INFLUENCE OF FRACTURE TOUGHNESS ON FATIGUE LIFE OF STEEL BRIDGES

FRITZ ENGINEERING LABORATORY LIBRARY

by

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# TABLE OF CONTENTS

ABSTRACT

1. INTRODUCTION
   1.1 Description of the Problem
   1.2 Outline of the Work

2. INFLUENCE OF THE FRACTURE TOUGHNESS ON THE FATIGUE BEHAVIOR
   2.1 Background
      2.1.1 Physical Behavior
      2.1.2 Crack Propagation Relationships
   2.2 Fatigue Life Estimations
      2.2.2 Results from Fatigue Tests
      2.2.3 Fatigue Life Calculations
   2.3 Fatigue Behavior of Bridge Steel
      2.3.1 Introduction
      2.3.2 Fatigue Behavior at Low $\Delta K$-values
         2.3.2.1 Review of Test Data
      2.3.3 Fatigue Behavior at Medium $\Delta K$-values
      2.3.4 Fatigue Behavior at High $\Delta K$-values
   2.4 Crack Propagation Relationships Developed by McEvily
      2.4.1 Introduction
      2.4.2 Curve Fit to $da/dN-\Delta K$ Measurements
      2.4.3 Curve Fit to the Paris Equation
      2.4.4 Minimum Crack Size

IV
### 2.4.5 Fatigue Life Correlation with Beam Test Results

Page 50

### 2.4.6 Fatigue Life Calculations for Structural Details

Page 51

### 2.4.7 Conclusions

Page 53

### 2.5 Proposed Fatigue Crack Propagation Relationship

Page 55

#### 2.5.1 Crack Growth Relationship Near $\Delta K_{Th}$

Page 55

#### 2.5.2 Crack Growth Relationship Near $K_C$

Page 57

#### 2.5.3 Fatigue Life Calculations

Page 58

##### 2.5.3.1 Small Size Cover-plated Beams

Page 59

##### 2.5.3.2 Full Size Cover-plated Beams

Page 62

##### 2.5.3.3 Beams with Stiffeners

Page 63

### 2.6 Conclusion

Page 64

#### 2.6.1 Summary

Page 64

#### 2.6.2 Results

Page 64

### 3. CASE STUDIES

Page 67

#### 3.1 Problem Statement and Solution Approach

Page 67

##### 3.1.1 Introduction

Page 67

##### 3.1.2 AASHTO Material Toughness Requirements for Steel Bridges

Page 67

##### 3.1.3 Principle of Linear Elastic Fracture Mechanics

Page 69

##### 3.1.4 Material Characterization

Page 73

##### 3.1.5 Stresses at the Crack Location

Page 75

##### 3.1.6 Calculation Concept

Page 76
3.2 Quinnipiac River Bridge 77
   3.2.1 Introduction 77
   3.2.2 Material Characterization and Stresses at the Critical Location 78
   3.2.3 Crack Growth Stages 80
   3.2.4 Analysis of Crack Growth 81
   3.2.5 Numerical Results 84

3.3 Glenfield Bridge 86
   3.3.1 Introduction 86
   3.3.2 Material Characterization 88
   3.3.3 Stress Distribution in the Vicinity of the Crack 91
   3.3.4 Crack Growth Stage 95
   3.3.5 Analysis of Crack Growth 97
   3.3.6 Conclusion 107

3.4 Lafayette Street Bridge 108
   3.4.1 Introduction, Material Characterization 108
   3.4.2 Stresses at the Critical Location 111
   3.4.3 Crack Growth Stages 112
   3.4.4 Analysis of Crack Growth 114
   3.4.5 Influence of the Fracture Toughness on the Fatigue Behavior 122

3.5 Yellow Mill Pond Bridge 122
   3.5.1 Introduction 122
   3.5.2 Material Characterization 124
   3.5.3 Stresses at the Critical Location 126
3.5.4 Description of the Fracture Surface, Crack Growth Stages 128
3.5.5 Analysis of Crack Growth 129
3.5.6 Influence of the Fracture Toughness and the Fatigue Behavior 136

3.6 Dan Ryan Viaduct 138
3.6.1 Introduction 138
3.6.2 Material Characterization and Stresses at the Critical Location 139
3.6.3 Description of the Fracture Surface, Crack Growth Stages 142
3.6.4 Analysis of the Crack Growth 143
3.6.5 Influence of the Fracture Toughness on the Fatigue Life 147

3.7 Welded Box Girders 150
3.7.1 Introduction, Description of the Cracks 150
3.7.2 Stresses at the Critical Location 151
3.7.3 Analysis of the Cracks at the Welded Corners 157
3.7.4 Fatigue Behavior of Cracks at the Welded Corners 162
3.7.5 Influence of the Fracture Toughness on the Fatigue Resistance 164

4. EFFECT OF PEENING ON THE FATIGUE LIFE 166
4.1 Introduction and Research Approach 166
4.1.1 Problem Statement 166
4.1.2 Methods of Peening 167
   4.1.2.1 Shot Peening 167
   4.1.2.2 Air Hammer Peening 168

VII
4.1.2.3 Other Peening Methods 169
4.1.3 Research Approach 169

4.2 Experiments 170
  4.2.1 Peening Tool 170
  4.2.2 Experiment Design 172
  4.2.3 Specimen Preparation and Measurement Technics 172
  4.2.4 Results 174
    4.2.4.1 Visual Inspection 174
    4.2.4.2 Surface Deformations 175
    4.2.4.3 Microscopic Inspection 176
    4.2.4.4 Hardness Study 177
  4.2.5 Comparison with Test Beams 179

4.3 Analysis 179
  4.3.1 Residual Stress Field 179
  4.3.2 Analytical Model 180
  4.3.3 Fatigue Life Calculation 182
    4.3.3.1 Influence of Depth of the Zone affected by Peening 184
    4.3.3.2 Influence of the Stress Range on $a_{Th}$ 185
    4.3.3.3 Influence of the Yield Strength on $a_{Th}$ 186
    4.3.3.4 Influence of Minimum Stress on $a_{Th}$ 187
  4.3.4 Remaining Fatigue Life 187

4.4 Summary and Conclusions 189

VIII
5. CONCLUSIONS

6. RECOMMENDATIONS FOR FURTHER RESEARCH

NOMENCLATURE

TABLES

FIGURES

REFERENCES

VITA

Page
191
195
199
203
231
421
431
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 Geometrical Properties of Test Beams</td>
<td>203</td>
</tr>
<tr>
<td>2.2 Fatigue Threshold Stress Intensity Range for Constant Amplitude Tests at Room Temperature</td>
<td>204</td>
</tr>
<tr>
<td>2.3 Fracture Toughness for Bridge Steels after Ref. 9</td>
<td>206</td>
</tr>
<tr>
<td>2.4 Results from Least-Square Curve-Fit, McEvily's Relationships</td>
<td>207</td>
</tr>
<tr>
<td>2.5 Fatigue Life for Small Size Cover-Plated Beams ( K_c = 110 \text{ MPa}\sqrt{\text{m}} ), ( a_i = 0.076 \text{ mm} )</td>
<td>208</td>
</tr>
<tr>
<td>2.6 Fatigue Life for Small Size Cover-Plated Beams, ( R = 0.6, K_c = 110 \text{ MPa}\sqrt{\text{m}}, \Delta K_{\text{Th}} = 3.3 \text{ MPa}\sqrt{\text{m}}, a_i = 0.076 \text{ mm} )</td>
<td>209</td>
</tr>
<tr>
<td>2.7 Fatigue Life for Small Size Cover-Plated Beams, ( \Delta \sigma = 55 \text{ MPa}, \Delta K_{\text{Th}} = 33 \text{ MPa}\sqrt{\text{m}}, R = 0.6, C = 3.16, \text{ Eq. 2.13} )</td>
<td>210</td>
</tr>
<tr>
<td>2.8 Fatigue Life for Full Size Cover-Plated Beams, ( \Delta \sigma = 55 \text{ MPa}, \Delta K_{\text{Th}} = 3.3 \text{ MPa}\sqrt{\text{m}}, R = 0.6, C = 3.16, \text{ Eq. 2.13} )</td>
<td>211</td>
</tr>
<tr>
<td>2.9 Fatigue Life for Type 3 Stiffeners, ( \Delta \sigma = 110 \text{ MPa}, \Delta K_{\text{Th}} = 3.3 \text{ MPa}\sqrt{\text{m}}, R = 0.6, C = 3.16, \text{ Eq. 2.13} )</td>
<td>212</td>
</tr>
<tr>
<td>2.10 Fatigue Life for Small Size Cover-Plated Beams, ( \Delta \sigma = 55 \text{ MPa}, \text{ da/dN from Fig. 2.66, } R = 0.6 )</td>
<td>213</td>
</tr>
<tr>
<td>2.11 Fatigue Life for Full Size Cover-Plated Beams, ( \Delta \sigma = 55 \text{ MPa}, \text{ da/dN from Fig. 2.66, } R = 0.6 )</td>
<td>214</td>
</tr>
<tr>
<td>2.12 Fatigue Life for Type 3 Stiffeners, ( \text{ da/dN from Fig. 2.66, } \Delta \sigma = 110 \text{ MPa, } R = 0.6 )</td>
<td>215</td>
</tr>
<tr>
<td>2.13 Influence of the Fracture Toughness on the Fatigue Life, Summary, ( \text{ da/dN from Fig. 2.66, } a_i = 0.076 \text{ mm} )</td>
<td>216</td>
</tr>
<tr>
<td>TABLE</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>------</td>
</tr>
<tr>
<td>3.1 AASHTO Notch Toughness Specification for Bridge Steel</td>
<td>217</td>
</tr>
<tr>
<td>3.2 Influence of the Fracture Toughness on the Fatigue Life, Quinnipiac River Bridge</td>
<td>218</td>
</tr>
<tr>
<td>3.3 Stress Range Histogram, Glenfield Bridge</td>
<td>219</td>
</tr>
<tr>
<td>3.4 Influence of the Fracture Toughness on the Fatigue Life, Lafayette Street Bridge</td>
<td>220</td>
</tr>
<tr>
<td>3.5 Stress Ranges, Dan Ryan Viaduct</td>
<td>221</td>
</tr>
<tr>
<td>3.6 Influence of the Fracture Toughness on the Fatigue Life, Dan Ryan Viaduct</td>
<td>222</td>
</tr>
<tr>
<td>3.7 Weld Characteristics for Corner Welds, Box Girder</td>
<td>223</td>
</tr>
<tr>
<td>3.8 Geometrical Properties of the Investigated Crack Shapes, Embedded Flaws, Box Girder</td>
<td>224</td>
</tr>
<tr>
<td>3.9 Fatigue Crack Growth for Cracks at the Welded Corners, Box Girder</td>
<td>225</td>
</tr>
<tr>
<td>3.10 Influence of the Fracture Toughness on the Fatigue Behavior, Box Girder</td>
<td>226</td>
</tr>
<tr>
<td>4.1 Experiment Design for Peening, Specimen Designation</td>
<td>227</td>
</tr>
<tr>
<td>4.2 Deformations after Peening</td>
<td>228</td>
</tr>
<tr>
<td>a) Surface Deformation</td>
<td></td>
</tr>
<tr>
<td>b) Grain Deformation</td>
<td></td>
</tr>
<tr>
<td>4.3 Threshold Crack Length $a_{th}$ for Peened Full Size Cover-plated Beams, A36 Steel, $t_d = 0.5$ m</td>
<td>229</td>
</tr>
<tr>
<td>4.4 Comparison between the Fatigue Life of Peened and Unpeened Cover-plated Beams</td>
<td>230</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Typical Stress Intensity Range-Crack Growth Rate Relationship for Bridge Steel</td>
<td>231</td>
</tr>
<tr>
<td>2.2</td>
<td>Crack Growth Stages in a Tension Flange and Resulting Fracture Toughness</td>
<td>232</td>
</tr>
<tr>
<td>2.3</td>
<td>Fatigue and Fracture Surface, Beam B5A, A588 Steel, W36x230, Failure Temperature $-79^\circ C$</td>
<td>233</td>
</tr>
<tr>
<td>2.4</td>
<td>Fatigue and Fracture Surface, Beam B5, A588 Steel, W36x230, Failure Temperature $-101^\circ C$</td>
<td>234</td>
</tr>
<tr>
<td>2.5</td>
<td>Comparison between McEvily's Relationships (Eq. 2.11 and Eq. 2.13) and Paris Law (Eq. 2.6)</td>
<td>235</td>
</tr>
<tr>
<td>2.6</td>
<td>Crack Growth Rate Relationship Developed by Forman (Eq. 2.15)</td>
<td>236</td>
</tr>
<tr>
<td>2.7</td>
<td>Crack Growth Rate Relationship Developed by Pearson (Eq. 2.16)</td>
<td>237</td>
</tr>
<tr>
<td>2.8</td>
<td>Design Stress Range Curves for Detail Category A to E</td>
<td>238</td>
</tr>
<tr>
<td>2.9</td>
<td>Crack Location</td>
<td>239</td>
</tr>
<tr>
<td></td>
<td>a) at Stiffener Welds</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) at Cover Plate End Welds</td>
<td></td>
</tr>
<tr>
<td>2.10</td>
<td>Fatigue Test Results for Type 3 Stiffeners</td>
<td>240</td>
</tr>
<tr>
<td>2.11</td>
<td>Comparison of Fatigue Test Results for Type 3 Stiffeners from Ref. 12,48,49,50</td>
<td>241</td>
</tr>
<tr>
<td>2.12</td>
<td>Definitions of Angles for Crack Shape Correction Factor and Free Surface Correction Factor</td>
<td>242</td>
</tr>
<tr>
<td>2.13</td>
<td>Crack Shape Measurements</td>
<td>243</td>
</tr>
<tr>
<td>2.14</td>
<td>Fatigue Life of Full Size Cover-plated Beams, Fatigue Life Defined using Fracture Criterion</td>
<td>244</td>
</tr>
<tr>
<td>2.15</td>
<td>Fatigue Life for Type 3 Stiffener for Different Crack Shape Relationships, Eq. 2.6, $a = 0.076$ mm</td>
<td>245</td>
</tr>
<tr>
<td>Figure</td>
<td>Page</td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>------</td>
<td></td>
</tr>
<tr>
<td>2.16</td>
<td>246</td>
<td></td>
</tr>
<tr>
<td>2.17</td>
<td>247</td>
<td></td>
</tr>
<tr>
<td>2.18</td>
<td>248</td>
<td></td>
</tr>
<tr>
<td>2.19</td>
<td>249</td>
<td></td>
</tr>
<tr>
<td>2.20</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td>2.21</td>
<td>251</td>
<td></td>
</tr>
<tr>
<td>2.22</td>
<td>252</td>
<td></td>
</tr>
<tr>
<td>2.23</td>
<td>253</td>
<td></td>
</tr>
<tr>
<td>2.24</td>
<td>254</td>
<td></td>
</tr>
<tr>
<td>2.25</td>
<td>255</td>
<td></td>
</tr>
<tr>
<td>2.26</td>
<td>256</td>
<td></td>
</tr>
<tr>
<td>2.27</td>
<td>257</td>
<td></td>
</tr>
<tr>
<td>2.28</td>
<td>258</td>
<td></td>
</tr>
<tr>
<td>2.29</td>
<td>259</td>
<td></td>
</tr>
<tr>
<td>2.30</td>
<td>260</td>
<td></td>
</tr>
<tr>
<td>2.31</td>
<td>261</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 2.16** da/dN - ΔK Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, $0 \leq R < 0.2$

**Figure 2.17** da/dN - ΔK Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, $0.2 \leq R < 0.4$

**Figure 2.18** da/dN - ΔK Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, $0.4 \leq R < 0.6$

**Figure 2.19** da/dN - ΔK Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, $0.6 \leq R < 0.8$

**Figure 2.20** da/dN - ΔK Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, $0.8 \leq R < 1.0$

**Figure 2.21** Fatigue Threshold Stress Intensity Range of Different Steels

**Figure 2.22** Comparison of Fatigue Threshold Stress Intensity Range with lower Bound Relationships

**Figure 2.23** Transition between Zone II and III, ΔK_T from Eq. 2.52, A36 Steel^{32}, R = 0, ΔK_T = 45.3 MPa√m

**Figure 2.24** Transition between Zone II and III, K_T from Eq. 2.52, A36 Steel^{32}, 0.33 < R < 0.38, ΔK_T = 29.6 MPa√m

**Figure 2.25** Transition between Zone II and III, ΔK_T from Eq. 2.52, A36 Steel^{32}, 0.09 < R < 0.11, ΔK_T = 41 MPa√m

**Figure 2.26** Curve Fit Eq. 2.11, A36 Steel^{9}, R = 0.1

**Figure 2.27** Curve Fit Eq. 2.13, A36 Steel^{9}, R = 0.1

**Figure 2.28** Curve Fit Eq. 2.11, A588 A Steel^{9}, R = 0.1

**Figure 2.29** Curve Fit Eq. 2.13, A588 A Steel^{9}, R = 0.1

**Figure 2.30** Curve Fit Eq. 2.11, A517 F Steel^{9}, R = 0.1

**Figure 2.31** Curve Fit Eq. 2.13, A517 F Steel^{9}, R = 0.1
2.32 Influence of $K_{Th}$ on Curve Fit, Eq. 2.11, A36 Steel, $R = 0.1$

2.33 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, $\Delta K_{Th} = f \text{ (Test)}, K_c = 110 \text{ MPa} \sqrt{m}$

2.34 Fatigue Life of Small Size Cover-plated Beams, $\Delta K_{Th} = f \text{ (Test)}, K_c = 110 \text{ MPa} \sqrt{m}$

2.35 Curve Fit, McEvily's Relationship (Eq. 2.11) and Paris Law (Eq. 2.6), $K_c = 110 \text{ MPa} \sqrt{m}$, $R = 0.1$

2.36 Curve Fit, McEvily's Relationship (Eq. 2.11) and Paris Law (Eq. 2.6), $K_c = 165 \text{ MPa} \sqrt{m}$, $R = 0.1$

2.37 Curve Fit, McEvily's Relationship (Eq. 2.13) and Paris Law (Eq. 2.6), $K_c = 110 \text{ MPa} \sqrt{m}$, $R = 0.1$

2.38 $(\pi a)^{1/2}$ - Crack Length, Small Size Cover-plated Beams

2.39 Minimum Crack Length for Fatigue Crack Propagation, Small Size Cover-plated Beams

2.40 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.11, $a_i = 0.076 \text{ mm}$, $\Delta K_{Th} = 3.3 \text{ MPa} \sqrt{m}$, $R = 0.6$

2.41 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, $a_i = 0.076 \text{ mm}$, $\Delta K_{Th} = 3.3 \text{ MPa} \sqrt{m}$, $R = 0.6$

2.42 Influence of Fracture Toughness on Crack Growth Rate, Eq. 2.11, $R = 0.1$

2.43 Influence of Fracture Toughness on Crack Growth Rate, Eq. 2.13, $R = 0.1$

2.44 Crack Growth Rate Predicted with Eq. 2.11, $C = 3.799$, $\Delta K_{Th} = 3.3 \text{ MPa} \sqrt{m}$, $R = 0.1$

2.45 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.11, $K_c = 110 \text{ MPa} \sqrt{m}$, $R = 0.6$

2.46 Fatigue Crack Growth Rates, Curve Fit with Beam Tests, Eq. 2.13, $C = 3.16$, $\Delta K_{Th} = 3.3 \text{ MPa} \sqrt{m}$, $R = 0.6$

2.47 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, $C = 3.16$, $\Delta K_{Th} = 3.3 \text{ MPa} \sqrt{m}$, $R = 0.6$
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.48</td>
<td>Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, C = 3.16, α = 0.025 mm, Δσ = 55 MPa, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>278</td>
</tr>
<tr>
<td>2.49</td>
<td>Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, C = 3.16, α = 0.025 mm, Δσ = 55 MPa, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>279</td>
</tr>
<tr>
<td>2.50</td>
<td>Fatigue Life of Full Size Cover-plated Beams, Eq. 2.13, C = 3.16, α = 0.025 mm, Δσ = 55 MPa, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>280</td>
</tr>
<tr>
<td>2.51</td>
<td>Fatigue Life of Full Size Cover-plated Beams, Eq. 2.13, C = 3.16, α = 0.025 mm, Δσ = 55 MPa, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>281</td>
</tr>
<tr>
<td>2.52</td>
<td>Fatigue Life of Full Size Cover-plated Beams, Eq. 2.13, C = 3.16, α = 0.025 mm, Δσ = 55 MPa, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>282</td>
</tr>
<tr>
<td>2.53</td>
<td>Fatigue Life of Type 3 Stiffener, Eq. 2.13, C = 3.16, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>283</td>
</tr>
<tr>
<td>2.54</td>
<td>Fatigue Life of Type 3 Stiffener, Eq. 2.13, C = 3.16, α = 0.025 mm, Δσ = 110 MPa, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>284</td>
</tr>
<tr>
<td>2.55</td>
<td>Fatigue Life of Type 3 Stiffener, Eq. 2.13, C = 3.16, Δσ = 110 MPa, ΔK_{th} = 3.3 MPavm, R = 0.6</td>
<td>285</td>
</tr>
<tr>
<td>2.56</td>
<td>Schematic of Fatigue Crack Growth Relationship</td>
<td>286</td>
</tr>
<tr>
<td>2.57</td>
<td>Influence of ΔK_{1} on Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel^{25}, R = 0.1</td>
<td>287</td>
</tr>
<tr>
<td>2.58</td>
<td>Influence of ΔK_{1} on Fatigue Crack Growth Rate Prediction, Eq. 2.64, Upper Bound, A533 B Steel^{25}, R = 0.1</td>
<td>288</td>
</tr>
<tr>
<td>2.59</td>
<td>Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel^{25}, R = 0.1</td>
<td>289</td>
</tr>
<tr>
<td>2.60</td>
<td>Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel^{25}, R = 0.3</td>
<td>290</td>
</tr>
<tr>
<td>2.61</td>
<td>Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel^{25}, R = 0.5</td>
<td>291</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td>------</td>
</tr>
<tr>
<td>2.62</td>
<td>Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel, R = 0.7</td>
<td>292</td>
</tr>
<tr>
<td>2.63</td>
<td>Influence of $\Delta K_2$ on Fatigue Crack Growth Rate Prediction, Eq. 2.68, R = 0.1</td>
<td>293</td>
</tr>
<tr>
<td>2.64</td>
<td>Influence of $\Delta K_2$ on Fatigue Crack Growth Rate Prediction, Eq. 2.69, R = 0.1</td>
<td>294</td>
</tr>
<tr>
<td>2.65</td>
<td>Comparison between Fatigue Crack Growth Rate Prediction from Eq. 2.68 and Eq. 2.69, A36 Steel, $\Delta K_T = 45.4 \text{ MPa}\sqrt{\text{m}}$, $K_c = 104 \text{ MPa}$, R = 0</td>
<td>295</td>
</tr>
<tr>
<td>2.66</td>
<td>Proposed Fatigue Crack Growth Rate Prediction, R = 0.6</td>
<td>296</td>
</tr>
<tr>
<td>2.67</td>
<td>Fatigue Life of Small Scale Cover-plated Beams, da/dN from Fig. 2.66, R = 0.6</td>
<td>297</td>
</tr>
<tr>
<td>2.68</td>
<td>Fatigue Life of Small Size Cover-plated Beams, da/dN from Fig. 2.66, $a_i = 0.025 \text{ mm}$, $\Delta \sigma = 55 \text{ MPa}$</td>
<td>298</td>
</tr>
<tr>
<td>2.69</td>
<td>$\Delta K - da/dN (\Delta K)$ and $\Delta K - N (\Delta K)$ for Small Size Cover-plated Beams, da/dN from Fig. 2.66, $a_i = 0.013 \text{ mm}$, $\Delta \sigma = 69 \text{ MPa}$, $K_c = 165 \text{ MPa}\sqrt{\text{m}}$, R = 0.6</td>
<td>299</td>
</tr>
<tr>
<td>2.70</td>
<td>$a - \Delta K (a)$ and $a - a(N)$ for Small Size Cover-plated Beams, da/dN from Fig. 2.66, $a_i = 0.025 \text{ mm}$, $\Delta \sigma = 69 \text{ MPa}$, R = 0.6</td>
<td>300</td>
</tr>
<tr>
<td>2.71</td>
<td>Fatigue Life of Full Size Cover-plated Beams, da/dN from Fig. 2.66, $K_c = \infty$, R = 0.6</td>
<td>301</td>
</tr>
<tr>
<td>2.72</td>
<td>Fatigue Life of Full Size Cover-plated Beams, da/dN from Fig. 2.66, $\Delta \sigma = 55 \text{ MPa}$, R = 0.6</td>
<td>302</td>
</tr>
<tr>
<td>2.73</td>
<td>Fatigue Life of Full Size Cover-plated Beams, da/dN from Fig. 2.66, $\Delta \sigma = 55 \text{ MPa}$, R = 0.6</td>
<td>303</td>
</tr>
<tr>
<td>2.74</td>
<td>Fatigue Life of Type 3 Stiffeners, da/dN from Fig. 2.66, $\Delta \sigma = 110 \text{ MPa}$, $K_c = \infty$</td>
<td>304</td>
</tr>
<tr>
<td>2.75</td>
<td>Fatigue Life of Type 3 Stiffeners, da/dN from Fig. 2.66, $a_i = 0.025 \text{ mm}$, $\Delta \sigma = 110 \text{ MPa}$, R = 0.6</td>
<td>305</td>
</tr>
<tr>
<td>2.76</td>
<td>$a - \Delta K (a)$ and $a - a(N)$ for Stiffeners, da/dN from Fig. 2.66, $a_i = 0.025 \text{ mm}$, $\Delta \sigma = 110 \text{ MPa}$, R = 0.6</td>
<td>306</td>
</tr>
</tbody>
</table>

XVI
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>AASHTO Material Toughness Requirements for A36 Steel, Zone II, Ref. 52</td>
<td>307</td>
</tr>
<tr>
<td>3.2</td>
<td>Crack Growth Rate Prediction used for Case Studies of Fatigue Life Calculations</td>
<td>308</td>
</tr>
<tr>
<td>3.3</td>
<td>Quinnipiac River Bridge, Crack Location</td>
<td>309</td>
</tr>
<tr>
<td>3.4</td>
<td>Schematic of Girder Cross Section and Crack Dimension, Quinnipiac River Bridge</td>
<td>310</td>
</tr>
<tr>
<td>3.5</td>
<td>Assumption for Residual Stress Distribution near Stiffener-Web Connection, Quinnipiac River Bridge</td>
<td>311</td>
</tr>
<tr>
<td>3.6</td>
<td>Crack Growth Stages, Quinnipiac River Bridge1,51</td>
<td>312</td>
</tr>
<tr>
<td>3.7</td>
<td>Crack Growth Stage II, Fatigue Crack Growth, Quinnipiac River Bridge</td>
<td>313</td>
</tr>
<tr>
<td>3.8</td>
<td>Crack Growth in Web at Intersection with Horizontal Stiffener, Test Beam90</td>
<td>314</td>
</tr>
<tr>
<td>3.9</td>
<td>Partially Loaded Circular Crack in Infinite Solid53</td>
<td>315</td>
</tr>
<tr>
<td>3.10</td>
<td>Through Crack in Infinite Solid</td>
<td></td>
</tr>
<tr>
<td></td>
<td>a) Partially Loaded</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) Splitting Forces</td>
<td></td>
</tr>
<tr>
<td>3.11</td>
<td>Stress Intensity Factor and Fatigue Life for Crack in Web, Circular Crack, Quinnipiac River Bridge</td>
<td>317</td>
</tr>
<tr>
<td>3.12</td>
<td>Stress Intensity Factor, Through Crack, Quinnipiac River Bridge</td>
<td>318</td>
</tr>
<tr>
<td>3.13</td>
<td>Crack Size for ( K_c = 110 \text{ MPa} \sqrt{\text{m}} ) and ( K_c = 165 \text{ MPa} \sqrt{\text{m}} ), Quinnipiac River Bridge</td>
<td>319</td>
</tr>
<tr>
<td>3.14</td>
<td>Schematic of 179 Glenfield Bridge58</td>
<td>320</td>
</tr>
<tr>
<td>3.15</td>
<td>Schematic of Girder Cross Section, Crack Dimension, Glenfield Bridge</td>
<td>321</td>
</tr>
<tr>
<td>3.16</td>
<td>Charpy V-Notch Test Results, Glenfield Bridge</td>
<td>322</td>
</tr>
<tr>
<td></td>
<td>a) Base Plate</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) Electroslag Weld Metal</td>
<td></td>
</tr>
<tr>
<td></td>
<td>c) Fusion Line, Heat Affected Zone</td>
<td></td>
</tr>
<tr>
<td></td>
<td>d) Repair Weld</td>
<td></td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td></td>
</tr>
<tr>
<td>3.17</td>
<td>Fracture Toughness of Electroslag Weld Metal, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.18</td>
<td>Stress Range Histogram at Crack Location, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.19</td>
<td>Residual Stress Distribution in Flange due to Electroslag Weldment, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.20</td>
<td>Repair Weld Sequence in Electroslag Weld\textsuperscript{57}, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.21</td>
<td>Residual Stress Distribution in Flange due to Electroslag Weldment after Repair, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.22</td>
<td>Residual Stress Distribution in Flange due to Repair Weld, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.23</td>
<td>Residual Stress Distribution in Flange due to Web-Flange Weldment, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.24</td>
<td>Fracture Surface, South Side, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.25</td>
<td>Fracture Surface, Probable Initial Crack, half Circular Crack Arrest, Electron Microscopically Investigated Zones (1-5), Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.26</td>
<td>Schematic of Crack Growth Stages, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.27</td>
<td>Investigated Crack Shapes, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.28</td>
<td>Striation Markings at Location 3, 48240X, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.29</td>
<td>Stress Intensity Factor for Elliptical Crack under Uniform Load</td>
<td></td>
</tr>
<tr>
<td>3.30</td>
<td>Stress Intensity Factor for Eccentrically Through crack in Finite Plate</td>
<td></td>
</tr>
<tr>
<td>3.31</td>
<td>Stress Intensity Factor at Location A due to Splitting Force at \textsuperscript{57}</td>
<td></td>
</tr>
<tr>
<td>3.32</td>
<td>Stress Intensity Factor around Initial Elliptical Flaw, Glenfield Bridge</td>
<td></td>
</tr>
<tr>
<td>3.33</td>
<td>Comparison between Stress Intensity Factor during Stage III and Material Toughness, Glenfield Bridge</td>
<td></td>
</tr>
</tbody>
</table>

XVIII
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.34</td>
<td>Stress Intensity Factor around Semicircular Crack, ( r = 51 \text{ mm} ), Glenfield Bridge</td>
<td>341</td>
</tr>
<tr>
<td>3.35</td>
<td>Stress Intensity Factor around Semicircular Crack, ( r = 64 \text{ mm} ), Glenfield Bridge</td>
<td>342</td>
</tr>
<tr>
<td>3.36</td>
<td>Stress Intensity Factor around Semicircular Crack, ( r = 76 \text{ mm} ), Glenfield Bridge</td>
<td>343</td>
</tr>
<tr>
<td>3.37</td>
<td>Comparison between Stress Intensity Factor during Final Fracture and Material Toughness, Glenfield Bridge</td>
<td>344</td>
</tr>
<tr>
<td>3.38</td>
<td>Schematic of Span and Cross Section of Main Girder at Crack Location, Lafayette Street Bridge</td>
<td>345</td>
</tr>
<tr>
<td>3.39</td>
<td>Schematic of the Crack in the Stiffener-Gusset Region, Lafayette Street Bridge</td>
<td>346</td>
</tr>
<tr>
<td>3.40</td>
<td>Charpy V-Notch Test Results and Material Toughness for Web Material, Lafayette Street Bridge</td>
<td>347</td>
</tr>
<tr>
<td>3.41</td>
<td>Residual Stress Distribution in Flange due to Web-Flange Weldment, Lafayette Street Bridge</td>
<td>348</td>
</tr>
<tr>
<td>3.42</td>
<td>Crack Growth Stages in Web of Lafayette Street Bridge</td>
<td>349</td>
</tr>
<tr>
<td>3.43</td>
<td>Fracture Surface of Removed Plug, 2nd Fracture, Lafayette Street Bridge</td>
<td>350</td>
</tr>
<tr>
<td>3.44</td>
<td>Crack Growth Models during Stage II, Lafayette Street Bridge</td>
<td>351</td>
</tr>
<tr>
<td>3.45</td>
<td>Maximum Stress Intensity Factor in Flange, Lafayette Street Bridge</td>
<td>352</td>
</tr>
<tr>
<td>3.46</td>
<td>Stress Distribution and Stress Intensity Factor during Stage III-Stage IV, Lafayette Street Bridge</td>
<td>353</td>
</tr>
<tr>
<td>3.47</td>
<td>Fracture Toughness and Fatigue Life for Crack in Web, Lafayette Street Bridge</td>
<td>354</td>
</tr>
<tr>
<td>3.48</td>
<td>Cracked cover-plated Beam at Yellow Mill Pond Bridge</td>
<td>355</td>
</tr>
<tr>
<td>3.49</td>
<td>Schematic of Yellow Mill Pond Bridge, Crack Location</td>
<td>356</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>3.50</td>
<td>Charpy V-Notch Test Results, Yellow Mill Pond Bridge</td>
<td>357</td>
</tr>
<tr>
<td>3.51</td>
<td>Fracture Toughness for Material Removed from the Web, Yellow Mill Pond Bridge</td>
<td>358</td>
</tr>
<tr>
<td>3.52</td>
<td>Fracture Toughness for Material Removed from the Flange, Yellow Mill Pond Bridge</td>
<td>359</td>
</tr>
<tr>
<td>3.53</td>
<td>Cross Section of B4, Stress Distribution and Location of the Neutral Axis, Yellow Mill Pond Bridge</td>
<td>360</td>
</tr>
<tr>
<td>3.54</td>
<td>Residual Stress Distribution for A242, W36X230 Flange (after Ref. 36)</td>
<td>361</td>
</tr>
<tr>
<td>3.55</td>
<td>Local Weld Residual Stress Distribution for Cover Plate with End-Weld (after Ref. 36)</td>
<td>362</td>
</tr>
<tr>
<td>3.56</td>
<td>Stress Intensity Factor for Semielliptical Surface Crack in Flange, a = 13 mm, c = 63 mm, Yellow Mill Pond Bridge</td>
<td>363</td>
</tr>
<tr>
<td>3.57</td>
<td>Stress Intensity Factor for Semielliptical Surface Crack in Flange, a = 25 mm, c = 139 mm, Yellow Mill Pond Bridge</td>
<td>364</td>
</tr>
<tr>
<td>3.58</td>
<td>Stress Intensity Factor and Crack Opening for Bending Specimen</td>
<td>365</td>
</tr>
<tr>
<td>3.59</td>
<td>Fatigue Life of B4, Span 11, C = 2.18x10^{-13} mm^{5.5} / N^{3/2} cycle</td>
<td>366</td>
</tr>
<tr>
<td></td>
<td>Yellow Mill Pond Bridge</td>
<td></td>
</tr>
<tr>
<td>3.60</td>
<td>Stress Intensity Factor for Crack in Flange, c = 7.0 a^{1.0}, Yellow Mill Pond Bridge</td>
<td>367</td>
</tr>
<tr>
<td>3.61</td>
<td>Fatigue Life for Crack in W36x230 Flange, ΔC = 8.1 MPa, Yellow Mill Pond Bridge</td>
<td>368</td>
</tr>
<tr>
<td>3.62</td>
<td>Schematic showing Box Girder Bent with Crack Location, Dan Ryan Viaduct</td>
<td>369</td>
</tr>
<tr>
<td>3.63</td>
<td>Charpy V-Notch Results, Web Material, Dan Ryan Viaduct</td>
<td>370</td>
</tr>
<tr>
<td>3.64</td>
<td>Fracture Toughness measured with Compact Tension Tests, 1 sec Loading Time, Dan Ryan Viaduct</td>
<td>371</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>---------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>3.65</td>
<td>Stress Range in Box Web Plate at Crack Location, Dan Ryan Viaduct</td>
<td>372</td>
</tr>
<tr>
<td>3.66</td>
<td>Fracture Surface of Box Web Plate, Pier 24, Dan Ryan Viaduct</td>
<td>373</td>
</tr>
<tr>
<td>3.67</td>
<td>Fracture Surface of Box Web Plate, Pier 25, Dan Ryan Viaduct</td>
<td>374</td>
</tr>
<tr>
<td>3.68</td>
<td>Fracture Surface of Box Web Plate, Pier 26, Dan Ryan Viaduct</td>
<td>375</td>
</tr>
<tr>
<td>3.69</td>
<td>Fatigue Life for Semielliptical Surface Crack Growing into Web from Exterior, Dan Ryan Viaduct</td>
<td>376</td>
</tr>
<tr>
<td>3.70</td>
<td>Schematic of Fracture Surface and Corresponding Analytical Model, Pier 26, Dan Ryan Viaduct</td>
<td>377</td>
</tr>
<tr>
<td>3.71</td>
<td>Stress Intensity Factor for Through Crack in Web, Dan Ryan Viaduct</td>
<td>378</td>
</tr>
<tr>
<td>3.72</td>
<td>F&lt;sub&gt;e&lt;/sub&gt; - Correction Factor for Through Crack near Inserted Plate, Dan Ryan Viaduct</td>
<td>379</td>
</tr>
<tr>
<td>3.73</td>
<td>Dimensions of the Box Girder, Gulf Outlet Bridge</td>
<td>380</td>
</tr>
<tr>
<td>3.74</td>
<td>Cracks Found in the Corner Weld, Box Girder</td>
<td>381</td>
</tr>
<tr>
<td>3.75</td>
<td>Temperature Distribution in a Semiinfinite Plate</td>
<td>382</td>
</tr>
<tr>
<td>3.76</td>
<td>Weld Sequence and Discretisation near Corner Weld, Box Girder</td>
<td>383</td>
</tr>
<tr>
<td>3.77</td>
<td>Residual Stress Variation During Weld Passes of Corner Weld, Box Girder</td>
<td>384</td>
</tr>
<tr>
<td>3.78</td>
<td>Residual Stress Distribution due to Corner Welds in Middle Plane of Web and Flange, Box Girder</td>
<td>385</td>
</tr>
<tr>
<td>3.79</td>
<td>Residual Stress Distribution near Corner Weld, Box Girder</td>
<td>386</td>
</tr>
<tr>
<td>3.80</td>
<td>Investigated Crack Shapes in Corner Weld, Box Girder</td>
<td>387</td>
</tr>
<tr>
<td>3.81</td>
<td>Definition of Crack Lengths, Box Girder</td>
<td>388</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>3.82</td>
<td>Stress Intensity Factor for Cracks in Corner Weld, Box Girder</td>
<td>389</td>
</tr>
<tr>
<td></td>
<td>a) Shape A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) Shape B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>c) Shape C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>d) Shape D</td>
<td></td>
</tr>
<tr>
<td>3.83</td>
<td>Stress Intensity Factor and Fatigue Life for Cracks in Corner Weld, $\Delta \sigma = 14$ MPa, Box Girder</td>
<td>391</td>
</tr>
<tr>
<td>3.84</td>
<td>Investigated Crack Shapes, Edge Cracks in Flange and Web Plate, Box Girder</td>
<td>392</td>
</tr>
<tr>
<td>3.85</td>
<td>Stress Intensity Factor and Fatigue Life for Edge Crack in Flange and Web Plate, Box Girder</td>
<td>393</td>
</tr>
<tr>
<td>4.1</td>
<td>Small Crack at Cover Plate End Weld, 40X</td>
<td>394</td>
</tr>
<tr>
<td>4.2</td>
<td>Peening Tool</td>
<td>395</td>
</tr>
<tr>
<td>4.3</td>
<td>Peened Cover Plate End Weld, Small Specimens Cut Out for Investigation</td>
<td>396</td>
</tr>
<tr>
<td>4.4</td>
<td>Mounted Specimen</td>
<td>396</td>
</tr>
<tr>
<td>4.5</td>
<td>Schematic of Set-Up to Measure Surface Deformations from Peening</td>
<td>397</td>
</tr>
<tr>
<td>4.6</td>
<td>Hardness Measurements Indentations in Peened Zone, Specimen D, 700X</td>
<td>398</td>
</tr>
<tr>
<td>4.7</td>
<td>Peened Cover Plate End</td>
<td>399</td>
</tr>
<tr>
<td></td>
<td>a) Specimen A (3 Passes, 0.21 N/mm$^2$)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) Specimen C (3 Passes, 0.34 N/mm$^2$)</td>
<td></td>
</tr>
<tr>
<td>4.8</td>
<td>Grain Deformation in Peened Region, 80X</td>
<td>401</td>
</tr>
<tr>
<td></td>
<td>a) Specimen A</td>
<td></td>
</tr>
<tr>
<td></td>
<td>c) Specimen C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>e) Specimen E</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) Specimen B</td>
<td></td>
</tr>
<tr>
<td></td>
<td>d) Specimen D</td>
<td></td>
</tr>
<tr>
<td></td>
<td>f) Specimen F</td>
<td></td>
</tr>
<tr>
<td></td>
<td>g) Specimen G</td>
<td></td>
</tr>
<tr>
<td>4.9</td>
<td>Measured Deformations, Influence of the Air Pressure</td>
<td>405</td>
</tr>
<tr>
<td></td>
<td>a) Surface Deformation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) Grain Deformation</td>
<td></td>
</tr>
<tr>
<td>4.10</td>
<td>Measured Deformations, Influence of the Number of Passes</td>
<td>406</td>
</tr>
<tr>
<td></td>
<td>a) Surface Deformation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>b) Grain Deformation</td>
<td></td>
</tr>
<tr>
<td>4.11</td>
<td>Knoop Hardness Measurements, Specimen G</td>
<td>407</td>
</tr>
</tbody>
</table>

XXII
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.12</td>
<td>Knoop Hardness Measurements, Specimen G₁</td>
<td>408</td>
</tr>
<tr>
<td>4.13</td>
<td>Knoop Hardness, Specimen D</td>
<td>409</td>
</tr>
<tr>
<td>4.14</td>
<td>Hardness Measurements Indentations in Specimen D, 100X</td>
<td>410</td>
</tr>
<tr>
<td>4.15</td>
<td>Peened Region of Beam B₁, 62.5X</td>
<td>411</td>
</tr>
<tr>
<td>4.16</td>
<td>Peened Region of Beam B₉, 62.5X</td>
<td>412</td>
</tr>
<tr>
<td>4.17</td>
<td>Peened Region of Beam B₁₄ with Fatigue Crack, 62.5X</td>
<td>413</td>
</tr>
<tr>
<td>4.18</td>
<td>Residual Stress Distribution Caused by Peening</td>
<td>414</td>
</tr>
<tr>
<td>4.19</td>
<td>Influence of the Minimum and Maximum Stress Intensity Factor on the Effective Stress Intensity Range</td>
<td>415</td>
</tr>
<tr>
<td>4.20</td>
<td>Zones Influenced by Peening</td>
<td>416</td>
</tr>
<tr>
<td>4.21</td>
<td>Influence of the Thickness of the Residual Compression Stress Zone on the Threshold Crack Length, Δσ = 41 MPa</td>
<td>417</td>
</tr>
<tr>
<td>4.22</td>
<td>Influence of the Stress Range on the Threshold Crack Length</td>
<td>418</td>
</tr>
<tr>
<td>4.23</td>
<td>Influence of the Yield Strength on the Threshold Crack Length</td>
<td>419</td>
</tr>
<tr>
<td>4.24</td>
<td>Influence of the Minimum Stress on the Threshold Crack Length</td>
<td>420</td>
</tr>
</tbody>
</table>
ABSTRACT

Fatigue cracks in structural members grow from small imperfections under cyclic loads. A load-carrying metallic member may eventually be destroyed by brittle fracture as the fatigue crack propagates and the material fracture toughness is reached. Unfortunately, existing crack growth rate relationships do not accurately predict the growth rate of cracks in structural steels over the entire stress intensity range.

In the first part of this investigation, a crack growth rate relationship is developed for structural steels, taking into account the threshold stress intensity range and the material fracture toughness. Next, using laboratory test results developed from beam specimens with cover plates and stiffeners, the crack growth relationship is applied incorporating different levels of fracture toughness and different initial flaw sizes. From this analysis and comparison, it was found: (1) that the fracture toughness has much less effect on the fatigue life than the initial flaw size, and (2) that high fracture toughness does not substantially increase the fatigue life.

