ACCELERATED PAVEMENT TESTING OF EXTENDED LIFE CONTINUOUSLY REINFORCED CONCRETE PAVEMENT SECTIONS

by

Erwin Kohler
Jeffery Roesler

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### Abstract

Ten CRCP sections were built at the ATREL full-scale testing facility, with five sections being tested under accelerated traffic loading conditions and five sections evaluated for induced transverse cracks. The opening and closing of cracks was measured at several transverse cracks, along with vertical deflections, transverse strains at the top of the slab, and internal pavement temperature. Two procedures were developed to determine crack width from the measurement of crack closing. These procedures used the horizontal deformation caused by changing the vertical load level or temperature condition to determine the in situ crack width. By calibrating the crack width model presented in the mechanistic-empirical pavement design guide (ME-PDG), crack width values obtained under different temperature conditions could be standardized. The model was adapted to predict crack width at any depth in the slab and an enhancement is proposed to use the model for predicting short-term changes in crack width. Continuous surveying of the pavements for more than two years and the sequential application of a large number of rolling-wheel loads on each section allowed for the collection of response data and observation of CRCP failure mechanism. Comparisons are presented regarding the elastic responses in sections with different design features (PCC thickness, steel content, and steel depth). Under conditions of small crack width (less than 0.15 mm), load transfer capacity at the transverse cracks remained intact despite heavy traffic loads and seasonal thermal cycles. Failure of the CRCP sections resulted from permanent deformation in the supporting layers. This report includes evaluation of crack width variability and use of FWD to characterize in situ crack width of field CRC pavements. Recommendations are made for the improvement of CRCP construction based on early age temperature, concrete drying shrinkage, and induction of transverse cracks.
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<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>ATLAS</td>
<td>Advanced Transportation Loading System</td>
</tr>
<tr>
<td>ATREL</td>
<td>Advanced Transportation Research and Engineering Laboratory</td>
</tr>
<tr>
<td>CRC</td>
<td>continuously reinforced concrete</td>
</tr>
<tr>
<td>CRCP</td>
<td>continuously reinforced concrete pavement</td>
</tr>
<tr>
<td>CS</td>
<td>crack spacing</td>
</tr>
<tr>
<td>CTE</td>
<td>coefficient of thermal expansion</td>
</tr>
<tr>
<td>CW</td>
<td>crack width</td>
</tr>
<tr>
<td>ESAL</td>
<td>equivalent single axle load</td>
</tr>
<tr>
<td>FWD</td>
<td>falling weight deflectometer</td>
</tr>
<tr>
<td>JPCP</td>
<td>jointed plain concrete pavement</td>
</tr>
<tr>
<td>Kips</td>
<td>1,000 pounds</td>
</tr>
<tr>
<td>LST</td>
<td>load spectra test, testing procedure to determine CW</td>
</tr>
<tr>
<td>LTE</td>
<td>load transfer efficiency</td>
</tr>
<tr>
<td>LTPP</td>
<td>long-term pavement performance</td>
</tr>
<tr>
<td>LVDT</td>
<td>linear variable displacement transducer</td>
</tr>
<tr>
<td>M-E PDG</td>
<td>Mechanistic-Empirical Pavement Design Guide</td>
</tr>
<tr>
<td>PCC</td>
<td>Portland cement concrete</td>
</tr>
<tr>
<td>RH</td>
<td>relative humidity</td>
</tr>
<tr>
<td>SRA</td>
<td>shrinkage- reducing admixtures</td>
</tr>
<tr>
<td>Tavg</td>
<td>average pavement temperature, obtained from measurements near the surface and near the bottom of the slab</td>
</tr>
<tr>
<td>Tdiff</td>
<td>temperature difference between the surface and the bottom of the pavement</td>
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Note on units: English units (inches, feet, pounds) are used throughout the text with the exception of crack width, which is always presented in millimeters.
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DISCLAIMER

The contents of this report reflect the view of the author(s) who is (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.
CHAPTER 1  INTRODUCTION

1.1. Brief description of the problem

A narrow crack width is essential in continuously reinforced concrete pavements (CRCP) to allow proper load transfer across the crack and to reduce pavement deterioration as a result of the crack. All concrete structures experience volumetric changes caused by shrinkage and cyclic variation in the temperature and moisture conditions, which are inevitable in the case of pavements due to being exposed continuously to the environment. The embedded reinforcement in CRCP makes an important difference in the way these volumetric changes affect pavement performance. The short concrete segments between transverse cracks respond to traffic and temperature loading, while the reinforcement maintains continuity between these segments. The ability of transverse cracks to prevent the pavement from responding as independent slabs is the most crucial performance factor in CRCP, and the concepts of crack width, crack face rotation, and crack deterioration represent the central part of this issue. A detailed look into the mechanics of crack width is needed to understand the process of CRCP deterioration. The specific mechanism in which load transfer capacity is gradually lost has to be explained in order to refine the mechanistic design approach to this type of pavement.

The opening and closing of cracks is a highly complex phenomenon when parameters such as temperature variations, drying shrinkage, and traffic load are included. Although crack width is acknowledged as the most critical issue on CRCP performance, the difficulties associated with its measurement have resulted in limited research efforts on the topic. The main obstacle has been measurement of a parameter whose magnitude is in the sub-millimeter range and that needs to be observed below the pavement surface. Results of crack width measurements taken from the full-scale experimental sections described in this report enable better characterization of CRCP behavior and performance.
1.2. Research objective and methodology

The overall objective of the proposed research is to determine the sequence and process of punchout distress under CRCP sections subjected to accelerated loading for variables such as percent steel, depth of reinforcement, and slab thickness. The influence of crack width on the CRCP section performance was of particular interest in this research. The majority of the analysis focuses on the quantification of horizontal and vertical slab movements at transverse cracks, which results in crack width changes.

The specific objectives of this research for extended life CRCP are:

1. To study failure mechanism in full-scale CRCP sections subjected to accelerated traffic loading for several design variables. The following five questions are expected to be answered with respect to transverse crack spacing, crack width, and load-related cracking based on the full-scale testing:
   - required thickness for extended life CRC pavement
   - necessity of two layers of steel reinforcement
   - design steel content for minimum crack width
   - ideal depth of steel from the PCC surface and
   - effect of uniformly induced crack spacing on the performance of the CRCPs are evaluated.

2. To measure crack width in transverse CRCP cracks through evaluation of mechanical responses of the pavement. This approach seeks also the crack width profile determination.

3. To evaluate the effect that temperature changes and load applications have on the crack width magnitude. This implies evaluation of seasonal and daily temperature cycles and the analysis of the instantaneous changes in crack width caused by a rolling wheel.
Ten, full-scale CRCP test sections were built and five sections loaded under accelerated traffic conditions at the Advanced Transportation Research and Engineering Laboratory (ATREL) full-scale testing facility. The sections were instrumented to capture vertical and horizontal deformations in the pavement, as well as pavement temperature. The sections were tested continuously from December 2002 to August 2004, and therefore crack width was determined under different thermal conditions for each section. The experimental research consisted of characterizing crack width magnitude and applying accelerated (damaging) loading to fail the pavement sections in a short period of time.

The methods for measuring crack width were perfected during the process of loading the sections. The problem of comparing measurements obtained in different seasons was approached making use of the crack width prediction model that is part of the Mechanistic-Empirical Pavement Design Guide (M-E PDG) that resulted from NCHRP Project 1-37A. This work also documents and explains the causes of the extensive cracking observed in the failed sections and compares the mechanism of failure under small crack width against currently accepted punchout failure models.

1.3. Contents of the document

Chapter 1 presents the introduction and background about continuously reinforced concrete pavements. Chapter 2 contains the literature review pertinent to crack width in CRCP, including previous studies, factors affecting crack width, and details of prediction models, especially the model from the M-E PDG. Chapter 3 describes the research approach, the design variables in the sections, the loading machine used to test the pavements, and the instrumentation employed. Chapter 4 explains the methods of crack width measurement, describing the effect of temperature and load in the horizontal movements at transverse cracks. Chapter 5 is devoted to the analysis of results of crack width (CW), showing crack profiles through the thickness and explaining how the crack width model in the M-E PDG was used to shift CW measurements to standard temperature conditions and the capacity of the model to predict CW at any slab depth and to predict short term variation in CW. It includes observations of the zero-stress temperature and a preliminary assessment of crack width variability with the Accelerated Transportation Loading
System (ATLAS) machine and a Falling Weight Deflectometer (FWD). Chapter 6 presents details of the CRCP cracking and the results of accelerated load testing of the different sections. Transverse cracks are analyzed regarding their progression in time, spacing interval, and width; results of loading tests show typical responses measured in elastic load range and the mechanism of pavement failure, and a brief evaluation of two layers of reinforcement steel. Chapter 7 presents recommendations for CRCP construction regarding early age temperature development, shrinkage, and transverse crack induction. Chapter 8 summarizes the findings and suggests future research.

1.4. **Historic development of CRCP**

1.4.1 **Background**

A continuously reinforced concrete pavement (CRCP) is constructed with no regular transverse joints and contains longitudinal bars of steel reinforcement. The reinforcement is made continuous by splicing the bars to make sections that can be several miles long. Transverse cracks form as a result of concrete drying and temperature shrinkage being restrained by the longitudinal steel reinforcement and slab-base friction. The subsequent cracks are held tightly together by the continuous steel reinforcement. CRCP is considered a pavement type for heavy traffic and where the delays associated with repairs and rehabilitation have to be minimized. A study of over 400 in-service PCC pavements revealed that CRCP outperforms other types of concrete pavement (Smith, 1998). The same results are well documented in Illinois (Gharaibeh, 1999), which along with the state of Texas, uses CRCP as their long-life pavement type.

Punchout distress is the most severe performance problem for CRCP (Darter, 1979; McCullough, 1980; Zollinger, 1990). A punchout (Figure 1.1) is a depression that occurs at the edge of the pavement as a result of a structural failure caused by the action of heavy wheel loads after transverse cracks have suffered loss of load transfer efficiency (LTE). Punchouts are also accompanied by voids under the concrete slab and deterioration of the concrete/steel interface at the crack.
1.4.2 CRCP projects and research

The first use of continuous reinforcement in a concrete pavement occurred near Washington D.C in 1921, several years before the next recorded case, which was in 1938 near Indianapolis (Burke 1968). Experimental projects took place in Illinois and New Jersey in 1947, California in 1949, and Texas in 1951. The results generated valuable and promising information regarding the design of CRC pavements, and consequently during the 50’s several more CRCP sections were built in other states, anticipating the construction during the 60’s of important sections of the interstate highway system.

In the 70’s there was a notable research interest in the performance of the aging interstate CRCP system. During the 80’s, the research revolved around rehabilitation with asphalt and concrete overlays (Yoder, 1981), and corrosion of reinforcement (Korfhage, 1982; Hagen, 1985). In the 90’s, there were important advancements in the understanding of concrete pavements, and many of these advancements were applicable to CRCP such as concrete curing and base drainage. Among the examples of research performed specifically on CRCP are the investigation of the mechanism of punchout development (Zollinger, 1989), and studies of early-age behavior (Suh, 1992; Kadiyala, 1993), cracking characteristics (Haque, 1998), thermal stresses (Nishizawa, 1998), along with efforts related to corrosion (Verhoeven, 1993). In recent years the study of mechanistic analysis of CRCP distresses (Zollinger 1999; Selezneva, 2003) and the improved numerical modeling of CRCP (Kim, 2001) are examples of the research trends. To date there has
been no full-scale testing of CRCP under controlled loading conditions, which can significantly assist in model development and verification of the early-age characteristics of CRCP (crack spacing, width, LTE) and its repeated load performance. This research effort has focused on using the experimental findings of full-scale testing to improve CRCP prediction models and understand the performance and failure of CRC slabs.
CHAPTER 2  CRACK WIDTH IN CRCP

2.1. Introduction

Transverse cracks in continuously reinforced concrete pavement are closely spaced (generally 2 to 10 feet apart) and their width is generally a fraction of the width measured in joints and cracks in unreinforced concrete pavements. A measurable crack width develops when the contraction forces in the concrete, such as drying shrinkage and thermal contraction, overcome the slab-base friction forces and, most importantly, the bond between the concrete and steel.

2.2. Factors affecting crack width

Several factors affect crack width in CRCP, ranging from geometrical parameters to material properties. The long list of factors implies that there is more than one solution to minimize crack width, and it is of great importance to find the most cost effective solutions.

2.2.1 Effect of temperature at concrete setting and temperature changes

The traditional formula to predict crack opening in Jointed Plain Concrete Pavements (JPCP) and Jointed Reinforced Concrete Pavements (JRCP) due to changes in temperature is the following:

\[ \Delta_cw = \alpha \cdot \Delta T \cdot L \cdot C \]  

[2.1]

where

\( \Delta_cw \) = change in crack width
\( \alpha \) = coefficient of thermal expansion of concrete
\( \Delta T \) = drop in pavement temperature, measured at slab mid-depth
\( L \) = length of slab; actually the sum of the two half-slabs adjacent to the crack
\( C \) = restraint coefficient based on friction between slab and base

Lee (2001) demonstrated for unreinforced concrete pavements that this simple formula represents the changes in crack width when each term of the equation is carefully determined, in particular regarding to the effective slab length and base friction coefficient. In reinforced
concrete, however, the restraint of the steel prevents the direct application of the aforementioned formula, even though the effect of wider cracks with greater temperature drop still applies. The concrete setting temperature and the thermal expansion coefficient of concrete have been found to be two of the most sensitive variables determining CRCP behavior (Kim et al, 2003).

2.2.1.1 Concrete setting temperature

Given that temperature is a major factor in crack width, it is understandable that important efforts have been dedicated to accurately model and predict the temperature at the time of concrete set (Suh, 1992; Rasmussen and McCullough, 1998; Ramaiah et al, 2002). High setting temperatures cause wider cracks once the pavement temperature drops to its normal range. To prevent high setting temperature it is very important to control concrete placement temperature especially during hot weather conditions. Paving operations during hot days should be minimized because the heat of hydration cannot be readily dissipated into the atmosphere.

Shindler and McCullough (2002) studied CRC pavement data from the Texas Rigid Pavement database to evaluate the effect of concrete temperatures on long-term CRC pavement performance. The age of the sections was from 11 years to 36 years old, with the average age at around 23 years. The average number of failures per 1,000 feet section was calculated, with failures defined as severe punchouts, plus asphalt or concrete patches. The setting temperature was not available but they used the average maximum daily air temperature during the month of placement for the analysis. The air temperature during placement was grouped into five categories, from 50 to 100°F in 10°F increments. Figure 2.1 provides an overall summary of the 337 CRC pavement sections analyzed. The results are presented as percentage of failures in each category with respect to the total number of failures. From this figure it may be seen that there is an increased number of failures as the air temperature at placement increases. More than 36 percent of all failures occurred in the sections that were placed under conditions where the air temperature at placement exceeded 90°F. Around 62% of the failures occurred when the sections were constructed at air temperature higher than 80°F.
2.2.1.2 Coefficient of thermal expansion

The coefficient of thermal expansion (CTE) for concrete mixes used in pavements in Illinois is measured as part of the Long Term Pavement Performance (LTPP) Program of the Strategic Highway Research Program (SHRP). As of September 2004, 71 cores from Illinois sections had been tested using the “Standard Test Method for the Coefficient of Thermal Expansion of Hydraulic Cement Concrete” (AASHTO Designation TP60-00). On average, the CTE from all cores is 5.7 με/°F (Harman, 2004). In a few cases the major aggregate type was identified, as presented in Table 2.1.

<table>
<thead>
<tr>
<th>Primary Agg Class</th>
<th>Average CTE (x10^-6 ε/°F)</th>
<th>Min-Max CTE (x10^-6 ε/°F)</th>
<th>No. of Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>5.1</td>
<td>3.9-5.6</td>
<td>7</td>
</tr>
<tr>
<td>Dolomite</td>
<td>6.2</td>
<td>5.6-6.8</td>
<td>10</td>
</tr>
<tr>
<td>All</td>
<td>5.7</td>
<td>3.9-7.3</td>
<td>71</td>
</tr>
</tbody>
</table>

The study by Schindler and McCullough (2002) also examined the effect of the type of aggregate on failure occurrence, and found that significantly more failures occur in sections with siliceous
river gravel as compared to limestone sections. The increased number of failures in the river gravel sections was attributed to its higher coefficient of thermal expansion and poorer bond characteristics.

Zollinger et al. (1999) proposed a set of construction guidelines for CRCP based on coarse aggregate types and weather condition. Aggregates were divided in categories according to their CTE, and different recommendations were made for concrete placement under different weather conditions for each category of aggregates.

2.2.1.3 Temperature differential

When the surface is warmer than the bottom of the slab, a narrower opening may be observed near the surface in joints and cracks as compared to the opening at the bottom. Two mechanisms act together to cause this deformation. The first is differential thermal expansion and the second is rotation of crack face. The differential expansion corresponds to deformation occurring along horizontal planes while the rotation of the faces is caused by curling. Poblete et al. (1988) showed that the joint opening at the surface of in-service PCC pavements is different from joint opening at the bottom in undoweled pavement slabs with the slab’s temperature gradient being the major factor. The effect of temperature differential in transverse cracks in CRCP seems to have been first mentioned in the literature by Friberg (1960), who attributed some of the changes in crack width measured by Witkoski and Shaffer (1960) to changes in “the slopes of warped concrete”.

2.2.2 Effect of drying shrinkage, creep and relative humidity

Drying shrinkage causes cracking in reinforced concrete, especially at early age when the concrete tensile strength has not fully developed. Shrinkage continues to widen cracks even after initial cracking has occurred. Since drying shrinkage has a faster development at the surface, there is a non-uniform shrinkage distribution and a tendency for the slab to curl upward, especially in jointed pavements (Eisenmann and Leykauf, 1990; Kadiyala and Zollinger, 1993; Heath and Roesler, 1999). As in the case of temperature shrinkage, the bond-slip interaction between concrete and steel controls the actual crack opening, thus even well predicted unrestrained shrinkage cannot be used directly to predict crack width. Since moisture loss is the
underlying cause of drying shrinkage, increases in the internal relative humidity partially reverse for some of the volumetric contraction of the pavement (Mindess and Young, 1981), hence making crack width smaller than expected from drying shrinkage alone. Irreversible drying shrinkage is the part of the total drying shrinkage during the first drying cycle that cannot be regained during subsequent wetting and drying cycles. A further complication is that the induced concrete shrinkage stresses are modified by tensile creep relaxation, which may even prevent cracking when shrinkage develops slowly (Neville, 1996).

Given the low strength of early aged concrete, and since moisture diffusion is typically very high during the first few days, shrinkage strain is one of the dominant factors in the development of initial cracking besides thermal contraction. Drying shrinkage depends to a great extent upon the water-cement ratio used to place the concrete pavement. Other factors related to the magnitude and rate of shrinkage are degree of hydration, moisture diffusivity, and the method of curing used during the concrete hardening process.

2.2.3 Effect of bond-slip between concrete and steel

Crack width increases with temperature drop and concrete shrinkage. Stress and strain vary inside each pavement segment according to the distance to the closest crack. If the crack extends through the slab thickness, and assuming that drying shrinkage and/or temperature drop has occurred, then at the crack face the steel is in a state of maximum tensile stress and the concrete stress is zero. Inside a panel, away from a crack, the steel is in compression because it is fully bonded to the contracting concrete. The transition from tension to compression occurs in the bond development or bond-slip zone. The bond stress is the interfacial shear that takes place at the boundary between steel bar surface and the concrete. In order to predict crack width, the concrete and steel stresses at increasing distance from the crack face need to be defined relative to the bond stress, which depends on the concrete strength and mechanical shape of the bearing face of the ribs on the longitudinal bar. The bond stress distribution represents one of the major complications in the accurate modeling of CRCP behavior.

Moore and Lewis (1962) studied bond failure in CRCP slabs. They developed a plastic gage that measured movement of the steel bar with respect to the concrete as the slabs were subjected to a
high number of vertical loads. They concluded: 1) that bond stress at points within a region of slip may considerably exceed the average bond stress over the region; 2) heavy repetitive vertical loads increased the magnitude of slip in regions adjacent to cracks; and 3) no slip was detected between reinforcement bars and the concrete beyond 12 inches from the crack.

The interaction between steel and concrete in the bond-slip region also has an effect in the formation of punchouts in CRCP. Pullout failure is the mechanism by which a void is created around the rebar by tension in the steel, as shown in Figure 2.2. These voids initiate near the crack face when the steel stress reaches a threshold and these stresses crush the concrete (Goto, 1971).

Field investigations by Zollinger (1989) found evidence of these voids around reinforcement even in early stages of punchout development as seen in Figure 2.3. His results suggest that there is a significant loss in bond within 2 inches of the transverse cracks. The pullout-cone failure reduces the bond stiffness between concrete and steel, allowing wider cracks, and at the same
time minimizes the effect of rebar as a load transfer mechanism and facilitates progression of crack deterioration and steel rupture.

Figure 2.3. Void around rebar and contribution to faulting (after Zollinger, 1989)

In addition to the effect on crack width, taking into account the bond slip relationship helps to better predict crack spacing, and the location of cracks could be estimated using a minimization of energy approach (Chen, 2003). More about the relationship between bond-slip and crack width will be discussed in section 2.5.

2.2.4 Effect of base friction

The sliding of the concrete slab along the base develops frictional force at the interface. Base friction is an important factor in the early development of CRCP cracks. As in the case of temperature effects, numerous investigations have looked into the effects of base friction on unreinforced concrete pavements, but little research is found regarding base friction for CRCP. The most common types of material used under the CRCP slabs are asphalt or cement-aggregate
mixtures (39 and 22% of LTPP CRCP sections, respectively), closely followed by gravel bases (20%). In addition to higher resistance to erosion, the first two types of base provide higher friction due to adhesion and interlock.

In the case of loose unbound bases, the sliding plane was observed at the slab-base interface. For stabilized bases, the sliding plane was observed down in the base (tenth of an inch beneath the interface). When a bond-breaker was used, such as thin asphalt layer on cement-stabilized base, the failure plane occurred at the interface of the thin asphalt layer and the cement-stabilized base. Friction force increases in a parabolic pattern with slab displacement (Lee, 2000). Since concrete contraction is zero at the mid-panel (assumed symmetry), the frictional force does not exist there, and is maximum near the crack faces. This is shown in Figure 2.4a. Figure 2.4b to Figure 2.4d present typical distribution of bond stress and concrete and steel stress.

![Figure 2.4. Typical distribution of base friction and other stresses over a segment of cracked CRCP (after Won et al, 1991)](image-url)
2.2.5 Reinforcing steel characteristics

There are several pavement design variables related to reinforcement steel bars that have significant effect on crack width. They include such factors as percentage of longitudinal steel, longitudinal bar diameter, depth of cover, number of layers of longitudinal steel, and even steel rib pattern characteristics.

2.2.5.1 Amount of steel

The purpose of the reinforcing steel is to limit the contraction/expansion movements of concrete. A very low percentage of steel will cause crack spacing slightly smaller than unreinforced concrete. In terms of crack spacing, steel percentages of 0.55 to 0.70 have provided suitable CRC pavement performance (Zollinger, 1999). A clear example of the effect of amount of steel in crack spacing is presented in Figure 2.5 from the experimental CRCP sections in Vandalia placed on soil (Burke, 1968). These results are for 7-inch pavement, but a similar trend was observed in the 8-inch sections. According to McCullough (1975), field observations and design theories confirm that crack width in CRC pavements decreases with an increase in percentage of longitudinal reinforcement.

![Figure 2.5. Crack spacing over time for various steel percentages for CRCP placed on soil (after Burke, 1968)](image-url)
2.2.5.2 Bar Size and Bond Characteristics

Bar size has an influence on crack development in that the restraint of the longitudinal steel depends on the bond area provided by the reinforcing bar (Zollinger, 1999). The stress transfer from the concrete to the longitudinal steel depends on the reinforcing steel surface area and the shape of the longitudinal steel surface. For the same percent of longitudinal steel, a smaller size bar results in a larger steel surface area. This increases stress transfer from the steel to the concrete and results in tighter cracks. The effect of epoxy-coated steel on crack width has also been studied. Results in two sections of interstate highway in central Oklahoma show that epoxy coating has no significant effect on crack spacing or crack width (Zwerneman, 1995).

2.2.5.3 Depth of Cover of Longitudinal Steel

The volumetric changes are greatest at the pavement surface and decrease with depth. If the steel is placed near the surface of the slab, the restraint to the induced movements increases which results in an increase in the number of transverse cracks. Figure 2.6 shows the effect of the depth of steel on crack spacing for Illinois CRC 7 and 8 inch pavements with deformed bars and wire fabric reinforcement (Dhamrait, 1973). A survey of CRC pavements in South Dakota shows an average crack spacing of 1.7 feet with the steel 2.5 inch below the surface, and an average spacing of 2.9 feet with the steel 3.68 inch below the surface (Won, 1991). An aspect related to the depth of steel is the use of two layers of longitudinal steel. The position of the top layer of steel has been shown to be significant in past studies and the use of two-layer placements has been adopted in Texas DOT construction standards for pavements thicker than 13 inch in order to maintain adequate bar spacing for construction purposes (Won, 1991). Two layers of reinforcing steel require two sets of transverse chairs to support the longitudinal steel, which have caused a weakened plane and transverse cracking when the two transverse chairs are placed in the same plane (Zollinger, 1999).
2.3. Variations in crack width

Crack width is not a constant value along a crack in concrete structures. Crack width varies with slab depth and it can vary horizontally from the center of the lane to the edges. Furthermore, it varies between different transverse cracks.

2.3.1.1 Permanent difference in crack width through slab thickness

Studies (Witkoski 1960; McGhee 1974) of pavement cores have shown a variation in crack width through the depth of the slab. The crack width decreased with depth and in some cases became almost non-existent at the bottom of the core. Other core studies showed that crack became discontinuous in the intermediate vicinity of the reinforcing bar and widened towards both the top and bottom surfaces. Cores taken from sections in Illinois (Lindsay, 1959) indicated clearly that cracks were widest at the surface (Figure 2.7).
The prediction of crack width has to make reference to the depth in the slab. The common points found in the literature to report crack width are either the pavement surface, where crack width can be readily measured, or at the depth of the steel reinforcement, where the balance of forces between concrete and steel are used to calculate a theoretical crack width.

2.3.1.2 Permanent difference in crack width along horizontal surface

Besides the changes with depth, crack width can also vary across the width of the pavement for a single transverse crack. Natural occurring cracks generally meander and sometimes do not cross the entire traffic lane. There are cracks that originate at an edge but become progressively narrower until they disappear. There are also cracks that originate within the lane but do not extend to either edge. Some cracks become divided and form Y-cracks. These patterns have been
reported in the literature by several authors (van Breemen, 1959; Tayabji 1998), and exemplify the difficulties in defining a crack width, even if well measured and at a fixed depth. Average crack width in a pavement section suffers from some lack of meaning when incomplete cracks are present.

2.3.1.3 Short-term variation of crack width through the slab thickness

Traffic loads create rotation of the crack faces. When a load is placed on top of a transverse crack, it produces crack opening at the bottom and crack closing at the top. The opposite is true when the load is moved some distance away from the crack. Figure 2.8 represents the changes in crack width along the depth, in terms of rotation, caused by load reversal. The first plot in Figure 2.8 is when the load is over the crack while the second plot is when the load is located some distance from the crack.

Temperature curling also causes crack rotation. A positive gradient (top warmer than bottom) helps to close the top part of the crack, while a negative gradient opens the crack up. These differential thermal movements are in addition to the linear contraction/expansion phenomena.
2.3.1.4 Measurement of crack width

Different means have been used to measure crack width in CRCP. They include crack width comparators, microscopes with scales, gages placed on embedded plugs, and displacement transducers (LVDTs). It is important to mention that there exists a difference in measuring absolute crack width and change in crack width. Crack comparators are extremely easy to use, but not very accurate. They provide a rough, direct measure of absolute crack width. Microscopes also allow for the measurement of absolute crack width. However, the use of microscopes is limited by the subjectivity of the readings made by different operators. The placement of the microscope at the same location from one measurement time to the next presents difficulties in reporting the long term variations in crack width.

Dial gages that precisely measure distance between fixed points across a crack are more accurate, and were used extensively in the early years of CRCP research. Gages made with invar offer the advantage of less error caused by temperature changes. Generally the fixed points in the concrete were marked with brass plugs inserted on the surface of the pavement. If the plugs are placed in the concrete before the crack has occurred, then the measurements represent absolute crack width. The results, however, represent the total change in length between the fixed points, and require a priori knowledge of the crack location before final concrete set. Linear variable displacement transducers (LVDTs) are now more common because they offer the advantage of automatic, continuous recording with high accuracy. Like dial gages, they measure the distance between two fixed points, and if the initial measurement is made before the crack occurred then they can give the absolute crack width. Typically, the fixed point measurement techniques with dial gages or LVDTs are used only to determine changes in crack width.

2.4. Existing crack width data

Given its relevance to the performance of the pavement structure, previous studies have included crack width measurement taken from in-service CRCP sections. Burke and Dhamrait (1968) reported CW measured with a microscope at “some distance down the surface” for 7 and 8 inches thick slabs, and at 1, 10, and 20 years after construction and for various steel contents.
The results ranged from 0.08 to 1.07mm, and are presented in Figure 2.9. The one percent steel content sections had the smallest CW and the crack widths did not significantly change after 10 years in service. These results were obtained at the Vandalia test pavement, which was the first experimental CRCP in Illinois. Although the construction methods have changed since this pavement was built, the measured CW data still serve as a reference.

![Figure 2.9. Crack widths in first experimental CRCP section in Illinois (after Burke and Dhamrait 1968)](image)

Dhamrait et al (1973) studied cores removed from 12 experimental CRC pavements constructed throughout Illinois during 1963-66. Crack width at the level of steel was measured in 108 of 151 cores. The rest of the cores were broken or disturbed when removed from the pavement. Crack width was found to be less than 0.2mm for 79 out of the 108 cores (73%). The other 29 cores had crack width measured at the depth of steel from 0.2 to 2.7mm.

Gharaibeh et al (1999) noted CW of 0.475, 0.793, and 0.831mm at depth of steel in 9-year old pavements with reinforcement located at increasing depths (2, 3, and 4 inches, respectively) in 8-inch slabs. McCullough (1981) presented measurements of CW at the surface, also with a microscope, and concluded that pavement with crack width of less than 0.51mm (measured in summertime) presented no spalling, and defined 1.0mm as maximum limit for design when the
pavement is at its coolest temperature. More recently, the same author proposed a maximum crack width for design of 0.63mm at 0°C (McCullough and Dossey, 1999). The European standard (PIARC, 1994) calls for crack width no larger than 0.5mm.

Regarding jointed concrete pavements, Chou et al. (2004) reported CW at joints in 16-inch thick airport slabs with 23 feet joint spacing. They used optical fiber sensors at 6 inches below surface and measured initial CW of 0.29mm. A maximum CW of 1.21mm was measured during the cold temperature season. CW at the Denver International airport pavement can be calculated from data presented by Rufino and Roesler (2004). Maximum CW at mid-depth in 16.5-inch slabs with 20 feet joint spacing was 0.50mm and 2.84mm across joints with tie bars and with dummy joints, respectively. For highway pavements, Lee (2001) analyzed joint movement from sections included in the Long-Term Pavement Performance (LTPP) monitoring program and concluded that strong linear relationships exist between joint opening and temperature. Using his effective ratio of joint movement, and assuming a maximum of 25°C drop in pavement temperature from the time of construction, maximum openings between 0.45mm and 3.15mm can be expected.

CW in CRCP is considerably smaller than in jointed pavements. In general, CW in CRCP can be expected to be smaller than 1.0mm, and most of the time smaller than 0.5mm, while CW in jointed pavements can exceed 3.0mm. It is possible for a CW of near zero to occur in CRCP depending on the setting temperature and the time of year the CW measurement was made.

2.5. Crack width prediction models and Mechanistic-Empirical Pavement Design Guide

Different models have been proposed to predict crack width in CRCP. The research presented here does not attempt to cover all the variables that determine crack width, but rather investigates into the measurement of absolute crack width and how different factors affect it. The Mechanistic-Empirical Pavement Design Guide, referred to hereinafter as M-E PDG, is a pavement design tool based on existing mechanistic-empirical technologies. It includes a module for prediction of performance of CRC pavement which is heavily based on crack width and for that reason has been included as part of this investigation. Three other existing models on crack width prediction are briefly described in this section.
2.5.1 Formulas developed by Vetter

Vetter (1932) developed formulas for crack spacing in reinforced concrete based on stress diagrams for drying shrinkage and drop in temperature. The steel, concrete, and bond stress distribution diagrams and relationships are presented in Figure 2.10 and Figure 2.11 (after Zollinger 1989).

