EVALUATION OF MAINTENANCE/REHABILITATION
ALTERNATIVES FOR CONTINUOUSLY REINFORCED
CONCRETE PAVEMENT

by

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A Report of the Investigation of
Determination of Optimum Maintenance Procedures
and Materials for Continuously Reinforced Concrete Pavement
Project IHR-901
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carried out by the
TRANSPORTATION RESEARCH LABORATORY
DEPARTMENT OF CIVIL ENGINEERING
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Evaluation of Maintenance/Rehabilitation Alternatives For Continuously Reinforced Concrete Pavement

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**Abstract**
An evaluation of several maintenance/rehabilitation methods for an Interstate Continuously Reinforced Concrete Pavement (CRCP) in Illinois has been conducted. Maintenance and rehabilitation needs are increasing rapidly due to aging and heavy truck traffic on the Interstate system. Thus, efficient methods are greatly needed. The design, construction, performance, and costs of several maintenance and rehabilitation methods were evaluated including patching, cement grout and asphalt undersealing, epoxying of cracks, and an asphalt overlay. Information gathered will be very useful in the development of future maintenance activities and rehabilitation projects. Nondestructive Testing (NDT) deflections, reflection cracking, cost, and statistical analyses were used to evaluate the maintenance and rehabilitation methods. Two experimental patches, one with a reduced splice length, the other with a welded splice, were recommended as alternatives to current patching techniques because of cost and construction time reductions. Cement grout undersealing significantly reduced peak deflections and was recommended for preventive maintenance or rehabilitation on a selective basis to fill voids and reduce pumping. Asphalt undersealing did not reduce deflections, but was recommended for use as a preventive maintenance treatment to protect the subbase and reduce pumping. The use of epoxy to bond wide cracks together failed due to large movements of the CRCP. The asphalt overlay significantly reduced deflections and should extend the life for several years. Nearly all wide cracks not patched in the existing CRCP (where some or all rebars ruptured) have reflected through the overlay after one year. The asphalt overlay placed over the most highly distressed portion of the project was a cost effective method.

**Key Words**
Pavement, Concrete, Distress, Performance, Design, Evaluation, Maintenance, Reinforcement, Construction, Patching, Rehabilitation, Overlay, Deflection, Grout, Underseal, Asphalt, Costs

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DISCLAIMER

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


PREFACE

This is the third report describing the work accomplished on Project IHR-901 entitled "Determination of Optimum Maintenance Procedures and Materials for Continuously Reinforced Concrete Pavement." Appreciation is expressed to Mr. Chris Sward for assistance during the project construction. Thanks are due to the project advisory committee for guidance and for reviewing this report. The committee includes: H. C. Bankie, B. J. Dempsey, W. L. Gamble, J. Santarelli, D. R. Schwartz, M. F. Thompson, C. P. Alexander, and John Ebers, Jr. Special appreciation is extended to several personnel from the Illinois Department of Transportation, District 3, for the extensive assistance given to the authors, including Mr. Philip J. Faletto, Mr. Claude F. Hershey, Mr. Ed Wallens, and Mr. Gary Sale. Thanks are due Mr. J. S. Dhamrait and Mr. Ken Wicks for conducting the Road Rater deflection measurements. Appreciation is also expressed to Maggie Ross for typing this report.

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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Chapter</th>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>DESCRIPTION OF THE FIELD SURVEY</td>
<td>3</td>
</tr>
<tr>
<td>2.1</td>
<td>Introduction</td>
<td>3</td>
</tr>
<tr>
<td>2.2</td>
<td>Initial Field Survey</td>
<td>3</td>
</tr>
<tr>
<td>2.2.1</td>
<td>Wide Transverse Cracks</td>
<td>4</td>
</tr>
<tr>
<td>2.2.2</td>
<td>Edge Punchouts</td>
<td>5</td>
</tr>
<tr>
<td>2.2.3</td>
<td>Failure of Slab Adjacent to a Patch</td>
<td>5</td>
</tr>
<tr>
<td>2.2.4</td>
<td>Failures Adjacent to a Joint in a Ramp</td>
<td>6</td>
</tr>
<tr>
<td>2.3</td>
<td>Core and Deflection Study</td>
<td>7</td>
</tr>
<tr>
<td>2.4</td>
<td>Selection of Rehabilitation Portion</td>
<td>9</td>
</tr>
<tr>
<td>2.5</td>
<td>More Extensive NDT Deflection Testing</td>
<td>9</td>
</tr>
<tr>
<td>2.5.1</td>
<td>Description</td>
<td>10</td>
</tr>
<tr>
<td>2.5.2</td>
<td>Deflection Profile Analysis</td>
<td>11</td>
</tr>
<tr>
<td>2.5.3</td>
<td>Results of the Statistical Analysis</td>
<td>12</td>
</tr>
<tr>
<td>3</td>
<td>EXPERIMENTAL PATCHING PROGRAM</td>
<td>43</td>
</tr>
<tr>
<td>3.1</td>
<td>Description of Experimental Patches</td>
<td>43</td>
</tr>
<tr>
<td>3.1.1</td>
<td>20 Inch Tied Lap Splice Patch</td>
<td>43</td>
</tr>
<tr>
<td>3.1.2</td>
<td>4-Inch Double Weld Splice</td>
<td>45</td>
</tr>
<tr>
<td>3.1.3</td>
<td>Other Experimental Patches</td>
<td>47</td>
</tr>
<tr>
<td>3.2</td>
<td>Construction Procedures</td>
<td>48</td>
</tr>
<tr>
<td>3.2.1</td>
<td>Saw Cutting Patches</td>
<td>48</td>
</tr>
<tr>
<td>3.2.2</td>
<td>Breakout Methods</td>
<td>49</td>
</tr>
<tr>
<td>3.2.3</td>
<td>Reinforcement</td>
<td>52</td>
</tr>
<tr>
<td>3.2.4</td>
<td>Concrete Placement</td>
<td>54</td>
</tr>
<tr>
<td>3.3</td>
<td>Patch Evaluation</td>
<td>55</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Visual Evaluation</td>
<td>55</td>
</tr>
<tr>
<td>3.3.2</td>
<td>NDT Evaluation</td>
<td>60</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Reflective Crack Survey</td>
<td>60</td>
</tr>
<tr>
<td>3.3.4</td>
<td>Cost Analysis</td>
<td>61</td>
</tr>
<tr>
<td>4</td>
<td>CEMENT GROUT UNDERSEALING</td>
<td>89</td>
</tr>
<tr>
<td>4.1</td>
<td>Introduction</td>
<td>89</td>
</tr>
<tr>
<td>4.2</td>
<td>Cement Grout Undersealing Procedures</td>
<td>90</td>
</tr>
<tr>
<td>4.3</td>
<td>Evaluation of Pressure Grouting</td>
<td>93</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Deflection Evaluation of Pressure Grouted Areas</td>
<td>94</td>
</tr>
<tr>
<td>4.3.2</td>
<td>Cost Analysis</td>
<td>96</td>
</tr>
</tbody>
</table>
5 ASPHALT UNDERSEALING ........................................ 107
  5.1 Introduction ............................................. 107
  5.2 Asphalt Undersealing Procedures ....................... 108
  5.3 Evaluation of Asphalt Undersealing .................... 111
      5.3.1 Deflection Evaluation of Asphalt Undersealed Areas ........................................ 111
      5.3.2 Cost Analysis ................................... 113

6 EPOXY CRACK SEALING ........................................ 120
  6.1 Introduction ............................................. 120
  6.2 Epoxy Crack Sealing Procedures ....................... 121
  6.3 Results from Epoxy Crack Sealing ..................... 122

7 ASPHALT OVERLAY AND UNDERDRAINS ......................... 124
  7.1 Introduction ............................................. 124
  7.2 Underdrains ............................................ 124
  7.3 Asphalt Overlay ....................................... 124
  7.4 Cost Analysis ....................................... 124

8 CONCLUSIONS AND RECOMMENDATIONS ......................... 134

REFERENCES .................................................. 137
<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Breakdown of Distress Shown in Figure 2.4</td>
<td>39</td>
</tr>
<tr>
<td>2.2</td>
<td>Initial Test Section for NDT Deflections</td>
<td>40</td>
</tr>
<tr>
<td>2.3</td>
<td>Modified Sections for Statistical Analysis of Deflections</td>
<td>41</td>
</tr>
<tr>
<td>2.4</td>
<td>Results of Statistical Analysis of NDT Deflections</td>
<td>42</td>
</tr>
<tr>
<td>3.1</td>
<td>Cracking Data for Patches</td>
<td>87</td>
</tr>
<tr>
<td>3.2</td>
<td>Patching Cost Data</td>
<td>88</td>
</tr>
<tr>
<td>4.1</td>
<td>1978 Cost Data for Various Cement Grouting Applications</td>
<td>106</td>
</tr>
<tr>
<td>7.1</td>
<td>Results from Statistical Analysis of Deflection</td>
<td>133</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>Accumulative Patching Requirements versus Accumulative 18-kip ESAL Traffic</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>Loading.</td>
<td></td>
</tr>
<tr>
<td>2.2</td>
<td>Results of Crack Survey.</td>
<td>15</td>
</tr>
<tr>
<td>2.3</td>
<td>Location of Distress Relative to Area Selected for Rehabilitation.</td>
<td>16</td>
</tr>
<tr>
<td>2.4</td>
<td>Wide Crack.</td>
<td>17</td>
</tr>
<tr>
<td>2.5</td>
<td>Wide Crack.</td>
<td>17</td>
</tr>
<tr>
<td>2.6</td>
<td>Formation of Longitudinal Crack Between Two Closely Spaced Transverse</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>Cracks - Early State of Edge Punchout Development.</td>
<td></td>
</tr>
<tr>
<td>2.7</td>
<td>Extended Development of Edge Punchout.</td>
<td>19</td>
</tr>
<tr>
<td>2.8</td>
<td>Edge Punchout.</td>
<td>20</td>
</tr>
<tr>
<td>2.9</td>
<td>Edge Punchout.</td>
<td>20</td>
</tr>
<tr>
<td>2.10</td>
<td>Distress Adjacent to a Patch.</td>
<td>21</td>
</tr>
<tr>
<td>2.11</td>
<td>Distress Adjacent to a Patch.</td>
<td>21</td>
</tr>
<tr>
<td>2.12</td>
<td>Disintegration of Subbase Beyond the Patch Boundaries.</td>
<td>22</td>
</tr>
<tr>
<td>2.13</td>
<td>Crack in a Patch Adjacent to a Joint in the Ramp.</td>
<td>23</td>
</tr>
<tr>
<td>2.14</td>
<td>IDOT Road Rater.</td>
<td>23</td>
</tr>
<tr>
<td>2.15</td>
<td>Crack Pattern and Deflections at Cores No. 1 &amp; 3  (1 in. = 2.5 cm, 1 ft.</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>= 0.3 m, 1 kip = 4.45 N)</td>
<td></td>
</tr>
<tr>
<td>2.16</td>
<td>Crack Pattern and Deflections at Cores No. 2 &amp; 4  (1 in. = 2.5 cm, 1 ft.</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>= 0.3 m, 1 kip = 4.45 N)</td>
<td></td>
</tr>
<tr>
<td>2.17</td>
<td>Area Before Core No. 1 was Taken</td>
<td>26</td>
</tr>
<tr>
<td>2.18</td>
<td>Faulting of Crack Before Core No. 1 was Taken</td>
<td>26</td>
</tr>
<tr>
<td>2.19</td>
<td>Core No. 1 - Concrete Deterioration.</td>
<td>27</td>
</tr>
<tr>
<td>2.20</td>
<td>Core No. 1 - Corrosion of Steel.</td>
<td>27</td>
</tr>
<tr>
<td>2.21</td>
<td>Edge Punchout Before Core No. 2 was Taken</td>
<td>28</td>
</tr>
<tr>
<td>2.22</td>
<td>Width of Crack at Core No. 2.</td>
<td>28</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>2.23</td>
<td>Core No. 2 - Concrete Deterioration</td>
<td>29</td>
</tr>
<tr>
<td>2.24</td>
<td>Core No. 2 - Corrosion of Steel</td>
<td>29</td>
</tr>
<tr>
<td>2.25</td>
<td>Core No. 3 - Very Tight Crack at Pavement Surface</td>
<td>30</td>
</tr>
<tr>
<td>2.26</td>
<td>Core No. 3 - Microscopic Crack Width at Mid-depth</td>
<td>30</td>
</tr>
<tr>
<td>2.27</td>
<td>Core No. 3 - Infiltration of Foreign Material</td>
<td>31</td>
</tr>
<tr>
<td>2.28</td>
<td>Core No. 3 - No Corrosion on Steel</td>
<td>31</td>
</tr>
<tr>
<td>2.29</td>
<td>Core No. 4 - Spalled Crack</td>
<td>32</td>
</tr>
<tr>
<td>2.30</td>
<td>Core No. 4 - Spall at Top of Core</td>
<td>32</td>
</tr>
<tr>
<td>2.31</td>
<td>Core No. 4 - Loss of Aggregate Interlock</td>
<td>33</td>
</tr>
<tr>
<td>2.32</td>
<td>Core No. 4 - Ruptured Steel and Concrete Deterioration</td>
<td>33</td>
</tr>
<tr>
<td>2.33</td>
<td>Typical Deflection Profile</td>
<td>34</td>
</tr>
<tr>
<td>2.34</td>
<td>Deflection Profile at a Distress</td>
<td>35</td>
</tr>
<tr>
<td>2.35</td>
<td>Deflection Peaks Reveal Underlying Voids</td>
<td>36</td>
</tr>
<tr>
<td>2.36</td>
<td>Low Deflections Indicate 12 in. Thick CRCP</td>
<td>37</td>
</tr>
<tr>
<td>2.37</td>
<td>Coefficient of Variation (COV) Versus Failures per 100 feet</td>
<td>38</td>
</tr>
<tr>
<td>3.1</td>
<td>20-Inch Tied Splice Patch</td>
<td>64</td>
</tr>
<tr>
<td>3.2</td>
<td>Effective Weld Length Versus Axial Strength for Welded Splices</td>
<td>65</td>
</tr>
<tr>
<td>3.3</td>
<td>Weld Design Used in the Welded Splice Patch</td>
<td>66</td>
</tr>
<tr>
<td>3.4</td>
<td>4-Inch Double Weld Splice Patch</td>
<td>67</td>
</tr>
<tr>
<td>3.5</td>
<td>Sawing of Partial Depth Cut</td>
<td>68</td>
</tr>
<tr>
<td>3.6</td>
<td>Ripper Saw</td>
<td>68</td>
</tr>
<tr>
<td>3.7</td>
<td>Full Depth Cut Made by Ripper Saw</td>
<td>69</td>
</tr>
<tr>
<td>3.8</td>
<td>Full Depth Cut Made by Ripper Saw</td>
<td>69</td>
</tr>
<tr>
<td>3.9</td>
<td>Hydrahammer</td>
<td>70</td>
</tr>
<tr>
<td>3.10</td>
<td>Reinforcing Steel Bent Upward from Mechanical Breakout</td>
<td>70</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>3.11</td>
<td>Drophammer</td>
<td>71</td>
</tr>
<tr>
<td>3.12</td>
<td>Initial Breakout Method (Not Recommended)</td>
<td>72</td>
</tr>
<tr>
<td>3.13</td>
<td>Cutting Reinforcement with a Hand Held Gas Saw</td>
<td>73</td>
</tr>
<tr>
<td>3.14</td>
<td>Improved Breakout Method</td>
<td>74</td>
</tr>
<tr>
<td>3.15</td>
<td>20-Inch Tied Lap in the Center of the Welded Splice Patch</td>
<td>75</td>
</tr>
<tr>
<td>3.16</td>
<td>Broken Welds</td>
<td>75</td>
</tr>
<tr>
<td>3.17</td>
<td>Broken Welds</td>
<td>76</td>
</tr>
<tr>
<td>3.18</td>
<td>Broken Bars</td>
<td>76</td>
</tr>
<tr>
<td>3.19</td>
<td>The Shaded Rebar, Which is Welded to Existing Reinforcement of the Right End of the Patch, Butts Against Existing Reinforcement at the Left End of the Patch</td>
<td>77</td>
</tr>
<tr>
<td>3.20</td>
<td>Placement of Concrete</td>
<td>78</td>
</tr>
<tr>
<td>3.21</td>
<td>Vibrating the Concrete</td>
<td>78</td>
</tr>
<tr>
<td>3.22</td>
<td>Time of Pour Versus Percent of Patches Cracked</td>
<td>79</td>
</tr>
<tr>
<td>3.23</td>
<td>Lap Steel Left in No-Steel Patch</td>
<td>80</td>
</tr>
<tr>
<td>3.24</td>
<td>Transverse Crack Through Full Depth of Slab</td>
<td>80</td>
</tr>
<tr>
<td>3.25</td>
<td>Type of Splice Versus Percent of Patches Cracked</td>
<td>81</td>
</tr>
<tr>
<td>3.26</td>
<td>Adjacent Lane Condition Versus Percent of Patches Cracked</td>
<td>82</td>
</tr>
<tr>
<td>3.27</td>
<td>Deflection Profiles Before and After Placement of a Welded Splice Patch</td>
<td>83</td>
</tr>
<tr>
<td>3.28</td>
<td>Deflection Profiles Before and After Placement of a Bituminous Patch</td>
<td>84</td>
</tr>
<tr>
<td>3.29</td>
<td>Deflection Profiles Before and After Placement of a No-Steel Patch</td>
<td>85</td>
</tr>
<tr>
<td>3.30</td>
<td>Reflection Cracking at Bituminous Patch Boundaries</td>
<td>86</td>
</tr>
<tr>
<td>4.1</td>
<td>Cement Grout Undersealing Hole Pattern</td>
<td>99</td>
</tr>
<tr>
<td>4.2</td>
<td>Gardner-Denver Drill Mounted on a Forklift</td>
<td>100</td>
</tr>
<tr>
<td>Figure</td>
<td>Description Rollers Page</td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>--------------------------</td>
<td></td>
</tr>
<tr>
<td>4.3</td>
<td>Compressive Strength of Cement Grout Slurry Versus Time... 101</td>
<td></td>
</tr>
<tr>
<td>4.4</td>
<td>Apparatus for Monitoring Vertical Movements of the Slab... 102</td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>Profile Showing Reduced Deflections After Cement Grout Undersealing..................................................... 103</td>
<td></td>
</tr>
<tr>
<td>4.6</td>
<td>Deflection Profiles Before and After Cement Grout Undersealing..................................................... 104</td>
<td></td>
</tr>
<tr>
<td>4.7</td>
<td>Deflection Profiles Before and After Cement Grout Undersealing..................................................... 105</td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>Asphalt Undersealing Hole Pattern..................................................... 116</td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td>Drill Rig Used in Asphalt Undersealing..................................................... 117</td>
<td></td>
</tr>
<tr>
<td>5.3</td>
<td>Deflection Profiles Before and After Asphalt Undersealing.. 118</td>
<td></td>
</tr>
<tr>
<td>5.4</td>
<td>Deflection Profiles Before and After Asphalt Undersealing.. 119</td>
<td></td>
</tr>
<tr>
<td>7.1</td>
<td>Illustration of Pipe Underdrains..................................................... 129</td>
<td></td>
</tr>
<tr>
<td>7.2</td>
<td>Water Flowing from Lateral Outlets..................................................... 130</td>
<td></td>
</tr>
<tr>
<td>7.3</td>
<td>Deflection Profiles Before and After the Asphalt Overlay... 131</td>
<td></td>
</tr>
<tr>
<td>7.4</td>
<td>Cost Effectiveness of an Asphalt Overlay Compared to Continual Patching..................................................... 132</td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 1
INTRODUCTION