In the second part of the investigation, the fatigue and fracture behavior of six bridges which have experienced cracking were analyzed, and the influence of different fracture toughness levels was examined.
It was found that: (1) cyclic applied live load stress eventually increases the crack size and the stress intensity factor, (2) brittle fracture may develop before fatigue crack growth can penetrate the plate thickness, and (3) brittle fracture is dependent on the residual stress conditions and the magnitude of the fracture toughness of the connected material. This study demonstrates that an increase in the material fracture toughness above levels currently used to control the minimum fracture toughness of steel bridges does not appreciably change the fatigue life of the welded connection. It is concluded that design, quality control, and the prevention of cracks in bridge members will produce more reliable fracture resistant details.

In the third part, the influence of peening on the fatigue behavior is investigated analytically, using the principles of fracture mechanics. It is known that small fatigue cracks at cover plate ends can be arrested by peening. Air-hammer peening cold works the material in a small region surrounding the crack and also introduces compressive residual stresses around the crack tip. In this investigation, it was found that the depth of the zone with compressive residual stresses depends on the peening intensity. Further, it was found that the depth of fatigue cracks which can be successfully repaired depends on the peening intensity, the stress range and the minimum stress. The influence of the yield strength was found to be small. The analytical results agree well with experimental observations made during fatigue tests of peened cover-plated beams.
1. **INTRODUCTION**

1.1 **Description of the Problem**

Structural details such as cover plates, flange and web attachments (i.e. stiffeners, lateral connection plates) when welded on beams are known to contain small weld discontinuities at the weld-base metal intersection (weld toe). Under repeated applied cyclic loads, fatigue cracks grow from these small imperfections and the load-carrying member can finally be destroyed by rapid fracture. Rapid fracture occurs when the stress intensity factor at the crack tip reaches the material toughness. The stress intensity at the crack tip depends on the applied load, on the residual stresses, on the crack length and the geometry of the structural detail. Many fractures have been preceded by fatigue crack growth. A question often raised is the degree of interaction between the fatigue crack growth characteristics and the fracture resistance of the detail. Will significant increases in material fracture resistance substantially improve the fatigue resistance of the detail and permit easier detection and repair of the fatigue cracks? In most bridge fracture examples the rapid fractures are of the cleavage type and are commonly termed "brittle".

Fatigue life tests of girders have been made using A36, A588, A441, and A514 steels. Different yield strengths and different levels of fracture toughness were represented. However, the fatigue life of the test girders was mainly a function of stress range and
seemed independent of the mechanical strength properties. Crack
growth rate tests in which crack growth per cycle, da/dN, was mea-
sured as a function of $\Delta K$ (the variation of the stress intensity
factor, K), have given results consistent with this finding. These
tests showed only a moderate influence of yield strength at the
crack growth threshold and showed no significant influence of frac-
ture toughness until the maximum $K$ (of the $\Delta K$ variation) approached
the critical $K$ for onset of rapid fracturing. From these findings
and the fact that most of the fatigue life occurs while the crack
is relatively small, a large influence of fracture toughness on fa-
tigue life would not be expected. This topic is examined in detail
in this thesis.

During the past few years fatigue crack growth has occurred in
several bridges. Frequently, prior to discovery, a large segment of
rapid fracture had occurred. It is desirable to determine whether
or not a steel of higher toughness will increase the fatigue life
so that fatigue cracks can be detected by regular bridge inspections
and that the members can be repaired prior to extensive damage.

The Quinnipiac River Bridge developed a fatigue crack from a
lack of fusion area in a groove welded splice in a horizontal stiff-
ener. The crack penetrated the web in fatigue crack growth and
fractured the web in unstable manner. The brittle fracture was
arrested when the crack entered the tension flange. The fracture
surface suggested that brittle fracture occurred at low temperatures
after the bridge was in service for about ten years. Would a higher
material toughness have affected the behavior in a significant way?
Another example of fatigue cracking followed by brittle fracture is the cracking of the Lafayette Street Bridge in St. Paul, Minnesota\textsuperscript{2}. A fatigue crack started from a large lack of fusion area in a lateral bracing gusset-to-transverse stiffener weld. After the fatigue crack had nearly propagated through the web thickness the remaining part of the web and the tension flange cracked rapidly. The three-span structure did not collapse because of its redundancy. Several other similar cases are discussed in Chapter 3.

The Quinnipiac River Bridge and the Lafayette Street Bridge both satisfied the requirements of the 1974 interim AASHTO Specification\textsuperscript{3,35} for fracture toughness. Both of these structures would be classified as redundant. The current AASHTO Requirements\textsuperscript{3} for fracture safe bridge design insure a minimum level of fracture toughness of requiring specified levels of Charpy V-Notch energy. The requirements were derived from fracture mechanics principles to insure enough toughness to develop the fatigue life of the detail. Minimum service temperatures must be satisfied according to temperature zones in the United States. The Charpy V-Notch energy absorption requirements are intended to provide enough toughness so that inelastic behavior occurs even at the lowest service temperature.

Experimental trials indicate that small fatigue cracks at the end weld of cover plates can be repaired by air hammer peening the weld toe. End welds of cover plates were also peened before any cyclic load was applied, and the so treated beam showed a better fatigue behavior than untreated beams. Shot peening is frequently
used in machine design to improve the fatigue behavior of various parts. The shot peening method was also investigated analytically; experimental work was only done on air hammer peened cover-plated beams. The air hammer peening is done by plastically deforming the region of the weld toe. This procedure introduces local residual stresses which prevent the full stress cycle from being effective. Crack growth can therefore be arrested or crack growth can be prevented. Also the material is cold worked such that small cracks cease to exist.

No analytical prediction of the influence of air hammer peening exists. Maximum crack sizes which can be successfully repaired by peening were determined with tests on full scale cover plated beams.

1.2 Outline of the Work

The first part of this dissertation provides a review of existing crack propagation relationships which take the effect of the fracture toughness on the fatigue life into account. Most of the research in the field of crack propagation was done with aluminum alloys (aircraft industry) and high strength steels (pressure vessels) so that many crack growth rate measurements are restricted to materials other than bridge steels. The suitability of the corresponding crack growth rate relationships for the prediction of the fatigue life of structural details is discussed. Based on the available experimental data a relationship was developed which takes the influence of the fracture toughness into account. Fatigue life
estimates for structural details cover plates and stiffener welded on beams are examined using different levels of material fracture toughness for the fracture criteria.

The same relationship which predicts the crack growth rate as a function of the stress intensity range is used to analyse the fracture behavior of several bridge structures. The fatigue behavior of some of these structures is investigated and the influence of the fracture toughness on the fatigue life is determined. The analysis is made using models provided by linear elastic fracture mechanics. Most of these solutions were derived from two dimensional conditions. The crack configurations in structural components are often of very complex nature, so that some simplifications were inevitable. However these calculation models can be verified by the known fracture behavior of the investigated structure.

Air hammer peening is a method to repair fatigue damage in structural details. In the third part the influence of air hammer peening of coverplated beams is investigated. To analytically predict the fatigue behavior, the residual stress field after peening has to be known. Estimates of the residual stress in the peened zone are made based on measurements. The influence of the peening variables such as air pressure and peening time are investigated. The tool geometry is kept constant for convenience. A model based on the principles of fracture mechanics is developed and the influence of the yield strength of the material, the minimum stress and the stress range is investigated. The analytically predicted
behavior is compared with the test results in full size coverplated beams provided by Ref. (23, 83).
2. INFLUENCE OF THE FRACTURE TOUGHNESS ON THE FATIGUE BEHAVIOR

2.1 Background

Experimental research on the fatigue behavior extends over the last 100 years. It was August Wöhler\textsuperscript{37} in 1870 who discovered that railroad axles may fracture under repeatedly applied loads. Since then more sophisticated experimental studies were undertaken to investigate the basic fatigue and fracture behavior and the behavior of structural components under cyclic applied loads.

2.1.1 Physical Behavior

Fatigue crack growth rates da/dN have been measured for a great number of loading conditions in different environments for various materials. With crack size, applied load and geometry known, these rates of growth have been related to the stress intensity range, $\Delta K$. In general three zones of fatigue crack growth rates have been defined as shown in Fig. 2.1.

The lower limit of the stress intensity range in Zone I is given by the threshold stress intensity range, $\Delta K_{\text{Th}}$. When the stress intensity range is smaller than $\Delta K_{\text{Th}}$, fatigue cracks will not propagate under cyclic applied loads. $\Delta K_{\text{Th}}$ is the smallest stress intensity range for which crack propagation occurs under cyclic applied loads. The threshold stress intensity range depends on the material and on the loading conditions. The loading conditions can be characterized by the stress intensity range and the R-ratio, where $R$ is defined in Eq. 2.1 and the stress intensity range $\Delta K$ in Eq. 2.2.
\[ R = \frac{K_{\text{Min}}}{K_{\text{Max}}} \quad (2.1) \]

\[ \Delta K = K_{\text{Max}} - K_{\text{Min}} \quad (2.2) \]

Where \( K_{\text{Max}} \) is the maximum stress intensity factor and \( K_{\text{Min}} \) the minimum stress intensity factor. With a given stress intensity range and R-ratio the loading conditions are completely defined. The maximum stress intensity, \( K_{\text{Max}} \), can be expressed in terms of Eqs. 2.1 and 2.2 as

\[ K_{\text{Max}} = \frac{\Delta K}{1-R} \quad (2.3) \]

The minimum stress intensity, \( K_{\text{Min}} \), can be expressed as

\[ K_{\text{Min}} = R K_{\text{Max}} = R \frac{\Delta K}{1-R} \quad (2.4) \]

For a given crack geometry Eq. 2.1 can also be expressed as

\[ R = \frac{\sigma_{\text{Max}}}{\sigma_{\text{Min}}} = \frac{P_{\text{Max}}}{P_{\text{Min}}} \quad (2.5) \]

Where \( \sigma \) is the stress and \( P \) the applied load. For stress intensity ranges slightly higher than \( \Delta K_{\text{Th}} \), cracks grow in a subcritical stable manner. It is very difficult to determine accurately the threshold stress intensity range because of the rapid decrease of the crack propagation rate for a small decrease of the stress intensity range. The threshold stress intensity range has been investigated by various researchers\(^{24,25,26,27}\) during the last two decades for various
materials and loading conditions. A review of some results is provided in Section 2.3.

Zone II (see Fig. 2.1) represents the fatigue crack propagation conditions above $\Delta K_{Th}$. The crack propagation rate is best represented as a function of $\Delta K$ using a straight line logarithmic function for steels. Thus, in Zone II the crack growth rate $da/dN$ can be expressed by the following equation which was introduced long ago by Paris

$$\frac{da}{dN} = C \Delta K^n$$

(2.6)

The average crack growth constant $C$ and the crack growth exponent $n$ for structural steels were determined by Hirt and Fisher from available test data. Barsom has examined the variation of $C$ and $n$ for different types of steels. All studies indicate that the $R$ ratio has a relatively small influence on the crack propagation rate in Zone II.

In Zone III (see Fig. 2.1) the crack growth rate increases faster than predicted by Eq. 2.6 for increasing stress intensity ranges. Equation 2.6 is widely useful. It is referred to hereafter as the simple power law (SPL).

When the maximum stress intensity factor approaches the material toughness, $K_c$, crack instability develops. The transition between Zone II and Zone III depends mainly on the yield strength of the material.
Fatigue life will be terminated when the maximum stress intensity at the crack front reaches the effective material toughness. The material toughness is a function of the temperature, strain rate and the plate thickness. For thicknesses greater than some value related to the toughness and yield strength, plane strain behavior results. For plane strain behavior the fracture toughness has a minimum value called $K_{IC}$. For service temperatures and strain rates encountered in bridge structures the plate thickness is not big enough to provide plane strain conditions. For a crack front in a flange approaching the back free surface the thickness of the remaining ligament influences the $K_C$ value. A very thin remaining ligament is very ductile and the fracture resistance $K_C$ is very high. In this study, brittle fracture was assumed to occur when the maximum $K$-value on the crack front reached $K_C$. Different stages of crack sizes and related $K_C$ values are shown schematically in Fig. 2.2.

In Ref. (11) and Ref. (12) it was reported that the complete failure of a beam during fatigue testing occurred when the crack had completely penetrated the flange and the deflection increased. The reported tests were carried out at room temperature and the cracks were in thin plates, so that a high fracture toughness resulted. Tests carried out on beams with thicker flanges at lower temperatures showed another failure behavior. Failure occurred as brittle fracture of the beam which destroyed the cross-section. Two fracture surfaces from Ref. (36) are shown in Figs. 2.3 and 2.4. The beams were cycled and experienced fatigue crack growth. They were also
cycled at low temperatures until fracture occurred. The critical temperature was -101°C for beam B5 and -73°C for beam B5A. The measured material $K_c$-values were 52 MPa$\sqrt{m}$ at -101°C for beam B5 and 136 MPa$\sqrt{m}$ at -73°C for beam B5A. Similar fracture behavior for cracks starting at other structural details such as stiffeners and attachments were also reported.  

2.1.2 Crack Propagation Relationships

Crack propagation rates $da/dN$ were measured by various researchers for various materials and loading conditions over the last years. $da/dN$ is usually related to the stress intensity range, $\Delta K$. These test data are then represented by various "Crack Propagation Laws." Researchers would fit the data points by an equation that represented fairly well the test results.

Most of the crack propagation relationships were empirical and gave the number of cycles as a function of the crack length. These observations are obviously only valid for a special type of test specimen. These empirical crack growth relationships could not account for other details and crack conditions; they are not suitable for fatigue life calculations of structural details.

Crack growth rate relationships may take the stress intensity range into account, but neglect the threshold stress intensity range and also the accelerated crack propagation conditions that are observed at the maximum stress intensity approaches the material fracture resistance. Paris first recognized that, within limits,
fatigue crack growth rate can be expressed as the simple power law given by Eq. 2.6.

Richards and Lindley\textsuperscript{15} found that crack growth is little affected by minimum stress and constraint (thickness) when $K_{\text{Max}}$ is less than 0.6 - 0.8 times $K_c$. They based this observation on the crack growth mechanism. As a result they concluded that Eq. 2.6 can be used to predict crack growth rates except at low and high values of $\Delta K$ (i.e. Zones I and III in Fig. 2.1).

Gurney\textsuperscript{28} observed that the crack growth exponent in Eq. 2.6 is affected by the material. Barsom\textsuperscript{9} determined $n$ for martensitic steel as 2.25 and for ferrite-pearlite steel as 3.0. However, an examination of the test data indicates that the crack growth exponent for martensitic steel is mainly governed by growth rates above $2.5\times10^{-5}$ mm/cycle. At lower growth rates about the same exponent was observed for martensitic and ferrite-pearlite steels using wedge-opening-loading (WOL) specimens. These tests were carried out at room temperature in air environment.

Fisher et al.\textsuperscript{11,12} investigated the fatigue behavior of beams with structural details welded on them. The structural details were cover plates, stiffeners and gussets. From a statistical analysis of the test results the crack growth exponent was found to be about 3.0, depending on the structural detail\textsuperscript{6,7}.

The crack growth constant $C$ in Eq. 2.6 was also determined by Hirt\textsuperscript{6} for bridge steels. He found that

14
\[ C = 1.24 \times 10^{-13} \frac{\text{mm}^{5.5}}{\text{N}^3 \text{ cycle}} \quad (\equiv 2.05 \times 10^{-10} \frac{\text{in}^{5.5}}{\text{kip}^3 \text{ cycle}}). \] A value of
\[ C = 2 \times 10^{-10} \frac{\text{in}^{5.5}}{\text{N}^3 \text{ cycle}} \] was usually employed. This value represents a mean value for the crack growth constant. The upper limit\(^{10}\) for growth rates was found to be
\[ 2.179 \times 10^{-13} \frac{\text{mm}^{5.5}}{\text{N}^3 \text{ cycle}} \quad (3.6 \times 10^{-10} \text{ in English units}). \] Barsom and Novak\(^{8}\) found \( C \) to be
\[ 3.995 \times 10^{-13} \frac{\text{mm}^{5.5}}{\text{N}^3 \text{ cycle}} \] for martensitic steel and
\[ 2.179 \times 10^{-13} \frac{\text{mm}^{5.5}}{\text{N}^3 \text{ cycle}} \] for ferrite-pearlite steels. Beam tests did not provide a significant difference in the values on \( C \) and \( n \) for either ferrite-pearlite steel and martensitic steel.

The SPL relationship proposed by Paris does not consider the accelerated crack growth rate as \( K_{\text{max}} \) approaches the material fracture toughness and the behavior near the threshold stress intensity range. Even the smallest stress intensity range yields crack growth. Eq. 2.6 is used because of its simplicity and because the most useful life in bridge structures is for \( \Delta K \)-values between about 5 MPa\(\sqrt{\text{m}} \) and 30 MPa\(\sqrt{\text{m}} \). To take the crack growth rate near the threshold stress intensity range \( \Delta K_{\text{Th}} \) into account Lukas and Klesni\(^{13}\) have shown that an equation of the form

\[ \frac{da}{dN} = C (\Delta K^n - \Delta K_{\text{Th}}^n) \quad (2.7) \]
describes the crack growth rate for zero mean load. Priddle\textsuperscript{14} postulated, that the crack growth rate can be described by Eq. 2.8
\[
\frac{da}{dN} = C \left( \Delta K - \Delta K_{Th} \right)^n
\]  \hspace{1cm} (2.8)

To account for the R-ratio $\Delta K_{Th}$ has to be a function of $R$.

Donahue et al\textsuperscript{16} suggested that $n = 2$ for Eq. 2.7 and determined the crack growth constant in Eq. 2.7 to be:
\[
C = \frac{4A}{\pi \sigma_Y E}
\]  \hspace{1cm} (2.9)

In the empirical relationship Eq. 2.9 the factor $A$ was thought to account for the influence of environmental conditions, $\sigma_Y$ is the yield strength and $E$ the young's modules of elasticity.

In order to predict the fatigue crack growth rate when the maximum stress intensity factor was near the fracture toughness, McEvily\textsuperscript{17} suggested that the equation given by Donahue (Eq. 2.7) be adjusted by an additional factor and obtained
\[
\frac{da}{dN} = \frac{4A}{\pi \sigma_Y E} \left( \Delta K^2 - \Delta K_{Th}^2 \right) \left( 1 + \frac{\Delta K}{K - K_{Max}} \right)
\]  \hspace{1cm} (2.10)

McEvily modified equation 2.10 to take minimum stress into account and changed the crack growth constant into the non dimensional factor. This yielded the relationship given in Eq. 2.11
\[
\frac{da}{dN} = \frac{C}{E^2} \left[ \Delta K^2 - \Delta K_{Th}^2 \left( R \right) \right] \left( 1 + \frac{\Delta K}{K - K_{Max}} \right)
\]  \hspace{1cm} (2.11)

An empirical relationship was used to estimate the dependency of the threshold stress intensity range as a function of $R$, Eq. 2.12
\[
\Delta K_{\text{Th}} (R) = \frac{1.2 \Delta K_{\text{Th}}^*}{1 + 0.2^{1+R \over 1-R}} 
\]

where \(\Delta K_{\text{Th}}^*\) is the threshold stress intensity range for \(R=0\).

McEvily\textsuperscript{19} subsequently altered Eq. 2.11 so that the crack growth rate was:

\[
\frac{da}{dN} = \frac{C}{E^2} \left[ \Delta K - \Delta K_{\text{Th}} (R) \right]^2 \left( 1 + \frac{\Delta K}{K_c - \Delta K} \right) 
\]

and to predict \(\Delta K_{\text{Th}} (R)\) for different \(R\)-ratios.

\[
\Delta K_{\text{Th}} (R) = \left( \frac{1-R}{1+R} \right)^{1/2} \Delta K_{\text{Th}}^* 
\]

Also Eq. 2.14 is an empirical relationship. Both relationships suggested by McEvily, Eqs. 2.11 and 2.13 will be used to predict the fatigue life of beams with structural details welded on it. Both equations are represented in graphical form in Fig. 2.5, for comparison also the relationship suggested by Paris is plotted.

Other crack growth relationships have been suggested. Forman's\textsuperscript{20} equation takes the crack growth rate at medium and high stress intensity ranges into account. However the stress intensity range near \(\Delta K_{\text{Th}}\) is neglected. This results in

\[
\frac{da}{dN} = C \frac{\Delta K}{K_c (1-R)-\Delta K} 
\]

Forman's equation is represented in graphical form in Fig. 2.6. The values of the crack growth constant and the crack growth exponent were determined by Hartbower\textsuperscript{21} on A514 steel with WOL-specimens. The \(R\)-ratio has a significant influence on the crack growth rate at medium stress intensity ranges (see Zone II, Fig.2.1).
This is not in accordance with the observation of Richard and Lindley\textsuperscript{15}. Also, Fisher et al.\textsuperscript{11,12} found that the R-ratio had little or no effect on the fatigue behavior of welded details. Hence, Eq. 2.15 does not appear suitable for fatigue life calculations of structural details in bridge steel. Forman developed his equation mainly for aluminum alloys. Later, Pearson\textsuperscript{18} modified Forman's equation in the following way:

\[
\frac{da}{dN} = C \frac{\Delta K^p}{[(1-R)K_c - \Delta K]^{1/2}}
\]

(2.16)

Also Pearson's equation shows an influence of the R-ratio at medium stress intensity ranges. The Forman-, Pearson- and McEvily Equations were developed for aluminum alloys, the constants in Eqs. 2.11, 2.13, 2.15 and 2.16 are not available in the literature for structural steels.

To account for accelerated crack growth rates in Zone III (see Fig. 2.1) the crack growth rate equations must be multiplied by an additional factor. The following relationships have been suggested:

\[
1 + \frac{1-R}{K_c(1-R)-\Delta K}
\]  
\text{(McEvily)}

\[
\frac{1}{K_c(1-R)-\Delta K}
\]  
\text{(Forman)}

\[
\frac{1}{K_c(1-R)-\Delta K}^{1/2}
\]  
\text{(Pearson)}

All relationships have in common that for $K_{\text{Max}}$ approaching the fracture toughness the crack growth rate increases to infinity. Other
relationships also exist which consider various other effects such as temperature, environment, test frequency, etc. These effects will not be considered here.

2.2 Fatigue Life Estimations

2.2.1 Introduction

In recent years efforts were made to predict the fatigue life of structural details by tests and analytically. An exact analytical prediction of the fatigue life depends on an accurate estimate of the stress intensity range, $\Delta K$. Zettlemoyer did extensive work developing $\Delta K$-values for structural details such as cover plates and stiffeners welded on beams. The fatigue life can be determined by integrating Eq. 2.17 between two limiting crack lengths.

$$\frac{da}{dN} = f(\Delta K)$$

(2.17)

Because $\Delta K$ is a function of crack length, closed form integration of Eq. 2.17 is only possible in a few relative simple cases.

The fatigue life estimates can be verified by experiments. Two major research programs were undertaken between 1966 and 1974 to define the fatigue strength of structural details welded on small beams. Based on these test results design rules were established (see Fig. 2.8). These studies found that the failure initiated at surface flaws due to non-metallic inclusions along the fillet weld connecting the cover plate or stiffener to the beam. These discontinuities reside in zones of high stress concentration at the weld
toe and act as crack initiation sites. Several structural details with the crack locations are shown schematically in Fig. 2.9. During the fatigue tests, a deflection criteria was used to define the usefull number of load cycles. An increase of the midspan deflection of about 0.5 mm was found to be equivalent to a crack size that was considered to be failure of the section. Roughly 70% of the flange was cracked at that time. This indirect criteria can not be used for the analytical fatigue life estimates. The fatigue life was assumed to be exhausted when brittle fracture of the remaining section occurred. The maximum stress intensity factor was equated to various levels of material toughness. This fracture criteria was also used in a recent experimental study. For comparison purposes calculations were also made with an infinite fracture toughness which provided an upper bound fatigue life. For an infinite fracture toughness the crack was assumed to completely penetrate the flange of the beam.

2.2.2 Results From Fatigue Tests

The experimental results from laboratory tests from Ref. (11) and Ref. (12) were used to evaluate the adequacy of the calculated fatigue life. Ref. (11) and Ref. (12) report the fatigue behavior of several details: rolled and welded beams, beams with cover plates of different dimensions welded in different ways on the beams, different types of stiffeners and gussets welded on the beams. Based on these test results, fatigue categories A to E in Fig. 2.8 were developed. Later the fatigue behavior of full size cover-
plated beams was investigated\textsuperscript{23,36} and the fatigue design
Category E' was defined. Full size beams with other structural de-
tails were also tested but showed no size effect. The current de-
sign categories for fatigue life are shown in Fig. 2.8 which
represents the lower confidence limit of the test data.

For comparison purposes the same beam sizes and dimensions
were used for the analytical calculations. The dimensions of the
beams are shown in Table 2.1. Based on the test results the follow-
ing regression equations for small scale cover-plated beams with
transverse end welds were developed in Ref. (11):

\[
\log N = 11.8873 - 3.095 \log \Delta \sigma
\]  \hspace{1cm} (2.18)

By making use of the standard error estimate for Eq. 2.18
(s = 0.101) it is possible to define the upper and lower 95% confi-
dence limit for 95% survival as:

\[
\log N = 11.8873 - 3.095 \log \Delta \sigma \pm 2s
\]  \hspace{1cm} (2.19)

The lower 95% confidence limit is identical to design category E in
Fig. 2.8.

Stiffeners fillet-welded to the tension flanges are designated
Type 3 in Ref. (12). For these type of stiffeners the following re-
gression equation provided a good estimate of the fatigue life:

\[
\log N = 13.5342 - 3.505 \log \Delta \sigma
\]  \hspace{1cm} (2.20)

The standard error of estimate for Eq. 2.20 was determined to be
0.1024 so that the upper and lower confidence limits are defined as:

\[
\log N = 13.5342 - 3.505 \log \Delta \sigma \pm 0.2048
\]  \hspace{1cm} (2.21)

Eq. 2.20 has a slope of -3.505 in the log N - log \Delta \sigma diagram. In the
AASHTO specifications the slope for the design curves in the log $N$ –
log $\Delta \sigma$ diagram is about $-3.0$ for all details. (It is not
exactly $-3.0$ because of round off values given in numerical form).
Also the Category C curve is used to describe the test results of
stiffeners type 3 and other similar details such as short shear
connectors, groove welds with reinforcements, etc. In general, as
larger amounts of experimental data become available so more com-
patible the exponent becomes with $-3.0$. For fatigue life cal-
culations a slope of $-3.0$ is usually used for all details. Also
the test results are reasonably represented with a slope of $-3.0$.
The test results of the stiffener tests are compared with the
regression line and the exponent $-3.0$ in Fig. 2.10. The line with
exponent $-3.0$ is given by Eq. 2.22.

$$\log N = 12.4534 - 3.0 \log \Delta \sigma$$  (2.22)

The standard error for Eq. 2.22 is bigger than for Eq. 2.21
(0.1281 vs. 0.1024). It is apparent from Fig. 2.10 that the test
results at the lower stress range level ($\Delta \sigma = 100$ MPa) tend to make
the slope smaller than $-3.0$. However, the test results are well
represented by Eq. 2.22. Kouba et al.\textsuperscript{48}, Gurney\textsuperscript{49} and Braitwaite\textsuperscript{50}
also conducted fatigue tests with beams with welded stiffeners.
Their test results are compared with Eq. 2.22 in Fig. 2.11. Most of
these test results fall within the scatterband predicted by Eq. 2.22
with a standard deviation of $s = 0.1281$. Eq. 2.22 will be used to
compare the results of the analytical studies.

Not enough data was available for a statistical analysis of the
test results on cover-plated beams with a flange thickness bigger
than 20 mm. Design category E' was constructed with the same slope as for the other design curves as a lower bound to the available test results. The equation for the design Category E' can be calculated from the numerical values given in the AASHTO specification.

\[ \log N = 10.875 - 2.954 \log \Delta \sigma \]  

(2.23)

Also here is the slope not exactly - 3.0 in the \( \log N - \log \Delta \sigma \) diagram because of the rounded values in the specifications.

In the research project described in Ref. (36) an other criteria to define the useful fatigue life of full size beams with cover plates was used. During the fatigue tests the cross-section containing the fatigue crack was cooled to temperatures between -43° C and -129° C. The load was cyclic applied until brittle fracture occurred. Two examples of the fracture surface are shown in Figs. 2.3 and 2.4. The test results reported in Ref. (36) are shown in Fig. 2.14. As can be seen from Fig. 2.14 all beams developed the fatigue life of design Category E'.

### 2.2.3 Fatigue Life Calculations

The stress intensity range \( \Delta K \) is often expressed as a function of a central through crack in an infinite plate under uniform axial tension, adjusted by superimposed correction factor

\[ \Delta K = F_e F_s F_w F_g \Delta \sigma (\pi a)^{1/2} \]  

(2.24)

\( F_e \) is the correction factor for the shape of a surface crack and is given by the following equation:
\[ F_e = \frac{1}{E(k)} \left( 1 - k^2 \cos^2 \phi \right)^{1/4} \]  

(2.25)

where angle \( \phi \) is the phase angle of the ellipse (see Fig. 2.12) and

\[ k^2 = 1 - (a/c)^2 \]  

(2.26)

where, \( a \), is the semiminor diameter and \( c \) the semimajor diameter of the ellipse. \( E(k) \) is the complete elliptical integral of the second kind.

\[ E(k) = \int_0^\frac{\pi}{2} \left( 1 - k^2 \sin^2 \beta \right)^{1/2} d\beta \]  

(2.27)

\( F \) is the front free surface correction factor. Different solutions are found in the literature for \( F_s \). For the analysis of the fatigue life of the beams with stiffeners or cover plates Zettlemoyer's\(^{22}\) solution was employed which gives the free surface correction in the form of Eq. 2.28.

\[ F_s = f \text{ (structural detail, crack geometry, plate geometry)} \]  

(2.28)

\( F_w \) is the correction factor for the finite width (also called back free surface correction factor). Because the fatigue crack grows from one side of the plate into the flange, the flange has a tendency to bend. This bending amplifies the back free surface correction. At most structural details the bending is somewhat restrained by the web and/or stiffener. For completely unrestrained bending the back free surface correction factor is given by\(^{22,53}\)

\[ F_w = Q \left( \frac{2b}{\pi a} \tan \frac{\pi a}{2b} \right)^{1/2} \]  

(2.29)

where

\[ Q = \frac{0.752 + 2.02 \frac{a}{b} + 0.37 \left(1 - \sin \frac{\pi a}{2b}\right)^3}{1.122 \cos \frac{\pi a}{2b}} \]  

(2.30)
b is the plate thickness. For a completely restrained flange Q is equal to 1.0. The influence of Q on the fatigue life is not very pronounced because most of the useful fatigue life is developed at small crack sizes. For small crack sizes (compared with the flange thickness). a/b is small and Q is about unity. For bigger cracks the bending effect is more pronounced.

$F_g$ is the stress gradient correction factor. Zettlemoyer\textsuperscript{22} developed an approximate equation for the stress correction factor for a crack growing from the toe of an end-welded cover-plated beam. This relationship was

$$F_g = \frac{\text{SCF}}{1 + \frac{1}{0.1473 \left( \frac{a}{b} \right)^{0.4348}}}$$  \hfill (2.31)

where SCF is the stress concentration factor. The stress concentration factor was determined at the toe of the weld for the uncracked section. The following function was derived from the numerical study:

$$\text{SCF} = 3.539 \log \frac{z}{t_{FL}} + 1.981 \log \frac{t_{Cp}}{t_{FL}} + 5.788$$  \hfill (2.32)

where $z =$ weld leg size

$t_{FL} =$ Flange thickness

$t_{Cp} =$ Cover plate thickness

For stiffeners welded on the flange, Zettlemoyer\textsuperscript{22} developed the following expression for the stress gradient correction factor.

$$F_g = \frac{\text{SCF}}{1 + \frac{1}{0.3546 \left( \frac{a}{b} \right)^{0.1543}}}$$  \hfill (2.33)

The stress concentration factor, SCF, was found to be
\[
SCF = 1.621 \log \frac{Z}{t_{f1}} + 3.963
\] (2.34)

The value of \( F_g \) for any position along the crack front can be obtained by substituting \( a^* \) as defined in Fig. 2.12 in Eq. 2.31 or 2.33.

Fatigue cracks originate along the fillet welds connecting the cover plate or stiffener to the flange. These cracks start as shallow surface cracks and grow through the flange as a semiellipse. During crack growth the shape tends to approach a semicircle. There is usually more than one crack initiation site along the weld toe and the cracks tend to coalesce and form larger cracks of irregular shape. The number of initiation sites and the degree of coalescence depends on the configuration and uniformity of the weld\(^44\). Transverse end welds at cover plate ends and stiffeners welded to the flange are usually made by hand. This increases the imperfections of the weld. The more imperfections the weld contains, the more crack initiation sites are present and the cracks coalesce more often.

The crack shape has a major influence on the growth characteristics of a fatigue crack. The ratio \( a/c \), the minor to the major semidiameter of the fatigue crack is important when calculating the correction factors. Gurney\(^45\) found that the importance of the crack growth relationship increases for increasing stress concentration factors. The stress concentration factor for small cover plated beams is equal to 6.78 (Eq. 2.32. The full size cover-plated beams
result in a value equal to 7.21. The crack shape during growth is therefore more important for the full size beams. Based on measurements on full scale beams the following relationship was established as a lower bound by Slockbower\textsuperscript{10} for crack coalescence:

\[
c = 3.549 a^{1.133} \quad [\text{mm}]
\]  
(2.35)

Eq. 2.35 provides a lower bound for the crack shapes, see Fig. 2.14. For single cracks developing at the end of a longitudinal weld Eq. 2.36 was found to describe the crack shape during early growth\textsuperscript{46}:

\[
c = 3.355 + 1.29 a \quad [\text{mm}]
\]  
(2.36)

Eq. 2.36 provides a straight line relationship; a half circular crack is approached, but never reached.

Crack shape relationships have been developed for cracks growing from stiffeners welded to beams. The shape for small single cracks was not adequately taken into account. The crack shape relationship after coalescence predicts average values. Crack shape measurements suggest the following two relationships for stiffeners

\[
c = 0.279 + a \quad [\text{mm}]
\]  
(2.37)

and

\[
c = 1.403 a^{0.9505} \quad [\text{mm}]
\]  
(2.38)

Eq. 2.38 provides a lower bound for the crack shape. The crack shape equations and the measured data are compared in Fig. 2.14.

Eqs. 2.37 and 2.38 were derived with a curve fit through the available data points. As can be seen from Fig. 2.13 the measurements are not evenly distributed over the full plate width. The curve fits are therefore not very accurate. As soon as there is more data available the crack shape relationships may be improved.
With the known stress intensity range $\Delta K$ (Eq. 2.24) the fatigue life of a structural detail can be calculated from Eq. 2.17.

Eq. 2.17 can be expressed as

$$dN = \frac{da}{f(\Delta K)} \quad (2.39)$$

The number of cycles $\Delta N$ between crack sizes $a_i$ and $a_f$ can be obtained by integration of Eq. 2.39 which yields

$$\Delta N = \int_{a_i}^{a_f} \frac{1}{f(\Delta K)} \, da \quad (2.40)$$

The initial crack size is very important for the fatigue life, but unfortunately it is also very difficult to establish. Several investigations have determined a lower bound initial crack size at the weld toe of stiffeners and cruciform joints to be 0.025 mm. The average initial crack size $a_i$ is between 0.076 mm and 0.10 mm and the upper bound is between 0.6 mm and 0.7 mm.

To obtain a numerical solution of the stress intensity range and integrate Eq. 2.40 an existing computer program developed by Albrecht$^{47}$ and Zettlemoyer$^{22}$ was modified for use with different crack propagation relationships. The cycle life was determined by numerical integration as

$$N = \sum_{j=1}^{m} \frac{1}{f(\Delta K)} \, \Delta a_j \quad (2.41)$$

The term $\frac{1}{f(\Delta K)}$ is treated as an integral, defined between two crack sizes $a_j$ and $a_j + 1$ using the 32-point gauss quadrature formula. This permits the use of larger crack increments $\Delta a_j$ without loss of accuracy.
The fatigue life can be calculated from Eq. 2.41 and Eq. 2.24 as long as the crack has an elliptical shape and the crack front does not reach the opposite surface of the flange. As the crack penetrates the flange thickness it becomes a through crack and the analytical model for $\Delta K$ must be altered. Experimental evidence has shown that 96% or more of the fatigue resistance is exhausted when the crack penetrates the flange. These fatigue tests were all carried out at a room temperature so that the fracture toughness of the flange plates was high. No rapid fracture occurred during these tests. The fatigue life given by Eq. 2.19 or 2.22 can be adjusted for 96% of the fatigue life. For small size cover plates Eq. 2.19 yields

$$\log (0.96N) = 11.8696 - 3.095 \log \Delta K$$

The 96% life for stiffeners welded on beams and for full size cover plates can be derived similarly. Fig. 2.15 shows that the difference between the 96% and the 100% life prediction is very small. The crack shape relationship and the initial crack size have a more pronounced influence on the fatigue life.

In Fig. 2.15 the fatigue lives for different crack growth relationships using the simple power law in Eq. 2.40 are shown. The fatigue life estimated using the crack shape equation (Eq. 2.35) developed by Stockbower and Fisher\textsuperscript{10} shows the best agreement with the tests. Stockbower predicts long shallow cracks with many initiation sites that coalesce. Because of the good agreement between the estimated life and the test results the crack shape relationship
provided by Eq. 2.35 will be used for cover plates and stiffener details.

2.3 Fatigue Behavior of Bridge Steel

2.3.1 Introduction

The basic behavior of bridge steels has been determined with experiments on compact tension specimens or on plate specimens with a center crack. The applied conditions are the cyclic applied load, the crack length a, the number of cycles and the geometry of the specimens. The test data has been used to develop da/dN - ΔK relationship in different ways. da/dN measurements for different loading conditions were made in all 3 zones (see Fig. 2.1) of fatigue crack growth.

2.3.2 Fatigue Behavior at Low ΔK-Values

Crack propagation rates at low stress intensity ranges are often smaller than predicted by Paris' SPL (Eq. 2.6). Even August Wöhler believed that a stress intensity range exists, below which no fatigue damage occurs. He reported on fatigue tests with more than 100 million cycles without failure. The highest stress intensity range for which no fatigue crack propagation occurs is called the threshold stress intensity range. Various research projects have examined this limit for various materials and loading conditions.

Klingerman tested several A36 steel plate specimens with center cracks under different loading conditions. He varied the
R-ratio and the stress intensity range and predicted the threshold value to be between 3.6 MPa√m and 5.8 MPa√m.

Paris et al.\textsuperscript{25} published crack growth data obtained with compact tension specimen fabricated from A533 grade B, class 1 pressure vessel steel. He investigated the dependency of the crack growth rate on the stress ratio and other conditions such as environment and temperatures. Threshold stress intensity ranges were established as lower limits. An investigation was also carried out on ASTM A508, class 2, forged steel\textsuperscript{25}.

Bucci et al.\textsuperscript{26} obtained test data on high strength ASTM A577 grade F steel. He also used compact tension specimen and established the crack growth threshold values for several stress ratios.

Several materials under various loading and environmental conditions were tested by Pook\textsuperscript{27}. The results for ferrous materials such as mild steel, low alloy steel and 18/8 austenitic steel are of interest to this study.

Unfortunately most of these studies do not deal with bridge steels. However, it appears that the behavior of these special steels is similar to the behavior of bridge steels in the region of the threshold stress intensity range.

2.3.2.1 Review of Test Data

Paris's tests at constant amplitude in room air at 24° C show a clear dependency of the fatigue threshold stress intensity range on the R-ratio. Very slow fatigue crack growth was measured and the
corresponding fatigue crack growth rates, da/dN, calculated. The fatigue threshold values were established from curve-fits or from run out tests. Run out tests are tests at a given crack length and stress condition for which no crack growth occurs. Paris found a threshold value of 8 MPa√m for R=0.1 and 3.0 MPa√m for R=0.8 for A533 B-1 steel. Similar data was obtained for A508-2 forged steel. The results are summarized in Table 2.2 and the da/dN - ∆K measurements are shown in Fig. 2.16 through 2.20 for the different R-ratios. To obtain one threshold value Paris used between ten and twenty data points for the curve fit.

The influence of the stress ratio is also apparent in Bucci's work. For A517 grade F steel he found that the ∆K_{Th} -value is 3.3 MPa√m for R=0.9 and about 5.5 MPa√m for R=0.2. His tests were carried out at constant amplitude with a frequency of 150 cps at room temperature. The tests were carried out with compact tension specimens.

Pook's investigation provides results on about 30 different materials and different environmental conditions. He only used a few data points to determine the ∆K_{Th} -value. This resulted in ∆K_{Th} = 6.6 MPa√m for R = 0.13 and 3.8 MPa√m for R = 0.75 for mild steel. ∆K_{Th} -values of the same order were found for his investigation of low alloy and 18/8 austenitic steel.

To determine the threshold stress intensity range a curve fit with the measurements provided by Klingerman on A36 steel was made. The function for the least square curve fit is
\[ \Delta K = C_1 e^{C_2 (da/dN)} \]  

(2.43)

The function of this type was chosen because as \( da/dN \rightarrow 0 \) the stress intensity range \( \Delta K \) approaches \( C_1 = \Delta K_{Th} \). This function for the curve-fit was only used for test series which provided more than 6 data points in the threshold region. A limit, above which the data points were not considered was established in order to obtain the curve fit in the threshold region. Based on these assumptions the fatigue threshold stress intensity range could be calculated. The suitability of the curve-fit provided by Eq. 2.43 was examined by comparing the results with the \( \Delta K_{Th} \)-values obtained by Paris, Bucci and Pook. The result from those investigators could be predicted within \( \pm 10\% \). The \( \Delta K_{Th} \)-values and the coefficient of correlation, \( r^2 \), are also shown in Table 2.2.

Klingerman's data on crack growth rates near \( \Delta K_{Th} \) was analysed with Eq. 2.43. The obtained \( \Delta K_{Th} \)-values and the \( \Delta K_{Th} \)-values from the other investigators\(^{25,26,27}\) are plotted in Fig. 2.21. At a stress ratio of \( R = 0.192 \), \( \Delta K_{Th} \) is 8.8 MPa√m, at a high stress ratio \( (R = 0.877) \), \( \Delta K_{Th} \) is 3.5 MPa√m for the A36 steel investigated by Klingerman.

A regression analysis gives the following equation for the A533 grade B class 1 steel.

\[ \Delta K_{Th} = 8.4 \ (1-0.85R) \quad [\text{MPa}\sqrt{\text{m}}] \]  

(2.44)

For the A36 steel the analysis yields

\[ \Delta K_{Th} = 10.2 \ (1-0.69R) \quad [\text{MPa}\sqrt{\text{m}}] \]  

(2.45)

The coefficient of correlation is 0.96 for the A533 B-1 steel and
0.92 for the A36 steel. Eqs. 2.44 and 2.45 are shown in Fig. 2.21 and compared with the test data.

It can be seen from Fig. 2.21 that the $\Delta K_{Th}$ -values for A36 steel are much higher than for the other steels. $\Delta K_{Th}$ measurements are difficult to acquire and test imperfections tend to increase $\Delta K_{Th}$. Eq. 2.45 can therefore be taken as an upper limit.

Harrison\textsuperscript{21} predicted that the fatigue crack propagation threshold range is between

$$\Delta K_{Th} = 7.6 \times 10^{-4} \ E \ \sqrt{\text{mm}}$$  \hspace{1cm} (2.46)

and

$$\Delta K_{Th} = 9.1 \times 10^{-4} \ E \ \sqrt{\text{mm}}$$  \hspace{1cm} (2.47)

For bridge steel the $\Delta K_{Th}$ -value is between 4.9 MPa$\sqrt{\text{m}}$ and 5.8 MPa$\sqrt{\text{m}}$ according to Eqs. 2.46 and 2.47. Harrison did not consider the influence of the R-ratio on $\Delta K_{Th}$.