![Stress distribution between cracks of CRC member subjected to shrinkage according to Vetter (Zollinger 1989)](image)

Figure 2.10. Stress distribution between cracks of CRC member subjected to shrinkage according to Vetter (Zollinger 1989)
Figure 2.11. Stress distribution between cracks of CRC member subjected to temperature drop according to Vetter (Zollinger 1989)

The terms in the formulas presented above are:

- $A_s, A_c$ : cross sectional area of steel and concrete
- $E_s, E_c$ : modulus of elasticity of steel and concrete
- $f_{sz}$ : steel tension stress due to shrinkage at the crack face
- $f_{tz}$ : concrete tension stress due to shrinkage strain at center of crack spacing
- $f_{t\phi}$ : concrete tension stress due to temperature drop at center of crack spacing
- $f'_{sz}$ : steel compressive stress due to shrinkage at center of crack spacing
- $L$ : maximum possible distance between cracks
- $t_m, t_t$ : temperature drop of the pavement at the surface and at the level of steel
- $u$ : uniformly distributed bond stress
- $x$ : distance from a crack measured along the reinforcement
- $y$ : bond development under a temperature drop
- $z$ : drying shrinkage
- $\Sigma_0$ : rebar circumference
- $\alpha_s, \alpha_c$ : coefficient of thermal expansion of steel and concrete
- $\varepsilon_s, \varepsilon_c$ : strain in steel and concrete
\( \phi_s \) : steel tension stress due to temperature drop at the crack face  
\( \phi'_s \) : steel tension stress due to temperature drop at center of crack spacing

Vetter’s approach was an early attempt to determine the amount of reinforcement necessary in CRC pavements. He showed that when reinforced concrete cracks due to shrinkage, the shrinking concrete grips the steel by bond in an extended region near the cracks, causing the concrete to go into tension. The bond force is assumed uniform in the region of grip near the cracks and zero in the central region between cracks. This action causes the steel near the cracks to go into tension and in the central region between cracks to go into compression. It is important to stress the fact that the concrete slips a little in the region of bond, but since the bars are deformed, bond forces continue to be developed. The formulas for the average crack spacing based on shrinkage and on temperature drop are, respectively:

\[
L = \frac{S_c^2}{q \cdot n \cdot p^2 \cdot u \cdot (z \cdot E_c - S_c)} \quad [2.2]
\]

and

\[
L = \frac{S_c^2}{q \cdot n \cdot p^2 \cdot u \cdot (\alpha_s \cdot t_m \cdot E_c - S_c)} \quad [2.3]
\]

where \( L, u, z, E_c, \) and \( \alpha_s \) are as described before, and

- \( S_c \): tensile strength of concrete
- \( q \): ratio of bond area (perimeter) to area of steel \((= \pi D^2/4 = 4/D)\)
- \( n \): modular ratio \((E_s/E_c)\)
- \( p \): percent reinforcement

A formula for the average crack spacing when both shrinkage and temperature drop occur simultaneously is derived by considering the combined stress diagram for the steel and concrete which is expressed in a simplified form as:

\[
L = \frac{S_c^2}{q \cdot n \cdot p^2 \cdot u \cdot (\alpha_s \cdot t_m \cdot E_c - S_c)} \quad [2.4]
\]
The last expression allows an interpretation of the effect of variables such as bond stress, percentage of reinforcement, or bar size (a higher $q$ value is obtained with smaller bars). Maximum crack width can be obtained by summing up the changes in slip between concrete and steel. Zuk (1959) developed a formula for crack width at the steel depth based on some of Vetter’s expressions for bond length and tensile stress in concrete. His formula for crack width comprised only the shrinkage effect, but the temperature can be easily added to obtain:

$$ CW = L(z + \alpha_c \cdot t_m) + \frac{f_i}{E_c} \left( L - \frac{f_i \cdot d_b}{4 \cdot u \cdot p} \right) $$ \[2.5\]

The limitations of this approach to analyze CRCP are that it assumes uniform bond stress and does not include slab deformation caused by environmental curling/warping.

### 2.5.2 Texas CRCP program

The CRCP software is a design tool that has been developed at the University of Texas at Austin, and whose first version dates from the mid 1970’s. CRCP-8, released in 1995, was the last version based on a one-dimensional analysis (similar to Vetter’s approach). Crack width in this version was determined using the following regression model, fit to data from pavement sections in Texas (Jimenez et al, 1992).

$$ CW = 0.028 + 740 \times Z - 260 \times 10^{-11} \times E_i + 29 \times \alpha \times \Delta T - 203 \times 10^{-4} \times p \times \phi $$ \[2.6\]

where,

- $CW$: crack width at the surface, inch
- $Z$: residual shrinkage, defined as the difference between the total expected shrinkage and the shrinkage that has occurred since construction to the time when the crack formed, inch/inch
- $E_i$: elastic modulus on the day the crack occurred, psi
- $\alpha$: thermal coefficient of the concrete, $1/°F$
- $\Delta T$: temperature differential, setting temperature minus temperature at time of measurement, $°F$
- $p$: reinforcing steel percentage (expressed as fraction)
- $\phi$: reinforcing bar diameter, inch.
Later versions of the program are based on results from two-dimensional finite element models (Kim et al., 1998; 2001), verified with runs of 3-D models, and are capable of including considerations of non-linear variation in temperature and drying shrinkage, non-linear bond-slip relationships between concrete and steel bars, and the ability of changing location of the longitudinal reinforcement. The CRCP program has a Window-based interface, and its outputs consist of time history of mean crack width, mean crack spacing, and mean steel stress at cracks, as well as punchout prediction in terms of failures per mile. The time history results are given at day 1 through 28, and then at 120 days. The crack width prediction is for width at the surface and parts of the mechanistic model were calibrated with field data. A validation study of the crack spacing prediction capability of the CRCP-8 program (Schindler, 2000), concluded that the software is especially accurate in the critical range of crack spacing less than 3 feet. The limitations of the CRCP model are that it predicts crack width only at the surface, it does not include slab-base friction, and it has been calibrated only with data from sections in the state of Texas.

2.5.3 Sato et al. model

Sato et al. (1989) developed a series of equations to calculate stress in the reinforcement in CRCP through the refinement of the bond stress distribution. Starting with the basic differential equation for bonding, they integrate it by regions (with multi-linear approximations) and used different boundary conditions to arrive at solutions of crack width at the top of the slab, bottom of the slab, and at the depth of the steel. The equations have been omitted here for brevity. Although rigorous, the equations by Sato et al. are difficult to implement and have not been used extensively.

2.5.4 Mechanistic-Empirical Pavement Design Guide (M-E PDG)

The M-E PDG (ERES, 2004) predicts failure of CRCP in terms of the accumulated fatigue damage associated with the formation of longitudinal cracks. This failure corresponds to edge punchouts, and its development is based on a sequence of events related to crack width, loss of load transfer, and foundation support changes. The formulas for crack spacing and crack width are based on work by Vetter (1933) and Reis et al (1965):
2.5.4.1 Crack spacing

The M-E PDG offers an expression for crack spacing, although an externally provided value for mean crack spacing can also be used (such as the one that would be obtained with the CRCP-8 program). Equation [2.7] determines crack spacing:

\[
L = \frac{f_{28} - C \sigma_0 \left(1 - \frac{2\zeta}{h}\right)}{\frac{f}{2} + \frac{U_m P}{c_1 d_b}}
\]  

[2.7]

where

- \(L\) : average crack spacing, inch
- \(f_{28}\) : concrete tensile strength at 28-days, psi
- \(C\) : Bradbury’s curling/warping stress coefficient
- \(\sigma_0\) : Westergaard’s nominal stress factor, based on maximum strain between pavement surface and slab bottom (curling and warping).
- \(\zeta\) : depth to steel layer, inch
- \(h\) : slab thickness, inch
- \(f\) : base friction coefficient based on base type
- \(U_m\) : peak bond stress, psi
- \(P\) : ratio of area of steel reinforcement to area of concrete, percent
- \(c_1\) : first bond stress coefficient
- \(d_b\) : reinforcing steel bar diameter, inch

The first bond stress coefficient \((c_1)\) depends on crack spacing, which requires equation [2.7] to be solved iteratively.

2.5.4.2 Crack width

Crack width is calculated at time “\(i\)” according to equation [2.8]:

\[
CW_i = L \cdot \left(e_{SHR_i} + \alpha_{PCC} \Delta T_i - \frac{c_{21} f_{\sigma i}}{E_{PCC}}\right)
\]  

[2.8]
Where

\[ CW \] : average crack width at the depth of the steel, inch
\[ L \] : average crack spacing, inch
\[ \varepsilon_{\text{SHR}} \] : unrestrained concrete drying shrinkage at the depth of the steel, strains
\[ \alpha_{\text{PCC}} \] : concrete coefficient of thermal expansion, 1/°F
\[ \Delta T_\text{c} \] : drop in PCC temperature from the concrete “set” temperature at the depth of the steel, °F
\[ c_2 \] : second bond slip coefficient
\[ f_{\sigma t} \] : maximum longitudinal tensile stress in the concrete at the steel level, psi
\[ E_{\text{PCC}} \] : concrete modulus of elasticity, psi

Subscript “\( i \)” is used in reference to the variables that change continuously during pavement life. The M-E PDG predicts these variables for each month in the life of the pavement. Some variables are repeated for the same month every year, while some others are assumed to vary according to four seasons, and some progress as the pavement ages. The time frame considered to determine changes in crack width does not need to be one month. Hourly changes can be computed as long as the input variables are properly taken into account.

The CW formula in M-E PDG can be interpreted as the crack width being the result of concrete drying and temperature shrinkage less the restraining effects from the steel and base friction expressed as a negative concrete strain. These three elements take into account all the parameters discussed earlier in this chapter that affect crack width. A more detailed look into the three elements of crack width is presented below.

**Shrinkage**

Unrestrained drying shrinkage is a function of the concrete’s ultimate drying shrinkage and internal relative humidity, as calculated in Equation [2.9]

\[
\varepsilon_{\text{SHR},i} = \varepsilon_{\infty} \left(1 - rh_{\text{PCC}}^3\right)_i
\]  

[2.9a]
where $\varepsilon_\infty$ is the concrete ultimate shrinkage and $\text{rh}_{PCC}$ is the relative humidity at the depth of the steel, which is a function of time, ambient humidity, water cement ratio, and depth below the pavement surface. The general formula and incremental formulas are:

$$\text{rh}_{PCC} = \text{rh}_a + (100-\text{rh}_a)f(t) \tag{2.9b}$$

$$\text{rh}_{PCC,i} = 0.5(\text{rh}_{PCC \text{ annual}} + \text{rh}_{PCC \text{ monthly}}) \tag{2.9c}$$

where,

$\text{rh}_{PCC,i}$ : relative humidity in the concrete at steel depth for each month $i$, %

$\text{rh}_a$ : average ambient relative humidity annual ($\text{rh}_a \text{ annual}$) or monthly ($\text{rh}_a \text{ monthly}$)

$$f(t) = \frac{1}{(1 + \frac{t}{35} \cdot (25.4 \cdot \zeta \cdot 1.35 \cdot \frac{\text{w}/\text{c} - 0.19}{4}))} \tag{2.9d}$$

$t$ : drying time, days

$\zeta$ : depth to steel, inches

$\text{w}/\text{c}$ : water/cement ratio

For ultimate shrinkage, the following formula is recommended:

$$\varepsilon_\infty = C_1 \cdot C_2 \cdot (26\text{w}^{2.1}(f'_c)^{0.28} + 270) \tag{2.9e}$$

where,

$\varepsilon_\infty$ : ultimate shrinkage, microstrain

$C_1$ : cement type factor: 1.0 for type I, 0.85 for type II, and 1.1 for type III cement

$C_2$ : type of curing factor: 0.75 if steam cured, 1.0 if cured in water or 100% relative humidity, and 1.2 if sealed during curing (curing compound)

$w$ : water content, lb/ft$^3$ for the PCC mix under consideration.

$f'_c$ : 28-day PCC compressive strength, psi.

Drying shrinkage also affects crack width indirectly because the difference in moisture content induces a warping stress.
Temperature drop

The deformation caused by the drop in temperature is the product of the coefficient of thermal expansion and the actual drop in temperature from the zero-stress temperature. The minimum temperature that the pavement can reach depends mostly on the climate at the pavement location, and is practically impossible to control, however the engineer has some control over the zero-stress temperature.

Zero-stress temperature can be input directly into the M-E PDG or it can be estimated from the monthly ambient temperature and cementitious content using the equation shown below, which is based on daytime construction with curing compound. The allowable range with this formula is from 60 to 120°F.

\[
T_z = CC \cdot 0.59328 \cdot H \cdot 0.5 \cdot 1000 \cdot \frac{1.8}{1.1 \cdot 2400} + MMT
\]  

[2.10]

where,
- \( T_z \): temperature at which the PCC layer exhibits zero thermal stress
- \( CC \): cementitious content, lb/yd³.
- \( H \): Heat of hydration per unit weight

\[ H = -0.0787 + 0.007 \cdot MMT - 0.00003 \cdot MMT^2 \]

\( MMT \): mean monthly temperature for month of construction, °F.

Concrete strain

The deformation in the concrete at the steel level is a function of the tensile stress \( f_{\sigma t} \), which is comprised of three terms, each with clear origin and significance:

\[
f_{\sigma t} = \frac{L \cdot U_m \cdot P}{c_{1i} \cdot d_b} + C\sigma_0 \left(1 - \frac{2c}{h}\right) + \frac{L}{2} f
\]  

[2.11a]

The first term of this formula accounts for the stress in the concrete caused by the presence of the steel, which is transmitted through the rebar deformations (ribs) and vary along the rebar. The maximum bond stress, \( U_m \), as well as the bond stress coefficients \( c_1 \) and \( c_2 \), come from assumed bond stress distributions and depend on crack spacing and concrete properties:

\[
U_m = 0.002 \cdot k_1
\]  

[2.11b]
where $k_1$ is called bond slip coefficient, and it is equal to 0.1172 times the compressive strength ($f'_c$) of concrete.

$$c_1 = 0.577 - 9.499 \times 10^{-9} \ln \epsilon_{\text{tot-}\zeta} + 0.00502 \ln \bar{L} \times (\ln \bar{L})$$  \[2.11c\]

$$c_2 = a_1 + \frac{b_1}{k_1} + \frac{c_1}{L^2}$$  \[2.11d\]

where $a$, $b$ and $c$ depend on $k_1$, $L$, and $\epsilon_{\text{tot-}\zeta}$, which is the total strain at the depth of steel, based on temperature drop and shrinkage, which should not be confused with $\epsilon_{\text{tot-}\Delta}$ shown later. The first and second bond slip coefficients, $C_1$ and $C_2$, are need for equations 2.11a and 2.8, respectively.

The second term of formula [2.11a] is the environmental tensile stress in the concrete. Bradbury’s correction factor ($C$) is obtained once the radius of relative stiffness is calculated. Westergaard’s nominal stress is calculated using an equivalent strain difference between pavement surface and slab bottom, which involves the estimation of temperature difference and moisture through the thickness of the pavement.

$$\epsilon_{\text{tot-}\Delta} = \alpha_{\text{PCC}} \cdot \Delta_{\text{eqv}} + \epsilon_{\infty} \cdot \Delta (1 - r_{\text{PCC}}^3)_{\text{eqv}}$$  \[2.11e\]

The last term of formula [2.11a] is the expression for concrete stress caused by base/slab friction. Typical values of base/slab friction coefficient are presented in the M-E PDG, and for instance, for asphalt treated bases, it ranges from 2.5 to 15.

Some formulas presented in the M-E PDG documentation have been left out, even though they are involved in crack width determination (for instance the coefficients in equation [2.11d] and the equivalent temperature and moisture differential in equation [2.11e]). Crack width predicted with the M-E PDG formulas will be used and discussed in Chapter 5. The intention here has been to present the calculation philosophy and the fundamental parts of the procedure. Additional formulas are included in the M-E PDG to compute time-dependent variables that affect crack spacing as well as crack width, in addition to expressions for the estimation of base
erodibility, load transfer efficiency, loss of shear capacity and number of punchouts per mile. These additional formulas and the ones that have been omitted here can be found in Appendix LL of the M-E PDG (ERES, 2004).

2.5.5 Sensitivity analysis of the crack width formula in the M-E PDG

A sensitivity analysis was performed to evaluate the effect of different parameters in the M-E PDG crack width formula. The seven variables presented in the general formula (Eq.2.8) were studied by initially assuming low and high values that were compared to the baseline case. To obtain the low and high values for the sensitivity analysis, initial minimum and maximum were considered for each variable based on extreme possible cases, and these extremes were trimmed down to account for more probable conditions. A list containing the minimum, low, baseline, high, and maximum values is presented in Table 2.2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>L (inch)</th>
<th>ε_{SHR} (μs)</th>
<th>α_{PCC} (μs/°F)</th>
<th>ΔT (°F)</th>
<th>ε_2</th>
<th>f_{σ1} (psi)</th>
<th>E_{PCC} (10^6 psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>12</td>
<td>0</td>
<td>2.30</td>
<td>0</td>
<td>0.99</td>
<td>206</td>
<td>3.0</td>
</tr>
<tr>
<td>Low</td>
<td>24</td>
<td>9</td>
<td>2.79</td>
<td>8</td>
<td>1.20</td>
<td>255</td>
<td>3.7</td>
</tr>
<tr>
<td>Baseline</td>
<td>48</td>
<td>86</td>
<td>5.00</td>
<td>38</td>
<td>2.00</td>
<td>491</td>
<td>8.0</td>
</tr>
<tr>
<td>High</td>
<td>120</td>
<td>493</td>
<td>6.71</td>
<td>72</td>
<td>2.90</td>
<td>648</td>
<td>9.3</td>
</tr>
<tr>
<td>Maximum</td>
<td>240</td>
<td>702</td>
<td>7.20</td>
<td>80</td>
<td>3.11</td>
<td>697</td>
<td>10.0</td>
</tr>
</tbody>
</table>

The possible minimum and maximum conditions were assumed as follows:

- Crack spacing, L: minimum one foot, maximum 20 feet. Values based on field data
- Drying shrinkage, ε_{SHR}: minimum zero shrinkage assuming 100 percent relative humidity, maximum 702 microstrain assuming 750 microstrain of ultimate shrinkage and 40 percent relative humidity
- Coefficient of thermal expansion, α_{PCC}: 2.3 and 7.2 microstrain per °F, minimum and maximum respectively as reported from cores from nationwide LTPP sections
- Temperature drop, $\Delta T$: minimum zero considering concrete pavement operation occurred at time of lowest annual temperature, maximum 80°F assuming paving at 100°F and minimum temperature 20°F.
- Second bond slip coefficient, $c_2$: 0.99 and 3.11 minimum and maximum respectively, combining extreme crack spacing, total shrinkage, and bond slip coefficient.
- Tensile stress, $f_{\sigma t}$: 206 and 697 psi minimum and maximum respectively, according to calculations using between 0.6 and 1.0 percent of steel, extreme crack spacing, temperature differential between 0 and 20°F, and base friction between 4 and 12.
- Elastic modulus of concrete, $E_{PCC}$: 3 to 10 millions psi.

The values used for the analysis were calculated from the extreme values. The “low” value for each parameter was obtained by increasing the minimum by 10 percent of the difference between min and max. The “high” value represents a reduction of 10 percent below the maximum assumed. The exception is the drying shrinkage where more reasonable numbers were judged based on possible internal relative humidity of 70 to 99 percent and ultimate shrinkage of 300 and 750 microstrain. The result of the sensitivity analysis is shown in Figure 2.12, which presents the change in crack width as a result of varying each parameter from the baseline case to the low or high value.

![Figure 2.12 Change in crack width due to variation in input parameters](image)

Figure 2.12 Change in crack width due to variation in input parameters
The more influential variables in the formula are the drying shrinkage and the temperature drop, followed by the crack spacing. The effect of each of these three parameters is shown in detail in Figure 2.13. The behavior of CW with respect to crack spacing, in which there is initially an increase and later a decrease, is mathematically explained by the fact that the crack spacing, $L$, is a factor to the entire CW formula and it is also present in the stress term of equation 2.8 (see Eq. 2.11a), thus creating a quadratic expression in $L$. This causes the nearly parabolic representation. This sensitivity analysis applies only to the baseline case assumed, varying one parameter at a time, but considering the effect that each change has on the other parameters to obtain the final effect on crack width.

Figure 2.13. Effect of the drying shrinkage, temperature drop, and crack spacing on crack width
2.6. Summary of Chapter 2

Chapter 2 presented the most relevant information found in the literature regarding crack width in CRCP. The main factors affecting crack width have been described: state of pavement temperature (drop in pavement temperature from the time of concrete setting, coefficient of thermal expansion, and temperature differential), drying shrinkage and concrete internal relative humidity, bond between concrete and steel, interface friction between the slab and base, and characteristics of the reinforcing steel. Crack width values reported in concrete pavement research indicated that 0.1 to 1 mm is a reasonable typical opening for transverse cracks in CRCP, and crack width can even be near zero depending on the conditions at time of measurements. It is clear that measurement of crack width is a difficult task and the major efforts have been carried out in the states of Texas and Illinois.

Different models to predict crack width were reviewed but there is a lack of field measurements to validate the predicted values. The crack width prediction formula that is part of the Mechanistic Pavement Design Guide was described in detail since it integrates the main factors that affect crack width and accounts for the effect of crack width magnitude on the shear capacity across transverse cracks.
CHAPTER 3  FULL-SCALE SECTIONS, INSTRUMENTATION, AND ATLAS LOADING SYSTEM

3.1. Introduction

The elements of the experimental research are introduced in this chapter, which includes the design and material characteristics of the pavement sections, the instrumentation used to capture the pavement responses, and the accelerated loading device that trafficked the CRCP sections.

3.2. Pavement test sections

Ten experimental sections were built at the Advanced Transportation Research and Engineering Laboratory (ATREL) in December 2001 with funding from the Illinois Department of Transportation. The objective of the sections was to study the failure mechanism of extended life CRC pavements. Sections 1 through 5 (lane 1) were loaded with accelerated traffic and the results are presented in this research. Sections 6 to 10 (lane 2) were built to compare the effect of induced cracks and were not subjected to trafficking. The results and benefits of crack induction in these sections are reported in Kohler and Roesler (2004) and in Chapter 7. All sections are approximately 85 feet long, arranged in two 500-foot lanes, with transition zones to accommodate change in steel content between sections, and have end restraints constructed to prevent excessive slab movement and to anchor the longitudinal steel. The layout and main design characteristics of the section are presented in Figure 3.1. The concrete surface layer was placed on top of 4 inches of BAM, 6 inches of aggregate subbase (ASB), and a compacted subgrade separated from the ASB by a nonwoven geotextile.

All the transverse cracks in lane 1 pavement sections developed naturally (there were no induced cracks in lane 1). The thickness of the concrete is 10 inches in sections 1, 2, and 3, and 14 inches in sections 4 and 5. The reinforcement of the slabs consists of a total of 26 longitudinal bars, spaced 5.5 inches apart. The bar size in sections 1 and 2 is #5 and #6, respectively. Sections 3 to 5 have #7 bars. The depth of the longitudinal reinforcement is 3.5 inches in sections 1, 2, and 3; in section 4 the depth is 4.5 inches; and in section 5 the reinforcement is split in two layers, at 3.5
and 7 inches. Transverse steel reinforcement (#4 bars) supported the longitudinal bars every 4 feet. Figure 3.2 shows the cross-sectional view of the pavement sections revealing the depth and horizontal spacing of the rebars.

Figure 3.1. Layout and basic design parameters of test sections

Figure 3.2. Cross sectional view of concrete and steel reinforcement

Details about design and particularly about construction of these experimental test sections, as well as specific information about the instrumentation, are available from Kohler et al. (2002).
3.3. **Construction material properties**

Concrete beams and cylinders were made at the time of concrete casting to determine the hardened concrete properties. Three concrete prisms were also sampled to determine the concrete coefficient of thermal expansion. In addition, Falling Weight Deflectometer (FWD) testing was performed one month after construction to estimate the in-situ pavement properties. The concrete mix design used for all sections is detailed in Table 3.1. Concrete test results are presented in Table 3.2.

<table>
<thead>
<tr>
<th>Table 3.1. Concrete mix design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (lb./cy)</td>
</tr>
<tr>
<td>Fly Ash (lb./cy)</td>
</tr>
<tr>
<td>Coarse Aggregate (lb./cy)</td>
</tr>
<tr>
<td>Fine Aggregate (lb./cy)</td>
</tr>
<tr>
<td>Voids</td>
</tr>
<tr>
<td>Design w/c Ratio</td>
</tr>
<tr>
<td>Air Entraining Admixture (oz./cwt)</td>
</tr>
<tr>
<td>Water Reducer (oz./cwt)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3.2. Average hardened concrete properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd Point Flexural strength 28-d (psi)</td>
</tr>
<tr>
<td>Elastic modulus from cylinders 28-d (psi)</td>
</tr>
<tr>
<td>Elastic modulus from FWD (psi)</td>
</tr>
<tr>
<td>Compressive strength 28-d (psi)</td>
</tr>
<tr>
<td>Coeff. of thermal expansion (°F)</td>
</tr>
</tbody>
</table>

The average modulus of the bituminous aggregate mixture (BAM) layer was backcalculated to be 250 ksi at a pavement temperature of 80°F. The effective k-value (of the subgrade and subbase) was determined with FWD to be approximately 200 psi/inch. Yield and ultimate strength of the reinforcement steel bars passed the minimum requirements of 60,000 and 90,000 psi respectively, verified by IDOT’s laboratory in Springfield.
3.4. Instrumentation

A variety of sensors were installed in the test sections in order to monitor the environmental and repeated load performance of the CRC pavement and to identify differences in pavement responses between test sections.

3.4.1 Crack movement

The sensor arrangement used to capture the crack movements at the edge of the pavement is shown in Figure 3.3. Horizontal movements were measured, via LVDTs, relative to each side of the crack, at different depths in the slab. Vertical movements were measured with two LVDTs suspended from a reference beam supported away from the loaded pavement edge. Vertical and horizontal sensors were placed concurrently at each monitored crack, as shown in Figure 3.4.

Figure 3.3. LVDT arrangement to measure crack movements at the edge
Individual sensors are also used to monitor daily and seasonal changes in crack width on the pavement surface. One LVDT is installed at one crack per section, as shown in Figure 3.5. The environmental-only sensors are part of the static instrumentation system, which continuously collects environmental responses from every section. Conversely, the instrumentation set-up presented in Figure 3.3, which belongs to the dynamic system, was moved along with the accelerated pavement testing device.

### 3.4.2 Strain gages

Strain gages were installed at time of pavement construction. Four sensors per section were mounted and zip-tied to thin steel chairs to hold them into the correct lateral position and at one inch under the CRCP surface, as shown in Figures 3.6. The type of gage used is specially designed to measure strain in concrete under a dynamic loading. The gages are sealed between thin resin plates and are 5 inches long by 0.5 inch wide by 0.2 inch thick. The gage length is 2.36 inches. The manufacturer is Tokyo Sokki Kenkyujo CO. They were transversely oriented and at a distance from the loaded edge where the maximum stress was expected under a rolling wheel. This distance was determined to be approximately 54.5 inches using ILLISLAB.
3.4.3 Data logging

The static and dynamic systems are independently controlled, each having their own data-logger. The static system operated on all the sections since the start of concrete placement in early December 2001. Two datalogger units were used for the collection of static instrumentation data, one for each lane, plus a third unit dedicated to an on-site weather station. The datalogger model was CR10X manufactured by Campbell Scientific Inc. Each of the units connected to the
pavement sensors were intended to record data related to the early age responses due to concrete material hydration and shrinkage. Thermocouples installed at various depths in each section provided valuable in-situ temperature profiles. The weather station allowed recording of air temperature, air relative humidity, wind speed, wind direction, and solar radiation.

The dynamic instrumentation system consisted of signal conditioning equipment from National Instrument, known as SCXI (Signal Conditioning eXtensions for Instrumentation). This unit was connected to a personal computer located in the trailer from where the ATLAS system was operated. SCXI is a modular platform for signal conditioning, and consists of multi-channel modules installed in a chassis. Three different types of modules were used: for strain gages, thermocouples, and LVDTs. This system was controlled with a program written with the software LabView® and is also connected to the ATLAS to record position of the wheel load. Wheel position was employed to synchronize the acquisition of the dynamic data.

3.5. Advanced Transportation Loading Assembly, ATLAS

The Advanced Transportation Loading Assembly (ATLAS) is the machine used to test full-scale pavement performance using accelerated, damaging loads. The characteristics of the ATLAS pertinent to the research program presented herein are listed in Table 3.3. A picture with a full view of the ATLAS is shown in Figure 3.7.

<table>
<thead>
<tr>
<th>Overall dimensions</th>
<th>124 x 12 x 12 feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load capacity</td>
<td>80,000 lbs</td>
</tr>
<tr>
<td>Tire load rating</td>
<td>Dual tires: 12,000 lbs</td>
</tr>
<tr>
<td></td>
<td>Aircraft tire: 55,700 lbs</td>
</tr>
<tr>
<td>Traffic length</td>
<td>85 feet max</td>
</tr>
<tr>
<td>Wheel traffic speed</td>
<td>10mph max</td>
</tr>
<tr>
<td>Loading conditions</td>
<td>Uni- or Bi-directional</td>
</tr>
<tr>
<td></td>
<td>Adjustable lateral position, fixed or distributed</td>
</tr>
</tbody>
</table>
All the load test operation is controlled with a personal computer from inside a trailer positioned next to the machine. The testing was carried out by application of a few thousand passes at a time, followed by inspection of the pavement, and then another round of loading. The sections were built so that the entire traffic length of the machine could be used, which is 85 feet. The load level used was fixed for the entire length of the section, and the loads used ranged from 5,000 to 55,000 pounds. The loading was primarily applied with a single aircraft tire with an inflation pressure of 210 psi. The tire had to be replaced once to complete testing in all five sections. Most of the testing was done at a wheel speed between 6 and 8 mph, mostly in bidirectional mode, and with fixed lateral position. The wheel load was applied at one or two inches from the edge of the pavement in order to produce the maximum feasible deflection and stresses in the concrete, and therefore accelerating pavement failure.

The machine was moved from one section to the next using its own crawling tracks. Most of the time the ATLAS was run inside a movable shelter made of synthetic fabric on an aluminum skeleton frame. The shelter protected the machine and the pavement section from direct sunlight and snow, but did not provide controlled environmental conditions. The doors were kept open when the weather was mild, to keep air temperature and moisture similar to the outdoors conditions. Figure 3.8 shows the ATLAS inside the shelter.
3.6. Testing sequence and procedure

The sequence of pavement testing with the ATLAS started in August 2002 and ended in August 2004. Section 1 and 2 were tested to failure. Then sections 4 and 5 were tested although pavement failure was not achieved. Finally section 3 was tested, resulting in the same failure pattern observed in sections 1 and 2. Extra time was initially spent in section 1 to refine details and procedures of crack instrumentation and to setup data collection synchronized with the passage of the loading wheel. Sections 4 and 5 were tested before section 3 because it was preferred to load these thicker sections in wintertime when the cracks are wider. Sections 1 through 5 were tested under different weather conditions, which need to be accounted for in the analysis. Table 3.4 presents a brief summary of testing in sections 1 to 5.

Table 3.4. Dates of testing, total load repetitions and pavement temperature range on experimental CRCP sections

<table>
<thead>
<tr>
<th>Testing info</th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 4</th>
<th>Section 5</th>
<th>Section 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total load repetitions</td>
<td>246,800</td>
<td>118,600</td>
<td>163,400</td>
<td>64,300</td>
<td>1,800</td>
</tr>
<tr>
<td>Pavement temperature range</td>
<td>34-80 °F (²)</td>
<td>75-95°F</td>
<td>25-50°F</td>
<td>40-65°F</td>
<td>64-80°F</td>
</tr>
</tbody>
</table>

¹ Includes time when no load was being applied.
² Most of the effective test was done during June 2003, when temperature was 60-80 °F
The testing procedure followed in each section consisted of an initial collection of responses to temperature loads only, followed by a period of loading at low load levels to obtain elastic responses and then heavy loading to accelerate damage.

3.7. Instrumented cracks

The study of crack movement and crack width presented in the following chapters is based on measurements taken from individual cracks in each section. Although more rigorous conclusions would be obtained if all transverse cracks had been instrumented, practical limitations such as the number of sensors and channels made it preferable to instrument about four cracks per section. The cracks that were instrumented were selected based on two criteria:

1- Most clearly visible cracks in the depth and width of the slab.
2- Cracks located along the entire section not immediately adjacent to another instrumented crack.