This report evaluates several maintenance/rehabilitation methods applied to an Interstate highway continuously reinforced concrete pavement (CRCP) in Illinois. Several experimental features were incorporated into the project. The results of this project provide considerable information useful in the development of other RRR projects, and also in applying several maintenance procedures.

Illinois has constructed over 2,000 two-lane miles of CRCP on heavily trafficked Interstate highways. Many of these CRCP have either exceeded their design traffic life or are experiencing concrete durability problems, and are requiring extensive patching maintenance. Thus, there is a great need to determine the most effective procedures for both maintaining and rehabilitating CRCP. The CRCP rehabilitation described herein represents the first such project in Illinois. This project includes three types of permanent patching (lapped splices, welded splices, and asphalt patches), epoxying of wide cracks, cement grout undersealing, underdrains, and an asphalt overlay. Construction techniques, costs, and related problems are described. Reflective cracking through the overlay after one year is discussed. Nondestructive testing (NDT) utilizing a heavy load Road Rater was used to evaluate structural deficiencies and the structural effect of the various experimental features. Four complete sets of NDT deflections were taken (1) before any construction, (2) after the patching, cement grout undersealing, asphalt undersealing, and crack epoxy, (3) 3 months after the asphalt overlay was placed, and (4) one year after the overlay was placed.
The report is presented in the following sequence:

Chapter 2 - describes the initial field condition survey, coring results, and NDT deflection analysis.

Chapter 3 - provides results from the experimental patching program.

Chapter 4 - describes the cement grout undersealing.

Chapter 5 - describes the asphalt undersealing.

Chapter 6 - explains the wide crack epoxy tests.

Chapter 7 - describes the underdrains and asphalt overlay construction and results obtained.

Chapter 8 - presented conclusions and recommendations.
Chapter 2
DESCRIPTION OF THE FIELD SURVEY

2.1 Introduction

The project consists of 6.6 miles (10.4 km) of 4 lane rural Interstate (I-57) identified as construction section 46-1. The structural section consists of an 8 in. (20 cm) continuously reinforced concrete pavement (CRCP) over a 4 in. (10 cm) cement aggregate mixture (CAM) subbase placed on grade. The soil in this area is generally an A-6 classification.

This section was chosen as a Resurfacing, Restoring, and Rehabilitation (R-R-R) project for several reasons but primarily due to its rate of distress occurrence. Figure 2.1 shows that the rate of distress is increasing sharply as the number of cumulative 18-kip (80 kN) ESAL's increases beyond the design traffic life of 4.8 x 10^6 ESAL's.

The project survey of the section consisted of an initial distress survey, a coring study, and an NDT deflection analysis.

2.2 Initial Field Survey

During the initial distress survey, conducted 6 months prior to construction, all the distresses occurring throughout the section were recorded and photographed. Also in the distress survey, the average crack spacing was determined. A diagram of the crack survey, as well as the average spacing value are shown in Figure 2.2.

The initial survey showed that the section was suffering from four major distress types: (1) wide transverse cracks, (2) edge punchouts and associated pumping, (3) failure of pavement adjacent to a patch, and (4) failures adjacent to a joint in a ramp. The section had no evidence of D-cracking or other related distress. Figure 2.3 shows the occurrence of the distress and Table 2.1 gives a complete breakdown of all the distresses.
2.2.1 Wide Transverse Cracks

As of November 1977 over 130 cracks were observed to have opened up between 0.05 to 0.75 in., (1 to 19 mm) and most had spalled and faulted from 0.1 to 0.7 in. (3 to 18 mm). Examples of such wide cracks can be seen in Figures 2.4 and 2.5.

At many of the cracks, the steel was visible from the top of the pavement and was observed to be ruptured. It was usually necked down due to corrosion, and the rupture was usually along the face of one side of the crack. At cracks where the steel was not visible, a straight edge was passed in the slab along the crack to determine if the steel was ruptured.

The crack usually begins to open up at the outer edge of the truck lane and then progressively spalls and faults across the lane. Most of the wide cracks (88 percent) extend only across the truck lane and hence are definitely related to repeated loading. Once the crack opens enough to allow water (and chlorides from deicing salts) to reach the reinforcement, corrosion and necking down of the rebars occurs. Also, load is transferred across the crack through aggregate interlock and steel reinforcement. Once the crack opens up wider than about 0.04 in. (1 mm) most of the aggregate interlock is lost which causes high vertical shear stresses in the steel reinforcement and, along with corrosion, leads to progressive steel rupture across the slab. Steel rupture was confirmed at 37 cracks (28 percent) and many others were suspected.

Experimental and analytical studies have shown that the steel at cracks is in tension and hence any yielding or rupture would cause the crack to open up. Possible contributing factors are loss of subbase support, which causes increased slab deflection and high stresses in the steel at the crack.
2.2.2. **Edge Punchouts**

Edge punchout is a name given to a small section of slab between two transverse cracks which has become depressed or punched downward relative to the surrounding pavement. Located at the edge of the slab, the edge punchout usually ranges from 2 to 6 ft. (0.6 to 1.8 m) across the lane, and may become "punched down" as much as 1-1/2 in. (3.8 cm) or more. They usually form between two closely spaced transverse cracks as shown in Figures 2.6 and 2.7. Excluding patch extensions, 16 of these distresses were observed (Figures 2.8 and 2.9).

It is believed that due to a large number of heavy load applications and loss of support along the edge from pumping, the aggregate interlock of cracks near the outer edge of the lane breaks down first, and gradually progresses toward the center of the lane. Due to this loss, the transverse stresses are increased and eventually a short longitudinal crack forms between the two transverse cracks about 24 to 72 in. (61 to 183 cm) from the edge of the slab and thus a small portion of the slab has no load transfer to the adjacent slab, and depresses into the base under continued loadings.

This type of distress may be caused by subbase pumping and loss of support. It was noted that at locations where this type of failure was repaired, the subbase was disintegrated. Whether this is due to moisture infiltration, freeze-thaw cycles, poor quality control during construction, a combination of these, or some other cause has not been determined.

2.2.3 **Failure of Slab Adjacent to a Patch**

The problem with patching at Manteno was not with the patches themselves, but with the adjacent slab. Nearly all of the 54 original patches
placed by state maintenance crews had given excellent performance (only 1 patch had deteriorated). However, the slab adjacent to 21 of the patches (39 percent) had deteriorated up to 4 feet (1.2 m) away from the patch. Figures 2.10 and 2.11 show a slab failure adjacent to a patch.

The failure of the CRCP near the patch is believed to be primarily caused by not extending the original patch to cover the entire existing deteriorated subbase (Figure 2.12). If the subbase beneath the CRCP surrounding the patch has disintegrated, the loss of support will cause higher stresses in the slab and result in punchout distress, as previously described.

At 23 locations there were wide transverse cracks near the patch, not always the first crack but possibly the third or fourth crack away from the patch end.

2.2.4 Failures Adjacent to a Joint in a Ramp

It was observed that a great percentage of the joints in the exit and merging ramps at the Manteno Interchange had caused a wide transverse crack to propagate across the CRCP. Many of these failures were patched, but a crack again formed in the PCC patch, as shown in Figure 2.13. It is believed that these cracks are caused by the amount of movement that exists at the joints (0.1 to 0.3 in. seasonally) and because the jointed ramps are tied to the CRCP this movement causes tensile stresses in the reinforcing steel that results in rupture. When the crack in the CRCP is forced open, repeated loading causes a breakdown of the aggregate interlock and shears the steel. This cause led to approximately 29 failures.

For more information on the CRCP distress types and their related mechanisms, the reader should refer to the initial report on this study (Reference 1).
2.3 Core and Deflection Study

A core and deflection study was undertaken in 1977. It was believed that the deflections and related core samples might help in determining the cause(s) for the high amount of distress in this project. NDT deflections were taken using a vibratory deflection device (Road Rater) owned by the Illinois Department of Transportation. The Road Rater, which can be seen in Figure 2.14 is capable of delivering a peak to peak vibratory load of up to 8 kips (35.5 kN). Deflections were taken continuously over a 200 foot (61 m) section and at four other sections within a test area of 650 feet (198 m) using a 5 kip (22.2 kN) peak to peak load at 15 cycles per second. The center of the loading plate was positioned 18 inches (46 cm) from the shoulder joint. The deflection measurements used are from the sensor in the center of the loading plate. Core samples were taken at four different locations that corresponded with certain deflections and distress areas. The cores were taken approximately 12 in. (30 cm) from the slab edge in an attempt to sample the second reinforcing bar. Figures 2.15 and 2.16 show the CRCP crack pattern, deflections, and the location of the cores taken.

Core No. 1 was taken at what first appeared to be a typical tight crack. It was noticed that this particular crack had double the deflection of the immediately adjacent cracks. Upon further examination of the crack, it was noticed that it had faulted approximately 0.04 in. (1 mm) and it was decided to take a core sample. It was very surprising to see the amount of corrosion and concrete deterioration that the core contained. Figures 2.17 to 2.20 show the crack before coring and the core that was taken. The increase in deflection at this crack was significant.

Core No. 2 was taken over a crack that typified the beginning stages of an edge punchout. The crack had faulted 0.08 in. (2 mm) and the very high deflections at the punchout were again significant. A short longitudinal crack had formed approximately 60 in. (152 cm) from the pavement edge and
the transverse crack spacing was 25 in. (63 cm). Again, there was extensive rebar corrosion and concrete deterioration contained in the core. Figures 2.21 to 2.24 are pictures of core no. 2.

Core no. 3 was taken over a very tight crack and it was believed that there should be no corrosion or deterioration. Figures 2.25 to 2.28 show this to be true. The crack became microscopic just below the surface of the slab and the core had to be broken apart to observe the steel. Although there is no corrosion of the rebar, it is interesting to note how far down from the top of the core that foreign material had infiltrated. As seen in Figure 2.27 it had almost reached the steel. The deflection at this crack is similar to that at other tight cracks.

Core no. 4 was taken over a crack that had spalled considerably but had not opened extensively as yet. Figures 2.29 to 2.32 show that the rebar had ruptured and that there was a void in the concrete around the rebar, just as in cores no. 1 and no. 2. The rebar was necked down from considerable corrosion.