Barsom\textsuperscript{9} suggested a conservative estimate for the $\Delta K_{Th}$ -value for martensitic steel, ferrite-pearlite steels and austenitic steels to be

$$\Delta K_{Th} = 7.0 \ (1-0.85R) \hspace{1cm} [\text{MPa} \sqrt{\text{m}}]$$  \hspace{1cm} (2.48)

Eq. 2.48 is valid for $R > 0.1$; for stress ratios $<0.1$ a $\Delta K_{Th}$ -value of 6.0 MPa$\sqrt{\text{m}}$ was suggested. A somewhat higher value for $R = 0$ was proposed by Paris who suggested $\Delta K_{Th} = 6.6$ MPa$\sqrt{\text{m}}$.

McEvily\textsuperscript{19} found the lower bound for the threshold stress intensity range to be

34
\[ \Delta K_{Th} = \left( \frac{1-R}{1+R} \right)^{1/2} \Delta K_{Th}^o \]  
(2.49)

\( \Delta K_{Th}^o \) is the fatigue threshold stress intensity range for \( R = 0.0 \) for \( \Delta K_{Th}^o = 6.6 \text{ MPa}\sqrt{\text{m}} \). Eq. 2.49 is plotted in Fig. 2.22. Eqs. 2.48 and 2.49 predict about the same \( \Delta K_{Th}^o \)-values. Both provide a lower bound for \( \Delta K_{Th} \) in the region between \( R = 0.0 \) and \( R = 1.0 \). For further calculations the relationship suggested by McEvily, Eq. 2.49 will be used.

2.3.3 **Fatigue Behavior at Medium \( \Delta K \)-Values**

Extensive studies of the fatigue crack growth properties in Zone II (see Fig. 2.1) has demonstrated that the fatigue crack growth rate is best predicted by the simple Power Law, Eq. 2.6.

The transition between Zone II and Zone III is the point of onset of accelerated stable fatigue crack propagation. In Zone III the propagation rate becomes bigger than predicted by the Paris Power Law. Barsom^8 shows that at zero minimum load the transition point \( K_T^o \) is at

\[ K_T^o = 0.04 \sigma_Y \]  
(2.50)

\( K_T^o \) is a function of yield stress and the modules of elasticity and is independent of the fracture toughness. Barsom derives \( K_T \) based on a crack-opening-displacement range \( \Delta \delta \). The onset of accelerated crack growth occurs at a COD (crack-opening-displacement) of 0.04 mm. The COD is given by

\[ \Delta \delta = \frac{\Delta K}{E \gamma} = 0.04 \quad [\text{mm}] \]  
(2.51)
For higher R-ratios, the transition between Zone II and Zone III occurs when the maximum stress intensity factor reaches the critical value of 0.04 mm. Hence the transition point can be approximated as:

$$\Delta K_T(R) = (1-R) K_T^0$$ (2.52)

Barsom's test data with R = 0.0 are shown in Fig. 2.23. Klingerman et al. conducted tests at different R-ratios between 0.09 and 0.38 at stress ranges of 110 MPa and 138 MPa on A36 steel. The investigated A36 steel had a yield strength of 252 MPa. The results and the $\Delta K_T$ after Eq. 2.51 and 2.52 are shown in Figs. 2.24 and 2.25. To predict $\Delta K_T(R)$ stress ratios of 0.1 and 0.35 were used.

Klingerman et al. concluded that the transition between Zone II and Zone III is associated with the transition from elastic to plastic net section stress. This criterion is applicable to laboratory plate specimens but is difficult to apply to structural members. In structural shapes the net section stress often remains in the elastic region even for large crack sizes.

2.3.4 Fatigue Behavior at High $\Delta K$-Values

Klingerman et al. also obtained fatigue crack growth data on A36 steel at high $\Delta K$-levels. The crack growth rates were higher than predicted by the simple Power Law. Klingerman concluded that elastic fracture mechanics concepts were valid even when applied to regions in which plasticity was evident. The K-value can be satisfactorily adjusted by the plastic zone correction.
Klingerman did his study on plate specimens with a center crack; to start the fatigue cracking he used a notch. He concluded that yielding of the net section had little effect on the crack growth rates.

The maximum load a specimen can sustain is given by the material toughness or the ultimate tensile stress. The procedure to measure the fracture toughness is defined in the ASTM E399 specifications. However, because of plastic deformation of the material and the thickness requirement this specification is not often applicable for bridge steels at test temperatures higher than -40° C. To measure the fracture toughness at temperatures higher than -40° C procedures such as the J-Integral, COD (crack opening displacement) and the R-curve procedure were developed. These procedures take the plastic deformation during testing into account. The $K_C$-value is a function of the energy absorption which is represented by the area under the $P-\delta$ curve of the test record. Therefore, the bigger the deformation at the critical load, the higher is the $K_C$-values. $K_C$-values of 200 MPa\(\sqrt{m}\) at -40° C for 10 mm thick compact tension specimens made of A36 steel were reported.

Barsom measured $K_C$-values for A36, A588 and A514 steel at room temperature with compact tension specimens. For A36 steel Barsom determined the $K_C$-values as a function of the maximum load, $P_{\text{Max}}$, independent of the deformation at fracture as:

$$K = \frac{P_{\text{Max}} f(a/w)}{B\sqrt{a}}$$  \hspace{1cm} (2.53)
is a function of the geometry and crack length. No adjustments for plasticity on the crack length was made. The experimental values of the investigation are listed in Table 2.3.

If the critical load calculated with the 5% secant effect is used, K-values about 1/3 the $K_c$ values from the maximum load are obtained for the A36 steel and the A588 steel, 2/3 of the $K_c$-values for the A514 steel results.

An extensive study of the fracture toughness for bridge steels was undertaken by Roberts et al. The results for eight different steels were evaluated. To determine the fracture toughness three point bend specimen of different thickness were used and tested under static, 1 sec and dynamic loading over a wide temperature range. Valid static tests are seldom available at temperatures higher than $-40^\circ$ C when the $K_c$ value is higher than 50 MPa\(\sqrt{m}\). The lowest recorded $K_c$ value at $-40^\circ$ C is for A242 25mm thick steel plate and is about 45 MPa\(\sqrt{m}\).

The study reported by Roberts et al. on the fracture toughness of full size welded steel beams only provides the 1 sec fracture toughness at very low temperatures. It is necessary to extrapolate for higher temperature conditions based on dynamic fracture toughness results.

It can be seen from the various investigations that the fracture toughness values differ with specimen geometry and test method. For this investigation the $K_c$-value will be assumed to be a function of
the maximum load, independent of the deformation at fracture. This means that large deformations at fracture of the specimen or member are required. Low $K_c$-values for minimum service temperatures of bridges are about 55 MPa√m and high toughness values are about 165 MPa√m.

2.4 Crack Propagation Relationships Developed by McEvily

2.4.1 Introduction

In Chapter 2.1.2 it was noted that McEvily proposed two different crack propagation relationships. Both consider the crack growth rate near the threshold stress intensity range and the accelerated crack growth rate as $K_{Max}$ approaches the material fracture resistance.

In 1973 McEvily\(^{17}\) proposed that the crack growth rate $da/dN$ be taken as:

$$\frac{da}{dN} = \frac{C}{E^2} \left( \frac{\Delta K}{\Delta K_{Th}} \right)^2 \left( 1 + \frac{\Delta K}{K_{c} - K_{Max}} \right) \quad \text{(see Eq. 2.11)}$$

In 1979, McEvily\(^{19}\) suggested the following modification

$$\frac{da}{dN} = \frac{C}{E^2} \left( \frac{\Delta K}{\Delta K_{Th}} \right)^2 \left( 1 + \frac{\Delta K}{K_{c} - K_{Max}} \right) \quad \text{(see Eq. 2.13)}$$

$K_{Max}$ in Eqs. 2.11 and 2.13 is defined as a function of $\Delta K$ and $R$ in Eq. 2.4. The only constant in Eqs. 2.11 and 2.13 is $C$, a dimensionless value for known material properties such as the modules of elasticity, the threshold stress intensity range and the fracture toughness.
The McEvily relationships were developed for Aluminum alloys\(^{17,19}\) such as RR58, L64, 2124 – T851 with a fracture toughness of about 40 MPa\(\sqrt{m}\). Based on the known threshold stress intensity range and material toughness the constant \(C\) in Eq. 2.10 and 2.11 can be determined in three different ways:

1.) \(\frac{da}{dN} - \Delta K\) measurements can be approximated with Eqs. 2.11 and 2.13 using a least square curve fit.

2.) Eqs. 2.11 and 2.13 can be approximated with a least square curve fit over a given range with the simple Power Law.

3.) The constant \(C\) can be adjusted based on the results of fatigue tests on laboratory specimens such as small size cover-plated beams or stiffeners welded on beams.

2.4.2 Curve Fit to \(\frac{da}{dN} - \Delta K\) Measurements

Only a few sets of crack growth data (\(\frac{da}{dN}\) vs \(\Delta K\)), material toughness (\(K_C\)) and threshold stress intensity range (\(\Delta K_{th}\)) are available in the literature. Barsom\(^9\) provides data on A36 steel, A588 grade A and grade B steels and for A514 grade E and grade F steels. A least square curve fit to determine \(C\) in Eqs. 2.11 and 2.13 was made for these data sets. Some curve fits are shown in Figs. 2.26 through 2.31. In these figures the straight line from the simple Power Law (Eq. 2.6) with

\[
n = 3.0
\]

and

\[
C = 1.211 \times 10^{-13} \frac{\text{mm}^{5.5}}{N \text{ cycle}}
\]

40
is also shown. All those figures are plotted to the log-log scale, the results are also summarized in Table 2.4. From these test results the average value of C is 9.6 cycle\(^{-1}\) and 12.2 cycle\(^{-1}\) for Eq. 2.13.

From Figs. 2.26, 2.27 and 2.28 it can be seen that the curve fit is good when \(\Delta K\) is larger than 25 MPa\(\rightleftharpoons m\). Eq. 2.11 overestimates the crack growth rates for \(\Delta K\) below 25 MPa\(\rightleftharpoons m\). The threshold stress intensity range was estimated for \(R = 0.1\) according to Eq. 2.49 as \(\Delta K_{Th} = 6\) MPa\(\rightleftharpoons m\). By increasing the \(\Delta K_{Th}\) value to an upper bound value of 9.5 MPa\(\rightleftharpoons m\) the curve fit of Eq. 2.11 is improved in the region below 25 MPa\(\rightleftharpoons m\) as can be seen in Fig. 2.32.

The crack growth relationship can be used to predict the fatigue life of structural details. For comparison purposes the life calculation is made for the same size initial flaws investigated under NCHRP Project 12-711. The numerical procedures and crack shape relationships given in Chapter 2.2 are used to predict the stress intensity. From Figs. 2.26 through 2.31 it is evident, that stress intensity ranges below \(\Delta K_{Th}\) yield no crack growth. The minimum crack size has to be big enough so that the stress intensity range exceeds \(\Delta K_{Th}\). Stress intensity factors equal or larger than \(K_c\) result in fracture. For \(K_{Max}\) approaching \(K_c\) the crack growth rate increases to infinity and the maximum stress intensity range can be determined from Eq. 2.4 by setting \(K_{Max}\) equal to \(K_c\). The maximum stress intensity range \(\Delta K_{Max}\) is given

\[
\Delta K_{Max} = K_c (1-R)
\]  

(2.54)
If $\Delta K$ reaches $K_{\text{Max}}$, the remaining ligament will fracture and terminate the fatigue life.

The predicted fatigue life for several small cover-plated beams is compared with test results in Table 2.5. It is also represented in graphical form in Figs. 2.34 and 2.35. The mean fatigue life and the upper and lower 95% confidence limits of the experimental data are compared with the predicted results. The fatigue life is calculated for a fracture toughness $K_c = 110 \text{ MPa}$\(\sqrt{\text{m}}\). The R-ratio is the R-ratio from the test conditions using the nominal stresses for Eq. 2.1. The average life determined in the tests is 184,200 cycles for a stress range of 138 MPa. The life predicted by Eq. 2.11 at the same stress level at an R-ratio of 0.091 (test conditions) and an initial flaw size of 0.076 mm is 43,800 cycles or 23.8% of the experimental results. Eq. 2.13 predicts 68,100 cycles or 37% of the experimental life. At a low stress range, $\Delta \sigma = 55 \text{ MPa}$ and a R-ratio of 0.555 the life calculated with Eq. 2.11 is only 11% of the experimental results and 27.9% with Eq. 2.13. The fatigue lives provided by Eqs. 2.11 and 2.13 underestimates the fatigue life if the usual initial crack size is assumed. For larger initial crack sizes the predicted fatigue life decreases further. The change in fatigue life for decreasing initial flaw size is also indicated in Figs. 2.34 and 2.35. For a very small initial flaw 0.013 mm, the predicted fatigue life is still less than the experimental results. At the lowest stress range tested no growth is predicted because the stress intensity range is smaller the $\Delta K_{\text{Th}}$. 
The R-ratio given by the minimum to the maximum test load does not describe the actual test conditions at critical locations satisfactorily. Due to welding of the cover plate on the beam, high residual tensile stresses are introduced at the location of the crack path. These residual tensile stresses increase the actual maximum stress up to the yield stress and also increase the minimum stress. The R-ratio must consider the local residual stress as well as the applied stress. The actual R-ratio is between 0.8 and 1.0. However the threshold stress intensity ranges are usually determined with compact tension test with a sharp fatigue crack. To measure $\Delta K_{Th}$, the applied $\Delta K$-value is decreased until no crack growth can be observed. This procedure does not simulate the actual situation in complex details. Fatigue cracks generally start from pores, inclusions and other discontinuities. These weld defects may not be as sharp initially as the fatigue cracks in the test specimens and higher threshold stress intensity range is expected. To simulate this condition, an R-ratio of 0.5 to 0.7 was used to predict the threshold stress intensity range.

The fatigue lives were reestimated for the R-ratio of 0.6 with Eqs. 2.11 and 2.13 for the same $K_c$ value (110 MPa√m) and $\Delta K_{Th}$-value (3.3 MPa√m) for different stress ranges. The results are shown in Table 2.6. The higher R-ratio shortens the fatigue life, but the large differences between the predicted and the measured fatigue life suggest that Eqs. 2.11 and 2.13 are not satisfactory.
2.4.3 Curve Fit to the Paris Equation

The da/dN vs \( \Delta K \) relationship in region II (see Fig. 2.1) has
been investigated by various researchers and can be considered as
well established. The crack growth rate can be predicted by the
simple power law, Eq. 2.6. The values of the crack growth constant
is \( 1.211 \times 10^{-13} \frac{\text{mm}^{5.5}}{N^3 \text{cycle}} \) and the exponent 3.0. Eq. 2.6
is represented by a straight line on the log da/dN vs log \( \Delta K \) dia-
gram. This relationship is valid for \( \Delta K \)-values between about
\( \Delta K_1 = 10 \text{ MPa} \sqrt{\text{m}} \) and \( \Delta K_2 = 40 \text{ MPa} \sqrt{\text{m}} \). \( \Delta K_2 \) depends on the yield
strength of the material and the R-ratio (Eq. 1.52).

McEvily's equations (Eq. 2.11 and 2.13) can be approximated be-
tween \( \Delta K_1 \) and \( \Delta K_2 \) by the simple power law using the least square
method. For a known \( \Delta K_{\text{Th}} \) and \( K_c \) the constant \( C \) in Eq. 2.11 is cal-
culated by Eq. 2.55

\[
\frac{C}{E^2} = C_1 \frac{\int \frac{\Delta K_2}{\Delta K_1} \left( \frac{\Delta K^2}{K_c - K_{\text{Max}}} \right) \left( 1 + \frac{\Delta K}{K_c - K_{\text{Max}}} \right) d\Delta K}{\int \left( \frac{\Delta K_2}{\Delta K_1} \right)^2 \left( 1 + \frac{\Delta K}{K_c - K_{\text{Max}}} \right)^2 d\Delta K}
\]

The value for \( C_1 \) from the SPL is \( 1.211 \times 10^{-13} \frac{\text{mm}^{5.5}}{N^3 \text{cycle}} \). The in-
tegration of Eq. 2.55 has to be done numerically. A similar ex-
pression can be written to calculate \( C \) for Eq. 2.13. It was stated
earlier that the \( \Delta K_{\text{Th}} \)-value has an influence on the curve fit. The
results for different \( \Delta K_{\text{Th}} \) values are shown in Figs. 2.35 through
2.37. Figs. 2.35 and 2.36 show the curve fits for Eq. 2.11 for
\( K_c = 110 \text{ MPa} \sqrt{\text{m}} \) and \( K_c = 165 \text{ MPa} \sqrt{\text{m}} \). Fig. 2.37 shows curve fits for Eq. 2.13. The R-ratio is constant and is 0.1.

The \( \Delta K \) limit for the integration has little influence on the crack growth constant. By varying \( \Delta K \) from 13.2 MPa\( \sqrt{\text{m}} \) to 5.5 MPa\( \sqrt{\text{m}} \) the crack growth constant in Eq. 2.11 changed from 3.7987 to 3.7945 for \( \Delta K_{Th} \) of 3.3 MPa\( \sqrt{\text{m}} \). This is a change in the crack growth rate of less than 1%. Figs. 2.35 and 2.36 show that Eq. 2.11 overestimates the crack growth rate for \( \Delta K_{Th} \) of 3.3 MPa\( \sqrt{\text{m}} \) or 7.7 MPa\( \sqrt{\text{m}} \) when \( \Delta K \) is less than 30 MPa\( \sqrt{\text{m}} \) when compared with the prediction by the Paris Power Law. When \( \Delta K_{Th} \) = 12.1 MPa\( \sqrt{\text{m}} \) the results in region II are in much better agreement with the simple power law. Eq. 2.13 is plotted in Fig. 2.37; Eq. 2.13 underestimates the crack growth rate for \( \Delta K_{Th} \) between 7.7 MPa\( \sqrt{\text{m}} \) and 12.1 MPa\( \sqrt{\text{m}} \) when compared with the Paris Law. For \( \Delta K_{Th} \) = 3.3 MPa\( \sqrt{\text{m}} \) Eq. 2.13 overestimates the crack growth rate for \( \Delta K \) between 6 MPa\( \sqrt{\text{m}} \) and 30 MPa\( \sqrt{\text{m}} \).

2.4.4 Minimum Crack Size

The crack propagation rate depends on the stress intensity range which is a function of the cyclic applied stress, the crack length and the geometry. A crack or internal flaw will not grow under cyclic loading if the \( \Delta K \)-value is smaller than the threshold stress intensity range \( \Delta K_{Th} \).

For known geometrical conditions, the ratio \( \frac{\Delta K}{\Delta \sigma} \) can be expressed as

\[
\frac{\Delta K}{\Delta \sigma} = (\pi a)^{1/2} \ CF(a) \tag{2.56}
\]
The correction factor \( CF(a) \) can be calculated using the numerical procedures given by Ref. 22 and summarized in Chapter 2.2. In Fig. 2.38 \((\pi a)^{1/2} CF(a)\) is plotted as a function of the crack length \( a \) for the small size cover- plated beams. The minimum crack length for a given stress range and threshold stress intensity range can be determined by equating \( \Delta K \) with \( \Delta K_{Th} \) in Eq. 2.56. The minimum crack length is the biggest crack size for which no crack growth under a given cyclic stress and threshold stress intensity range occurs. The transition point in Fig. 2.38 between Eq. 2.36 and Eq. 2.35 is for a crack length of 1.39 mm. For crack lengths bigger than 1.39 mm the increase in \((\pi a)^{1/2} CF(a)\) is very rapid.

Fig. 2.39 shows the relationship between the minimum crack length for different stress ranges as a function of \( \Delta K_{Th} \). With a minimum crack size known for a given stress range and threshold stress intensity range, the fatigue life can be calculated. The fatigue lives for \( \Delta K_{Th} = 3.3 \text{ MPa}\sqrt{\text{m}}, K_c = 110 \text{ MPa}\sqrt{\text{m}} \) and \( K_c = 165 \text{ MPa}\sqrt{\text{m}} \) are shown for different stress ranges in Figs. 2.40 and 2.41. The smallest crack size for crack propagation is 0.017 mm for \( \Delta \sigma = 55 \text{ MPa} \). The fatigue life provided by Eq. 2.11 for \( K_c = 165 \text{ MPa}\sqrt{\text{m}} \) is somewhat shorter than for \( K_c = 110 \text{ MPa}\sqrt{\text{m}} \). This is because the crack growth constant was determined as a function of \( K_c \). By solving Eq. 2.55 between the limits 13.2 MPa\sqrt{\text{m}} and 44 MPa\sqrt{\text{m}} the resulting crack growth constant is 3.799. For the same limits, \( K_c = 165 \text{ MPa}\sqrt{\text{m}} \) Eq. 2.55 yields 4.506 for the constant in Eq. 2.11. The predicted fatigue crack growth rate for \( K_c \) equal 165 MPa\sqrt{\text{m}} is therefore bigger at smaller \( \Delta K \)-values which decreases the resulting
fatigue life. The fatigue crack growth relationship according to Eq. 2.11 is shown in Fig. 2.42 and for Eq. 2.13 in Fig. 2.43. The fatigue life estimated for small size cover plates is about the same as can be seen in Figs. 2.40 and 2.41. However the fatigue lives for test specimens with a higher $K_C$-value should be longer and not shorter as predicted. If the same constant C is used for $K_C$ equal 110 MPa$\sqrt{m}$ and 165 MPa$\sqrt{m}$ the fatigue life will be longer for the higher $K_C$-value as can be seen from the crack growth rate prediction in Fig. 2.42 and 2.43. This problem only arises because of the curve-fitting-procedure and can be solved by making the curve fit for $K_C = 165$ MPa$\sqrt{m}$ and then keeping the value C constant for other $K_C$-values, see Fig. 2.44.

From Figs. 2.40 and 2.41 it is apparent that the fatigue life calculated with the crack growth relationship given in Eq. 2.11 is shorter than predicted by the tests. This is due to the poor curve fit in the region of low $\Delta K$, see Figs. 2.31 and 2.32. To improve the curve fit, the value of $\Delta K_{th}$ has to be increased. The increase in $\Delta K_{th}$ leads to an increase in the minimal crack size. For $\Delta \sigma = 55$ MPa and $\Delta K_{th} = 7.7$ MPa$\sqrt{m}$ the minimum crack size for crack propagation is 1.95 mm. For this crack size the fatigue life is infinite, for an initial crack size slightly larger than 1.95 mm (1.98 mm) the fatigue life is 1.1152x10$^6$ cycles. To determine the actual initial crack size to make the fatigue life the same as predicted by the test$^{11}$, the following procedure has to be used.

A large number of cycles is obtained at crack sizes between
1.95 mm and 1.98 mm. Over this small increase of crack length the crack growth relationship can therefore be solved analytical. The number of cycles is obtained by integrating Eq. 2.57.

\[
dN = \frac{da}{da/dN} \tag{2.57}
\]

where \( da/dN \) is given by the relationship by McEvily, Eq. (2.11)

Eq. 2.57 yields

\[
dN = \frac{da}{E^2 \left( \Delta K^2 - \Delta K_{Th}^2 \right) \left( 1 + \frac{\Delta K}{K_{c} - K_{Max}} \right)} \tag{2.58}
\]

Over a small interval \( \Delta a \), the second factor in the denominator remains constant \((= C_2)\). The stress intensity range \( \Delta K \) can be written as

\[
\Delta K = \Delta \sigma \sqrt{a} \tag{2.59}
\]

and the crack growth threshold as

\[
\Delta K_{Th} = \Delta \sigma \sqrt{a_{Th}} \tag{2.60}
\]

For \( \Delta K \approx \Delta K_{Th} + q \approx q_{Th} = \text{constant} \). Eq. 2.58 can be written as

\[
dN = \frac{E^2}{C \Delta \sigma^2 q^2 C_2} \frac{da}{a - a_{Th}} \tag{2.61}
\]

and integrated between \( a \) and \( \bar{a} \), the numbers of cycles is given as:

\[
N = \frac{E^2}{C \Delta \sigma^2 q^2 C_2} \log \left( \frac{\bar{a} - a_{Th}}{a - a_{Th}} \right) \tag{2.62}
\]

Eq. 2.62 gives the number of cycles between the limits \( a \) and \( \bar{a} \). For a crack length bigger than \( \bar{a} \) the numbers of cycles is calculated using the numerical procedure, because \( q \) is a function of \( a \) and does not remain constant. The total number of cycles is
\[ N_{\text{tot}} = N_{\text{a}} + N_{\text{a}}^{\text{Max}} = a \left( K_c \right) \] (2.63)

Eq. 2.63 can be solved to provide the same results as the experimental data. For a stress range of 55 MPa an initial crack size slightly larger than \( a_{\text{Th}} \) results; for \( \Delta K_{\text{Th}} = 7.7 \text{ MPa}\sqrt{\text{m}} \) the initial crack size \( a_{\text{i}} = 3.41 \text{ mm} \). With these initial crack sizes the fatigue life calculated with Eq. 2.11 is the same as predicted by the tests.

For \( \Delta \sigma = 138 \text{ MPa} \) the fatigue life for an initial crack size of 0.076 mm is about the same as for \( \Delta K_{\text{Th}} = 7.7 \text{ MPa}\sqrt{\text{m}} \) and \( \Delta K_{\text{Th}} = 12.1 \text{ MPa}\sqrt{\text{m}} \) as observed by tests (see Fig. 2.45). However, if the same initial crack size is used at a stress range of 55 MPa, the fatigue life drops well below the 95% survival limit. If the initial crack size is small enough to satisfy the fatigue strength at \( \Delta \sigma = 138 \text{ MPa} \), the fatigue life for \( \Delta \sigma = 55 \text{ MPa} \) is infinity because \( \Delta K(a_{\text{i}}) \) is less than \( \Delta K_{\text{Th}} \). It is not rational to have different initial flaw sizes for different stress ranges in order to be compatible with the experimental test data.

As it can be seen in Fig. 2.41, Eq. 2.13 also predicts a fatigue life that is too short for the small size cover-plated beams at stress ranges between \( \Delta \sigma = 55 \text{ MPa} \) and \( \Delta \sigma = 138 \text{ MPa} \) when \( \Delta K_{\text{Th}} = 3.3 \text{ MPa}\sqrt{\text{m}} \). This results because Eq. 2.13 overestimates the crack growth rate when \( \Delta K \) is bigger than 4.9 MPa\sqrt{\text{m}}. The stress intensity factor for \( \Delta \sigma = 138 \text{ MPa} \) and an initial crack size of
0.076 mm is 13.5 MPa√m and even for Δσ equal to 55 MPa the K-value is larger than 5.9 MPa√m for a flow size larger than 0.13 mm. Most of the fatigue life is consumed at small crack sizes and an overestimation of the crack growth rate at these crack sizes changes the fatigue life significantly.

For ΔK_{Th} = 7.7 MPa√m the fatigue lives increase and are too high, because of the low predicted crack growth rate when ΔK is about 10 MPa√m.

2.4.5 Fatigue Life Correlation with Beam Test Results

Instead of determining the constant C in Eqs. 2.11 and 2.13 curve fitting to da/dN vs ΔK measurements or the da/dN vs ΔK estimates from the Paris Power Law, the constant C can also be obtained from the fatigue tests described in Ref. 11. The constant C has to be determined so that the calculated fatigue life is the same as provided by the tests. For these calculations ΔK_{Th} was taken as 3.3 MPa√m, K_c as 110 MPa√m the R-ratio 0.5, and an initial crack size of 0.076 mm. The resulting constant in Eq. 2.11 is 1.45 cycle\(^{-1}\) and in Eq. 2.13 3.16 cycle\(^{-1}\) to predict the same life as the tests. Both relationships with these two constants are compared with the da/dN - ΔK measurements in Figs. 2.26 through 2.31. These models do not agree with the crack growth rate measurements. The constant C is determined so that the average life is the same as predicted by the tests. The fatigue lives at stress ranges higher than 60 MPa for an initial flow size of 0.076 mm are somewhat
short and at lower stress ranges the fatigue life is too long. As can be seen from Fig. 2.46 these differences are very small. The fatigue life prediction from these growth rates fall within the scatterband of the test data provided by Eq. 2.19. The influence of the initial flow is more pronounced at low stress levels. More fatigue life at these stress levels result from low ΔK values and at these low ΔK-values the da/dN predictions change more rapidly for a short interval of ΔK.

Fig. 2.47 compares predicted fatigue life provided by Eq. 2.13 for C = 3.16, ΔK_th = 3.3 MPa√m, R = 0.6 and different K_c values with the variation provided by the experimental data. The results suggest that the material fracture toughness does not have a major influence on the fatigue resistance once reasonable levels of fracture toughness are achieved.

2.4.6 Fatigue Life Calculations for Structural Details

The da/dN relationship provided by Eq. 2.13 permits the fatigue life of various structural details to be calculated with the procedure outlined in Chapter 2.2. The fatigue life for different stress ranges will not be proportional to −3.0 in the log N − log Δσ diagram. At high stress range levels (Δσ bigger than 50 MPa) this effect is very small, but at stress range levels less than 50 MPa the calculated fatigue lives are greater than provided by the experimental test data (see Fig. 2.47).

The fatigue life for small size cover-plated beams for K_c-values
of 55 MPa√m, 110 MPa√m and 165 MPa√m for a stress range of 55 MPa is shown in Fig. 2.47 and tabulated in Table 2.7. For an initial crack size of 0.076 mm the fatigue life for $K_c = 110$ MPa√m is taken as 100% the fatigue life for $K_c = 55$ MPa√m is 89% and for $K_c = 165$ MPa√m is 103.1%. The same small variation in fatigue life can also be obtained by keeping the number of cycles constant (for $K_c = 110$ MPa√m and $a_i = 0.076$ mm) and varying the initial flaw size. The initial flaw size for $K_c = 55$ MPa√m to have the same fatigue life as to $a_i = 0.076$ mm and $K_c = 110$ MPa√m has to be 0.030 mm and for $K_c = 165$ MPa√m the initial flaw size would be 0.117 mm. A small variation in initial flaw size is seen to have the same effect on the fatigue life as the evaluated changes in material toughness.

From Fig. 2.48 it is evident that the increase in useful fatigue life is very small at all toughness levels once the fatigue crack reaches a length of about 4 mm. In this figure the fatigue life for an initial flaw size of 0.025 mm is plotted. The fatigue life between the crack size 0.025 mm and 0.076 mm is also affected by the toughness, see Table 2.7.

The fatigue life is not proportional to fracture toughness. This is because the increase of $da/dN$ for $\Delta K$-values bigger than 20 MPa√m is very rapid.

Fig. 2.49 shows increments of $\Delta K$ and the number of cycles for various crack lengths as a function of the crack length. There is almost no increase of $\Delta K$ for a crack length smaller than 2 mm.
Most of the usefull fatigue life is provided at these small crack sizes.

The fatigue life of full size cover-plated beams at a stress range of $\Delta \sigma = 55$ MPa and on initial crack size of 0.076 mm is compared in Fig. 2.50 with design Category E' for the three different levels of fracture toughness. All three toughness levels provide fatigue lives longer than required by design Category E'. The fatigue life for these beams is summarized in Table 2.8 and in Fig. 2.51 and 2.52. It is visually apparent that any increase in life diminishes rapidly as the fracture toughness increases.

A similar comparison is provided for stiffeners welded to the web and flange in Table 2.9 and in Figs. 2.53 through 2.55.

2.4.7 Conclusion

McEvily's crack growth relationships are attractive, mainly because, given material toughness, the threshold stress intensity range and a selected value of the dimensionless constant C, one equation for $da/dN$ with the natural dimensions of length, can be used over the entire $da/dN$ range. However, the relationship does not yield consistent results nor is it compatible with fatigue tests of structural details.

$$\frac{da}{dN} = \frac{C}{E^2} (\Delta K_{Th}^2 - \Delta K_{Th}^2) \left(1 + \frac{\Delta K}{K_c - K_{Max}}\right)$$ (see Eq. 2.11)

cannot accurately represent the room temperature fatigue data for bridge steels. Crack growth data indicate that crack
propagation is proportional to \((\Delta K)^{3.0}\). Eq. 2.11 is roughly proportional to \((\Delta K)^{2.0}\). This influence is more dominant at low R-ratio.

For high R-ratios (R>0.6) the crack growth rate provided by

\[
\frac{da}{dN} = \frac{C}{E^2} (\Delta K - \Delta K_{Th})^2 \left(1 + \frac{\Delta K}{K_C - K_{Max}}\right)
\]

(see Eq. 2.13)

provides a better prediction of fatigue behavior. The fit to measured data of \(da/dN - \Delta K\) has to be very carefully done especially when the measurements are not distributed over the full range of \(\Delta K\). Test data at high levels tends to obscure the influence of \(da/dN\) measurements at low \(\Delta K\) levels. For low levels of fracture toughness the crack growth rate at \(\Delta K = 15\) MPa\(\sqrt{m}\) is 1.54 times larger for \(K_C = 55\) MPa\(\sqrt{m}\) than for \(K_C = 110\) MPa\(\sqrt{m}\) (see Fig. 2.46). At the same stress intensity level the difference between the crack growth rate for \(K_C = 110\) MPa\(\sqrt{m}\) and \(K_C = 165\) MPa\(\sqrt{m}\) is only 1.08. The crack growth rate at low stress range levels has a very pronounced influence on the fatigue life.

The implications of McEvily's crack growth relationship is not compatible with test data on a variety of structural steel details.
2.5 Proposed Fatigue Crack Propagation Relationship

Section 2.1.1 and 2.3 have shown that three different zones of fatigue crack propagation can be identified. The region near the threshold stress intensity range and the region where the maximum stress intensity factor approaches the material fracture toughness are dependent on the stress ratio $R$. The region between these limits is independent of the $R$-ratio. Therefore, crack propagation behavior is best represented in each zone by different functions. Test results demonstrate that the middle zone of the $da/dN$ vs $\Delta K$ relationship is best represented by the simple power law, Eq. 2.6.

2.5.1 Crack Growth Relationship Near $\Delta K_{Th}$

Near the threshold stress intensity range the function representing the crack growth rates has to fulfill the following boundary conditions

- For $\Delta K$ approaching $\Delta K_{Th}$ the crack growth rate has to approach zero.

- At a given point $\Delta K_1$ the crack growth rate has to be the same as predicted by the Paris Power Law.

- The slope at $\Delta K_1$ has to be the same as for the Paris Power Law.

The first condition can be satisfied by choosing a function of the form

$$\frac{da}{dN} = C_1 (\Delta K_1^{n_1} - \Delta K_{Th}^{n_1})$$

(2.64)
The crack growth constant \( C_1 \) and the crack growth exponent \( n_1 \) have to be selected so that they satisfy the 2nd and 3rd conditions. The second condition can be written as
\[
C \Delta K_1^n = C_1 (\Delta K_1^{n_1} - \Delta K_{Th}^{n_1}) \tag{2.65}
\]
and the third condition yields
\[
C n \Delta K_1^{n-1} = C_1 n_1 \Delta K_1^{n_1-1} \tag{2.66}
\]
Eqs. 2.65 and 2.66 have to be solved simultaneously for \( n_1 \) and \( C_1 \).
The Paris Power Law coefficient \( C \) is \( 1.211 \times 10^{-13} \frac{mm}{N^3 \text{cycle}} \) and \( n \) is taken as 3.0. For known values of \( \Delta K_{Th} \) and \( \Delta K_1 \), \( n_1 \) can be determined from the transcendent equation, Eq. 2.67
\[
\frac{C n \Delta K_1^{n-1}}{n_1 \Delta K_1^{n_1-1}} (\Delta K_1^{n_1} - \Delta K_{Th}^{n_1}) - C \Delta K^n = 0 \tag{2.67}
\]
Eq. 2.67 was solved numerically using the secant method. The proposed crack growth rate relationship is shown in Fig. 2.56.

Eq. 2.64 is shown in Fig. 2.57 and compared with \( \frac{da}{dN} \) - \( \Delta K \) measurements from Ref. 25. Eq. 2.64 is plotted for \( \Delta K_1 \) values between 13.4 MPa\(\sqrt{m}\) and 87.9 MPa\(\sqrt{m}\). For increasing values of \( \Delta K_1 \), \( n_1 \) approaches 3.0 and \( C_1 \) approaches the crack growth constant \( C \).

Fig. 2.57 also shows that \( \Delta K_1 \) has little influence on the crack growth rate when it exceeds 22 MPa\(\sqrt{m}\). As a result \( \Delta K_1 \) was taken as a constant equal 22 MPa\(\sqrt{m}\) independent of the R-ratio.

An upper bound to the crack growth propagation rate is given by the simple power law with a crack growth constant.
\[ C = 2.179 \times 10^{-13} \frac{\text{mm}^{5.5}}{N^{3/2}} \]. This upper bound solution is shown in Fig. 2.58 and compared with the test data plotted in Fig. 2.57. \( \Delta K_{Th} \) in Fig. 2.58 was selected as the lower bound value given in Eq. 2.14.

Figs. 2.59 through 2.62 compare Eq. 2.64 for R-ratios of 0.1, 0.3, 0.5 and 0.7 with crack growth data from Ref. 25. The curve fit for low R-ratio is good. For R-ratios of 0.5 and 0.7 the predicted growth rate is not satisfactory. The predicted crack growth rate is less than desired. The results suggest that a better fit would result by utilizing the simple power law for \( \Delta K \) values larger than \( \Delta K_{Th} \).

2.5.2 Crack Growth Relationship Near \( K_C \)

The boundary conditions for the crack growth rate in Zone III are almost the same as Zone I.

- For \( K_{Max} \) approaching \( K_C \) the crack growth rate has to approach infinity.
- At a given point \( \Delta K_2 \) the predicted crack growth rate has to be compatible with the simple power law.
- The slope from any relationship at \( \Delta K_2 \) has to be the same as for the simple power law.

Two different relationships to predict the crack growth rate at high \( \Delta K \) levels were investigated.
\[
\frac{da}{dN} = C_2 \left( \frac{1}{K_c (1-R) - \Delta K} \right)^{n_2}
\] (2.68)

and

\[
\frac{da}{dN} = C_3 \left( \frac{\Delta K}{K_c (1-R) - \Delta K} \right)^{n_3}
\] (2.69)

Both relationships fulfill the first boundary condition. Eq. 2.68 is represented in graphical form in Fig. 2.63, Fig. 2.64 shows Eq. 2.69 for different $\Delta K_2$ values. The values of $C_2$ and $n_2$ were determined in the same way as described in Section 2.5.1. Figs. 2.63 and 2.64 indicate the value of $\Delta K_2$ has a significant influence on the curve fit. $\Delta K_2$ is a function of the yield stress and the R-ratio. $\Delta K_2$ is the transition point between Zone II and Zone III and is defined as $\Delta K_T$ in Eq. 1.52. Fig. 1.65 shows Eqs. 1.68 and 1.69 for the upper and lower bound of crack growth rate estimates. The $da/dN - \Delta K$ measurements for A36 steel from Ref. 8 are compared with the predicted results. This shows that Eq. 1.68 fits the measured data better. This relationship was used in subsequent studies. Fig. 1.66 summarizes the crack growth rate relationship derived for the various regions of interest.

2.5.3 Fatigue Life Calculations

The crack growth rate relationship shown in Fig. 2.66 can be used to predict the fatigue life of various structural details. Three different structural details were examined: Small size cover-plated beams, full size cover-plated beams and beams with stiffeners (stiffener Type 3). The general procedure and the
geometrical conditions for the fatigue life calculations are given in Section 2.2. Due to the high residual tensile stresses from welding, the R-ratio was again taken as 0.6. Based on the R-ratio the threshold stress intensity range is 3.3 MPa√m (Eq. 2.14). The material was assumed to have a yield stress of 215 MPa. This resulted in a $K_T$ value of 44 MPa√m for a R-ratio of 0.0. The $\Delta K_T$ value decreases to 17.6 MPa√m for $R = 0.6$ (Eq. 2.52).

Three different material toughness values were examined, a low toughness value of $K_c = 55$ MPa√m, a medium toughness value of 110 MPa√m and a high toughness value of 165 MPa√m. The $da/dN - \Delta K$ relationship is shown in Fig. 2.66. For the fatigue life calculation stress ranges that are probable in bridge structures were chosen: 55 MPa and 69 MPa for small cover-plated beams, 55 MPa for full size cover-plated beams and 110 MPa for the beams with stiffeners. The stress range must be large enough to exceed the threshold stress intensity range for the assumed initial flaw sizes.

2.5.3.1 Small Size Cover-Plated Beams

The fatigue strength of small size cover-plated beams is shown in Fig. 2.67 where the mean and the confidence limits for the test data are also given. The predicted behavior for 3 different initial crack sizes of $a_i = 0.025$ mm, 0.076 mm and 0.76 mm at stress ranges of $\Delta\sigma = 55$ MPa and 69 MPa are compared with test data. The results are compared for all three levels of fracture toughness, $K_c$. The
fracture toughness has little influence on the fatigue behavior as it can be seen in Fig. 2.68. Also shown in Fig. 2.68 is the fatigue life for a crack completely penetrating the flange plate. (This fatigue life is indicated as \( K_c = \infty \) in the figures and tables). The fatigue lives for the different fracture toughness is summarized in Table 2.10.

Different crack flanges were observed during the testing of the beams reported in Ref. 11 and Ref. 12. Most of the fatigue life is generated during growth in stage 1. Stage 1 is the propagation as a semielliptical crack from the initial condition until the crack front reaches the opposite flange surface. The final small ligament was usually broken because the applied stress exceeded the tensile strength of the material. A through crack results. Brittle fracture never occurred at these tests at room temperature. About 95% or more of the fatigue life was consumed during stage 1. The remaining fatigue life was generated during the second stage. The second stage is the crack growth as a through crack. Eventually one crack front reaches the edge of the flange and a large edge crack results. During this crack stage the crack grew very rapidly and the fracture mode changed from a flat to a slant fracture appearance. The plane of the crack turns and does not remain perpendicular to the flange plate. The total fatigue life, including the life during stage 2 is also indicated in Fig. 2.68.

The shortest fatigue life results from the lowest material
toughness value and is $2.52 \times 10^6$ cycles (100%) for the initial crack size of 0.025 mm. For $K_c = 110$ MPa√m the life is slightly longer, $2.54 \times 10^6$ cycles or 100.4%. At the high toughness level the life is 100.5%. If the fracture toughness is infinite and the Paris Power Law alone is applied the fatigue life is 100.9%. For an infinite toughness the fatigue life is the life to penetrate the full flange thickness.

Fig. 2.69a shows the crack growth relationship with linear scales for $K_c = 110$ MPa√m and $R = 0.6$. Fig. 2.69b shows the number of cycles versus $\Delta K$ for the small size cover-plated beams. At $\Delta K$ levels below 10 MPa√m a large number of cycles results. This is due to the fact that $\Delta K$ is not a linear function of the crack length. The $\Delta K$ level as a function of the crack length is shown in Fig. 2.70a. Fig. 2.70 also demonstrates why the difference in fatigue life for different levels of fracture toughness is so small. $\Delta K$ levels greater than 40 MPa√m result in crack growth rates that are large and increase so fast that negligible increments of fatigue life results. Fig. 2.70 also demonstrates that most of the fatigue life is developed at small crack sizes. The total fatigue life of a cover-plated beam is the area under the curve in the diagram in Fig. 2.70b. Small changes in the initial crack size have a much larger influence on the fatigue life than the final crack size which is dependent on the material fracture resistance.
2.5.3.2 Full Size Cover-Plated Beams

Fig. 2.71 compares the predicted fatigue resistance of full size cover-plated beams with the fatigue test results of small size cover-plated beams and with the design category E'. The design category E' was derived from test results on full scale cover-plated beams as a lower bound to the test results. The predicted fatigue life is bounded by the tests on the small size cover-plated beams and design category E'. The differences in fatigue life are small for different $K_C$ values as can be seen from table 2.11 and Fig. 2.72. The fatigue life in Fig. 2.71 is only shown for $K_C = \infty$ and for different initial flaw sizes. For the initial flaw size of 0.076 mm the fatigue life for $K_C = 55 \text{ MPa}\sqrt{m}$ is 150,600 cycles smaller than for $K_C = \infty$, 109,100 cycles smaller for $K_C = 110 \text{ MPa}\sqrt{m}$ than for $K_C = \infty$ and 102,900 cycles smaller than $K_C = 165 \text{ MPa}\sqrt{m}$. The difference in fatigue life is therefore very small compared with the difference of fatigue life between the initial flaw sizes of 0.076 mm and 0.76 mm. The fatigue life for a crack to grow from $a_i = 0.076 \text{ mm}$ to 0.76 mm is 213,300 cycles. For an initial flaw size of 0.076 mm and a low fracture toughness level of 55 MPa$\sqrt{m}$ the fatigue life exceeds the requirements of the design category E'.