The technique to measure horizontal crack movement at various depths was developed during testing of the first two sections, and for that reason a more consistent instrumentation plan was implemented for sections 3 to 5. The number of datalogger channels was increased from 16 to 24 on sections 3 to 5. The original 16-channel capacity allowed collecting from 8 vertical and 8 horizontal LVDTs, while the new capacity allowed for 8 vertical and 16 horizontal sensors. Location of the sensors along the 85-ft long sections and the instrumented depths are presented in Table 3.5. Sensors at top (“t”) and bottom (“b”) were placed at about one inch from the indicated surface. Sensors at mid-top (“mt”) and mid-bottom (“mb”) were placed at third-points in the thickness.
Table 3.5. Instrumented cracks and depth of sensors.

<table>
<thead>
<tr>
<th>Section</th>
<th>Station (ft)</th>
<th>Depth (*)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19.8</td>
<td>t</td>
</tr>
<tr>
<td></td>
<td>24.4</td>
<td>t, m</td>
</tr>
<tr>
<td></td>
<td>51.4</td>
<td>t, b</td>
</tr>
<tr>
<td></td>
<td>61.9</td>
<td>t, m, b</td>
</tr>
<tr>
<td></td>
<td>72.2</td>
<td>t</td>
</tr>
<tr>
<td></td>
<td>78.1</td>
<td>b</td>
</tr>
<tr>
<td>2</td>
<td>21.6</td>
<td>t, m, b</td>
</tr>
<tr>
<td></td>
<td>29.0</td>
<td>t</td>
</tr>
<tr>
<td></td>
<td>34.0</td>
<td>t</td>
</tr>
<tr>
<td></td>
<td>48.2</td>
<td>t</td>
</tr>
<tr>
<td></td>
<td>57.1</td>
<td>t, b</td>
</tr>
<tr>
<td>3</td>
<td>14.9</td>
<td>t, mt, mb, b</td>
</tr>
<tr>
<td></td>
<td>23.4</td>
<td>t, mt, mb, b</td>
</tr>
<tr>
<td></td>
<td>43.7</td>
<td>t, mt, mb, b</td>
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(*) Note. t=top, m=mid-depth, b=bottom, mt=mid-top, mb=mid-bottom
CHAPTER 4 DETERMINATION OF CRACK WIDTH

4.1. Introduction

This chapter describes how the crack closing movements were utilized to determine crack width (CW) at the transverse cracks on the CRCP experimental sections. The influence of pavement temperature and of moving wheel loads on CW is first introduced, to explain the process of crack width determination.

4.2. Pavement responses to temperature and to moving wheel loads

The most influential factor in CW is pavement temperature and therefore a thorough understanding of the daily and seasonal Tavg and Tdiff are needed for the test section location. Tavg is the internal pavement temperature obtained as the average of top and bottom of the slab. Tdiff is the temperature difference through the slab, calculated as the temperature at top minus temperature at the bottom.

4.2.1 Pavement temperatures

Air temperature, as well as pavement temperature, oscillates in daily and annual cycles. For the site of testing in Rantoul, Illinois, the highest pavement temperature (Tavg) is about 100°F during summertime while the lowest is close to 20°F in wintertime. Figure 4.1 presents pavement temperature data collected every 30 minutes, obtained from measurements at one and nine inches below the surface in 10-inch slabs, for the two-year period of pavement testing in this research.

During summer days the difference between the low and high Tavg can exceed 20°F (early morning to afternoon on the same day), while in winter that difference is usually less than 10°F. Temperature at the bottom of the slab changes at a lower rate compared to the changes near the surface, which leads to temperature differentials. Figure 4.2 shows the temperature differential Tdiff measured every 30 minutes, indicating a wider range in summer than in winter.
Figure 4.1. Average pavement temperature trend, two years data

Figure 4.2. Hourly pavement temperature differential two-year data

Pavement temperature profiles for morning and afternoon, in winter and summertime are presented in Figure 4.3.
The temperature at the top of the pavement was taken at one inch below the surface because at this depth it is less affected by sudden changes caused by wind, rain, and cloudiness (solar radiation). Other studies involving temperature in concrete pavements have also used top temperature measured at one inch or more from the top of the pavement instead the actual slab surface (Choubane and Tia, 1982, Armaghani et al. 1987, Nishizawa et al. 1998).

Plots of Tdiff versus Tavg reveal a hysteretic behavior. Figure 4.4 shows this phenomenon for typical days in winter and summer. Rapidly increasing temperature at the top leads to increase in Tavg during the day, especially during the morning hours. When top temperature begins to decrease, Tavg remains high before it starts to decrease. During night hours Tavg is decreasing, and Tdiff is low and changes slowly, until a new day bring warmer temperature to the top, and with it another cycle starts.
4.2.2 Crack width versus temperature

Transverse cracks open and close with the daily and seasonal temperature cycles. Figure 4.5 is an example that presents the variation in Tavg and Tdiff, along with daily crack movement at various depths. The upper plot shows how Tavg increased during the daylight hours and decreased during night. The lower plot shows that the crack closes as the temperature increases, and then it opens as the temperature goes down (note some delay of Tavg with respect to the changes in Tdiff).

The data in Figure 4.5 reveals that the largest range of movement occurred at the sensor located near the surface, and the least movement was observed at sensors near the slab bottom. The daily movement at various depths using the data presented in Figure 4.5 is now shown in Figure 4.6. The term “daily movement” is defined here as the short-term horizontal movement of cracks caused by temperature changes within periods of 24 hours. It is determined by taking the maximum difference in crack width from each sensor (from minimum to maximum opening). The daily movement obviously depends on the temperature variation on the sampled days.
Figure 4.5. Temperature and horizontal crack movement at various slab depths.

Figure 4.6. Daily movement at various depths in a single crack.

Figure 4.7 presents daily movement averaged from the four cracks in section 3 and the four cracks in section 4. These measurements were taken in May and March of 2004, when average pavement temperatures were about 69°F and 41°F for sections 3 and 4, respectively. From the graphs in Figure 4.7, it is clear that there is greater crack movement as the measurement is taken
closer to the surface of the slab. The total change in Tavg in this 24-hour period in section 3 was 5.4°F and in section 4 was 6.5°F. Total change in Tdiff was 6.0°F and 6.8°F. The percent of steel in section 3 is 1.09 and in section 4 is 0.78.

![Diagram](image)

Figure 4.7. Daily movement at various depths, a) Average of all cracks in section 3; b) Average of all cracks in section 4.

### 4.2.3 Wheel loads

The wheel loads applied with the ATLAS machine traversed the section repeatedly. Each load application caused deformations in the slab as the wheel moved from one end of the section to the other. The loads applied to the pavement were at the edge of the slab, and they were in general higher than normal highway traffic loads. As a reference, heavy trucks operated on highways have axle loads that are around 18 kips (1 kip = 1,000 pounds) if it is a single axle, or about 34-kips if a tandem. Typical truck tires are rated to carry about 12 kips when in dual configuration, which means a single axle with dual tires could take up to 24 kips. The load used to test pavement elastic response was 10 kips applied on a single wheel. High load levels from 30 to 55 kips were used for the accelerated load testing. Normal and high loads were all applied at the edge, because that is the critical condition for stresses and deflections in CRCP. The outside wall of the tire traveled the section at one or two inches from the actual edge of the pavement. Loading at the edge had the purpose of accelerating fatigue damage in the concrete by increasing the tensile bending stresses.
Loading with the ATLAS offers a much better simulation of traffic-induced damage compared to fixed loads (such as slab tests in laboratory settings), while at the same time provides a rapid test for researchers compared with tests on in-service highways, which can take the entire design life to determine the performance. The use of the ATLAS permitted accurate control of the number and load level of wheel pass applications.

![Figure 4.8. Wheel load at the pavement edge](image)

### 4.2.4 Crack width versus wheel loads

Vertical deformations and horizontal crack movements were recorded at each instrumented crack for every inch movement of the wheel along the section. Figure 4.9 shows an example of measured vertical deformations during a single wheel pass. Because of variability in material, support conditions, and crack geometry, the vertical deflection was not the same at all instrumented cracks. The continuous recording of pavement response generates the lines of influence.
When concrete slabs bend under mechanical loading near a crack, the faces of the crack rotate, with the horizontal distance between them varying with depth. When the wheel is located over a crack, the load causes the upper part of the crack to reduce or close. When the wheel load is at a distance, approaching or leaving the crack, an opening movement at the surface is produced. In summary, as the wheel moves through the section, the upper part of each crack is subjected to an open-close-open sequence, as shown in Figure 4.10. The activity at the bottom part of the cracks is the opposite, experiencing a sequence of close-open-close. In both cases, the measured crack’s response hits the highest point when the wheel is directly over the crack. Figure 4.11 presents an example of horizontal crack movement at top and bottom of the slab.
The amount of crack closing caused by the wheel load at the top of the slab varies with the load level being applied and the pavement temperature profile. Two types of loading experiments were designed in order to measure and verify crack width for the several test sections. The first
type of load test consisted of keeping a constant load level throughout the test as changes in the slab temperature profile resulted in changes in the crack width. In the second loading type, various load levels were applied during a period of a few minutes in order to limit the effects of temperature profile changes. These experiments are called respectively the temperature and load spectra tests.

4.3. Temperature spectra tests

Crack widths along the depth of the slab are affected by changes in the pavement temperature profile. If crack width magnitude oscillates on a daily basis, so does the extent of the horizontal movements that can be caused by the vertical load. A temperature spectra test is defined here as the study of horizontal crack movements as they vary with the pavement temperature profile under a fixed, repeated load level.

Figure 4.12 presents typical opening and closing movements caused by a 35-kip rolling wheel load as measured with the top sensor while Tavg went from 27 to 54°F. The data corresponds to section 5 and it shows that for Tavg below 30°F, the temperature did not significantly affect the closing movement. This meant the crack was sufficiently wide such that the load level applied did not bring the adjacent crack faces together. For temperatures greater than 50°F the measured closing reached a minimum and the crack behaved as if it was fully closed, with the only movement being the elastic compression of the concrete under the 35 kips wheel load. Between 30 and 50°F the magnitude of crack closing decreased as the pavement temperature increased. This behavior of crack closing movement at the top of the crack is used to calculate the true value of crack width. The opening at the top, which is the result of the load being applied at a distance from the crack, remained constant for all ranges of temperature.
Figure 4.12. Effect of pavement temperature on horizontal crack movement at top of the slab

Typical horizontal crack movement at the bottom of the slab is presented in Figure 4.13. The opening movement, which when measured at the bottom corresponds to the response at the instant the wheel load is directly on top of the crack, increased with temperature due to the axis of rotation of the crack faces shifting upward. The closing movement was relatively constant.

Figure 4.13. Effect of pavement temperature on horizontal crack movement at bottom of slab
Only the closing movement measured when the wheel is on top of the crack (as in Figure 4.12) is used to try to determine crack width, because the crack faces need to be forced into contact. Opening movement by itself does not provide information on the absolute crack width. Furthermore, in order to collect good data from a temperature spectra test, it is necessary for the pavement to be exposed to a range of temperatures that is wide enough to identify the region in which crack closing is dominated by temperature.

Crack closing and crack width magnitude are two different parameters and are only equal under certain conditions. Crack closing is how much the crack closes by the action of the load, while crack width is the actual distance between crack faces (at a specified depth and temperature state). Crack closing equals crack width when the applied load forces the crack faces to come into intimate contact. The amount of crack closing (at the depth of the sensor) caused by a given load can be categorized into one of three stages:

- **Wide crack closing**: The crack is wide enough to remain open under loading, i.e., the crack does not close, only the opening is reduced.
- **Restricted crack closing**: The crack begins slightly open but during loading the crack faces come into contact with each other.
- **Closed crack compression**: The crack is fully closed before loading and only the elastic compression of the concrete is measured as the load passes over the crack.

Figure 4.14 is a schematic representation of the vertical and horizontal deformation at the crack and shows the three stages of crack closing.
As shown previously, the maximum rotation of the crack faces occurs at the same time the pavement experiences the largest vertical deflection. The magnitude of horizontal crack movement primarily depends on the load level and the temperature conditions at the time of the measurement.

The interpretation of certain temperature, $T_i$, on crack closing according to the temperature spectra model can be seen in Figure 4.15. If $T_i$ is lower than $T_1$, then crack closing does not depend on temperature and the crack closes to the full extent permitted by the load (see region A in Figure 4.15). Furthermore, the two adjacent slabs do not come into contact with each other and the crack width magnitude cannot be determined. If $T_i$ is between $T_1$ and $T_2$ then the initial crack opening becomes fully closed under loading (see region B). The crack width magnitude has to be calculated by subtracting the concrete compression component from the measured crack closing, depicted as the vertical line shown in Figure 4.15, region B. Finally, if $T_i$ is higher than $T_2$, then the crack has been closed by the temperature deformations prior to loading, and the only movement that is measured is the elastic compression of concrete (region C in Figure 4.15). The crack closing model shown in the expressions in Figure 4.15 applies both to $T_{avg}$ and $T_{diff}$.
Temperature spectra tests were completed on pavement sections 1 and 2 in 2002. Load testing in section 1 was carried out under a wide range of pavement temperatures (see table 3.4, chapter 3), which allowed capturing responses at low and high temperatures. Temperatures at time of testing in section 2, however, were high, as the testing was conducted exclusively during summertime. To construct the plots shown in Figure 4.16 it was necessary to use data collected between the months of December and June. The plots show the opening and closing (positive and negative values respectively) caused by a 35-kips load and measured near the top of the slab. The horizontal axis in the plot on the left is the average temperature of the pavement (Tavg), while the plot on the right involves the temperature difference (Tdiff).
Individual curves versus Tavg follow the crack closing trend indicated in the model. In the curves versus Tdiff, the predominant effect of Tavg in crack closing makes it hard to distinguish any trend. In general the relationship between Tavg and Tdiff and their combined effect on crack width makes graphical interpretation more difficult. The closing at the three crack locations were different, which meant they did not have the same width. Differences between crack closing at the top and at mid-depth were observed in the cracks located at 24.4 and 61.9, where sensors at mid-depth revealed smaller crack width than at the top (the other cracks were only instrumented at top).

Approximate crack widths in section 1 are presented in Table 4.1 based on the temperature spectra method. The temperature Tavg at which the crack closing curve begins to be controlled by temperature was found for each crack (T1 from model in Figure 4.15) so that crack width can be obtained directly from crack closing at that temperature. As mentioned, crack width in section 2 could not be obtained because the temperatures during the time of testing were high and the horizontal movement observed falls in the category of closed crack compression.
There is a certain combination of temperatures (Tavg and Tdiff) and load levels that can close the crack and allow for the determination of the crack width magnitude. However, the process of obtaining the data is difficult because of the need of temperature range, and because the analysis is complex given the combined effects of Tavg and Tdiff. This was the main reason for the development of the load spectra test presented in the next section. For the testing of the sections 3 to 5 it was decided that the application of a range of load levels during a short period of time, instead of application of constant load over a range of temperatures, would provide a more accurate and rapid approach to determine crack width.

### 4.4. Load spectra tests

In a load spectra test a wide range of load levels are applied to the pavement, and greater horizontal movements are obtained as higher loads are exerted. Typical influence lines for horizontal movement at two load levels are presented in Figure 4.17, in which, closing at the top of the slab occurs directly under the load, while closing at the bottom happens when the load is at a distance from the crack.

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<th>CW (mm)</th>
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<td>42.0</td>
<td>1.1</td>
<td>0.098</td>
</tr>
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</table>
Figure 4.17. Horizontal crack activity at top and bottom of the slab under rolling wheel load, two load levels

The maximum opening and closing values are plotted versus load level for the horizontal sensors located at the various depths: 1 inch from the surface (Top), H/3 from the surface (Mid-top), 2H/3 from the surface (Mid-bottom), and 1 inch from the bottom (Bottom). A typical resulting plot is presented in Figure 4.18.
Crack opening, represented by positive horizontal movement, is approximately linear with the load level for all sensor locations. Crack closing is also initially linear with load, but it changes in slope after a certain load, hence it can be approximated by a bilinear curve. This bilinear behavior is highlighted in the plots and represents the essence of the crack width determination procedure. The change in slope in the curve of crack closing versus load occurs at a load level that fully closes the crack without causing compression on the crack faces. It is the point at which the crack changes from a state of no contact at the depth of the sensor to a state of compression. Crack width is defined as the crack closing movement at this closure load. Loads greater than the closure load produce compressive deformations in the concrete that do not allow the actual crack closing to be determined unless the exact compressive deformation is known. The process used to calculate crack width from the plot of crack closing is shown in Figure 4.19 and consisted of intersecting best-fit lines traced over the first and last portion of the curve to obtain the closing load. At the closing load, the measured closing crack movement is considered equal to the crack width at that temperature profile and slab depth.
Figure 4.19. Crack width from crack closing measurements in load spectra tests

The results of load spectra tests (LST) clearly show the amount of free closing at the instrumented depth, and the point of the change in slope of the crack closing is the crack width. The magnitude of crack width obtained with LST represents the crack opening as it impacts the mechanical response to rolling wheel loads. The crack measurements with the LST are more accurate than visual methods and also enable crack width determination with depth.

This loading procedure and crack width calculation method was performed on each instrumented crack in test sections 4, 5 and 3 (the order in which they were tested). Four cracks were instrumented in sections 3 and 4, but there was only one crack in section 5 to instrument. Four sensors were installed at all cracks. In order to facilitate the execution of LST, the software that controls the ATLAS was modified to allow for continuous wheel passes at automatically increasing load levels. These load levels ranged from 6 to 51 kips for sections 4 and 5, and ranged from 6 to 36 kips in section 3. The load increment was always 3 kips. The reason for the difference in maximum load is that section 3 had a 10-inch thick slab while sections 4 and 5 had

66
14-inch slabs. To reduce variability in the results and improve the fitting of the bilinear curve, the closing movement at each load level was taken as the average of 3 to 10 consecutive passes at each load level. Using the ATLAS, three passes could be completed in 60 seconds (with the wheel traveling in both directions at 6 mph), and consequently a complete test with 16 different load levels could be finished in about 16 minutes. Given the short duration of the test, the temperature conditions at the end of the test were always practically the same as the beginning.

Several LST were performed on each of the three sections in order to determine the crack width at various temperature conditions. The results of crack width from top to bottom at each crack and under various temperature conditions are shown for sections 3, 4 and 5 in Table 4.2, Table 4.3, and Table 4.4, respectively. Note that sensors at top and bottom were placed at 1 inch below the surface and 1 inch above the base.

Table 4.2. Crack width (microns) and pavement temperature (°F) for load spectra tests on section 3

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Table 4.3. Crack width (microns) and pavement temperature (°F) for load spectra tests on section 4

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67
Table 4.4. Crack width (microns) and pavement temperature (°F) for load spectra tests on section 5

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<td>25.7</td>
<td>7.3</td>
<td>4.0</td>
<td>4.3</td>
</tr>
</tbody>
</table>

4.5. Analysis of measured crack width

4.5.1 Variability in the results

Reliable crack width results were obtained in sections 3, 4, and 5 using the load spectra test and the arrangement of four sensors with slab depth. Approximate crack widths were obtained in section 1 using the temperature spectra data. Crack width in section 2 could not be obtained because no load spectra test was performed and the temperatures during the time of testing were always high. As it was mentioned before, several load spectra test allowed collecting crack width under different temperature conditions, i.e., combination of Tavg and Tdiff. Variation of crack width with temperature, along with the crack width profile, is presented in Figure 4.20, which shows one representative crack from each section.
The following observations can be made at this point based on the results presented in Tables 4.2 to 4.4 and in Figure 4.20:

• There is great variability in crack width at the various transverse cracks within the 85 feet long section. In fact, the widest crack in section 3 is about 80 percent greater that the narrowest one. In section 4, this value is 140 percent. Only the most visible cracks of each section were considered for the measurement, so an even higher variability can be expected if each transverse crack width is assessed.

• Crack widths at the mid-bottom and bottom locations were much smaller than the crack width near the top of the slab, as expected. The crack closing movements measured with the sensors located in the lower part of the slab occurred when the load was away from the crack. The load spectra tests indicated again that the crack width profile was decreasing with depth.

• $T_{avg}$ and $T_{diff}$ significantly affected the measured crack width at a single transverse crack location. As expected, CW decreased for higher $T_{avg}$ and $T_{diff}$.

• The changes in temperature affect crack width mostly in the upper portion of the slab.
4.5.2 Crack width at depth of steel

The crack width at the steel depth needs to be evaluated against existing models that predict crack width. Given that measurements were taken at various depths in the slabs, crack width at the depth of the steel can be calculated by interpolating crack width measured above and below the known steel depth. Crack width at the depth of the steel in section 1 was obtained from measurements in the two cracks instrumented at top and mid-depth, and it was estimated in the other two cracks based on width ratio from the instrumented cracks. No crack width data was obtained in section 2. For cracks in sections 3 to 5 the results were averaged from the various temperature conditions. Table 4.5 presents crack width at the depth of the steel for all evaluated sections.

<table>
<thead>
<tr>
<th>Section</th>
<th>Crack ID</th>
<th>Tavg (°F)</th>
<th>Tdiff (°F)</th>
<th>CW above steel</th>
<th>CW below steel</th>
<th>CW at steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cr.24.4</td>
<td>43.3</td>
<td>0.5</td>
<td>0.089</td>
<td>0.049</td>
<td>0.058</td>
</tr>
<tr>
<td></td>
<td>Cr.51.4</td>
<td>39.8</td>
<td>-4.1</td>
<td>0.083</td>
<td>-</td>
<td>0.054</td>
</tr>
<tr>
<td></td>
<td>Cr.61.9</td>
<td>39.8</td>
<td>-4.1</td>
<td>0.118</td>
<td>0.65</td>
<td>0.077</td>
</tr>
<tr>
<td></td>
<td>Cr.72.2</td>
<td>42.0</td>
<td>1.1</td>
<td>0.098</td>
<td>-</td>
<td>0.063</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>41.2</td>
<td>-1.7</td>
<td>0.097</td>
<td>0.057</td>
<td>0.063</td>
</tr>
<tr>
<td>3</td>
<td>Cr.14.9</td>
<td>65.6 to 78.0</td>
<td>0.9 to 7.6</td>
<td>0.030</td>
<td>0.005</td>
<td>0.011</td>
</tr>
<tr>
<td></td>
<td>Cr.23.4</td>
<td>65.6 to 78.0</td>
<td>-5.2 to 7.6</td>
<td>0.026</td>
<td>0.003</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td>Cr.47.3</td>
<td>78.0</td>
<td>7.6</td>
<td>0.025</td>
<td>0.005</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td>Cr.57.2</td>
<td>78.0</td>
<td>7.6</td>
<td>0.025</td>
<td>0.006</td>
<td>0.014</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>71.8</td>
<td>1.9</td>
<td>0.027</td>
<td>0.005</td>
<td>0.011</td>
</tr>
<tr>
<td>4</td>
<td>Cr.24.1</td>
<td>28.0 to 43.6</td>
<td>0.6 to 5.8</td>
<td>0.046</td>
<td>0.034</td>
<td>0.036</td>
</tr>
<tr>
<td></td>
<td>Cr.28.8</td>
<td>28.0 to 43.6</td>
<td>0.6 to 5.8</td>
<td>0.026</td>
<td>0.003</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td>Cr.44.0</td>
<td>43.6</td>
<td>5.8</td>
<td>0.059</td>
<td>0.032</td>
<td>0.032</td>
</tr>
<tr>
<td></td>
<td>Cr.63.1</td>
<td>43.6</td>
<td>5.8</td>
<td>0.035</td>
<td>0.021</td>
<td>0.021</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>35.7</td>
<td>2.2</td>
<td>0.050</td>
<td>0.031</td>
<td>0.033</td>
</tr>
<tr>
<td>5</td>
<td>Cr.14.5</td>
<td>50.5 to 63.2</td>
<td>0.9 to 7.5</td>
<td>0.023</td>
<td>0.007</td>
<td>0.012</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>54.5</td>
<td>3.4</td>
<td>0.023</td>
<td>0.007</td>
<td>0.012</td>
</tr>
</tbody>
</table>

Since the temperature conditions prevalent at time of testing in these sections were different, conclusions cannot be established about crack width from one section to another. The issue of a standard temperature state to compare crack width between sections is addressed in Chapter 5, along with discussion of crack width profiles and prediction models.
The measured crack widths were smaller than reported in the literature mainly due to the season of construction (low temperatures) and the relatively young age of the sections. The implications of small crack width in pavement performance and transverse cracking characteristics are discussed in Chapter 6.

4.6. Summary of Chapter 4

This chapter explained the use of the crack closing to determine crack width. It was shown that both temperature parameters, Tavg and Tdiff, vary considerably during daily and seasonal cycles. Rolling wheel loads trigger a sequence of opening and closing at the cracks that depends on the initial crack width and is particular to the depth at which the movement is measured. When the wheel is located over a crack, the load causes the upper part of the crack to reduce or close. When the wheel load is at a distance, approaching or leaving the crack, an opening movement at the surface is produced.

The temperature and load spectra tests were developed to calculate the crack width based on crack closing movements. A load spectra test consists of detecting the amount of crack closing needed to actually shut the crack and it does this by using several load levels under constant temperature conditions. A temperature spectra test comprises a constant load level that is applied to the section and the crack closing varies with the change in pavement temperature. It was demonstrated that the load spectra test provided the most feasible method to measure the true magnitude of the crack width. The results obtained with this new method are associated with the mechanical response of the crack to rolling wheel loads and therefore are less prone to the subjectivity of visual methods.
CHAPTER 5  ANALYSIS OF CRACK WIDTH RESULTS

5.1.  Introduction

Continuously reinforced concrete pavements have numerous transverse cracks that occur naturally at variable spacing. These cracks do not affect the performance of the structure unless they widen and result in a loss of load transfer efficiency. The capacity to transfer vertical loads across a crack depends on factors such as the thickness of the slab, the roughness of the crack’s face, the dowel action of the reinforcement steel, and the crack width.

This chapter presents the analysis of the crack width measurements that were presented in Chapter 4 with respect to the crack width profile (with slab depth), temperature standardization, short-term crack width prediction, and zero-stress temperature.

5.2.  Crack width profile

Crack width is not necessarily uniform along the depth of the slab and that affects the aggregate interlock and the shear capacity of the cracks. Crack width profile along the thickness of the slab is discussed in this section through visual observation of the profile and from the results of the crack instrumentation under different temperature and loading conditions.

5.2.1  Visual observations

Crack width profiles were observed at the slab’s edge in each section and on pavement cores. Transverse cracks at the edge were found to be generally smaller at increasing distance from the surface, as shown in Figure 5.1, in which the crack turns invisible to the eye near the bottom of the slab. This is consistent with the idea of crack formation occurring at the surface because of the greater drying shrinkage and temperature contraction. A pattern in which the cracks “look” wider toward the surface was observed in most cracks and on all pavement sections.
Four-inch diameter cores were extracted from the pavement sections at the location of several transverse cracks. The cracks in the cores were clearly visible near the top and the bottom of the pavement and sometimes disappeared near the steel depth. This represents a pattern different to what was observed on the edge of the slab, where cracks were smaller at the bottom than anywhere else. Figure 5.2 shows a core from section 1 with a detailed view of crack width at the bottom.
Although the cracks were visible from top, bottom and on the sides of most of the cores, they were at every point smaller than 0.2mm, which is the smallest increment of the crack gage used to quantify crack width. In fact, cracks were so tight in all cores that the halves could not be separated manually (only attempted on cores with no reinforcement). Figure 5.3 shows that at the holes left after core extraction it was evident that the crack width became smaller with depth from the surface.
Some of the pavement cores extracted at transverse cracks were obtained at the location of a reinforcement bar. Those at the rebars were sawn parallel to the reinforcement in order to expose crack width at the vertical plane of the steel. It was verified that the cracks were not visibly detectable near the steel bar, although they were visible around the perimeter of the core.

These observations suggest that the profile of transverse cracks with pavement depth has a “V” shape at locations away and in-between reinforcement bars, but it has its narrowest point at the steel depth in the immediacy of the longitudinal steel. This is illustrated in Figure 5.4.
Another observation of transverse cracks is that they propagated vertically downward through the slab thickness, even though they meander across the slab. The crack did not offset more than two inches between the top and bottom surface.

5.2.2 Profile measurements

The crack width profile is not fixed but varies according to the pavement temperature conditions. Figure 5.5 presents measured CW profiles during a 24-hour period at the four instrumented transverse cracks in section 3. The data is presented every hour. The cracks are wider near the surface, and in some cases they widen again toward the bottom of the slab, particularly in crack 23.4. No extrapolation has been made for crack widths at the surface or bottom of the 10-in slab. These profiles were all measured at the edge of the slab.
Figure 5.5. Crack width profiles during 24-hour in section 3

Figure 5.6 shows crack width and temperature information related to the data presented in Figure 5.5. Average crack width calculated every hour from the four sensors in each crack is presented in part (a) of the figure. The cracks are open during the morning hours but become fully closed during the afternoon and open up again at night. Part (b) presents pavement temperature at three locations: near the surface, at mid-depth and near the bottom. Part (c) shows the average and difference between pavement temperature at top and bottom.

The particular thermal conditions shown in Figure 5.6, which make the cracks open during parts of the day, are not typical throughout the year but are likely typical of good performing field CRCPs. During warmer days these cracks are fully closed for a larger portion of the day or even closed throughout the entire day, while in the cold season they stay mostly open. On this particular day, June 25, the average pavement temperature ranged from 68 to 74°F.

Temperature at the bottom of the pavement is delayed with respect to temperature at the surface. The average distance between crack faces, which is indicative of the crack’s load transfer
capacity, is delayed with respect to the time of the lowest ambient temperature (represented by temperature at the surface of the pavement). In the case presented in Figure 5.6, the crack is at its widest (average) between 8am and 9am even though the lowest ambient temperature had occurred around 7:30am.

Figure 5.6. Average width (a) and pavement temperature (b and c) versus time during data collection of crack width profile in section 3
Pavement section 4 was evaluated in wintertime, and due to the lower temperatures the cracks in this section do not close completely at any time of the day. Individual crack profiles are shown in Figure 5.7.

Figure 5.7. Crack width profiles during 24-hour in section 4 for four instrumented locations

Average width in each crack is presented in Figure 5.8 along with temperature information. Note that the scales in the charts are different than the ones used for section 3 because of the greater crack width and lower temperatures. Pavement temperature on January 31 ranged from 21 to 37°F. Overall crack width in section 4 for this particular day is about 0.080 mm during the early morning hours, and it lowered to about 0.020 mm in the afternoon. For section 3, measured in June, the maximum width did not reach to 0.025 mm, and it was zero for approximately 8 hours that particular day.
The results presented above for sections 3 and 4 showed the effect of temperature on crack width profile. During a 24-hour period, temperature has the largest effect on crack width changes whereas factors such as long-term drying shrinkage, length of bond-slip zone, and subbase friction are constant or change negligibly.
Studying crack width and its profile is important because the deterioration of the transverse cracks cause loss of load transfer capacity. The wear and abrasion of the fractured concrete faces is a consequence of the repeated traffic loads.

Figure 5.9 shows crack width at top and bottom of the slab as the wheel load travels along section 3. In the unloaded condition, when the wheel load is more than 10 to 15 feet away from the crack, the top of the crack is wider than the bottom. This condition is reversed when the load is in the vicinity of the crack. The top of the crack closes or compresses (negative width) while the bottom of the slab opens up. The data shown corresponds to a 21-kip wheel load pass recorded at 11am on the same day reported in Figure 5.6 (relevant because of the temperature). This load level is representative of a half-axle load of a heavy truck. In all the cases, the crack width at the bottom is greater than at the top at the instant the wheel is exactly at the crack. The crack width profiles when the load is above the crack and at 15 feet after the crack are shown for each crack in Figure 5.10.

Figure 5.9. Crack width at top and bottom of the slab under rolling wheel load
A more detailed example of typical horizontal crack mechanics is presented in Figure 5.11 with data from section 4. Figure 5.11 shows crack width at four depths in the 14-inch slab, as a 45-kip load moves through the pavement section, with detailed profiles at various wheel offsets. The crack experiences a shape reversal as it goes from a condition of open-at-top to a condition of open-at-bottom. This shape reversal may be one cause of the transverse crack deterioration along with differential vertical movement across the crack face.