These results indicate that when a crack shows any faulting at all (i.e., as little as 0.04 in. (1 mm)) it is reasonable to assume corrosion has been significant enough to reduce the cross-sectional area of the steel and/or there has been a considerable loss of aggregate interlock. At each of the three locations that had faulted, the deflections are markedly greater than at non-faulted cracks immediately adjacent. The high deflections may also be an indication of a void under the slab. Water was introduced into core no. 4 for 10 minutes from a hose connected to a water source. The water never filled up the core hole. Thus, there must have been extensive voids beneath the slab at this location. The cement aggregate mixture (CAM) subbase could not be cored at any of the four holes (it disintegrated during coring).
It is believed that deflections can be used to locate areas that may be in early stages of distress. However, for future study, it should be noted that the magnitude of load needed to get adequate information is relatively high. As shown in Figure 2.16 a vibratory load of even 2 kips (8.9 kN) does not adequately produce deflections to indicate the true loss of support. Thus it is recommended that loads of 5 kips (22.2 kN) or more be used so that distressed areas can be located.

2.4 Selection of Rehabilitation Portion

The overall construction section was 6.6 miles (10.6 km) in length. Available funds were very limited for the project. The distress survey shown in Figure 2.3 was used to determine the distribution on badly distressed areas with the intent of optimizing the rehabilitation efforts. This profile and the statistical summary given in Table 2.1 show that a large proportion of the distress is contained within the central portion of the project.

Therefore, a section of approximately 3.1 miles was selected for actual rehabilitation. Sufficient funds were available for a realistic rehabilitation effort over this length.

2.5 More Extensive NDT Deflection Testing

In the spring of 1978, prior to the beginning of construction, a more extensive nondestructive test deflection (NDT) program was conducted on the project. The NDT deflections were used in the construction layout of the project to locate patch boundaries and to select areas for undersealing. The deflections were also used in evaluating various maintenance procedures by comparing them with deflections taken after construction. Finally, the
deflections were analyzed statistically in an attempt to find a correlation with the amount of existing distress.

2.5.1 Description

The IDOT heavy load Road Rater was used to measure deflections. As previously illustrated in this chapter, a load in excess of 2-kips (8.9 kN) was required to produce a deflection indicative of the true slab support. Therefore, a 5-kip (22.2 kN) peak to peak load was used at a frequency of 15 cycles per second. The loading plate was centered 18 inches (46 cm) from the longitudinal shoulder joint in the truck lane.

Six test sections were selected along the project. The six sections referred to as Sections A through F, ranged from 2000 to 3000 feet (610 to 914 m) in length, and were chosen to facilitate the evaluation of treatments. Data on the six test sections is given in Table 2.2. Two of the sections were cement grout undersealed, two were asphalt undersealed and two were control sections in which the only treatment prior to the asphalt overlay was patching. Major distresses were repaired in all sections (and throughout the project) by patching (e.g. punchouts, slab failure adjacent to patches and selected wide cracks).

The deflection readings were usually taken at 50 foot (15 m) intervals, with the exception of one section where 25 foot (7.6 m) intervals were used. In addition to the deflections at regular intervals, concentrated deflection measurements were taken at every crack along certain portions of the sections and near selected distress within the test sections. This consisted of deflections at every transverse crack for 15 to 20 feet (4.6 to 6.1 m) on either side of the distress.
2.5.2 Deflection Profile Analysis

The deflection measurements were plotted to form deflection profiles. A typical profile is shown in Figure 2.33. The deflection profiles revealed some structural characteristics of the CRC slab that could not have been identified by a visual survey. The deflection profile at a severe distress not only showed a deflection peak at the point of the failure (such as an edge punchout) but also revealed the extent of the distress in the adjacent CRC slab. As can be seen in Figure 2.34, above average deflections continue on either side of the peak at the failure for several feet. This probably reflects damage to the subbase due to a combination of water intrusion and instability of the slab resulting in pumping. The ability of the deflections to reveal the extent of distress was used in determining patch boundaries. Where no failures existed, deflection peaks usually revealed underlying voids, as shown in Figure 2.35. This proved useful in selecting areas for undersealing. The NDT results also revealed a short length of the project that exhibited very low deflections indicating considerable structural strength relative to the other areas of the project. This particular section was a 900-foot (274 m) strip of CRCP in Section D which was discovered to be a 12-inch (30 cm) thick slab placed on grade, compared to the normal cross section of an 8-inch (20 cm) slab on top of a 4-inch (10 cm) cement treated subbase placed on grade. The deflection profile for this section, shown in Figure 2.36 shows significantly lower deflections for the extra thick area as compared to the adjacent normal thickness. There was no distress in this area.

The specific situations mentioned above are additional evidence in demonstrating the ability of the Road Rater deflections to indicate the structural condition of the CRCP. They also show the potential value of
deflections in the design of a rehabilitation project.

2.5.3 Results of the Statistical Analysis

The NDT deflections were analyzed statistically in an attempt to find a correlation between deflections and amount of distress. The original deflection data was modified before being analyzed. The concentrated deflections at distresses were eliminated, leaving only the deflections taken at regular intervals. It was believed that the regular interval deflections would indicate the overall condition of the particular section. Also, the regular interval deflections which coincided with an obvious distress were eliminated. For the analysis, the 12 inch (30 cm) CRCP was separated from the rest of Section D.

Since the 12 inch (30 cm) CRCP was located near the center of Section D, the adjacent pavement before and after it was also separated into separate sections. The entrance and exit ramp areas of Section E were eliminated. The remainder of Section E was split into two separate sections, one north of the ramp area and one south of the ramp area. All of the above modifications are contained in a data listing for the test sections in Table 2.3.

It was believed that the analysis would provide a correlation between one of the deflection statistics and existing distress. Since deflections are used as an indication of the strength of the CRCP/subgrade, the statistic thought to be the most significant in determining the amount of distress was the mean section deflection. However, results from the analysis, listed in Table 2.4, showed otherwise. There was no correlation between mean deflections and amount of distress. The variable showing the best correlation with amount of distress was the coefficient of variation (COV) of deflection. The graph of COV versus failures per 100 ft (30 m) is shown in Figure 2.37,
which displays an increasing amount of distress with increasing COV. A sharp increase in the amount of distress is especially noted when the COV increases much beyond 0.15. Of course this does not mean that average deflection is not important in determining the behavior of CRCP. It is quite probable that a section with relatively high deflections, such as section B, is in the process of deteriorating and may display an increase in distress rates and COV in the near future. However, it does seem quite logical that the COV would be a good indicator of the amount of distress. No matter whether mean deflections are high or low, a high COV reveals that there is a large variation of deflections in the section (including high peak deflections). These peaks reflect localized structural weaknesses in the CRCP, and these types of structural weaknesses lead to edge punchouts and deteriorated wide cracks from repeated traffic loadings. A simple analogy would be a chain only being as strong as its weakest link. This could further explain why a section displaying high deflections, but yet having a reasonable COV, does not possess a high amount of distress. If the entire section has overall average higher deflections, this will not result in as many distresses as high localized deflections.

One definite finding of the statistical analysis is that COV correlates with the amount of existing distress for this particular project. Whether this correlation is true for other CRCP projects is not known.
Figure 2.1. Accumulative Patching Requirements versus Accumulative 18-kip ESAL Traffic Loading

CRCP
I-57
6.6 Miles
(9 Years Data)

Log-Normal Distribution
\( \mu = 8.6568 \)
\( \sigma = 0.87 \)
Figure 2.2. Results of Crack Survey
Figure 2.3. Location of Distress Relative to Area Selected for Rehabilitation
Figure 2.4. Wide Crack

Figure 2.5. Wide Crack
Figure 2.6. Formation of Longitudinal Crack Between Two Closely Spaced Transverse Cracks - Early State of Edge Punchout Development
Figure 2.7. Extended Development of Edge Punchout
Figure 2.8. Edge Punchout

Figure 2.9. Edge Punchout
Figure 2.10. Distress Adjacent to a Patch

Figure 2.11. Distress Adjacent to a Patch
LONGITUDINAL CRACKING

SPALLING

FAULTING

SUBBASE DISINTEGRATED BEYOND PLANNED PATCH BOUNDARIES

X - MARKS INDICATE THE PLANNED BOUNDARIES OF THE PATCH

Figure 2.12. Disintegration of Subbase Beyond the Patch Boundaries
Figure 2.13. Crack in a Patch Adjacent to a Joint in the Ramp

Figure 2.14. IDOT Road Rater
Figure 2.15. Crack Pattern and Deflections at Cores No. 1 & 3
(1 in. = 2.5 cm, 1 ft. = 0.3 m, 1 kip = 4.45 N).
Figure 2.16. Crack Pattern and Deflections at Cores No. 2 & 4
(1 in. = 2.5 cm, 1 ft. = 0.3 m, 1 kip = 4.45 N).
Figure 2.17. Area Before Core No. 1 was Taken

Figure 2.18. Faulting of Crack Before Core No. 1 was Taken
Figure 2.19. Core No. 1 - Concrete Deterioration

Figure 2.20. Core No. 1 - Corrosion of Steel
Figure 2.21. Edge Punchout Before Core No. 2 was Taken

Figure 2.22. Width of Crack at Core No. 2
Figure 2.23. Core No. 2 - Concrete Deterioration

Figure 2.24. Core No. 2 - Corrosion of Steel
Figure 2.25. Core No. 3 - Very Tight Crack at Pavement Surface

Figure 2.26. Core No. 3 - Microscopic Crack Width at Mid-depth
Figure 2.27. Core No. 3 - Infiltration of Foreign Material

Figure 2.28. Core No. 3 - No Corrosion on Steel
Figure 2.29. Core No. 4 - Spalled Crack

Figure 2.30. Core No. 4 - Spall at Top of Core
Figure 2.31. Core No. 4 - Loss of Aggregate Interlock

Figure 2.32. Core No. 4 - Ruptured Steel and Concrete Deterioration
Figure 2.33. Typical Deflection Profile.
Figure 2.34. Deflection Profile at a Distress.
Figure 2.36. Low Deflections Identify 12 in. Thick CRCP Slab Instead of the 8 in. CRCP and 4 in. Cement Treated Subbase.
Figure 2.37. Coefficient of Variation (COV) Versus Failures per 100 feet
Table 2.1. Breakdown of Distress Shown in Figure 2.4.

<table>
<thead>
<tr>
<th>Type of Distress</th>
<th>No. in Total Project Length</th>
<th>% in Overlay Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original Concrete Patches</td>
<td>NBL 26</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td>SBL 18</td>
<td>77</td>
</tr>
<tr>
<td>Concrete &quot;add on&quot; patches</td>
<td>NBL 6</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>SBL 4</td>
<td>75</td>
</tr>
<tr>
<td>Existing Bituminous Patches</td>
<td>NBL 22</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td>SBL 14</td>
<td>86</td>
</tr>
<tr>
<td>Bituminous Patch Extensions to Concrete Patches</td>
<td>NBL 9</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>SBL 6</td>
<td>83</td>
</tr>
<tr>
<td>Punchouts (excludes patch extensions)</td>
<td>NBL 10</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>SBL 6</td>
<td>83</td>
</tr>
<tr>
<td>Wide Cracks with faulting and/or ruptured steel</td>
<td>NBL 76</td>
<td>74</td>
</tr>
<tr>
<td></td>
<td>SBL 54</td>
<td>55</td>
</tr>
<tr>
<td>Wide Cracks near Patches</td>
<td>NBL 17</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>SBL 6</td>
<td>50</td>
</tr>
<tr>
<td>Confirmed Steel Ruptures</td>
<td>NBL 19</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>SBL 17</td>
<td>59</td>
</tr>
<tr>
<td>Patches and Failure at Ramps</td>
<td>NBL 20</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>SBL 9</td>
<td>100</td>
</tr>
<tr>
<td>Section</td>
<td>Length (ft.)</td>
<td>Old Patches</td>
</tr>
<tr>
<td>---------</td>
<td>--------------</td>
<td>-------------</td>
</tr>
<tr>
<td>A</td>
<td>66600 + 2000</td>
<td>0</td>
</tr>
<tr>
<td>B</td>
<td>577 + 50</td>
<td>2000</td>
</tr>
<tr>
<td>C</td>
<td>535 + 50</td>
<td>2250</td>
</tr>
<tr>
<td>D</td>
<td>575 + 0</td>
<td>2800</td>
</tr>
<tr>
<td>E</td>
<td>620 + 0</td>
<td>3000</td>
</tr>
<tr>
<td>F</td>
<td>664 + 0</td>
<td>2000</td>
</tr>
</tbody>
</table>

Table 2.2: Initial Test Section for NDT Deflections.
Table 2.3. Modified Sections for Statistical Analysis of Deflections

<table>
<thead>
<tr>
<th>Section</th>
<th>Stations (ft.)</th>
<th>Length (ft.)</th>
<th>Old Patches</th>
<th>New Patches</th>
<th>Wide Cracks</th>
<th>Total Failures</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>686+00 → 666+00</td>
<td>2,000</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>B</td>
<td>577+50 → 555+00</td>
<td>2,250</td>
<td>0</td>
<td>9</td>
<td>7</td>
<td>16</td>
</tr>
<tr>
<td>C</td>
<td>535+00 → 563+00</td>
<td>2,500</td>
<td>1</td>
<td>16</td>
<td>8</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>(minus bridge)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D1</td>
<td>575+00 → 589+00</td>
<td>1,400</td>
<td>4</td>
<td>8</td>
<td>13</td>
<td>25</td>
</tr>
<tr>
<td>12&quot; Section</td>
<td>589+00 → 598+00</td>
<td>900</td>
<td>0</td>
<td>0</td>
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<tr>
<td>D2</td>
<td>598+00 → 625+00</td>
<td>700</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>E1</td>
<td>620+00 → 625+00</td>
<td>500</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>E2</td>
<td>632+50 → 640+00</td>
<td>750</td>
<td>3</td>
<td>2</td>
<td>2</td>
<td>7</td>
</tr>
<tr>
<td>F</td>
<td>664+00 → 690+00</td>
<td>2,600</td>
<td>0</td>
<td>9</td>
<td>4</td>
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Table 2.4. Results of Statistical Analysis of NDT Deflections

<table>
<thead>
<tr>
<th>Section</th>
<th>Total Failures</th>
<th>Length (ft.)</th>
<th>Failures/100 ft.</th>
<th>Deflection $\bar{x}$ ins. ($10^{-3}$)</th>
<th>$\sigma$ ($10^{-3}$)</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0</td>
<td>2,000</td>
<td>0.00</td>
<td>7.7395</td>
<td>1.076</td>
<td>0.139</td>
</tr>
<tr>
<td>B</td>
<td>16</td>
<td>2,250</td>
<td>0.71</td>
<td>10.0500</td>
<td>1.898</td>
<td>0.189</td>
</tr>
<tr>
<td>C</td>
<td>25</td>
<td>2,500</td>
<td>1.00</td>
<td>7.8182</td>
<td>1.518</td>
<td>0.194</td>
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<tr>
<td>D1</td>
<td>25</td>
<td>1,400</td>
<td>1.79</td>
<td>7.4120</td>
<td>1.754</td>
<td>0.236</td>
</tr>
<tr>
<td>12&quot; Section</td>
<td>0</td>
<td>900</td>
<td>0.00</td>
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CHAPTER 3: EXPERIMENTAL PATCHING PROGRAM

This chapter describes the experimental patching program. Construction procedures, performance, and costs of the experimental patches are presented. Also, the experimental patches are compared with the standard patching methods used in Illinois.