If the fatigue life for $K_C = 55 \text{ MPa}\sqrt{m}$ is taken as 100%, then the fatigue life for $K_C = 110 \text{ MPa}\sqrt{m}$ is 102.5% and 102.9% for $K_C = 165 \text{ MPa}\sqrt{m}$. For the assumed fracture toughness of infinity the fatigue life for a crack to grow through the flange is 109.3%. In Fig. 2.73 the fatigue life distribution as a function of the crack length is shown.
2.5.3.3 Beams With Stiffeners

The predicted fatigue life of stiffeners is given in Table 2.12 and in Figs. 2.74 through 2.76. The predicted fatigue behavior using Zettlemoyer's model is compared with the test results in Fig. 2.74. Only the fatigue behavior for different initial crack sizes is plotted ignoring $K_c$. The difference between the predicted fatigue resistance at all levels of fracture resistance is very small as it can be seen in Table 2.12. If the fatigue life for $K_c = \infty$ is taken as 100% then the life for $K_c = 55 \text{ MPa} \sqrt{\text{m}}$ is 97.7%, the fatigue life for $K_c = 110 \text{ MPa} \sqrt{\text{m}}$ is 99% and 99.2% for $K_c = 165 \text{ MPa} \sqrt{\text{m}}$ for an initial crack size $a_1 = 0.025 \text{ mm}$. The difference in fatigue resistance for different $K_c$ values is apparent in Fig. 2.75. Up to a crack length of $a = 6 \text{ mm}$ the predicted fatigue behavior is the same for different $K_c$ values. The material toughness does not affect the crack growth rate until $\Delta K$ exceeds $\Delta K_T$. As can be seen in Fig. 2.75, when $a > 6 \text{ mm}$ no significant increase in fatigue life results as $K_c$ increases. Fig. 2.76 shows the stress intensity range change for $\Delta \sigma = 110 \text{ MPa}$ as a function of the crack length. The increase in $\Delta K$ is constant up to $\Delta K \approx 30 \text{ MPa} \sqrt{\text{m}}$, when $\Delta K$ exceeds 40 $\text{ MPa} \sqrt{\text{m}}$, the increase is very rapid. This is the reason why there is a small difference in final crack length for $K_c = 110 \text{ MPa} \sqrt{\text{m}}$, $K_c = 165 \text{ MPa} \sqrt{\text{m}}$ and $K_c = \infty$. The increase is fatigue life for crack lengths bigger than $a(\Delta K_T)$ is very small because of the fast rising $\Delta K$ value and the high crack growth rates.
2.6 Conclusions

2.6.1 Summary

The fatigue strength of structural details was evaluated using two mathematical models to predict the crack growth rate. Results were obtained by integrating the $\frac{da}{dN} - \Delta K$ relationship proposed by McEvily (Eq. 2.13) and by integrating the relationship developed in Section 2.5. Both relationships consider the crack growth rate near $\Delta K_{th}$ and the accelerated crack growth rate as $K_{max}$ approaches the fracture resistance of the material, $K_c$.

An R-ratio of 0.6 was assumed for the fatigue life calculation. This high R-ratio is a simplistic adjustment for local residual tensile stresses. The high R-ratio decreases the threshold stress intensity range. At an R-ratio of 0.6, the threshold stress intensity range was taken to be 3.3 MPa√m.

Fatigue lives for 3 different structural details were calculated and compared with test results from the laboratory studies. The fatigue life was terminated when the maximum stress intensity factor along the crack front reached the material fracture toughness. For comparison purposes three different levels of fracture toughness were examined.

2.6.2 Results

McEvily's relationships do not fit the basic crack growth data nor the fatigue test data from the laboratory tests made on beams with
structural details. When McEvily's relationships are fitted to the measured experimental data on test beams they do not fit the $da/dN - \Delta K$ data. McEvily's relationship indicates that there is an influence of the fracture toughness on the crack propagation rate at both low and medium stress intensity ranges. The exact prediction of crack growth rates is of major importance at low $\Delta K$ - values because most of the fatigue life is developed at these low levels. Calculations of fatigue lives presented in Ch. 2.4 to determine the influence of the fracture toughness are therefore only of qualitative nature. The McEvily relationship overestimates the influence of the fracture toughness and predicts fatigue lives that are too long for high fracture toughness levels compared to fatigue lives for low levels of fracture toughness. This is because of the influence of the fracture toughness on the crack propagation rate at low stress intensity ranges.

Because the McEvily relationship does not describe the fatigue behavior of compact tension specimens and of beams accurately enough, the relationship presented in Section 2.5 was developed. The proposed relationship is based on the simple power law and adjusted for the influence of the threshold stress intensity range in Zone I and the accelerated crack growth in Zone III. Fatigue life calculations based on the integration of the proposed relationship agree well with the test results from the laboratory studies.

The relative fatigue lives calculated with the proposed
relationship for different structural details for the three levels of fracture toughness are shown in Table 2.13. The fatigue life for a $K_c$ - value of 55 MPa$\sqrt{m}$ is taken as 100% for this comparison. Then the fatigue life for $K_c = \infty$ is 101% for the small size cover- plated beams and 102.6% for Type 3 stiffeners. The biggest increase results for full size cover-plated beams, the fatigue life increases to 109.7% for $K_c = \infty$. The calculated fatigue lives for an initial crack size of 0.076 mm falls within the scatter band of plus or minus one standard deviation for all three levels of fracture toughness. The reason for the more pronounced increase in fatigue life for the full size cover-plated beams is the larger flange thickness. In the thick plate, the increase of the stress intensity factor with increasing crack length is not so rapid as it is for the thinner plates. The rapid increase of the stress intensity factor is not due to the increase due to a larger crack. This increase is only proportional to the square root of the crack length. The large increase is caused by the back free surface correction factor which increases according to Eq. 2.29 very rapidly for the crack approaching the back surface.

The increase in fatigue life for an increasing fracture toughness is not very pronounced. Small changes in the initial crack size effect the fatigue life much more
3. CASE STUDIES

3.1 Problem Statement and Solution Approach

3.1.1 Introduction

During the last two decades, several bridges have cracked in the U.S.A. and elsewhere in the world. Most apparent and known disasters are the ones involving complete loss of the structure such as the collapse of the Point Pleasant Bridge in West Virginia in 1967 or the Kings Bridge in Melbourne in 1962. Cracks have developed in several other bridge structures; the bridges did not collapse completely because of the more favorable statical system. The destroyed member could then be restored and the bridge remained in service.

In the following chapters several bridges which cracked are analysed to investigate the influence of fracture toughness on the fracture behavior and service life of the details. Most of these structures were designed in accordance with the AASHTO Specifications. Most of the structures had material that satisfied the AASHTO material toughness requirements. These requirements were adopted in 1974, but many of the investigated bridges were designed and constructed prior to that time.

3.1.2 AASHTO Material Toughness Requirements for Steel Bridges

In 1974 the American Association of State Highway and
Transportation Officials (AASHTO) incorporated a Charpy V-Notch impact requirement in their specification. The requirements are shown in Table 3.1. The toughness requirements are only a part of the fracture control plan for steel bridges and regulate the material aspects. Other factors affecting the fracture behavior are design, fabrication, erection, inspection and use.

With the above toughness requirements, segments of unstable crack propagation cannot be prevented in bridges as fatigue resistance is exhausted. The requirements only eliminate the possibility of brittle fracture initiation in carefully designed, fabricated and inspected structures. When basic design rules are violated and extensive fatigue crack growth occurs, the structure will eventually experience brittle fracture. The toughness requirements do not prevent unstable crack growth in structures, which have serious faults.

The basic idea of the current toughness requirements is to make sure that the material has a moderate toughness and can tolerate cracks which are large enough to be discovered. Because $K_{IC}$ tests (compact tension tests or three point bend tests) are very expensive and difficult to carry out, the Charpy V-Notch test is used as a reference test for the AASHTO requirements. An empirical relationship exists between the CVN energy absorption and fracture toughness. Loading times in bridge structures are about 1 sec which corresponds to a strain rate of less than $10^{-3}$ sec$^{-1}$ on the elastic plastic boundary near the crack tip. A temperature shift to relate the
intermediate loading rate of bridge structures to the impact loading
during Charpy V-Notch tests is needed. An other temperature shift
is needed to relate the minimum service temperature with the temper-
ature for the minimum $K_{1c}$. These temperatures are indicated in
Fig. 3.1.

3.1.3 Principle of the Linear Elastic Fracture Mechanics

The analysis of fatigue crack growth and brittle fracture
behavior is made using linear elastic fracture mechanics concept
(LEM). Linear elastic fracture mechanics assumes a linear load-
displacement behavior of the structure until fracture occurs. The
stress field in the vicinity of the crack tip can be characterised
by a single parameter, the stress intensity factor $K$. The stress
intensity factor considers the crack geometry, the loading condi-
tions and the geometrical conditions of the solid containing the
crack. The stress intensity factor can be written in the following
form, Eq. 3.1

$$K = K \text{ (Crack length, Load, Geometry)} \quad (3.1)$$

Because the body containing the crack is assumed to be elastic, the
principle of super position is valid. The total stress intensity
factor is the sum of each component

$$K = \Sigma K_i = K_{\text{LL}} + K_{\text{DL}} = K_{\text{RS}} + \ldots \quad (3.2)$$

where $K_{\text{LL}}$ is the stress intensity factor due to live load, $K_{\text{DL}}$ is
due to the dead load of the structure and $K_{\text{RS}}$ is due to residual
The stress intensity factor is often expressed in the form\(^4\)

\[ K = \sigma(a)^{1/2} \cdot CF(a) \tag{3.3} \]

where \(\sigma(a)^{1/2}\) is the stress intensity factor of a through crack in an infinite plate under uniform tension. The geometrical conditions are taken into account by \(CF(a)\). \(CF(a)\) is the product of several different correction factors including the free surface correction factor, the finite width correction factor (≡ back free surface correction factor) and the stress gradient correction factor.

Several difficulties are encountered when analysing the actual cracks in bridge structures: The fracture mechanics solution are generally for well defined crack shapes in well defined solids for well defined loading conditions. Most solutions are for two dimensional behavior. Cracks in bridge structures are often of highly irregular shape and not well defined because of the corroded fracture surface. Very often a crack existed for several years in a bridge member before it was discovered, removed and prepared for investigation. Minute details on the fracture surface are often destroyed because the two fracture surfaces rub together and are subjected to environmental conditions.

The initial flaw size can seldom be determined exactly. The initial crack may vary from a micro crack less than 1 mm in size up to a crack of several cm. This depends greatly on the quality
control used at time of fabrication and whether or not fatigue crack propagation has enlarged the crack.

The behavior in the vicinity of a crack tip is not purely elastic as assumed by the methods of LEM. Some plastic deformation occurs in the low strength materials (such as bridge steels with a yield strength up to 500 MPa to 600 MPa) because the section sizes are not thick enough to maintain plane strain conditions under service loading rates (∼1 sec load rise time) and normal service temperatures (T > −40º C). The plastic deformation near the crack tip can be taken into account by considering the plastic zone correction. At the crack tip where stresses exceed the elastic limit, plastic deformation occurs which creates plastic zones surrounding the crack tip. The half-size of the plastic zone, $r_Y$, can be estimated as

$$r_Y = \frac{1}{2\pi} \left( \frac{K}{\sigma_Y} \right)^2 \quad (3.4)$$

for plane stress conditions. For a crack front approaching the back free surface of a plate, the unbroken ligament becomes thinner and thinner. The small ligament is not constrained and plane strain conditions exist at the crack tip. The plastic zone size for plane strain conditions is 1/3 of the plastic zone size for plane stress conditions. The transition between plane stress and plane strain starts when the plastic zone is half of the remaining unbroken ligament. The $K$-value can be calculated by taking the plastic zone into account. Instead of using the physical crack size as a pseudo crack length $a'$ of
\[ a' = a + r_Y \] (3.5)

has to be considered. Eqs. 3.3 and 3.5 may be solved simultaneously by an iterative procedure.

The fatigue crack propagation is only slightly affected by the plastic zone. If the plastic zone due to the cyclic applied stress is about one half of the remaining ligament then accelerated fatigue crack growth occurs. The fatigue life can be calculated using Eq. 3.6

\[ \frac{da}{dN} = \frac{1}{2} \Delta \delta \] (3.6)

where \( \Delta \delta \) the crack opening range is:

\[ \Delta \delta = \frac{\Delta K^2}{E \sigma_Y} \] (3.7)

Because the \( \Delta K \)-value from the live load stress is low the plastic zone is also small and therefore the zone where accelerated crack growth occurs is small. This effect can be neglected in civil engineering structures.

Often it is not possible to solve Eqs. 3.3 and 3.5 simultaneously because the remaining ligament is smaller than the plastic zone. Calculations based on the elastic behavior are then made.

The fatigue life of a detail can be calculated using the principle outlined in Chapter 2.2.3. The simple power law can be used to estimate the number of cycles. The fatigue life is a function of the third power of the stress intensity range.

\[ N \propto (\Delta K)^3 \] (3.8)
Obviously the $\Delta K$ value has a pronounced effect on the fatigue life. The crack growth rate prediction as a function of the $\Delta K$ as shown in Fig. 3.2 is used. The influence of the threshold stress intensity range and the fracture toughness are included.

3.1.4 Material Characterization

The desired material properties are often not available for a detailed analysis of the cracking behavior of a bridge component. Very often insufficient material can be removed from the bridge to adequately assess the desired material properties of the base metal and weldment.

The fracture resistance is best defined by $K_c$ values obtained from compact tension tests or three point bend tests. Often the removed material is not thick enough for plane strain behavior at the desired test temperature of $T > -40^\circ\text{C}$. Since yield strengths are generally less than 500 MPa inelastic behavior results and it is necessary to estimate the elastic plane strain fracture toughness using the $J$-integral approximation. Very often the fracture toughness must be estimated from Charpy V-Notch test results. The CVN-test results can be converted to $K_{I_d}$ values using the empirical relationship developed by Rolfe and Barsom\textsuperscript{52}.

\[ K_{I_d} = (6.46 \times 10^{-4} \text{E CVN})^{1/2}; \text{[MPa}\sqrt{\text{m}}, \text{MPa, J]} \] (3.9)

The minimum $K_{I_d}$ value is obtained from the Irwin empirical relationship as modified by Shoemaker and Rolfe\textsuperscript{88}.

73
\[ K_{Id} = 0.113 \sigma_{Yd} \text{ [MPa} \sqrt{\text{m}}, \text{ MPa}] \] (3.10)

where \( \sigma_{Yd} \) is the dynamic yield strength at the NDT temperature. The difference between the temperature at fracture and the NDT temperature is usually small and can be neglected.

The resulting \( K_{Id} \) values have to be shifted in order to obtain the fracture toughness at an appropriate loading rate such as \( K_{Ic} \) or \( K_{Ilsec} \). The empirical relationship for the full temperature shift is

\[ T_s = 120 - 0.12 \sigma_{YS} \] (3.11)

\( T_s \) is the temperature shift in °C and \( \sigma_{YS} \) is the static yield strength [MPa] at room temperature. The value corresponding to 1 sec loading time is often approximated using \( 3/4 T_s \). Since most fracture toughness values are on the rising portion of the toughness-temperature relationship considerable errors may result.

The yield strength is often measured for different specimen sizes under different (some times unknown) test conditions. To compensate for the temperature and load rise time, the yield strength was corrected by using Irwin's empirical relationship.

\[ \sigma_Y = \sigma_{YS} + \frac{666,420}{(T+273)(\log 2 \times 10^{-10}t)} - 189 \text{ (MPa)} \] (3.12)

where \( \sigma_Y \) is the yield strength at temperature \( T \) [°C] for a load rise time \( t \) [sec]. \( \sigma_{YS} \) is the yield strength for the load rise time of 60 sec (static) at room temperature in MPa. Because of the three dimensional stress conditions in the bridge component it is
possible to have a yield strength higher than measured by simple tensile specimens.

The crack growth behavior at the fatigue crack growth threshold and for propagating cracks is defined for homogenous materials. Fatigue crack growth often initiates in a faulty weld with unknown material properties. The threshold limit for fatigue crack propagation may be very low or not existent in the inhomogenous weld materials encountered near lack of fusion regions.

3.1.5 Stresses at the Crack Location

Often the stress conditions at the crack location are not well known. The stress due to the truck traffic using the bridge are generally not know. The magnitude of the actual stress spectrum often has to be estimated from measurements made on similar structures and the design stresses. Generally the number of cycles is not available and must be estimated from the ADTT (average daily truck traffic). The stress due to the weight of the structure (DL) can be determined but its magnitude is small compared with the residual stress.

Residual stresses are often of unknown magnitude at the location of the crack. Often the residual stress field is complicated because of weld geometry and weld sequence. Different welds sometimes intersect at the location of the crack and result in a complex internal stress field. The residual stress field is often
estimated based on measurements made on similar details.

3.1.6 Calculation Concept

Several uncertainties are involved when estimating the stress intensity range. A ± 15% variation in ΔK will introduce ± 50% variation in fatigue resistance, because the fatigue life depends on the 3rd power of the stress intensity (see Eq. 3.4). The general source of variation in fatigue life comes from the variability in stress gradient, crack shape and uncertainty in initial flaw.

The crack size at the end of slow stable crack propagation and the onset of rapid fracture depends on the total stress intensity factor as calculated with Eq. 3.2 and not only on the stress intensity range. The major contribution on the stress intensity factor usually is the residual stress. The exact magnitude of the residual stress is often not known.

The stress intensity factor is locally elevated by an irregular crack front. The irregular crack front of the initial or final crack is sometimes caused by a lack of fusion area or by crack coalescence. The fracture resistance along the crack front is not always constant because of irregularities of the weld metal, inclusions and pores.
3.2 Quinnipiac River Bridge

3.2.1 Introduction

In November 1973 a large crack was discovered in the suspended span of the fascia girder of the Quinnipiac River Bridge. The bridge is located near New Haven, Connecticut on Interstate 95. The Quinnipiac River Bridge is a noncomposite structure with welded steel girders and a concrete slab. The span is 50.3 m long and 2.8 m high. The structure is shown in Fig. 3.3 and the cross section of the girder with the crack in Fig. 3.4.

Investigations\textsuperscript{1,51} showed that the crack grew in different stages. Fatigue crack growth started at a lack of fusion area in a longitudinal stiffener groove weld. Fatigue crack growth occurred until the crack reached a critical length and fractured the web. When the crack was finally discovered, it was approximately 1.4 m long and had also partially penetrated the bottom flange. One of the questions asked was if brittle fracture could have been prevented by using a steel with a higher fracture toughness.

Material was removed from the girder near the crack and standard ASTM Type A Charpy V-Notch tests were carried out in order to verify if the material fullfills the requirements of the AASHTO Specifications for toughness. The Charpy tests for the web material gave an energy absorption of 27 joule at 4\textdegree C and 47 joule at 4\textdegree C for the flange material. Both materials satisfied the requirements for zone 2 of the 1974 interim AASHTO Specification (see Table 3.1).
3.2.2 Material Characterization and Stresses at the Critical Location

At the critical location the web and the flange are fabricated from A36 steel. Tensile coupons provided a yield strength of 254 MPa and an ultimate strength of 420 MPa.

The results from the Charpy V-Notch tests were used to estimate the fracture toughness at the time of brittle fracture. It was estimated, that for the expected loading conditions the fracture toughness was about 130 MPa√m to 150 MPa√m. Compact tension tests at a loading time of 0.1 sec confirmed this value.

Brittle fracture is arrested when the fracture toughness of the material is higher than the stress intensity factor driving the crack. It can be assumed that the fracture toughness necessary to arrest the dynamic growth is equal to the dynamic fracture toughness. The dynamic fracture toughness estimated with Eq. 3.9 is 52 MPa√m.

At the critical location the following stresses are present: Stress from the live load (σ_{LL}), stress from the dead load (σ_{DL}) and residual stress from welding (σ_{RS}).

The effective live load stress range (calculated with Miner's Rule) at the critical location is 8.1 MPa. The maximum live load stress was taken as 13.8 MPa. An average daily truck traffic (ADTT) of about 4300 crossed the bridge resulting in about 1'600'000 random stress cycles per year. In the bottom flange the maximum live load stress is 22.8 MPa.
The dead load stress, $\sigma_{DL}$, the stress due to the weight of the structure was 34.1 MPa at the level of the longitudinal stiffener to the web. During welding the weld material and some of the connected plates are in a liquid state. During cooling these materials are restrained from shrinking and residual tensile stresses are developed. These residual tensile stresses are in equilibrium with residual compression stresses elsewhere on the section. Measurements on cycled I-beams were used to estimate the distribution of residual stress. The residual stress measurements were made on A36 steel beams. The exact residual stress distribution is not known. Three different distributions were investigated as shown schematically in Fig. 3.5. For each distribution it was assumed that the residual stresses in the stiffener alone were in equilibrium.

The residual stresses in the stiffener are not very large. The stiffener is not continuous because of the weld containing the large lack of fusion area. The estimated stress distribution in the horizontal stiffener is indicated in Fig. 3.5. The compression stress is assumed to be $\sigma_y/4$ and the tensile stress $\sigma_y/2$. This distribution is assumed for all following cases. The length of the residual tensile stress block in the web was varied between 76 mm and 178 mm as shown in Fig. 3.5. The magnitude of $\sigma_{RS}^-$ was assumed to be the yield stress (254 MPa). The compression residual stress was assumed equal to half the yield stress (127 MPa).

Other tensile or compression residual stresses may have been
present at the critical location at the time of brittle fracture. These could include stresses from temperature gradients and stresses from partially blocked supports or from support settlements. Because of their unknown magnitude these stresses were not considered here.

3.2.3 Crack Growth Stages

Visual examination and examinations with a transmission electron microscope of replicas made of the fracture surface showed that different stages of crack growth could be distinguished. The different stages of crack growth are shown schematically in Figs. 3.6 and 3.7. The fracture originated in a lack of fusion area between two connected lengths of a horizontal stiffener. Only a surface weld pass existed which did not penetrate the full depth of the stiffener; the unfused part of the weld between the two segments of the horizontal stiffeners acted as an initial crack. During handling, shipping and early service life this part broke completely and a large circular crack with a radius of about 11.5 cm resulted. (Stage I in Fig. 3.6) Later on the crack front penetrated the web in stable fatigue crack growth driven by the cyclic applied loads from the truck traffic (Stage II). Brittle fracture occurred after the crack had fully penetrated the web. This unstable crack growth was arrested when the crack tip moved into the bottom flange. The upper front was also arrested in the web in a zone of low stresses. At crack arrest the stress intensity
factor driving the crack dropped below the dynamic fracture toughness.

Calculations based on assumptions of the truck traffic were in good agreement with estimates of the rate of crack growth using measured striation spacing 1,51.

Stage II is of interest for fatigue crack growth. This stage begins when the crack front reaches the web surface and ends when brittle fracture of the web occurs. Stage IIa, Fig. 3.7 can be analytically modeled as a circular crack in a solid. As soon as the crack front reaches the opposite free surface of the web, two independent crack fronts are formed. (Stage IIb). These crack fronts have the tendency to become perpendicular to the surfaces of the web. Eventually the crack fronts are perpendicular and the crack can be analytically modeled as a through crack in a plate.

3.2.4 Analysis of Crack Growth

During Stage II fatigue crack growth, the crack changes its shape; different analytical models have to be considered for these different shapes. Since the longitudinal stiffeners were welded on the web, a complex stress pattern exists at the location of the crack. Therefore several estimates have to be used to calculate the value of the maximum stress intensity factor.

Stage II a

During Stage IIa the crack front lies on a circle with the
center at the edge of the stiffener, see Fig. 3.7a. During the experimental work on NCHRP Project 12-15(3) some details similar to the ones that existed at the Quinnipiac River Bridge were tested and the crack surfaces were exposed. A typical crack surface is shown in Fig. 3.8. For this test the horizontal stiffener was completely unfused and crack growth only occurred in the longitudinal welds and web. The picture shows the semicircular crack front. The stress intensity factor for a flat circular crack in an infinite solid under uniform applied stress like the live and dead load stress is

\[ K = \frac{2}{\pi} \sigma (\pi a)^{1/2} \]  

(3.13)

where

\[ \sigma = \sigma_{LL} + \sigma_{DL} \]

For the residual stress the stress intensity factor was to be estimated by superposition: the full surface of the circular crack has to be loaded with the residual stress \( \sigma_{RS} \). The influence of the compressive residual stress can be obtained by loading the crack surface over the area \( 0 < r < c \) with a stress \( \sigma \), where

\( \sigma = 2\sigma_{RS} \). The stress intensity for a partially loaded circular crack as shown in Fig. 3.9 is

\[ K = \frac{2\sigma}{(\pi a)^{1/2}} (a - (a^2 - c^2)^{1/2}) \]

(3.14)

c is indicated in Fig. 3.9.

To take the increase of the stress intensity factor due to the finite width of the web plate into account, the K-value has to be multiplied by the finite width correction factor
\[ F_w = \left( \frac{2b}{\pi a} \tan \frac{\pi a}{2b} \right)^{1/2} \]  \hspace{1cm} (3.15)

where \( b \) is the half the plate width or the width of the horizontal stiffener plus the thickness of the web.

**Stage II b**

As soon as the crack front reaches the back free web surface there are two curved crack fronts moving up and down the web as shown in Fig. 3.7b. These crack fronts can not be described by the circular crack model. The crack fronts eventually develop a through crack in the web. The crack length \( 2a \) is an average crack length measured on the center line of the web.

**Stage II c**

Both crack fronts eventually become more or less perpendicular to the web surfaces as shown in Fig. 3.7c. This crack stage can best be described by a through crack in an infinite plate. For the dead load and live load a uniform tensile stress at the horizontal stiffener is assumed using the magnitude of stress at the horizontal stiffener. The variation in bending stress can be neglected because of the uncertainty of other estimates such as the magnitude and distribution of the residual stress due to the welding of the stiffener on the web.

For an uniformly loaded through crack the stress intensity factor is

\[ K = \sigma (\pi a)^{1/2} \]  \hspace{1cm} (3.16)
taking the correction for the plastic zone into account and solving
Eq. 3.4 and Eq. 3.16 simultaneously a K value of

$$K = \frac{\sigma (\pi a)^{1/2}}{1.0 - \frac{1}{2} \left(\frac{\sigma}{\sigma_y}\right)^2} 1/2$$  \hspace{1cm} (3.17)

At the location of the stiffener stresses as high as the yield strength are present. Eq. 3.17 reduces to

$$K = \sigma (2\pi a)^{1/2}$$  \hspace{1cm} (3.18)

The crack length a for the through crack is the average crack length measured on the center line.

As soon as the crack tip enters the compressive residual stress zone, the stress intensity factor can be estimated by superposition. The crack surface is assumed to be loaded with $\sigma_{RS}$ over its entire surface. The influence by the tensile stress is estimated using the solution for a partially loaded through crack$^{53}$ as shown in Fig. 3.10 a

$$K = \frac{2}{\pi} \sigma \sin^{-1} \left(\frac{c}{a}\right) (\pi a)^{1/2}$$  \hspace{1cm} (3.19)

c is defined in Fig. 3.10.

3.2.5 Numerical Results

The stress intensity factor as a function of the crack length is shown in Figs. 3.11 and 3.12. In Fig. 3.11a the maximum stress intensity factor with and without the finite width correction for the circular crack penetrating the web is shown. In Fig. 3.12 the stress intensity factor adjusted for the plastic zone is shown; the
crack length is measured on the centerline of the web. The stress in Eq. 3.18 is the sum of the live load stress, dead load stress and residual stress.

During Stage IIa, the circular crack length was measured from the end of the stiffener. The minimum crack length for the crack entering the web is 114 mm, the width of the horizontal stiffener. As soon as the crack enters the web, the stress intensity becomes larger than the low level of fracture toughness of 55 MPa√m so that brittle fracture occurs as soon as a few load cycles sharpened the crack tip enough. The plane stress plastic zone radius (Eq. 3.4) about 4 mm for a K-value of 55 MPa√m, less than the web thickness so that brittle fracture is triggered. For a K-value of 110 MPa√m the plastic zone is 16 mm, which is larger than the web thickness so that no brittle fracture occurs because of the plastic deformation. Brittle fracture for material with a fracture toughness of 110 MPa√m occurs when the crack size measured on the middle line of the web produces a K-value for a through crack of the same magnitude. At this stage, the crack length 2a is 42 mm, the crack is 84 mm long on the web surface on the stiffener side. When the crack front reaches the opposite web surface, the stress intensity factor is 143 MPa√m. The actual fracture occurred in the Quinnipiac River Bridge at about this level of fracture toughness. The K-Value of 165 MPa√m is reached for a crack length 2a of 96 mm measured on the middle line. The crack shapes for Kc of 110 MPa√m and 165 MPa√m are shown in a schematic in Fig. 3.13.
As soon as the crack front leaves the zone of high residual tensile stress, the K-value decreases very rapidly with increasing crack length, see Fig. 3.12. The change from tension to compression residual stresses is not as abrupt as assumed. The positive stresses from the dead load and the live load prevent the K-value from becoming negative.

As long as the crack front does not completely penetrate the web, the fatigue life is estimated by replacing \(\Delta K\) in the Paris Power Law by the product of Eq. 3.13 and 3.15. The stress range \(\Delta \sigma\) is 8.1MPa, \(b\) in Eq. 3.15 is the width of the stiffener plus the web thickness. The fatigue lives are indicated in Table 3.2 and in Fig. 3.11b. After the crack penetrated the web completely the model for fatigue crack growth using the circular crack is still valid. The length \(b\) in Eq. 3.15 is the crack length plus \(t_w/2\).

The fatigue lives in Fig. 3.11b and Table 3.2 do not include the life to completely fracture the stiffener weld joint. If this connection fractured only due to bridge traffic, the estimated fatigue life is between \(2 \times 10^6\) and \(20 \times 10^6\) cycles\(^{51}\).

3.3 Glenfield Bridge

3.3.1 Introduction

On January 28, 1977 a tugboat captain spotted a large crack developing in the Interstate 79 Glenfield Bridge over the Ohio River Back Channel at Neville Island near Pittsburgh\(^{54}\).

The cracked girder is part of a continuous structure with three
spans of 68.9 m, 109.7 m and 68.9 m. The fracture occurred in the middle of the 109.7 m long center span. The location of the fracture, plan view and the cross-section are shown in Fig. 3.14. Due to the fractured girder the deflection at midspan increased about 10 cm without significant damage to the road surface. The crack in the tension flange was about 5 cm wide when discovered.

The superstructure of each span includes two main girders spaced at 12.2 m with transverse floor beams trusses spaced at 7.6 m. These floor beam trusses support the longitudinal stringers. The girders support a 21.6 cm thick concrete roadway. The design assumed that there is no composite action between the steel girders and the concrete slab. However, the observed behavior showed, that at least some composite action was present after the girder fractured.

The bridge was opened to traffic on September 3, 1976. The bridge carries the traffic in four lanes over the Ohio River. At the fractured cross section the girder is composed of a 3353 mm by 13 mm web and 89 mm by 762 mm flanges. At the crack location the web and the bottom flange were fabricated from A588 steel.

Near the crack a vertical stiffener is welded to the web; it was not welded to the bottom flange (tight fit). At the casualty section an electroslag weld (shop weld) was used to splice the 89 mm thick plates that form the tension flange. Shop records indicated that repairs had been made to the original weld. Radiographs were made before and after the repair and noted on the NDT.
reports but these records were never located after the fracture. About 5 cm from the fractured cross section a submerged arc welded splice had been made in the web. The geometry of the girder and the crack are shown schematically in Fig. 3.15.

On January 28, 1977, the day when the fractured web was observed, the temperature dropped about 15° C to 20° C (from 10° C to -10° C) within 90 minutes as a cold front passed through the area. On January 17, 1977, the temperature had reached a low value of -27° C, when an earlier cold front passed through the area.

A large section (0.7 m x 1.5 m) of the girder containing the crack in the tension flange was removed from the structure for a detailed investigation. The girder was subsequently repaired with a bolted splice and the bridge was reopened to traffic.

From a visual inspection it was concluded, that the fracture was related to the electroslag welded joint in the flange. Similar electro slag weldments existed at other locations in the tension flange of the same structure. An extensive study was undertaken to determine the causes of the fracture and to help evaluate the probability of cracking in other electroslag welded connections in the structure.

3.3.2 Material Characterization

The material removed from the fractured girder was used to make specimens for materials tests in order to determine the
material properties. Test data from the mill report was also available.

The base material (A588 steel) of the bottom flange has a yield stress of 420 MPa and an ultimate strength of 580 MPa according to the mill report. Charpy V-Notch tests were made from the base material, the electroslag weld metal, the fusion line/heat affected zone and of repair weldment material. The results of the Charpy V-Notch tests are shown in Figs. 3.16 a-c. The test of the base material showed good toughness with an impact energy of 20 joule at 0° C, see Fig. 3.16a.

The material from the heat affected zone (HAZ) and the electroslag weldment showed a CVN value of 20 joules at -11° C. The results for these two materials are shown in Figs. 3.16 b and c.

Charpy V-Notch tests were also carried out on the multiple span repair weld in which much of the initial flaw was imbedded. Due to the limited material available only 5 tests could be carried out at -18° C and 5 at 4° C. The test results are shown in Fig. 3.16 d. The average value at -18° C is 8 joule and 19 joule at 4° C.

Benter\(^{60}\) provides some data for \(K_{IC}\) fracture toughness for electroslag weldments connecting A588 steel. Data is also available from tests carried out at Lehigh\(^{61}\) on similar steel (A537G) with electroslag weldments. These tests are summarized in Fig. 3.17 and compared with the dynamic fracture toughness values estimated
from the Charpy V-Notch test data using Barsoms correlation, Eq. 3.9. It can be seen, that the static fracture toughness for the weld metal and the heat affected zone reported in Ref. 60 varies between 60 MPa√m and 80 MPa√m for a temperature of -40° C. Similar variation of resistance can be expected from the electroslag weldment and from the multiple pass repair weld based on the Charpy V-Notch results. The Charpy V-Notch test results for the electroslag weld metal of the casualty girder show a transition temperature of about 20° C. As can be seen from Fig. 3.16b the CVN-values increase rapidly over a small increase of temperature. The predicted $K_{Id}$-value also changes rapidly in the same temperature region (see Fig. 3.17). The static fracture toughness values were estimated shifting the dynamic values. The maximum temperature shift expected for these materials is between 65° C and 70° C. For 1 sec loading time a temperature shift of 50° C results. The resulting K-values for the intermediate loading rate for the electroslag weldment are shown in Fig. 3.17. The mean dynamic fracture toughness almost coincides with the upper bound of the static fracture toughness based on the estimate from Benters' data. The lower bound of the $K_{Id}$ and $K_Q$ tests are also in agreement. Because of the temperature shift, the transition temperature is now at -20° C. The CVN-values for the fusion line and the heat affected zone of the electroslag weld (Fig. 3.16 c) and the repair weld material (Fig. 3.16 d) are located near the lower limits and the transition temperature is therefore higher for these conditions.
The $K_{Id}$-values for these materials are indicated in Fig. 3.17. The fusion line, heat affected zone and weld repair show less resistance than the electroslag weld metal.

The estimated variation in fracture toughness at an intermediate loading rate is cross hatched in Fig. 3.17. A fracture toughness between 75 MPa$\sqrt{m}$ and 120 MPa$\sqrt{m}$ is expected at $-20^\circ$ C.

Benter$^{60}$ also reported on studies on the fatigue crack growth rate in electroslag weld metal and in the adjacent heat affected zone. He found these growth rates were comparable or smaller than the untreated base material. Therefore the same constants in the Paris equation were used for the electroslag weld and the heat affected zone than for the base metal.

3.3.3 Stress Distribution in the Vicinity of the Crack

At the crack in the flange, stresses due to different loads and residual stresses are present. Only stresses in the longitudinal direction are considered. These stresses are caused by the dead load (weight of the structure), the live load (traffic), from welding (residual stresses) and from temperature gradients over the cross section of the continuous bridge.

From the design calculations it was found that the dead load stress at the cracked section is 130 MPa. This calculation is based on the assumption of noncomposite behavior which is reasonably compatible with the construction of the structure. The dead load stress
in the bottom flange is not affected by the composite behavior of the bridge. It is assumed that the stress is uniformly distributed over the flange plate.

The design live load is based on a lane load. The design live load plus impact results in a live load stress range of 52 MPa. Stress history measurements were made during the summer of 1978 and resulted in the stress spectrum shown in Fig. 3.18 and tabulated in Table 3.3. The stress events given in Table 3.3 yield an equivalent stress range (Miners Rule) of 5.1 MPa. The peak stress was 14.1 MPa. The equivalent stress range considered all the recorded 1892 stress events. The stress due to the actual traffic is obviously much smaller than the design stress. The difference between the design and the measured stress results from several factors. This includes the number of vehicles on the bridge, the magnitude of impact, composite behavior and the three dimensional behavior of the structure. The measured live load stress is mostly due to the passage of single trucks.

Residual stresses in the bottom flange are from different origins. Longitudinal stresses are initially introduced in the flange by the electroslag welding procedure. During electroslag welding, the two plates are connected by liquid metal. The hot metal cools faster on the outside than on the inside of the weld. This results in residual stresses in the longitudinal direction of the weld. Near the weld surface, the material is in compression and tension develops in the center of the plate. The assumed initial
residual stress distribution is shown in Fig. 3.19 for the electroslag weld. The stresses are indicated for three horizontal planes in the flange: near the interior surface, at the middle of the plate and near the exterior surface. It was assumed that the stresses vary linearly between these locations. This initial residual stress distribution was later altered by repair welds.

During inspection (X-Ray) of the original electroslag weld several flaws were found which required repair. The repair was made in different stages by air arc gouging and replacing the material with manual weld passes. Different stages of the repair are shown schematically in Fig. 3.20. The metallographic examination indicated that one repair was made by gouging a boat shaped cavity through the weld and in the connected plate in the longitudinal direction. This excavation was located about 5 cm from the center line of the flange and was made from the bottom flange surface. At the crack plane the repair weld was through the flange thickness. A schematic of the repair welds at the crack plane and along the member is shown in Fig. 3.20. The removing of the material also affected the residual stress in this area. The assumed residual stress distribution is shown in Fig. 3.21 for the electroslag weld after gouging out the crack. The residual stresses shown in Fig. 3.21 are in equilibrium with the residual stresses away from the repair weld in the flange and web. The repair weld was made manually; the repair cavity was filled by depositing many passes of weld metal. The resulting residual stress distribution from the repair weld is shown in Fig. 3.22.
After repair of the flange plate, the web was attached using automatic submerged arc fillet welds. The web-flange welds only alter the residual stresses in the electroslag weld in a finite region. The submerged arc fillet welds heat and melt the flange plate and the electroslag weldment as they pass along the plate. This does provide some relief to the residual stresses in existence prior to making the weld. It is believed that this release is small compared with the undisturbed zones and that the changes can be neglected. The residual stress field due to the flange-web fillet welds is shown in Fig. 3.23. This distribution was approximated based on residual stress measurements on welded steel girders with flange and web plate of similar dimensions and fabricated from the same grade of steel.36

Residual stress also exists due to other sources that are not considered here. Flame cutting the flange plates introduces tensile residual stresses. However, these stresses are maximum at the edges of the flange and do not appreciably affect the residual stress field near the middle of the flange. Residual stresses may also be present due to rolling and straightening the plate. The residual stress fields to be considered in the fracture analysis of the Glenfield Bridge girder are summarized in Figures 3.21, 3.22 and 3.23.

Stresses in the girder flange are also caused by temperature gradients through the depth of the bridge cross section. On the day the final fracture was discovered, the temperature dropped rapidly in the Pittsburgh area when a cold front moved through. The concrete
slab cools much slower down than the completely unprotected steel girders. It was estimated$^{56}$ on the basis of strain measurements that a temperature difference between the slab and the bottom flange of 8° C would result in thermal tensile stresses of about 28 MPa in the bottom flange.

3.3.4 Crack Growth Stages

From a visual inspection of the crack surface it was apparent that the crack grew in different stages through the tension flange. One side of the fracture surface is shown in Fig. 3.24 and a close-up of the initial crack region is given in Fig. 3.25. Below the web and 6 mm below the interior flange surface a zone with a black oxide appearance and irregular contours is visually apparent. The fracture surface was flat and coated with a light oxide. The irregular shaped discontinuity is about 70 mm wide and 50 mm deep. One of the longitudinal web-flange fillet welds can be seen to penetrate into the discontinuity near the top flange surface. The absence of shear lips at this region suggests that the discontinuity was partially exposed at the flange surface, prior to making the web-flange connection. Both the top and the bottom flange surfaces show small shear lips except at the black zone. Examination of the crack surface with the electron microscope indicated$^{57}$ that interdendritic separations in the weld metal of the electroslag weldment had occurred. Hot cracking apparently developed during welding and high temperature oxides formed on the fracture surface. This zone provided the
initial flaw and is designated Stage I in Fig. 3.26.

Fracture surface replicas were obtained at 5 locations around the black area for an electron microscope investigation\textsuperscript{57}. The fractographic investigation showed striation markings around the periphery of the initial discontinuity. An example of markings from one location is shown in Fig. 3.28. Striation markings are a sign of fatigue crack growth. The zone where fatigue crack growth occurred is indicated as Stage II in Fig. 3.26. This zone was only one or two millimeters wide (No exact measurements could be made because of the irregular shape and the corrosion of the fracture surface). Measurements of the striation spacings showed that they were between $1.1 \times 10^{-5}$ mm and $1.0 \times 10^{-4}$ mm apart. Fatigue crack growth is the extension mode of this stage.

During Stage III brittle fracture (crack instability) occurred in parts of the flange and web. The brittle fracture was arrested when the crack reached a semicircular size of 76 mm radius as shown in Fig. 3.25. The zone within this semicircular region was slightly corroded. One location along the semicircular crack front was investigated and striation markings were found there as well. This stage of fatigue crack growth is designated as Stage IV in Fig. 3.26. The zone where fatigue crack growth occurred is very small (less than 1 mm). Crack growth between Stage I and Stages IV is believed to have occurred before January 28, 1977 because of the rust found on the fracture surface. The final fracture of the bottom flange (Stage V) and the web was observed on January 28, 1977\textsuperscript{54}. The
appearance of this surface is crystalline and distinctly different from the origin.

The crack front in the web did not move along the weld in the web. The dimensions for the different crack shapes that were investigated are shown in Fig. 3.27.

3.3.5 Analysis of Crack Growth

Stage I

Stage I is believed to have occurred during the electroslag welding and the weld repair as hot cracking. Its cause is reported elsewhere and is not the subject of this investigation. The initial discontinuity in the weldment was taken as the initial flaw size for this investigation.

Stage II

During Stage II fatigue crack growth developed from the initial flaw. The crack in the flange was modelled as an elliptical shaped flaw subjected to uniformly applied stress. The diameter of the ellipse was taken such that the ellipse circumscribed the irregular initial flaw shape. The shape and size of the ellipse is shown in Fig. 3.26 and 3.27. The stress intensity factor for an elliptical crack in a finite plate under uniform tension is given by Eq. 3.20, (see Fig. 3.29).

\[ K = \sigma (\pi a)^{1/2} F_e F_w \]  \hspace{1cm} (3.20)

where \( F_e \) is the correction factor for the shape, Eq. 2.25. \( F_w \) takes
the finite thickness of the plate into account. The finite width correction factor was developed from the solution for an excentrical through crack in a finite plate \(^5\) (p. 11.2, Ref. 53). Reference 53 provides the finite width correction factor in graphical form as a function of the geometry as indicated in Fig. 3.30.