5.2.3 Importance of crack width profiles

A larger contact area at the crack’s face is preferred because it implies lower shear stresses. The shear stresses produced by a given vertical load (heavy truck axle) are the lowest when the crack is fully closed, while the stresses increase as the crack partially widens and the contact surface become smaller. In the absence of vertical load, the upper portion of the crack is wider and this condition is reversed when heavy traffic loads are situated symmetrically over the crack. An implication of this profile reversal is that at the time of the highest deflection, right before the load crosses the crack, it is the upper part of the crack face that carries most of the shear stress. The measurements shown in Figure 5.11 clearly depict this process, which has been used by other researchers to explain the spalling failure observed in in-service CRC pavements.
5.3. Crack width at standard temperature conditions

The five pavement sections that formed part of this experiment were constructed with design features that should affect crack width. In order to compare crack width between sections, it is necessary to bring the measurements to a common pavement temperature condition and location within the slab depth. The M-E PDG crack width formula was used as a translation mechanism to convert measured crack width at an arbitrary temperature to a standard temperature. The CW model, presented in detail in section 2.5, was used to shift crack width measurements because of its ability to capture the effect of temperature. The formula was calibrated to fit the measured CW magnitudes at different temperatures for each instrumented transverse crack and then the calibrated formula was used to predict CW at 32°F and zero temperature differential at the steel
depth. This assumed standard condition, which also corresponds to water freezing temperature, was selected to make sure the cracks were not fully closed and therefore have a finite width that can be compared and discussed.

5.3.1 Refinement of inputs for the M-E PDG

The M-E PDG software was used to model the constructed sections. The values selected for each input parameter are presented below along with support information. The inputs are divided into those that are specific for each section and those that are common for all of them. The section-specific parameters are slab thickness, percent of steel, steel bar diameter, and depth of steel from the surface. This information was presented in Chapter 3. The common inputs cover climate and concrete and steel material properties. Some are not directly used in the crack width formula but are needed to run other modules of the M-E PDG software. The month of December was used as construction date. Climate information corresponded to the weather station in Champaign, IL, less than 30 miles south of the project’s location. The four structural layers characterized were the following: concrete, asphalt base, granular subbase, and subgrade. Cement type I was used for the concrete mix, with the total content of cementitious material being 605 lb/cy, and a water/cement ratio of 0.4. The aggregate type was limestone. Coefficient of thermal expansion was measured on prisms and cylinders and was 3.54 microstrains per °F. Concrete compressive strength and elastic modulus at 28 days was measured in cylinders made at time of construction and the results were 5,680 psi and 6.8x10^6, respectively. Thermal conductivity and heat capacity were assumed as recommended: 1.25 BTU/hr-ft-°F and 0.28 BTU/lb-°F respectively. Zero-stress temperature of concrete was 70°F, as determined with thermocouples embedded in the pavement. This value corresponded to the temperature at the time of concrete setting, and it was determined as 95% of the highest internal temperature during the hours following concrete placement. An ultimate shrinkage of 601 microstrain (at 40% R.H.) was derived with the formula presented as equation 2.2.38 in Appendix LL of the M-E PDG documentation (ERES, 2004). The 50% default value was used for reversible shrinkage, and 35 days was the default used as the time to develop 50% of ultimate shrinkage. Base type was asphalt treated with an erodibility index of 3. Base/slab friction coefficient was set to 7.5. The thickness of the asphalt treated base was 4 inches and the reference temperature was set at 70°F. Effective binder content was 11%, air voids was 8.5%, and total unit weight was 148 pcf based
on field quality control measurements. A Superpave binder, PG64-22, was used and the default aggregate gradation was used. The thermal conductivity of the asphalt was assumed 0.67 BTU/hr-ft-°F, and the heat capacity was 0.23 BTU/lb-°F. The subbase consisted of crushed stone 6 inches thick. Modulus of the granular layer was internally calculated in the program to be 42,000 psi, with a Poisson’s ratio of 0.35 and 0.5 for the coefficient of lateral pressure. Plasticity index was assumed to be 1, and the amount of material passing #200 sieve and material passing #4 sieve were as defined for a CA6 coarse aggregate gradation by the Illinois DOT: 8% and 43% respectively. Subgrade material was characterized in the first 3 sections as AASHTO A-2-4. The subgrade modulus was set at 11,000 psi as it correlates well with the backcalculated k-value of 200 psi/in (Kohler et al., 2002). Plasticity index was calculated from samples before construction (Kohler et al., 2002) and entered as 9.7, with 6.8% material passing the #200 sieve and 95.5% passing the #4 sieve. The shoulder type was gravel. A zero temperature difference was considered for effective permanent curl/warp. The most important inputs common to all pavement sections are summarized in Table 5.1.

5.3.2 Predicted crack width with the M-E PDG

Time-dependent crack width prediction in the M-E PDG software is affected by pavement temperature, humidity, and other variables such as fluctuation of subgrade support and gain in concrete strength. The direct results of crack width from the M-E PDG program for the first 3 years of pavement life are presented in Figure 5.12, which shows the seasonal variation of crack width and its increase with age. Highlighted in the plot is the predicted crack width, in millimeters, for each section at the approximate age when the actual measurements were taken. These highlighted values are denoted as “prediction at the time of interest” for each section and are used later to compare with actual measurements. As mentioned before, the range of temperatures at the time when CW measurements were taken in each section makes it impossible to directly compare CW between sections. Some sections were tested with temperatures as low as 28°F (section 4) while others as high as 95°F (section 2).
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Value</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCC 28 day compressive strength</td>
<td>$f'_{c28}$</td>
<td>psi</td>
<td>5,680</td>
<td>Measured in cylinders</td>
</tr>
<tr>
<td>PCC elastic modulus</td>
<td>$E_{PCC}$</td>
<td>psi</td>
<td>6,890,000</td>
<td>Measured in cylinders</td>
</tr>
<tr>
<td>PCC 28 day tensile strength</td>
<td>$f_t$</td>
<td>psi</td>
<td>501</td>
<td>Calculated $^a$</td>
</tr>
<tr>
<td>PCC Modulus of Rupture</td>
<td>MR</td>
<td>psi</td>
<td>745</td>
<td>3rd point measurement</td>
</tr>
<tr>
<td>PCC Water/cement ratio</td>
<td>w/c</td>
<td>Fraction</td>
<td>0.4</td>
<td>Mix design</td>
</tr>
<tr>
<td>PCC coefficient of thermal expansion</td>
<td>$\alpha_{PCC}$</td>
<td>1/oF</td>
<td>$3.54 \times 10^{-6}$</td>
<td>Measured in prisms</td>
</tr>
<tr>
<td>PCC Poisson's ratio</td>
<td>$\mu_{PCC}$</td>
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<td>Default value</td>
</tr>
<tr>
<td>Ultimate shrinkage</td>
<td>$\epsilon_{\infty}$</td>
<td>unit less</td>
<td>$601 \times 10^{-6}$</td>
<td>Calculated (Eq 2.2.38 App. LL) $^b$</td>
</tr>
<tr>
<td>Shoulder joint stiffness</td>
<td>Js</td>
<td>unit less</td>
<td>0.04</td>
<td>Granular (Table 7 App. LL) $^b$</td>
</tr>
<tr>
<td>LTE of base (alone)</td>
<td>LTE$_{Base}$</td>
<td>%</td>
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<td>ATB or CTB base (Table 8 App. LL) $^b$</td>
</tr>
<tr>
<td>Base thickness</td>
<td>$h_{Base}$</td>
<td>inch</td>
<td>4</td>
<td>BAM base, design</td>
</tr>
<tr>
<td>Base friction coefficient</td>
<td>F</td>
<td>unit less</td>
<td>7.5</td>
<td>ATB mean value (Table 6 App. LL) $^b$</td>
</tr>
<tr>
<td>Percent subgrade passing # 200 sieve</td>
<td>$P_{200}$</td>
<td>%</td>
<td>6.8</td>
<td>Measured</td>
</tr>
<tr>
<td>Percent subgrade passing # 4 sieve</td>
<td>$P_{4}$</td>
<td>%</td>
<td>95.5</td>
<td>Measured</td>
</tr>
<tr>
<td>Temperature at time of concrete set</td>
<td>$T_{set}$</td>
<td>°F</td>
<td>70</td>
<td>Est. from temp. measured in first 72 hours</td>
</tr>
</tbody>
</table>

$^a$ The formula for tensile strength is $f_t = 0.7 \times (488.5 + E_{PCC}/23000)$

$^b$ M-E PDG APPENDIX LL: Punchouts in Continuously Reinforced Concrete Pavements (ERES, 2004)
5.3.3 Initial comparison of predicted and observed crack spacing and crack width

Crack width obtained with the M-E PDG software is at the depth of the steel. Since the steel in sections 1, 2 and 3 is at the same depth (3.5 inches), the corresponding curves shown in Figure 5.12 can be directly compared. A careful observation of the plot reveals that crack width in section 1 was always greater than in section 2, and both of them are greater than in section 3. This shows that CW decreased as the amount of steel increased. Reinforcement steel in section 4 is located at 4.5 inches from the surface and cannot be compared with any other curve in the plot. Section 5 has two layers of steel (at 3.5 and 7 inches) and this feature is not modeled in the M-E PDG program. A major assumption was made for the two layer steel case by considering the steel to be at an average depth of 5.25 inches. In reality the presence of the upper layer would maintain tighter cracks, however only half of the total steel content of the section is in the upper layer.
Predicted results shown in Figure 5.12 were based on the mean crack spacing from the M-E PDG software for each pavement section, according to equation 5.1 below. Table 5.2 presents crack spacing for each section and the calculated terms in the formula. The actual average crack spacing observed in the sections is presented in Table 5.3. A comparison between predicted and average observed crack spacing is shown in Figure 5.13. In this figure, the error bars correspond to plus or minus five percent from the mean crack spacing.

\[
L = \frac{f_{28} - C\sigma_0 \left(1 - \frac{2\zeta}{h}\right)}{f + \frac{U_s P}{c_i d_s}}
\]  
(formerly Eq. 2.7) [5.1]

<table>
<thead>
<tr>
<th>Section</th>
<th>(L) (feet)</th>
<th>(f_{28})</th>
<th>(C\sigma_0 \left(1 - \frac{2\zeta}{h}\right))</th>
<th>(f)</th>
<th>(\frac{U_s P}{c_i d_s})</th>
</tr>
</thead>
<tbody>
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<td>4.9</td>
<td>501</td>
<td>40.746</td>
<td>7.5</td>
<td>4.122</td>
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<td>40.746</td>
<td>7.5</td>
<td>5.193</td>
</tr>
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<td>Section 3</td>
<td>3.7</td>
<td>501</td>
<td>40.746</td>
<td>7.5</td>
<td>6.663</td>
</tr>
<tr>
<td>Section 4</td>
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<td>501</td>
<td>15.968</td>
<td>7.5</td>
<td>3.569</td>
</tr>
<tr>
<td>Section 5</td>
<td>5.9(^a)</td>
<td>501</td>
<td>N/A</td>
<td>7.5</td>
<td>N/A</td>
</tr>
<tr>
<td>100% steel(\zeta = 3.5\text{ in.})</td>
<td>5.09</td>
<td>501</td>
<td>15.968</td>
<td>7.5</td>
<td>4.189</td>
</tr>
<tr>
<td>100% steel(\zeta = 7\text{ in.})</td>
<td>5.26</td>
<td>501</td>
<td>0</td>
<td>7.5</td>
<td>4.189</td>
</tr>
<tr>
<td>50% steel(\zeta = 3.5\text{ in.})</td>
<td>6.92</td>
<td>501</td>
<td>15.968</td>
<td>7.5</td>
<td>2.094</td>
</tr>
<tr>
<td>50% steel(\zeta = 7\text{ in.})</td>
<td>7.14</td>
<td>501</td>
<td>0</td>
<td>7.5</td>
<td>2.094</td>
</tr>
<tr>
<td>100% steel(\zeta = 5.25\text{ in.})</td>
<td>5.14</td>
<td>501</td>
<td>11.1776</td>
<td>7.5</td>
<td>4.189</td>
</tr>
</tbody>
</table>

\(^a\) Spacing calculated as average of various cases to account for two layers of steel.
Table 5.3. Actual crack spacing statistics

<table>
<thead>
<tr>
<th>Section</th>
<th>Number of cracks</th>
<th>Crack spacing, (feet)</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td>Min</td>
<td>Max</td>
<td>STDV</td>
</tr>
<tr>
<td>Section 1</td>
<td>15</td>
<td>4.6</td>
<td>0.9</td>
<td>26.1</td>
<td>7.81</td>
</tr>
<tr>
<td>Section 2</td>
<td>27</td>
<td>3.0</td>
<td>0.9</td>
<td>9</td>
<td>1.98</td>
</tr>
<tr>
<td>Section 3</td>
<td>33</td>
<td>2.6</td>
<td>0.8</td>
<td>5.6</td>
<td>1.2</td>
</tr>
<tr>
<td>Section 4</td>
<td>15</td>
<td>4.8</td>
<td>0.7</td>
<td>14.2</td>
<td>4.84</td>
</tr>
<tr>
<td>Section 5</td>
<td>4</td>
<td>20.7&lt;sup&gt;a&lt;/sup&gt;</td>
<td>4.4</td>
<td>34.4</td>
<td>12.04</td>
</tr>
</tbody>
</table>

<sup>a</sup> Only four cracks developed in this section

Mean crack spacing from the test sections follows the same trend as the prediction: it decreases from section 1 to section 3 and increases again for section 4. The difference between predicted and average observed crack spacing is 0.3, 1.3, 1.1, and 0.7 feet for sections 1 through 4, respectively. More detailed analysis regarding crack spacing, including the variability observed within each section, is presented in Section 6.2.2.
Regarding crack width, Table 5.4 shows the results of predicted and measured values. This initial comparison indicates that the predicted crack width is consistently higher than the measured values for the sections considered. There were no measured crack width data in section 2, and the predicted crack width for section 5 lacks validity for comparison purposes due to the double layer of steel and reduced number of cracks on that section. Therefore, from crack width evaluation in sections 1, 3, and 4, the predicted CW is on average 6.3 times greater than the measured CW (the ratio of predicted to measured CW was respectively about 3, 10, and 7 for these three sections).

### Table 5.4. Predicted and measured crack width

<table>
<thead>
<tr>
<th>Section</th>
<th>CW (mm)</th>
<th>CS (feet)</th>
<th>Individual CS (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M-E PDG</td>
<td>Measured</td>
<td>M-E PDG</td>
</tr>
<tr>
<td>Section 1</td>
<td>0.169</td>
<td>0.063</td>
<td>4.9</td>
</tr>
<tr>
<td>Section 2</td>
<td>0.115</td>
<td>-</td>
<td>4.3</td>
</tr>
<tr>
<td>Section 3</td>
<td>0.108</td>
<td>0.011</td>
<td>3.7</td>
</tr>
<tr>
<td>Section 4</td>
<td>0.238</td>
<td>0.033</td>
<td>5.5</td>
</tr>
<tr>
<td>Section 5</td>
<td>0.179</td>
<td>0.012</td>
<td>5.9</td>
</tr>
</tbody>
</table>

\* Cr.1 stands for the first crack where CW was measured

The predicted CW values were obtained using either measured or estimated properties from the full-scale test sections and the measured values were considered accurate based on the methodology described in Chapter 4. Therefore several reasons exist why there is a difference between measured and predicted CW.

- **Crack spacing:** predicted crack widths consider the calculated mean crack spacing of the section while measured crack widths are affected by the actual lengths of the adjacent panels. The observed mean spacing for each section is similar to the calculated values, but what really matters is the individual crack spacing associated with the cracks that were instrumented. The measured crack width used for the comparative analysis was taken as the average of all instrumented cracks in each section.

- **Temperature:** predicted crack widths use pavement temperature at the depth of the steel based on historic data from a nearby weather station. The EICM (Enhanced Integrated Climate Model) used in the M-E PDG produces concrete temperature at various points
through the thickness of the PCC layer based on ambient temperature and material properties. “Average nightly monthly temperature at steel depth” is what the program internally uses but such values are not reported in the M-E PDG outputs and therefore are unknown. *The measured crack widths used for comparison are simply the average of the values obtained under the various thermal conditions.*

There are different approaches that can be taken at this point in order to offer a more rigorous analysis of the differences in predicted and observed crack width. Since the procedure uses a great number of variables there are many ways of adjusting the formula to reduce the discrepancies. The following section offers a brief sensitivity analysis of the M-E PDG for several key variables suspected of being influential on the crack spacing and width magnitudes.

**5.3.4 M-E PDG crack spacing and width sensitivity**

Two slab thicknesses were analyzed. For 8-in slabs the steel was simulated at 2.5, 3.0, 3.5 and 4 inches below the surface. For 14-in slabs, the depths were 2.5, 3.5, 4.5, and 7.0 inches. Three temperatures at time of concrete setting were analyzed, 70, 110 and 140°F. The main inputs are presented in Table 5.5. The drying shrinkage is a function of moisture content in the concrete, and a moisture (relative humidity) profile was assumed based on recent measurement. The coefficient of thermal expansion (CTE) of concrete is somewhat low, but it corresponds to values obtained from prisms prepared at time of construction of the CRCP sections.
Table 5.5. Inputs for crack spacing and crack width analysis

<table>
<thead>
<tr>
<th>Input</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (psi)</td>
<td>4,200</td>
</tr>
<tr>
<td>Tensile strength (psi)</td>
<td>455</td>
</tr>
<tr>
<td>Elastic modulus of concrete (psi)</td>
<td>6,300,000</td>
</tr>
<tr>
<td>Ultimate drying shrinkage (in/in)</td>
<td>660x10^{-6}</td>
</tr>
<tr>
<td>Modulus of subgrade reaction, k (psi/in)</td>
<td>255</td>
</tr>
<tr>
<td>Thermal coef. of concrete (1/F)</td>
<td>3.54x10^{-6}</td>
</tr>
<tr>
<td>Steel content (%)</td>
<td>0.78</td>
</tr>
<tr>
<td>Rebar diameter, size (#)</td>
<td>6</td>
</tr>
<tr>
<td>Concrete set temperature (F)</td>
<td>70, 110, and 140</td>
</tr>
<tr>
<td>Concrete temperature at time of analysis (F)</td>
<td>32</td>
</tr>
<tr>
<td>Temperature differential at time of analysis (F)</td>
<td>0</td>
</tr>
<tr>
<td>Concrete relative humidity (%) --variable</td>
<td>65-98</td>
</tr>
<tr>
<td>Subbase friction coefficient</td>
<td>7.5</td>
</tr>
</tbody>
</table>

Crack spacing results are shown in Figure 5.14. The results of crack spacing calculation show that longer crack intervals are obtained with steel placed deeper in the slab. Longer intervals are obtained when the setting temperature is lowered. Taking the 70°F setting temperature for example, by using steel at 2.5 in instead of at mid-depth, crack spacing is reduced from 76 to 52 inches in the thin slab. In the thick slab that reduction is from 80 to 64 inches.

Figure 5.14. Crack spacing versus depth of steel
To compare crack width for different depth of steel, it was preferred to assume the same crack spacing for all cases, so as to isolate the effect of the depth of steel in crack width. According to the formulas, transverse cracks would be closed (zero width) at the depth of steel for the case of 8 in slab with reinforcement at 2.5 in. Cracks in all other cases have a finite width that increases with steel depth, and especially with setting temperature. A crack spacing of 48 was used in the analysis, and the results are shown in Figure 5.15. The DG2002 formula predicts crack width only at the depth of the steel, which can be misleading when trying to compare different designs. To more appropriately compare crack width between the cases analyzed, it is convenient to estimate the width at the same depth, e.g., at the surface. An assumption has to be made to obtain crack width at a depth different than the depth of the steel. This assumption is that the concrete tensile stress caused by the steel restraint is a fixed value and it can be applied to any depth, in the same way that subbase frictional stresses are uniform through the depth. Figure 5.15 shows crack width at the depth of steel (which vary from one point to another in a given curve) and also at 1 and 3 inches below the surface. The moisture profile has a profound impact on crack width in the upper portion of the slab.

Figure 5.15. Crack width versus depth of steel (cw at depth of steel)
5.3.5 Individual calibration

The purpose of using the M-E PDG is to normalize crack width measurements obtained from the different sections so that they can be compared to one another. No global formula is currently available to describe the crack width from time zero to any future time and for all design features. Therefore, a calibrated constant is derived for each individual crack to match its measured behavior. In order to calibrate the crack width formula with the measured values at various temperatures, the formula was entered into a spreadsheet to estimate CW, using the actual temperature parameters Tavg and Tdiff. To that effect, the measured average pavement temperature (Tavg) was used as the temperature at the depth of the steel (Tsteel), and the measured temperature differential (Tdiff) replaced the estimated equivalent temperature difference (Δteqv). All the other inputs were left as they were entered for the DG software runs except for crack spacing. The exact crack spacing for each instrumented crack was used instead of the predicted average spacing for each section.

The average monthly relative humidity in the concrete (rh_pcc) calculated at depth of steel from measured ambient humidity (rh_a) was used, and ranged from about 85% to 90%. These rh_pcc values resulted in relatively high concrete drying shrinkage \( ε_∞(1 - rh_{pcc}^3) \) which caused the predicted CW to be much wider than the measured CW magnitudes. The calibration constant CC in Eq. 5.2 is the adjustment parameter proposed by the M-E PDG to fit the results of the formula to measured crack width data, if available.

\[
CW_i = CC \cdot L \cdot \left( e_{SHRI} + α_{pcc} ΔT ∈ \frac{ε_2, f_{σi}}{E_{pcc}} \right)
\]  

(based on Eq. 2.8) [5.2]

When the average monthly rh_pcc values were used, the calibration constant needed to be reduced considerably in order to balance the high calculated concrete shrinkage. The average CC to fit the formula for all cracks was 0.16. This small calibration constant effectively reduced the sensitivity of CW to important variables (e.g. temperature) in the formula and was deemed inapplicable. Much better results were obtained when the concrete relative humidity was adjusted to a higher value. The CRC pavement likely experienced higher rh_pcc relative to the predictive formula because of the use of a protective tent that covered the sections during testing.
The formula was calibrated to each crack using an $r_{	ext{pcc}}=96\%$. Measured and predicted crack width using the calibrated formula are shown in Figure 5.16. The charts show one crack from each section, and each data point corresponds to a crack width measured at the temperature conditions described in the graph. The calibration constant $CC$ for the crack is also shown in the plot. Details of crack width and calibration constant $CC$ for all the cracks are presented in Table 5.6. The average calibration constant $CC$ for all the studied cracks was 0.51.

![Section 1](image1.png)
![Section 3](image2.png)
![Section 4](image3.png)
![Section 5](image4.png)

Figure 5.16. Examples of crack width measured at various temperature conditions and predicted with calibrated M-E PDG formula

Table 5.6. Average crack width at standard temperature conditions and at depth of steel ($T_{\text{avg}} = 32^\circ\text{F}$ and $T_{\text{diff}} = 0$) with individual crack’s calibration constant

<table>
<thead>
<tr>
<th>Section</th>
<th>Average CW (mm)</th>
<th>Individual CW (mm)</th>
<th>Calibration Constant CC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cr.1   Cr.2   Cr.3   Cr.4</td>
<td>Cr.1   Cr.2   Cr.3   Cr.4</td>
</tr>
<tr>
<td>Section 1</td>
<td>0.116</td>
<td>0.115   0.085   0.145   0.118</td>
<td>0.70   0.60   0.66   0.60</td>
</tr>
<tr>
<td>Section 2</td>
<td>-</td>
<td>-       -       -       -</td>
<td>-       -       -       -</td>
</tr>
<tr>
<td>Section 3</td>
<td>0.064</td>
<td>0.076   0.046   0.068   0.066</td>
<td>0.92   0.31   0.77   0.46</td>
</tr>
<tr>
<td>Section 4</td>
<td>0.031</td>
<td>0.044   0.023   0.035   0.022</td>
<td>0.75   0.38   0.22   0.12</td>
</tr>
<tr>
<td>Section 5</td>
<td>0.081</td>
<td>0.081   -       -       -</td>
<td>0.40   -       -       -</td>
</tr>
</tbody>
</table>
It can be seen from the plots in Figure 5.16 that predicted CW follows the changes in temperature in the same manner as the measured CW, i.e. the crack closes with higher Tavg and higher Tdiff. The model correctly represented the sensitivity of CW to changes in the pavement temperature profile. This feature of the M-E PDG crack width formula was validated. The calibrated formula was then used to standardize crack width to the desired temperature conditions. The average CW at standard temperature conditions on sections 1, 3, 4, and 5 ranged from 0.031 to 0.116 mm (see Table 5.6). Table 5.6 reports the CW at the depth of the steel which is different for sections 4 and 5 compared with sections 1 to 3. The CW on section 2 could not be measured because the cracks were fully closed at time of measurement. Section 5 contained a double layer of steel, not explicitly modeled in M-E PDG, and a reduced number of cracks, which makes any conclusion regarding crack width difficult. The analysis of crack width between sections is presented in Chapter 6, section 6.2.3.

5.4. Prediction of crack width profile with M-E PDG

5.4.1 Modification of the crack width formula

The M-E PDG formula predicts crack width only at the depth of the steel, which can be misleading when trying to compare different design features. To more appropriately compare crack width between the cases analyzed, it is convenient to estimate the width at a standard reference depth, for instance at the slab surface \((z=0)\). The M-E PDG formula can be used to predict crack width at depths other than the depth of the steel, but an additional assumption needs to be made in order to apply the formula. This assumption is that the concrete tensile stress caused by the steel restraint is a fixed value and it can be applied at any depth, in the same way that subbase frictional stresses are treated as uniform through the depth. The formulation for crack width is again presented as follows:

\[
cw = L \left( \varepsilon_{\text{shri}} + \alpha_e \cdot \Delta T_e - \frac{c_2 \cdot f_{\sigma}}{E_{\epsilon_i}} \right) \quad \text{(formerly Eq. 2.8)}
\]

(formerly Eq. 2.8) [5.3]
\( \bar{L} \): Average crack spacing

\( \varepsilon_{SHR,i} \): unrestrained concrete drying shrinkage at depth of steel

\( \alpha_c \): Concrete CTE

\( \Delta T_{\xi} \): drop in concrete temperature from zero-stress temperature

\( c_2 \): Bond-slip coefficient

\( f_{\sigma} \): Maximum tensile stress in the concrete

\( E_c \): Concrete modulus of elasticity

The expression for tensile stress in the concrete is

\[
f_{\sigma} = \frac{LU_m P_b}{c_1 d_b} + C_i \sigma_{\varepsilon_0} \left( 1 - \frac{2 \varepsilon_{\xi}}{h} \right) + \frac{L}{2} f_{\sigma} \text{ (formerly Eq. 2.11a)}
\]

\[ [5.4] \]

which is the restraint of the steel, plus environmental curling and warping, plus slab-base frictional restraint.

**Restraint of steel**

\[
\frac{LU_m P_b}{c_1 d_b}
\]

\( L \): crack spacing

\( U_m \): Peak Bond Stress

\( P_b \): Percent steel, fraction

\( c_1 \): Bond stress coefficient

\( d_b \): Reinforcing steel bar diameter, inch

The tensile stress resulting from steel restraint is a function of the depth of the steel because \( c_1 \) is calculated based on concrete strains at the depth of steel \( \varepsilon_{\text{tot-\xi}} \).

\[
c_1 = 0.577 - 9.499e^{-9} \left( \frac{\ln \varepsilon_{\text{tot-\xi}}}{(\varepsilon_{\text{tot-\xi}})} \right) + 0.00502 \bar{L} * (\ln \bar{L}) \text{ (formerly Eq. 2.11c)}
\]

\[ [5.5] \]

where \( \varepsilon_{\text{tot-\xi}} \) depends on temperature drop \( \Delta T_{\xi} \), and drying shrinkage \( \varepsilon_{SHR,i} \).
Environmental curling and warping

\[ C_i \sigma_{oi} \left(1 - \frac{2\zeta}{h}\right) \]

\( C_i \) is an adjustment coefficient for finite size of the panel, and \( \sigma_{oi} \) is the stress calculated from the strain differential of the slab, which is

\[ \varepsilon_{\text{tot}} \Delta_m = \alpha_{\text{PCC}} \Delta_{\text{eqv}} \Delta_m + \varepsilon_\infty \Delta(1 - r \rho_{\text{PCC}}^3)_{\text{eqv}} \]  

(formerly Eq. 2.11e) \[5.6\]

The factor \( \left(1 - \frac{2\zeta}{h}\right) \) adjusts the stress to the depth \( \zeta \), which does not need to be depth of steel and can be replaced by the depth of interest ‘z’.

Friction restraint

\[ \frac{L}{2} f \]

\( L \) is crack spacing, and \( f \) is the friction coefficient, which is independent of depth of steel or depth at which crack width is sought.

The equation for crack width can be re-defined to calculate the crack width at any depth in the slab \( z \), as the following:

\[ \text{cw}(z) = L \left\{ \varepsilon_{\text{SIR1}}(z) + \alpha_e \cdot \Delta T_z - \frac{c_{z,\sigma}}{E_{c, i}} \left( \frac{L \rho_m P_h}{c_{v, d, b}} + C_i \sigma_{oi} \left(1 - \frac{2z}{h}\right) + \frac{L}{2} f \right) \right\} \]  

\[5.7\]

The drop in temperature can be assumed uniform along the thickness, but it could also be refined if a zero-stress temperature profile and instantaneous temperature profile are both available (to calculate temperature drop at specific depths). The moisture profile affects directly the term \( \varepsilon_{\text{SIR1}} = \varepsilon_\infty \left(1 - r \rho(z)^3\right) \). It is necessary to know the internal moisture of the concrete at the depth
of interest (or better yet, the profile along the thickness) of the slab in order to obtain crack width at the specific depth of interest ‘\( z \)’.

5.4.2 Sensitivity analysis

Since the M-E PDG model for crack width has been modified to predict not only at the depth of the steel but at any slab depth, the new formula can be used to study the sensitivity of the model to the two most important variables, humidity and temperature. To perform the sensitivity analysis the standard case used is section 4 (\( h=14 \) in., depth of steel = 4.5 in., \( P=0.78\% \), bar diameter = 0.875in).

Humidity

Two internal relative humidity (RH) profiles are analyzed for the slab. The first is a simplified profile consisting of a linear increase from 80% RH at the surface to 99% RH at the bottom. The second profile is named “realistic” and is based on measurements performed at various depths during August 2004. The realistic profile is mathematically modeled in two parts, the first one varying linearly from 65% to 78% in the top half inch of the slab and the second one following the equation \( RH = 100 - 11 \cdot z^{-0.99} \), where \( z \) is the depth in the slab. Both profiles are shown in Figure 5.17.

![Figure 5.17. Relative humidity profiles](image)

Figure 5.17. Relative humidity profiles
Temperature

Two average pavement temperature values are considered: 32 and 43°F. Two temperature differentials from top to bottom of the slab are considered: the first is with the surface 20°F warmer and the second with the surface 15°F cooler than the bottom. Linear variation through the depth is assumed.

Figure 5.18 shows the effect of the changes in relative humidity profile, average temperature and temperature differential. A dryer concrete implies a greater crack width, and the sensitivity to this parameter is considerable. A change in average pavement temperature causes a nearly uniform change in crack width along the thickness of the slab, and a change in the temperature differential produces rotation of the crack faces. As seen in Figure 5.18, relative humidity is a significant factor in the prediction of crack width based on the M-E PDG model.

Figure 5.18. Change in crack width profile under the effect of change in (a) relative humidity profile, (b) average temperature, and (c) temperature differential
5.5. Short-term prediction of CW using M-E PDG

5.5.1 Hourly CW model and data

In theory, the M-E PDG model may be applied to predict not only crack width on a monthly basis, but also on an hourly basis. The purpose of hourly CW prediction is to determine the factors affecting short-term changes in CW and by knowing the hourly CW, the effects of wheel loading on crack deterioration may be determined during accelerated pavement testing. The first step to examine the model is to have continuous readings of crack width from the experimental sections. Continuous readings were obtained using the following methods:

(a) By knowing CW at one point in time (by a load spectra test) and then applying the changes in CW measured continuously.

(b) If the total measured crack closing under load for a range of temperatures is known, then the CW can be determined by the temperature spectra method.

A typical comparison between measured and predicted hourly crack width is shown in Figure 5.19 for data measured at the steel depth in section 4 during the end of January 2004 (concrete age=25 months, temperatures under 40°F). The M-E PDG formula is applied using the actual average temperature in the depth of the pavement and the temperature difference collected during testing, which are shown in Figure 5.20. For the hourly prediction, the calibration constant was reset to its default value, CC=1. The relative humidity at the depth of the steel was chosen to obtain as good a match with the measured CW curve. This relative humidity at the depth of steel value used is 96 percent and is assumed constant over the three days of analysis. Under these assumptions the changes in crack width are only due to temperature because everything else in the CW formula is constant. It can be seen that the model responds to temperature with the correct trend, however, the predicted magnitude or amplitude is smaller than the measured.
The arbitrary assumption of a constant relative humidity of 96% was done in the initial model assessment to determine if temperature effects were the only factor in short-term CW changes. In an effort to quantify variation of relative humidity and to assess typical values, RH measurements were taken in the full-scale CRCP sections, as explained next.
5.5.2 Relative humidity measurements

Relative humidity sensors (model Sensirion SHT75) were installed at different depths in the slab in August 2004 to monitor hourly RH profile changes. Although humidity is also affected by other variables, only the relationship with the temperature of the slab was studied. Using data collected during a two-week period, RH behaved inversely to the temperature when both parameters were measured near the pavement surface, but inside the concrete slab the relationship was proportional. Figure 5.21 presents RH and temperature for the cases at 0.5 inch and 4.5 inches from the surface.