3.1 Description of Experimental Patches

Four types of experimental patches were used on this project. The term experimental is used because the patches do not follow the "Standard Specifications for Road and Bridge Construction" for the State of Illinois. The largest percentage of the experimental patching concentrated on lap length and splicing methods. A total of 89 patches were constructed in the project to repair edge punchouts, wide cracks, and other localized distress. Due to the large amount of distress, only the more severe were patched. Therefore, quite a few wide cracks where some or all of the reinforcement had ruptured were not patched.

3.1.1 20 Inch Tied Lap Splice Patch

Current Illinois specifications require a 36-inch (91 cm) tied lap splice for patches in CRCP that are greater than 10 feet (3 m) in length. This requirement appears to be overly conservative. In both field and laboratory testing, significantly shorter tied splices have been successfully used. Splice lengths of 15 to 20 inches (38 to 51 cm) have been implemented and are performing adequately on many patches in Illinois. Laboratory tests performed at the University of Maryland found that "the 20 inch splice length in No. 5 deformed-bar reinforcement was adequate in all respects at all concrete ages." (Ref. 3) Taking this information into
account, the project team selected a 20-inch (51 cm) tied splice as shown in Figure 3.1. This design was used for patches ranging from 5 ft (1.52 m) to 17 ft (5.18 m) in length.

The 20-inch (51 cm) tied splice patch has significant advantages over the standard 36-inch (91 cm) tied splice patch. First of all, the shorter splice length results in large savings of construction time which converts into a cost savings. The shorter splice requires an end portion breakout of 24 inches (61 cm) while the standard 36-inch (91 cm) splice requires an end portion breakout of 40 inches (102 cm). The end portion breakout is the distance between the partial depth cut and the full depth cut in a standard patch. The 40 and 24 inches (102 and 61 cm) lengths are 4 inches (10 cm) longer than the splice lengths to insure that the minimum splice length requirements are met. The concrete in this area is broken out by hand methods, and care must be taken not to damage the existing reinforcing steel. This particular part of the patch construction often proves to be the single most time consuming event (Ref. 2). By decreasing the area requiring hand breakout (40% decrease in end portion breakout area for a full lane width patch 10 ft (3 m) in length), the shorter splice length will result in significant construction time savings. This time savings could prove to be especially valuable in a rehabilitation project where a large number of patches are to be placed.

The 20-inch (51 cm) tied lap splice may also result in a material savings in the situations where a distress can be adequately repaired by a patch less than 10 feet (3 m) in length. Currently, most patches constructed in Illinois are 10 feet (3 m) or greater in length. This is due to the
specification restriction to leave the steel in place for a patch less than 10 feet (3 m) in length, and to remove the concrete by hand methods for the entire length of the patch without damaging the reinforcement. If a 20-inch (51 cm) splice were used, the minimum patch length could be reduced considerably to about 5 feet (1.5 m) assuming all of the existing distress was contained in the 12-inch (30 cm) center portion. This shorter patch length would require less new concrete and reinforcing steel resulting in material cost savings in addition to the aforementioned savings in construction time.

3.1.2 4-Inch Double Weld Splice

An alternative to the tied splice for steel reinforcement is the welded splice. The current Illinois specifications do not allow welding between new and existing reinforcement.

Welded splices have been tested both in the field and in the laboratory. One district maintenance crew with the capability of welding at the patch site uses a single 6-inch (15 cm) welded lap to connect new reinforcement to the existing steel. They have been very successful using this alternative splicing method. In laboratory tests run at the University of Illinois using maintenance crew field welded samples, the project team established that in static testing, double weld splices with effective weld lengths greater than 7 in. (18 cm) were able to develop full yield strength and 90% of ultimate strength for Grade 40 Rebar. A double weld splice contains two welds, one on each side of the rebar, for the length of the splice. Figure 3.2 shows the relationship between effective weld length and the ultimate axial strength for both single and double welds. A 4-inch (10 cm) double weld splice was selected as the design for the project. The 4-inch (10 cm)
double weld consisted of two 1/4 inch (0.6 cm) continuous welds, 4 inches (10 cm) in length, one on each side of the rebars (Figure 3.3). This is sometimes referred to as a double flare-v-groove weld. This design was used for patch lengths of 3 to 14.4 feet (0.9 to 4.4 m). The double weld was chosen over a single weld because it reduces the eccentricity of the load.

The design for the welded lap patch also calls for a 20-inch (51 cm) tied splice in the center to allow for temperature movements. (Figure 3.4) This will prevent the rebars from buckling before the concrete has time for adequate strength gain. It should also keep the welded lap splices from rotating due to eccentric tensile loads until the surrounding concrete can gain strength to confine these rotations.

For crews possessing welding capabilities, the 4-inch (10 cm) double weld splice patch should have the same advantages as the 20-inch (51 cm) tied splice patch but to an even greater extent. The increase in labor and equipment costs due to the welding may be more than offset by a further reduction in the end portion breakout from the current 40 inches (102 cm) for a standard patch, to 24 inches (61 cm) for the 20-inch (51 cm) tied splice patch, to only 8 inches (20 cm) for the 4-inch (10 cm) double weld patch. The 4-inch (10 cm) double weld splice patch could decrease the minimum length of a patch to 3 feet (0.9 m) for example, allowing an 8-inch (20 cm) end portion breakout and a 20-inch (51 cm) center portion.

As explained previously, the shorter lap length would mean a substantial savings in both construction time and materials.
This type of patch could be especially useful in repairing distresses that are contained in a short length such as wide open cracks where the steel has ruptured. Instead of a standard 10-foot (3 m) patch a double weld splice patch as short as 36 inches (91 cm) could be used, as long as the entire distress could be contained in the 20-inch (51 cm) center portion as earlier described.

3.1.3 Other Experimental Patches

Of the 89 patches constructed, 43 were 4-inch (10 cm) double weld splice patches and 40 were 20-inch (51 cm) tied lap splice patches. The 6 remaining patches were evenly divided between bituminous patches and concrete patches containing no reinforcing steel.

Bituminous patches do not restore the continuity of the CRC slab and provide no load transfer across the joint. This leads to potential distress in the CRC adjacent to the patch. The bituminous patch also provides no method of keeping the joints tight when movement of the CRC slab occurs due to temperature changes. Three bituminous patches were constructed in order that their performance could be monitored both as an alternative patching method and to see how they would perform in a rehabilitation project under an asphalt overlay.

A no-steel concrete patch, like the bituminous patch, does not restore the continuity of the CRC slab and provides no load transfer. Also, like bituminous patching, no-steel patching is cheaper than standard patching due to savings in construction costs. The no-steel patches were also placed in order to observe their performance in a rehabilitation project (in combination with an asphalt overlay).
3.2 Construction Procedures

The priorities in patch construction in a rehabilitation project are somewhat different than those for placing one or two patches in a single day as a part of maintenance operations. Therefore, the construction procedures (particularly the breakout methods) used in this project differ from the typical standard patch construction as described in an earlier report by Maxey, Darter, and Smiley (2).

3.2.1 Saw Cutting Patches

The general contractor used two methods of saw cutting patches: a gas powered concrete saw, and a ripper saw.

The hand operated concrete saw shown in Figure 3.5 was used to make the partial depth cuts at the ends of a patch. The cutting depth could be set on the saw, but was checked with a rule in order to insure that the steel was not being cut. The cutting was performed dry with high speed abrasive blades. Each blade would usually complete the cuts for two patches. An average of 35 minutes was required to make the two partial depth cuts for each patch. One laborer on the saw and one flagman were able to make the partial depth cuts for approximately 10 patches in an 8 hour day.

The full depth cuts were not placed in the conventional manner. A ripper saw, shown in Figure 3.6, was positioned at the center line of the patch. The blade then cut full depth and 4 ins (10 cm) wide as shown in Figures 3.7 and 3.8. Most of the full depth cuts were made at wide cracks with ruptured steel so there was less steel to cut through. Each cut could be made in about 15 minutes. Those that had to be cut through steel took longer. This method
was abandoned after a couple weeks due to considerable difficulty with the saw.

3.2.2 Breakout Methods

Several types of equipment were used on the project for the breakout of patches. A hydrammer was used extensively and for longer patches it seemed to be the most efficient. The hydrammer is a large hydraulic hammer, in this case mounted on a crawler backhoe as shown in Figure 3.9. The hydrammer was used to break up the surface of the concrete and was driven deep within the center breakout area. With this piece of equipment the break area was fairly well confined. In some cases, the hammer was used to within 6 inches (15 cm) of the partial depth cut with no apparent crack propagation across the cut into the adjacent pavement. When the hammer was driven deeply into the patch area much of the steel would be bent upward as can be seen in Figure 3.10. This was not a problem as long as the steel had been cut at the lap ends. The hydrammer breaks the patch fairly rapidly but there was a great deal of nonproductive time travelling between patches. Additional equipment necessary for operation include a large compressor and a truck to pull it. Labor includes the tractor operator and an oiler. On those patches where the hydrammer was used the breakup could be completed in 10 minutes or less.

On a few patches the breakup was done with a drophammer (Fig. 3.11). This method was much slower than the hydrammer because of the necessity to reposition the tractor a number of times during the breakout. The drophammer also caused more fracturing of the concrete than did the hydrammer.
This did not cause any problems if the steel had been cut. The drophammer has the following advantages: 1) operates with no additional equipment requirements, 2) only an operator is needed, and 3) travel time between patches is much shorter.

The patch breakout steps were done in an assembly line manner in that a given piece of equipment, such as the saw or hydrahammer, would move from patch to patch completing its specific task, then the patch area would remain unchanged until the next step was performed. This is an added option that exists for a rehabilitation project and not for maintenance patching. Each patch was done in a slightly different manner due to the different equipment options used for each step, however the basic procedures can be described.

One method was used on patches where no full depth cut was made, or when one ripper cut was made down the center of the patch along a wide crack. The basic steps are diagrammed in Fig. 3.12. After the partial depth cuts were made, the entire surface of the patch area was broken by the hydrahammer or the drophammer. Then a work crew, usually 3 to 4 men, would pick and shovel the loose concrete from the patch area onto the adjacent slab. This debris was later picked up by a loader and emptied into a dump truck, adding about 20 minutes per patch. Additional concrete would be broken up with pneumatic hammers and the pieces removed by hand and with shovels. The jackhammers would then be used between the reinforcing bars in the lap area. The broken concrete was removed and an air gun was used to blow out the finer debris to expose the steel. The steel could then be marked to the proper length and cut with a hand held gas powered saw, as shown in Figure 3.13.
The remainder of the concrete could then be removed with little disturbance to the steel in the end area which was to remain in place. The center of the patch could be removed fairly quickly but the laps took longer because of working between the bars, as it was important to be careful not to damage the bars. A small air hammer was used to trim the edges and when finished, the patch was blown out with an air gun. This procedure took between 2 and 2-1/2 hours. In comparison, according to Reference 2, the typical 10-foot (3 m) patch takes on the average 4 hours to break out. The equipment required includes a pavement breaker (either the hydrahammer or dropheammer), a compressor, usually two jackhammers, and hand tools. However, because there is a good chance that the steel will be damaged, and that the damage may be transmitted to the adjacent CRCP, this method is not recommended.

An improved method was used on patches constructed later in the project. The basic steps are shown in Figure 3.14. After the partial depth cuts were made, a crew would use air hammers on the ends of the patch to remove the concrete above the reinforcement. The steel was then cut to the proper length using the handheld, gas powered saw shown in Figure 3.13. The hydrahammer was then used to rapidly break up the concrete in the center of the patch. A backhoe was then used to remove the concrete and steel from the center of the patch, and load it into a dump truck. The concrete remaining beneath the steel in the end areas was then broken up with air hammers and the patch was finished by the same procedure as the other method. Using the backhoe for removal speeds the process considerably, however, care must be taken not to damage the subbase. This method was advantageous because cutting the steel prior to using the pavement breaker made the steel less
susceptible to damage and less vibration was transmitted to the adjacent CRC slab. The method was also quicker. The average time to complete the breakout was from 1-1/2 to 2 hours. The equipment requirements were the same as for the other method. The job foreman claimed that the contractor can save money by removing the concrete with the backhoe and suggested that a minimum patch size of 5 feet (15 m) be specified so that the shovel could be used.

Several variations on the two methods described were used, but these are the most representative of the work performed in breaking out the patches.

3.2.3 Reinforcement

The tied patches required that the new steel be tied in place with 20 inches (51 cm) minimum lap at both ends. The steel for the tied patches was measured and cut in the yard and then transported to the patch. The steel was tied in place by laborers with small wire ties, two ties per bar at each lap. The ties were about 4 inches (10 cm) long with a loop in each end. A small metal hook was put through the loops and the hook was then used to twist the tie tight. The method was quick and easy. The tied patches were generally larger than the welded ones and chairs were often used to support the steel. Because the ties allow for longitudinal movement of the steel, problems of bending or breaking of the steel due to temperature movements of the CRCP before concrete placement were not experienced. It required two laborers 15 to 20 minutes to complete the tying procedure on an average patch.
For the welded patches the special provisions of the project required 8 inches (20 cm) of weld per bar, one-half on each side of the bars. The bars could be stacked vertically provided that 2.5 inches (6 cm) minimum clearance to the surface was maintained. A 20-inch (51 cm) tied lap in the center of the patch was required as shown in Figure 3.15.

The bars were welded by one pass on each side of the bars creating two continuous 4-inch (10 cm) welds at each end. The bars for a patch were pre-cut as they were for the tied patches. The existing steel has to be straightened in order for the new bars to lay flat on the old ones which was necessary to achieve a good, fast, high quality weld. With the old bars straight the welder would set the new ones on top and secure them with vise grips. It was soon found to be more efficient to have laborers tie the bars in place and then let the welder make all the welds. This is quicker and would save time and money. A Lincoln Arc Welder SA 2020 and E70XX electrodes were used. An average of 1 hour was spent on welding the rebars for one patch.

The first three welded patches contained reinforcing bars welded on both ends without a 20-inch (51 cm) tied lap in the center. These patches were allowed to remain overnight without being filled with concrete. The next morning the reinforcement was found to have broken. In some cases the welds broke (Figures 3.16 and 3.17) and in others the reinforcing bars broke (Figure 3.18). The breakage was due to contraction of the CRCP during the night. The daily movement was then monitored at several open patches where steel reinforcement had not yet been placed. The differential movements measured overnight across a patch were found to be up to 0.4 inch (1 cm) depending on the adjacent lane condition. Those patches with breakage were
corrected and the proper center japping procedure was used on all remaining patches as called for in the special provisions. The only other problem encountered was at patches with narrow widths. Sometimes a new rebar welded at one end would butt against the existing bar at the other end. (Figure 3.19) This would cause the bar to bow up in the center when temperature movements of the CRCP occurred. Therefore, it was important to check that this situation did not occur.