The finite width correction factor for the ellipse shown in Figs. 3.26 and 3.27 varies between 1.00 and 2.06, depending on the location of the crack front. Because of the highly irregular shape of the initial flaw, the stress intensity factor may vary locally as well. To take this effect into account Eq. 3.20 was corrected by an empirical crack shape correction factor C.

\[
K = \sigma (\pi a)^{1/2} F_e F_w C \tag{3.21}
\]

The electron microscope investigation showed that some striation markings were between \(1.1 \times 10^{-5}\) mm and \(1.0 \times 10^{-4}\) mm apart. Striation markings are caused by cyclic applied loads. Every large load cycle may produce one marking. There is also a level of stress intensity range below which crack propagation can occur but no striation markings can be detected. Striation markings are only detectable at high magnification and over a small region of crack growth. Striations are generally only visible in bridge steel when the growth rate \(\text{da/dN} > 2.5 \times 10^{-6}\) mm/cycle. On a microscopical scale the crack front does not advance steadily; the advance of the crack front is arrested for some load cycles as when the crack front reaches a grain boundary of the material or the stress cycles fall below the crack growth threshold. The macroscopic crack growth rate, as expressed by the
simple power law is an average of the different components of the microscopic growth rate. \(^{64}\) Microscopic growth is manifested by crack growth striations, growth because of void coalescence, growth due to clevage fracture and growth due to corrosion.

Quantitative expressions have been developed for the striaion mechanism of growth. Estimates are not available on the number of load cycles and their magnitude that are needed to advance the crack front through grain boundaries. Bates and Clark\(^ {55}\) developed an empirical relationship for constant cycle loading between the stress intensity range and the distance between the striations. They found that the distance \(\Delta s\) can be approximated by

\[
\Delta s = 5.4 \left( \frac{\Delta K}{E} \right)^{2.1}
\]

Eq. 3.22 was developed from measurements made on aluminum alloy under constant cyclic stress amplitude and a minimum stress intensity factor equal to zero (\(R=0\)). In order to generalize the equation for use with other materials, \(\Delta K\) was normalized by dividing by Youngs modulus of elasticity. Other studies\(^ {62}\) have indicated that the \(\Delta K\) value estimated from striation measurements is only accurate to within 40%. This relationship was extended by Hertzberg and von Euw\(^ {62}\) for \(R\)-ratios larger than zero. They related the striation markings to an effective stress intensity range \(\Delta K_{\text{eff}}\). Their test data was acquired on 2024-T3 aluminum alloy and a more general relationship was developed as:

\[
\Delta s = 24 \left( \frac{\Delta K_{\text{eff}}}{E} \right)^{2}
\]

\(99\)
where $\Delta K_{\text{eff}}$ was determined empirically for 2024-T3 aluminum as

$$
\Delta K_{\text{eff}} = (0.5 + 0.4R) \Delta K
$$

(3.24)

This relationship takes into account the fact that the crack will be closed near the tip during part of the unloading cycles even when $R > 0$. This behavior was first proposed by Elber\textsuperscript{63} and verified by compliance measurements.

Eq. 3.22 and 3.23 were both developed for constant amplitude stress cycles. The traffic crossing a bridge causes random variable loads and stresses. Measurements of stress range suggest that most of the stress cycles will be below the fatigue crack growth threshold. Those cycles that exceed the crack growth threshold will only infrequently exceed the level of crack growth $(2.5 \times 10^{-6} \text{ mm cycle})$ that can be detected from the striation markings. Hence the crack front will be advanced between striation marks by a relatively large number of stress cycles. This will cause the measured striations to be spaced further apart than predicted by either Eq. 3.22 or 3.23. Or conversely any estimate of $\Delta K$ based on striation markings would be overestimated. Albrecht's\textsuperscript{78} laboratory studies on fatigue striations at weld toe cracks made with A588 steel showed that the striation marks only exist on less than 10% of the fracture surface (about 70% of the fracture surface of aluminum shows striation markings). Some of the remaining fracture surface is covered with so called quasistriations, whose spacings are an order of magnitude larger than the expected growth rate for the A588 steel. Albrecht also found that if an overload is applied between every 10 and 100
cycle, then the average striation spacings are further apart than predicted by Bates and Clark equation. Bucci\textsuperscript{73} observed, that at low $\Delta K$ very small percentage overload (order at 10%) can cause crack growth delay. Striation markings are therefore useful for verifying the fracture mode but quantitative statements are very difficult to base on these markings. It is also possible that small markings may have been whipped out by the cleaning of the fracture surface.

For an empirical crack shape correction factor $C = 1.0$ in Eq. 3.21 and with the $R$-ratio equal to 0.8, a stress intensity range between 5.4 MPa$\sqrt{m}$ and 16.2 MPa$\sqrt{m}$ results from the striation measurements. Eq. 3.22 yields a $\Delta K$-value of 1.2 MPa$\sqrt{m}$ resulting from the effective Miner-stress range of 5.1 MPa. Both the Paris Power Law and Eq. 3.22 indicate that for a $K$-value of 5.4 MPa$\sqrt{m}$ on the average only every 19th cycle would produce a marking. By assuming that stress cycles larger than 10.3 MPa produce striation markings and that the smaller stress ranges increase the crack length then every 23rd cycle larger than 10.3 MPa would produce one marking. The relationship for the effective $K$-value due to the applied $\Delta K$-value (Eq. 3.24) was developed on homogenous materials. In homogenous materials the crack front is straight and the applied $\Delta K$-value can be related to the effective $\Delta K$-value. In the weld material, however, the crack front is irregular and the crack closing effect is affected by adjacent crack fronts.

If a correction factor of 1.59 is introduced to take the local
irregularities into account, then the estimated crack growth rate \( da/dN \), based on the measured random variable stresses in the flange under traffic and the measured striation spacing provide comparable conditions. About every 19th stress cycle would be expected to produce detectable crack growth.

**Stage III, Brittle Fracture**

During cold temperature and under dead and live load stress the embedded elliptical flaw became unstable and brittle fracture occurred. The brittle fracture was arrested when the stress intensity factor was less than the dynamic fracture toughness and/or the crack tip reached a zone with a higher fracture toughness.

The contributions to the stress intensity factor due to residual stress, due to live load can be obtained by superposition of these different effects. Since the live load stress and the dead load stress are nearly uniformly distributed over the crack surface, Eq. 3.20 can be used to estimate their contribution to the stress intensity factor. The stress intensity factor due to the varying residual stress field can be obtained by numerical integration of the K-values due to a splitting force applied at a point on the crack surface. The K-value for the concentrated force shown in Fig. 3.31 is given in Ref. 42. The stress intensity at one location on the crack front is

\[
K = \frac{P}{\pi} \frac{a^{1/2}}{\ell^{1/2}} \left( \frac{\mathcal{K}}{R} \right)^{1/2} \frac{1}{(\frac{1}{a} - \frac{1}{2} - 1)^{1/2}} \frac{1}{(1 - K^2 \cos^2 \phi)^{1/4}}
\] (3.25)
The parameters in Eq. 3.25 are defined in Fig. 3.31. The crack surface was approximated by a 0.7 mm by 0.5 mm mesh. A computer program was used for the integration. A uniform stress distribution was also numerically integrated and compared with the solution provided by Eq. 3.20. This comparison showed that the numerical solution was within ±10% of the exact value depending on the location on the crack front. A correction factor for each point on the crack tip was calculated and this individual correction factor was used to adjust the increment of stress intensity factor due to the residual stress. The stress intensity factor must also be adjusted by the finite width correction factor as described in Stage II.

The stress intensity factor along the elliptical crack front as a result of residual stress, and the dead and live load stresses are shown in Fig. 3.32. The estimated maximum K-value is 104 MPa√m and occurs directly under the web-flange connection.

The maximum K-value of 104 MPa√m is compared with fracture resistance in Fig. 3.33. The material resistance is the material resistance at the intermediate loading range from comparison it can be seen that crack instability would be possible at temperatures below -8°C. It seems highly probable that crack instability developed from the original defect at temperatures below -20°C. It was necessary for the initial flaw to be sharpened by cyclic live load. The initial crack tip was not as sharp and the K-value did not make the material resistance until nearly six months of traffic had used the structure and the temperature had decreased to a very low
level.

Stage III, Crack Arrest

The original brittle fracture was arrested after the crack had assumed a semicircular shape with a radius of about 76 mm. The crack was arrested because the K-value decreased at the tip and/or because the crack tip entered a zone of higher material toughness. Fig. 3.17 indicates that the material toughness can vary at a given level of temperature.

The stress intensity factor of the semicircular crack due to the changed residual stress distribution can be calculated using the same procedure outlined in the first part of stage III. The K-value must also be corrected by factors that take the finite width of the plate and the free surface into account. For an edge crack

\[
\frac{F_w}{F_s} = \left( \frac{2b}{\pi a} \tan \frac{\pi a}{2b} \right)^{1/2} \frac{0.752 + 2.02 \frac{a}{b} + 0.37 (1 - \sin \frac{\pi a}{2b})^3}{\cos \frac{\pi a}{2b}}
\]

(3.26)

a is the crack length, b the plate width. Equation 3.26 overestimates the \( \frac{F_w}{F_s} \) correction factors for a circular crack. The maximum crack length a for a circular crack is at 90°. The adjacent material restrains the plate from bending and the crack from opening. The finite width correction factor was calculated using an equivalent crack length. For the equivalent crack length, a *, it is assumed that the stress intensity factor for a semicircular (or semielliptical) crack in an infinite plate is the same as for an
edge crack in a semi-infinite plate. The stress intensity factor for an edge crack is [53]

\[ K = 1.1215 \sigma (\pi a^*)^{1/2} \]  \hspace{1cm} (3.27)

and for a semielliptical surface crack [53]

\[ K = F_s \frac{\sigma (\pi a)^{1/2}}{E (k)} (1 - k^2 \cos^2 \phi)^{1/4} \]  \hspace{1cm} (3.28)

The free surface correction factor for the semielliptical surface crack varies between 1.0 and 1.30 [42]. For convenience, a value equal to 1.1215 was used, the same as for the edge crack. Eqs. 3.27 and 3.28 can be used to estimate the equivalent crack length \( a^* \) as

\[ a^* = a \frac{1}{E'(k)} (1 - k^2 \cos^2 \phi)^{1/2} \]  \hspace{1cm} (3.29)

For a semicircular crack Eq. 3.29 reduces to

\[ a^* = 0.405a \]  \hspace{1cm} (3.30)

The finite width correction factor can be expressed as

\[ F_w = \frac{1}{1.1215} \left( \frac{2b}{\pi a^*} \tan \frac{\pi a^*}{2b} \right)^{1/2} \frac{0.752 + 2.02}{a/b + 0.37} \frac{\left(1 - \sin \frac{\pi a^*}{2b}\right)^3}{\cos \frac{\pi a^*}{2b}} \]  \hspace{1cm} (3.31)

The crack sizes shown in Fig. 3.27 were evaluated for the various stress conditions. Figs. 3.34, 3.35 and 3.36 show the resulting K-values which include the effect of the residual stress due to the electroslag welding, the repair weld, the web to flange weld and the stress due to the dead and live load. The analysis suggests that the maximum K-value at the time the crack arrested was about 70 MPa√m.
The maximum K-value is compared with the dynamic fracture toughness in Fig. 3.33. This comparison suggests that the lowest temperature for crack arrest is about 7° C. It is probable that the irregular crack front and blunting as the crack enlarged provided a higher level of fracture resistance than indicated. The stress intensity for crack sizes less than 76 mm is smaller than 70 MPa√m. This likely assisted in decreasing the velocity at which the crack was moving and helped arrest the crack at the 76 mm radius.

Stage IV

After brittle fracture was arrested, the crack size was again increased by fatigue crack growth. Fatigue crack growth was verified by striation markings at crack tip location 5 (see Fig. 3.35). The crack tip was sharpened by cyclic loading and the resulting stable crack growth. About 1 mm of crack growth was observed.

Stage V

When the temperature dropped and decreased the fracture toughness in January 1972 the stress conditions were again adverse and the remaining section of the tension flange fractured. The cold front that passed through the Pittsburgh area on January 28, 1977 produced a temperature gradient in the structure and resulted in thermal tension stresses in the bottom flange. The K-value at the moment of the fracture, considering the residual stresses, dead and live load stresses and thermal stress assumed to be about 28 MPa, was 81 MPa√m. The estimated K-value at fracture is compared with
the material toughness in Fig. 3.37. The estimated K-value falls within the static fracture resistance band at the temperature believed to exist on January 28, 1977 when the crack was first observed. Fig. 3.37 suggests, that for temperatures below -17° C brittle fracture occurs.

3.3.6 Conclusions

Brittle fracture of the tension flange of a girder in the Glenfield Bridge developed because a large initial weld defect enlarged in fatigue and exceeded the fracture resistance of the material. Variations in material toughness and changes in the stress field as a result of residual stresses from welding permitted the crack to arrest at a relatively large crack size. For large crack sizes the fatigue crack growth rate is high even under small cyclic applied stresses. Hence fracture of the girder was inevitable.

A low level of fracture toughness ie. \( K_c = 55 \text{ MPa}\sqrt{\text{m}} \) would have provided about the fatigue life as a girder with a fracture toughness of 110 MPa\(\sqrt{\text{m}} \). A crack as large as fabricated into the I-79 girder is predicted to become unstable as soon as the cyclic stresses sharpen the initial flaw. The stress including temperature stress in the bridge are sufficient to produce instability. A higher fracture toughness would not have affected the behavior significantly because a small increase in the crack size increases the finite width correction so rapidly that fracture will occur.
The initial flaw in the Glenfield Bridge girder was large enough to limit the fatigue resistance to a relatively short life. Fracture would occur as soon as the stress in the bottom flange due to the differential temperature was increased enough to trigger the fracture. Higher levels of fracture toughness would not have materially altered the behavior of the bridge.

3.4 Lafayette Street Bridge

3.4.1 Introduction, Material Characterization

On May 7, 1975, one of the main girders of the Lafayette Street Bridge over the Mississippi River in St. Paul, Minnesota was discovered to be cracked. The Lafayette Street Bridge is a three span structure with spans of 82.3 m, 110.3 m, and 76.4 m. The crack was discovered in the middle span of the east girder of the south bound bridge. The crack was located about 36.2 m away from a pier. The cross section at the location of the crack consists of two welded main girders, about 3.5 m high and a concrete slab which carries the roadway. The bridge was designed for noncomposite action between the steel girders and the concrete slab. A schematic of the spans and the cross section are shown in Fig. 3.38. The crack, when detected, had completely fractured the bottom (tension) flange and had extended up the girder web close to the top flange. The crack was found when an excessive deflection with respect to the adjacent structure was investigated. The gusset plate was partially cracked near a vertical stiffener as can be seen
in Fig. 3.39. Portions of the web, flange and gusset were removed from the bridge for laboratory studies.

After the crack was repaired a second crack was discovered about 29 m away from the same pier. The crack was about 25 mm long on the web surface when detected. A plug with the second crack in it was removed from the bridge for further study.

The structure was opened to traffic on November 13, 1968 and was closed for road surface repairs for about 160 days during summer 1974 (Closed from May 20 to October 25, 1974). The daily truck traffic crossing the bridge was estimated from available records to be 1500 trucks. For the period of time between November 13, 1968 and May 7, 1975 about 3.3 Million trucks had crossed the structure.

The main girder at the critical location was fabricated out of A441 steel (web and flange), the gusset plate was A36 steel. The A441 steel had a yield strength of 317 MPa in the flange and 370 MPa in the web. The yield strength of the A36 material was found to be 260 MPa.

In Fig. 3.40a the results from Charpy V-Notch tests made on web material are shown. The CVN values are converted to $K_{Id}$ values using Eq. 3.9. The temperature shift of 75° C is used to obtain the $K_{Ic}$-values and 56° C to obtain the $K_{I 1 sec}$ (see Eq. 3.11). In addition four compact tension specimen were fabricated and tested at -18° C, -32° C, -84° C and -101° C. The test at -18° C was
made at an intermediate strain rate (1 sec from zero to maximum load), the other tests were static tests. The two tests at the lowest temperature fulfilled the requirements of the E399 specifications for linearity of the P-Δ plot. The thickness requirement is violated by all specimens. The K-values from the compact tension tests and the K-values obtained with the CVN-results are shown in Fig. 3.40b. For better comparison the tests value measured at -18° C (1 sec loading time) is converted to a static K-value using a temperature shift of 19° C. It can be seen, that the test results at -18° C and -32° C are well below the expected values from the CVN-results. The analysis using the J-integral method would have predicted higher values than the $K_Q$-analysis. The $K_Q$ analysis only takes the yield load into account and neglects plastic deformation. Higher K-values can therefore be expected than predicted by the $K_Q$-value. An extrapolation of the CVN results predicts K-values at the intermediate loading rate of 120 MPa√m to 140 MPa√m for temperatures of -22° C. This temperature corresponds to the minimum service temperature recorded in St. Paul prior to the discovery of the fracture during the winter of 1974-1975.

The dynamic fracture toughness is estimated with Eq. 3.10. A value of 50 MPa√m for the web and 45 MPa√m for the flange material results.
3.4.2 Stress Distribution at the Critical Location

In the uncracked girder the following stresses in longitudinal direction are present: Dead load stress, live load stress and residual stresses due to local welding.

Noncomposite action was assumed for the design. The design calculations provide a dead load stress of 58.5 MPa in the bottom flange of the girder and 51.6 MPa at the location of the gusset plate. The live load range due to the design HS 20 truck loading generates a stress range of 32.3 MPa in the girder flange and 28.5 MPa at the web to flange connection. Strain measurements of a single HS 20 truck provided a stress range of about 14 MPa. Based on the design stress range and the few measurements the Miner stress range was estimated to be about 13.8 MPa at the location of the gusset and 15.6 MPa in the bottom flange.

The biggest residual stresses are caused by welding the gusset and stiffeners to the web and by welding the web to the flange. These residual stresses were estimated separately. The welding of the gusset and stiffener to the web was assumed to produce stresses in the longitudinal direction equal to the yield strength.

The stiffener was not welded to the bottom flange. The stress distribution of the residual stresses due to the web to flange welding is shown in Fig. 3.41.
3.4.3 Crack Growth Stages

The investigation concluded that crack growth occurred in different stages. The different stages are shown in Fig. 3.42.

Stage I

The crack surface in the gusset indicated that the transverse and longitudinal single level groove welds were not completely fused through the plate thickness to the back up bar. A large initial flaw existed in the gusset plate in a plane perpendicular to the applied cyclic stress from truck traffic. The size of this initial flaw near the weld connecting the gusset to the stiffener varies between 4 mm and 9 mm to 10 mm near the web. The unfused zone is about 150 mm long and of irregular shape.

Stage II

The cyclic applied stress due to the truck traffic load increased the size of the flaw in the gusset and started the crack into the web. Eventually it completely penetrated through the thickness of the gusset plate. Fatigue crack growth was verified by a fractographic examination with the electron microscope. Fatigue striations were detected at several locations in the gusset plate-transverse stiffener weld. The fatigue crack grew into the web, at the same time it was propagating in the transverse weld. Due to corrosion of the fracture surface the size and the shape of the zone where slow stable fatigue crack growth occurred in the web is not clearly visible. From examination of the plug, where
similar crack growth occurred, it can be concluded that the fatigue crack entered the web in different parallel planes. The crack had nearly propagated through the web at the time of discovery. No cracking was visible in the transverse weld from the top surface of the gusset plate. From the crack surfaces shown in Fig. 3.43 it can be concluded that the fatigue crack growth zone is rather long near the surface of the gusset side.

The fatigue crack reached a critical size after it had propagated 9 mm to 10 mm into the web. It propagated through the web in an unstable manner and also destroyed the tension flange.

Stage III

Stage III is the rapid propagation of the crack tip through the web and the flange. Rapid propagation of the crack moving up the web was arrested when the crack tip was about 100 mm above the gusset plate. It is believed that the bottom flange was completely fractured. The fracture surface of Stage III of the web and the bottom flange were coated with an oxide. The gusset plate still remained partially unfractured. The crack in the gusset plate was about 250 mm long and had been arrested by the bolt holes in the gusset plate.

Stage IV

After the unstable crack growth was arrested the cyclic applied stress due to the traffic load increased the crack size in the web. Electron microscope pictures show clearly striation markings in
the web about 100 mm above the gusset plate. At the same time slow
stable fatigue crack growth also occurred in the gusset plate.
However, small ligaments between the crack tip and the bolt holes
and between the bolt holes show signs of ductile tearing.

Stage V

The girder finally cracked up to the top flange and the crack
was detected and repaired after the remaining part of the web was
completely destroyed. At time of the repair the deflection of the
bridge at the cracked section had increased to 190 mm.

That the girder fractured in different stages can also be con-
cluded from the different coating of oxide on the crack surface.
Also the deflection measured on March 20, 1975 indicated the various
modes of crack extension existed as the bridge had deflected 64 mm
at that time.

3.4.4 Analysis of Crack Growth

Stage I

The cause of the initial crack is due to fabrication which
resulted in a lack of fusion region in the single bevel transverse
groove weld. The initial flaw in the 13 mm thick gusset plate was
about 9 mm to 10 mm deep near the web and several centimeters long.

Stage II

The estimated time for the fatigue crack to penetrate the

114
The gusset plate is difficult to estimate because of the highly irregular crack shape of the initial flaw and because of the varying, irregular material properties of the faulty weld. The initial crack can best be described as a quarter elliptical surface crack with a $a/c$-ratio (minor to major axis of the ellipse) between $1/6$ and $1/8$. Because of the near by vertical stiffener and web connection the stress flaw due to the applied load is disturbed in the gusset plate. The gusset plate, without the web, with only the stiffener welded on it, can be considered as a cruciform joint as investigated by Frank. For the applied stress, Frank developed the following expression for the stress intensity factor $K$.

$$K = k_T F_F \left( \frac{a}{t} \right) \frac{1.0-0.12(1.0-a/c)}{E(k)} \left( \cos \frac{\pi a}{2b} \right)^{-1/2} \sigma(\pi a)^{1/2}$$

(3.32)

where $F_F \left( \frac{a}{t} \right)$ the stress concentration decay polynomial developed by Frank is:

$$F_F \left( \frac{a}{t} \right) = 1.0 - 3.215 \left( \frac{a}{t} \right) + 7.897 \left( \frac{a}{t} \right)^2 - 9.288 \left( \frac{a}{t} \right)^3 - 4.086 \left( \frac{a}{t} \right)^4$$

(3.33)

For $k_T$ Frank determined the value of 2.64. This stress intensity factor was developed for a symmetrical cruciform joint and for two cracks growing from either outside surface into the plate. Eq. 3.33 has to be adjusted for the plate thickness $t$ instead of $t/2$. However the crack grows from the inside of the plate towards the free surface. The expression $\frac{a}{t}$ in Eq. 3.33 has to be replaced by $\frac{t_G - a}{t_G}$ where $t_G$ is the thickness of the gusset plate, 13 mm.
Originally Frank used the secant correction factor to take the finite width into account. However for \( a/t_G \) ratio larger than 0.4 the tangent correction factor as shown in Eq. 3.15 is more accurate to predict the influence of the finite width. The plate width \( b \) in Eq. 3.15 is the thickness of the gusset plate. The finite width correction factor considers the gusset plate to be completely restrained from bending. Actually the gusset plate is restrained from bending by the web. For the cyclic applied stress range of 13.8 MPa and a gusset plate thickness \( t_G \) of 13 mm the fatigue life can be estimated by substituting Eq. 3.34 in the crack growth equation (Eq. 2.6) and integrating to determine the fatigue life.

\[
\Delta K = 2.64 F_F \left( \frac{t_G}{t_G} \right) \left\{ \frac{1.0 - 0.12 (1.0 - a/c)}{E(k)} \right\} \left( \frac{2t_G}{\pi a} \tan \frac{\pi a}{2t_G} \right)^{1/2} \Delta \sigma (\pi a)^{1/2} \quad (3.34)
\]

For an initial flaw size of \( a_1 = 3.7 \) mm a fatigue life of \( 1.38 \times 10^6 \) cycles results in penetration of the gusset plate. For an initial flaw of 8.6 mm, the life is \( 3.69 \times 10^6 \) cycles. The structure sustained about \( 3.3 \times 10^6 \) cycles; the fatigue life estimate is in good agreement with the observed life of the structure. The influence of larger \( a/c \)-ratios on the fatigue life between \( a/c = 1/6 \) and \( a/c = 1/8 \) is 5.5%. It can therefore be assumed that the gusset was completely cracked from the transverse stiffener before March 1975.

Although a satisfactory model is not available to describe
its growth, the crack was simultaneously propagating into the girder web. The fatigue crack moving into the web was modeled as part of a circular crack with a radius \( r = 38 \text{ mm} \) (corresponds to \( a/c = 1/7.9 \) for crack growth in the gusset plate) as shown in Fig. 3.44. Where the crack tip enters the web, the stress range is locally increased by the vertical stiffener welded to web and the end of the longitudinal weld connecting the gusset to the web. The stress concentration decay polynomial developed by Frank, Eq. 3.33 was used to estimate the \( F_g \) correction factor for this crack extension. The expression \( \frac{t_G - a}{t_G} \) in Eq. 3.33 has to be replaced by \( \frac{a - r}{t_w} \) where \( t_w \) is the thickness of the web (13 mm). For an embedded circular crack the crack shape correction factor is \( \frac{2}{\pi} \) and the free surface correction factor \( F_s = 1.0 \). The stress concentration factor \( K_T \) in Eq. 3.32 is larger than 2.64 because of the additional longitudinal gusset-web weld. For the stiffener-web connection alone the \( K_T \) factor is 2.64. The resulting stress intensity factor is

\[
\Delta K = K_T F_F \left( \frac{a-r}{t_w} \right) \frac{2}{\pi} \left( \frac{2(r+t_w)}{\pi a} \tan \frac{\pi a}{2(r+t_w)} \right)^{1/2} \Delta \sigma (\pi a)^{1/2} \quad (3.35)
\]

For an assumed \( K_T \) value of 2.98 the crack front penetrates the web (9.5 mm) using \( 3.3 \times 10^6 \) applied load cycles. At a depth of 9.5 mm fatigue striations were still found. At this stage, the crack is 56 mm long on the inside web surface \( (c = 28 \text{ mm}) \).

This analytical model for the crack shape overestimates the crack length measured on the web surface compared with the fracture behavior observed at the location of the second crack (location
where the core was removed).

**Stage III**

During Stage III the web and the bottom flange fractured brittle. As stated earlier, the stress in the web at the location of the gusset was very high; the two vertical welds that connect the stiffeners to the web and the longitudinal welds connecting the gusset to the web raised the stress locally up to the yield point. The stress intensity factor for a circular crack and loaded over the cross hatched area as shown in Fig. 3.44a can be obtained by superposition of Eqs. 3.13 and 3.14. For the crack size a equals to 48 mm and a stress the magnitude of the yield strength (370 MPa) a K-value of 54.6 MPa√m results. This result has to be adjusted by the finite width correction factor provided by Eq. 3.15. For a plate width of b = r – t_w and a crack length of 48 mm a value of F_w equal to 2.58 results. Therefore a K-value of 140 MPa√m is obtained. At the minimum temperature measured in St. Paul (-22° C) the lower bound fracture toughness is estimated from the CVN tests is of the same order of magnitude for the intermediate loading range (see Fig. 3.40).

The crack in the web can also be modeled as a semielliptical surface crack with a = 9.5 mm and c = 28 mm. For this semielliptical crack the maximum stress intensity factor is 96.5 MPa√m (including the finite width correction factor of 1.39 (for a = 9.5 mm and b = t_w)). The two results are in fair agreement.
The estimate of the stress intensity factor for the web crack neglected the influence of the restraint of the gusset upon the opening of the crack in the web. Ref. 36 has shown that this influence is less than 10%. For this case this influence is even smaller because of the relative large crack in the gusset.

Brittle fracture is arrested when the K-value is smaller than the dynamic fracture resistance $K_{Id}$ of the material. After the crack propagated through the web thickness K can be calculated for a through crack, Eq. 3.16. The finite width of the web can be neglected because of the relative small crack length compared with the height of the web plate. The applied stress on the web surface is at the yield strength as long as the crack is in the region of the welds from the stiffener and gusset. No crack arrest occurs for the crack front moving towards the bottom flange.

The K-value in the bottom flange can be estimated using the procedure outlined in section 3.3. The estimated K-values are compared in Fig. 3.45 with the dynamic material toughness of the flange. The dynamic material toughness is always smaller than the estimated K-value and crack arrest can not occur.

The crack front that moved upwards in the web was arrested after the crack was about 100 mm above the gusset plate. As a result of the principle stress direction the crack front did not move perpendicular to the flange and the crack tip moved out of the high tensile residual stress zone. At this stage the crack can be modeled as a large edge crack in a semi-infinite plate. For a
crack length of 10 cm the finite width of the web can be neglected
($F_w = 1.0$). The free surface correction factor is $F_s = 1.122$.
The full crack opening was also prevented by the gusset plate which
acted as a flange. The presence of the gusset plate reduces the
K-value in the web. Its effect can be calculated by applying a
closing force on the web crack at the level of the gusset plate.

The estimated stress distribution along the crack path is
indicated in Fig. 3.46. Up to 200 mm high in the web the crack
is in the zone of high residual tensile stresses ($\sigma = \sigma_Y$). The
web plate away from the stiffener is in residual compression
(residual stress due to the web-flange weld), a stress of 69 MPa
is assumed. The resulting K-value due to this stress distri-
bution is shown in Fig. 3.46. The plotted K-value is obtained by
superposition of the following three components. The total
crack length is loaded with a compressive stress of 69 MPa, the
K-value is calculated with Eq. 3.27. In the region near the
bottom flange the solution for a partially loaded crack, Eq. 3.19
has to be employed to estimate K. The effect of the closing
force due to the gusset plate on the K-value is obtained from
Eq. 3.36, see Fig. 3.10.

\[
K = \frac{2P}{(\pi a)^{1/2}} \frac{a}{(a^2 - b^2)^{1/2}} \quad (3.36)
\]

where $b = a - 152$ mm and $P$ the closing force is.
To estimate the magnitude of the closing force it is assumed that the remaining, uncracked part of the gusset sustains stresses as high as the yield strength. For the calculation it was assumed that about 168 mm of the gusset were uncracked. The resulting closing force is distributed over the web thickness.

From Fig. 3.46 it can be seen that the K-value for the web crack is less than the dynamic fracture toughness for a crack length of 275 mm. Crack arrest therefore results. At this stage the deflection of the bridge had increased a significant amount.

**Stage IV, Stage V**

During Stage IV the crack propagated in the web and in the gusset as a result of fatigue. This stage was not investigated because the useful fatigue resistance of the bridge member was exhausted when the initial brittle fracture of the web and bottom flange occurred.

The fatigue crack growth decreased the closing force developed in the gusset plate. A higher K-value at the crack tip of the web crack resulted and brittle fracture of the remaining part of the web was triggered. The K-value without the closing force of the gusset plate is also indicated in Fig. 3.46b as a function of the crack length.
3.4.5 Influence of the Fracture Toughness on the Fatigue Behavior

The fatigue life of the Lafayette Street Bridge was assumed to be exhausted when brittle fracture during Stage III occurred. Although the bridge did not collapse and was still in service for a time after the tension flange was fractured, the deflection exceeded the tolerable limits and such a large crack is also undesirable. Therefore it was assumed, that the life of the structure was exhausted after brittle fracture developed in the girder web.

The actual life of the bridge is set as 100%. Then the life for the low level of fracture toughness (55 MPa√m) is 31.2%, for the fracture toughness of 110 MPa√m is 90.6% and for the high K-value of 165 MPa√m 104.9%. Or, expressed in days, the bridge would have been usefull for 1.9 years if the $K_c$ value would have been 55 MPa√m, 5.5 years for the medium K-value or 6.3 years for the high toughness value of 165 MPa√m.

The stress intensity factor and the fatigue life as a function of the crack length is shown in Fig. 3.47 and in Table 3.4.

3.5 Yellow Mill Pond Bridge

3.5.1 Introduction

In October - November 1970, during a routine cleaning and repainting of the Yellow Mill Pond Bridge, a large crack was discovered in one of the cover-plated steel beams of the east bound
span II bridge. The Yellow Mill Pond Bridge is located in Bridgeport, Connecticut on Interstate 95. The crack had developed in the bottom flange of a rolled interior beam at the end weld of a cover plate 2.8 m from the support of the 34.6 m long simply supported span. The crack initiated at a fillet weld toe of the west end of a cover plate end weld on beam 4. The tension flange and part of the web was completely cracked when discovered. The crack extended about 400 mm up the web. A photograph of the cracked girder is shown in Fig. 3.48. The section containing the crack was removed from the structure for inspection and material tests. The fractured girder and two adjacent girders with smaller cracks were repaired with bolted splices.

The Yellow Mill Pond Bridge complex consists of 28 simple supported pairs of spans and several on and off ramps. The bridge crosses the Yellow Mill Channel, highways and a railroad track. Span II, where the fracture occurred, is a landspan about 10 m above ground. Each bridge carries three traffic lanes in one direction. Each bridge consists of 7 or 8 longitudinal girders fabricated from rolled sections. All girders are reinforced with two cover plates of different length on the tension flange and with one cover plate on the compression flange. Most of the rolled sections are W36x230 A242 steel profiles except for the fascia girders. Both spans 10 and 11 are 35.05 m long (center line support). The girders support a 185 mm thick concrete deck which is protected by an overlay of bituminous concrete. A schematic of span 10 and 11 is
shown in Fig. 3.49.

The bridge was designed in 1955 and opened to traffic in 1958. The design assumed composite action between the concrete deck and the steel girders. One major aspect in designing the structure was the vertical clearance between the channel and the bridge super structure. To improve the clearance rolled beams reinforced with multiple cover plates were used instead of built up members.

At the fractured section the girder is a rolled W36x230 profile with a cover plate welded on it. The cover plate is 32 mm thick, 381 mm wide and 28.96 m long. The corners of the cover plate were rounded (r = 76 mm) but not tapered. The fillet welds at the cover plate ends are manually made with the weld leg size between 13 mm and 15 mm.

After the major crack was found the bridges were inspected several times and other fatigue cracks were detected in 1970, 1974, 1976, 1977, and 1979\textsuperscript{10,23}. Several methods have been employed to repair the fatigue damage\textsuperscript{10}.

3.5.2 Material Characterization

The girders of the Yellow Mill Pond Bridge were made from A242 steel with a yield stress of 394 MPa for the material removed from the web and 398 MPa for the flange material. Charpy V-Notch test results from the web and flange material were available. These test results are summarized in Fig. 3.50. The material removed

124
from the web shows a transition temperature of 20 joules at -23° C. The flange provided 20 joules at +15° C. The difference in the transition temperature is due to the different thickness of the web and flange. An excessive amount of manganese was found in the flange material, which may have increased the transition temperature. The CVN test results were used to estimate the fracture toughness for a dynamic loading rate, an intermediate loading rate and a static loading rate using Eq. 3.11. The results are shown in Fig. 3.51 for the web material and in Fig. 3.52 for the flange material.

Three point bend specimens were fabricated from the material removed from the web. Due to the limited material available only four specimens 177.8 mm by 44.5 mm by 19.1 mm were made. The specimens were oriented in the rolling direction so that the fatigue crack was oriented in the same direction as in the girder. The specimens were tested at three different temperatures, one specimen at -40° C, two at -29° C and one at -18° C. The loading time for the fracture test was 1.5 sec, comparable with the loading time in bridge structures. The three point bend tests could not be evaluated using the ASTM E399 specification, because the material did not satisfy the test requirements. The J-integral procedure was employed for a $K_J$-estimate. The J-integral results were converted to values of critical stress intensity factor $K_c$ by

$$K_c = (J E)^{1/2}$$  \hspace{1cm} (3.37)

where $J$ the energy release rate per unit crack extension is. The
resulting fracture toughness values are compared with the fracture toughness estimated from the CVN tests in Figs. 3.51 and 3.52. The fracture toughness from the three point bend specimens is in good agreement with the fracture toughness for the intermediate loading rate.

3.5.3 Stresses at the Critical Location

The stress in the bottom flange near the cover plate end weld are caused by the dead load, the live load and cooling after rolling and welding. The structure was designed to act compositely so the neutral axis of the cross section was above the centroid of the rolled section. The location of the neutral axis depends on the ratio of the moduli of elasticity of the concrete and the steel and on the effective width of the concrete deck. The most conservative assumptions resulted in the neutral axis being about 280 mm above the neutral axis of the rolled section alone. However, the dead load stress distribution depends on the way the bridge was constructed. Usually the dead load is only carried by the steel structure alone. The dead load stress in the bottom flange at the crack location was estimated to be 27.6 MPa. The stress distribution in the girder is shown in Fig. 3.53.

The short time traffic loads act on the composite section. Considerable efforts have been made to determine the stress in the bottom flange due to the traffic. Strain measurements were made in 1971, 1973 and 1976. Traffic counts were made

126
simultaneously. Strain ranges were measured with electrical strain
gages and with mechanical recording equipment over longer period of
time. All the stress measurements were made on span 10. However,
both spans (10 and 11) are similar in construction and subjected to
the same traffic, so that the measurements were usefull to estimate
the stress at the casualty location. Miner's effective stress
range at the west end of Span 10 at the weld toe was 8.1 MPa. The
smallest stress range in the stress spectrum that is used to cal-
culate the Miner effective stress range has a significant influence
on the magnitude of the equivalent stress range. The smaller the
minimum stress is, the smaller the equivalent stress range, however
more stress cycles result. The number of vehicles, the number of
stress cycles and the minimum stress range are dependent on each
other. For this analysis the minimum stress range was taken as
2.1 MPa, the Miner effective stress range is between 11.0 MPa and
13.1 MPa and each truck crossing the structure was assumed to pro-
duce between 1.0 and 1.8 stress cycles estimated on the basis of
the experimental measurements. A maximum stress range of 72.4 MPa
was recorded during the measurements made in 1976 and was believed
to be caused by a multiple truck presence on the bridge.

Based on the available ADTT counts, the total number of trucks
crossing the bridge was estimated to be about \(21 \times 10^6\) between the
opening of the bridge and the detection of the first crack in 1970.
Hence the initial cracked detail sustained between \(21 \times 10^6\) and
\(37.8 \times 10^6\) stress range cycles. The distribution of the stress is
shown in Fig. 3.53 for the composite action of the bridge.

The residual stress distribution in the rolled W36x230 profiles with welded cover plates was investigate and reported in Ref. 36. The investigation was made using an analytical model and available experimental measurements. These measurements were made on A588 steel beams with a yield strength of 458 MPa. The residual stresses in the Yellow Mill Pond girders were modified to be proportional to the different yield strength. The resulting distribution in the bottom flange is shown in Fig. 3.54 for the residual stresses due to rolling and in Fig. 3.55 due to welding the cover plate on the beam.

### 3.5.4 Description of the Fracture Surface, Crack Growth Stages

The fracture surface was heavily corroded and partially covered with paint when discovered. It was obvious that the crack existed for some time before it was detected. Because the surface was so corroded, it was very difficult to establish the different crack growth stages. No electron microscope investigation was made. It is believed, that at least three different stages contributed to the final fracture.

The crack started at the toe of the transverse fillet weld connecting the cover plate to the flange and grew completely through the tension flange. This type of crack growth was expected based on laboratory tests. There was a change in the crack path
when it encountered on inclusion condition near the center of the flange. Small ligaments at the edges of the flange tip showed signs of tensile fracture. Fatigue crack growth also extended about 5 cm into the web. At that location there was evidence of necking.

Crack instability occurred in the web and extended about 230 mm up the web. After the crack was arrested it continued to grow under the cyclic applied load due to the traffic until it was discovered on November 2, 1970. It had extended about 400 mm up the web.

It is believed, that a very large overload passed over the bridge in September 1970. However, no records of the magnitude of this load or the stresses are available.

3.5.5 Analysis of Crack Growth

From a visual inspection of the crack surface it was concluded, that no brittle fracture occurred in the flange. Brittle fracture occurred after the tension flange was completely cracked and the crack front was about 5 cm up the web. Stress intensity factors due to the dead load stress, the live load stress and the residual stress for two elliptical crack sizes were calculated and compared with the estimated fracture toughness of the flange material. The stress intensity factors were estimated using the numerical procedure outlined in Section 3.3.5 (see Fig. 3.31) for an elliptical surface crack. The K-value corresponding to a crack
length of a 13 mm (crack front at about mid thickness of the flange) and for a of 25 mm (Flange is almost fractured ($t_{fl} = 32$ mm). From a preliminary inspection it was concluded that crack instability had developed when the crack was about halfway through the flange. Later it was concluded that the crack path had only changed its direction due to the delamination. The crack length along the cover plate end was estimated from Eq. 2.35 to be 126 mm ($c = 63$ mm) for the 13 mm crack and 277 mm ($c = 139$ mm) for the larger crack size of 25 mm. Inspections at the other cover plate ends have demonstrated that the cracks are very often the full width of the cover plate end weld. An analysis was also made for a crack width $2c$ of 178 mm for the smaller crack length and 356 mm for a of 25 mm. The two investigated crack shapes are shown in Figs. 3.56 and 3.57.

The stress intensity factor obtained by integrating Eq. 3.24 over the crack surface was corrected by the front free surface correction factor and by the back free surface correction factor. Both factors are a function of the crack length and the plate thickness. The back free surface correction factor takes the boundary conditions of the plate into account. The web and the cover plate restrain the flange plate from bending which keeps the back surface straight. Hence the combined correction factor given in Eq. 3.38 can be used

$$F = (1.0 + 0.122 \cos^4 \frac{\pi a}{2b}) \left( \frac{2b}{\pi a} \tan \frac{\pi a}{2b} \right)^{1/2}$$  \hspace{1cm} (3.38)

Eq. 3.38 was derived from the correction factor for a double edge notch specimen. No calculation of an equivalent crack length was
made because the crack is very shallow and long and therefore acts more like an edge crack with a restrained back surface.

The correction factor shown in Eq. 3.38 can also be obtained by the method of superposition of the two independent correction factors $F_s$ (Eq. 2.28) and $F_w$ (Eq. 3.15). The calculated correction factor is slightly larger, but the difference is only about 2.5%. This small difference is not significant so Eq. 3.38 was used for all further calculations.

The stress intensity factor for the dead load and the live load stress was determined from the closed form solution and corrected by the front-and back free surface correction factor. The stress gradient correction factor for crack lengths larger than 3 mm is unity.

The numerical results are shown in Figs. 3.56 and 3.57. The plotted $K$-value includes all correction factors. Without traffic the maximum stress intensity factor is 123 MPa$\sqrt{m}$ for a crack length of 25 mm. The maximum observed traffic load ($\sigma = 72.4$ MPa) would increase the $K$-value to 156 MPa$\sqrt{m}$. Comparison with the material fracture toughness shows that the girder flange would likely fracture at temperatures below 0° C. During the winter time the temperature is less than 0° C in Bridgeport, but no fracture was observed. This likely results because of the following reasons:

- The dynamic fracture toughness was estimated from Charpy V-Notch test results and a temperature shift was applied to obtain the
estimated fracture toughness. The empirical estimation of the toughness based on the Charpy results is valid in the transition region, values for the comparison are at the upper end of the transition region and some error is possible. The temperature shift is an empirical relationship and may also involve some error. The material toughness may therefore be greater than estimated. Tests on material removed from the web exhibited a high toughness and considerable yielding developed at the crack tip of the compact tension specimens even at -20°C. These values are plotted for comparison purposed in Fig. 3.52.

- The residual stresses were estimated from measurements on other sections so that deviations are possible. As can be seen in Fig. 3.56 and 3.57 the residual stress due to welding of the cover plate and from rolling contribute between 70% (traffic load included) and 90% (no traffic load) to the total stress intensity factor.

- The finite width correction factor is a conservative estimate, it assumes an infinite large edge crack (c + ∞).

- The remaining ligament is very small (5.9 mm). Plastic deformations are likely to occur which increases the material toughness.

- A maximum stress due to live load of 72.4 MPa was observed during the summer of 1976. No other such large stress ranges were
observed. Hence the maximum stress range is likely less than 72.4 MPa over much of the service life.

- A heavier load at higher temperatures may have blunted the crack tip (warm prestressing) so that it lost its sharpness and reduced the stress intensity factor.

It was observed on the bridge that cracks at other cover plate end welds were halfway through the flange and extended over the cover plate width. The influence of crack width \( c \) on the stress intensity factor is small. The crack shape factor for several crack sizes was calculated with Eq. 2.25 and varies between 0.933 for \( a = 2.5 \) mm to 0.958 for \( a = 13 \) mm. If the crack geometry relationship

\[
c = 7.0 \ a^{1.0}
\]  

(3.39)

is assumed, the crack shape correction factor is constant (0.972).