![Relative humidity and temperature data in the pavement](image)

Figure 5.21. Relative humidity and temperature data in the pavement (a) at 0.5 inches from the surface, and (b) at 4.5 inches from the surface

These findings are in accordance to those by Grasley and Lange (2004) who noted that within a partially saturated porous material such as concrete, the relationship between temperature and relative humidity is the reverse of that in the open environment. Relative humidity in such a material is controlled primarily by the state of the capillary meniscus. When temperature is
increased, water expands, pore fluid pressure increases, and this leads to an increase in internal RH.

A linear regression for RH as a function of temperature at 4.5 inches below the surface resulted in RH = 81 + 0.16*T(°F), R^2 = 0.29, n = 308, for a 13 day period (see Figure 5.22). Even though this formula was developed with limited data and under particular temperature conditions, a relationship like this can be used in the crack width formula to see if improvements to the prediction are obtained. As mentioned previously, humidity sensors were installed at various depths in the thickness of the concrete, 0.5, 1.5, 3.0 and 4.5 inches from the top of the pavement. The results indicate that humidity increases with depth, as shown in Figure 5.23. Below approximately two inches from the surface the relative humidity stays at values higher than 90 percent. Additional measurements (not shown) indicated internal RH between 98 and 99 percent near the bottom of the slab.

Figure 5.22. Relative humidity versus temperature at 4.5 inches below pavement surface.
5.5.3 Re-evaluation of shrinkage components in M-E PDG CW model

The shrinkage component in the M-E PDG crack width formula uses the relative humidity to adjust for moisture changes:

\[ \varepsilon_{SHRI} = \varepsilon_\infty \left(1 - r_{h_{PCC}}^3\right) \]  
(formerly equation 2.9a)  

\[ [5.8] \]

The term \( \varepsilon_\infty \) is the ultimate drying shrinkage and \( r_{h_{PCC}} \) is the relative humidity in the concrete at the depth of interest. Two things deserve comment regarding the formula: the use of ultimate shrinkage and the use of the cube of the relative humidity.

Using the ultimate drying shrinkage multiplied by the factor \( (1-r_{h_{PCC}}^3) \) implies that in the case of 100 percent humidity the shrinkage component in the crack width formula becomes zero, which theoretically is incorrect. The M-E PDG states what other researchers have found regarding that the total shrinkage that concrete experiences needs to be separated into reversible and irreversible components; however this concept is not used in the crack width formula for CRCP in the M-E PDG.
The other aspect is the cubic exponent for relative humidity as it relates to the strain in the concrete. Research has shown that the volumetric changes caused by variations in moisture content are explained by the negative pressure within the fluid in the micropores of the concrete (Persson 1998, Lura 2003, Jensen and Hansen 1996, Grasley et al 2003). With increase in temperature, water expands and reduces the surface tension, which reduces the negative pressure in the pores, and therefore allows for expansion in the concrete in addition to thermal expansion. Governing this process is the Kelvin-LaPlace equation (Grasley et al 2003), which shows an approximate linear relationship between pore pressure or volumetric strain and RH. The basis for using a cubic power for the effect of relative humidity in shrinkage in the M-E PDG is not clear and should be re-evaluated.

5.5.4 Prediction with variable relative humidity

The prediction of crack width with the M-E PDG model was presented in Figure 5.19 for the case of constant internal relative humidity. Field measurements indicated that inside the slab the humidity can be approximated by a linear model based on temperature. Unfortunately the RH measurements were taken after the crack width measurements had ended, and therefore there is no simultaneous data of crack width, relative humidity, and temperature for the field test sections. The application of the linear model for relative humidity to the crack width data resulted in an enlarged amplitude of the predicted crack width, but it also caused a shift in the curve upward because the calculated humidity was in a lower range (88 to 90 percent). Figure 5.24 shows the predicted crack width that would result if the hypothetical model RH=77+0.5*Tavg is assumed (where Tavg is the pavement temperature in °F). Such a model was not obtained from measurements, but the following observations should be taken into account:

- A positive trend of relative humidity with temperature was obtained in the field and is supported by theory.
- The pavement temperature at the time of RH measurements was 64 to 80°F, and the RH readings were 91 to 95%. Applying the linear model with the temperatures at which the CW measurements were taken, (36 to 40°F) reduces considerably the relative humidity.
These measurements and other research (Zollinger 2003) indicate that RH should be above 90%.

- A more comprehensive model for RH, that uses not only pavement temperature as a predictor but also climatic conditions would be helpful to clarify if the simple hypothetical model is viable.

### 5.5.5 Prediction with additional curling term

Another approach to try to improve the hourly prediction capacity of the M-E PDG CW model is to give more weight to the already existing effect of temperature differential. An additional term $R \cdot T_{diff}$ was included into equation 5.3, which results in equation 5.9:

$$ c_w = L \left\{ e_{SHR_i} + \alpha_c \cdot \Delta T_v - \frac{c_2 \cdot f_\sigma}{E_{\psi i}} + R \cdot T_{diff} \right\} $$  \[5.9\]

![Figure 5.24. Hourly crack width predicted with hypothetical linear model for relative humidity](image)

Figure 5.24. Hourly crack width predicted with hypothetical linear model for relative humidity
The Westergaard formula for temperature curling stress is \( \frac{E}{2(1-\nu)} \cdot \alpha \cdot T_{\text{diff}} \), where \( E \) is the elastic modulus, \( \nu \) is Poisson ratio, and \( \alpha \) is the coefficient of thermal expansion. Rotations of the crack faces cause an increase in CW at the upper part of the slab if the surface is colder. The relationship between curling stress and CW depends then on the depth of analysis. The expression that multiplies \( T_{\text{diff}} \) was calibrated for the particular data shown in Figure 5.24 by combining into the factor R: a) the existing parameters of the Westergaard formula, b) the effect of CW depth of interest, and c) a calibration factor. If \( R = -1.38 \cdot 10^{-5} \) is used in Eq. 5.9 and the RH is held constant at 90 percent, the results are in close proximity to the measured crack width, as shown in Figure 5.25. Note that the prediction curve is “ahead” of the measured data, which is in accordance to the observed delayed response of crack width to changes in temperature. If this approach is to be implemented, it seems appropriate to use nonlinear temperature profile or data from approximately one hour before the time of the desired prediction.

![Figure 5.25. Hourly crack width predicted with calibrated temperature differential term](image)

The use of the extra term in the CW formula is justified for short-term prediction because of the need for a more sensitive response. Prediction of a representative crack width value for an entire month, as in the M-E PDG, does not require the level of refinement of an hourly model.
5.5.6 Short term prediction of crack width comments

The M-E PDG crack width formula requires complex expressions and calibrations to account for all the factors involved. It is intended in the M-E PDG to predict monthly behavior of crack width, and an application in much shorter time intervals was attempted with collected data. The CW predictions were less variable relative to the measured crack width when a constant RH was used. No relative humidity measurements were available from the time of CW testing. The CW formula is very sensitive to RH values because it uses the ultimate drying shrinkage which is related to the RH cubed. The purpose of having a model to predict hourly changes in crack width is to correct or standardize measurements taken at different times of the day and to combine the effects of crack width changes with the results of continuous accelerated load testing. Investigations into the hourly variation of RH inside the concrete pavement revealed that if more accurate RH data is taken into consideration in the CW formula, it would increase the CW amplitudes making the hourly model more sensitive. Another approach to increase amplitude of the hourly prediction was to add another term to put emphasis on the temperature differential effect. The result showed that it is plausible to include temperature differential to refine short-term predictions.

5.6. Zero-stress temperature

5.6.1 Estimated zero-stress temperature

The effect pavement temperature has on the crack width magnitude is controlled to a large extent by the zero-stress temperature. Given the importance of the zero-stress temperature for crack width, a more detailed analysis of this parameter was necessary for the APT sections. The zero-stress temperature, \( T_{zs} \), is defined in the M-E PDG as the temperature at which the concrete hardens sufficiently to cause cracks to open when the concrete temperature drops below its value, and it constitutes one of the three terms in the M-E PDG formula for crack width (equation 5.3).
Schindler (2002) described the theoretical development of early-age concrete temperatures and thermal stresses over time in a fully restrained specimen. The process is shown in Figure 5.26. The concrete is plastic at placement and stresses do not start to develop until enough hydration products have formed to cause final setting, which occurs at time $t_f$. The hydration of cement with water is exothermic in nature, and this causes the concrete temperature to increase beyond the setting temperature. The restrained expansion of the concrete caused by the temperature leads to the development of compressive stresses until the peak temperature ($T_{\text{max}}$) is reached at time $t_a$. During this phase, the hydrating paste is still developing structure, the strength is low, and most of the early-age compressive stresses are relaxed (Springenschmid and Breitenbücher, 1991, Westman, 1999). As concrete temperature starts to drop, the compressive stresses are relieved until the concrete temperature falls below the zero-stress temperature ($T_{zs}$). This condition is reached at time $t_{zs}$, where the stress condition changes from compression to tension for the first time.

The behavior of the concrete temperature development after placement is a complex problem. It is primarily affected by the temperature of the concrete at placement, the ambient temperature,
the type and quantity of the cementitious materials, the solar radiation intensity, and the boundary conditions of the pavement. Schindler (2002) also studied the relationship between peak temperature and the zero-stress temperature and proposed a simplified procedure in which $T_{zs}$ can be obtained as a percentage of $T_{\text{max}}$. The reduction factors are 8, 6, and 4.5 percent depending whether the concrete is placed in hot, normal, or cold weather conditions (defined as air temperature above 80°F, between 65 and 80°F, or below 65°F, respectively).

As mentioned in Chapter 2, zero-stress temperature can be input directly into the M-E PDG or it can be estimated from monthly ambient temperature and cement content using the equation shown below, which is based on daytime construction with curing compound. The allowable range with this formula is from 60 to 120°F.

$$T_z = CC \cdot 0.59328 \cdot H \cdot 0.5 \cdot 1000 \cdot \frac{1.8}{1.1 \cdot 2400} + MMT$$  \hspace{1cm} (formerly Eq. 2.10)  \hspace{1cm} [5.8]

where,

- $T_z$ : temperature at which the PCC layer exhibits zero thermal stress
- $CC$ : cementitious content, lb/yd³.
- $H$ : Heat of hydration per unit weight
- $H = -0.0787 + 0.007 \cdot MMT - 0.00003 \cdot MMT^2$
- $MMT$: mean monthly air temperature for month of construction, °F.

This equation is to be used when the temperature on the day of construction is not known, and the best information available is the historic mean temperature for that month. Assuming temperatures scenarios between 45 and 55°F during the day of concrete placement, the zero-stress temperature would be between 61 and 75°F (values obtained considering 460 lb/cy).

The zero-stress temperature can also be estimated using pavement temperature data collected during the hours after placement. Figure 5.27 shows concrete internal temperature and the temperature differential for each of the five sections of this study during the 72-hours following construction. Five thermocouples were embedded through the thickness of the slab in each section. The internal temperature was calculated as the average on the three central thermocouples (in the depth) and the difference was obtained using the ones at top and bottom.
The rise in temperature caused by the heat of hydration can be observed in the initial part of the plot.

Figure 5.27. Concrete internal temperature and temperature difference for 3 days after pouring

Note that the temperature difference from top to bottom of the slab for all sections in the period following the peak temperature is slightly negative (about 2 to -4°F between midnight and 6am). This is different from most rigid pavements, which experience higher positive differences in
temperature from top to bottom after the peak slab temperature. The majority of concrete pavement construction occurs during hot sunny days with the solar radiation plus the heat of hydration causing the slab to set, in a flat condition, with the upper portion much warmer than the bottom. During times when the temperature difference is more negative than the temperature difference at setting, the slab tends to curl upward causing tensile stress at the top of the slab. In the experimental sections, this phenomenon seems to be less relevant because the slab remains flat or curled downward most of the time, with the edge in contact with the base layer, which provides support and reduces stress.

Table 5.7 presents the exact time of concrete placement, the peak temperature, and the time of peak temperature for each of the five sections. The peak temperature increased as sections were paved later in the day (except for section 3).

Table 5.7. Concrete placement time and peak internal temperature

<table>
<thead>
<tr>
<th>Section</th>
<th>Placement time</th>
<th>Peak temperature (°F)</th>
<th>Time of peak temp.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>8:30 AM</td>
<td>76.1</td>
<td>8:30 PM</td>
</tr>
<tr>
<td>Section 2</td>
<td>10:30 AM</td>
<td>76.7</td>
<td>12:00 AM</td>
</tr>
<tr>
<td>Section 3</td>
<td>11:30 AM</td>
<td>73.8</td>
<td>1:30 AM</td>
</tr>
<tr>
<td>Section 4</td>
<td>12:30 PM</td>
<td>86.0</td>
<td>2:30 AM</td>
</tr>
<tr>
<td>Section 5</td>
<td>1:30 PM</td>
<td>88.8</td>
<td>3:30 AM</td>
</tr>
</tbody>
</table>

The paving took place on December 3rd 2001, and the five sections were paved between 7:30 am and 3:30 p.m. The peak internal temperature occurred between 8:30pm and 3:30am on the night of the paving day, approximately 12 to 14 hours after pouring. The air temperature was 45°F at the beginning of paving operations and increased throughout the day. The temperature of the fresh concrete was 60°F for the first two sections, 63°F for sections 3 and 4, and 66°F for section 5. Black plastic sheets covered the sections during the weeklong curing time.

Table 5.8 presents zero-stress temperature for each section using Schindler’s simplified procedure in which Tzs is obtained as the peak temperature reduced by 4.5 percent (cold weather paving).
Table 5.8. Zero-stress temperature with simplified procedure

<table>
<thead>
<tr>
<th>Section</th>
<th>95.5% of peak temp (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>72.7</td>
</tr>
<tr>
<td>Section 2</td>
<td>73.2</td>
</tr>
<tr>
<td>Section 3</td>
<td>70.5</td>
</tr>
<tr>
<td>Section 4</td>
<td>82.1</td>
</tr>
<tr>
<td>Section 5</td>
<td>84.8</td>
</tr>
</tbody>
</table>

5.6.2 Crack closing temperature from CW measurements

The analysis of crack width versus pavement temperature provides information on the temperature above which the cracks become closed. The analysis was performed for crack width at the depth of the steel. In a given section, each crack has its own closing temperature that was calculated with a linear regression to zero crack width. An example from section 4 is presented in Figure 5.28 and shows that the crack would be closed when the pavement temperature is about 52.6°F. Crack width was determined for this crack to be 0.044 mm at the standard pavement temperature of 32°F (shown in Table 5.6).

The scatter comes from the effect of the temperature differential through the thickness of the slab. Closing temperatures for the individual cracks are presented in Table 5.9 along with the section average. There is variability within each section and between sections. Some of the regressions suffer from low coefficients of determination due to the aforementioned scatter and
the fact that the range of temperature in which the CW measurements were obtained in some sections was small.

<table>
<thead>
<tr>
<th>Section</th>
<th>Cr.1</th>
<th>Cr.2</th>
<th>Cr.3</th>
<th>Cr.4</th>
<th>Section average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
<td>68.8</td>
<td>66.3</td>
<td>71.4</td>
<td>65.5</td>
<td>68.0</td>
</tr>
<tr>
<td>Section 3</td>
<td>75.7</td>
<td>73.8</td>
<td>71.0</td>
<td>72.1</td>
<td>73.1</td>
</tr>
<tr>
<td>Section 4</td>
<td>52.6</td>
<td>59.2</td>
<td>57.7</td>
<td>49.0</td>
<td>54.6</td>
</tr>
<tr>
<td>Section 5</td>
<td>66.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>66.4</td>
</tr>
</tbody>
</table>

5.6.3 **Discussion of zero-stress temperature and crack closing temperatures**

A graph comparing the zero-stress temperature calculated with the simplified method (taking the 95.5 percent of the peak temperature) and the crack closing temperature (Table 5.9) is presented in Figure 5.29, along with the zero-stress temperature calculated with Equation 5.8.

![Figure 5.29. Zero-stress temperature calculated with two methods and the crack closing temperature for each section](image)

The two calculation methods provide similar results, although both have limitations. The simplified method uses a single reduction factor for “cold weather paving” for the case of temperature under 65°F, but it is possible to better refine it for lower temperatures. In fact the
lower the air temperature during construction, the lower is the percentage of the peak temperature that is considered for zero-stress temperature (Schindler, 2002). The method of equation 5.8 should be considered less reliable since it uses only ambient temperature and cement content, neglecting other particular pavement details.

Zero-stress temperature is a concept easily understood in relation to crack width at early age: there exists a certain temperature that represents the limit at which the cracks would change from open to closed status or vice versa. As the transverse cracking pattern fully develops over the first years of the pavement life, the zero-stress temperature of the early age becomes different from the closing temperature at later ages. The closing temperature is expected to be higher than \( T_{zs} \) given the additional drying shrinkage that makes the cracks wider, but at the same time it is reasonable to expect lower closing temperature considering the overall expansion of the length of the pavement due to later age cracks and debris. After transverse cracks have formed they do not completely disappear, which means that even a closed crack (\( CW=0 \)) adds to the total length of slab, if it is not restrained. The cracks behave as closed, with their faces in contact and full aggregate interlock, but they do not fit perfectly back as they did before cracking. Crack closing temperatures resulted in lower than the calculated zero-stress temperature in three of the four sections with data. These results give credence to the idea of possible reduction in effective crack width caused by pavement expansion.

5.7. Study of variability of crack width

Results obtained from the experimental sections evaluated with the ATLAS indicate dispersion of crack widths, even within homogeneous sections. It is important to approximately know the variability of crack width in a CRCP section to estimate its expected distribution. Faulted punchouts will likely develop adjacent to wide cracks because a considerable loss of load transfer capacity is necessary to develop the failure. Knowledge of the distribution of crack width will allow a more realistic relationship between punchouts and crack width since the frequency of wide cracks should correlate better with punchouts than the average crack width. Since assessment of crack width variability is necessary to improve punchout prediction models, two approaches were implemented on the experimental sections to obtain crack width from larger sample sizes (measuring \( CW \) on a greater number of transverse cracks per uniform
section). The first method consisted of using the ATLAS as the loading mechanism, while the second method involved impulse loading with the falling weight deflectometer (FWD). A summary of the findings can be found in Kohler and Roesler (2006).

5.7.1 CW measurement with the ATLAS

5.7.1.1 Testing procedure
The procedure called load spectra tests, which was explained in Chapter 4, was used to determine crack width. The difference with the earlier application is that the horizontal sensors that measure crack closing were installed on the surface of the slab instead on the side of the slab. These LVDT sensors were placed at about one inch from the edge of the pavement. The wheel load was applied adjacent to the sensors near the edge to maximize the crack closing deformations. Initially eight sensors were installed on consecutive transverse cracks in section 3 followed by tests with 8 additional sensors, and finally a test with 31 sensors. The testing was performed during February, March, and May 2005. Photographs of the sensors and the ATLAS’s wheel are presented in Figure 5.30.
5.7.1.2 CW results

The eight sensors installed at the beginning of section 3 were used to collect crack closing movement when the pavement temperature near the surface (1 inch below) was 48.0°F and near the bottom (at 9 inches) was 40.2°F. For several weeks 8 additional sensors were installed at the end of the section and a test was run when temperatures were 38.3°F and 32.6°F near the top and bottom, respectively. Finally, 31 sensors were installed to monitor 31 out of 33 cracks present in section 3. The average pavement temperature was 49.1°F. For the first test, called herein dataset 1, the applied load ranged from 6 to 45 kips, at 3-kip increments. Loads up to 42-kips were used in the second test, designated as dataset 2. Loads ranged from 6 to 45 kips in the last dataset (called dataset 3). For each dataset, crack width was obtained as the average crack movement of several wheel passes at each load level. A typical plot of crack movement versus load level is presented in Figure 5.31, in which the change in slope is interpreted as indication of full crack closing. Since the sensors are attached to the surface, crack width is reported at the surface not the depth of steel. Figure 5.32 presents the result of crack width measurements for the three
datasets. Datasets 1 and 2 were instrumented at the following stations (ft): 2.5, 4.4, 5.6, 6.4, 8.1, 9.0, 11.6, 13.5, 56.6, 60.7, 66.4, 71.0, 74.5, 76.6, 78.7, and 80.2.

Figure 5.31. Determination of crack width from crack closing (typical)

Figure 5.32. Measured crack width along section 3
The first observation from the plot is that crack width in dataset 1 is smaller than in dataset 2 due to the higher pavement temperatures during testing. The second observation that all datasets have similar trends of crack widths with section stationing. The final observation is the considerably smaller crack width at the end of the section (stations 56.6 to 80.2 feet) compared to the beginning of the section (stations 2.5 to 13.5) based on dataset 2 measurements. This is linked to the damage observed on the slab after the repeated heavy loading. As a reference, a map of the cracks in section 3 is presented in Figure 5.33. The greater crack widths at the beginning of the section can be interpreted as the cause of the damage as the first punchout extended from station 0.0 to about station 26.0 ft. Another interpretation could be that the crack width was greater because of the repeated load damage. It is not clear which of the two interpretations is more appropriate.

The rate of horizontal movement is different before and after the crack faces are brought into contact by the action of the load, and depend on multiple factors such as slab thickness, depth and amount of reinforcement steel, and pavement temperature profile. Figure 5.34 shows a higher rate of movement observed at low load levels, when the crack is still open, which translates into a steeper initial slope of crack width versus load. Lower rates of movement occur at higher load levels because the faces of the crack are in contact and the concrete is subjected to elastic compression.
Figure 5.34. Difference in measured closing rate when the crack is open (lower loads) and when the crack has closed (higher loads).

5.7.2 CW measurement with the FWD

5.7.2.1 Testing procedure
The load spectra test (LST) proved to be effective in obtaining crack width, but required the application of a range of vertical loads. The ATLAS provides an ample range of loads, which facilitates the testing, but cannot be considered practical for field application and therefore an alternative source for vertical load was investigated. The load application through a Falling Weight Deflectometer was first tried in March 2004, when one crack in sections 6, 7, and 8 were tested. During April 2005 two more testing sessions were conducted with specific objectives of obtaining crack width variations within a single test section. On April 16th 2005 IDOT’s FWD was used to test all 22 cracks in section 8 and 15 cracks in section 9. Load levels ranging from 9 to 24 kips were used. On April 25th 2005 a second FWD was used to test 25 cracks in sections 1, 2, and 8. Figure 5.35 shows IDOT’s FWD on section 8.
When the ATLAS was used for crack width evaluation the sensors were connected to the data acquisition system inside the ATLAS control trailer. A portable system needed to be developed for the CW testing with the FWD. A single LVDT was attached to the pavement surface every time the testing operation moved from one crack to the next. The testing sequence consisted of positioning the FWD load plate symmetrically over the crack, at about 6 to 12 inches from the edge (see Figure 5.36). The block holding the LVDT was then glued to the pavement along with the reaction bracket on the opposite side of the crack. After 60 seconds the FWD loading and data collection program commenced. At the end of the loading sequencing, the sensor is removed and moved to the next transverse crack.

The data collection system included a signal conditioner for the LVDT, a 12V battery-powered datalogger, and a laptop computer with a PCMCIA DAQ card that connects to the datalogger, all mounted on a cart that can be moved along with the FWD, as shown in Figure 5.37.
5.7.2.2 CW results

During the first experiment in March 2004, a Heavy Weight Deflectometer was used to apply loads that varied for each test, but were approximately 16.5, 28, 37, and 48 kips. The results indicated crack width of 13 and 20 microns in sections 6 and 7 respectively, with only one crack
being measured per section. The width of the crack in section 8 could not be determined. The temperature at the pavement surface was 44, 47, and 49°F at the time of the corresponding evaluation in sections 6, 7, and 8.

The higher pavement temperature affected the first FWD testing conducted in April 2005. Crack width could only be determined on three of the 22 evaluated cracks in section 8, and in two of the 16 cracks in section 9. The measurements were carried out between 10:30am and 1:00pm when the pavement temperature was rising. Temperature near the surface went from 63.0°F to 81.9°F and at the bottom from 51.9°F to 58.7°F, resulting in an average pavement temperature during the testing of about 64.5°F.

The second testing conducted in April 2005 was scheduled for the early morning, but the forecast for heavy rainfall forced the testing to be shifted to the preceding evening. This testing resulted in crack width determination on 5 out of 7 tested cracks in section 1, 5 out of 12 in section 2, and 4 out of 6 in section 8. The testing started at about 6:30 pm where the surface began at 65.7°F and the bottom was at 61.3°F. The testing ended at 9:30 pm when the surface was 53.6°F and the bottom was 60.9°F. The average pavement temperature during the testing was approximately 61.0°F. The air temperature also dropped from 60.2°F to 48.7°F during the testing.

5.7.3 CW variability results

The small quantity of collected data makes it difficult to infer results of crack width variability. To assess variability of crack width, all the sampled cracks need to form a homogeneous population, which means that crack width has to come from a section with uniform design features and have been obtained under similar temperature conditions. During high temperatures all cracks may be closed. As the pavement cools, some cracks would open and the standard deviation between cracks would increase as the temperature decreased. The actual variability for a section could theoretically be obtained only when all the active cracks have non-zero crack width.
A summary of CW measured per uniform section grouped by loading source is presented in Table 5.10. Individual results of CW per section are shown in Table 5.11. Data collected with IDOT-FWD attempted to collect CW in sections 8 and 9, but unfortunately CW could only be obtained in a small fraction of the total number of tested cracks due to high testing temperatures. Sections 8 and 9 also had induced cracks that were believed to have smaller, average crack widths than Lane 1 sections. Testing with the FWD on April 25 intended to capture CW along the unloaded edge of the pavement in sections 1 and 2, but a limited number of the cracks had open cracks. The best set of data to study crack width variability corresponds to the 31 cracks measured with the ATLAS (shown in Figure 5.32 as dataset 3) due to the large data set at one temperature condition. The only limitation of these measurements is that they were obtained from the loaded edge of the pavement which exhibited punchout damage.

### Table 5.10. Summary of crack width measured for variability study

<table>
<thead>
<tr>
<th>Load source</th>
<th>Section</th>
<th>Date &amp; time</th>
<th>Tavg (°F)</th>
<th>Tdiff (°F)</th>
<th>Nr. of cracks tested</th>
<th>Nr. Of cracks w/ measured CW</th>
<th>CW-avg (μm)</th>
<th>CW-stdev (μm)</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>ATLAS</td>
<td>3 (1)</td>
<td>2/15/05 4:07 PM</td>
<td>44.1</td>
<td>7.8</td>
<td>8</td>
<td>7</td>
<td>41.1</td>
<td>9.92</td>
<td>24%</td>
</tr>
<tr>
<td></td>
<td>3 (2)</td>
<td>3/03/05 2:29 PM</td>
<td>35.4</td>
<td>5.7</td>
<td>16</td>
<td>16</td>
<td>51.8</td>
<td>24.14</td>
<td>47%</td>
</tr>
<tr>
<td></td>
<td>3 (3)</td>
<td>5/05/05 4:52 AM</td>
<td>49.1</td>
<td>0.9</td>
<td>31</td>
<td>31</td>
<td>59.1</td>
<td>11.6</td>
<td>19.6%</td>
</tr>
<tr>
<td>IDOT-</td>
<td>8</td>
<td>4/16/05 10:30 AM</td>
<td>61.0</td>
<td>18.4</td>
<td>22</td>
<td>3</td>
<td>9.2</td>
<td>3.6</td>
<td>39%</td>
</tr>
<tr>
<td>FWD</td>
<td>9</td>
<td>4/16/05 12:30 PM</td>
<td>69.0</td>
<td>22.8</td>
<td>15</td>
<td>2</td>
<td>7.0</td>
<td>2.6</td>
<td>37%</td>
</tr>
<tr>
<td>FWD</td>
<td>1</td>
<td>4/25/05 8:17 PM</td>
<td>61.5</td>
<td>0.1</td>
<td>7</td>
<td>5</td>
<td>7.4</td>
<td>1.8</td>
<td>25%</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>4/25/05 7:06 PM</td>
<td>63.5</td>
<td>4.4</td>
<td>12</td>
<td>5</td>
<td>7.8</td>
<td>1.8</td>
<td>23%</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>4/25/05 10:04 PM</td>
<td>57.3</td>
<td>-7.3</td>
<td>6</td>
<td>4</td>
<td>16.9</td>
<td>6.3</td>
<td>37%</td>
</tr>
</tbody>
</table>
One challenge for FWD testing is determining the approximate crack closing temperature or zero-width temperature for a section. This temperature is important because $T_{avg}$ must be less than this value in order to successfully obtain crack width data. Furthermore, the temperature cannot be too low since the FWD can only close the cracks approximately 30 to 40 microns at 200 kN. In order to determine this temperature, preliminary testing must be completed on the section to determine the crack closing rates (microns/kN – see Figure 5.34) to resolve if the
cracks are open or closed. An estimate can also be made of the closing temperature based on the relationship between the zero-stress and zero-width temperature calculation (Kohler 2005). In order to determine with confidence the change in crack closing rate with the LST method, at least 5 or 6 load levels should be used with the FWD.

A crack width distribution plot was prepared (Figure 5.38) based on the ATLAS dataset 3, as presented in Table 5.11. Figure 5.39 presents all datasets shown in Table 5.11. From dataset 3 on section 3 (naturally-developing transverse cracks), the 31 measured cracks had an average CW of 59.6 microns, a standard deviation of 11.6 microns and a coefficient of variation of 19.6 percent. For the rest of the CW test data the coefficient of variation ranged from 23 to 47 percent.

Figure 5.38. Crack width distribution from ATLAS testing section 3 dataset 3
Crack width distribution from the Figure 5.39(c) dataset, involving 31 measurements, was compared to the normal distribution. The Kolmogorov–Smirnov test was used and indicated that a normal distribution cannot be rejected at a 0.05 level of significance. Figure 5.40 shows a normal distribution plot of the predicted crack width distribution versus the actual dataset from Figure 5.39(c). In order to improve the predicted distribution of crack width on a test section, the
following Weibull distribution proposed originally by Selezneva et al (2003) for crack spacing
distribution was utilized.

\[
P(CW_U \geq CW \geq CW_L) = 100 \left[ e^{-\left(\frac{CW_L - CW_{MIN}}{\alpha}\right)^{\beta}} - e^{-\left(\frac{CW_U - CW_{MIN}}{\alpha}\right)^{\beta}} \right]
\]

(1)

where:

\[
P(CW_U \geq CW \geq CW_L) = \text{ probability of crack width in interval}
\]

\[
CW_U \geq CW \geq CW_L \text{ (%)}
\]

\[
CW_U = \text{ upper limit of crack width interval (microns)}
\]

\[
CW_L = \text{ lower limit of crack width interval (microns)}
\]

\[
CW_{MIN} = \text{ minimum crack width (microns)}
\]

\[
\alpha = \text{ regression shape parameter}
\]

\[
\beta = \text{ regression shape parameter}
\]

\[
\alpha = \frac{CW - CW_{MIN}}{\Gamma \left(1 + \frac{1}{\beta}\right)}
\]

(2)

where:

\[
\bar{CW} = \text{ mean crack width (microns)}
\]

\[
\Gamma(1+\beta^{-1}) = \text{ regression shape parameter}
\]

The shape parameters \(\beta\) and \(\Gamma\) are determined from calibrated formulas presented by Selezneva et al (2003). Since the Weibull distribution was found to represent crack spacing distribution, these same shape parameter formulations were used to predict the crack width distribution. Figure 5.40 shows the Weibull distribution is a better match to measured data relative to the normal distribution. Further crack width datasets are required to validate that the calibrated
Weibull shape parameters proposed by Selezneva can be confidently used for crack width distribution prediction.

![Graph showing predicted and actual distribution of crack width for CRCP test section 3.](image)

Figure 5.40. Predicted and actual distribution of crack width for CRCP test section 3.