For the bituminous patches all the steel was removed from the patch area. However, 8 to 10 inches (20 to 25 cm) of lap steel was left at the ends of the no-steel concrete patches.

3.2.4 Concrete Placement

The concrete placement was similar to that of a typical patch. The concrete was a rich 7-bag mix with an air entraining additive. The average air content was 5 to 6% and the slump was usually around 3 inches (7.6 cm). When the concrete arrived, the patch was prewetted and a cement slurry was brushed along the patch edges. The concrete was spread from one end of the patch to the other in one lift (Figure 3.20). Consolidation of the concrete was aided by use of a vibrator, as shown in Figure 3.21, especially at patch ends and around the reinforcing bars. The concrete was then struck off and finished. The process of pouring and finishing the patch usually took somewhere between 10 and 15 minutes with the exception of a couple of extra long patches. A liquid membrane curing compound was then applied.

Several 6 x 6 x 30 inch (15 x 15 x 76 cm) beams were taken in order
to sample the flexural strength of the concrete used. Using center point loading the flexural strength at 24 hours averaged about 400 psi, with a low of 300 psi and a high of 500 psi. All of the 7-day tests showed flexural strengths above 600 psi. The patches reached the specified flexural strength of 600 psi before opening to traffic.

The asphalt concrete for the bituminous patches was placed in three equal lifts, and compacted after each lift.

3.3 Patch Evaluation

Three methods were used in evaluating the experimental patches. A short term visual evaluation was conducted from the time the patches were constructed until they were covered by the asphalt overlay. Non-destructive testing using a Road Rater was used to evaluate the structural effect of the various patch types. A reflective crack survey was conducted one year after construction in order to determine if the patch joints maintained adequate load transfer and minimal horizontal opening.

3.3.1 Visual Evaluation

The visual evaluation consisted of regular inspections of every patch beginning immediately after construction. Any cracking found in a patch was measured, recorded, and photographed. The inspections continued until the asphalt overlay was placed, with the final result being a very comprehensive set of records of all the distresses found in each individual patch. These records were then used to evaluate various construction variables and determine which were most significant as far as development of cracking.

The visual evaluation revealed four variables that appeared to have significant effects on the amount of cracking in a patch. These variables
were: 1) the time the patch was poured (A.M. or P.M.); 2) the type of 
ap used (welded or tied); 3) the adjacent lane condition (tight or wide 
rears); and 4) the length of the patch. The data used in evaluating 
these variables is contained in Table 3.1.

There was a large difference in the amount of longitudinal and trans-
verse cracking between patches placed in the morning and those placed in 
the afternoon (Figure 3.22). Over 46% of the A.M. patches were cracked 
as compared to just 7.5% of the P.M. patches. When the cracking is sepa-
rated into longitudinal and transverse cracking, 36% of the A.M. patches 
contain longitudinal cracking and 18% of the A.M. patches contain trans-
verse cracking. Of the 7.5% of the P.M. patches showing cracking, all 
contained longitudinal cracking only. Therefore, the A.M. patches not 
only contain more total cracking than the P.M. patches, but they also are 
the only ones containing transverse cracking. It is believed that the 
transverse cracks are caused by contraction of the CRCP resulting in tensile 
stresses during the first night (and possibly succeeding nights) after the 
patch is placed, and that they are more likely to occur in patches poured 
in the A.M. because the concrete is much "stiffer" (but still very weak in 
tension) during the evening hours when the CRC is contracting, than the 
concrete poured in the P.M. When transverse cracking develops this early 
in the patch life, it is considered to be a more serious problem than a 
typical CRCP crack that forms due to shrinkage stresses. When a transverse 
crack forms early in the patch life it often begins to break down fairly 
soon. The vertical shear stresses from traffic loading destroy the cement 
paste-aggregate bond of the aggregates near the crack. The net result is 
rapid loss of aggregate interlock at the crack and eventual deterioration 
of the patch (2).
Cracking in the no-steel patches support the results of the tied and welded patches concerning time of pour. Two of the three no-steel concrete patches developed transverse cracks, and both were placed in the A.M. The one P.M. patch did not crack. The no steel patches also support the belief that the A.M. patches that crack transversely do so in tension. In the two patches that cracked transversely, the crack appeared at one end just beyond 8 to 10 inches (20 to 25 cm) of reinforcement left in the ends of the patch (Fig. 3.23). The reinforcement transfers tensile stresses to the concrete through bond when the adjacent CRC slab begins to contract at night. At the end of the 8-10 in (20-25 cm) reinforcement, the concrete is not strong enough in tension to carry the tensile force and a crack forms through the entire depth of the patch as can be seen in Figure 3.24. Of course, this situation is even more severe when the adjacent lane condition is a wide crack with ruptured steel forcing the patch to carry the stresses for the full pavement width, as will be discussed later.

The longitudinal cracking is believed to be caused by the expansion of the CRCP causing compressive stresses during the P.M. after the A.M. pour. Most of the longitudinal cracking was associated with patches poured in the A.M. and occurred directly over the rebars.

The visual evaluation also revealed a difference in the amount of total cracking between tied lap patches and welded lap patches. The inspections found 49% of the welded patches contained some type of cracking compared to 17% of the tied lap patches (Fig. 3.25). However, when the cracking was separated into longitudinal and transverse, the difference between the tied lap and welded lap patches was found to be in the amount
of longitudinal cracking. The tied and welded lap patches contained almost equal amounts of transverse cracking, 15% and 14% respectively. The large difference in longitudinal cracking, though not totally explainable, was in some part due to bowing of the bars in the welded patches where the reinforcing bars butted into one another as was explained in the construction procedures.

When the adjacent lane condition was related to patch cracking the results were not at all surprising. Patches placed directly opposite to a wide crack in the adjacent lane showed a very high amount of cracking (75%), while those patches adjacent to sound tight cracks showed a much smaller amount (24%) of cracking (Fig. 3.26). The problem with patching adjacent to a wide crack is that the patch must withstand the unrestrained longitudinal movement of both lanes. This large movement often causes a transverse crack to form in the new patch directly opposite the wide crack in the adjacent lane.

The only known solution to the problem of patching adjacent to wide cracks is to limit the longitudinal movement of the slab due to temperature changes. This can be done by either patching during periods of moderate temperature change or attempt to control slab temperature change by covering the slab with insulation or by spraying with water.

The fourth variable which appeared to be significant was patch length. The patches were divided into those greater than 5 ft (1.5 m) long and those less than or equal to 5 ft (1.5 m). The short patches (≤ 5 ft) displayed a higher percentage of cracking than the long patches. This difference in cracking was found however, to be closely related to the type of
lap. A large percentage of the short patches were welded patches, therefore the variable of patch length is just repeating the results found from comparing tied patches to welded patches.

The worst possible combination of patching conditions occurs by selecting the most critical level of each of the four variables. The patch condition that gives the highest percentage of total cracking and transverse cracking, is a short welded patch, poured in the morning adjacent to a wide crack in the other lane.

After individual comparisons were made of each variable using the data in Table 3.1, multiple regression analyses were conducted using all the individual variables, plus all two and three way combinations. The regression analyses were not conducted to develop equations to predict the amount of cracking, but primarily were used to find which variables were most significant in determining the performance of a patch. Separate analyses were completed for longitudinal and transverse cracking.

The adjacent lane condition was found to be the most dominant variable related to transverse cracking. The next most significant was the two way combination of adjacent lane condition and time of pour. These results are similar to the comparisons made previously.

For longitudinal cracking one particular combination was by far the most significant. The combination of patch length, type of splice, and adjacent lane condition was much more significant than any of the others in relating to longitudinal cracking.

Several conclusions can be made from the visual evaluation. Patches should be poured in the afternoon to avoid the subsequent cracking that usually results in patches poured in the morning. The transverse cracking
is believed to be caused by tensile forces. The adjacent lane condition is the most dominant variable affecting patch performance. When patching a lane that is adjacent to a wide crack (ruptured steel), efforts should be made to control the temperature change of the slab, or the patching should be done during period of moderate temperature changes. It is recommended that the lowest truck volume lane be patched first. Thus even if this patch cracks the traffic loadings will be fewer. And finally, the tied and welded patches performed equally well.

3.3.2 NDT Evaluation

The NDT consisted of deflection readings taken by the IDOT Road Rater before and after patch construction. The deflections could then be compared in order to evaluate the structural effect of the different types of patches.

From the deflection profiles, both the welded lap and tied lap patches restored the continuity of the pavement (Figure 3.27).

On the other hand the bituminous patches made no contribution towards re-establishing continuity at the patch area and did not limit deflections as can be seen by the profile shown in Figure 3.28. The effect of the loss of continuity will be observed over future years at these patch areas.

Figure 3.29 shows that one no-steel patch did reduce the deflection peak but the others were not checked. These areas will also be observed in the future.

3.3.3 Reflective Crack Survey

Reflective cracking from the concrete slab is the major cause of deterioration of an asphalt overlay. One year after the asphalt overlay was
constructed a crack survey was conducted. One object of the study was to see which patch joints, if any, could be identified as the underlying cause of reflective cracks.

Five patches were identified as the underlying cause of reflective cracks. Included in these five patches were all three of the bituminous patches placed on the project. One example of this is shown in Figure 3.30. Of the other two patches, one was a no steel concrete patch and one was a tied splice patch. Both the no steel patch and the tied splice patch had transverse cracks prior to the overlay. The crack in the tied splice patch was adjacent to a joint in a ramp which must have forced an open crack in the patch. Therefore, one year after construction, there are almost no reflective cracks near the reinforced concrete patches, but all bituminous patches had reflective cracks at their ends.

3.3.4 Cost Analysis

The purpose of the experimental patching was to find alternative patching methods that performed adequately and reduced cost. A cost analysis was conducted to compare the bid costs of the tied and welded lap patches to average costs calculated for a standard 10 x 12 foot (3 x 3.6 m) patch constructed by Illinois DOT maintenance crews and by private contractors. The bid costs for the experimental patches and the average cost for a standard 10 x 12 patch by a private contractor include profit and overhead, while the average cost calculated for district maintenance crews does not. From Reference 2, the average cost for a 10 x 12 foot (3 x 3.6 m) patch for IDOT district maintenance crews in the year 1977 was $85.00 per square yard ($102 per square meter), with a range of values from $67 to $120 per square yard
($80 - $144 per square meter). The average total cost for the 10 x 12 foot (3 x 3.6 m) patch would be $1133.33. With traffic control costs included, private contractors bid prices for a 10 x 12 foot (3 x 3.6 m) patch are in the range of $100 to $110 per square yard. If an average cost of $105 per square yard is used the total cost is $1400. As can be seen in Table 3.2, the unit bid costs for the tied lap and weld lap patches don't appear to represent any savings over the costs mentioned above. The bid costs for tied lap patches and welded lap patches are $89.98 per square yard and $105 per square yard respectively. However, these are unit costs. In the case of a narrow, well confined distress a noticeably large savings could result if the minimum width of these patches was adequate. Using minimum costs for each type of patch shown in Table 3.2, a tied lap patch would save over $500 compared to maintenance costs. A welded lap patch would save $700 compared to maintenance costs, or close to $1000 compared to private contractor bid costs. The savings for maintenance crews could be even larger than shown considering that the bid costs include overhead and profit, and the maintenance crew costs do not. In a rehabilitation project such as this one where there were many wide cracks, the savings could amount to quite a large sum if these relatively narrow distresses could be covered by a smaller type patch.

For example, the use of the small patches saved an average of $583 per patch, or a total of $48,386 on this project. The savings were calculated using a cost of $1400 for a 10 x 12 patch compared to an average cost of $817 for the smaller patches actually placed. The average cost of $817 was simply the total cost of all welded and tied patches divided by the number of these patches.
The utilization of these patches by IDOT maintenance crews may also result in significant cost savings.

The cost analysis has shown that the experimental patches are capable of saving money. The 20-inch (51 cm) tied lap should save money in all patching uses. If overhead and profit were not included its unit cost would more than likely be lower than the average unit cost of a standard patch constructed by a district maintenance crew. The welded lap patch with its higher unit cost, will save money when it is used in situations where it can take advantage of its much smaller minimum width.
Figure 3.1. 20-Inch Tied Splice Patch
Figure 3.2. Effective Weld Length Versus Axial Strength for Welded Splices
Figure 3.3. Weld Design Used in the Welded Splice Patch
Figure 3.4. 4-Inch Double Weld Splice Patch
Figure 3.5. Sawing of Partial Depth Cut

Figure 3.6. Ripper Saw
Figure 3.7. Full Depth Cut Made by Ripper Saw

Figure 3.8. Full Depth Cut Made by Ripper Saw
Figure 3.9. Hydrahammer

Figure 3.10. Reinforcing Steel Bent Upward from Mechanical Breakout
Figure 3.11. Drophammer
Step 1. Surface Breakup by Mechanical Methods (drophammer or hydrammer).

Step 2. Hand Removal of Debris from Step 1, Breakout of Remaining Concrete with Jackhammers Down to the Steel, Cut Steel to Proper Length.

Step 3. Mechanical Breakup of Center Breakout Area, Hand Breakup of End Portion Breakout Area, Mechanical or Hand Removal of Debris from Center Breakout Area Depending on the Length of the Patch, Hand Removal of Debris from End Portion Breakout Area.

Figure 3.12. Initial Breakout Method (Not Recommended)
Figure 3.13. Cutting Reinforcement with a Hand Held Gas Saw
Step 1. Breakout in End Sections Using Jackhammers down to the Steel. Cut all Steel Rebars Across the Patch at the Proper Lap Length.


Figure 3.14. Improved Breakout Method (recommended).
Figure 3.15. 20-Inch Tied Lap in the Center of the Welded Splice Patch

Figure 3.16. Broken Welds When No Tied Lap was Placed in Center of Patch and Patch was Left Open Over Night.
Figure 3.17. Broken Welds (see Figure 3.16).

Figure 3.18. Broken Bars (see Figure 3.16).
Figure 3.19. The Shaded Rebar, Which is Welded to Existing Reinforcement of the Right End of the Patch, Butts Against Existing Reinforcement at the Left End of the Patch.
Figure 3.20. Placement of Concrete

Figure 3.21. Vibrating the Concrete
Figure 3.22. Time of Pour Versus Percent of Patches Cracked
Figure 3.23. Lap Steel Left in No-Steel Patch

Figure 3.24. Transverse Crack Through Full Depth of Slab Occurring at the End of 12 ins. of Reinforcement as Shown in Figure 3.23. No Tied Steel was Placed in Patch.
Figure 3.25. Type of Splice Versus Percent of Patches Cracked
Figure 3.26. Adjacent Lane Condition Versus Percent of Patches Cracked
Figure 3.27. Deflection Profiles Before and After Placement of a Welded Splice Patch
Figure 3.28. Deflection Profiles Before and After Placement of a Bituminous Patch
Figure 3.29. Deflection Profiles Before and After Placement of a No-Steel Patch
Figure 3.30. Reflection Cracking at Bituminous Patch Boundaries
Table 3.1. Cracking Data for Patches.