Considering all these uncertainties the resulting stress intensity is a reasonable estimate. It was apparent on the crack surface that small ligaments fractured in rapid crack growth. This was evident from the small shear lips. However, no brittle fracture occurred in the flange although the stress intensity factor was quite high and near the estimated fracture toughness.

Crack instability occurred when the crack tip was in the web:

The crack in the web was modeled as an edge crack under bending loading.\(^5\)
\[
K = \sigma (\pi a)^{1/2} \left( \frac{2b}{\pi a} \tan \frac{\pi a}{2b} \right)^{1/2} \frac{0.923 + 0.199 (1\sin \frac{\pi a}{2b})^4}{\cos \frac{\pi a}{2b}} \quad (3.40)
\]

\(\sigma, a\) and \(b\) are indicated in Fig. 3.58. The stress intensity factor is not governed by the applied stress alone. The stress intensity factor is governed by the opening at the crack tip due to deflections of the bridge girders. The crack opening \(v\) is

\[
v = \frac{4a}{E} \left( 0.8 - 1.7 \frac{a}{b} + 2.4 \left( \frac{a}{b} \right)^2 + \frac{0.66}{(1-\frac{a}{b})} \right) \quad (3.41)
\]

\(v\) is indicated in Fig. 3.58. The applied traffic load causes the bridge to deflect and the crack is opened. Some load transfer to adjacent girders takes place because of the increased flexibility of the cracked girder. This causes increased forces to be carried by the diaphragms between the girders and resulted in fatigue failures in some of the bolts. If it is assumed that the heavy over load produced 69 MPa stress in the girders and that the cracked girder did not carry load over half its length between the transverse diaphragms (3.5 m), then the crack would about 1.2 mm open at the flange surface. The stress intensity factor can be calculated from the opening of the crack using Eq. 3.40 and 3.41. A stress intensity factor of 275 MPa√m results.

The stress intensity factor is determined by the crack opening and the crack opening is due to the deflection of the structure. The crack opening, for a given stress is independent of the crack length. For an increasing crack length the \(K\)-value decreases. This leads to crack arrest when the crack length becomes 260 mm.
Subsequent stress cycles due to traffic will continue to increase the crack size. The fatigue life was essentially exhausted as soon as the bottom flange was cracked.

Since the opening of the structure approximately $21 \times 10^6$ trucks crossed the structure and produced between $21 \times 10^6$ and $37.8 \times 10^6$ load cycles at the cover plate ends. Fatigue life estimations were made using the correction factors suggested by Zettelmooye\textsuperscript{22}. If an initial crack size of 1.3 mm and the conservative crack shape equation (Eq. 3.39) are assumed, $43.8 \times 10^6$ cycles result from the fatigue life calculation using the Paris Power Law (Eq. 2.6) with the lower bound crack growth constant $C = 2.18 \times 10^{-13}$ mm$^{5.5}$ for an effective stress range of 13.1 MPa. The fatigue life for different initial conditions and crack shape relationships are shown in Fig. 3.59. For comparison Category E' of the AASHTO Specification is also plotted. The predicted fatigue life of the Yellow Mill Pond Bridge details are indicated by the cross hatched area. It can be seen that the fatigue life estimates are in good agreement with the observed behavior. The conservative assumptions for the initial conditions are justified because the Yellow Mill Pond structure has more than 200 other cover plate ends with similar stress ranges subjected to the same number of load cycles. The west end of the cover plate on beam B4 in span 11 was the first to fail. Cracks were also observed at other details and later repaired.

It was stated earlier that the initial flaw size has a large influence on the fatigue resistance. If the initial conditions are
varied to obtain the $37.8 \times 10^6$ cycles then an initial crack between 2 mm ($\Delta \sigma_{\text{Miner}} = 13.1 \text{ MPa}$) and 4 mm ($\Delta \sigma_{\text{Miner}} = 11.0 \text{ MPa}$) is estimated. The initial crack size at the cover plate end removed from the Yellow Mill Pond Bridge could not be determined from the examination of the fracture surface because of corrosion.

3.5.6 Influence of the Fracture Toughness on the Fatigue Behavior

The fatigue life of cover-plated beams is known to be exhausted when the flange is completely cracked. The residual fatigue life corresponding to crack growth in the web can be neglected because it is so small. Also the load carrying capacity is not sufficient. At the Yellow Mill Pond bridge the load was redistributed to the adjacent girders.

In Fig. 3.60 the stress intensity factor due to residual stresses from rolling and welding and due to dead and live load are shown as a function of the crack length for three different magnitudes of maximum life load. It can be seen that the contribution of the live load to the total stress intensity is small. For crack length smaller than half the flange thickness the total stress intensity factor is proportional to the square root of the crack length. The correction factors are nearly constant for crack length between 2 mm and 16 mm. For crack length larger than half the flange thickness the contribution of the weld residual stress decreases but the finite width correction factor increases rapidly.
Also indicated in Fig. 3.60 are the three investigated levels of toughness. For the low level of 55 MPa√m the critical crack length is 5.1 mm, for 110 MPa√m 19.6 mm and 26.2 mm for the high level of 165 MPa√m. For the estimation of the crack length the maximum traffic load of 72.4 MPa was assumed.

The fatigue life as a function of the crack length is shown in Fig. 3.61. If the fatigue life for the highest level of fracture toughness is assumed to be 100%, then the fatigue life for $K_c = 55 \text{ MPa}\sqrt{\text{m}}$ is only 73.3%, but almost the same life can be expected for $K_c = 110 \text{ MPa}\sqrt{\text{m}}$ as for $K_c = 165 \text{ MPa}\sqrt{\text{m}}$. 
3.6 Dan Ryan Viaduct

3.6.1 Introduction

A large crack was discovered in one of the steel box bents supporting the elevated track of Chicago's Mass Transit System (EL) of the Dan Ryan Line on January 4th 1978. Immediate inspection showed that two adjacent bents were also completely cracked. The structure was subsequently closed to traffic. The structure was reopened to traffic on January the 15th, 1978, after shoring towers were completed. In January 1978, low temperatures of \(-20^\circ\) C were recorded in Chicago\(^{75}\).

The viaduct at Clark and 18th Streets is part of the Lake - Dan Ryan Rapid Transit Line. The fractured bents were part of a 12 m high and 820 m long viaduct. The structure is welded steel construction with a super-structure of continous and suspended plate girders which carry a cast-in-place concrete trough. The rails of the two tracks are on wooden ties resting on ballast. The alignment of the tracks and the structure are on a 120 m radius curve.

The bents have a boxed shaped cross section with two column legs and a horizontal box member spanning between (see Fig. 3.62). The top flanges of the plate girders pass over the top flanges at the horizontal boxes; the bottom flanges pierce the boxes through flame cut slots in the lower portion of the box web plates. The girders intersect the boxes at different angles. The bents and the girders are fabricated from A36 steel.
The structure was designed in 1967 with the design criteria and procedures compatible with different codes, such as the American Railway Engineering Association (AREA) Code. The structure was built in 1968/1969. The girders were framed into the box girders to achieve a smooth uncluttered appearance. The structures' appearance was an important consideration and this type of construction minimizes the height. After traffic operations began, the structure was inspected several times. The last inspection before the cracks were discovered was in July 1976; no defects were found at that time.

The initial field examination of the fracture indicated that the cracks started at the welded junction of the plate girder flange tip and the box side plate. All three cracks completely severed the bottom flange of the box girders and the webs. Openings of about 20 mm were measured at the bottom flange. The cracks had arrested near or in the top flange. The cracks in the bottom flanges were inclined approximately 20 degrees to a normal to the longitudinal axis of the bent. Two crack surfaces were found to be slightly corroded, the third one was heavily coated with oxide. Samples were removed from each horizontal box girder for laboratory investigation.

3.6.2 Material Characterization and Stresses at the Critical Location

The material removed from the box girders was used to prepare specimens for tensile tests, chemical analysis, Charpy V-Notch tests,
Compact Tension tests, metallographic examination and fractographic investigation. The tests were carried out at different laboratories in accord with ASTM specifications. The physical and the chemical tests indicated that the material conformed to the ASTM requirements for A36 steel.

The yield strength (0.2% offset) measured with flat tensile specimens was determined to be 235 MPa and the tensile strength 480 MPa. The yield strength for 1 sec loading time at -20° C is 330 MPa (Eq. 3.12). The Charpy V-Notch tests provided impact energy greater than the AASHTO minimum requirement for Zone 2 of 20 joule at 4° C. The transition temperature was between 4° C and 20° C for 20 joule. The results of the CVN tests are shown in Fig. 3.63, the averages were calculated from 12 tests at each temperature.

Compact tension specimen were tested at one second loading time at three different temperatures. The tests did not fulfill the requirements of the ASTM E399 specification. Nonlinearity required the use of the J-integral method to estimate the critical stress intensity factor. The results are shown in Fig. 3.64. A critical plain strain material toughness $K_{IC}$ (average of 3 specimens) of 88 MPa√m was measured at -20° C.

Based on the dimensions obtained from the contract drawings a recalculation of the stresses was made using several finite element discretions. It was found that the magnitude of the tensile and compression stresses due to the dead load, design live load, impact
and centrifugal forces were in the order of 140 MPa and the shear stresses 85 MPa. The calculated stresses were generally within the allowable limits of the design specifications. At one location in the web of the box girder a shear stress was found to be 124% of the allowable stress. It was concluded that this overstress did not effect the safety of the structure.

The tensile stress range in the box girder at the tip of the girder flanges which framed into the box girder (location of the crack) was calculated to be up to 42 MPa for the live load (both tracks loaded), impact (29.4%, AREA Specification) and centrifugal forces. However, most service stress cycles are produced by only one train crossing over the structure at a time. The impact factor given in the specifications provides a maximum design value; the effective distribution of the impact factor can be assumed by a two parameter Rayleigh probability density function. The maximum impact is the design impact, the minimum impact is 0%. From the distribution curve a mean impact factor of about 12% results. Only one track at a time was usually loaded, which produces stresses at the critical location between 0.5 and 0.8 times $\Delta\sigma_{\text{Max}}$. The calculated stress ranges are shown in Table 3.5.

On an average weekday 467 trains (in both directions) pass over the viaduct; the trains vary in length between 2 and 8 cars. However, each train produces one load cycle at the critical locations. Fig. 3.65 shows the stress time relationship for the tensile stress when trains with 4 and 6 cars pass over the structure. During the
total life of the structure (8 years) $1.364 \times 10^6$ load cycles were applied.

High residual stresses due to welding the flange to the box girder web are present in the crack region. A total stress as high as the yield stress can be expected due to the residual stress, dead load stress and live load stress combined.

3.6.3 Description of the Fracture Surface, Crack Growth Stages

A typical box girder bent is shown in Fig. 3.62. In bents 24 and 25 the fracture occurred in girder 4. In bent 26 the crack occurred in girder 3. The fracture surface of the webs of the box girders adjacent to the stringer flange tip were examined to determine the cause of the cracking. Schematics and photographs of the fracture surfaces are shown in Figs. 3.66, 3.67, and 3.68. The fracture surfaces show that the weldment connecting the flange tip to the box-web had large unfused areas. Paint was found to penetrate from the inside of the box into these areas. Surface replicas were made and examined with the transmission electron microscope \(^{75}\). From the observations made with the electron microscope it was concluded that the fatigue cracks grew from the lack of fusion area as well as from the exterior surface at the weld toe one the box girder web. Striation markings were found on all three fracture surfaces. The marks were between $2 \times 10^{-5}$ mm and $5 \times 10^{-5}$ mm apart.

The observations indicated that crack growth occurred in
different stages. Stage I corresponds to the initial imperfection which was the large lack of fusion area at the flange tip in the girder web. Stage II was the development of fatigue crack growth from the weld toe on the outside surface and from the lack of fusion area. Stage III is the brittle fracture of the web plate. From the slightly different degrees of corrosion on the fracture surface it was concluded that brittle fracture occurred almost at the same time in two bents. The third surface with the heavy oxide coating fractured earlier.

The fracture of pier 26 was evaluated in this study. The similar fracture surfaces of piers 24 and 25 suggested comparable fracture behavior.

3.6.4 Analysis of the Crack Growth

Fatigue Crack Growth

Simultaneous fatigue crack growth occurred in the box web flange connection at the exterior weld toe and at the large lack of fusion area. Fatigue crack growth at the exterior weld toe was observed over the full flange thickness as shown in Fig. 3.68. The fatigue crack had therefore many origins and is very shallow. As the crack shape ratio a/c approaches zero, the crack shape factor $F_e \rightarrow 1.0$, (Eq. 2.25). The depth of the crack is very small compared with the web thickness so that the finite width correction factor was taken as 1.0. The free surface correction factor becomes 1.122 for the edge crack in an infinite solid. The stress gradient

143
correction factor $F_g$ is given by Eq. 3.42 for a weld angle of 45° (see Ref. 72).

$$ F_g = 4.21 \frac{1}{1 + 7.30 \frac{a}{0.576}} $$  \hspace{1cm} (3.42)

The crack length in Eq. 3.42 is measured in m. About 50% of the web at the welded connection at pier 26 is unfused. This was taken into account by increasing the stress range by a factor of two. This results in the following stress intensity range

$$ \Delta K = 2.444 \Delta \sigma \frac{1}{1 + 7.30 \frac{a}{0.576}} \left( \frac{\pi a}{1} \right)^{1/2} $$  \hspace{1cm} (3.43)

It was assumed that the unfused segment remains constant in Eq. 3.43. This assumption is not correct, but the increase due to fatigue crack growth is very small compared with the original unfused area. The assumed finite width correction factor is valid as long as the crack length remains small compared with the plate width. At fracture the crack growing from outside into the web was about 3 MPa deep compared with the 19 mm thickness of the web.

The fracture surface of the web plate at the bent 26 shows a large unfused area of irregular shape. Formulas for stress intensity factors for such irregular shaped flaws as shown in the schematic in Fig. 3.68 are not available in closed form solution. The preexisting unfused area can be approximated as a semielliptical surface crack. The semidiameter $a$ and $c$ were choosen so that the ellipse approximates the natural flaw. The resulting stress intensity factor along the crack front is only an average value and
it may vary locally because of the irregular crack front. Sharp corners will elevate the K-factor locally.

All relationships for the estimation of the stress intensity factor assume an infinite sharp crack tip. The natural flaw due to the lack of fusion does not have a continuous sharp crack tip. Some fatigue cycles are needed to sharpen the tip and initiate crack growth.

The stress intensity factor for the semielliptical surface crack in the web at the at the flange tip is governed by Eq. 3.3 and the following correction factors:

- The finite width correction factor $F_w$, Eq. 3.15 where $b$ is the thickness of the web. Because crack growth also occurs in the same plane at the outside web surface, the width $b$ does not remain constant.

- The crack shape correction factor $F_e$, Eq. 2.25. For a constant ratio $a/c$ of the axis the correction factor remains constant.

- The front free surface correction factor $F_s$ was defined by Eq. 2.28.

- The stress gradient correction factor $F_g$ given in Eq. 3.42 was derived for a crack growing from the exterior into the web. The stress gradient correction factor for the crack growing from the load of fusion area is derived from Eq. 3.42, the crack length $a$ has to be replaced by $t_w - a$ as shown in Eq. 3.44.

$$F_g = 4.21 \frac{1}{1 + 7.30 (t_w - a)^{0.576}}$$

(3.44)
As soon as the two crack fronts connect a through crack results. The remaining small fused area shown in Fig. 3.70 was neglected; a through crack of the length 2a of about the thickness of the flange plate results.

The fatigue life for a crack growing in the web plate from the exterior is shown in Fig. 3.69. Also plotted are the estimated cycles of stress from trains crossing the Dan Ryan Viaduct. The results plotted in Fig. 3.69 suggest that the initial crack size was about 0.08 mm. Fatigue crack growth continued until a through crack developed in the web plate.

Brittle Fracture

As soon as the crack front reached the crack tip growing from the unfused area, a through crack resulted in the web plate. The crack length, 2a, of the through crack is 39 mm (see Fig. 3.70). This is somewhat less than the thickness of the inserted flange plate because the welds around the flange plate fused the region near the top and bottom surface of the flange plate. The small fused areas within the region of the through crack were neglected because they are very thin compared with the web thickness and are of irregular shape. The material properties of that area are also ill defined. A few applied load cycles would likely crack this small area.

The K-value for a through crack in an infinite plate is given by Eq. 3.18. Eq. 3.18 takes the plastic zone correction into
account and is shown in Fig. 3.71 for an applied stress of 330 MPa. At a crack length of 39 mm, a K-value of 116 MPa√m results. At a crack length of 39 mm, a K-value of 116 MPa√m results. At the time of the fracture the material toughness was estimated to be 88 MPa√m so brittle fracture was a logical consequence.

Brittle fracture did not occur at smaller crack sizes because the fatigue life was consumed changing the crack shape from a semielliptical surface flaw into a through crack. The stress intensity factor did not exceed the material fracture resistance during this transition.

3.6.5 Influence of the Fracture Toughness on the Fatigue Life

For material with a fracture toughness less than 116 MPa√m the fatigue and fracture behavior would have been identical. 1.364 million stress cycles cracked the small fused area and a through crack resulted with a crack length 2a of 39 mm and a K-value of 116 MPa√m. The transition from the initial connection of the interior and exterior cracks is very rapid and does not significantly effect the fatigue resistance. During this transition, crack instability will develop when the material fracture toughness is less than 116 MPa√m. Hence no significant difference in behavior will occur.

For materials with a fracture toughness higher than 116 MPa√m brittle fracture does not occur immediately and the cyclic applied
load increases the crack length. From fatigue tests it is known that the crack growth rate for such details with a large through crack is faster than predicted by Eq. 3.16 and the SPL. This indicates that the cyclic applied stress is not uniformly distributed as assumed at a simple trough crack. A stress gradient correction has to be introduced into the calculations. The stress gradient correction factor is of primary importance to the fatigue life estimates but does not significantly alter the total stress intensity factor as a result of residual stress in the loads. This stress applied on the crack surface is already at the yield strength.

The stress gradient correction factor has to be introduced because of the disturbed stress flaw due to welding the inserted flange plate to the web. The stress gradient correction factor determined by Norris\textsuperscript{72} considers the variation in stress for a crack growth path perpendicular to the web surface along the weld toe. The $F_g$ factor for a through crack front perpendicular to the web surfaces and adjacent to the weld toe of the inserted detail is not available. An estimate of the decay of the $F_g$ factor was made to estimate the fatigue life increment after development of a through crack.

It can be assumed that the $F_g$ factor for the through crack is the same order of magnitude as the value for a semielliptical surface crack. Norris determined a maximum value of about 4.0 (see Eq. 3.42). As the crack propagates away from the inserted
plate, the stress gradient correction factor decays to one. The distance over which the stress flaw is disturbed by the detail was assumed to be $t_F \ell / 2$ and a linear or a quadratic decay function was assumed to account for the decay. The variation of the correction factor is shown in Fig. 3.72.

The maximum crack size $2a$ for a material toughness of 165 MPa$\sqrt{m}$ is 79 mm. The fatigue life for a through crack growing from a equals 19 mm to 39 mm has to be calculated step wise because the $F_g$ factor cannot be determined as a closed function. The following functions were employed:

$$
\begin{align*}
a < \frac{t_F \ell}{2} & \quad F_g = 4.0 \\
\frac{t_F \ell}{2} \leq a < t_F \ell & \quad F_g = F_g(a) \quad (3.45) \\
t_F \ell \leq a & \quad F_g = 1.0
\end{align*}
$$

If a linear decay function is assumed, then 498 400 additional stress cycles are needed to increase the crack size $2a$ from 39 mm to 79 mm. 773300 stress cycles result when a quadratic decay function is used.

The fatigue lives for the different fracture toughness levels are summarized in Table 3.6.
3.7 Welded Box Girders

3.7.1 Introduction, Description of the Cracks

The tension members of some tied arch bridges consist of welded built up sections. These sections are often welded box girders fabricated of high strength steel. High stresses due to the dead load and the live load are carried by these sections. These members are of high structural importance, because a fracture would result in a catastrophic failure of the bridge structure.

The box girders in the Gulf Outlet Bridge are made of four plates, the web plates are 711 mm high and 19 mm thick, the flanges are 635 mm wide and 16 mm thick. The plates are welded at the four corners, inside and outside of the box. The dimension of a box girder are shown in Fig. 3.73. The bottom flange is perforated at several locations, but the slots are far apart and small enough so that they can be neglected.

An examination of the longitudinal outside corner welds of the box girder revealed small cracks in the weld material and heat affected zone of the base plate. Some of these cracks were below the weld surface and were detected after slightly grinding the weld surface. Other cracks were also found in the inside corner weld at different locations. The distribution of these cracks was completely random but confined to regions with manually made welds. It is believed that these cracks are caused by hydrogen induced cold cracking. One of the largest cracks found and one average
crack are shown in Fig. 3.74. The geometry of the cracks was determined by breaking open cores removed from the box corner. The size and the shape of the cracks was also estimated by grinding the weld and base material away until no indication of a crack could be detected. The cracks were not only located in the weld metal, they also extend into the heat affected zone of the base material.

Because the cracks at the intersection between the web and the flange are oriented perpendicular to the main stress field due to the dead load and live load an investigation was made to determine if these cracks can propagate under the cyclic applied live load stress. High residual tensile stresses are also present in the box section particularly at the box corners so that the possibility of brittle fracture has to be examined as well. A brittle fracture might destroy the structure.

3.7.2 Stresses at the Critical Location

The bottom chord in a tied arch bridge is mainly subjected to high tensile forces due to the dead and live load and to residual stresses. From the design calculations a dead load stress of 252 MPa existed in the tie girder box of the Gulf Outlet Bridge. The dead load stress is assumed to be uniformly distributed over the cross section.

The live load stress range is difficult to estimate. The design live load stress, including an impact factor of 7.5%, is
48 MPa. For the determination of the live load stress the bridge was assumed to be fully loaded and that only axial forces are present in the tie. A reanalysis indicated, that in the center part of the span also bending stresses exist in the box girder\textsuperscript{91}. Measurements to determine the actual stress range are therefore needed. Based on the design calculations and on the results from the reanalysis the Miner stress range is estimated to be 14 MPa and the highest regularly encountered live load stress range is 28 MPa.

The residual stress due to the welding of the box was estimated using a procedure outlined in Ref. 79. Originally the method was derived to predict the deflections in heat cambered simple beams. To predict the camber of the beams the residual stresses and strains are calculated using a finite discretisation of the cross section. The calculation of the residual stresses assumed an elastic-perfectly plastic stress-strain response and temperature dependent material properties. The variation of the temperature as a function of the time and the location on the cross section for each weld pass has to be known\textsuperscript{79}. It has been shown that the temperature distribution at some discrete time intervals has to be known in order to minimize the calculation efforts\textsuperscript{80}. This approach predicts the residual stress distribution due to the welding and cooling off within an acceptable degree of error.

The determination of the temperature distribution in the web and flange plate due to the heat input of welding is based on the principles of heat transfer in a thin semi-infinite plate. The
temperature at a given location in the plate is a function of the heat input, the position of the location relative to the moving heat source (electrode) and various thermal properties of the plate. For the calculation of the temperature in the plate the following assumptions were made.

- The heat losses through the surfaces of the plate can be neglected. (Radiation)
- The temperature at some distance from the heat source remains unchanged.
- The heat source is a point source; the effects at the edge are neglected.
- The physical properties of the material are temperature independent. (only for the calculation of the temperature distribution, not for the residual stress distribution).

In addition to these four basic assumptions several other assumptions had to be made in order to predict the variation of the temperature (and therefore the residual stress) over the plate thickness.

- The temperature is dependent on the distance between the heat source and the location of the point for which the temperature has to be predicted.
- Half the heat produced by welding the web-flange connection goes into the web plate and half the heat goes into the flange plate, independent of the different thickness.
- Due to radiation and melting of the electrode, 15% to 35% of the produced heat is lost. For the calculation a constant value for heat input of 85% of the produced heat is assumed.

The total heat produced was estimated using Eq. 3.46

$$Q = AVt \quad (3.46)$$

where 
- $Q =$ total heat generated in watt sec
- $A =$ Current in Amperes
- $V =$ Voltage
- $t =$ time in sec

It has been shown that the general equation for the temperature distribution in a semi-infinite plate due to a point heat source moving with a constant velocity along the edge is

$$T_{ij} = T_{\text{room}} + \frac{Q}{\pi k} e^{-\frac{\lambda v \xi}{h}} \frac{K_0(\lambda v \xi)}{h} \quad (3.47)$$

where
- $T_{ij} =$ Temperature at point $i$, time $j$
- $T_{\text{room}} =$ ambient Temperature
- $Q =$ heat input into the plate
- $k =$ thermal conductivity
- $1/2\lambda = k/s =$ thermal diffusivity
- $s =$ specific heat of the material
- $\xi =$ density of the material
- $v =$ Velocity of the welding electrode
- $\xi = v t_j =$ the distance from the heat source to the cross section of interest at the time increment $t_j$
- $K_0 =$ modified Bessel function of second kind, zero order
\[ r = (r_1^2 + y_1^2)^{1/2} \] where \( y_i \) is the distance from the edge of the plate to the location \( i \)

\[ h = \text{thickness of the plate (assumed 18 mm for the web and flange plate)} \]

Eq. 3.47 represents the steady state of heat transfer. Fig. 3.75 shows an example of the temperature distribution in the flange due to one weld pass. To avoid numerical complication, the time increment for which the temperature is calculated is dependent on the speed of the electrode moving along the plate edge.

Different weld passes are made during the fabrication of a box girder. First, the plates are temporarily assembled with small tack welds. Residual stresses due to these tack welds are not considered here. Then the longitudinal weld inside the box are made manually. Later the automatic submerged arc welds at the outside corners of the box are made. Because of the automatic welding procedure the current, the voltage and the speed are higher. Finally several manual passes are made on the outside over the submerged arc weld. It is assumed that the weld and the base material cool completely before the next pass is made. The weld sequence for a corner connection is shown in Fig. 3.76 the characteristics of each weld pass are indicated in Table 3.7. It is assumed that the total welding at a corner can be simulated with five passes. After finishing all welds in one corner, the welds of the corner diagonally opposite are made.

Due to the limitations in the calculation procedure given in
Ref. 79 for the residual stress distribution the two corners connecting the top flange to the webs had to be made simultaneously. However this limitation does not affect the residual stress distribution, because the temperature at the center line of the flange plate remains constant, see Fig. 3.75. Also the four corners are so far apart, that no interaction of the residual stress results.

The electrodes used to weld the webs to the flanges were E80 ($\sigma_Y \sim 500$ MPa). The submerged arc welds were made using L60 wire. Because they are used to connect material of a higher yield strength, their yield strength will be elevated. A yield strength of about 550 MPa was assumed.

The residual stresses due to welding were calculated using a computer program developed at the University of Missouri\textsuperscript{79}. To apply the computer program the cross section was divided into finite elements; part of the mesh near the corner weld is shown in Fig. 3.76. Each weld pass was simulated by adding an additional element. A coarser mesh was used away from the weld region. There is no need to use a finer mesh near the weld region, because the temperature can not be predicted accurate enough for a finer mesh.

The residual stress is calculated at the end of each weld pass. This distribution is used as the initial condition for the next weld pass. Fig. 3.77 shows the variation of the residual stress in some elements during the production of five weld passes.

Based on the residual stress at the center of each element the total distribution can be derived using linear interpolation. The
orientation is indicated in Fig. 3.80. The stress intensity factor is shown for the dead and live load stress. This $K$-value is almost constant along the circumference because of the ratio $a/c$. Superimposed is the $K$-value resulting from the residual stress. The stress intensity factors are corrected by the finite width and the free surface correction factor.

The stress intensity factor for the smallest crack shape (Shape A), is shown in Fig. 3.82a. This crack is completely embedded in the weld material and the adjacent heat affected zone. As it can be seen from Fig. 3.82a the maximum stress intensity factor is about 55 MPa√m. The stress intensity factor is maximum for an angle of 360°. At this location the crack front is in a zone of high residual stress.

For an increasing size of the ellipse the stress intensity factor increases for a phase angle of 270°. At this location the crack length $a$ in Eq. 3.26 increases and results in an increase of $F_w$. The maximum stress intensity factor for cracks in the corner weld is shown in Fig. 3.83 as a function of the crack length. The crack length is defined in Fig. 3.81.

The crack front moves into the web and the flange plates as soon as the embedded crack becomes too large to be accommodated in the weld area of the web-flange welded connection. The crack front splits up with one front located in the web and the other in the flange plate. Typical crack propagation stages are shown in Fig. 3.84. These crack configuration can be treated as two
completely independent edge cracks. Both crack fronts (the front in the web and the front in the flange) propagate at the same rate, because the cyclic applied stress is uniform. The total crack opening is almost the same because the residual stress distribution in the web is almost identical to the stress distribution in the flange. Hence the crack in the web can be analysed independent of the crack in the flange. The stress intensity factors for the cracks in the web and flange are almost the same.

The analysis of a crack with an irregular crack front and stresses varying over the crack length and plate thickness is made with a procedure derived in Ref. 36, Appendix A. The numerical procedure is based upon the solution of a semi-infinite straight fronted three dimensional crack. To employ this solution with an integration of the stress over the crack surface, the plate containing the crack has to be repeated from $-\infty$ to $+\infty$. In order to obtain the $K$-value for different boundary conditions the solution has to be normalized by $1/\pi$ and integrated from $-\pi/2$ to $\pi/2$ over the entire crack length. The stress intensity factor is shown in Eq. 3.48\textsuperscript{36}.

$$K \sim \frac{1}{\pi} \int_{-\pi/2}^{\pi/2} \int_{0}^{a} \sigma(a, \theta) \, d\theta \, da$$

Eq. 3.48 was evaluated numerically. For the integration the crack surface was devided into finite elements, 20 elements in crack direction and 10 elements over the initial plate thickness were used. The stress intensity factor for uniformly applied load was 160
stress distribution shown in Fig. 3.79 results.

After the first welding pass, the corner weld inside of the box had a residual stress as high as the yield point of the material. Subsequent welds reduced the stress to about $1/2 \sigma_Y$ (see Fig. 3.77). Compression stresses equal to $1/2 \sigma_Y$ are present about 4 cm away from the corners in the web and flange. The final distribution of the residual stress in the middle of the plates is shown in Fig. 3.79.

Residual stress distributions depend also on the support conditions during welding and the cooling procedure. This effect is not investigated here.

3.7.3 Analysis of the Cracks at the Welded Corners

The cracks in the corner welds are of different shapes and sizes. Several shapes that were evaluated are shown in Fig. 3.80. It was assumed that the cracks can be analyzed as elliptical cracks circumscribing the flaw.

The total stress intensity factor is calculated using the principle of superposition. The $K$-value due to the uniformly applied dead and live load stress is calculated using Eq. 3.3 and the crack shape correction factor given in Eq. 2.25.

The $K$-value due to the residual stress shown in Fig. 3.79 was estimated using the procedure shown in Section 3.3.5, Eq. 3.25. Due to the principle of superposition of the dead load, live load
and residual stresses, stresses as high as 850 MPa result. The yield strength of the material is about 860 MPa.

As the cracks are in the material of limited thickness, the K-factor has to be adjusted by the finite width and free surface correction factors.

All the cracks were close to the free surface. The material between the crack front and the weld surface is so thin, that it was easily ground away. This ligament is so small that it plastically deforms without initiating a brittle fracture. The dead load transmitted through this ligament is so small that it can be neglected. Instead of an internal elliptical crack, a U-shaped surface crack results.

Some of the larger cracks penetrated through the free surface and were visible without grinding. The finite width and free surface correction factors were determined by Eq. 3.26. The width in Eq. 3.26 is the distance between the two weld surfaces as indicated in Fig. 3.81. The crack length a is the distance between the free surface and the crack front.

**Numerical Results**

The geometrical conditions for the crack shapes examined are summarized in Table 3.8. The four crack shapes indicated in Fig. 3.80 were numerically evaluated and the results are summarized in Fig. 3.82 a-d. The stress intensity factor is shown as a function of the phase angle $\phi$. The phase angle is defined in Fig. 2.12 and the
also calculated using this numerical procedure and compared with the closed from solution. A small difference (\( \approx 4\% \)) resulted and was neglected. For the analysis of the crack front in the web it was assumed that the front edge is free to bend and the back surface is restrained from bending. The correction factor that results in the same as for a double edge notched plate with a plate width of half the web width (318 mm).

The stress intensity factor for the uniformly applied stress \( (\sigma_{LL}, \sigma_{DL}) \) was estimated for the closed form solution. The total stress intensity factor was obtained by superposition. The stress intensity factor due to stress applied on the two weld areas outside the plates containing the corner crack were calculated separately and added. The stress over the weld was replaced by a splitting force.

Numerical results were obtained for different crack conditions. The crack stages examined are shown in Fig. 3.84. The resulting K-values as a function of crack length are summarized in Fig. 3.85a. It was found that the major contribution to the total stress intensity factor results from the dead and live load stress of about 300 MPa. The reduction in the K-value due to the compressive residual stress is small. Hence the stress intensity factor is always increasing with increasing crack length. A level of 165 MPa\( \sqrt{m} \) is reached when the crack length is about 57 mm, see Fig. 3.85a.
3.7.4 Fatigue Behavior of the Cracks at the Welded Corner

It is of primary importance to know if fatigue crack propagation under the cyclic applied live load will occur for the crack shapes shown in Fig. 3.80. If fatigue crack growth under the cyclic applied live load stress is likely, then the cracks will increase in size and brittle fracture will eventually result. If the crack geometry and the applied stress are small enough no crack growth will occur. Cracks may be tolerated if the structure's factor of safety is adequate against fracture throughout its service life. In order to establish the reserve against brittle fracture, the dimensions of the largest crack and the fracture toughness of the material has to be known for the lowest anticipated service temperature.

The cracks at the intersection of the web and flange are in a zone of high tensile residual stresses. The R-ratio is close to unity which decreases the threshold stress intensity range to a minimum value of 2.75 MPa√m. The usual flaws in the weld metal cold (i.e. pores and slags) have contributed to the formation of transverse cold cracks in the weld. These cracks are irregular in shape so that locally a higher stress intensity factor may exist. The crack tip is very sharp, because the cracks were caused by cold cracking.

The cyclic applied equivalent Miner stress range was approximated as 14 MPa√m for axial stress and bending. For the crack initiation, the maximum possible stress range under service loads
has to be considered. The maximum stress range due to the traffic load was assumed to be 28 MPa.

The stress intensity range for crack shapes A through D are summarized in Table 3.9. It can be seen that the stress intensity range is larger than $\Delta K_{Th}$ for all the crack shapes. Crack shape B, C, and D propagate even under the equivalent Miner stress range of 14 MPa. Fatigue crack propagation is likely for all these cracks. About $11.3 \times 10^6$ load cycles are needed for crack shape C to propagate through the weld area and penetrate through the inside weld. As demonstrated in Fig. 3.89a, the increase in the total stress intensity factor is rapid as the crack approaches the back free surface. The remaining ligament is very small when $K$ exceeds 100 MPa$\sqrt{\text{m}}$. The ligament may deform plastically providing the surface layer has increased toughness. A material with a poor fracture toughness of 55 MPa$\sqrt{\text{m}}$ will likely fracture during this stage. If the material is reasonably good there is a chance that instability will not develop. The stress intensity factor and the fatigue cycles as a function of crack length for crack fronts in the web and the flange are shown in Fig. 3.85b. Also here an extremely slow crack propagation can be observed.

The crack propagation is based on the principles of fatigue crack growth. In this case other mechanisms such as stress corrosion cracking and corrosion assisted fatigue crack growth may increase the crack size and growth rate.
3.7.5 Influence of the Fracture Toughness on the Fatigue Resistance

Cracks starting in the corner weld exhibit three different propagation stages. During the first stage the crack is completely embedded in the connection of web and flange. The stress intensity factor and the fatigue resistance during this stage are shown in Fig. 3.83. The second stage is the transition phase and during the third stage one crack front is in a web plate and the second in a flange plate. The fatigue resistance and the change in stress intensity factor are shown in Fig. 3.85. No estimates of the fatigue life during the transition phase was made. If it is assumed that fatigue crack growth is initiated by a crack described by shape B then only a few load cycles are needed to shapen the crack tip and brittle fracture results if a low fracture toughness of 55 MPa√m is assumed.

For materials with higher fracture toughness, no brittle fracture will occur as long as the crack is embedded in the web-flange connection. For crack sizes corresponding to a stress intensity larger than 100 MPa√m the remaining ligament is so small that it may plastically deform. However, the risk of brittle fracture is great during this phase of crack growth. A crack with a shape slightly larger than shape III provides a K-value of 110 MPa√m, 84 x 10^6 cycles at 14 MPa are required to propagate a crack from shape B to this size. An additional 40 x 10^6 load cycles are required to propagate to a crack size which results in a K-value of 165 MPa√m.
The fatigue lives corresponding to the different fracture toughness levels are indicated in Table 3.10.
4. EFFECT OF PEENING ON THE FATIGUE LIFE

4.1 Introduction and Research Approach

4.1.1 Problem Statement

Fatigue cracks which develop at the end weld of cover plates and at welds connecting stiffeners to flanges, originate at the weld toe and propagate into the flange. The location of a fatigue crack at a cover plate end is shown in schematic in Fig. 2.9. A picture of a small fatigue crack (0.75 mm deep) is shown in Fig. 4.1. This crack was detected in the flange of a full size cover-plated beam tested under research project NCHRP 12-15(12), see Ref. 23. It can be seen in Fig. 4.1 that the crack initially penetrated into the flange in the heat affected zone. This is a zone of high residual tensile stress as well. The residual stress distribution due to the cover plate end weld was summarized in Fig. 3.55. The high residual tensile stresses decrease the threshold stress intensity range so that crack initiation and growth can occur at relatively low applied stress ranges.

Different methods can be employed to increase fatigue life or prevent fatigue cracks from initiating without reducing the stress range or drastically changing the geometry of the detail. One method very often used in the design and manufacturing of machine parts is the introduction of residual compressive stresses at the point where cracks develop. The residual compressive stresses are known to reduce the effective stress intensity range at the location
where cracks will form and also decrease the R-ratio. The decreased R-ratio increases the threshold stress intensity range, see Eq. 2.14.

Residual compressive stresses may be introduced in different ways, eg., by a reversed applied load, by an over load, or by peening. The peening method is investigated in this section as laboratory studies are available on fatigue damaged details treated by peening.

It is sufficient to treat only a small area on girders reinforced with cover plates in order to improve the fatigue life.

4.1.2 Methods of Peening

4.1.2.1 Shot Peening

Residual stresses can be introduced by shot peening. Shot peening has been successfully applied to various machinery parts in order to improve the fatigue behavior over the last 40 years [85]. In shot peening the surface of the machinery part is bombarded with small round hard particles. Every particle acts like a tiny peening hammer. On the treated surface a small layer of compressive residual stress is formed. The residual stresses prevent fatigue cracks from initiating [89]. The thickness of the layer of compressive stresses depends on the material and the peening conditions. A method to estimate the depth of the layer of compressive stresses was developed by J.O. Almen [85]. The thickness is related to the deformation of a standard strip which was peened. The thickness of compression stress is between 0.05 mm and 0.5 mm [84]. The magnitude
of the compressive residual stresses is about half the yield strength depending on the material and the peening procedure.

The shot peening method is used to extend the fatigue life of springs, shafts, axles etc. The parts are peened locally or on the full surface.

4.1.2.2 Air Hammer Peening

A small region can also be peened with an air hammer. The weld toe is air hammer peened until the zone exhibits plastic deformation. No measurements are available which relate the peening intensity (zone of compressive stresses and surface deformations) to the air pressure, tool geometry, and peening time. Qualitative statements have been employed to describe the peening intensity. Terms like "Peening was continued until the crack was no longer visible" or "Peening was continued until the weld toe became smooth" are used to describe the peened condition.

Air hammer peening of weld toes at cover plate end welds was found to be an effective way of prolonging the fatigue life of cover-plated beams. It was found, that the fatigue strength is improved by at least one fatigue design category if the minimum stress was low or the peening carried out with the dead load in place. Peening was successfully applied to full size cover-plated beams in the laboratory. Cover plate ends at the Yellow Mill Pond Bridge were also peened in 1976 in order to repair fatigue damage and to improve their fatigue behavior. No longterm results are available
on the effectiveness of peening the Yellow Mill Pond Bridge.

In addition to residual compression stresses which are introduced by peening, the material is plastically deformed and cold worked so that small fatigue cracks are altered.

4.1.2.3 Other Peening Methods

The surface of a machinery part may also be peened using a bundle of wires. Each wire acts like a small peening hammer. Peening is continued until all the surface is treated.

Peening can also be performed with flap brushes which are rotated with a hand held tool. A wheel equipped with flexible flaps carrying the shot particles is rotated against the work piece. The rotation causes the shot particles to impact against the surface. Penetration depths of the residual compression zone between 0.02 mm and 0.38 mm can be achieved, depending on the material and the tool.

Shot peening and flap wheel peening introduce compressive residual stresses over broad surface areas. Peening has to be done in very localized areas in order to prevent cracks from growing at the cover plate end welds. In addition the depth of the residual stress field has to be large, in order to arrest small cracks.

4.1.3 Research Approach

Experimental data on the fatigue behavior of peened full size
cover-plated beams is available. From the experiments reported in Ref. 23, recommendations were derived giving the maximum crack sizes which can be successfully retrofitted by peening. The fatigue strength of peened full size cover plate beams was also determined during these experiments. However, no mathematical model is available to predict the fatigue behavior of a peened cover-plated beam detail.

The effect of air hammer peening depends on various parameters such as the tool size, the air pressure, the peening time and on uncontrolled parameters such as the operator. Several cover plate ends were peened under controlled conditions and the residual stress field was estimated. From the residual stress field and the crack propagation models reviewed earlier, the influence of peening was evaluated. These results were compared with the fatigue behavior of the peened full size cover-plated beams.

4.2 Experiments

4.2.1 Peening Tool

The same peening tool was used for these experiments as was used for the peening of the weld toes described in Ref. 23. The peening was performed with a handheld Ingersoll Rand Model 1940 pneumatic air hammer operated at different air pressures. The air hammer has a piston diameter of 29 mm and a stroke of 25 mm.

The peening tool is shown in Fig. 4.2; it has a radius of 19 mm
about one axis and a 3 mm radius about the second axis. All the edges of the tool are ground smooth.

The peening tool is hand held at an angle between 60° to 70° with respect to the flange. The angle is also indicated in Fig. 4.2.

4.2.2 Experiment Design

The controlled variables for a given peening tool are the air pressure and peening time. The peening tool is usually moved back and forth along the weld toe to obtain a smooth peened surface. The speed with which the tool is moved is not constant and depends on the operator and the toughness of the weld. For these experiments the operator was instructed to maintain a constant speed. The peening time is expressed in terms of the number of peening passes. A peening speed of 3.6 mm/sec was measured and kept constant throughout the experiments.

The laboratory tests reported in Ref. 23 were peened at an air pressure of 0.17 N/mm². During the peening it was observed that as soon as the valve for the air hammer was opened, the air pressure dropped between 0.03 N/mm² to 0.07 N/mm². The air pressures reported here are the static air pressures.

By varying the air pressure and the number of peening passes 6 different intensities of peening could be obtained. The controlled variables and the experiment design are summarized in Table 4.1. At the lowest air pressure not much surface deformation was observed and it was difficult to operate the tool at the
highest air pressure. It was very hard to maintain the direction of the tool along the cover plate end weld at the high air pressure. For comparison purposes 3 peening passes were made on the base plate (away from any welding) with an air pressure of 0.28 N/mm². This reference test was made on specimen G, see Table 4.1. To maintain the peening direction the air hammer was held perpendicular to the plate surface.

All peening was done in the laboratory; the test beam was positioned so that it was easily accessible for peening.

4.2.3 Specimen Preparation and Measurement Techniques

Peening was carried out on a previously tested beam from NCHRP Project 12-15(2)²⁹. The dimensions of the beam and cover plate correspond to the dimensions given in Table 2.1 (Beam B16, see Ref. 23, was used). The rolled section and the cover plate were A36 steel with a dynamic yield strength of 287 MPa and a tensile strength of 424 MPa according to the mill report. Calculations are also made for A588 Steel. The properties of the A588 steel reported in Ref. 23 were employed; the yield strength is 400 MPa and the tensile strength 547 MPa.