Crack width variability is a function of the temperature state of the pavement and decreases as the cracks close. This trend is presented in Figure 5.41, although caution should be exercised in reading this plot as the sample size is not the same for all the points shown and temperature difference affects this trend also. Given the effect of pavement temperature on crack width variability, it is suggested here that the statistical distribution of crack width in a homogeneous section of pavement be determined or calculated at the standard temperature of 32°F with zero temperature differential.
5.8. Summary of Chapter 5

Crack width is recognized as one of the most influential parameters that affects the performance of continuously reinforced concrete pavements. This is due to the importance that load transfer capacity has on the level of stresses experienced by the slab. Tight cracks imply good load transfer, and therefore reduced stresses and lower rates of distress development. On the other hand, wide cracks represent a serious problem to the pavement structure because of insufficient load transfer capacity, and water and oxygen infiltration that can affect the pavement support layers and cause reinforcement corrosion. Despite the importance of crack width in CRCP, there is scarce literature and field data dealing with this subject.

This chapter presented data on the vertical crack width profile through the depth of the concrete. Visual observations at the edge of the slab and in pavement cores pointed to a profile where the crack is more open near the surface and has its tightest point at the bottom or at the level of the reinforcement. The measurements of crack width at different depths corroborated the visual observations, and added valuable information with respect to the variations of the profiles as the pavement was subjected to temperature changes and traffic loading. Temperature can fully close...
the cracks or make them stay open, as it was shown with data from sections evaluated in different seasons.

The results of crack width obtained under different thermal conditions over a period of two years made it necessary to find a way to standardize the CW data. The M-E PDG has a procedure that includes a comprehensive crack width model for CRCP, and it was tested against measured crack width data. Since crack spacing is part of the crack width formula, a comparison of this variable was made and showed that the model predicted similar average crack spacing to that observed in the sections. It was found that the model over predicted crack width by an overall factor of six. Individual calibrations were performed for each crack, with the only objective to use the calibrated formulas to predict crack width under certain standard temperature conditions. The standard conditions were defined as 32°F interior temperature with zero temperature differential. Under these standard conditions the crack width in the individual cracks of each pavement section ranged from 0.031 to 0.116mm.

The M-E PDG formula predicts crack width only at the depth of the steel. The formula was analyzed and reformulated to use it to predict crack width at any depth within the slab especially at the slab surface. A sensitivity analysis was performed assuming two different humidity profiles, two average temperatures, and two temperature differentials. The results indicated high sensitivity of the formula to the relative humidity in the concrete.

The M-E PDG formula could not directly predict short-term variations in crack width, when using hourly inputs since it was derived originally for a monthly CW prediction. An hourly predictive model based on the M-E PDG was first developed which only utilized the measured temperature and temperature differences in the CW prediction. The actual variation in crack width was greater than the predicted variation. Hourly relative humidity changes were assessed based on field RH data. It was found that relative humidity increases with increasing temperature when the measurements are taken inside the concrete, while the opposite occurs near the surface (humidity behaves inverse to temperature, as in the ambient). A second approach to improve the
model was also investigated, and consisted of adding a term proportional to the temperature
differential, Tdiff, into the CW formula. The results of both strategies are effective for these test
sections but cannot be considered generally applicable since they were empirically implemented.

The falling weight deflectometer has been validated as a nondestructive measuring device to
determine surface crack width on CRCP. The procedure requires application of as many load
levels as practically possible, with five load levels minimum, and the capability to record
horizontal crack movement in the range of expected width, (up to 500 microns) with
approximately one micron resolution.

The best available set of data from this study consisted of crack width measurements on 31
cracks in one section with the average pavement temperature of 49.1°F and temperature
differential of 0.9°F. In this case, the average crack width was 59 microns with a standard
deviation of 11.6 microns. The distribution of crack width could be described by a Weibull
distribution similar to past work completed for crack spacing distribution by Selezneva (2003).
The Weibull distribution shape parameters used for previous crack spacing prediction were
successfully used to predict the distribution of crack width.

The zero stress temperature ($T_{zs}$) was estimated from two existing methods presented in the
literature. The average crack closing temperature for each section was estimated based on the
measured CW data. The crack closing temperature was lower than the estimated $T_{zs}$ from the
existing models. As the pavement section matures, $T_{zs}$ and crack closing temperature diverge
due to the fact that additional cracks and debris result in an apparent expansion in the fixed
length CRCP system which reduces the temperature needed to close the cracks.
CHAPTER 6  PERFORMANCE OF CRCP WITH SMALL CRACK WIDTH

6.1. Introduction

This chapter presents the results of accelerated loading on five full-scale CRCP test sections. In addition to the crack width determination effect described in Chapters 4 and 5, four of the five test sections were trafficked with loads that could result in significant fatigue damage levels to the pavement. Loading in sections 1, 2, and 3 was terminated when the pavement exhibited extensive cracking levels. Section 4 was loaded with an equivalent number of load repetitions and load levels, but no new cracks developed. Section 5 was only loaded to collect response data. This chapter documents and explains CRCP transverse cracking characteristics for each test section, presents the measured pavement responses from the accelerated loading tests, and describes the observed mechanism of failure on the sections relative to current understanding and models. A detailed description of the failure of each individual section can be found in Appendix A.

6.2. Transverse cracking

The design characteristics of each section, coupled with some construction factors, created differences in the transverse cracking from one section to another. However, crack spacing, crack width, and other aspects of the cracking pattern found in the experimental sections are similar to what could be expected from CRCP built to withstand real traffic conditions.

6.2.1 Crack progression

Regular crack surveys were performed to evaluate crack progression. The first crack surveys were carried out every few days after construction, and then monthly for the first 18 months. The location of transverse cracks and their progression over time is shown in Figure 6.1. The last survey, performed 30 months after construction, includes data not recorded before in sections 1 and 5. Section 1 was not originally included because it was being subjected to accelerated load testing first, and in section 5 no cracks were initially found, although closer examination revealed
later that a couple of existing cracks had not been detected probably because of transverse surface tining. The first transverse cracks observed were in section 3, while cracks were not seen in sections 2 and 4 until after the first month. All cracks propagated either during the first or second winter, when the concrete was contracting relative to the steel. No new cracks were developed on the surface during the warmer months of the year (April-October).

The majority of the cracks could be easily identified. However, some cracks initiated at one or both sides of the edges but could not be observed in the interior of the slab. Deciding the number of cracks in such cases is somewhat arbitrary and different approaches would give slightly different results. The method used counted only cracks that reached the edge. Cracks that extended for less than one foot were not counted, as well as those that merged with another crack within 4 feet from the edge.

6.2.2 Crack spacing

A stable pattern of transverse cracks was obtained after 18 months. A statistical summary and detailed crack spacing information is presented in Table 6.1. The average crack spacing was calculated using the concrete segments between the first and the last crack on each section, therefore discarding the space at the ends of the section. An alternative calculation method for
the average spacing is dividing the total length of the section, 85 feet, by the total number of segments (number of cracks + 1), and is presented in the summary as “entire section average”. The first method was selected for the analysis because of the limited length of the sections.

Table 6.1. Crack spacing summary and detail for all sections, in feet

<table>
<thead>
<tr>
<th>Summary</th>
<th>S1</th>
<th>s2</th>
<th>s3</th>
<th>s4</th>
<th>s5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of cracks</td>
<td>15</td>
<td>27</td>
<td>33</td>
<td>15</td>
<td>4</td>
</tr>
<tr>
<td>Average CS</td>
<td>4.64</td>
<td>3.00</td>
<td>2.58</td>
<td>4.78</td>
<td>20.67</td>
</tr>
<tr>
<td>Min</td>
<td>0.9</td>
<td>0.9</td>
<td>0.8</td>
<td>0.7</td>
<td>4.4</td>
</tr>
<tr>
<td>Max</td>
<td>26.1</td>
<td>9.0</td>
<td>5.6</td>
<td>14.2</td>
<td>34.4</td>
</tr>
<tr>
<td>STDV</td>
<td>6.77</td>
<td>2.29</td>
<td>1.20</td>
<td>4.31</td>
<td>15.16</td>
</tr>
<tr>
<td>Entire Section Avg.</td>
<td>5.31</td>
<td>3.04</td>
<td>2.50</td>
<td>5.31</td>
<td>17.00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Crack location (loc.) and crack spacing (CS) in feet</th>
<th>loc. CS</th>
<th>loc. CS</th>
<th>Loc. CS</th>
<th>loc. CS</th>
<th>Loc. CS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 -</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>19.9 - 4.1</td>
<td>2.5</td>
<td>1.5</td>
<td>1.0</td>
<td>1.3</td>
<td>15.4</td>
</tr>
<tr>
<td>24.0 - 26.1</td>
<td>4.0</td>
<td>2.7</td>
<td>2.3</td>
<td>3.4</td>
<td>23.0</td>
</tr>
<tr>
<td>50.1 - 1.3</td>
<td>6.7</td>
<td>1.2</td>
<td>5.7</td>
<td>1.4</td>
<td>24.1</td>
</tr>
<tr>
<td>51.4 - 6.1</td>
<td>7.9</td>
<td>1.3</td>
<td>7.1</td>
<td>1.4</td>
<td>25.5</td>
</tr>
<tr>
<td>57.5 - 4.4</td>
<td>9.2</td>
<td>1.8</td>
<td>8.5</td>
<td>1.2</td>
<td>26.2</td>
</tr>
<tr>
<td>61.9 - 10.9</td>
<td>11</td>
<td>1.3</td>
<td>9.7</td>
<td>1.8</td>
<td>28.8</td>
</tr>
<tr>
<td>72.8 - 0.9</td>
<td>12.3</td>
<td>2.2</td>
<td>11.5</td>
<td>0.8</td>
<td>32.2</td>
</tr>
<tr>
<td>73.7 - 1.2</td>
<td>14.5</td>
<td>1.4</td>
<td>12.3</td>
<td>1.7</td>
<td>33.0</td>
</tr>
<tr>
<td>74.9 - 3.2</td>
<td>15.9</td>
<td>5.3</td>
<td>14</td>
<td>2.6</td>
<td>44.0</td>
</tr>
<tr>
<td>78.1 - 2.4</td>
<td>21.2</td>
<td>3.3</td>
<td>16.6</td>
<td>3</td>
<td>48.9</td>
</tr>
<tr>
<td>80.5 - 1.4</td>
<td>24.5</td>
<td>2.9</td>
<td>19.6</td>
<td>3.9</td>
<td>63.1</td>
</tr>
<tr>
<td>81.9 - 1.2</td>
<td>27.4</td>
<td>1.5</td>
<td>23.5</td>
<td>4.1</td>
<td>66.1</td>
</tr>
<tr>
<td>83.1 - 0.9</td>
<td>28.9</td>
<td>2.3</td>
<td>27.6</td>
<td>2.4</td>
<td>71.4</td>
</tr>
<tr>
<td>84.0 - 0.9</td>
<td>31.2</td>
<td>2.8</td>
<td>30</td>
<td>1.5</td>
<td>72.5</td>
</tr>
<tr>
<td>84.9 -                                                                                   31.5</td>
<td>3</td>
<td>82.3</td>
<td>-</td>
<td></td>
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</tr>
<tr>
<td>85</td>
<td>35.0</td>
<td>5.8</td>
<td>34.5</td>
<td>1.1</td>
<td>85</td>
</tr>
<tr>
<td>40.8 - 2.5</td>
<td>35.6</td>
<td>4.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>43.3 - 2.7</td>
<td>39.8</td>
<td>3.7</td>
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<tr>
<td>46.0 - 0.9</td>
<td>43.5</td>
<td>1.7</td>
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<tr>
<td>46.9 - 1.1</td>
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<td></td>
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<tr>
<td>48.0 - 9</td>
<td>47.3</td>
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<td></td>
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</tr>
<tr>
<td>57.0 - 2.3</td>
<td>49.8</td>
<td>2.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>59.3 - 8.8</td>
<td>52.2</td>
<td>4.9</td>
<td></td>
<td></td>
<td></td>
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<td>68.1 - 2.9</td>
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<td>2.7</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>71.0 - 7</td>
<td>59.8</td>
<td>4.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>78.0 - 2.4</td>
<td>64</td>
<td>5.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80.4 -                                                                                   69.6</td>
<td>1.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>85</td>
<td>71.5</td>
<td>2.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>74.2</td>
<td>3.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>77.5</td>
<td>1.7</td>
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<td>79.2</td>
<td>2.8</td>
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<td>82</td>
<td>1.5</td>
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<tr>
<td>83.5</td>
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<td></td>
</tr>
<tr>
<td>85</td>
<td></td>
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</tbody>
</table>
For the discussion that follows, it is necessary to keep in mind the design parameters of the various sections. Sections 1, 2 and 3 are all 10-inch slabs, with the reinforcement located 3.5 inches below the surface (top of steel), and steel contents of 0.55, 0.80, and 1.09 percent. Sections 4 and 5 are 14-inch slabs, with 0.78 percent of reinforcement, placed in one layer (4.5 inches) and two layers (3.5 and 7.0 inches), respectively. All the sections contain the same number of longitudinal steel bars, but the diameter was changed, such that sections 1 and 2 have each #5 and #6 bars, and sections 3 to 5 have #7 bars. There are transition zones between sections (10 feet long), and anchorage lugs were used at both ends of the CRCP to minimize movements.

The highest number of cracks occurred in section 3, which is in the middle of the lane and has the greatest steel content. The average crack spacing is 2.58 feet. In section 2, the average crack spacing is 3.00 feet with the only design difference between them being the steel content. The additional steel in section 3 with respect to section 2 resulted in a reduction of crack spacing. Section 1 has even less steel than section 2, and the average crack spacing raised to 4.64 feet. Some movement may have occurred at the lug end of section 1, which had the effect of increasing the average spacing. It is not clear whether 4.64 feet corresponds truly to the approximate crack spacing of the section 1 design, but it is reasonable to expect that had section 1 been at an interior location, the spacing would be between 3.00 and 4.64 (larger than section 2 and shorter than observed). Crack spacings between 3.0 and 5.0 feet are generally considered appropriate for good performing CRCP, and therefore sections 1 to 3 can represent desirable field sections.

Section 4 is four inches thicker and has approximately the same steel content as section 2, although placed at 4.5 instead of 3.5 inches from the surface. Section 4 has half the number of cracks as section 2, and therefore it can be concluded that the two effects, thicker slab and deeper steel, combined to generate a spacing of 4.78 feet in section 4 that is considerably higher than the 3.00 feet in section 2. Existing field data and models show an increase in thickness and deeper reinforcement result in larger crack spacing. Section 5 is difficult to assess since only four cracks developed over the entire pavement length. It can be attributed to possible movement at
the end lug, but it can also be the result of the double layer of steel effect. Prediction of crack spacing with the double layer of steel is uncertain with existing models. The presence of reinforcement at only 3.5 inches from the top would reduce the spacing compared to section 4, while the fact that only one half of the total steel is at 3.5 inches (and the rest at mid-depth) would increase the spacing.

Crack spacing in sections 1 to 4 can be compared with the findings of Selezneva et al (2003) which come from CRCP sections surveyed during the Long-Term Pavement Performance (LTPP) program:

- Mean crack spacing in the LTPP sections varies from 1.0 to 7.5 feet. Individual crack spacing ranges from 0.25 to 10 feet.
- The ratio between the standard deviation of crack spacing and the average spacing was found to be roughly constant among the sections of the study and about 0.56, implying higher variability in sections with greater crack spacing.
- The vast majority of punchouts (90 percent) develop on CRCP panels that have transverse cracks spaced from 1 to 2 feet.
- Transverse crack spacing distribution is not normal but skewed with a long right tail, and can be approximated by a Weibull distribution model.

Assuming that the 47 LTPP sections represent “typical” CRC pavements, it can be concluded that mean crack spacing in the experimental sections 1, 2, 3 and 4 are within the expected range. The coefficient of variation in these four sections is 1.44, 0.76, 0.47, and 0.90. This indicator was obtained with sample sizes (number of segments between cracks along each section) of 14, 26, 32, and 14, respectively. The number of segments in each section contained in the range of 1 to 2 feet in length is 7, 10, 14, and 5, which represents between 33 and 47 percent of segments that could be punchout candidates. Crack spacing was not statistically checked to see if it follows the Weibull distribution model suggested by Selezneva et al (2003), but the observed and predicted curves are shown in Figure 6.2.
In summary, the crack spacing in all sections (except section 5) can be considered within a typical range, with no indication of higher or lower propensity for punchout development than traditional CRCP sections. The effect of the design parameters such as steel content, steel depth, and thickness, reflected known trends. The effect of double layer steel on crack spacing unfortunately could not be assessed due to the small number of transverse cracks in section 5.

### 6.2.3 Crack width

As mentioned in chapter 5, crack width cannot be compared from one section to another based only on the direct measurements because of the different temperature conditions in which each section was evaluated. Using calibrated formulas for each crack based on the M-E PDG model, it was possible to convert the measured crack width to a standard temperature. Table 6.2 presents the average crack width for each section based on several independent cracks under several temperature conditions. Table 6.3 presents the crack width corrected for temperature and at the
depth of the steel using the locally calibrated M-E PDG CW model for each individual crack, as was explained in section 5.3.4.

Table 6.2. Crack width measurements and respective temperature

<table>
<thead>
<tr>
<th>Section</th>
<th>CW</th>
<th>Approx.Pavement temperature (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.063</td>
<td>41</td>
</tr>
<tr>
<td>2</td>
<td>NA</td>
<td>85</td>
</tr>
<tr>
<td>3</td>
<td>0.011</td>
<td>72</td>
</tr>
<tr>
<td>4</td>
<td>0.033</td>
<td>36</td>
</tr>
<tr>
<td>5</td>
<td>0.012</td>
<td>54</td>
</tr>
</tbody>
</table>

Table 6.3. Standard crack width at fixed temperature

\((T_{avg} = 32\,\text{oF} \text{ and } T_{diff} = 0\,\text{oF})\)

<table>
<thead>
<tr>
<th>Section</th>
<th>CW at depth of steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.116</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>0.064</td>
</tr>
<tr>
<td>4</td>
<td>0.031</td>
</tr>
<tr>
<td>5</td>
<td>0.081 (^{(1)})</td>
</tr>
</tbody>
</table>

\(^{(1)}\) depth corresponds to the average depth of the two layers of steel

Average crack width in section 3 is about one half of crack width in section 1, both at 3.5 inches below the surface. Reinforcement steel in section 4 is at 4.5 inches from the surface and therefore it can’t be directly compared to sections 1 and 3. Using the modified M-E PDG CW formula, section 4 crack width was calculated at 3.5 inches and resulted in approximately the same value of 0.031 mm. This means that cracks in section 4 are on average one half the widths of cracks in section 3. The measured crack width in section 5 is reported at 5.25 inches from the surface since this is the average depth of the two layers of steel. When interpolated to 3.5 inches, crack width in section 5 is 0.148 mm, although this is based on a small number of transverse cracks in the section.

Increase in percent of reinforcement from section 1 (0.55 percent) to section 3 (1.09 percent) effectively reduced crack width. Section 4 was expected to present wider cracks than section 3 for three reasons: (1) it has less reinforcement, 0.78 percent versus 1.09; (2) longer crack
spacing; and (3) the reinforcement is deeper in the slab. However, crack width in section 4 was smaller and the reason is not totally clear. Each section’s average crack width comes from measurements in only four cracks. Sampling more cracks per section would lead to more accurate results, as was recommended in Chapter 5.

6.3. Longitudinal crack development

In order to fail the CRCP test sections under an accelerated time schedule, a significant number of load repetitions at high load levels had to be applied to the sections. This translated into longitudinal cracking in three of the five pavement sections. Section 4 did not develop longitudinal cracking even though the loading scheme was similar to sections 1, 2 and 3. Loading in section 5 was completed only to collect response data since section 4 could not be failed and was the same thickness as section 5.

The longitudinal cracks initiated 4 to 5 feet from the loaded edge at transverse cracks and eventually propagated toward the pavement edge with more load applications. At the end of loading, they resembled half-moon cracks. The places where the longitudinal crack extended to the edge coincided with longer panels. The presence of other closely spaced transverse cracks allowed the crack propagation to continue advancing longitudinally. The half-moon cracks had the potential to develop faulting and the enclosed area was considered a punchout. A cascade effect created failures associated with the original punchouts, affecting a considerable length of the sections. The length of each punchout was between 10 and 40 feet. Secondary longitudinal cracks formed closer to the edge in sections 1 and 2. Figure 6.3 presents crack maps of all sections after the accelerated loading was finished.
Figure 6.3. Crack maps at end of ATLAS loading
Figure 6.4 shows a picture of the half-moon cracks in section 1 (visibility of cracks was enhanced for the picture by spraying water over them).

Figure 6.4. Photograph of half-moon cracks in section 1

6.4. Load levels and total traffic loading

Total loading in most sections was relatively similar and consisted of thousands of wheel passes with load level varying between 10 and 55 kips. When converted into equivalent single axle loads (ESALs), the total loading in sections 1 to 4 was between 627 and 911 million ESALs. ESALs were only needed to give an approximation of the damage applied to each section, but for a mechanistic analysis ESALs are not required. The load sequence started with load levels similar to current half-axle loads of heavy trucks on highways, 10 kips. After initial passes at
that load level, which accounted for about the first 10,000 repetitions, the load was increased to damaging levels of 30 or 35 kips. Higher loads were applied toward the end of the testing. The history of load level versus pass number is shown for each section in Figure 6.5. The loading was applied for several hundreds passes at a time, and then a visual pavement inspection was performed.

![Figure 6.5. ATLAS Load history in each section](image)

The load testing ended in sections 1, 2, and 3 when the pavement had extensive punchout failures. In section 4, the loading ended when the amount of total traffic had reached a similar level to the other sections, without developing a punchout failure. Section 5 was not subjected to damaging loads due to mechanical limitations with the ATLAS, the reduced number of
transverse cracks that existed on the section, and performance observed in section 4. Only 12.5 millions ESALs were applied on section 5. A summary of the tested sections is presented in Table 6.4.

Table 6.4. Summary of tested sections

<table>
<thead>
<tr>
<th></th>
<th>Section one</th>
<th>Section two</th>
<th>Section three</th>
<th>Section four</th>
<th>Section five</th>
</tr>
</thead>
<tbody>
<tr>
<td>Duration of testing&lt;sup&gt;(1)&lt;/sup&gt;</td>
<td>12 months (6/16/02-6/23/03)</td>
<td>11 weeks (6/30/03-9/13/03)</td>
<td>10 weeks (5/26/04-8/4/04)</td>
<td>9 weeks (1/3/04-3/6/04)</td>
<td>2 weeks (4/6/04-4/15/04)</td>
</tr>
<tr>
<td>Total load repetitions</td>
<td>246,800</td>
<td>118,600</td>
<td>163,400</td>
<td>64,300</td>
<td>1,800</td>
</tr>
<tr>
<td>Total ESALs</td>
<td>911 M</td>
<td>778 M</td>
<td>627 M</td>
<td>764 M</td>
<td>12.5 M</td>
</tr>
<tr>
<td>Approx ESALs at first failure</td>
<td>511 M</td>
<td>230 M</td>
<td>548 M</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Maximum load applied</td>
<td>50 Kips</td>
<td>50 Kips</td>
<td>55 Kips</td>
<td>55 Kips</td>
<td>35 Kips&lt;sup&gt;(3)&lt;/sup&gt;</td>
</tr>
<tr>
<td>Pavement temperature range</td>
<td>34-80 °F&lt;sup&gt;(2)&lt;/sup&gt;</td>
<td>75-95°F</td>
<td>64-80°F</td>
<td>25-50°F</td>
<td>40-65°F</td>
</tr>
<tr>
<td>Failure description</td>
<td>Extended punchouts</td>
<td>Extended punchouts</td>
<td>Extended punchouts</td>
<td>Section did not fail</td>
<td>Response loading only</td>
</tr>
</tbody>
</table>

<sup>(1)</sup> Includes time when no load was being applied.

<sup>(2)</sup> Most of the effective test was done during June 2003, when temperature was 60-80 °F

<sup>(3)</sup> Loads up to 55 kips were used, but less than 10 passes were applied at each load level higher than 35 kips

The wheel load applications were converted into Equivalent Single Axle Loads (ESALs) by using load equivalency factors. The load repetitions with the ATLAS were applied with a single aircraft wheel, which correspond to half an axle, therefore the equivalent axle load corresponds to twice the specified ATLAS load. An equivalent axle load factor (EALF) defines the damage per pass of an axle of any load relative to the damage to a pavement per pass of a standard axle, usually the 18-kip single-axle load. The most common method to determine EALF is based on results of the AASHO Road Test. For rigid pavements, the factors depend on several parameters such as pavement thickness and terminal serviceability. For this research, an approximate EALF was used according to equation 6.1, where Lx is the single axle load.
\[ EALF = \left( \frac{Lx}{18} \right)^{4.3} \] [6.1]

Figure 6.6 presents the equivalency factors calculated for a 10-inch pavement, and the fitted exponential approximation. Note, the maximum legal load for single axles in Illinois is 20 kips (max gross weight 80 kips for vehicles with 5 or more axles) per Illinois Vehicle Code 625 ILCS sec 5/15-111.

![Figure 6.6](image)

Figure 6.6. AASHTO Equivalent Axle Load Factor for rigid pavement and exponential approximation

Another factor was applied to account for channelized trafficking of the ATLAS wheel at the pavement edge. Results from the PCA design method (PCA 1984) established that when 6 percent of the heavy traffic is driven at the edge, it causes an equal amount of damage to 100 percent of the traffic distributed in the wheel path (for the cases when the edge has no tied shoulder). The equivalent damage ratio (EDR), developed as part of the IDOT Mechanistic-Empirical Design Procedure for JPCP (Zollinger and Barenberg 1989), has the same concept as the PCA edge damage factor. For a 10-inch slab, the EDR for a JPCP is 0.05 or 20 times worse damage at the edge versus loading in the wheel path.
Wheel loads of 50 kips (which would correspond to a single axle load of 100 kips) are unrealistic for highway vehicles and the channelized loading at the edge deviates from actual vehicle lateral distributions. However, the use of ESALs allowed for a simple, direct comparison of total traffic loading on the various pavement sections. Although the total number of ESALs is very high compared to known highway traffic, the procedure employed is considered valid as the best tool to compare between sections.

6.5. Typical pavement responses to traffic loading

6.5.1 Data collection, storage and processing
Signals coming from the sensors were collected in a synchronized manner with the passage of the loaded wheel. For every pass of the wheel a complete set of responses was scanned from the sensors in order to determine maximum, minimum and unloaded responses. This scanning was performed every one inch along the section. The unloaded value corresponds to the sensor reading taken at the beginning of the pass, before the load had been applied. This allows for the determination of the rebound response and permanent deformation values. Rebound values are defined as the difference between maximum responses (maximum or minimum) and the unloaded value, and represent the effect of the load during one pass. The maximum, minimum and unloaded values from all sensors were saved for each pass. A time-history response was recorded for each sensor, as the wheel rolled over the section, every 10 to 20 passes. For a more detailed review of each section responses see Appendix A.

6.5.2 Vertical deflections
Vertical sensors recorded the deflection of the slab under loading. Figure 6.7 presents a typical plot of rebound deflection measured over the entire testing period of section 3. Four cracks were instrumented in each section, except in section 5. Daily temperature cycles affected the deflection because of slab curling that affected the support conditions at the slab edge. The three load levels employed in this section are presented. Higher deflections were obtained with increased load levels and load repetitions.
6.5.3 Transverse strain

Strains were measured close to the pavement surface (1-inch), at an approximate distance from the edge (4.5 ft.) such that the highest tensile strains in the transverse direction were captured. To account for the maximum strain, which is experienced at the surface, the measured strain is amplified by a factor of 5/4 or 7/6 based on the distance from the neutral axis to the strain gage and the distance to the surface. To estimate the approximate stress, the results (in strains) have to be multiplied by the elastic modulus of concrete, which is about 7 million psi. Four strain gages were embedded along each section, but since the testing spanned over two years, not all of them were operable by the time the section was loaded. In section 1 the four sensors could be read, but only three sensors were usable in section 2 and section 4, and one sensor in sections 3 and 5. Figure 6.8 shows strains measured at one of the gages embedded in section 2.
6.5.4 Crack width

Crack opening was affected by changes in temperature, but more importantly by the deformations caused by the wheel loading. A higher vertical deflection caused more horizontal compression near the slab surface, and a wider opening at the bottom of the slab. The plots in Figure 6.9 to Figure 6.12 present typical opening and closing (positive and negative movement, respectively) at the instrumented depths in the slab: 1.0, 3.7, 6.3, and 9.0 inches below the surface. The data correspond to the crack located at station 14.9 in section 3. Horizontal crack movements were in all sections strongly affected by average pavement temperature and temperature differential. The major responses to load are crack closing near the surface and crack opening near the bottom of the slab.

Figure 6.8. Typical strain plot
Figure 6.9. Horizontal crack movement at a sensor located at top (z=1.0”)

Figure 6.10. Crack movement at a sensor located at mid-top (z=3.7”)
Figure 6.11. Crack movement at a sensor located at mid-bottom (z=6.3”)

Figure 6.12. Crack movement at a sensor located at bottom (z=9.0”)

Figure 6.13 shows crack movements at the crack at station 44.0 in section 4 and gives an example on how the increase in both the average pavement temperature and temperature differential affected the crack closing at the top and the opening at the bottom of the slab.
Load transfer efficiency (LTE) was calculated from measurements of vertical deflection at the transverse cracks. Two values of load transfer efficiency are obtained in each wheel pass, the LTE on the approach side and the LTE on the leave side. LTE calculated with a rolling wheel is different from the LTE calculated with load at a fixed position as in the case of falling weight deflectometer test. When loading is applied bi-directionally, there are four LTE values for each
crack. An algorithm was programmed to obtain LTE from the deflection on both sides of a crack, ensuring that calculation is performed at the instant when the load is only at one side of the crack. Figure 6.14 shows the average LTE values (approach and leave LTE, unidirectional) for each of the four instrumented cracks in section 3. LTE is higher than 95% in all the cracks as it was the case in most other sections (the exception was section 1). At this high LTE value, there was a small temperature effect, which caused LTE to oscillate on a daily basis.

![LTE graphs (14.9, 23.4, 47.3, 57.2)](image)

Figure 6.14. Load transfer efficiency at four cracks in section 3

### 6.6. Elastic responses and comparison between pavement sections

For every test section, pavement responses under the action of the ATLAS wheel load were recorded continuously at each sensor. Influence lines were collected for transverse strains and horizontal and vertical movements at the instrumented cracks. The effect of the moving wheel
load is represented by the rebound responses, defined as the peak value measured as the wheel passes over the sensor compared to the reading after the wheel has moved away from the sensor. These responses depend on the load level being applied and are also a function of the thermal state of the pavement. These responses typically vary along a uniform section due to variability in material and support conditions.

6.6.1 Effect of slab curling on elastic responses

The effect that the slab curling has on the pavement responses makes it difficult to compare strain and deflection measurements between sections that have been tested in different seasons. Figure 6.15 shows rebound vertical deflections measured continuously over a two-week period for a fixed load level of 30 kips. These measurements were obtained from section 3 during the summertime when the daily thermal cycles were more pronounced. Figure 6.15 also shows the temperature difference through the thickness of the slab, which fluctuated each day during this time of the year between -4 and 8°F approximately (note the temperature scale is inverted in the chart). The slab’s temperature differentials were not as large as expected due to the sheltering effect of the ATLAS and the encapsulating tent structure. Despite the fixed load level, the rebound deflections were not constant but varied with thermal conditions in the slab. Deflections in the morning could be 20 percent higher than in the late afternoon.

Transverse strain measurements for the same section and load level (30 kips) are presented in Figure 6.16. The plot shows more scatter as the strain gage signals were not as clear as the LVDT’s. Similar to the deflection measurements, the transverse bending strain, measured with embedded sensors at 4.5 feet from the edge, increased with decreasing temperature differential. For instance, a variation of 40 microstrains measured one inch below the slab surface can be observed within a few hours and is equivalent to approximately 350 psi of additional surface tensile stress in the early morning with respect to the values during the afternoon.
Similar behavior was observed in all pavement sections where the edge uplift caused by transverse curling increased the rebound deflection and the transverse strain near the top of the slab. An example of the vertical displacement measured at the edge of the slab due to daily temperature cycles, with no applied wheel load, is shown in Figure 6.17. The data comes from section 1 and it was collected during the month of September. In the early morning, when the temperature differential, $T_{\text{diff}}$, is about $-7^\circ\text{F}$, the edge is lifted more than 0.20 mm. By 3 p.m. the
edge has moved downward and remains down (possibly in full support) until $T_{\text{diff}}$ become lower than about 0°F. The edge is then lifted up again during the night hours. The movement follows different paths during the periods of heating and cooling of the pavement, in a hysteretic behavior caused by the non-linearity of the thermal profile. Comparable results have been reported for in-service undoweled concrete pavements (Poblete et al. 1988).