<table>
<thead>
<tr>
<th>TIME OF POUR</th>
<th>TYPE</th>
<th>LENGTH</th>
<th>TIED SPLICE (40)</th>
<th>WELDED SPLICE (43)</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>SHORT (13)</td>
<td>LONG (27)</td>
<td>SHORT (31)</td>
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<td></td>
<td></td>
<td>(6)</td>
<td>(12)</td>
<td>(17)</td>
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<tr>
<td></td>
<td>AM</td>
<td>42</td>
<td>a. 1, 17%</td>
<td>a. 3, 25%</td>
<td>a. 9, 53%</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>b. 1, 17%</td>
<td>b. 1, 8%</td>
<td>b. 9, 53%</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>c. 0, 0</td>
<td>c. 3, 25%</td>
<td>c. 0, 0</td>
</tr>
<tr>
<td></td>
<td>PM</td>
<td>25</td>
<td>a. 0, 0</td>
<td>a. 2, 18%</td>
<td>a. 0, 0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>b. 0, 0</td>
<td>b. 2, 18%</td>
<td>b. 0, 0</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>c. 0, 0</td>
<td>c. 0, 0</td>
<td>c. 0, 0</td>
</tr>
<tr>
<td></td>
<td>AM</td>
<td>14</td>
<td>a. 0, 0</td>
<td>a. 1, 50%</td>
<td>a. 10, 91%</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>b. 0, 0</td>
<td>b. 0, 0</td>
<td>b. 7, 64%</td>
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<td></td>
<td>c. 0, 0</td>
<td>c. 1, 50%</td>
<td>c. 6, 55%</td>
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<tr>
<td></td>
<td>PM</td>
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<td></td>
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<td></td>
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<td>c. 0, 0</td>
<td>c. 0, 0</td>
<td>c. 0, 0</td>
</tr>
</tbody>
</table>

(   ) - Indicates number of that type of patch
a. Number of patches cracked; % of that type cracked
b. Number of patches cracked longitudinally; % of that type cracked longitudinally
c. Number of patches cracked transversely; % of that type cracked transversely
<table>
<thead>
<tr>
<th>PATCH TYPE</th>
<th>UNIT COST</th>
<th>MINIMUM DIMENSIONS</th>
<th>MINIMUM AREA</th>
<th>MINIMUM COST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Illinois Patch (District Maintenance)</td>
<td>$85.00/yd²</td>
<td>10' x 12'</td>
<td>13.33 yd²</td>
<td>$1133.33</td>
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<tr>
<td>Standard Illinois Patch (Private Contractors)</td>
<td>$105/yd²</td>
<td>10' x 12'</td>
<td>13.33 yd²</td>
<td>$1400.00</td>
</tr>
<tr>
<td>20 Inch Tied Lap Splice</td>
<td>$89.98/yd²</td>
<td>5' x 12'</td>
<td>6.67 yd²</td>
<td>$599.87</td>
</tr>
<tr>
<td>4 Inch Double Weld Splice</td>
<td>$105/yd²</td>
<td>3' x 12'</td>
<td>4.0 yd²</td>
<td>$420.00</td>
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</tbody>
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CHAPTER 4
CEMENT GROUT UNDERSEALING

4.1 Introduction

Cement grout undersealing refers to the pressurized pumping of a cement grout slurry beneath the slab and/or subbase to reduce pumping, decrease deflections, and restore uniform support to the slab. It does not refer to raising the slab. Excessive lifting of the slab can be harmful to the performance of the CRCP structure by causing the formation of new voids that may lead to high stress concentrations in the slab.

Several factors made it quite obvious that voids existed under the slab and/or subbase. Distress such as edge punchouts and deteriorated transverse cracks indicated localized loss of support. Deflections measured with the IDOT heavy load Road Rater (called Non-Destructive Testing, NDT) revealed the existence of voids, and signs of pumping were evident throughout the project. If nothing was done to fill these voids, water would continue to collect and pumping action would continue to remove the fine material. The size of the voids would increase, causing localized loss of support which is a major factor in accelerating the deterioration of CRCP (Ref. 1) as was illustrated in Figure 2.2. Regardless of whether or not the project was overlaid, action had to be taken to fill the voids or distress would continue to occur at an increasing rate.

Cement grout undersealing was one technique chosen to fill the voids in order to arrest pumping, reduce deflections, and return uniform support to the slab. If the undersealing was successful in doing this, the life of the pavement would be extended.
4.2 Cement Grout Undersealing Procedures

The first step was to select the areas to be treated. Two types of cement grout undersealing were used: blanket grouting and localized grouting. Large areas that displayed overall poor performance were chosen for blanket grout undersealing. An area selected for blanket treatment was grouted completely from one end to the other, regardless of the local conditions of the slab. In localized grouting only the areas showing pumping or localized loss of support were treated, plus other special areas including adjacent to patches and roadway cross pipes where sags have occurred, and lug areas. Pumping areas and special areas were easily located by a visual survey, but this was not the case with areas having localized loss of support. The Road Rater was used to locate areas with higher than average deflections indicating localized loss of support. Some type of heavy deflection device is a necessity to locate areas for selective grout undersealing.

In this project, selective and blanket grouting were used in both the truck and passing lanes. This is usually not the case because pumping is dependent on traffic loadings and usually occurs primarily in the truck lane. Therefore, selective grouting is recommended predominantly for the truck lane unless there are signs of local loss of support and pumping in the passing lane. If this is the case then areas in both lanes should be treated.

Once the areas for cement grout undersealing were selected, a hole pattern was chosen. An optimum hole pattern is one which provides complete coverage of grout with minimum overlap. Several patterns were tested and the amount of material pumped was monitored. The pattern which was
eventually chosen had three holes every 14 lineal feet (3.4 m) of pavement, with the centerline hole staggered (Figure 4.1). This pattern seemed to yield a maximum of pumped material for a minimum number of holes used.

This pattern appeared to give good coverage when a trench was opened along the slab for the placement of underdrains. A thin layer of pressure grout material was nearly continuous along the edge of the slab either between the slab and subbase, or beneath the subbase. However, a closer spacing may give even better coverage, especially in the case of selective pressure grouting in badly pumping areas. Since selective grout undersealing usually treats localized loss of support, a closer spacing than the one used for blanket grouting may be beneficial. This would insure that the areas containing the largest voids would be completely filled without raising the pressure to the point where the slab would be lifted. A small reduction of spacing from 14 feet to say 10 feet would result in little additional total cost because the cost of drilling is relatively low compared to the cost of the cement grout slurry for average hole spacings.

The holes were 2 inches (5 cm) in diameter and were made with a drill mounted on a forklift shown in Figure 4.2. The holes were drilled to a depth of 18 to 24 inches (46 to 61 cm) in order to fill voids at both the subgrade-subbase interface and the subbase-slab interface. The drill was quite fast (approximately 20 seconds per hole) and had the capability to cut through reinforcement. This is a valuable capability. When a drill without this capability hits steel reinforcement it must be withdrawn and a new hole started, plus the unfinished hole must be filled with cement grout. This problem was encountered in the drilling described in Chapter 5.
After the holes were drilled the cement grout slurry was prepared. The slurry was a mixture of Type 1 Portland Cement, limestone dust, and water. The special provisions called for a limestone dust meeting the following gradation requirements:

- Passing No. 30 Sieve 100%
- Passing No. 100 Sieve 92 ± 8%
- Passing No. 200 Sieve 82 ± 18%

It also was to be free of harmful impurities and foreign material.

A slurry consistency was required that would pump easily, be capable of filling small voids, and still be able to develop adequate strength and durability. The mix design selected was six (6) 94-pound bags of Type 1 Portland Cement, 24 cubic feet (0.68 m³) of limestone dust, and 90 gallons of water yielding approximately 1-1/2 cubic yards (1.15 m³) of slurry. This mix formed a slurry which flowed well. The strength gain of the cement grout slurry with time is shown in Figure 4.3. The strength characteristics of this mix are comparable with most cement grout slurries used for cement grout undersealing.

All of the mixing and proportioning was done on the grout plant truck unit. First, the water and cement were added to one of the mixers. The grout plant contained two mixers allowing a batch to be mixing while another was being pumped. The bags of cement were loaded on the grout plant by a front end loader, and were then added to the mixer by hand. After the cement and water were loaded into the mixer, the limestone dust was added. The limestone dust was loaded onto a conveyor by a front end loader, which carried it to the mixer. The slurry was allowed to mix thoroughly, and was then ready for pumping.
The grout packer nozzle was inserted into the hole in the slab and the pumping was started. Pavement movements were monitored by the apparatus shown in Figure 4.4. The gauge indicated the amount of relative movement of the slab near the hole with respect to the shoulder. Pumping was continued until either there was (1) a significant movement of the gauge, or (2) slurry was forced up through a nearby crack/joint or drilled hole. Vertical slab movements were kept below 0.10 inches (2.5 mm). As was mentioned before, lifting of the slab should be kept to an absolute minimum. Even 0.10 inches (2.5 mm) may be more than necessary. In some cases water was forced out at the longitudinal shoulder joint. When this occurred pumping was continued until slurry of a thicker consistency was forced out through the joint. It should be noted that the grout slurry has the capability to gain strength in the presence of water. This is a very valuable capability because it is quite likely that water will be present in some voids.

After pumping, the nozzle was removed and a wooden plug was immediately inserted into the hole. When the back pressure had subsided so that the slurry would not be forced out the hole (usually about 10 to 15 minutes after the completion of pumping) the plug was removed and the hole was filled with a sand cement grout.

4.3 Evaluation of Pressure Grouting

A complete evaluation of pressure grouting cannot be made for several years, since only actual pavement performance will determine its cost effectiveness. One evaluation can be made by comparing NDT deflections taken after grouting with the NDT deflections before grouting. Also, an approximate cost effectiveness analysis is presented.
4.3.1 Deflection Evaluation of Pressure Grouted Areas

Approximately three weeks after the completion of the pressure grouting, NDT deflections were taken at all the locations measured in the initial NDT analysis described in Section 2.5. The IDOT Road Rater was again used, and all loading conditions were duplicated as closely as possible. These deflections could be compared with the initial deflections to assess the structural effect of the cement grout undersealing.

It was known that deflections fluctuate substantially with changes in such variables as the temperature of the slab and the moisture conditions of the subgrade, among others. Therefore, it was necessary to normalize the deflections before a valid comparison could be made. A normalization factor was computed for the second set of deflections by equalizing the means for the control sections for the initial and second sets of deflections. Since the control sections received no treatment other than patching, prior to the overlay, any difference between the initial and second set of mean deflections for these sections would be due to changes in seasonal variables. This normalization factor would also include the effect of the additional traffic loadings accumulated since the initial deflections. The second set of mean deflections for the control sections showed a 12% decrease over the initial mean deflections. Thus the second set of deflections was multiplied by a normalizing factor of 1.12. A valid comparison of the first and second sets of mean deflections could now be made.

Examination of the deflection profiles produced no clear conclusions. For some profiles, such as the one in Figure 4.5, the deflections were reduced after grouting, but this was not usually the case. More typical profiles are shown in Figures 4.6 and 4.7 with the two lines crossing back and forth over each other.
The statistical analysis consisted of t-tests run between the means of the two sets (before and after grouting) of deflections for each of the cement grouted sections. Initially, the overall analysis showed no significant change in the mean deflections after grouting. However, when deflections at points from the initial NDT analysis greater than the mean ($\geq 7 \times 10^{-3}$ inches) were separated and compared to those at the same points after grouting, there was a significant decrease in deflection (9% decrease in mean deflection). When even larger deflections ($\geq 8 \times 10^{-3}$ inches) were separated and tested, the decrease after cement grouting was even more significant (16% decrease in deflection).

From these analyses it was concluded that cement grouting was effective in decreasing higher than average deflections, but had little or no effect on average or lower than average deflections. This meant that blanket cement grouting was having little or no effect on average or lower than average deflection areas, resulting in a possible inefficient use of material. It indicates that the pumping of grout material under a sound pavement is of no benefit as far as reducing deflections. However, the forcing of pressure grouting slurry into an area where small voids exist may reduce future pumping.

The main finding of this first analysis is that selective cement grouting is a more efficient way to apply pressure grouting material than blanket grouting along the entire project.

A second set of statistical analyses was completed on NDT deflections taken after the addition of the asphalt overlay. T-test comparisons were conducted between the initial deflections and the third set of deflections
taken after the overlay. This allowed a comparison to be made between the
effect of the asphalt overlay on the control sections and the effect of the
combination of cement grouting and asphalt overlay. The results of the
analyses were dominated by the effect of the asphalt overlay on reducing
deflections. There was no significant difference between the reductions in
deflections for the control sections and the cement grouted sections.

4.3.2 Cost Analysis

The first part of the cost analysis is concerned with comparing blanket
with selective cement grout undersealing.

Using bid costs, the blanket unit cost of grouting was $3.35 per square
yard of CRCP in the two traffic lanes for the given hole pattern. The unit
cost of selective grouting was more expensive than blanket grouting because
all of the areas chosen for selective grouting possessed higher than average
deflections which required more grout slurry. The average amount of slurry
pumped under the CRC slab in selective grouting was 0.50 cubic feet per square
yard of pavement, while blanket grouting averaged 0.40 cubic feet of slurry
per square yard of CRCP. When a reasonable range for coverage is used, say
10% to 40%, selective grouting is much cheaper than blanket grouting. At
10% coverage selective grouting costs $0.40 per square yard ($0.48 per sq.
m.) of CRCP, while for 40% coverage the cost was $1.60 per square yard ($1.91
per sq. m). Even at 40% coverage, the cost of selective grouting was less
than half the $3.35 per square yard ($4.00 per sq. m) cost for blanket grout-
ing. Cost values for blanket grouting and various coverages of selective
grouting, separated into their basic components, are shown in Table 4.1.
The second part of the cost analysis is devoted to determining the extra pavement life that the different grouting treatments must provide to be cost effective. The following analysis makes several simplifying assumptions, but is adequate for its intended purpose. The basis used for determining cost effectiveness is the annual cost of the asphalt overlay, since its life can be estimated fairly accurately using the IDOT design nomograph for asphalt composite pavements. The expected life of a 4-inch (10 cm) asphalt overlay over an 8-inch (20 cm) CRCP is approximately 11 years for the expected traffic conditions. The 1978 cost of the asphalt is $5.75 per square yard ($6.88 per sq. m) of CRCP, or $0.52 per square yard ($0.62 per sq. m) per year of service life. Therefore, the asphalt overlay plus grouting must provide an annual cost of no more than $0.52/SY to be as cost effective as the asphalt overlay alone. The total initial cost of this alternative is

\[
\begin{align*}
$5.75 & \text{ Asphalt Overlay/SY} \\
- 3.33 & \text{ Grout Underseal/SY for 100\% Coverage} \\
$9.08 & \text{ Total}
\end{align*}
\]

Therefore, the total life required during the first overlay period is computed as

\[
\frac{$9.08/SY}{$0.52/SY/yr.} = 17.5 \text{ yrs.}
\]

The asphalt overlay and grout alternative must last at least 17.5 to be as cost effective as the asphalt overlay (or and extra 6.5 years).*

*Note: The following assumptions were made in the cost effectiveness analysis. (1) Each alternative requires similar routine maintenance, (2) each initial treatment is repeated at the end of each period and (3) the interest rate is equal to the inflation rate.
Similarly, selective grouting at 10% and 40% coverages were required to provide approximately one year and three years of additional pavement life respectively, to be as cost effective as the asphalt overlay.