About 10 cm long pieces along the longitudinal cover plate welds were peened for each given specimen indicated in Table 4.1. For the preliminary study peening was done along a cover plate end weld. All other measurements and peening was done along the longitudinal weld of the cover-plated beam. The peened cover plate was
in the tension region during the fatigue tests. The toe of the cover plate longitudinal weld is much smoother than the end weld. The cover plate and welds were made manually; the cover plate longitudinal welds were made using an automatic submerged arc weld process.

Test specimens were saw cut from the middle region of the peened strip for further study. A peened cover plate end weld with the location of the removed specimens is shown in Fig. 4.3. Specimens about 10 mm x 15 mm x 6 mm were saw cut from the peened region and mounted for further investigation in bakelite or translucite.

The surface of the specimen was polished in different steps up to a minimum polishing powder of 0.3 microns grain diameter. An example of a mounted specimen is shown in Fig. 4.4. To make the grain structure of the metal visible the surface was etched using a 1% to 2% Nital solution (2 parts HNO₃, 98 parts alcohol). On the specimen shown in Fig. 4.4 the surface deformation from the peening, the welded zone and the heat affected zone are visible.

The prepared specimens were investigated with a stereo microscope at magnifications between 40X and 140X. The measurements were made on photographs taken with a Zeiss Axiomat using Polaroid Film (102 mm x 127 mm).

The surface deformations of the peened region were measured using a dial gage for measurements of 0.000 1 inch (≈ 0.0025 mm). The set-up for the measurements is shown schematically in Fig. 4.5.

173
By moving the specimen on a stable steel plate the maximum surface deformation was measured. A reference line for these measurements was established by measuring the surface outside the peened region at 3 mm intervals. Several measurements at different locations were taken across the peened region.

To verify the depth of the plastically deformed zone due to the peening, a microhardness survey was also carried out on different specimens. The specimens mounted in bakelite were tested in a microhardness tester with an applied load of 100 grams. The dimensions of the indentation was measured and converted to Knoop Hardness numbers. The indenter has a longitudinal angle of 127° 30' and a transverse angle of 130°. By measuring the length of the longer diagonal with a microscope the area of the indentation could be calculated. Three indentations of the hardness study are shown in Fig. 4.6 at a magnification of 700X. The distance between two measurements is 0.05 mm in the peened zone and 0.1 mm or 0.2 mm in the zone away from the peened region. Several measurements on the same specimen were carried out perpendicular to the peened surface.

4.2.4 Results

4.2.4.1 Visual Inspection

A visual inspection of the peened region indicated that the surface becomes smoother by applying more peening passes. Also the peened surface is smoother when peened at a low air pressure than it is when the surface is peened at higher air pressure. This
results because at the lower air pressure the tool can be guided more easily than at higher air pressures. Fig. 4.7a, b,c show three peened regions. Specimen A shows a smooth surface when peened at 0.21 N/mm². At higher air pressures ripples appear as shown in Fig. 4.7b. These ripples are smoothed out as more peening passes are applied (see Fig. 4.7c).

The peened region also becomes wider as the air pressure increases and as the number of passes increases. It is not possible to guide the tool on the same line, so that more peening passes result in a larger deformed zone. It is easier to guide the tool at low air pressure. No quantitative measurements of the width of the peened region and the ripples were made.

4.2.4.2 Surface Deformations

The surface deformation was measured as shown in Fig. 4.5. The results of these measurements are shown in Table 4.2 and Figures 4.9a and 4.10a. About 5 measurements were made on each specimen. Table 4.2 shows the average value (\(\bar{x}\)) and the average value plus/minus one standard deviation (\(\bar{x} \pm s\)).

Fig. 4.9a indicates that the surface deformation does not increase with increasing air pressure once a limit is attained. At the highest air pressure the standard deviation increases as it is more difficult to operate the tool at high pressure. More increase of surface penetration can be obtained by increasing the number of peening passes.
4.2.4.3 Microscopic Inspection

Figures 8a-g show the polished and etched surfaces of the specimens A through G. Indicated in the figures is the depth of plastically deformed grains. Also visible is the heat affected zone. All the pictures are at 80X magnification. Unfortunately the grain structure is not as clearly visible in the heat affected zone as it is in the base metal. Also the edge of the specimen was not clearly visible. Polishing rounded the edges slightly and reduced the sharpness of the photograph. The depth of penetration was measured on one specimen for each degree of peening.

The depth of plastically deformed grains are given in Table 4.2 and in graphical form in Fig. 4.9b and 4.10b. As can be seen from Fig. 4.9b, the depth of deformed grains increases with increasing air pressure. The depth of penetration for 0.21 N/mm$^2$ and 0.28 N/mm$^2$ pressure is almost the same. This is because only one measurement was made on specimens A & B.

An increasing number of peening passes increases the depth of the plastically deformed zone. Fig. 4.10b suggests that once a level between 0.4 mm and 0.5 mm is attained, the additional peening passes do not increase the zone of deformations further. In Fig. 4.8g the deformed grain structure for specimen G is shown. The depth of the deformed grains is 0.58 mm and the surface deformation is 0.79 mm. The specimen was peened with the same intensity as specimen B which had 3 passes at 0.28 N/mm$^2$. However the deformation are much greater for specimen G than for specimen B. This
resulted because the air hammer could be held perpendicular to the plate surface and the hardness of the weld material was greater than the hardness of the base plate.

4.2.4.4 Hardness Study

The Knopp Hardness was measured on specimen G, G₁ and Specimen D. Specimen G₁ was made at the same peening intensity as specimen G on a different cross section. The peening of specimens G and G₁ was on the base plate alone. The results from the micro-hardness studies are shown in Fig. 4.11, 4.12 and 4.13. The hardness in Fig. 4.11a, 4.12a and 4.13a are the measured hardnesses. To eliminate the measurement errors and the local variation of the hardness due to the local inhomogeneity of the material, averages were calculated and plotted. Fig. 4.11b, 4.12b and 4.13b show the average of three adjacent measurements which were calculated as

\[ KN_i = \frac{(KN_{i-1} + KN_i + KN_{i+1})}{3} \]  

(4.1)

KN is the Knoop hardness. Five measurements were included in the average shown in Figs. 4.11c, 4.12c and 4.13c and seven measurements in the plots given in Figs. 4.11d, 4.12d and 4.13d.

Figs. 4.11, 4.12 and 4.13 show the hardness of base plate with increasing depth, the hardness in the peened region and the depth of the plastically deformed grains. Fig. 4.11 shows the hardness of specimen G. For the base plate about 140 points were measured. The measurements 1 and 2 of the peened region have about 200 points.
The measurements in Fig. 4.11 suggest that the peening has an influence well beyond the zone of the visible deformed grains. The hardness of the base plate is about the same as the hardness measured below the peened zone at a depth of 4 times the thickness of the plastically deformed zone.

The size of the zone influenced by peening is about twice as large as where plastically deformed grains were observed in specimen G₁, (see Fig. 4.12). At this depth the hardness of the base plate is about the same hardness of the measurement through the peened region. At a depth of about 2 mm the hardness of the base plate is higher than at a depth of 6 mm. If the same hardness is assumed for the 2 mm and 6 mm depths than it can be concluded that the zone where the hardness is affected by peening is between 2 and 3 times \( t_D \).

The hardness indentations for the measurements made on specimen D are shown in Fig. 4.14. Also shown in this figure is the depth of the plastically deformed grains. The heat affected zone can also be seen in this figure.

The same observations can be made from the hardness measurements on specimen D. The size of the zone influenced by peening is about 1.5 times as large as the zone where visible grain deformation was observed.

From the hardness measurements it can be concluded that the material is affected by peening well beyond the depth of the visible deformed grains. The depth of the affected zone is between 2 and 4 times the depth of the zone of the deformed grains.
4.2.5 Comparison with Test Beams

Specimens were prepared from three different test beams that were investigated in NCHRP project 12-15(2). Pictures of the specimens at 62.5 x magnification are shown in Figs. 4.15, 4.16 and 4.17. The specimen removed from B1 does not have a fatigue crack intersecting the surface, specimens B9 and B14 have fatigue cracks crossing the polished area. From Fig. 4.17 it is apparent that beam B14 had a fatigue crack when it was peened. The fatigue crack, now at an angle of about 40° to the surface is clearly visible in the deformed area.

The surface deformation and the depth of the plastically deformed grains was measured on each specimen. An average surface deformation of about 0.26 mm was measured. The plastically deformed zone was measured to be about 0.38 mm deep. The depth of the plastic deformed zone varied between 0.12 mm and 0.50 mm. The large variation is caused by the way the peening was done. When a fatigue crack was discovered, peening was continued until the crack was not visible. Fig. 4.9 indicates that the test beams were peened at an equivalent of 3 to 9 passes. The peening speed was not always constant when peening the test beams.

4.3 Analysis

4.3.1 Residual Stress Field

Peening of the weld toe region introduces residual compression
stresses near the surface. The deformed grains (see Figs. 4.8a-g) and the deformed cracks (see Fig. 4.17) indicate that the material is cold worked in this region. Cold working of the material increases the yield strength up to 200%\(^8\). A yield strength of the material in the zone of plastically deformed grains of 1.5 \(\sigma_Y\) can be assumed. Below this zone the material is still affected by the peening. In a zone between \(t_D\) and 3\(t_D\) the hardness of the material is greater than the base plate. In this zone the material is in residual compression. The magnitude of the stresses is \(\sigma_Y\). The transition between the region with 1.5 \(\sigma_Y\) and \(\sigma_Y\) is gradual. A transition zone 20% the size of the \(\sigma_Y\) zone was assumed. The stress distribution is shown in Fig. 4.18. The residual compression stresses are in equilibrium with residual tensile stresses below the peened region. It was assumed that the magnitude of the residual tensile stress is half the yield strength.

The residual stresses due to welding of the cover plate on the beam are eliminated by peening near the flange surface. The total residual stress distribution in the peened region is shown in Fig. 4.18.

4.3.2 Analytical Model

It was observed earlier that the crack growth rate is a function of the stress intensity range. The stress intensity range is given by
\[ \Delta K = K_{\text{Max}} - K_{\text{Min}} \]  

(see Eq. 2.2)

It was also shown that the crack growth rate is independent of the mean stress. This statement is valid as long as the minimum stress intensity factor \( K_{\text{Min}} \) is greater than zero. A minimum stress intensity factor greater than zero means that the crack is always open and that the crack surfaces are never in contact. When \( K_{\text{Min}} \) is less than zero, the crack surfaces may come in contact for some time during a stress cycle and prevents the entire stress intensity range from contributing to crack propagation. For the fatigue life calculation only the stress range of the separated crack surfaces needs to be taken into account.

Different cases of stress intensity range are shown in Fig. 4.19. For a minimum stress intensity factor \( K_{\text{Min}} \) greater than zero, the effective stress intensity range is calculated using Eq. 2.2. This case is indicated as Case A in Fig. 4.19. Case B shows the effective stress intensity range when \( K_{\text{Min}} \) is less than zero and \( K_{\text{Max}} \) is larger than zero. The effective stress intensity range is equal to \( K_{\text{Max}} \). In case C both \( K_{\text{Min}} \) and \( K_{\text{Max}} \) are less than zero. The crack is always closed and the effective stress intensity range is zero. The effective stress intensity range calculation is summarized in Eq. 4.2.

\[ K_{\text{Min}} > 0, \quad \Delta K_{\text{eff}} = K_{\text{Max}} - K_{\text{Min}} \quad (4.2a) \]

\[ K_{\text{Min}} < 0, K_{\text{Max}} > 0, \quad \Delta K_{\text{eff}} = K_{\text{Max}} \quad (4.2b) \]
Fatigue life calculations were made using the effective stress intensity range.

The threshold stress intensity range is also affected by peening. The R-ratio is less than zero in the peened region and a threshold stress intensity range of 6.6 MPa√m results for this condition (see Eq. 2.14).

Peening the surface of the flange cold works the material at the crack location and the crack shape is changed in this layer. A crack that is peened has to be considered as a center through crack. The cold worked ligament is so small that no load will be transferred and it can be neglected. The stress intensity range for a peened crack was estimated using the same models that were used for the unpeened semielliptical surface crack.

The K-value due to the residual stress field introduced by peening produces a closing force on the crack surfaces. The stress intensity factor for a crack in an infinite plate due to a stress field varying along the crack length can be calculated using the relationships shown in Fig. 3.10b. The K-value is obtained by replacing P by \( \sigma \)a and by integrating over the crack length. For the stress distribution shown in Fig. 4.18, a numerical procedure was employed to obtain the stress intensity factor. This K-value has to be corrected for conditions at the structural detail. It has to be adjusted by the finite width correction factor, the free
surface correction factor and the crack shape correction factor. The stress gradient correction factor was taken into account through the numerical calculation of the K-value. \( F_w, F_s, \) and \( F_e \) are the same as for the structural detail.

Fatigue life calculations for cover plated beams were made using the procedures given in Chapter 2. The maximum and minimum stress intensity factors were calculated using Eq. 4.3.

\[
K_{\text{Max}}, K_{\text{Min}} = K_{\text{peen}} + \Delta K
\]  

(4.3)

The effective stress intensity range for the fatigue life calculation is determined according to Eq. 4.2.

4.3.3 Fatigue Life Estimates

Two problems are of interest when determining the fatigue resistance of peened beams: 1) What is the maximum crack size which can be successfully arrested by peening so that further crack growth is prevented and 2) What is the remaining fatigue life for a peened beam which contains a larger crack or is subjected to a larger stress range which offsets the beneficial effects of peening. The peening intensity was assumed to provide a depth of plastically deformed grains equal to 0.5 mm with the surface deformation equal to 0.3 mm. The total depth of the peened zone was estimated from the hardness study.

The total crack depth which can be successfully arrested by peening depends on the depth of the residual stress zone, the yield
strength of the material, the applied stress range and the minimum stress level. The influence of each of these parameters was investigated.

The maximum crack depth which can be successfully peened is the sum of the physical crack length \( a_{\text{Th}} \) for which no further crack growth occurs and the surface deformation. The surface deformation was observed to be between 0.2 mm and 0.3 mm. For these calculations a value of 0.3 mm was assumed. The maximum crack size which can be successfully repaired is schematically shown in Fig. 4.20.

4.3.3.1 Influence of Depth of the Peened Zone on \( a_{\text{Th}} \)

The hardness measurements made on specimens \( G, G, \) and \( D \) suggest that the total depth of the residual compression stress zone due to peening is between 2 and 4 times the depth of the plastically deformed grains. Fig. 4.21 shows the \( K \)-values corresponding to the peening residual stresses for a depth of penetration of \( t_D \), \( 2 t_D \), \( 3 t_D \). The \( K \)-value decreases until the crack tip enters a region of residual tensile stress which depends on the depth of penetration \( n t_D \). As soon as the crack tip enters the zone of residual tensile stress the \( K \)-value increases and eventually becomes positive.

The stress intensity range corresponding to an applied stress range of 41 MPa for a crack at the end of a cover plate weld is also shown in Fig. 4.21. The effective stress intensity range \( \Delta K_{\text{eff}} \) was evaluated from superposition using the relationships provided by
Eq. 4.2. The values of $\Delta K_{\text{eff}}$ are plotted as a solid line in Fig. 4.21. When $t$ equals $t_D$, $\Delta K_{\text{eff}}$ exceeds $\Delta K_{\text{Th}}$ for crack sizes larger than 2.3 mm. This crack depth is marked as $a_{\text{Th}}$ in Fig. 4.21. If deeper peening penetration is provided and $t$ equals 2 times $t_D$, the threshold crack size $a_{\text{Th}}$ is 3.3 mm. This increases to 4.3 mm for $t$ equals 3 times $t_D$.

The maximum crack size which can be successfully arrested depends to a large extent on the zone of residual compression stresses. The value of $a_{\text{Th}}$ can vary between 2.3 mm and 4.3 mm with a mean value of 3.3 mm when $\Delta \sigma$ equals 41 MPa. The results suggest that a fatigue crack up to 3.6 mm deep ($a_{\text{Th}}$ plus surface deformation) can be arrested in full size cover plated beams if the stress range does not exceed 41 MPa.

4.3.3.2 Influence of the Stress Range on $a_{\text{Th}}$

The effective stress intensity range $\Delta K_{\text{eff}}$ is plotted in Fig. 4.22 for stress ranges of 28 MPa, 41 MPa and 55 MPa. The total thickness of the compressive stress layer was taken as 3 times $t_D$. This shows that the effective stress range equals the applied stress range for crack depths larger than 3.5 mm. This crack depth is designated as $a_r$ in Fig. 4.22. Crack lengths larger than 3.5 mm result in a minimum $K$-value larger than zero. Hence, the $R$-ratio increases for increasing crack length and the threshold stress intensity range decreases. The stress intensity threshold range depends on the minimum and maximum stress intensity factors. Each stress range
produces a different threshold stress intensity range for crack lengths larger than $a_r$. For crack length larger than $a_r$, $\Delta K_{Th}$ is 6.6 MPa$\sqrt{m}$ as indicated in Fig. 4.22.

$\Delta K_{\text{eff}}$ is always less than 6.6 MPa$\sqrt{m}$ for a stress range of 28 MPa provided the crack length is less than 3.6 mm. $\Delta K$ is larger than $\Delta K_{Th}$ for crack length less than $a_r$ when the stress range is greater than 33 MPa. The threshold crack length, $a_{Th}$, is 3.2 mm for $\Delta \sigma = 41$ MPa and 2.9 mm for $\Delta \sigma = 55$ MPa. The maximum crack length for which no fatigue crack growth occurs is summarized in Table 4.3 for these stress range conditions. The values in Table 4.3 do not include the surface deformations.

4.3.3.3 Influence of the Yield Strength on $a_{Th}$

Fig. 4.23 shows the stress intensity factors after peening for two different types of steels that have been peened at the weld toe. The yield strength for the A36 steel is 287 MPa and 400 MPa for the A588 steel. The threshold crack length, $a_{Th}$, for a stress range of 42 MPa is also indicated for both types of steel. The difference is very small (A588: $a_{Th} = 3.3$ mm, A36: $a_{Th} = 3.2$ mm). This results because the depth of the compression residual stress field is not much different and the K-value from peening is equal to zero at the same crack length. When the stress range exceeds 42 MPa, the influence of the yield strength is more pronounced.
4.3.3.4 Influence of the Minimum Stress on $a_{Th}$

When dead load is applied to a peened detail, the applied load decreases the magnitude of the residual compressive stress. The effect of various levels of minimum stress is summarized in Fig. 4.24. The effective stress intensity range $\Delta K_{eff}$ is less than the applied stress intensity range would be without peening. The peened surface layer is still in residual compression because the tensile minimum stress is smaller than the compressive stress introduced by peening. The beneficial compressive stress is reduced with higher minimum stress. Increasing minimum stress decreases the magnitude of $a_{Th}$. Fig. 4.24 shows the effective K-value for different minimum stresses as a function of the crack length.

4.3.4 Remaining Fatigue Life

The additional fatigue life for full size cover-plated beams which were peened with crack sizes larger than $a_{Th}$ was also estimated. The results are shown in Table 4.5.

Fig. 4.22 shows the effective stress intensity range for different stress ranges as a function of the crack length. The fatigue life is obtained by using $\Delta K_{eff}$ and integrating the Paris Law.

Fig. 4.22 indicates that $a_{Th}$ is larger than $a_r$ for stress ranges less than 33 MPa. Peened beams with cracks slightly larger than $a_{Th}$ will therefore show no difference in their fatigue behavior when compared to unpeened beams with the same crack length.

187
at stress ranges less than 33 MPa.

For stress ranges larger than 33 MPa, $a_{Th}$ is smaller than $a_r$. Fatigue life estimates have been made using $\Delta K_{eff}$ between the limits $a_{Th}$ and $a_r$. Within these limits $\Delta K$ is less than $\Delta K$ for the unpeened beams. Fatigue lives for peened beams containing cracks with lengths between $a_{Th}$ and $a_r$ are longer than the fatigue lives of unpeened beams containing the same crack length. This results because $\Delta K_{eff}$ is less than $\Delta K$. The fatigue behavior for beams with cracks larger than $a_r$ is the same for peened or unpeened beams because $\Delta K_{eff}$ is equal to $\Delta K$.

Table 4.4 compares the fatigue lives at two stress range levels for different initial crack lengths for both the peened and unpeened cover plates. For crack sizes larger than $a_r$, the fatigue resistance is not influenced by peening. The $da/dN - \Delta K$ relationship developed in Chapter 2 for growth rates near $\Delta K_{Th}$ is used for the fatigue life estimates.

The fatigue life for the unpeened beams for crack length between $a_{Th}$ and the flange thickness was set at 100%. Over the same limits of crack length, the fatigue life for peened beams is 102.4% for a stress range of 41 MPa. The difference is larger for increasing stress ranges. It is 115.4% at $\Delta \sigma$ equals 55 MPa.
4.4 Summary and Conclusions

Based on a visual and microscopic inspection and on hardness measurements it can be concluded that the peening intensity is affected by both air pressure and the number of peening passes. An increase in air pressure increases the surface deformation and the depth of plastically deformed grains. Variation in penetration increases with increasing air pressure. Multiple pass peening at a lower air pressure results in more surface penetration and a deeper zone with plastically deformed grains. The measured peening intensities were comparable with the measured peening intensity of test beams from NCHRP project 12-15(2)\textsuperscript{29}. A depth of plastically deformed grains of 0.5 mm and a surface deformation of 0.3 mm were consistently obtained.

The residual compression zone is deeper than the zone of deformed grains. A total depth between 2 and 4 times the depth of deformed grains is in residual compression. The layer near the surface is cold worked which elevates the yield strength.

The evaluation showed that the threshold crack length is affected by the depth of the residual compression zone. The yield strength of the base material was found to have little influence on the threshold crack length. The threshold crack length is affected by the minimum stress and by the magnitude of the stress range. The larger the minimum stress, the smaller the maximum crack length for which no crack growth occurs. For larger stress ranges a\textsubscript{Th} becomes smaller.
Peened beams with a crack length larger than $a_{Th}$ show practically no significant increase in fatigue life when compared with unpeened beams having the same crack length. The peening is either successful and prevents further crack growth or unsuccessful and no fatigue life increase results.

The analytical study agrees well with the observations made during the test of the full size cover-plated beams. Cracks with a depth of about 3 mm could be successfully repaired if the dead load stress was small or applied prior to peening.
5. CONCLUSIONS

The findings and conclusions of this study are based on the theoretical analysis, the case studies and observations made during the experimental work. The influence of fracture toughness on fatigue behavior was investigated for stiffeners and cover-plate beam tests. In addition, six bridge structures which developed cracking were also used to assess the influence of fracture toughness on the crack development. The main conclusions are:

(1) None of the existing relationships accurately predicted the crack growth rate over the full range of expected behavior of bridge steels.

(2) The Paris Law with a crack growth constant of

\[ 1.21 \times 10^{-13} \frac{\text{mm}}{N \text{Cycles}}^{5.5} \]

and a crack growth exponent of 3.0 was found to provide the most reliable prediction of crack growth rate at medium stress intensity range.

(3) The influence of the threshold stress intensity range was taken into account by assuming the crack growth rate was zero for \( \Delta K < \Delta K_{\text{th}} \). For \( \Delta K > \Delta K_{\text{th}} \) the crack growth rate was predicted by the Paris Law.

(4) The influence of accelerated crack growth rates near the fracture toughness of a given material can be predicted by a power function. This permits the crack growth rate to converge to infinity when the maximum...
stress intensity factor approaches the limiting fracture toughness of the material. For most practical purposes, the crack growth rate can be predicted using the Paris Law up to a maximum stress intensity factor equal to the fracture toughness. For $K_{\text{Max}}$ greater than $K_c$, crack instability occurs.

(5) The influence of the fracture toughness on the fatigue behavior of structural details is not a major factor once a reasonable value of fracture toughness (70 MPa√m to 90 MPa√m) is attained. Fatigue life is much more affected by the variation of the initial flaw and the magnitude of the stress range.

(6) Linear elastic fracture mechanics can be used to predict the fatigue and fracture behavior of bridge components. The influence of the restraint (thickness, three dimensional stress conditions) on the fracture toughness needs further investigation in order to predict the fracture behavior of thin unbroken ligaments.

(7) A lower bound threshold stress intensity range is 2.75 MPa√m. This value was used to evaluate crack growth initiation in bridge structures. It takes the high R-ratio due to residual stress into account.

(8) Stress intensity range estimates based on striation markings are not accurate as a result of random variable
loading. However, striation markings can be used to verify the crack growth mechanism.

(9) The fracture behavior of bridge components is not significantly altered by changing the fracture toughness of the material from 110 MPa√m to 165 MPa√m. Fatigue crack growth increases the crack length to that high stress intensity factors and fracture at high material toughness results.

The influence of peening fatigue damaged details was evaluated. This evaluation provided the following observations:

(1) The zone influenced by air-hammer peening is deeper than the zone in which visible grain deformations can be observed. The zone affected by peening and the resulting residual compressive stress field is between two and four times the zone of deformed grains.

(2) The depth affected by peening depends on the air pressure and the peening time (number of passes). The depth affected by peening can be measured from the surface deformation and the zone where the material is cold worked.

(3) The maximum crack depth which can be successfully repaired depends on the peening intensity, the stress range and the minimum stress. The influence of the yield strength of the material can be neglected.
(4) The maximum crack depth which can be repaired by peening was predicted to be about 3 mm. This is in good agreement with the experimental observations made during the testing of full size cover-plated beams.

(5) Cover-plated beams peened after the cracks were larger than the zone affected by peening exhibit no improvement in their fatigue behavior. Their fatigue resistance is the same as unpeened beams containing comparable fatigue cracks.
6. RECOMMENDATIONS FOR FURTHER RESEARCH

The work reported in this dissertation examined the influence of fracture toughness on the fatigue resistance of structural details. The principles of fracture mechanics were used to investigate this parameter. Additional research is needed to provide better models and increase the reliability of the predicted resistance. It is suggested that consideration be given to the following details:

(1) Additional experimental work is needed to verify the influence of fracture toughness on the fatigue behavior. Tests on different details made of steel with low, medium and high fracture toughness need be carried out. A detail suitable for these tests is a detail where the fracture toughness has a relative large influence on the fatigue behavior. One such detail is a box corner weld with a crack starting in the corner weld and growing into the web and flange plates. Among the uncertainties are the residual stress distribution and the adequateness of the model for a typical corner crack.

(2) The residual stress distribution due to fabrication of the structural detail has a significant influence on the fracture behavior. The residual stress distribution at many commonly used details is not well defined. Research is needed to establish the residual stress distribution more accurately and also to assess the influence of variation of the residual stress field on the fatigue
and fracture behavior. Tests of a detail having different residual stress distributions should be carried out to establish the significance of the residual stress variation. Different residual stress distributions may result from different fabrication procedures.

(3) Fractures in bridge structures have often occurred at moderate temperatures. Considerable plastic deformations can develop at the crack tip and linear elastic fracture mechanics is not fully applicable. Often the fracture mode is not under plane strain conditions. Research is needed to account for the large plastic deformations at the crack tip and to establish the degree of constraint conditions.

(4) During slow stable fatigue crack growth, the crack has to be analysed using different fracture mechanics models. These models do not consider the transition conditions. Of particular concern is the transition from a surface crack to a through crack. The back surface correction has to be considered and its influence on the fatigue and fracture resistance during such transitions.

(5) Initial flaw sizes are often of highly irregular shape. Local irregularities in the crack front are common and cannot be readily taken into account by the stress intensity factor models. Research is needed to establish the significance of these conditions.
(6) Striation markings are an indication of fatigue crack growth. The influence of the random stress cycles and variations in the R-ratio on the striation spacing is not known. These relationships need to be investigated further for bridge steels so that an appropriate relationship can be developed between the striation spacing and the stress intensity factor.

(7) Only a few structural details were investigated in this study. Fatigue cracking has been observed in details that were not treated in this dissertation. Fatigue cracks may initiate in lack of fusion areas of groove welds and in other locations. An investigation of these cracks should be carried out.

(8) Peening is one method that can arrest fatigue damage and retrofit cover-plated beams with small cracks. The analysis provided in this report has indicated that multiple peening passes at a given air pressure level can provide optimum resistance to further crack propagation. Details other than cover plate ends may also be repaired by peening. The fatigue behavior of these details should be investigated analytically and experimentally.

(9) Another reliable method to retrofit fatigue damage is the gas tungsten arc remelting (GTA) of the zone where fatigue damage occurs. An analytical model of the GTA
remelting needs to be developed in order to assess the significance of the various factors that contribute to increased resistance.
NOMENCLATURE

\( a \)
- crack size, minor semidiameter of elliptical crack

\( a' \)
- pseudo crack length \((a' = a + r_y)\)

\( a^* \)
- equivalent crack length

\( a_i \)
- initial crack length

\( a_f \)
- final crack length

\( a_r \)
- crack length for which \( \Delta K = \Delta K_{\text{eff}} \)

\( a_{\text{Th}} \)
- crack length at crack growth threshold

\( A \)
- constant in crack growth relationship, electrical current

\( b \)
- plate width; remaining ligament for compact tension or three point bend specimen

\( b_1 \)
- distance defined in Fig. 3.30

\( B \)
- thickness of compact tension or three point bend specimen

\( c \)
- major semidiameter of elliptical crack

\( C \)
- crack growth constant; empirical crack shape correction factor; a constant

\( CF(a) \)
- combined total correction factor for stress intensity

\( CVN \)
- Charpy V-Notch impact test value

\( e \)
- crack eccentricity

\( E \)
- Young's modules of elasticity

\( E(k) \)
- complete elliptical integral of second kind

\( F_c \)
- crack shape correction factor

\( F_{P(a/t)} \)
- stress concentration decay polynomial

\( F_g \)
- stress gradient correction factor

\( F_s \)
- front free surface correction factor

\( F_w \)
- finite width correction factor

199
\( J \) energy release rate per unit crack extension

\( k \) thermal conductivity

\( k^2 \) constant defining the crack shape

\( K \) stress intensity factor

\( K_{lc} \) critical static plane-strain material toughness

\( K_{ld} \) critical dynamic plane-strain material toughness

\( K_I (1 \text{ sec}) \) critical 1 sec loading time plane-strain material toughness

\( K_c \) critical stress intensity factor

\( K_{DL} \) stress intensity factor due to dead load

\( K_{Max} \) maximum stress intensity factor

\( K_{Min} \) minimum stress intensity factor

\( K_{LL} \) stress intensity factor due to live load

\( K_{RS} \) stress intensity factor due to residual stress

\( K_t \) stress concentration factor

\( \Delta K_T \) stress intensity range at transition from elastic to elastic-plastic behavior

\( \Delta K_{T_0} \) stress intensity range at transition from elastic to elastic-plastic behavior, \( R = 0 \)

\( \Delta K \) stress intensity range

\( \Delta K_{eff} \) effective stress intensity range

\( \Delta K_{Th} \) stress intensity range at crack growth threshold

\( \Delta K_{Th^*} \) stress intensity range at crack growth threshold for \( R = 0 \)

\( K_N \) Knopp hardness

\( K_o \) modified Bessel function of the second kind, zero order

\( \lambda \) distance defined in Fig. 3.31

\( \log \) logarithm to base 10

\( n \) crack growth exponent
N  fatigue life
P  applied load
P_{\text{Max}}  maximum applied load
P_{\text{Min}}  minimum applied load
q  an auxiliary constant
Q  coefficient for back free surface correction, generated heat
r  radius of circular crack, distance defined in Fig. 3.31
r_Y  radius of plastic zone
R  stress ratio, distance defined in Fig. 3.31
s  standard deviation, specific heat of the material
\Delta s  striation spacing
SCF  maximum stress concentration factor
t  loading time, plate thickness
T  temperature
t_{Cp}  thickness of cover plate
t_d  thickness of zone with plastic deformation
t_{Fl}  thickness of flange
t_G  thickness of gusset
t_{St}  thickness of stiffener
t_W  thickness of web
T_s  temperature shift
v  crack opening at crack mouth, velocity
V  voltage
W  width of compact tension or three point bend specimen
\bar{x}  arithmetic average
\( z \)   weld leg size
\( \alpha \)   proportionality factor
\( \delta \)   crack opening displacement (COD)
\( \xi \)   density of the material
\( \theta \)   angle of ellipse
\( \phi \)   phase angle of ellipse
\( \sigma \)   applied stress
\( \sigma_{DL} \)   stress due to dead load
\( \sigma_{LL} \)   stress due to live load
\( \sigma_{\text{Max}} \)   maximum stress
\( \sigma_{\text{Min}} \)   minimum stress
\( \sigma_{RS} \)   residual stress
\( \sigma_{Y} \)   effective yield stress
\( \sigma_{YD} \)   dynamic yield stress
\( \sigma_{YS} \)   0.2% offset tensile bar yield stress at room temperature with static loading
\( \Delta \sigma \)   stress range
<table>
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<th>$T_{Cp}$ (mm)</th>
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<td>Pook 27</td>
<td>-</td>
<td>0.33</td>
<td>5.1</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.50</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.64</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.75</td>
<td>2.5</td>
</tr>
<tr>
<td>Material (reference)</td>
<td>Number of Data Points</td>
<td>R Stress Ratio</td>
<td>$\Delta K_{Th}$ from Ref. (MPa√m)</td>
</tr>
<tr>
<td>---------------------</td>
<td>-----------------------</td>
<td>----------------</td>
<td>---------------------------------</td>
</tr>
<tr>
<td>18/8 Austenitic Steel</td>
<td>-</td>
<td>0.0</td>
<td>5.9</td>
</tr>
<tr>
<td>Pook 27</td>
<td>-</td>
<td>0.33</td>
<td>5.9</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.62</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.74</td>
<td></td>
</tr>
<tr>
<td>A36 24</td>
<td>9</td>
<td>0.192</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>0.244</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.627</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>0.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.732</td>
<td></td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>0.812</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>0.829</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>0.875</td>
<td></td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>0.877</td>
<td></td>
</tr>
</tbody>
</table>

* Not enough data points for curve fit
### Table 2.3
Fracture Toughness for Bridge Steels After Ref. 9

<table>
<thead>
<tr>
<th>Steel</th>
<th>( K_c ) (MPa√m)*</th>
<th>( K_c ) (MPa√m)**</th>
</tr>
</thead>
<tbody>
<tr>
<td>A36</td>
<td>40.4</td>
<td>104.3</td>
</tr>
<tr>
<td>A588-A</td>
<td>44.2</td>
<td>118.4</td>
</tr>
<tr>
<td>A588-B</td>
<td>45.8</td>
<td>128.6</td>
</tr>
<tr>
<td>A514-E</td>
<td>117.6</td>
<td>182.1</td>
</tr>
<tr>
<td>A514-F</td>
<td>101.9</td>
<td>145.0</td>
</tr>
</tbody>
</table>

\[
K_c = \frac{P \cdot f(a/w)}{B\sqrt{a}}
\]

*\( P = P \) (5% secant offset)  
**\( P = P_{Max} \)
<table>
<thead>
<tr>
<th>Steel</th>
<th>$\Delta K_{Th}$ (MPa/\sqrt{m})</th>
<th>$K_C$ (MPa/\sqrt{m})</th>
<th>R</th>
<th>$C_{\text{Eq. (2.11)}}$ (cycle$^{-1}$)</th>
<th>$C_{\text{Eq. (2.13)}}$ (cycle$^{-1}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A36</td>
<td>6.0</td>
<td>104.4</td>
<td>0.1</td>
<td>8.355</td>
<td>11.144</td>
</tr>
<tr>
<td>A588 A</td>
<td>6.0</td>
<td>118.7</td>
<td>0.1</td>
<td>8.746</td>
<td>11.602</td>
</tr>
<tr>
<td>A588 B</td>
<td>6.0</td>
<td>134.1</td>
<td>0.1</td>
<td>15.666</td>
<td>19.711</td>
</tr>
<tr>
<td>A517 E</td>
<td>6.0</td>
<td>182.4</td>
<td>0.1</td>
<td>8.558</td>
<td>10.596</td>
</tr>
<tr>
<td>A517 F</td>
<td>6.0</td>
<td>145.0</td>
<td>0.1</td>
<td>6.520</td>
<td>8.154</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td>9.6</td>
<td>12.2</td>
</tr>
</tbody>
</table>
Table 2.5
Fatigue Life For Small Size Cover-Plated Beams

\( K_c = 110 \text{ MPa} \sqrt{\text{m}}, a_i = 0.076 \text{ mm} \)

<table>
<thead>
<tr>
<th>( \Delta \sigma ) (MPa)</th>
<th>R</th>
<th>( \Delta K_{\text{th}} ) (MPa(\sqrt{\text{m}}))</th>
<th>( N_1 ) (cycles)</th>
<th>( N_2 ) (cycles)</th>
<th>( N ) (cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>0.555</td>
<td>3.5</td>
<td>346x10^3</td>
<td>876x10^3</td>
<td>3.13x10^6</td>
</tr>
<tr>
<td>83</td>
<td>0.455</td>
<td>4.0</td>
<td>136x10^3</td>
<td>235x10^3</td>
<td>897x10^3</td>
</tr>
<tr>
<td>110</td>
<td>0.385</td>
<td>4.4</td>
<td>69x10^3</td>
<td>101x10^3</td>
<td>368x10^3</td>
</tr>
<tr>
<td>138</td>
<td>0.091</td>
<td>6.0</td>
<td>44x10^3</td>
<td>68x10^3</td>
<td>184x10^3</td>
</tr>
</tbody>
</table>
Table 2.6
Fatigue Life for Small Size Cover-Plated Beams

$R = 0.6$, $K_c = 110$ MPa$\sqrt{m}$, $\Delta K_{Th} = 3.3$ MPa$\sqrt{m}$, $a_i = 0.076$ mm

Eq. 2.11

<table>
<thead>
<tr>
<th>$\Delta \sigma$ (MPa)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq. 2.18* C = 1.0</td>
<td>41 7.63x10^6</td>
<td>7.96x10^6</td>
<td>5.49x10^6</td>
<td>829x10^3</td>
<td>2.09x10^6</td>
</tr>
<tr>
<td></td>
<td>55 3.13x10^6</td>
<td>3.33x10^6</td>
<td>2.29x10^6</td>
<td>346x10^3</td>
<td>875x10^3</td>
</tr>
<tr>
<td></td>
<td>83 897x10^3</td>
<td>1.20x10^6</td>
<td>828x10^3</td>
<td>125x10^3</td>
<td>316x10^3</td>
</tr>
<tr>
<td></td>
<td>110 368x10^3</td>
<td>595x10^3</td>
<td>411x10^3</td>
<td>62x10^3</td>
<td>157x10^3</td>
</tr>
<tr>
<td></td>
<td>138 184x10^3</td>
<td>338x10^3</td>
<td>233x10^3</td>
<td>35x10^3</td>
<td>90x10^3</td>
</tr>
</tbody>
</table>

Eq. 2.13

<table>
<thead>
<tr>
<th>$\Delta \sigma$ (MPa)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
<th>$N$ (cycle)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq. 2.18* C = 1.0</td>
<td>41 7.63x10^6</td>
<td>38.3x10^6</td>
<td>12.1x10^6</td>
<td>3.14x10^6</td>
<td>8.37x10^6</td>
</tr>
<tr>
<td></td>
<td>55 3.13x10^6</td>
<td>9.26x10^6</td>
<td>2.93x10^6</td>
<td>759x10^3</td>
<td>2.02x10^6</td>
</tr>
<tr>
<td></td>
<td>83 897x10^3</td>
<td>2.28x10^6</td>
<td>721x10^3</td>
<td>186x10^3</td>
<td>498x10^3</td>
</tr>
<tr>
<td></td>
<td>110 368x10^3</td>
<td>959x10^3</td>
<td>303x10^3</td>
<td>78x10^3</td>
<td>209x10^3</td>
</tr>
<tr>
<td></td>
<td>138 184x10^3</td>
<td>496x10^3</td>
<td>157x10^3</td>
<td>41x10^3</td>
<td>108x10^3</td>
</tr>
</tbody>
</table>

* Test Data
Table 2.7
Fatigue Life For Small Size Cover-Plated Beams

\[ \Delta \sigma = 55 \text{ MPa}, \Delta K = 3.3 \text{ MPa} \sqrt{\text{m}}, R = 0.6, C = 3.16, \text{ Eq. (2.13)} \]

<table>
<thead>
<tr>
<th>Kc (MPa\sqrt{m})</th>
<th>a\text{Max} (mm)</th>
<th>N (Cycle)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>5.0</td>
<td>383x10^3</td>
</tr>
<tr>
<td>110</td>
<td>6.5</td>
<td>404x10^3</td>
</tr>
<tr>
<td>165</td>
<td>7.2</td>
<td>410x10^3</td>
</tr>
</tbody>
</table>

- a = 0.025 mm to 0.076 mm
- a = 0.076 mm to a\text{Max} = f (Kc)

<table>
<thead>
<tr>
<th></th>
<th>2.61x10^6</th>
<th>2.93x10^6</th>
<th>3.02x10^6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total</td>
<td>2.99x10^6</td>
<td>3.33x10^6</td>
<td>3.43x10^6</td>
</tr>
</tbody>
</table>
Table 2.8
Fatigue Life For Full Size Cover-Plated Beams
\( \Delta \sigma = 55 \text{ MPa}, \Delta K_{Th} = 3.3 \text{ MPa}\sqrt{\text{m}}, R = 0.6, \text{ Eq. (2.13)} \)

<table>
<thead>
<tr>
<th>( K_c ) MPa( \sqrt{\text{m}} )</th>
<th>( a_{\text{Max}} ) mm</th>
<th>( N ) (Cycle)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>10.9 mm</td>
<td>122x10³</td>
</tr>
<tr>
<td>110</td>
<td>16.6 mm</td>
<td>130x10³</td>
</tr>
<tr>
<td>165</td>
<td>19.4 mm</td>
<td>133x10³</td>
</tr>
</tbody>
</table>

a = 0.025 mm to 0.076 mm

1.23x10⁶ 1.66x10⁶ 1.78x10⁶

Total
1.35x10⁶ 1.79x10⁶ 1.91x10⁶
## Table 2.9

Fatigue Life For Type 3 Stiffeners

\[ \Delta \sigma = 110 \text{ MPa}, \Delta K_{Th} = 3.3 \text{ MPa}\sqrt{m}, R = 0.6, C = 3.16, \text{ Eq. (2.13)} \]

<table>
<thead>
<tr>
<th>N (Cycle)</th>
<th>( K_C = 55 \text{ MPa}\sqrt{m} )</th>
<th>( K_C = 110 \text{ MPa}\sqrt{m} )</th>
<th>( K_C = 165 \text{ MPa}\sqrt{m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_{Max} = 7.6 \text{ mm} )</td>
<td>( 1.66 \times 10^6 )</td>
<td>( 1.73 \times 10^6 )</td>
<td>( 1.75 \times 10^6 )</td>
</tr>
<tr>
<td>( a = 0.025 \text{ mm to 0.076 mm} )</td>
<td>( 2.37 \times 10^6 )</td>
<td>( 2.76 \times 10^6 )</td>
<td>( 2.87 \times 10^6 )</td>
</tr>
<tr>
<td>( a_{Max} = f(K_C) )</td>
<td>( 4.03 \times 10^6 )</td>
<td>( 4.49 \times 10^6 )</td>
<td>( 4.62 \times 10^6 )</td>
</tr>
</tbody>
</table>

212
Table 2.10
Fatigue Life For Small Size Cover-Plated Beams $\Delta \sigma = 55$ MPa
d$\text{a}/dN$ From Fig. 2.66, $R = 0.6$

<table>
<thead>
<tr>
<th>$K_c$ (MPa$\sqrt{m}$)</th>
<th>$a_{\text{Max}}$ (mm)</th>
<th>$K_c$ (MPa$\sqrt{m}$)</th>
<th>$a_{\text{Max}}$ (mm)</th>
<th>$K_c$ (MPa$\sqrt{m}$)</th>
<th>$a_{\text{Max}}$ (mm)</th>
<th>$K_c = \infty$</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>5.0</td>
<td>110</td>
<td>6.5</td>
<td>165</td>
<td>7.2</td>
<td></td>
</tr>
</tbody>
</table>