![Figure 6.17. Edge uplift due to daily changes in temperature differential](image)

6.6.2 Test section comparison based on elastic responses

The testing procedure followed in each section consisted of an initial loading at 45kN to obtain elastic responses over a period of 24 hours, followed by heavy loading to accelerate pavement damage. The average vertical deflection under 45kN loading for sections 1 through 5 is presented in Figure 6.18. Dispersion bars are included in the plot to account for the variability within the section (range), i.e., measurements at four different locations, and the aforementioned temperature effect. Typical temperatures ranges for $T_{\text{avg}}$ and $T_{\text{diff}}$ are also presented in the chart in the form of representative ranges during the hours of testing in each section.
Deflections measured in sections 1, 2, and 3 were approximately twice the deflections observed in the thicker slabs of sections 4 and 5. The additional reinforcement in section 3 (1.09 percent) with respect to sections 1 and 2 (0.55 and 0.80 percent respectively) did not significantly alter the elastic deflection measurements. At time of testing in section 2, the subgrade was saturated due to heavy rainfall, which could help to explain the higher deflections that this section experienced compared to sections 1 and 3. The 14-inch sections experienced similar deflections values to each other, irrespective of the presence of single or double steel layer.

6.7. Failure mechanism for CRCP under small crack width

The three sections that failed had extended punchouts and maintained high load transfer efficiency even after punchout failure had occurred.

6.7.1 Traditional assumption on formation of punchouts

The following is a list of field conditions that typically occur for a punchout to develop, according to the M-E PDG (ERES 2004), and they are the basis for their punchout prediction model:

- Presence of narrow transverse crack spacing (2 foot or less) in the crack spacing distribution.
- Loss of load transfer efficiency (LTE) across the transverse cracks due to aggregate interlock deterioration from excessive crack opening and heavy repeated loads.
- Loss of support along the pavement edge due to base erosion (void creation).
- Negative temperature gradients through the slab thickness along with drying shrinkage gradient, which further magnify tensile bending stresses.
- Passages of heavy axles causing repetitive cycles of excessive tensile bending stresses leading to longitudinal fatigue cracking that is defined as a punchout failure.

These stages of pavement deterioration that lead to longitudinal cracking and punchout are based on work by Zollinger (1989), who postulated that the fundamental cause of punchout distress is a loss of subbase support enhanced by reduction in pavement bending stiffness. The reduction in bending stiffness is caused by widened cracks and pullout fracture around the rebars. A schematic diagram of the formation of punchout distress, defined by Zollinger (1989), is shown in Figure 6.19.

![Diagram](image)

Figure 6.19. Formation of punchout distress (Zollinger 1989)

The prediction of punchouts in the M-E PDG includes the determination of a shear capacity loss due to aggregate wear-out. This assumes that as the slab is subjected to load applications the vertical crack surfaces are subjected to cycles of shear loading between the two sides that leads to aggregate wear-out and a resultant decrease in the crack’s load transfer capacity.
The M-E PDG distinguishes two cases with respect to the loss of shear capacity depending on the crack width. When the ratio between crack width and slab thickness is greater than 0.0038 (equivalent to 1 mm of CW in a 10-inch slab) the deterioration process occurs faster. How much faster depends on the level of stresses derived from traffic and the number of traffic repetitions. The formula for loss of shear capacity is presented in equations 6.2a and 6.2b. As a reference, the factor that multiplies stresses and traffic (first bracketed term in the equations) is shown in Figure 6.20 for slab thicknesses from 8 to 14 inches.

![Figure 6.20. Crack width related factor in shear capacity loss](image)

\[ \Delta s_i = \sum_j \left( \frac{0.005}{1+1 \cdot \left( \frac{cw_i}{h_{PCC}} \right)^{-5.7}} \right) \left( \frac{n_{ji}}{10^6} \left( \frac{\tau_j}{\tau_{ref,i}} \right) \right) ESR_i \text{ if } cw/h_{PCC} < 3.8 \]  

[6.2a]

\[ \Delta s_i = \sum_j \left( \frac{0.068}{1+6 \cdot \left( \frac{cw_i}{h_{PCC}} - 3 \right)^{-1.98}} \right) \left( \frac{n_{ji}}{10^6} \left( \frac{\tau_j}{\tau_{ref,i}} \right) \right) ESR_i \text{ if } cw/h_{PCC} > 3.8 \]  

[6.2b]

Where:
Δς_\text{i} = \text{loss in shear capacity during monthly increment } i \text{ due all load applications } j \\
cw_\text{i} = \text{crack width calculated for each monthly increment } i, \text{ (in units of mils)} \\
h_{\text{PCC}} = \text{slab thickness (in units of inch)} \\
n_{\text{ji}} = \text{Number of efficient axle load applications for monthly increment (}i\text{) and load level (}j\text{) (no traffic wander).} \\
τ_{\text{ij}} = \text{corner shear stress on the transverse crack due to load level (}j\text{) during monthly increment (}i\text{)} \\
τ_{\text{refi}} = \text{Reference shear stress derived from the PCA test results for monthly increment } i \\
ESR_\text{i} = \text{equivalent shear ratio that is an adjustment factor for lateral traffic wander.} \\

The plot shows that for crack width smaller than about 0.75mm there is no effect of thickness on loss of shear capacity and below 0.150mm width, the loss of shear capacity is insignificant. CW and LTE data from full-scale testing corroborated part of the model, since all crack widths were smaller than 0.15mm and shear capacity remained intact throughout loading.

6.7.2 Punchout formation in this study

Pavement in sections 1, 2, and 3 failed in similar manners. Based on the geometry of the pavement failures and from the results of the instrumentation, the punchout fracture occurred as a result of permanent deformation of the support layers and occurred rapidly without much change in the rebound deformation before the failure. This is similar to the failure mechanism in the M-E PDG where loss of support is the first stage in the development of a punchout failure.

6.7.2.1 Crack geometry at punchouts

The crack formation and settlement process can be explained using an example from section 1. Figure 6.21 offers a close up on the damaged pavement from station 53 to 86, with the cracks as seen from the surface as well as from the edge. Scale problems make difficult to appreciate the importance of cracks as seen from the edge, hence Figure 6.22 depicts edge cracks in more detail, covering only the zone from station 75 to 83, and its evolution over a period of a week of
loading. Figure 6.23 shows pictures of the cracks as seen from the top and from the edge of the pavement on one of the punchouts. Figure 6.24 shows the ends of the crack at another punchout. Cracks on the edge were only in the vertical direction before the beginning of failure. There are inclined cracks on the upper part of the slab, as a result of secondary compression failures above the rebar due to high slab deflections. When the half-moon cracks eventually propagate to the slab edge, they appear very inclined with depth.

6.7.2.2 Failure in terms of deflections

Maximum and unloaded deflections are measured during each wheel pass, and its difference is defined as rebound deflection. Theoretically, the unloaded deflection should remain unaltered if no damage occurs to the pavement, reflecting the fact that after each load application the slab returns to its initial position. The maximum deflection represents the vertical deformation of the pavement system when subjected to load, and it should theoretically remain constant without permanent deformation if no damage has occurred. This first deviates from theory when temperature curling is included in the analysis, because it induces slab deformations that vary the contact condition between the slab and the base layer. The second and most important deviation from theory results when permanent or plastic deformation is considered under application of heavy loads.
Figure 6.21. Top and edge view of cracking pattern from station 86 to 53 ft. Each grid line is one foot in length.

Figure 6.22. Crack deterioration along slab edge (on June 5 – top and June 13, 2003 - bottom)
Figure 6.23. Cracks and deformation at a punchout
The following analysis addresses the damage process in CRCP, neglecting temperature effects. The instrumentation tells what happens at the top of the slab, while the events in the underlying layers can only be inferred. Consider the following scenarios, represented in Figure 6.25.

i) **Elastic deformation in slab and supporting layers:** both maximum and unloaded deflections remain the same after each pass. Low load levels are being applied and no damage or permanent deformation is accumulating in the CRCP system.
ii) **Elastic deformation in the slab and permanent deformation in the subbase:** maximum deflections increase as more heavy-load repetitions are applied, but changes to the unloaded and rebound deflections are small because the slab returns to the original position after each load repetition. The increase in maximum deflection is related to the creation of permanent deformation or voids beneath the slab. The slab is progressively subjected to higher flexural stresses, and receives less support from the subbase to distribute the load.

iii) **Permanent deformation in both slab and subbase:** the flexural strength of the concrete slab is exceeded which progressively fractures and seats the concrete slab, with the unloaded and loaded deflection increasing during this stage along with a significant change in the rebound deflections.

![Diagram of vertical deformation at the edge of CRC pavement system]

**Figure 6.25.** Vertical deformation at the edge of CRC pavement system

To illustrate how the collected data supports the aforementioned mechanism, the results of section 1 punchout failures are presented. Figure 6.26 shows maximum and unloaded deflection from sensor D-24.4e during one loading round (40-kips), which consisted of 3,000 passes. Deflections increased suddenly at some point between passes 244,000 and 244,500.
Figure 6.26. Maximum and unload deflection, sensor D-24.4e

Figure 6.27 presents details of data in Figure 6.26 for a shorter span of passes, along with the rebound deflection. Unloaded deflection increased 0.5mm in less than 50 passes. This is the time frame when the concrete slab fractured. During this 50 passes the maximum deflection also increased, but less than the unloaded deflection. The rebound deflection was constant but actually dropped once the concrete fractured due to the elimination of the void beneath the slab.

The same behavior observed in sensor D-24.4e was observed in every sensor located inside a punchout area, which tells that identical mechanisms developed for all tested sections. This behavior indicated that there was void creation occurring under the slab. The voids formation is related to deformation in the asphalt and granular subbase layer and to subgrade compaction and erosion. In sections 1 and 2, visible pumping of fines occurred at the edge prior to punchout failure.
6.7.2.3 Permanent deformation at the slab

Pavement deformation was measured with a Dipstick® profiler on the loaded edge on section 1 as the loading progressed. The result is shown in Figure 6.28 along with a crack map of the section. The major punchouts occurred from station 65 to 82 ft., where the slab ended up almost 20 mm below the initial position at the site of peak deformation. The profiles also show how the rest of the section subsided under accelerated loading along the edge of the slab. An estimation of the permanent deformation on section 2 (original position of deflection sensor not recorded) was made based on unloaded deflection, and is presented in Figure 6.29. This plot supports the findings from section 1 that permanent deformation under the slab was the primary contributor to the punchout failure of the test sections with small crack widths and high shear capacity.
Figure 6.28. Longitudinal profiles at the loaded edge and crack map in section 1
6.8. **Comparison between observed failure and M-E PDG**

Between the traditional mechanism of CRC pavement failure utilized in the M-E PDG and the failure observed in the experimental sections of this study there is certain level of agreement and a clear difference. The points of agreement are:
- Loss of support has to occur for a punchout to happen
- Capacity to transfer load across transverse cracks does not decrease if crack width is smaller than 0.150 mm

The difference discovered in this experimental testing was that the longitudinal cracks started and developed with little decrease in load transfer efficiency across the transverse cracks. Traditionally the loss of LTE is cited as the cause of the increased stresses that generate the longitudinal cracks; however, it was found that under conditions of small crack width the longitudinal cracks start and develop with the shear capacity intact. The sequence of events that lead to faulted punchouts is shown in Figure 6.30. It is plausible that CRCP field sections may exhibit a loss in LTE with time. However, FWD testing of Illinois CRCP sections located on interstates generally shows high load transfer efficiencies (>85%) over time.

The high elastic deformations experienced at the slab edge near transverse cracks caused permanent deformation of the supporting layers and a subsequent void beneath the slab which eventually led to the longitudinal cracking (punchout) failure. The M-E PDG model will predict longer CRCP life if there is no loss of load transfer efficiency.
6.9. Evaluation of double layer steel reinforcement

6.9.1 Background

Two-layers of steel have been adopted as a standard for concrete slab thickness greater than 13 inches in Texas (Won 2004). Although Texas DOT has been using two layer steel reinforcement for 15 years, they have not done any comparison on the performance. Texas DOT believes two layers of steel should theoretically perform better than a single layer due to the higher steel bond area to concrete volume ratio, better consolidation is achieved, and first layer of steel is close to the surface minimizing crack widths.

Zollinger et al. (1999) found the number of layers of reinforcement, the vertical positioning of the steel, and curing program affected cluster cracking. The greater the steel depth from surface the less effect curing depth will have on cluster crack development. However, this will increase the surface crack width magnitudes. Two layers of transverse chairs are needed for two layers of longitudinal steel. Tang et al. (1996) reported that Texas DOT had found a high incidence of transverse cracking when the transverse chairs of the two layers coincide with each other. Two layer of steel are typically placed on top of each other allowing for better consolidation between the bars. No published information appears to be available on performance of CRCP with double layer of steel.

6.9.2 Load testing

Sections 4 and 5 are the 14-inch sections constructed as part of this project. They contain 0.78 percent reinforcing steel and both sections have #7 bars. Section 4 has a single layer at 4.5 inches below the surface, while section 5 has two layers of steel, one at 3.5 and the other at 7 inches. The two layers of longitudinal steel in section 5 were staggered, so that the spacing between rebars remained the same as with one layer. This spacing is 5.5 inches. Pictures of the reinforcement in section 5 are presented in Figure 6.31 and Figure 6.32. A picture of reinforcement in section 4 is shown in Figure 6.33.
Figure 6.31. Side view of double layer reinforcement in section 5

Figure 6.32. Angle view of double layer reinforcement in section 5
Heavy traffic loading was applied to sections 4 and 5 as part of the CRCP testing. Elastic responses were measured in both sections and can be compared to estimate the effect of the double layer steel. Responses at 10 and 35 kips will be compared in terms of rebound vertical deflections, transverse strain near the surface, and transverse crack openings. Four cracks were instrumented in section 4 and one crack in section 5.

The loading sequence in section 4 consisted of 10,000 initial passes at 10 kips, followed by 25,000 loading passes at 35 kips. Higher loads (45 and 55 kips) were applied later seeking failure of the pavement. Section 5 was only tested for elastic responses, with 50 initial passes at 10 kips and approximately 1,800 passes at 35 kips.

Another difference between the accelerated loading on sections 4 and 5 was the pavement temperature. Section 4 was tested mostly during February, while section 5 was tested in April. Average pavement temperature in section 4 went from around 32°F when testing at 10 kips, to 38°F when testing at 35 kips, and the differential was about -2°F and increased up to +7°F. Section 5 was tested at 40°F for those passes at 10 kips, and warmed up to 66°F during the days of testing at 35 kips, with differential temperature from 0 to +9°F.
6.9.2.1 Rebound vertical deflections:

Rebound deflections in section 4 and 5 were similar, both at 10 and at 35 kips load level as seen in Table 6.5.

<table>
<thead>
<tr>
<th>Section</th>
<th>Load 10 kips</th>
<th>35 kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.09-0.13</td>
<td>0.30-0.64</td>
</tr>
<tr>
<td>5</td>
<td>0.10-0.11</td>
<td>0.37-0.53</td>
</tr>
</tbody>
</table>

6.9.2.2 Transverse strain

Strain was measured at 1 inch below the surface and at 4.5 feet from the edge, at the location of the highest tensile strains in the transverse direction. Three gages were used in section 4 and one gage in section 5. Strain in section 5 was about one half of that in section 4. The average and min-max values are presented in Table 6.6.

<table>
<thead>
<tr>
<th>Section</th>
<th>Load</th>
<th>10 kips</th>
<th>35 kips</th>
</tr>
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<tbody>
<tr>
<td>4</td>
<td>Average</td>
<td>18</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Min-Max</td>
<td>5-51</td>
<td>2-121</td>
</tr>
<tr>
<td>5</td>
<td>Average</td>
<td>8</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>Min-Max</td>
<td>5-11</td>
<td>13-19</td>
</tr>
</tbody>
</table>

The location of the strain gage in section 5 was at station 61 ft, while the only crack in that section was at 15 ft, which means the sensors responded to an uncracked slab. In section 4 the distance between the sensors and the nearest crack was less than 4 inches for two of the sensors, and 5 feet for the third.

The most probable cause of lower bending deformation in section 5 is the absence of transverse cracks near the location of the sensor. In order to explore another possibility, the contribution of transverse steel was studied. The chairs supporting the upper layer of steel consist of 3-#2 bars.
They are placed 4 feet apart in between the transverse steel supporting the lower layer, which consist of #4 bars. A schematic diagram is shown in Figure 6.34.

![Diagram](image)

Figure 6.34. Transverse steel reinforcement in sections 4 and 5

The additional moment of inertia provided by the steel was insignificant when compared to the moment of inertia of the concrete. Section 4 has 0.029% reinforcement in the transverse direction while section 5 has 0.051%, but the transverse steel in section 5 is located closer to the neutral axis, therefore its contribution to bending resistance is similar and negligible in both cases.

6.9.2.3 Crack width

The closing measured at 35 kips is used here because it gives more consistent results than at 10 kips (lower signal to noise ratio). The closing of the cracks measured at the top and mid-top sensors (1 and 5 inches from surface) are presented below for the various temperature conditions experienced during loading. The last column is the estimated crack width under conditions of uniform 32°F temperature through the thickness. The width is calculated at the depth of steel for section 4 (4.5 inches) and the average depth of steel for section 5 (5.25 inches).
Table 6.7. Crack opening (microns)

<table>
<thead>
<tr>
<th>Section</th>
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<th>Cr.2</th>
<th>Cr.3</th>
<th>Cr.4</th>
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<tr>
<td>Top</td>
<td>25-54</td>
<td>43-66</td>
<td>43-78</td>
<td>25-50</td>
</tr>
<tr>
<td>Mid-top</td>
<td>14-27</td>
<td>41-50</td>
<td>28-51</td>
<td>14-30</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Standard crack width at 32°F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
</tr>
<tr>
<td>Mid-top</td>
</tr>
</tbody>
</table>

Not much can be concluded regarding crack width since section 5 developed only one transverse crack, and thus it is expected to be wider.

6.9.3 Consolidation concerns

One concern of 14-inch CRCP is constructability especially with 0.80% steel content. Consolidating the concrete below the steel depth will be difficult especially with the top layer of steel at 4.5 inches from the surface. The concrete placed in these experimental CRCP test sections had a 3- to 4-inch slump, whereas concrete placed with a slip-form paver has a specification range of 0.5- to 1.5-inch slump. In the field, use of a lower slump mixture with steel located in the upper third of the slab will require more compactive energy to assure proper concrete density.

Two pavement cores were taken from section 4 and two from section 5 to evaluate the compaction of the concrete. In each section, one core was extracted at a transverse crack and the other at an intact location. Visual inspection of the cores does not reveal differences in consolidation between cores in section 4 and 5. Concrete density was measured from the cores. Three of them had a segment of rebar embedded, thus the density of the core was corrected to obtain the concrete density. The concrete core densities were obtained in the dry and saturated surface dry (SSD) conditions per ASTM C1084, and are presented in Table 6.8 and Figure 6.35.
Table 6.8. Concrete density

<table>
<thead>
<tr>
<th>Core</th>
<th>Total volume (in³)</th>
<th>Dry weight (lb)</th>
<th>Saturated weight (lb)</th>
<th>Saturated immersed weight (lb)</th>
<th>Steel volume (in³)</th>
<th>Steel weight (lb)</th>
<th>Corrected dry concrete density (lb/ft³)</th>
<th>Corrected saturated concrete density (lb/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4a</td>
<td>167.23</td>
<td>14.4</td>
<td>14.54</td>
<td>8.45</td>
<td>0.00</td>
<td>0.000</td>
<td>148.79</td>
<td>148.98</td>
</tr>
<tr>
<td>4b</td>
<td>166.50</td>
<td>14.6</td>
<td>14.78</td>
<td>9.06</td>
<td>2.56</td>
<td>0.724</td>
<td>146.26</td>
<td>156.12</td>
</tr>
<tr>
<td>5a</td>
<td>169.56</td>
<td>14.7</td>
<td>14.84</td>
<td>9.04</td>
<td>2.33</td>
<td>0.660</td>
<td>145.08</td>
<td>155.06</td>
</tr>
<tr>
<td>5b</td>
<td>148.34</td>
<td>12.9</td>
<td>13.02</td>
<td>8.84</td>
<td>2.18</td>
<td>0.617</td>
<td>145.21</td>
<td>189.96*</td>
</tr>
</tbody>
</table>

*too high and not realistic

Figure 6.35. Dry and saturated density of concrete obtained from cores

Density results do not support a lack of consolidation below the double layer of reinforcement in section 5. Dry density for each sample is less than 2 percent from the average value. The saturated density in core 5b was too high and thus cannot be considered realistic. (Note, the field sections were internally consolidated by a hand-held vibrator.)

Cores from section 4 were 14.2 and 14.1 inches in length, and the bottom revealed the contact with the underneath layer of BAM. The cores from section 5 were 13.5 and 13.4 inches, and the interface with the BAM was not identifiable.
6.9.4 Findings and conclusions

No detrimental effect could be determined due to the presence of the double layer reinforcement steel, although the natural development of transverse cracking in section 5 makes it difficult to assess its impact on crack spacing and width. No significant structural response differences were found between sections 4 and 5.

6.10. Structural design requirements for heavily trafficked CRCP

The design guide developed by NCHRP 1-37A (DG2002) was run to check for the required slab thickness for an extended-life CRCP and to assess the sensitivity of the design to changes in the pavement features and material properties. The extended-life CRCP was used on a six-lane facility with an ADT of 70,000 (2020 estimate), an ADTT of 25,000, and a 0.5 directional distribution. The design lane had 74 percent of the one-way trucks traveling in it. The vehicle class distribution was based on IDOT’s weigh station data from I-55 (Bolingbrook). The vehicle class distributions are shown in Table 6.9 below. The default hourly distribution of trucks and axle load distribution factors specified in the DG2002 software were utilized. No traffic growth factors were assumed over the 40 years. Based on the ADTT and axle load distributions for a 40-year pavement life, the traffic was characterized as 230.1 million ESALs or 135.6 million heavy trucks for all cases analyzed.

Table 6.9. Vehicle class distribution

<table>
<thead>
<tr>
<th>FHWA Vehicle Class</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage</td>
<td>2.8</td>
<td>8.5</td>
<td>3.0</td>
<td>0.0</td>
<td>13.0</td>
<td>63.5</td>
<td>0.8</td>
<td>8.4</td>
</tr>
</tbody>
</table>

The concrete was assumed to have a modulus of rupture of 650 psi at 28-days and coefficient of thermal expansion (COTE) of $6.3 \times 10^{-6} \degree F$. The climatic data used for the analysis was taken from Chicago, IL. The following pavement cross-section was assumed: CRCP surface layer, 6
inch BAM, 12 inch aggregate subbase, and ML soil. The soil had a k-value of 200 psi/in. The following design features were analyzed with the DG2002 software:

- Slab thickness – 10, 12, or 14 inches
- Steel Content – 0.6%, 0.8%, or 1.0%
- Bar Diameter - #5, #6, or #7
- Depth to Steel – 3.5 & 7 inch (only for 12” and 14”)
- Shoulder Type - Asphalt
- Construction Season – August and November
- Base Type – BAM, CTB

The DG2002 was allowed to generate the mean crack spacing based on the material and design feature inputs. The failure of the CRCP section was assumed to be 10 punchouts per mile. Two reliability levels were run: 50 and 95 percent. The results of the design software runs are summarized in Table 6.10 for the 6-inch BAM base and a construction date of August.
Table 6.10. CRCP design runs for August construction and BAM base

<table>
<thead>
<tr>
<th>Run</th>
<th>Thickness (in)</th>
<th>Bar Size</th>
<th>Steel Ratio (%)</th>
<th>Steel Depth (inch)</th>
<th>Punchout per mile 50%(yrs)</th>
<th>Punchout per mile 95%(yrs)</th>
<th>Crack Spacing1 (inch)</th>
<th>Crack Width2 (mils)</th>
<th>LTE3 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>#5</td>
<td>0.6</td>
<td>3.5</td>
<td>10.3</td>
<td>3.1</td>
<td>36.4</td>
<td>8.2-19.7</td>
<td>42</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>#6</td>
<td>0.6</td>
<td>3.5</td>
<td>7.7</td>
<td>3.0</td>
<td>43.6</td>
<td>10.2-23.7</td>
<td>42</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>#6</td>
<td>0.8</td>
<td>3.5</td>
<td>13.4</td>
<td>3.1</td>
<td>32.5</td>
<td>7.5-17.5</td>
<td>55</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>#7</td>
<td>0.8</td>
<td>3.5</td>
<td>10.7</td>
<td>3.1</td>
<td>38.2</td>
<td>9.0-20.8</td>
<td>55</td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>#7</td>
<td>1.0</td>
<td>3.5</td>
<td>16.7</td>
<td>3.1</td>
<td>30.1</td>
<td>6.5-16.2</td>
<td>72</td>
</tr>
<tr>
<td>6</td>
<td>10</td>
<td>#7</td>
<td>1.0</td>
<td>5.0</td>
<td>14.6</td>
<td>3.1</td>
<td>35.8</td>
<td>8.0-19.0</td>
<td>72</td>
</tr>
<tr>
<td>7</td>
<td>12</td>
<td>#6</td>
<td>0.6</td>
<td>3.5</td>
<td>13.4</td>
<td>10.4</td>
<td>44.8</td>
<td>10.5-24.0</td>
<td>42</td>
</tr>
<tr>
<td>8</td>
<td>12</td>
<td>#6</td>
<td>0.8</td>
<td>3.5</td>
<td>23.6</td>
<td>19.4</td>
<td>33.4</td>
<td>7.5-18.0</td>
<td>55</td>
</tr>
<tr>
<td>9</td>
<td>12</td>
<td>#7</td>
<td>0.6</td>
<td>3.5</td>
<td>10.0</td>
<td>8.3</td>
<td>51.3</td>
<td>12.0-28.0</td>
<td>42</td>
</tr>
<tr>
<td>10</td>
<td>12</td>
<td>#7</td>
<td>0.8</td>
<td>3.5</td>
<td>18.3</td>
<td>15.2</td>
<td>39.3</td>
<td>9.5-21.5</td>
<td>55</td>
</tr>
<tr>
<td>11</td>
<td>12</td>
<td>#7</td>
<td>0.8</td>
<td>6.0</td>
<td>17.2</td>
<td>14.3</td>
<td>45.8</td>
<td>10.5-24.0</td>
<td>55</td>
</tr>
<tr>
<td>12</td>
<td>12</td>
<td>#7</td>
<td>1.0</td>
<td>3.5</td>
<td>30.1</td>
<td>20.1</td>
<td>30.9</td>
<td>7.0-16.5</td>
<td>72</td>
</tr>
<tr>
<td>13</td>
<td>14</td>
<td>#6</td>
<td>0.6</td>
<td>3.5</td>
<td>20.6</td>
<td>17.3</td>
<td>46.5</td>
<td>11.0-25.0</td>
<td>42</td>
</tr>
<tr>
<td>14</td>
<td>14</td>
<td>#7</td>
<td>0.6</td>
<td>3.5</td>
<td>16.6</td>
<td>13.4</td>
<td>53.1</td>
<td>13.0-29.0</td>
<td>42</td>
</tr>
<tr>
<td>15</td>
<td>14</td>
<td>#7</td>
<td>0.8</td>
<td>3.5</td>
<td>29.4</td>
<td>25.4</td>
<td>40.8</td>
<td>10.0-22.0</td>
<td>55</td>
</tr>
<tr>
<td>16</td>
<td>14</td>
<td>#7</td>
<td>1.0</td>
<td>3.5</td>
<td>40.0</td>
<td>40.0</td>
<td>32.0</td>
<td>7.5-17.5</td>
<td>97</td>
</tr>
<tr>
<td>17</td>
<td>14</td>
<td>#7</td>
<td>1.0</td>
<td>7.0</td>
<td>40.0</td>
<td>40.0</td>
<td>36.6</td>
<td>8.0-19.0</td>
<td>93</td>
</tr>
<tr>
<td>18*</td>
<td>14</td>
<td>#7</td>
<td>0.8</td>
<td>3.5</td>
<td>29.9</td>
<td>26.5</td>
<td>40.8</td>
<td>10.0-22.0</td>
<td>55</td>
</tr>
</tbody>
</table>

1,2 – Crack spacing and crack width range at end of design life
3 – LTE at end of the design life
* Changed built-in curl from –10 to 0°F

As a reference design, a 10-in. jointed plain concrete section with 1.5 inch dowels was run with the same materials, cross-section, and traffic. The results showed 100 percent slab cracking at the end of the 40-year design life for 95 percent reliability. If the slab thickness was increased to 12-in., the slab cracking was only 19 percent for a 95 percent reliability. IRI and faulting criteria
were not considered for this fatigue only analysis. However, according to the DG2002 procedure additional slab thickness would be required to meet acceptable levels of faulting and IRI.

For additional CRCP analysis, the base was changed to CTB and several cases were re-run at the 14 inch thickness to try and achieve a design with 0.8 percent steel content. The results can be viewed in Table 6.11. Changing the base from BAM to CTB did not change the design as seen when comparing runs 15 and 19. The time of the year of the construction had a tremendous impact on the punchouts predicted (zero stress temperature decreases significantly). For November construction, the 14-inch section passed under 0.8 percent steel and 12-inch CRCP passed the failure criteria for 50 percent reliability. By changing the construction month, the zero temperature stress of the CRCP section is decreased resulting in tighter crack widths. Runs 22 and 23 show that there is no difference in punchouts predicted if the crack spacing is 54.4 inches or 42 inches. When tied concrete shoulders are added a 10-inch CRCP passes the 40-year design life at 50 percent reliability.

Table 6.11. CRCP design runs for November construction and CTB base

<table>
<thead>
<tr>
<th>Run</th>
<th>Thickness (in)</th>
<th>Bar Size</th>
<th>Steel Ratio (%)</th>
<th>Steel Depth (inch)</th>
<th>Punchout per mile 50% (yrs)</th>
<th>Punchout per mile 95% (yrs)</th>
<th>Crack Spacing¹ (inch)</th>
<th>Crack Width² (mils)</th>
<th>LTE³ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>19*</td>
<td>14</td>
<td>#7</td>
<td>0.8</td>
<td>3.5</td>
<td>29.3</td>
<td>25.4</td>
<td>40.8</td>
<td>10.0-22.0</td>
<td>55</td>
</tr>
<tr>
<td>20</td>
<td>14</td>
<td>#7</td>
<td>0.8</td>
<td>3.5</td>
<td>40.0</td>
<td>40.0</td>
<td>51.6</td>
<td>7.0-13.0</td>
<td>100</td>
</tr>
<tr>
<td>21</td>
<td>14</td>
<td>#7</td>
<td>0.8</td>
<td>7.0</td>
<td>40.0</td>
<td>40.0</td>
<td>62.9</td>
<td>7.0-14.0</td>
<td>100</td>
</tr>
<tr>
<td>22+</td>
<td>12</td>
<td>#6</td>
<td>0.8</td>
<td>3.5</td>
<td>40.0</td>
<td>22.3</td>
<td>54.4</td>
<td>1.7-4.8</td>
<td>100</td>
</tr>
<tr>
<td>23$+</td>
<td>12</td>
<td>#6</td>
<td>0.8</td>
<td>3.5</td>
<td>40.0</td>
<td>22.3</td>
<td>42.0</td>
<td>2.0-5.0</td>
<td>100</td>
</tr>
<tr>
<td>24#+</td>
<td>10</td>
<td>#6</td>
<td>0.8</td>
<td>3.5</td>
<td>40.0</td>
<td>7.0</td>
<td>53.6</td>
<td>1.5-4.5</td>
<td>100</td>
</tr>
<tr>
<td>25#$</td>
<td>10</td>
<td>#6</td>
<td>0.8</td>
<td>3.5</td>
<td>40.0</td>
<td>25.0</td>
<td>54.3</td>
<td>3.0-6.5</td>
<td>100</td>
</tr>
</tbody>
</table>

¹² Crack spacing and crack width range at end of design life
³ LTE at end of the design life
* Construction season August
+ MOR =600psi, E=3350 ksi, ult. shrinkage=450microstrain, COTE=4.5e-06/°F
$ Crack spacing fixed at 42 inches
# Tied concrete shoulders
† MOR=700psi, E=4e06psi

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The expected sensitivity of the design guide to depth of steel and slab thickness was not seen. The most influential factors were changing the construction season from August to November (zero stress temperature change) and adding a tied concrete shoulder. The bar diameter or the bond area relative to the concrete volume is also important for crack spacing and width development. Using a smaller diameter bar, when possible at the same steel percentage decreases the crack width and predicted punchouts.

6.11. Summary of Chapter 6

The progression of cracks was followed from the time of construction until a stable pattern developed. Average crack spacing interval ranged from 2.6 feet in section 3 to 4.8 feet in section 4 with high section variability. The effect of the design parameters such as steel content, steel depth, and thickness on crack spacing was as expected based on existing model predictions. Measured crack widths on each section were small compared to values presented in the literature. Section 1 had an average crack width at the depth of the steel of 0.116 mm. Crack widths were smaller in all the other sections, with the minimum of 0.031 mm observed in section 4.