Although this economic analysis was approximate and several simplifying assumptions were made, it does show that it is questionable that blanket grouting would be cost effective (i.e., that it could extend the pavement life by 6.5 years or more). Even though it costs much more than selective grouting, it is no more effective in reducing overall deflections. It is very possible that selective grouting could prove to be cost effective since it only needs to extend pavement life by 1-3 years. The final answer to the question of cost effectiveness will be answered only by the monitoring of the field performance of these treatments over time.

At this time selective grouting seems to be a much more cost effective treatment than blanket grouting. It places the material where it will have the most beneficial effect, and shows promise as an effective way to extend pavement life. The process of selective grouting will require visual surveys and heavy load NDT in order to locate areas displaying loss of support.

On the other hand, blanket grouting does not seem to be cost effective when pumped into areas of CRCP which display no sign of voids (both visually and deflections). There is very little chance that blanket grouting would be cost effective except in the case when an entire project shows signs of pumping.
Figure 4.1: Cement Grout Undersealing Hole Pattern
Figure 4.2. Gardner-Denver Drill Mounted on a Forklift
Figure 4.3: Compressive Strength of Cement Grout Slurry Versus Time
Figure 4.4. Apparatus for Monitoring Vertical Movements of the Slab
Figure 4.5. Profile Showing Reduced Deflections After Cement Grout Undersealing
Figure 4.6. Deflection Profiles Before and After Cement Grout Undersealing
Figure 4.7. Deflection Profiles Before and After Cement Grout Undersealing
Table 4.1. 1978 Cost Data for Various Cement Grouting Applications.

<table>
<thead>
<tr>
<th>TREATMENT</th>
<th>UNIT COST OF SLURRY</th>
<th>AVE. AMOUNT OF SLURRY Pumped</th>
<th>TOTAL COST OF SLURRY</th>
<th>COST OF DRILLING</th>
<th>TOTAL COST OF TREATMENT</th>
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<tr>
<td>Blanket Cement Grouting</td>
<td>$6.73/ft³</td>
<td>0.40 ft³/yd²</td>
<td>$2.69/yd²</td>
<td>$0.64/yd²</td>
<td>$3.33/yd²</td>
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<tr>
<td>Selective Cement Grouting 100% Coverage</td>
<td>$6.73/ft³</td>
<td>0.50 ft³/yd²</td>
<td>$3.36/yd²</td>
<td>$0.64/yd²</td>
<td>$4.00/yd²</td>
</tr>
<tr>
<td>Selective Cement Grouting 40% Coverage</td>
<td>$6.73/ft³</td>
<td>0.20 ft³/yd²</td>
<td>$1.35/yd²</td>
<td>$0.25/yd²</td>
<td>$1.60/yd²</td>
</tr>
<tr>
<td>Selective Cement Grouting 10% Coverage</td>
<td>$6.73/ft³</td>
<td>0.05 ft³/yd²</td>
<td>$0.34/yd²</td>
<td>$0.06/yd²</td>
<td>$0.40/yd²</td>
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Chapter 5

ASPHALT UNDERSEALING

5.1 Introduction

Asphalt undersealing refers to the pressurized pumping of hot asphalt beneath the slab and/or subbase to arrest pumping and restore uniform support by filling the underlying voids. Asphalt undersealing does not strengthen the pavement section but if the voids are properly filled, uniform support is restored and pumping is suspended. Water resistant properties of the asphalt also help limit future pumping by preventing the intrusion of water into the subgrade/subbase. The slab should not be raised during this treatment. Significant lifting of the slab can be harmful, inducing stress concentrations into the CRCP.

As described in Chapter 4, the conditions of the project clearly indicated that voids existed beneath the pavement. If these voids were not filled, water would continue to collect in them and then be forced out by traffic action, taking fine material with it. This pumping action would increase the size of the voids, resulting in localized loss of support. The loss of support from beneath a pavement slab accelerates deterioration. Therefore, prior to overlaying, pumping must be stopped and uniform support must be returned to the slab in order to slow the deterioration process. One treatment selected for this purpose was asphalt undersealing. Asphalt undersealing had long been used for rehabilitating pavement, and if it could properly fill the voids and stabilize the slab, the rate of deterioration would be slowed and the service life of the pavement would be prolonged.
5.2 **Asphalt Undersealing Procedures**

Two 4000 foot (1219 m) long sections of pavement were chosen for asphalt undersealing. The two sections treated, covered all of Sections D and F from the NDT analysis in Section 2.4. This would accommodate the comparison of deflections taken after asphalt undersealing with those taken before undersealing. Asphalt undersealing was used as a blanket treatment only.

The hole pattern chosen for asphalt undersealing is shown in Figure 5.1. Holes were staggered 4 ft (1.2 m) on either side of the centerline, and spaced at 10 ft (3 m) longitudinally. With this pattern, asphalt was rarely forced up in adjacent holes, indicating minimum or no overlap. When trenches were opened for underdrains a nearly continuous layer of asphalt filler was present along the edge of the slab. Although this hole pattern appeared to give good coverage, the longitudinal spacing is larger than average spacings used for typical asphalt undersealing. A more typical longitudinal spacing for staggered holes in adjacent lanes is 4 to 6 feet (1.2 - 1.8 m).

The holes were 1-1/2 inches (4 cm) in diameter and were made with a drill mounted on a small tractor (Figure 5.2). The holes were drilled to a depth just below the bottom of the CRC slab (8 to 12 inches (20 to 30 cm)). The holes were also blown out with compressed air as they were being drilled. This was done to blow water and debris out of the hole to aid in pumping the asphalt. Drilling took between 45 and 60 seconds per hole, unless reinforcing steel was encountered. The drill was not capable of cutting the reinforcement, so if reinforcement was encountered
during drilling, the drill had to be pulled out and a new hole started. Then the unfinished hole had to be filled with sand-cement grout. This slowed the drilling process considerably. A drill capable of cutting the reinforcement is a definite asset.

A special asphalt cement must be used for asphalt undersealing. A typical asphalt cement used for undersealing usually possesses a low penetration and a high softening point. It should also have a viscosity suitable for pumping when heated to temperatures of 400°F to 450°F (Ref. 4). An IDOT specified type PAF-3 asphalt cement was used on the project.

The hot asphalt (400°F to 475°F) was delivered in bulk tankers, weighed for payment purposes, then emptied into applicator trucks. The pavement was splashed with water around the hole and areas where the asphalt might be forced up to allow for easy cleanup. The water kept the asphalt from sticking to the surface of the pavement. After a water saturated cardboard square was placed over the hole to insure a good seal during pumping, the tapered nozzle was inserted into the hole and held firmly in place. The asphalt is then pumped under the pavement until substantial vertical movement occurs. Pavement movement was monitored visually. If asphalt was forced up through a joint or crack before noticeable vertical movement of the slab, the asphalt was allowed to partially congeal, then pumping was continued. The amount of asphalt necessary to fill a hole varied from 21 to 57 gallons per hole, with an average value of 36 gallons per hole. This converted to a range 0.11 to 0.29 cubic feet of asphalt per square yard of pavement, with an average of 0.18 cubic feet per square yard. This is a much smaller quantity than the amount of slurry used in pressure grouting indicating that possibly the asphalt undersealing was
not completely filling all of the voids. The deeper drilling of the grouting must also have allowed the filling of many voids beneath the subbase (as was observed from the side trenches), which was not the case for the asphalt undersealing.

After filling, the nozzle was removed and a temporary wooden plug immediately inserted. When the asphalt had hardened sufficiently to prevent back flow, the wooden plug was removed and the hole was filled with portland cement sand grout. Finally, cleanup crews would scrape up the extruded asphalt from the pavement.

Visual monitoring of pavement movements during the undersealing process was thought to be very inadequate. A check was made to find out the accuracy of visual monitoring of the undersealing crew chief. Using the differential movement apparatus described in Chapter 4 (Figure 4.4), fifteen holes were monitored for changes in differential elevation between the shoulder and slab before and after undersealing. The movements were monitored directly opposite a hole and also at the midpoint between holes. Directly opposite the hole the shoulders as well as the slab were being lifted. At the midpoint the shoulders were no longer being lifted, however, the slab was being raised up to 0.60 in. (1.5 cm). This proved that visual monitoring was grossly inadequate. It allowed too much material to be pumped under the pavement, resulting in significant lifting of the slab. Whether the slab was raised enough to cause serious damage is not known, however, no additional slab cracking was observed. During cement grout undersealing, vertical pavement movement was limited to 0.10 in. Asphalt undersealing should be continued only until the slab begins to lift.
In order to recognize this condition some form of mechanical monitoring must be implemented.

During the application of asphalt undersealing certain safety precautions must be taken. Due to the high temperature and pressure used in pumping the asphalt, workmen should wear proper face masks, asbestos gloves, and heavy clothing. In addition, a shield is necessary to protect passing vehicles from asphalt that may be sprayed out if the nozzle does not maintain a tight seal in the hole.

5.3 Evaluation of Asphalt Undersealing

A complete evaluation of asphalt undersealing cannot be made until actual pavement performance is determined in the next several years. Only actual pavement performance will determine the total cost effectiveness of this treatment. The effect of asphalt undersealing on deflection can also be evaluated by comparing NDT deflections taken before asphalt undersealing, after asphalt undersealing, and after the asphalt overlay. Also, an approximate cost effectiveness analysis is presented with other cost comparisons.

5.3.1 Deflection Evaluation of Asphalt Undersealed Areas

Approximately two weeks after the completion of asphalt undersealing, NDT deflections were taken at all the locations measured in the initial NDT analysis described in Section 2.4. As explained in Section 4.3.1, all loading conditions were duplicated as closely as possible. The deflections from the asphalt undersealed areas taken before and after treatment could be compared to assess its structural effect. The deflec-
tions taken after undersealing were normalized by being multiplied by 1.12. The calculation of this normalizing factor is described in Section 4.3.1.

Examination of the deflection profiles revealed a surprising result. The deflections taken after asphalt undersealing showed significant increase over corresponding deflections taken before undersealing at many points. Typical profiles from asphalt undersealed sections are shown in Figures 5.3 and 5.4.

The statistical analysis showed similar results to those found from examination of deflection profiles. The net mean deflection for undersealed sections after asphalt undersealing was $9.0 \times 10^{-3}$ inch compared to a net mean deflection of $6.7 \times 10^{-3}$ inches for the same sections before asphalt undersealing (34% increase in mean deflection). The mean deflection included all of Section F plus all of Section D except for the 12 inch (30 cm) thick CRC as designated in Chapter 2. The 12 inch (30 cm) thick CRC also revealed large deflection increases after asphalt undersealing. The most obvious reason for the deflection increase is that too much material was pumped under the slab due to inadequate monitoring of vertical slab movements.

Another statistical analysis was conducted on a third set of NDT deflections taken after the asphalt overlay was placed. In this analysis, a T-test comparison was conducted between the initial deflection and the deflection taken after completion of the asphalt overlay. A comparison was made to see if the combination of asphalt undersealing and asphalt overlay reduced deflections more than an asphalt overlay alone. The results of the analysis were dominated by the reducing effect of
the asphalt overlay on deflections. There was no significant difference between the reduction in deflections for control sections and asphalt undersealed sections.

5.3.2 Cost Analysis

The cost of asphalt undersealing calculated using bid costs and pumped quantities was $2.86 per square yard of CRCP in the traffic lanes. Without considering the relative amount of material used, the cost of asphalt undersealing compared favorably with the cost of blanket cement grout undersealing. Compared with grout undersealing costs from Table 4.1, asphalt undersealing costs less than blanket grouting, but significantly more than the 40% coverage selective grouting. However, it must be realized that on the average, more than twice as much cement grout slurry volume was pumped than was asphalt cement per unit area of slab treated. Therefore, the reason asphalt undersealing was competitive with grouting, from a cost perspective, was due to the much smaller amount of asphalt pumped.

A more revealing comparison is made between unit cost of pumping materials. The unit cost of asphalt cement was $1.90 per gallon which converts to $14.21 per cubic foot, while the calculated unit cost for cement grout slurry was only $6.73 per cubic foot. This shows that if identical pavement sections were treated using equal amounts of cement grouting and asphalt cement undersealing, the cost of asphalt would be over twice as much as the cost of grouting.

The cost of asphalt undersealing was evaluated using the approximate cost effectiveness analysis method in Chapter Four. Using the
same assumptions the asphalt undersealing in combination with the asphalt overlay would have to extend the life of the pavement 6.5 years to be as cost effective as the asphalt overlay.

Although only the performance of the pavement over time will determine the actual cost effectiveness of asphalt undersealing, several recommendations can be made. Due to its relative high unit cost, asphalt undersealing does not appear to be well suited for treating projects with extensive large voids such as this one. With its lower unit cost and ability to gain strength in the presence of water, cement grout is a much better alternative for filling large voids. Other problems, besides cost, have been found concerning the treatment of large voids with asphalt undersealing. On one asphalt undersealing project it was reported that when slabs were removed after undersealing, voids deeper than one inch were commonly found only partially filled. In most instances, a thin layer of asphalt was found along the top of voids in contact with the pavement, but the balance of the voids were unfilled. The suggested explanation was that the asphalt cement pumped under the slab was unable to displace water that was present in the large voids. This would result in the asphalt floating to the top of the water (Ref. 5).

Despite its high unit cost and possible problems with filling large voids, asphalt undersealing may be cost effective as an early preventive maintenance treatment. Asphalt undersealing may be considered as a preventive maintenance technique at the first sign of small voids (Ref. 6 recommends asphalt undersealing to fill voids generally less than half an inch thick). Asphalt undersealing would provide a thin protective
layer under the slab that should limit water intrusion into the subbase/subgrade, suspending further pumping. Also, limiting asphalt undersealing to the filling of small voids should minimize costs, while still providing significant benefits.
Figure 5.1. Asphalt Undersealing Hole Pattern
Figure 5.2. Drill Rig Used in Asphalt Undersealing
Figure 5.3. Deflection Profiles Before and After Asphalt Undersealing
Figure 5.4: Deflection Profiles Before and After Asphalt Undersealing

INITIAL DEFLECTIONS

AFTER ASPHALT UNDERSEALING

DEFLECTION × 10⁻³ in

STA.

00+00 579+00

00+00 580+00

00+00 581+00

00+00 582+00

00+00 583+00

00+00 584+00

00+00 585+00
Chapter 6

EPOXY CRACK SEALING

6.1 Introduction

Epoxy has been used for structural grouting and repairs of concrete since the late 1950's. The injection of epoxy into cracks was the first technique for restoring the integrity of cracked concrete. Today, epoxy is widely used in the concrete industry. It is used for anything from a sealant to a bonding agent. It is especially used to fill cracks in concrete both to restore structural integrity and also to seal the area from moisture.