$a_{\text{i}} = 0.025$ mm
$\Delta K_T = 17.6$ MPa$\sqrt{m}$

<table>
<thead>
<tr>
<th>$a_{\text{i}}$ (mm)</th>
<th>$\Delta K_T$ (MPa$\sqrt{m}$)</th>
<th>$a_{\text{Max}} = F(K_c)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>16.4x10$^3$</td>
<td>27.1x10$^3$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28.6x10$^3$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>40.2x10$^3$</td>
</tr>
</tbody>
</table>

Total $2.52x10^6$ $2.54x10^6$ $2.54x10^6$ $2.55x10^6$
Table 2.11
Fatigue Life For Full Size Cover-Plated Beams, $\Delta\sigma = 55$ MPa
\(da/dN\) From Fig. 2.66

<table>
<thead>
<tr>
<th>$K_c$</th>
<th>$a_{\text{Max}}$</th>
<th>$\Delta K_T$</th>
<th>$a_{\text{Max}} = f(K_c)$</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>55 MPa(\sqrt{m})</td>
<td>10.9 mm</td>
<td>1.57x10^6</td>
<td>63x10^3</td>
<td>1.64x10^6</td>
</tr>
<tr>
<td>110 MPa(\sqrt{m})</td>
<td>16.6 mm</td>
<td>1.57x10^6</td>
<td>104x10^3</td>
<td>1.68x10^6</td>
</tr>
<tr>
<td>165 MPa(\sqrt{m})</td>
<td>19.4 mm</td>
<td>1.57x10^6</td>
<td>111x10^3</td>
<td>1.69x10^6</td>
</tr>
<tr>
<td>$\infty$</td>
<td>$t_F$</td>
<td>1.57x10^6</td>
<td>215x10^3</td>
<td>1.79x10^6</td>
</tr>
</tbody>
</table>
Table 2.12
Fatigue Life For Type 3 Stiffeners, \( da/dN \) From Fig. 2.66,
\( \Delta \sigma = 110 \) MPa, \( R = 0.6 \)

<table>
<thead>
<tr>
<th>( K_c ) (MPa( \sqrt{\text{m}} ))</th>
<th>( a_{\text{Max}} ) (mm)</th>
<th>( \Delta K_T ) (MPa( \sqrt{\text{m}} ))</th>
<th>( a_{\text{Max}} = F (K_c) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>7.5</td>
<td>2.24 \times 10^6</td>
<td>78 \times 10^3</td>
</tr>
<tr>
<td>110</td>
<td>11.2</td>
<td>2.24 \times 10^6</td>
<td>109 \times 10^3</td>
</tr>
<tr>
<td>165</td>
<td>12.1</td>
<td>2.24 \times 10^6</td>
<td>114 \times 10^3</td>
</tr>
<tr>
<td>( \infty )</td>
<td>( t_F )</td>
<td>2.24 \times 10^6</td>
<td>134 \times 10^3</td>
</tr>
</tbody>
</table>

Total: 2.32 \times 10^6  2.35 \times 10^6  2.36 \times 10^6  2.38 \times 10^6
Table 2.13

Influence Of The Fracture Toughness On The Fatigue Life

Summary

$da/dN$ From Fig. 2.66, $a_i = 0.076$ mm

<table>
<thead>
<tr>
<th>Detail</th>
<th>55[MPa√m]</th>
<th>110[MPa√m]</th>
<th>165[MPa√m]</th>
<th>$\infty$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small Size Cover-Plated Beams</td>
<td>100%</td>
<td>100.4%</td>
<td>100.5%</td>
<td>101.0%</td>
</tr>
<tr>
<td>$\Delta \sigma = 55$ MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Full Size Cover-Plated Beams</td>
<td>100%</td>
<td>102.7%</td>
<td>103.0%</td>
<td>109.7%</td>
</tr>
<tr>
<td>$\Delta \sigma = 55$ MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beams with Type 3 Stiffeners</td>
<td>100%</td>
<td>101.5%</td>
<td>101.7%</td>
<td>102.6%</td>
</tr>
<tr>
<td>$\Delta \sigma = 110$ MPa</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 3.1  AASHTO Notch Toughness Specification for Bridge Steels\textsuperscript{52}

<table>
<thead>
<tr>
<th>Steel</th>
<th>Thickness</th>
<th>CVN Impact Value, J</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>** Zone 1 * **</td>
<td>** Zone 2 * **</td>
</tr>
<tr>
<td>ASTM</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A36</td>
<td>up to 10 cm mechanically fastened</td>
<td>20 at 21°C</td>
</tr>
<tr>
<td>A572</td>
<td>up to 5 cm welded</td>
<td>20 at 21°C</td>
</tr>
<tr>
<td>A440</td>
<td></td>
<td>20 at 21°C</td>
</tr>
<tr>
<td>A441</td>
<td></td>
<td>20 at 21°C</td>
</tr>
<tr>
<td>A242</td>
<td></td>
<td>20 at 21°C</td>
</tr>
<tr>
<td>A588**</td>
<td>up to 10 cm mechanically fastened</td>
<td>20 at 21°C</td>
</tr>
<tr>
<td></td>
<td>up to 5 cm welded</td>
<td>20 at 21°C</td>
</tr>
<tr>
<td></td>
<td>over 5 cm welded</td>
<td>27 at 21°C</td>
</tr>
<tr>
<td>A514</td>
<td>up to 10 cm mechanically fastened</td>
<td>34 at -1°C</td>
</tr>
<tr>
<td></td>
<td>up to 6 cm welded</td>
<td>34 at -1°C</td>
</tr>
<tr>
<td></td>
<td>between 6 cm – 10 cm welded</td>
<td>47 at -1°C</td>
</tr>
</tbody>
</table>

\* Zone 1: Minimum service temperature -18°C and above  
Zone 2: Minimum service temperature from -18°C to -34°C  
Zone 3: Minimum service temperature from -34°C to -51°C  

\*\* If the yield point of the material exceeds 450 MPa, the temperature for the CVN value for acceptability shall be reduced by 8°C for each increment of 69 MPa above 450 MPa.
Table 3.2
Influence of the Fracture Toughness on the Fatigue Life, Quinnipiac River Bridge

<table>
<thead>
<tr>
<th>$K_c$ (MPa√m)</th>
<th>Circular Crack, a (mm)</th>
<th>Through Crack, a (mm)</th>
<th>N (cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>114.3</td>
<td>NA</td>
<td>0</td>
</tr>
<tr>
<td>110</td>
<td>121.7</td>
<td>21.1</td>
<td>3.38x10^6</td>
</tr>
<tr>
<td>143 *</td>
<td>125.4</td>
<td>35.7</td>
<td>3.61x10^6</td>
</tr>
<tr>
<td>165</td>
<td>128.8</td>
<td>47.5</td>
<td>4.40x10^6</td>
</tr>
</tbody>
</table>

* Actual Behavior
Table 3.3
Stress Range Histogram, Glenfield Bridge

<table>
<thead>
<tr>
<th>Max Stress (MPa)</th>
<th>Number of Events</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1.4</td>
<td>322</td>
<td>17.0</td>
</tr>
<tr>
<td>1.4 - 2.9</td>
<td>682</td>
<td>36.0</td>
</tr>
<tr>
<td>2.9 - 4.2</td>
<td>340</td>
<td>18.0</td>
</tr>
<tr>
<td>4.2 - 5.6</td>
<td>137</td>
<td>7.2</td>
</tr>
<tr>
<td>5.6 - 7.0</td>
<td>121</td>
<td>6.4</td>
</tr>
<tr>
<td>7.0 - 8.4</td>
<td>157</td>
<td>8.3</td>
</tr>
<tr>
<td>8.4 - 9.9</td>
<td>110</td>
<td>5.8</td>
</tr>
<tr>
<td>9.9 -11.2</td>
<td>20</td>
<td>1.1</td>
</tr>
<tr>
<td>11.2 -12.6</td>
<td>2</td>
<td>0.1</td>
</tr>
<tr>
<td>12.6 -14.1</td>
<td>1</td>
<td>0.1</td>
</tr>
</tbody>
</table>
Table 3.4
Influence of Fracture Toughness on Fatigue Life of the Lafayette Street Bridge

<table>
<thead>
<tr>
<th>Stress Intensity (MPa√m)</th>
<th>Crack Length * (mm)</th>
<th>Fatigue Life (cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>3.5</td>
<td>1.03x10^6</td>
</tr>
<tr>
<td>110</td>
<td>8.1</td>
<td>2.99x10^6</td>
</tr>
<tr>
<td>140</td>
<td>9.50</td>
<td>3.30x10^6</td>
</tr>
<tr>
<td>165</td>
<td>10.3</td>
<td>3.46x10^6</td>
</tr>
</tbody>
</table>

* Crack Tip penetration into Web.
<table>
<thead>
<tr>
<th>Pier #</th>
<th>$\Delta\sigma$ from Ref. 77 (MPa)</th>
<th>$\Delta\bar{\sigma}$ (MPa)*</th>
<th>Value used for Calculation (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>60.3</td>
<td>26.1 - 41.8</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>24.5</td>
<td>10.6 - 17.0</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>33.6</td>
<td>14.5 - 23.3</td>
<td>20.7</td>
</tr>
</tbody>
</table>

* 0.5 rsp 0.8 times $\Delta\sigma$ with 12% impact
### Table 3.6
Influence of the Fracture Toughness on the Fatigue Life, Dan Ryan Viaduct

<table>
<thead>
<tr>
<th>$K_c$ (MPa√m)</th>
<th>N (cycles)</th>
<th>N (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>$1.36 \times 10^6$</td>
<td>8</td>
</tr>
<tr>
<td>88 **</td>
<td>$1.36 \times 10^6$</td>
<td>8</td>
</tr>
<tr>
<td>110</td>
<td>$1.36 \times 10^6$</td>
<td>8</td>
</tr>
<tr>
<td>165</td>
<td>$1.86 \times 10^6$ *</td>
<td>10.9</td>
</tr>
</tbody>
</table>

* Linear decay function for $F_g$ is assumed

** Actual Behavior
Table 3.7

Weld Characteristics for Corner Welds, Box Girder

<table>
<thead>
<tr>
<th>Weld #</th>
<th>Current (Amp)</th>
<th>Voltage (V)</th>
<th>Speed (mm/sec)</th>
<th>Yield Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>280</td>
<td>28</td>
<td>0.04</td>
<td>550</td>
</tr>
<tr>
<td>2</td>
<td>750</td>
<td>33</td>
<td>8.5</td>
<td>550</td>
</tr>
<tr>
<td>3</td>
<td>280</td>
<td>28</td>
<td>0.04</td>
<td>550</td>
</tr>
<tr>
<td>4</td>
<td>280</td>
<td>28</td>
<td>0.04</td>
<td>550</td>
</tr>
<tr>
<td>5</td>
<td>280</td>
<td>28</td>
<td>0.04</td>
<td>550</td>
</tr>
</tbody>
</table>
Table 3.8
Geometrical Properties of the Investigated Crack Shapes
for Embedded Flaws, Box Girder

<table>
<thead>
<tr>
<th>Crack Shape</th>
<th>Minor Semi-diameter a (mm)</th>
<th>Major Semi-diameter C (mm)</th>
<th>Thickness of remaining ligament * (mm)</th>
<th>( F_e = \frac{1}{E(k)} )</th>
<th>( F_w \cdot F_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>3.7</td>
<td>4.4</td>
<td>0.5</td>
<td>0.716</td>
<td>1.174</td>
</tr>
<tr>
<td>B</td>
<td>4.6</td>
<td>6.2</td>
<td>0.5</td>
<td>0.727</td>
<td>1.267</td>
</tr>
<tr>
<td>C</td>
<td>6.2</td>
<td>7.8</td>
<td>-1.1</td>
<td>0.705</td>
<td>1.432</td>
</tr>
<tr>
<td>D</td>
<td>7.8</td>
<td>9.3</td>
<td>-2.3</td>
<td>0.693</td>
<td>1.953</td>
</tr>
</tbody>
</table>

* The - sign indicates that the ellipse containing the flaw reaches outside the weld surface.
Table 3.9

Fatigue Crack Growth for Cracks at the Welded Corners, Box Girder

<table>
<thead>
<tr>
<th>Crack Shape</th>
<th>$\frac{\Delta K}{\Delta \sigma}(\sqrt{\text{m}})$</th>
<th>$\Delta K_{\text{max}}$ for $\Delta \sigma = 28 \text{ MPa}$ and $\phi = 270^\circ$, (MPa$\sqrt{\text{m}}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.179</td>
<td>5.0</td>
</tr>
<tr>
<td>B</td>
<td>0.221</td>
<td>6.2</td>
</tr>
<tr>
<td>C</td>
<td>0.267</td>
<td>7.5</td>
</tr>
<tr>
<td>D</td>
<td>0.348</td>
<td>9.7</td>
</tr>
</tbody>
</table>
Table 3.10  
Influence of the Fracture Toughness of the Fatigue Behavior, Box Girder

a) **Corner Weld Cracks**

<table>
<thead>
<tr>
<th>$K_C$ (MPa$\sqrt{m}$)</th>
<th>$a$ (mm)</th>
<th>$N$ (Million Cycles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>55</td>
<td>NA</td>
<td>0</td>
</tr>
<tr>
<td>110</td>
<td>25</td>
<td>84</td>
</tr>
<tr>
<td>165</td>
<td>58</td>
<td>124</td>
</tr>
</tbody>
</table>

226
Table 4.1

Experiment Design, Specimen Designation

a) Cover Plate Weld Toe

<table>
<thead>
<tr>
<th>Air Pressure, N/mm²</th>
<th>0.21</th>
<th>0.28</th>
<th>0.34</th>
<th>0.41</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 passes</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>6 passes</td>
<td>E</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9 passes</td>
<td>F</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

b) Base Plate

| 3 passes | G    |
Table 4.2
Deformations After Peening, mm

a) Surface Deformation*  

<table>
<thead>
<tr>
<th>Air Pressure, N/mm²</th>
<th>0.21</th>
<th>0.28</th>
<th>0.34</th>
<th>0.41</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 passes</td>
<td>(0.048)</td>
<td>(0.145)</td>
<td>(0.173)</td>
<td>(0.132)</td>
</tr>
<tr>
<td></td>
<td>0.089</td>
<td>0.180</td>
<td>0.188</td>
<td>0.178</td>
</tr>
<tr>
<td></td>
<td>(0.130)</td>
<td>(0.215)</td>
<td>0.203</td>
<td>(0.224)</td>
</tr>
<tr>
<td>6 passes</td>
<td>(0.173)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.224</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(0.275)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9 passes</td>
<td>(0.229)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.300</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(0.371)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

b) Grain Deformation  

<table>
<thead>
<tr>
<th></th>
<th>0.150</th>
<th>0.150</th>
<th>0.188</th>
<th>0.376</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 passes</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 passes</td>
<td>0.437</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9 passes</td>
<td>0.526</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* \( \bar{x} - s \), \( \bar{x} \): Average  
\( s \): Standard Deviation
Table 4.3
Threshold Crack Length for Peened Cover-plated Beams
A36 Steel, \( t_D = 0.5 \) mm

<table>
<thead>
<tr>
<th>( \Delta \sigma ) (MPa)</th>
<th>( t_D )</th>
<th>( 2t_D )</th>
<th>( 3t_D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>3.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>41</td>
<td>2.3</td>
<td>3.2</td>
<td>4.30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(3.3)*</td>
<td></td>
</tr>
<tr>
<td>55</td>
<td>2.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

in mm

* A588 Steel

\( a_{Th} \) does not include the surface deformation
Table 4.4

Comparison between the Fatigue Life of Peened and Unpeened Cover-plated Beams

\[ \Delta \sigma = 41 \text{ MPa} \quad \Delta \sigma = 55 \text{ MPa} \]

<table>
<thead>
<tr>
<th></th>
<th>( a_i ) to ( a_{\text{Th}} )</th>
<th>( a_{\text{Th}} ) to ( a_r )</th>
<th>( a_r ) to ( a = t_{FL} )</th>
<th>( a_{\text{Th}} ) to ( a_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_i )</td>
<td>( \infty )</td>
<td>( 168 \times 10^3 )</td>
<td>( 1.72 \times 10^6 )</td>
<td>( 1.89 \times 10^6 )</td>
</tr>
<tr>
<td>( a_{\text{Th}} )</td>
<td>( 2.03 \times 10^6 )</td>
<td>( 125 \times 10^3 )</td>
<td>( 1.72 \times 10^6 )</td>
<td>( 1.85 \times 10^6 )</td>
</tr>
<tr>
<td>( a_r )</td>
<td>( \infty )</td>
<td>( 259 \times 10^3 )</td>
<td>( 731 \times 10^3 )</td>
<td>( 990 \times 10^3 )</td>
</tr>
<tr>
<td>( a_{\text{Th}} )</td>
<td>( 790 \times 10^3 )</td>
<td>( 126 \times 10^3 )</td>
<td>( 731 \times 10^3 )</td>
<td>( 858 \times 10^3 )</td>
</tr>
<tr>
<td>( a_i = 0.076 \text{ mm} )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fig. 2.1 Typical Stress Intensity Range-Crack Growth Rate Relationship for Bridge Steel
$K_{C1} \approx K_{IC}$

$K_{C2} > K_{C1}$

$K_{C1} < K_{C3} < K_{C2}$

Fig. 2.2 Crack Growth Stages in a Tension Flange and Resulting Fracture Toughness
Fig. 2.3 Fatigue and Fracture Surface, 36 Beam B54, A588 Steel, W36x230, Failure Temperature -79°C
Fig. 2.4 Fatigue and Fracture Surface, Beam 35, A588 Steel, W36X230, Failure Temperature -101°C
Fig. 2.5 Comparison between McEvily's Relationships (Eq. 2.11 and Eq. 2.13) and Paris Law (Eq. 2.6)
Fig. 2.6 Crack Growth Rate Relationship Developed by Forman\textsuperscript{20}, (Eq. 2.15)
Fig. 2.7 Crack Growth Rate Relationship Developed by Pearson, (Eq. 2.16)
(a) Stiffener

(b) Cover Plate

Fig. 2.9 Crack Location
Mean and 95% Confidence Limits for 95% Survival (stiffener type 3)
Slope: -3.0

Fig. 2.11 Comparison of Fatigue Results for Type 3 Stiffeners from Ref. 12, 48, 49, 50
Fig. 2.12 Definitions of Angles for Crack Shape Correction Factor and Free Surface Correction Factor
Fig. 2.13 Crack Shape Measurements
Fig. 2.14 Fatigue Life of Full Size Cover-plated Beams, Fatigue Life Defined using Fracture Criterion
Fig. 2.15 Fatigue Life for Type 3 Stiffener for Different Crack Shape Relationships, Eq. 2.6, $a_1 = 0.076$ mm
Fig. 2.16 $\frac{da}{dN} - \Delta K$ Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, $0 \leq R < 0.2$
Fig. 2.17  \( \frac{da}{dN} \) vs \( \Delta K \) Measurements near Fatigue Threshold
Stress Intensity Range, Test Results from
Literature, \( 0.2 \leq R < 0.4 \)
Fig. 2.18  da/dN - ΔK Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, 0.4 ≤ R < 0.6
Fig. 2.19  $\frac{da}{dN} - \Delta K$ Measurements near Fatigue Threshold
Stress Intensity Range, Test Results from Literature, $0.6 \leq R < 0.8$
Fig. 2.20 da/dN – ΔK Measurements near Fatigue Threshold Stress Intensity Range, Test Results from Literature, 0.8 ≤ R < 1.0
Fig. 2.21 Fatigue Threshold Stress Intensity Range of Different Steels
Fig. 2.22 Comparison of Fatigue Threshold Stress Intensity Range with lower Bound Relationships
Fig. 2.23 Transition between Zone II and III, $\Delta K_T$ from
Eq. 2.52, A36 Steel$^{32}$, $R = 0$, $\Delta K_T = 45.3$ MPa$\sqrt{m}$
Fig. 2.24 Transition between Zone II and III, $K_T$ from Eq. 2.52,
A36 Steel$^{32}$, $0.33 < R < 0.38$, $\Delta K_T = 29.6$ MPa$\sqrt{m}$
Fig. 2.25 Transition between Zone II and III, $\Delta K_T$ from Eq. 2.52, A36 Steel, $0.09 < R < 0.11$, $\Delta K_T = 41$ MPa$\sqrt{m}$
Fig. 2.26 Curve Fit Eq. 2.11, A36 Steel\(^9\), R = 0.1
Fig. 2.28 Curve Fit Eq. 2.11, A588 A Steel, $R = 0.1$
Fig. 2.30 Curve Fit Eq. 2.11, A517 F Steel, R = 0.1
Fig. 2.33 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, $\Delta K_{th} = f$ (Test), $K_c = 110$ MPa$\sqrt{m}$
Fig. 2.35 Curve Fit, McEvily's Relationship (Eq. 2.11) and Paris Law (Eq. 2.6), $K_C = 110 \text{ MPa} \sqrt{\text{m}}$, $R = 0.1$
Fig. 2.36 Curve Fit, McEvily's Relationship (Eq. 2.11) and Paris Law (Eq. 2.6), $K_C = 165$ MPa $\sqrt{m}$, $R = 0.1$
Fig. 2.37 Curve Fit, McEvily's Relationship (Eq. 2.13) and Paris Law (Eq. 2.6), $K_C = 110$ MPa$\sqrt{m}$, $R = 0.1$. 

- $C = 4.582$
- $C = 6.051$
- $C = 8.257$
Fig. 2.38 \((\pi a)^{1/2}\) - Crack Length, Small Size Cover-plated Beams

\[ b = 3.353 + 1.29a \]

\[ b = 3.549a^{1.133} \]
Fig. 2.39 Minimum Crack Length for Fatigue Crack Propagation, Small Size Cover-plated Beams
Fig. 2.40 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.11, 
$a_i = 0.076$ mm, $\Delta K_{Th} = 3.3$ MPa$\sqrt{m}$, $R = 0.6$
Fig. 2.41: Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13

\[ a_i = 0.076 \text{ mm}, \Delta K_{th} = 3.3 \text{ MPa} \sqrt{\text{m}}, R = 0.6 \]
Fig. 2.42 Influence of Fracture Toughness on Crack Growth Rate, Eq. 2.11, R = 0.1
Fig. 2.43 Influence of Fracture Toughness on Crack Growth Rate, Eq. 2.13, $R = 0.1$

C = 5.46
C = 4.58
ΔK

$\Delta K_{th}$

$da/dN$ (mm/cycles)

$\Delta K$ (MPa m$^{1/2}$)

$10^{-1}$ $10^{-2}$ $10^{-3}$ $10^{-4}$ $10^{-5}$ $10^{-6}$
Fig. 2.44 Crack Growth Rate Predicted with Eq. 2.11, $C = 3.799$, $\Delta K_{th} = 3.3 \text{ MPa}\sqrt{\text{m}}$, $R = 0.1$
Fig. 2.46 Fatigue Crack Growth Rates, Curve Fit with Beam Tests\textsuperscript{11}, Eq. 2.13, $C = 3.16$, $\Delta K_{Th} = 3.3$ MPa$\sqrt{m}$, $R = 0.6$
Fig. 2.47  Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13,
\[ C = 3.16, \Delta K_{th} = 3.3 \, \text{MPa}\sqrt{\text{m}}, R = 0.6 \]
Fig. 2.48  Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, $C = 3.16$, $a_i = 0.025$ mm, $\Delta \sigma = 55$ MPa, $\Delta K_{Th} = 3.3$ MPa$\sqrt{m}$, $R = 0.6$
Fig. 2.49 Fatigue Life of Small Size Cover-plated Beams, Eq. 2.13, $C = 3.16$, $a_i = 0.025$ mm, $\Delta C = 55$ MPa, $\Delta K_{th} = 3.3$ MPa√m, $R = 0.6$
Fig. 2.50 Fatigue Life of Full Size Cover-plated Beams, Eq. 2.13, C = 3.16, 
\( a_i = 0.025 \text{ mm}, \Delta \sigma = 55 \text{ MPa}, \Delta K_{th} = 3.3 \text{ MPa} \sqrt{m}, R = 0.6 \)
Fig. 2.51 Fatigue Life of Full Size Cover-plated Beams,
Eq. 2.13, C = 3.16, $a_i = 0.025$ mm, $\Delta \sigma = 55$ MPa,
$\Delta K_{Th} = 3.3$ MPa$\sqrt{m}$, $R = 0.6$
Fig. 2.52  Fatigue Life of Full Size Cover-plated Beams,
Eq. 2.13, $C = 3.16$, $a_1 = 0.025$ mm, $\Delta C = 55$ MPa,
$\Delta K_{Th} = 3.3$ MPa$\sqrt{m}$, $R = 0.6$

282
Fig. 2.53 Fatigue Life of Type 3 Stiffener, Eq. 2.13, $C = 3.16, \Delta K_{Th} = 3.3 \text{ MPa} \sqrt{m}, R = 0.6$
Fig. 2.54  Fatigue Life of Type 3 Stiffener, Eq. 2.13,
C = 3.16, a_i = 0.025 mm, \Delta \sigma = 110 MPa,
\Delta K_{Th} = 3.3 MPa\sqrt{m}, R = 0.6

C = 55 MPa\sqrt{m} 
\text{t}_{Fl} = 12.7 \text{mm}
Fig. 2.55 Fatigue Life of Type 3 Stiffener, Eq. 2.13,
$C = 3.16$, $\Delta \sigma = 110$ MPa, $\Delta K_{th} = 3.3$ MPa$\sqrt{m}$,
$R = 0.6$
\[ \frac{da}{dN} \text{ (log scale)} \]

- \( \Delta K_{Th} < \Delta K \leq \Delta K_1 \): \( \frac{da}{dN} = C \left( \Delta K^n - \Delta K_{Th}^n \right) \)
- \( \Delta K_1 < \Delta K \leq \Delta K_2 \): \( \frac{da}{dN} = C \Delta K^n \)
- \( \Delta K_2 < \Delta K < K_{max} \): \( \frac{da}{dN} = C \left( \frac{1}{K_C (1-R) - \Delta K} \right)^n \)

\[ \Delta K \text{ (log scale)} \]

\[ \Delta K_{Th} \] \quad \[ \Delta K_1 \] \quad \[ \Delta K_2 \] \quad \[ K_{max} = f(K_C) \]

Fig. 2.56 Schematic of Fatigue Crack Growth Relationship
Fig. 2.57 Influence of $\Delta K$ on Fatigue Crack Growth Rate Prediction,
Eq. 2.64, A533 B Steel$^{25}$, $R = 0.1$
Fig. 2.58 Influence of $\Delta K$ on Fatigue Crack Growth Rate Prediction, Eq. 2.64, Upper Bound, A533 B Steel, $R = 0.1$
Fig. 2.59  Fatigue Crack Growth Rate Prediction, Eq. 2.64, A 533 B Steel, $R = 0.1$
Fig. 2.60 Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel, 25°C, R = 0.3
Fig. 2.61 Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel, $R = 0.5$
Fig. 2.62 Fatigue Crack Growth Rate Prediction, Eq. 2.64, A533 B Steel, $R = 0.7$
Fig. 2.63 Influence of $\Delta K_{2}$ on fatigue crack growth rate prediction, Eq. 2.68, $R = 0.1$

$\frac{da}{dN}$ (mm/cycle)

$44 \text{MPa} \sqrt{m}$

$\Delta K (\text{MPa} \sqrt{m})$

$10^{-1}$ $10^{-2}$ $10^{-3}$ $10^{-4}$

20 30 40 50 100 150
Fig. 2.64 Influence of $\Delta K_2$ on Fatigue Crack Growth Rate Prediction, Eq. 2.69, $R = 0.1$
Fig. 2.65 Comparison between Fatigue Crack Growth Rate Prediction from Eq. 2.68 and Eq. 2.69, A36 Steel, $\Delta K_T = 45.4$ MPa$\sqrt{m}$, $K_c = MPa$, $R = 0$
Fig. 2.66 Proposed Fatigue Crack Growth Rate Prediction, $R = 0.6$. 

$\Delta K_{Th} = 3.3 \text{ MPa} \sqrt{\text{m}}$

$K_c = 165 \text{ MPa} \sqrt{\text{m}}$

$\Delta K_{T} = \text{Graphical Representation}$

$\frac{da}{dN} \text{ (mm/cycle)}$
Fatigue Life Using Model Ref. 22

Eq. 2.19

\[ \Delta \sigma = 0.076 \text{ mm} \]

\[ K_c = 110 \text{ MPa} \sqrt{m} \]

\[ a_i = 0.076 \text{ mm} \]

\[ a_i (\text{mm}) \]

\[ 0.025 \quad 0.076 \quad 0.76 \]

\[ 55 \quad 110 \quad 165 \]

\[ N (\text{cycles}) \]

\[ 10^5 \quad 10^6 \]

Fig. 2.67 Fatigue Life of Small Scale Cover-plated Beams, da/dN from Fig. 2.66, R = 0.6
Fig. 2.68 Fatigue Life of Small Size Cover-plated Beams, da/dN from Fig. 2.66, a_i = 0.025 mm, Δσ = 55 MPa
Fig. 2.69 $\Delta K - \frac{da}{dN}$ ($\Delta K$) and $\Delta K - N$ ($\Delta K$) for Small Size Cover-plated Beams, $\frac{da}{dN}$ from Fig. 2.66
$a_i = 0.013$ mm, $\Delta \sigma = 69$ MPa, $K_c = 165$ MPa$\sqrt{m}$, $R = 0.6$
Fig. 2.70 $a - \Delta K (a)$ and $a - a(N)$ for Small Size Cover-plated Beams, $da/dN$ from Fig. 2.66 $a_1 = 0.025$ mm, $\Delta \sigma = 69$ MPa, $R = 0.6$
Fig. 2.72 Fatigue Life of Full Size Cover-plated Beams, da/dN from Fig. 2.66, Δσ = 55 MPa, R = 0.6
Fig. 2.73 Fatigue Life of Full Size Cover-plated Beams, da/dN from Fig. 2.66, $\Delta \sigma = 55$ MPa, $R = 0.6$
Fig. 2.74 Fatigue Life of Type 3 Stiffeners, da/dN from Fig. 2.66, $\Delta \sigma = 110$ MPa, $K_c = \infty$
Fig. 2.75 Fatigue Life of Type 3 Stiffeners, da/dN from Fig. 2.66, \( a_i = 0.025 \text{ mm} \), \( \Delta \sigma = 110 \text{ MPa} \), \( R = 0.6 \)
Fig. 2.76 $a - \Delta K(a)$ and $a - N(a)$ for Stiffeners, $da/dN$ from Fig. 2.66, $a_i = 0.025$ mm, $\Delta \sigma = 110$ MPa, $R = 0.6$
Fig. 3.1  AASHTO Material Toughness Requirements for A36 Steel, Zone II, Ref. 52
Fig. 3.2 Crack Growth Rate Prediction used for Case Studies of Fatigue Life Calculations
Fig. 3.3 Quinipiac River Bridge, Crack Location
Fig. 3.4  Schematic of Girder Cross Section and Crack Dimension, Quinnipiac River Bridge
Fig. 3.5 Assumption for Residual Stress Distribution near Stiffener-Web Connection, Quinnipiac River Bridge
Fig. 3.6 Crack Growth Stages, Quinnipiac River Bridge\textsuperscript{1,51}
Fig. 3.7 Crack Growth Stage II, Fatigue Crack Growth, Quinipiac River Bridge
Fig. 3.8 Crack Growth in Web, Stage IIa, Test Beam
Fig. 3.9 Partially Loaded Circular Crack in Infinite Solid
Fig. 3.10  Through Crack in Infinite Solid
Fig. 3.11 Stress Intensity Factor and Fatigue Life for Crack in Web, Circular Crack, Quinnipiac River Bridge
Fig. 3.12 Stress Intensity Factor, Through Crack, Quinnipiac River Bridge
Fig. 3.13  Crack Size for $K_c = 110 \text{ MPa}\sqrt{\text{m}}$ and $K_c = 165 \text{ MPa}\sqrt{\text{m}}$, Quinipiac River Bridge
Fig. 3.14  Schematic of I79 Glenfield Bridge

320
Fig. 3.15  Schematic of Girder Cross Section, Crack Dimension, Glenfield Bridge
Fig. 3.16 Charpy V-Notch Test Results, Glenfield Bridge
c) Fusion Line, Heat Affected Zone

d) Repair Weld

Fig. 3.16 Continued
Fig. 3.17  Fracture Toughness of Electroslag Weld Metal, Glenfield Bridge
Fig. 3.18 Stress Range Histogram at Crack Location, Glenfield Bridge
Fig. 3.19  Residual Stress Distribution in Flange Due to Electroslag Weldment, Glenfield Bridge
a) 1st. Repair (weld passes on the fusion line)

b) 2nd. Repair (primary repair, after gouging out defect)

c) 3rd. Repair (weld passes near fusion line)

Fig. 3.20  Repair Weld Sequence in Electroslag Weld$^{57}$, Glenfield Bridge

327
Fig. 3.21 Residual Stress Distribution in Flange due to Electroslag Weldment after Repair, Glenfield Bridge
Fig. 3.22 Residual Stress Distribution in Flange due to Repair Weld, Glenfield Bridge
Fig. 3.23 Residual Stress Distribution in Flange due to Web-Flange Weldment, Glenfield Bridge
Fig. 3.24 Fracture Surface, South Side, Glenfield Bridge
Fig. 3.26  Schematic of Crack Growth Stages, Glenfield Bridge
Fig. 3.27  Investigated Crack Shapes, Glenfield Bridge
Fig. 3.28 Striation Markings at Location 3, 48240X, Glenfield Bridge
\[ y = a \sin \phi \]
\[ x = c \cos \phi \]
\[ K = \sigma (\pi a)^{1/2} \cdot F_e \]
\[ F_e = \frac{1}{E(k)} (\sin^2 \phi + \left(\frac{a}{c}\right)^2 \cos^2 \phi)^{1/4} \]
\[ E(k) = \int_0^{\pi/2} (1 - k^2 \sin^2 \theta)^{1/2} \, d\theta \]
\[ k^2 = 1 - \left(\frac{a}{c}\right)^2 \]

Fig. 3.29 Stress Intensity Factor for Elliptical Crack under Uniform Load
\[
K_{IA} = \sigma (\pi a)^{1/2} F_{wa} \left( \frac{a}{b_1}, \frac{e}{b} \right)
\]

\[
K_{IB} = \sigma (\pi a)^{1/2} F_{wb} \left( \frac{a}{b_1}, \frac{e}{b} \right)
\]

Fig. 3.30 Stress Intensity Factor for Eccentrical Through Crack in Finite Plate
AB = \ell
OA = R
OB = r

Splitting Forces \( P \) at \( B \)

\[
K = \frac{P\sqrt{a}}{\pi^{3/2}\ell^2} \sqrt{\frac{r}{R}} \frac{\sqrt{\frac{1}{a^2} - 1}}{\left\{1 - k^2 \cos^2 \phi \right\}^{1/4}}
\]

Fig. 3.31 Stress Intensity Factor at Location A due to Splitting Force at \( B \)
Fig. 3.32  Stress Intensity Factor around
Initial Elliptical Flaw, Glenfield Bridge
Fig. 3.33 Comparison between Stress Intensity Factor during Stage III and Material Toughness, Glenfield Bridge
Fig. 3.34 Stress Intensity Factor around Semicircular Crack, $r = 51$ mm, Glenfield Bridge
Fig. 3.35  Stress Intensity Factor around
Semicircular Crack, $r = 64$ mm,
Glenfield Bridge
Fig. 3.36  Stress Intensity Factor around Semicircular Crack, 
$r = 76$ mm, Glenfield Bridge
Fig. 3.37 Comparison between Stress Intensity Factor during Final Fracture and Material Toughness, Glenfield Bridge
Fig. 3.38  Schematic of Span and Cross Section of Main Girder at Crack Location, Lafayette Street Bridge
Fig. 3.39  Schematic of the Crack in the Stiffener-Gusset Region, Lafayette Street Bridge
Fig. 3.40 Charpy V-Notch Test Results and Material Toughness for Web Material, Lafayette Street Bridge
Fig. 3.41 Residual Stress Distribution in Flange due to Web-Flange Weldment, Lafayette Street Bridge
Fig. 3.42  Crack Growth Stages in Web of Lafayette Street Bridge
Fig. 3.43 Fracture Surface of Removed Plug, Second Fracture, Lafayette Street
a) Fatigue Crack Growth in Gusset

b) Fatigue Crack Growth in Web

Fig. 3.44 Crack Growth Models during Stage II, Lafayette Street Bridge
Fig. 3.45 Maximum Stress Intensity Factor in Flange, Lafayette Street Bridge
Fig. 3.46 Stress Distribution and Stress Intensity Factor during Stage III-Stage IV, Lafayette Street Bridge
Fig. 3.47 Fracture Toughness and Fatigue Life for Crack in Web, Lafayette Street Bridge
Fig. 3.48 Cracked Cover Plate Beam at Yellow Mill Pond Bridge
Fig. 3.50 Charpy V-Notch Test Results, Yellow Mill Pond Bridge
Fig. 3.51 Fracture Toughness for Material Removed from the Web, Yellow Mill Pond Bridge
Fig. 3.52 Fracture Toughness for Material Removed from the Flange, Yellow Mill Pond Bridge
Fig. 3.53  Cross Section of B4, Stress Distribution and Location of the Neutral Axis, Yellow Mill Pond Bridge
Fig. 3.54  Residual Stress Distribution for A242, W36X230 Flange (after Ref. 36)
Fig. 3.55  Local Weld Residual Stress Distribution for Cover Plate with End-Weld (after Ref. 36)
Fig. 3.56 Stress Intensity Factor for Semielliptical Surface Crack in Flange, $a = 13$ mm, $c = 63$ mm, Yellow Mill Pond Bridge
Fig. 3.57  Stress Intensity Factor for Semielliptical Surface Crack in Flange, a = 25 mm, c = 139 mm, Yellow Mill Pond Bridge
\[ K = \sigma \left( \frac{\pi a}{\pi a} \right)^{1/2} \left( \frac{2b}{\pi a \tan \frac{\pi a}{2b}} \right)^{1/2} \frac{0.923 + 0.199 \left( \frac{1 - \sin \frac{\pi a}{2b}}{\cos \frac{\pi a}{2b}} \right)^4}{(1 - a/b)^2} \]

\[ v = \frac{4\sigma a}{E} \left[ 0.8 - 0.7 \left( \frac{a}{b} \right) + 2.4 \left( \frac{a}{b} \right)^2 + \frac{0.66}{(1 - a/b)^2} \right] \]

Fig. 3.58 Stress Intensity Factor and Crack Opening for Bending Specimen
Fig. 3.59 Fatigue Life of B4, Span 11, $C = 2.18 \times 10^{-13} \text{ mm}^{5.5} \text{ N}^{-1} \text{ cycle}^{-1}$ Yellow Mill Pond Bridge
Fig. 3.60  Stress Intensity Factor for Crack
in Flange, c = 7.0 a^{1.0}, Yellow
Mill Pond Bridge
Fig. 3.61  Fatigue Life for Crack in W36×230 Flange, $\Delta \sigma = 8.1$ MPa, Yellow Mill Pond Bridge
Fig. 3.62 Schematic showing Box Girder Bent with Crack Location, Dan Ryan Viaduct
Fig. 3.63  Charpy V-Notch Results, Web Material, Dan Ryan Viaduct
Fig. 3.64 Fracture Toughness measured with Compact Tension Tests\textsuperscript{75}, 1 sec Loading Time, Dan Ryan Viaduct
Fig. 3.65 Stress Range in Box Web Plate at Crack Location, Dan Ryan Viaduct
Fig. 3.66 Fracture Surface of Box Web Plate, Pier 24, Dan Ryan Viaduct
Fig. 3.67 Fracture Surface of Box Web Plate, Pier 25, Dan Ryan Viaduct
Fig. 3.68 Fracture Surface of Box Web Plate, Pier 26, Dan Ryan Viaduct
Fig. 3.69  Fatigue Life for Semielliptical Surface Crack Growing into Web from Exterior, Dan Ryan Viaduct
Fig. 3.70 Schematic of Fracture Surface and Corresponding Analytical Model, Pier 26, Dan Ryan Viaduct
Fig. 3.71  Stress Intensity Factor for Through Crack in Web, Dan Ryan Viaduct
Fig. 3.72 $F_g$ - Correction Factor for Through Crack near Inserted Plate, Dan Ryan Viaduct
Fig. 3.73 Dimensions of the Box Girder, Gulf Outlet Bridge
Fig. 3.74 Cracks Found in the Corner Weld, Box Girder

a) Average Crack

b) Largest found Crack
Fig. 3.75 Temperature Distribution in a Semi-infinite Plate
Fig. 3.76  Weld Sequence and Discretisation near Corner Weld, Box Girder
Fig. 3.77 Residual Stress Variation During Weld Passes of Corner Weld, Box Girder

384
Fig. 3.78 Residual Stress Distribution due to Corner Welds in Middle Plane of Web and Flange, Box Girder
Fig. 3.79 Residual Stress Distribution near Corner Weld, Box Girder

386
Fig. 3.80 Investigated Crack Shapes in Corner Weld, Box Girder
Fig. 3.81 Definition of Crack Lengths, Box Girder
Fig. 3.82 Stress Intensity Factor for Cracks in Corner Weld, Box Girder

389
Fig. 3.82 Continued
Fig. 3.83 Stress Intensity Factor and Fatigue Life for Cracks in Corner Weld, $\Delta \sigma = 14$ MPa, Box Girder
Fig. 3.84 Investigated Crack Shapes, Edge Cracks in Flange and Web Plate, Box Girder
Fig. 3.85 Stress Intensity Factor and Fatigue Life for Edge Crack in Flange and Web Plate, Box Girder
Fig. 4.1 Small Fatigue Crack at Cover Plate End Weld. 40X
Fig. 4.2 Peening Tool
Fig. 4.3 Peened Cover Plate End Weld, Small Specimens Cut Out for Investigation

Fig. 4.4 Mounted Specimen
Fig. 4.5 Schematic of Set-Up to Measure Surface Deformations from Peening
Fig. 4.6 Hardness Measurements Indentations in Peened Zone, Specimen D. 700X
Fig. 4.7a Peened Cover Plate End, Specimen A
(3 Passes, 0.21 N/mm²)

Fig. 4.7b Peened Cover Plate End, Specimen C
(3 Passes, 0.34 N/mm²)
Fig. 4.7c Peened Cover Plate End, Specimen F
(9 Passes, 0.28 N/mm²)
Fig. 4.8 Grain Deformation in Peened Region, Specimens A & B. 80X
401
Fig. 4.8 (cont.) Grain Deformation in Peened Region, Specimens C & D. 80X

402
Fig. 4.8 (cont.) Grain Deformation in Peened Region, Specimens E & F. 80X

403
Fig. 4.8 (cont.) Grain Deformation in Peened Region, Specimen G. 80X
Fig. 4.9 Measured Deformations, Influence of the Air Pressure
Fig. 4.10 Measured Deformations, Influence of the Number of Passes
Fig. 4.11 Knopp Hardness Measurements, Specimen G
Fig. 4.12 Knoop Hardness Measurements, Specimen $G_1$

408
Fig. 4.13 Knoop Hardness, Specimen D

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**Diagram Description**

- **Knoop Hardness**
  - Graphs (a), (b), (c), and (d) show variations in Knoop hardness over different distances (t).
  - Graph (a) highlights the zone of deformed grains, marked by measurement and base plate.
  - The graphs are labeled with specific axes indicating t (mm) and hardness values.
Fig. 4.14  Hardness Measurements Indentations in Specimen D.  100X
Fig. 4.15 Peened Region of Beam B1. 62.5X
Fig. 4.16  Peened Region of Beam B9.  62.5X
Fig. 4.17 Peened Region of Beam B14 with Fatigue Crack. 62.5X
Fig. 4.18 Residual Stress Distribution Caused by Peening
Fig. 4.19 Influence of the Minimum and Maximum Stress Intensity Factor on the Effective Stress Intensity Range
Fig. 4.20 Zones Influenced by Peening
Fig. 4.21 Influence of the Thickness of the Residual Compression Stress Zone on the Threshold Crack Length, $\Delta \sigma = 41$ MPa
Fig. 4.22 Influence of the Stress Range on the Threshold Crack Length
Fig. 4.23 Influence of the Yield Strength on the Threshold Crack Length, $\Delta \sigma = 41$ MPa
Fig. 4.24 Influence of the Minimum Stress on the Threshold Crack Length
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