Three specific areas were proposed where the use of epoxy for crack sealing could be applied to the maintenance of CRCP: (1) areas of close crack spacing, (2) punchouts that are just beginning to form, and (3) cracks which are beginning to work and open. The third proposed area was the one tested in this project.

Almost 90 cracks had opened up between 0.05 inches (1 mm) and 0.75 inches (19 mm) within this project. As mentioned in Chapter Three only the most severe wide cracks were patched. Of the remaining wide cracks which were not patched, 61 were treated with epoxy. Many of these cracks were beginning to work and widen and a majority contained ruptured steel. Most of the wide cracks were subject to significant movements (opening/closing). As the cracks widened most of the aggregate interlock was lost, resulting in high vertical shears in the steel under wheel loads. This would eventually lead to progressive failure by rupture across the lane
as described in Chapter Two, unless something was done to bond the two sides of the crack together.

The epoxy was designed to fill the cracks and transfer shear by epoxy-concrete bond. The epoxy would in effect "weld" the slab back together, returning continuity to the slab and arresting further deterioration. If this were possible, it would save a tremendous amount of money because the only other alternative would be patching.

6.2 **Epoxy Crack Sealing Procedures**

Preparation of the cracks for epoxy application consisted of blowing all dust and loose concrete out of the crack. Pieces of loose concrete lodged in the crack were usually broken loose with a small hammer. The crack was blown out using an air jet, clearing the crack of all dust and debris. The cleaning operation on all 61 cracks was completed in less than 4 hours.

The epoxy was then mixed in small quantities near the point of application. The epoxy was a two-component high modulus, low viscosity, moisture insensitive material. Fluidity of the epoxy was very important since gravity filling was used. Non-contaminated oven dry silica sand was used as a mineral filler. The first use of the epoxy was without any sand to ensure coating of all surface areas. The first fill is continued until the crack remains full. (At the beginning of the fill some epoxy will usually leak out.) A second crew follows refilling the cracks as necessary. The sand is mixed with the epoxy for the second fill of the larger cracks, and is also spread on the surface of the epoxy to provide greater durability.
6.3 Results from Epoxy Crack Sealing

It was hoped that the epoxy would return continuity to the slab, forming a cost savings alternative to the patching of wide cracks. At a cost of $7.44 per lineal foot ($89.28 per 12 foot (3.6 m) lane width crack) epoxy crack sealing could repair a wide crack for a fraction of the cost of a patch. Unfortunately, the epoxy did not perform the way it was designed.

The wide cracks on the project were the focal points of significant temperature movements of the CRC slab. The epoxy was relatively brittle and could not withstand these movements. The epoxied cracks ruptured within 2 days to 3 weeks depending on the severity of the temperature movements at the specific crack. The epoxy usually failed by one of two modes; 1) failure in the epoxy, or 2) failure of the epoxy-concrete bond. Contributing to the failure of the epoxy-concrete bond is the problem of obtaining a clean bonding surface. It is very difficult to obtain adequate bond to the concrete at the face of a working crack.

Deflection measurements reinforced what was already known. The epoxy failed to decrease deflections or to increase crack efficiency by transferring shear forces. This was expected since all of the wide cracks opened up within 2 days to 3 weeks of the application of the epoxy.

The final test for the epoxy crack sealing was the reflective crack survey conducted approximately 10 months after the project was overlaid. Seven wide cracks had been documented and intentionally left untreated. Of the seven cracks left untreated, three reflected through the asphalt overlay (43 percent). For the wide cracks that were treated with epoxy, 32 out of 61 (32 percent) reflected through the overlay. Thus, there
was no significant difference in treated and untreated cracks reflecting through the asphalt overlay.

The epoxy was simply unable to withstand the temperature movement of the slab. After the epoxy broke down and the crack reopened, no support was provided by the epoxy. This was indicated by the results of deflection measurements and the reflection crack survey.

Epoxy crack sealing is not recommended for any wide crack that has opened up across the entire lane. Rupture of the reinforcing steel has likely occurred and significant temperature movements of the slab may exist. However, in the case where a wide crack has opened only part way across the lane, the application of epoxy may slow the rate of deterioration and prevent or delay a failure that would require patching.
CHAPTER 7

ASPHALT OVERLAY AND UNDERDRAINS

7.1 Introduction

The final stage of the R-R-R project was the placement of pipe underdrains along the edges of the pavement and the application of an asphalt overlay.

7.2 Underdrains

The pipe underdrains (4 inch (10 cm) diameter) were placed 4 to 6 inches (10 to 15 cm) from both edges of the CRC slab and at a depth of at least 30 inches (76 cm) below the top of the slab as shown in Figure 7.1. Underdrain lateral outlets were placed at approximately 500 feet (152 m) intervals. After placement of the pipe underdrains, water was found flowing from many of the outlets, as shown in Figure 7.2, revealing the amount of water held by the subbase/subgrade. The underdrains should reduce the amount of free water within the structural section reducing the adverse effects of pumping and freeze-thaw cycles.

7.3 Asphalt Overlay

The entire project was resurfaced with 4 inches (10 cm) of asphalt concrete, with the exception of one 700 foot (213 m) section. This 700 foot (213 m) section was overlaid with an additional 1-1/2 inch (3.8 cm) bituminous binder overlay for a total thickness of 5-1/2 inches (13.8 cm). The 4 inch (10 cm) asphalt overlay consisted of a 1 inch (2.5 cm) leveling binder, a 1-1/2 inch (3.8 cm) bituminous concrete binder course, and a 1-1/2 inch (3.8 cm) bituminous concrete surface course. A 38 foot (11.6 m)
pavement width (two 12 foot (3.6 m) lanes, a 3-1/2 foot (1 m) inside shoulder, and a 9-1/2 foot (2.9 m) outside shoulder) was overlaid at a cost of $5.75 per sq. yd. of CRCP in the traffic lanes. The asphalt overlay provided a smooth riding surface as well as a structural improvement.

The performance of the asphalt overlay was evaluated with two sets of NDT deflections, one shortly after the application of the overlay (Nov. 1978) and the other about 10 months from the date of overlay (July 1979). A reflection crack survey was also conducted.

Both set of NDT deflections were measured at the same points as for the initial NDT analysis. The Road Rater was used for both sets and loading points were duplicated as close as possible to those of the initial deflections.

The first set of deflections after the overlay was compared to the initial deflections. As shown in a typical profile in Figure 7.3, the deflections were reduced substantially along the entire project. The deflections were separated into the original test sections set up in Table 2.2 and analyzed statistically. The mean deflections from the initial NDT deflection were compared with mean deflection after the overlay. The deflections from the 5-1/2 inch (13.8 cm) thick overlay section were separated from Section E and designationed Section EX. All sections showed substantial deflection decreases. The results are shown in Table 7.1. Section EX showed the largest percentage decrease in deflection, obviously due to the extra thickness of asphalt concrete, along with Section A, of 51%. The 12 inch (30 cm) thick pavement section showed
the smallest percentage reduction in deflections at 29%.

The second set of deflections, taken 10 months after the overlay was compared to the first set of deflections after the overlay. Data for this comparison is also contained in Table 7.1. The new deflections for the second set showed a significant increase over the mean deflections for the first set of deflections after the overlay (28-40%). However, the second set of deflections still showed a decrease over the initial deflections taken prior to construction of the project (8-39%). The increase in deflections was not surprising. The first deflections taken after the overlay were measured in two consecutive days in November, when the average temperature was 35°F (2°C). The second set of deflections taken after the overlay were measured the following July when the average temperature was approximately 75°F (24°C). Asphalt pavement deflections increase with increases in temperature, so the deflection change is reasonable.

The reflection crack survey was conducted 10 months after the placement of the asphalt overlay. The survey revealed that distresses which were not adequately repaired usually reflected through the overlay. Many of the wide cracks which were not patched caused reflection cracks in the asphalt overlay.

The asphalt overlay provided a smooth riding surface and substantially reduced deflections. However, if the underlying pavement is not adequately repaired these distresses will reflect through the asphalt overlay requiring maintenance of some type in the future. For optimum performance of an asphalt overlay the underlying pavement should be adequately restored.
7.4 Cost Analysis

The cost effectiveness of the asphalt overlay depends upon the costs to maintain the pavement without an overlay. The project could conceivably be maintained by extensive patching for many years into the future. This would require considerable maintenance funds, lane closure time, and greater roughness over the next 10-20 years. A comparison is made to determine the approximate cost effectiveness of the asphalt overlay.

Future traffic over the next 20 years was estimated using historical trends. The future patching required if no overlay was placed was estimated using statistical theory and the trend of patch placement over the previous 10 year period. Figure 2.2 shows that the cumulative area of patching approximately follows a log-normal distribution with 18-kip equivalent single axle loads (ESAL). This is reasonable since many materials approximately follow a log-normal fatigue failure loading distribution. Utilizing this distribution (calculating the mean and standard deviation from the data shown in Figure 2.2) and the expected future traffic, the future patching over the 6.6 mile project was estimated. Assigning a typical cost of $100/SY for patching, the accumulated cost over the next 20 years is plotted in Figure 7.4.

The asphalt overlay was placed over only 3.1 miles of the 6.6 mile project due to the high concentration of distress in this area (this area contained about 85% of all existing distress). The costs of pre-overlay patching over the entire project plus the asphalt overlay is the total 1978 cost. Future repair over the next 20 years was assumed as follows:

- Approximately 15% of total project patching over the 6.6 miles has occurred in areas outside of the 3.1 mile asphalt overlay. This proportion is assumed to continue into the future.
• Only a small amount of patching will be required within the asphalt overlay. This is estimated at 5% of what would have occurred without the overlay.

The accumulated cost of this alternative over the next 20 years is shown in Figure 7.4.

The point of intersection of the two curves represents the approximate time when the accumulated costs of the overlay is equal to that of the patching only alternative. Thus, the overlay would need to last at least 12 years (or 1990) to be as cost effective as the patching only alternative. This is about the age predicted using the IDOT design manual as previously discussed.

It must be emphasized that this is only an approximate analysis as many factors have been ignored (such as salvage value, lane closure effect when patching, interest and inflation rates, etc.). Also, the rate of deterioration of the CRCP is considerably higher than on most other projects. If the rate of deterioration was less (the rate of patching less), the asphalt overlay would have to last much longer in order to be as cost effective as continual patching. Another point can be made relative to the length of overlay. If the overlay would have been placed over the entire 6.6 mile project, the initial cost would of more than doubled. Thus the Figure 7.4 AC overlay curve would have shifted upward, and the intersection of the two curves would not occur until 20-30 years. In this case the overlay would need to last at least this long to be cost effective, which is unlikely. Thus, the asphalt overlay is believed to be cost effective only when placed over the most severely deteriorated portions of the project.
Figure 7.1. Illustration of Pipe Underdrains
Figure 7.2. Water Flowing from Lateral Outlets
Figure 7.4. Cost Effectiveness of an Asphalt Overlay Compared to Continual Patching
Table 7.1. Results from Statistical Analysis of Deflection.

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Chapter 8

CONCLUSIONS AND RECOMMENDATIONS

An evaluation of several maintenance/rehabilitation techniques on Interstate CRCP has been conducted. The results from this evaluation be used to improve rehabilitation and maintenance methods.

The following conclusions and recommendations are based upon the results of the evaluation:

1. The project was chosen for rehabilitation because of its existing distress and the increasing rate of deterioration. The portion of the project containing the most distress was selected for RRR work. The project contained four major distress types: edge punchouts, wide cracks, failure adjacent to a patch, and failures adjacent to a joint in a ramp.

2. When a transverse crack shows any faulting (i.e., as little as 0.04 inches (1 mm)), it is reasonable to assume corrosion has been significant enough to reduce the cross-sectional area of the steel and/or there has been a slight widening of the crack and considerable loss of aggregate interlock.

3. It is believed that deflections can be used to locate areas that may be in early stages of distress (i.e., underlying voids). But, the magnitude of load needed to get adequate information is relatively high. A load of 5 kips (22.2 kN) or more is recommended.

4. The coefficient of variation (COV) of the deflection measurements over a given section correlates with the amount of distress for that section. A high COV indicates high deflection peaks which reveal localized structural weaknesses in the CRCP and foundation.
5. The adjacent lane condition (tight or wide crack) was the dominant variable in determining the amount of early cracking in a new concrete patch. The next most significant variable was the combination of time of pour and adjacent lane condition. A patch poured in the morning adjacent to a wide crack was very likely to develop transverse cracks due to large stresses induced by temperature movement of the adjacent CRCP during the first few nights.

6. Both the 20 inch (51 cm) tied lap splice patch and the 4 inch (10 cm) double weld splice patch performed adequately in all stages of evaluation. They are proposed as alternative patching methods since they saved an average of $583 per patch when compared to the cost of a standard 10 x 12 foot (3 x 3.6 m) patch constructed by private contractors. The welded splice patch is very cost-effective where a wide crack distress exists across both lanes. Bituminous patches did not reduce deflection peaks at failed areas and resulted in reflective cracks in the overlay at each end of the patches.

7. Cement grout undersealing was effective in reducing above average deflections. A blanket treatment over the entire project is probably not cost effective unless an entire project shows signs of pumping and loss of support. It is recommended for preventive maintenance or rehabilitation use as a selective treatment at the following locations: areas showing signs of pumping, adjacent to patches and roadway cross pipes, lug areas, and areas with localized loss of support. A heavy deflection device is required for selective pressure grouting to locate areas displaying loss of support. Mechanical monitoring of vertical pavement movements is necessary to limit lifting of the slab.
8. Asphalt undersealing caused

Vertical movements of the slab were only monitored visually by the contractor. When relative vertical movements were checked mechanically, they ranged up to 0.6 inches (15 mm) indicating overpumping of asphalt cement. The unit cost of asphalt cement was more than double the unit cost of cement grout slurry used in pressure grouting. Due to its high unit cost asphalt undersealing may not be cost-effective for use in filling large voids. However, it is recommended as a preventive maintenance treatment. It should provide a protective layer under the slab which will minimize water intrusion into the subbase/subgrade and limit further pumping.

9. The use of epoxy to restore the continuity of the slab at a wide transverse crack was unsuccessful. Due to substantial adjacent pavement movements the epoxy broke down and the crack reopened within 2 days to 3 weeks of application. Epoxy crack sealing is not recommended for wide working cracks that are across an entire lane.

10. The asphalt overlay was effective in reducing deflections. Underlying distress which was not adequately repaired reflected through the overlay. Adequate structural restoration of the underlying CRCP will reduce future maintenance and extend the life of an asphalt overlay.

11. If the entire project length had been overlayed, it is almost certain that it would not have been as cost effective as continual patching. However, by overlaying only the most severely deteriorated portions of the project the asphalt overlay is likely to be as cost effective as continual patching.
REFERENCES