Accelerated traffic loading at the slab’s edge with the ATLAS resulted in longitudinal cracks in sections 1, 2, and 3. The cracks started at transverse cracks, at 4 to 5 feet from the slab’s edge, and propagated toward the pavement’s edge creating half-moon cracks. Several of these extended punchouts were observed in each of the failed sections. These all occurred under heavy wheel loads of 30 kips or higher. The ATLAS testing showed there were apparent punchout performance differences between sections 1, 2, and 3 (511, 230, and 548 million ESALs, respectively). Section 2 had the lowest ESALs to failure and 0.8% steel. The use of the ESAL concept applied to accelerated pavement testing under changing temperatures, moisture contents, and channelized loading at the slab edge has its limitations and conclusions made should be done so cautiously and not over generalized. A conclusion that 0.8% steel content is worse than 0.6% is likely not correct due to differences in climatic conditions during testing and the fact that the
softest subgrade, which had necessitated undercutting and backfilling with aggregate during construction, resided under section 2.

Typical pavement responses during loading tests were presented. Vertical deflection, transverse strain, and crack widths were shown to vary with accumulated traffic and temperature cycles. Load transfer efficiency was also affected by temperature but remained greater than 90 percent for the duration of the tests on sections 2 (0.8% steel content) and 3 (1.09% steel content).

The failures observed in sections 1, 2 and 3 indicate that the repeated transverse bending of the slab created permanent deformation in the support layers near the slab edge that eventually led to punchout failure under small crack widths with high shear capacity. This method of failure differs from the traditional punchout sequence of events in that a reduction in load transfer efficiency is not the cause of the CRCP deterioration.
CHAPTER 7   RECOMMENDATIONS FOR IMPROVED CRCP CONSTRUCTION

High performance CRC pavements are achieved not only by using better design tools but also by refining construction practices in order to ensure that the design assumptions are met. This chapter discusses three areas of research that could be implemented and are aimed at reducing transverse crack width. A reduction in the average crack width subsequently increases the shear transfer capacity across the transverse cracks and ultimately increases the slab’s resistance to fatigue. Due to susceptibility of crack width to pavement temperature, it seems evident that reduced crack width would mean that the transverse cracks will be fully closed (zero crack width) during a larger portion of the pavement life. This concept could be stated as reducing the crack closing temperature or zero width temperature. The three CRCP construction recommendations presented here refer to early-age temperature conditions, drying shrinkage, and induction of transverse cracks.

7.1. Early-age concrete temperature

Crack width is substantially impacted by the concrete temperature development during the first 24 to 72 hours after placement. Although the relationships between zero-stress temperature, zero-width temperature, and maximum temperature at time of curing are still a matter of controversy, the reduction in the internal pavement temperature around the time of concrete setting is a desirable goal. Estimated crack closing temperatures were presented in Chapter 5 and are on the order of 55 to 73°F. The experimental sections were built in cooler weather which contributed to the excellent pavement performance observed under accelerated loading.

7.1.1 Background

Work conducted for the Texas Department of Transportation (Schindler 2002) has addressed the issue of early age temperature development in CRC pavements. During hydration of concrete
under field conditions, the concrete temperature development is determined by the balance between heat generation from the cementitious materials and heat exchange with the structure and its surroundings. The surroundings could either be an additional source of heat or a heat sink. Figure 7.1 presents a simplification of the process used to predict concrete temperatures under field conditions, which is categorized into the following three components:

I. **Concrete Heat of Hydration**: Numerous factors influence the concrete heat of hydration and the interaction of these factors are very complex. The cement chemical composition, cement fineness, amount of cement, water-cement ratio, presence of mineral and chemical admixtures, and the hydration temperature primarily influence the heat of hydration.

II. **Environmental Effects**: As is the case with most chemical reactions, the hydration of cement is strongly affected by its current temperature and moisture state. Environmental conditions fluctuate diurnally, and parameters such as ambient air temperature, wind speed, relative humidity, solar radiation, and cloud cover are constantly changing values. This causes the hydration behavior under field conditions to be very different from hydration under laboratory conditions.

III. **Heat Exchange**: In concrete placed under field conditions, heat will be transferred to and from the surroundings. Heat transfer mechanics have to be considered to model the transient heat exchange. As shown in Figure 7.1, the effects of various parameters including base temperature, curing methods, type of support materials, aggregate type used, slab thickness, and concrete surface color should all be accounted for in a heat transfer model.

The temperature model as well as concrete setting and thermal stress models developed for TxDOT were calibrated (Schindler 2002) by comparing the models’ predictions to those values measured in CRCP sections paved under different thermal conditions. The sections are listed in Table 7.1. The temperatures at slab mid depth, one inch from the top and one inch from the bottom were recorded at half-hour intervals, over a 72-hour period.
Figure 7.1. Primary model components and the variables considered (Schindler 2002)

Table 7.1. Texas CRCP construction sites used for model calibration (Schindler 2002)

<table>
<thead>
<tr>
<th>Construction Site</th>
<th>Date (year 2000)</th>
<th>Description</th>
<th>Temperature Ranges (°F)</th>
<th>Air a</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dallas, IH 45</td>
<td>May 5-7</td>
<td>13¼-inch</td>
<td>66.2 - 78.8</td>
<td>73.4 - 102.2</td>
<td></td>
</tr>
<tr>
<td>Houston, US 59 South</td>
<td>May 11-13</td>
<td>13-inch</td>
<td>78.8 - 84.2</td>
<td>82.4 - 111.2</td>
<td></td>
</tr>
<tr>
<td>Dallas, SH 190</td>
<td>Aug 4-6</td>
<td>12-inch</td>
<td>80.6 - 100.4</td>
<td>89.6 - 143.6</td>
<td></td>
</tr>
<tr>
<td>Houston, FM 529</td>
<td>Aug 25-27</td>
<td>10-inch</td>
<td>73.4 - 102.2</td>
<td>89.6 - 123.8</td>
<td></td>
</tr>
<tr>
<td>El Paso, Loop 375</td>
<td>Aug 17-19</td>
<td>11-inch</td>
<td>66.2 - 91.4</td>
<td>86 - 109.4</td>
<td></td>
</tr>
<tr>
<td>Dallas, IH 30</td>
<td>Sept 29-Oct 1</td>
<td>13¼-inch</td>
<td>57.2 - 86</td>
<td>73.4 - 104</td>
<td></td>
</tr>
<tr>
<td>Houston, US 59 North</td>
<td>Oct 19-21</td>
<td>15-inch</td>
<td>60.8 - 82.4</td>
<td>78.8 - 104</td>
<td></td>
</tr>
</tbody>
</table>

Note: a Air temperature range during day of paving

Numerous variables affect the development of concrete temperatures, concrete setting, and the zero-stress temperature. Schindler (2002) performed a sensitivity analysis under three environmental circumstances: hot, normal, and cold paving conditions (each condition defined by a particular range of air temperature and maximum solar radiation). The results of the sensitivity study were based on the worst case from the three paving environments analyzed. The sensitivity is rated as ‘high’, ‘moderate’, ‘low’ and ‘none’, and their definitions were selected by engineering judgment based on the change in the predicted result relative to the baseline case, and can be found in Schindler 2002 (page 328). The sensitivity ratings are summarized in Table
7.2. The maximum concrete temperature and zero-stress temperatures are significantly impacted by numerous variables from all five input categories. The base temperature may also be a significant factor in Illinois due to the use of a BAM base which can act as a heat source for the hydrating concrete.

Table 7.2. Sensitivity results obtained for each variable

<table>
<thead>
<tr>
<th>Variable</th>
<th>Sensitivity Rating</th>
<th>Maximum Concrete Temperature</th>
<th>Final Set Time</th>
<th>Zero-Stress Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Paving Environment</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>General Variables</strong></td>
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<tr>
<td>PCC Thickness</td>
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<td></td>
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<tr>
<td>Subbase Thickness</td>
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<td>None</td>
<td>Low</td>
<td></td>
</tr>
<tr>
<td>Subbase Type</td>
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<td>Low</td>
<td></td>
</tr>
<tr>
<td>Subgrade Thickness</td>
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<td>None</td>
<td>None</td>
<td></td>
</tr>
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<td>Time of Placement</td>
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<td><strong>Mixture Proportion</strong></td>
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<td>Cement Factor</td>
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<tr>
<td>w/cm ratio</td>
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<td>Low</td>
<td>Low</td>
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</tr>
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<td>Class C Ash Content (CaO= 29%)</td>
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<td>High</td>
<td></td>
</tr>
<tr>
<td>Class F Ash Content (CaO= 14%)</td>
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<td>High</td>
<td>Moderate</td>
<td></td>
</tr>
<tr>
<td>Class F Ash Content (CaO= 5%)</td>
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<td>High</td>
<td>High</td>
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<tr>
<td>GGBF Slag Content</td>
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<td>High</td>
<td></td>
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<td><strong>Materials Characterization</strong></td>
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<tr>
<td>Cement Type</td>
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<td>High</td>
<td></td>
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<tr>
<td>Blaine Value</td>
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<td>High</td>
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<tr>
<td>Activation Energy</td>
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<tr>
<td>Hydration time parameter</td>
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<td>Hydration slope parameter, β</td>
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<tr>
<td>Ultimate degree of hydration, αu</td>
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<td>Aggregate Type</td>
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<td>Coefficient of Thermal Expansion</td>
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<td><strong>Environmental Variables</strong></td>
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<td>Relative Humidity</td>
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<td>Wind Speed</td>
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<td>Solar Radiation</td>
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<td>Cloud Cover</td>
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<td>Low</td>
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<td>Deep ground temperature</td>
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<td>None</td>
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<td><strong>Construction Variables</strong></td>
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<td>Concrete Placement Temp.</td>
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<td>White wash base</td>
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<tr>
<td>Curing method</td>
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<td></td>
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<td>Color of plastic sheet</td>
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<td>None</td>
<td>High</td>
<td></td>
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<tr>
<td>Curing Blanket thickness</td>
<td>High</td>
<td>None</td>
<td>High</td>
<td></td>
</tr>
</tbody>
</table>

From the list of construction variables that affect maximum concrete temperature and the zero-stress temperature, the only ones that the TxDOT models identify as of high impact are the
concrete placement temperature, the color of plastic sheets, and the curing blanket thickness. Noteworthy is also the time of placement (hour of the day) although it was not categorized as a construction variable in Table 7.2.

An effort should be made to keep the concrete temperature as low as economically practical in warm weather paving. By controlling the temperature of the ingredients, the temperature of the fresh concrete can be regulated (ACI 305, 2000). This is currently the approach adopted by most states, since they specify a maximum concrete temperature at placement to mitigate the detrimental effects of hot weather placement.

Illinois DOT specifications related to concrete pavement placement temperature included in the 2002 version of the “Standard Specifications for Road and Bridge Construction”, Articles 1020.13-c (Protection from low temperatures) and 1020.14-a (Temperature Control for Placement). Effective January 1, 2004, the text of Article 1020.14-a was revised and the current text is presented below as extracted from the “Special Provision for Curing and Protection of Concrete Construction”:

**Temperature Control other than Structures.** The temperature of concrete immediately before placing shall be not less than 10°C (50°F) nor more than 32°C (90°F). Aggregates and/or water shall be heated or cooled as necessary to produce concrete within these temperature limits.

When the temperature of the plastic concrete reaches 30°C (85°F), an approved retarding admixture shall be used or the approved water reducing admixture in use shall have its dosage increased by 50 percent over the dosage recommended on the Department’s Approved List of Concrete Admixtures for the temperature experienced. The amount of retarding admixture to be used will be determined by the Engineer. This requirement may be waived by the Engineer when fly ash compensated mixtures are used.

Plastic concrete temperatures up to 35°C (96°F), as placed, may be permitted provided job site conditions permit placement and finishing without excessive use of
water on and/or overworking of the surface. The occurrence within 24 hours of unusual surface distress shall be cause to revert to a maximum 32°C (90°F) plastic concrete temperature.

Concrete shall not be placed when the air temperature is below 5°C (40°F) and falling or below 2°C (35°F), without permission of the Engineer. When placing of concrete is authorized during cold weather, the Engineer may require the water and/or the aggregates to be heated to not less than 20°C (70°F) nor more than 65°C (150°F). The aggregates may be heated by either steam or dry heat prior to being placed in the mixer. The apparatus used shall heat the mass uniformly and shall be so arranged as to preclude the possible occurrence of overheated areas which might damage the materials. No frozen aggregates shall be used in the concrete.

7.1.2 Recommendation

When designing a CRCP, the drop in temperature from the time of construction to the minimum temperature that the pavement will experience is a major factor in determining the amount of reinforcement. Instead of specifying fresh concrete temperature, IDOT specifications could be geared toward in-situ temperatures that should not exceed certain limits anytime during the first 72 hours after concrete placement.

This type of specification will allow contractor innovation during the selection of the mixture constituents and their proportions. The contractor will now be able to consider and optimize the cost of cooling the mixture versus the use of mineral and/or chemical admixtures during hot weather placement conditions. The contractor is in the position to schedule the paving activity at different times of the day, or even different times of the year (if possible), since this has been shown to significantly impact the maximum in place concrete temperature development.

Two actions would be required from IDOT to implement this type of specification:

1. An early-age concrete temperature prediction model needs to be adopted by IDOT and made available to the contractor. A commercial application sponsored by FHWA called HIPERPAV is an option. Texas DOT is planning to include their model into
ConcreteWorks, which is available at no cost but at the current stage has been developed mainly for mass concrete applications. A third alternative involves developing a model particular for IDOT.

2. Specify the measurement of pavement internal temperature in terms of procedure (responsibilities), equipment (sensors and dataloggers) and instrumentation geometry (depths and frequency or yardage).

7.2. Drying shrinkage

7.2.1 Background
Shrinkage of concrete (both drying and autogenous) is caused by the removal of water from capillary pores. The cement paste contracts and the aggregate does not, which results in restraining stresses. The simplest methods for reducing shrinkage include:

- Lower paste content
- Moderate w/cm (to limit both drying and autogenous)
- Larger coarse aggregate (CA) top-size

Since aggregates do not shrink, lowering the paste content simply dilutes the shrinking volume. Low w/cm leads to high autogenous shrinkage, while high w/cm leads to high drying shrinkage. The optimum w/cm appears to be moderate level. Having a larger CA top-size appears to reduce shrinkage in a couple of ways: first, it allows for use of a lower paste content at the same slump. Second, the larger aggregates provide more restraint for the shrinking paste. There are a couple of ways that shrinkage could be limited in the specifications:

- Measured shrinkage specification using ASTM C157
- Maximum strength specification
- Maximum cementitious materials content and minimum and maximum w/cm
There are also some limitations to the methods described above. Specifying a measured shrinkage specification using the ASTM C 157 standard does not account for any autogenous shrinkage, which could be high for w/cm ratios less than 0.40. Autogenous shrinkage is primarily an early-age phenomenon, and the ASTM procedure requires wet curing for 28 days. One could meet the specification and still have high early shrinkage. The limitation to the maximum strength specification is that the specification could be achieved simply by using a low quality aggregate rather than by reducing paste content or increasing w/cm. One could meet the specification and still have a high shrinkage material.

The use of shrinkage-reducing admixtures (SRA) should be beneficial in reducing the width of transverse cracks. However, the potential for lower concrete shrinkage at early ages may increase the spacing of the transverse cracks and subsequent thermal contraction could lead to wide cracks. The use of SRA in CRCP should be thoroughly evaluated before field implementation since it is not clear how this admixture would affect CRCP performance.

7.2.2 Recommendation

IDOT should consider specifying a maximum cementitious content and a minimum and maximum w/cm. This specification language does not suffer from the limitations described above for the first two possible specifications for limiting shrinkage. A possible value for the minimum w/cm is approximately 0.39-0.40. This value is low enough to permit the minimum strength requirements to be easily obtained, yet is high enough to limit severe autogenous shrinkage. SRA should be evaluated by IDOT in order to determine its potential for use in future IDOT applications. Specifications regarding lower shrinkage in CRCP should be related to transverse crack induction to minimize the adverse effect of low shrinkage on crack spacing.

7.3. Induction of transverse cracks

Continuously reinforced concrete pavements are traditionally built to have cracks occur naturally or passively under the restraint of the steel with the mean spacing based on steel content,
concrete to steel bond strength, base friction, concrete material properties, and environmental conditions. Since the recommendations in this report include reduction of drying shrinkage, there is an implied risk of greater crack spacing if no actions are taken. Transverse crack induction is a technique that was studied as part of this research project and it has also been tested in the past. Results indicate that induction helps to ensure adequate crack spacing and has some other benefits for extended life CRCP. More detailed information can be found in a paper by Kohler and Roesler (2004).

7.3.1 Background

7.3.1.1 Past experience with active crack control on CRCP

On rigid pavements, the effectiveness of saw-cutting is well known and has been widely used to reduce random crack formation in jointed plain and jointed reinforced concrete pavements. In the early 1990s, the Texas DOT constructed CRCP sections to look at the effects of coarse aggregate and curing methods on crack control. McCullough (1999) recommended active crack control for CRCP placed in temperatures exceeding 90ºF and constructed with aggregates that have a high coefficient of thermal expansion (CTE). Zollinger and Soares (1999) suggested the following guidelines for active crack control in CRC pavements based on the results of the Texas DOT test sections. In the summer months, for low CTE and high concrete fracture toughness use passive crack control and skewed transverse chairs. For high CTE and low fracture toughness use shallow saw-cut notches. Passive crack control can be used in winter months but for aggregates with high CTE and low fracture toughness, saw-cutting in combination with mid-depth crack inducers should also be implemented to minimize delamination.

7.3.1.2 Experimental Sections

As mentioned in Chapter 3, two lanes of pavements were built for this project. Lane 1 was left to develop natural transverse cracks and was loaded with the ATLAS, while in lane 2 the cracks were induced and the sections were not loaded with traffic. The induction of the cracks in lane 2 was carried out by two methods:
- **Sawing.** A Soff-Cut® saw was used to cut a 1.5-inch notch on top of the CRCP surface, based on work by Jeong et al. (2001). The early entry saw-cutting occurred approximately four hours after concrete placement.

- **Automated Tape Insertion.** This innovative procedure consists of creating a weakened plane by means of a plastic film inserted in the fresh concrete, as shown in Figure 7.2a. An automated tape insertion device was developed and used to expedite the construction process, as shown in Figure 7.2b. The insertion material was plastic tape 3-mil thick by 3-in. deep.

![Figure 7.2. Execution of crack induction by automated tape insertion. a) Close up of plastic film, b) Insertion device](image)

Induced crack spacing was set at 2, 4 or 6 feet with sawing and tape insertion used in an alternating pattern. There were a total of 37 saw-cut locations and 72 locations with tape insertions, totaling 109 induced cracks, as shown in Figure 7.3.
Regular crack surveys were performed over the first 18 months to evaluate crack progression in the CRCP sections. Although it is difficult to verify the full propagation of the crack from the bottom of the notch to the bottom of the slab, a good estimation of whether a complete crack has formed was obtained by examining both edges of the slab. The first crack surveys were performed every few days after construction, and subsequently at 30-day intervals.

Crack development and spacing
Figure 7.4 shows crack development in both lanes, by presenting the location of all cracks at the time of each crack survey. There were more cracks earlier in the active crack lane and at
regularly spaced intervals. Section 1 was not included in this analysis because it was subjected to accelerated load testing. Cracks were not seen in section 5 due to its thickness (14-in.) and location adjacent to the lug anchors. The movement of the lug anchors also affected the crack development in section 10. Figure 7.5 shows the actual spacing on each section as of July 2003 along with a 5-point moving average. The uniformity of the crack spacing on the active crack control lane relative to the passive crack control lane can be viewed in Figure 7.5.

![Figure 7.4](image)

Figure 7.4. Location and time progression of cracks on Lane 1 (passive crack control) and on Lane 2 (active crack control)
Figure 7.5. Crack spacing on lane 1 (sections 1-5) and lane 2 (sections 6-10)

Figure 7.6 presents the progression of cracks in both lanes. Almost all the cracks in lane 2 occurred in the first 30 days. However, it took two winters for the number of cracks in lane 1 to equal the number of cracks in lane 2. In both lanes, new cracks appeared during the winter months when the concrete was contracting relative to the steel, whereas no new cracks were seen during the warmer months of the year (April-October).
Figure 7.6. Crack development. a) Total number of cracks per lane. b) Percent of induced cracks on Lane 2 over the first 3 months, according to induction type.

For the induced cracks, the tape insertion method developed cracks slightly faster than the saw-cutting method, as seen in Figure 7.6b. For both induction types, almost all cracks occurred in the first 30 days after construction. Only four of the 109 crack-induced locations had not propagated a full-depth crack after 1 year and after the second winter (15 months), this number was reduced to one. Eleven passive cracks occurred in lane 2 as seen in Figure 7.4. Section 7 had the 6 passive cracks that formed between induced locations. A rebar locator determined that five out of the six occurred directly over the transverse rebar chairs. Two other natural cracks occurred near the lugs, and 3 natural cracks formed in the non-induced part of section 10. The reason passive cracks primarily occurred in section 7 is not clear but it could be related to the depth of steel relative to the concrete surface.
From Figure 7.4, the majority of early cracking reported in lane 1 comes from section 3, which is in the middle of the lane and has the greatest steel content. Sections 2 and 4 started to develop more cracks after the first month. In section 3, the average crack spacing was 3.2 feet after one week but reduced to 2.1 feet at the end of year one. In section 2, the reduction in crack spacing, from one week to one year, was 9.3 to 2.8 feet, and in section 4 it changed from no cracks to an average crack spacing of 16.8 feet. Figure 7.7 shows the progression of crack spacing and distribution for section 2, which is representative of the non-induced sections. From May 2002 to March 2003 (5 to 15 months pavement age), the amount of panels increased from 12 to 38, and the mean crack spacing reduced from 6.3ft to 2.0ft.

![Graph showing crack spacing distribution in section 2, May 2002 and March 2003](image)

**Figure 7.7. Crack spacing distribution in section 2, a) May 2002 and b) March 2003**

**Crack shapes and patterns**

The visual surveys revealed no Y-cracks, divided cracks, or meandering cracks on the active crack control sections (see Figure 7.8 for definitions). The first passive crack in lane 2, i.e. a
crack at a non-induced location, appeared 5 months after construction. Lane 1 exhibits meandering cracks, divided cracks, and Y-cracks. Based on field surveys from the literature, Y-cracks and divided cracks have a higher propensity to deteriorate and spall more rapidly under traffic loading. A picture of typical divided cracks and a Y-crack on lane 1 is shown in Figure 7.9.

Figure 7.8. Crack shapes and patterns associated with defective passive cracks
Figure 7.9. Surface defects of natural cracks. a) Y-crack; b) and c) divided crack
Lane 2 has one location where cluster cracking exists. As shown in Figure 7.4, there are four natural cracks, which developed in adjacent induced 4-ft panels, thus creating a set of cluster cracks. As mentioned earlier, the majority of these passive cracks in lane 2 coincided with the transverse reinforcement chairs. Cluster cracks are numerous in lane 1 considering the length of the test sections. Figure 7.5 shows that there are multiple locations on sections 2 and 3 where cluster cracking has occurred.

Crack face characteristics
The induction system in lane 2 resulted in earlier cracking than the natural crack sections. Higher LTE and cyclic shear resistance is expected from cracks that have formed earlier, assuming equivalent crack widths. Accelerated load test data from 5 specially instrumented cracks in section 1 seem to corroborate previous findings. The LTE values obtained during load testing were high, ranging from 74 to 96 percent. Out of the 5 instrumented cracks in section 1, the two earlier age cracks measured 94 percent LTE, while the 3 later developing cracks averaged 86 percent LTE. The final punchout failure occurred at the later age developing cracks. The average LTE on lanes 1 and 2, based on Falling Weight Deflectometer (FWD) measurements, dropped in both lanes from 95 to 93 percent. No conclusions on the crack face characteristics for active and passive cracks can be made based on the FWD test results.

To further study crack face characteristics, six 4-inch cores were taken at transverse cracks, and then split to expose the fractured surfaces. The cores were scanned with a distance-measuring laser to quantify the surface profile. A picture of the split cores is shown in Figure 7.10, and the locations are presented in Figure 7.11. Results of the surface profiles have not been analyzed and work continues in this regard based on methods developed by Chupanit and Roesler (2005).
7.3.2 Recommendations

It is recommended that IDOT implement trial sections of CRCP with induced transverse cracks especially for thick CRCP sections ($\geq 12$ inches) to minimize early-age distresses, high crack spacing variability, and other types of undesirable crack shapes and patterns. The study
presented here demonstrated that induced cracks propagate quickly and provide uniform spacing between transverse cracks. Almost all induced cracks had reached the bottom of the slab by the time the pavement was one month old. In the sections with non-induced cracks, it took two winters to reach a similar level of transverse cracking. Having the cracks originate and propagate early, benefits the pavement by preventing thermal movements in panels that are initially very long, and progressively become smaller, thus the oldest cracks are subjected to more thermal movement than later developing cracks. As a result, early developing cracks should also have better load transfer capacity. In a field implementation study, an automated crack induction device on the paver, which inserts the tape, would be ideal to avoid a separate construction operation. If early entry saw-cutting was necessary, this operation would have to be done after the curing application and final set of the concrete. Another aspect of the future field investigation would be to better understand the optimal crack spacing for Illinois’ climatic conditions and materials.

Based on the experimental sections, there are reasons to believe that the benefits that can be expected from inducing cracks in CRCP will exceed the extra cost at the time of construction. Both induction methods tried during this research (Soff-Cut and plastic film insertion) worked well and did not significantly slow down construction productivity. Although double layer reinforcement did not show measurable differences compared to the single layer steel sections, an added security for this pavement type would be using a crack induction technique. The ultimate impact on production and costs would only be known if a future field investigation was conducted to determine the cost to benefit ratio.
CHAPTER 8   CONCLUSIONS AND RECOMMENDATIONS

8.1. Conclusions

Ten full-scale experimental sections of continuously reinforced concrete pavement (CRCP) were constructed. Five sections were tested under accelerated loading, while the five remaining sections were monitored to study the effect of transverse crack induction techniques. The ten test sections were constructed with 10- or 14-inch concrete thickness, steel contents of 0.6%, 0.8% or 1.1%, concrete cover depths of 3.5 or 4.5 inches, and reinforcing steel placed in one or two layers. The base material was a 4-inch BAM on top of a 6-inch aggregate subbase and geotextile fabric. The test sections were loaded with the ATLAS device by means of a single aircraft tire. The wheel was trafficked along the pavement edge at load levels between 10,000 and 50,000 pounds. The main objective of the full-scale test sections and accelerated loading was to determine the sequence and process of punchout distress development in CRCP sections with respect to slab thickness, percent steel, and depth of reinforcement. The influence of crack width on the failure of CRCP sections was of particular focus in this research project. Crack width and vertical movement were investigated and measured at selected cracks in the loaded section by using special instrumentation and applying simulated traffic loads. Continuous survey of the pavement for two years and the sequential application on each section of a large number of rolling-wheel loads at high load levels allowed for the observation of responses and failure mechanisms.

The following conclusions are drawn from the study.

1. Crack width magnitude can be obtained from analyzing the crack closing movement from application of vertical loads. This allows for measurement of crack width that is related to the structural response of the crack to the traffic loads instead of a visual assessment. Two methods were developed, temperature spectra and load spectra tests, and they serve to quantify crack width magnitude based on the changes in crack closing
movements. The load spectra method is much superior to the temperature spectra method since it can be completed in less than one hour to obtain the true crack width.

2. Variation of crack width with pavement temperature conditions is extremely important and needs to be taken into account when comparing measurements obtained under different thermal conditions. Both the average temperature through the depth and the temperature difference from top to bottom of the slab significantly affect crack width and cannot be ignored.

3. The crack width prediction model in the Mechanistic-Empirical Pavement Design Guide (M-E PDG) was found to capture the effect of thermal conditions once it was calibrated for each specific crack. It significantly overestimated the measured crack width without calibration. The calibrated model can be used to shift crack width from one set of temperature conditions to another. It was used to convert measurements obtained in different seasons to a standard temperature condition of 32°F uniform through the thickness.

4. Investigation of vertical profile of transverse cracks indicated that crack width is considerably greater near the surface of the slab. The narrowest point of the profile is located either at the depth of the steel or at the bottom of the slab, depending on the horizontal distance to the closest reinforcement bar. Modifications implemented to the M-E PDG crack width formula permits its use to predict crack width at any depth in the slab, not only at the depth of the steel as originally formulated. The predictions compared well against measured profiles. Depth of crack width measurements is relevant since it needs to be clearly specified when crack width is reported and must be consistent when comparing results from different sections.

5. An hourly crack width prediction model was suggested, based on modifying the M-E PDG formula, to account for the effect of hourly temperature changes on the magnitude of the measured crack width.

6. Zero-stress temperature (Tzs) is an important factor in predicting the mean crack spacing and crack width. However, as the CRCP section ages, the temperature to close the crack (Tzs) differs from the measured crack closing temperature. The reason for the
discrepancy is that cracks can never fully-close once they are created, and ingress of fines or debris into the cracks reduces the crack closing temperature.

7. The falling weight deflectometer has been validated as a nondestructive measuring device to determine surface crack width on CRCP. Crack width variability can be determined using the load spectra test from ATLAS or the FWD. Crack width measurements should be conducted during temperature conditions when the majority of cracks are open. With the average pavement temperature at 49.1°F and temperature differential of 0.9°F, the average crack width (for 31 cracks on section 3) was 59 microns with a standard deviation of 11.6 microns. The distribution of crack width could be described by a Weibull distribution similar to past work completed for crack spacing distribution.

8. Induction of transverse cracks offers promising results to limit the occurrence of undesired crack patterns (such as Y-cracks, cluster, divided, and meandering cracks), as well as effectively controlling crack spacing, and reducing crack width.

9. Under conditions of small crack width (less than 0.15 mm), load transfer capacity at the transverse cracks remains intact despite traffic loads and seasonal thermal cycles. With small crack width, crack deterioration and loss of load transfer capacity is minimal. Failure is not controlled by transverse crack deterioration, but caused by the permanent deformation under the slab. The punchout failures observed originated at about 4 to 5 feet from the slab edge and propagated initially parallel to the pavement but eventually turned toward the edge. Continued application of load resulted in new failures connected with the existing ones in a cascading effect.

8.2. Practical recommendations

The full-scale testing of the extended-life CRC pavements has demonstrated the capacity of this type of pavement to sustain repeated heavy loading. CRCP should be considered one of the most attractive paving options for highly trafficked corridors. The following recommendations are
based on the research findings and are targeted at specifically addressing questions originally posed in the research problem statement to IDOT.

8.2.1 Thickness requirements

The deflection responses of the 14-inch sections were less than the 10-inch section deflections. The 14-inch sections did not develop punchout failures as seen in the 10-inch sections. The three 10-inch sections tested were able to sustain 230 (Section 2), 511 (Section 1), and 548 million ESALs (Section 3) before punchout distress was observed. The 14-inch CRCP, which withstood 764 million ESALs without failure, can virtually be termed a “Perpetual Concrete Pavement”. Based on the accelerated pavement testing, the required thickness for extended-life CRCP appears to be between 11 and 14 inches given adequate supporting layers, crack spacing, and crack width magnitude.

8.2.2 One versus two layers of reinforcing steel

Two-layer steel designs have been proposed as a means to increase the space in between reinforcement bars and facilitate concrete consolidation, which presents problems in thick sections (more than 12 inches) where the total steel content, if placed in one layer, results in congested reinforcement. The ATLAS testing showed very little difference in deflection response testing between Section 4 (single layer steel) and Section 5 (double layer steel). Furthermore, there was no measurable difference in unit weight from consolidation between the one layer and two layers of reinforcing steel. However, the concrete placed in these test sections had a 3- to 4-inch slump, whereas concrete placed in a slip-form application has a specification range of 0.5- to 1.5-inch slump. Use of a lower slump mixture will require more compactive energy to assure proper concrete density and thus every effort should be made to promote adequate internal vibration. Two layers of steel may increase the likelihood of inadequate consolidation especially when using it in conjunction with a slip-form paver. If two layers of reinforcing steel are going to be used, it is recommended to use the smallest diameter bar possible and align the bars vertically (on top of one another). Finally, it is recommended that the transverse reinforcing steel and transverse chairs not be aligned vertically on top of one another.
8.2.3 Steel content

Overall, the difference in percentage of steel between the test sections in lane 1 could not be detected with any certainty from the ATLAS testing and the confidence in the calculated ESAL levels. The primary difference between the sections with respect to steel percentages was the mean crack spacing and crack width, which was smaller for sections with higher reinforcement ratios. A steel content of 0.8% appears to be quite adequate for extended-life CRC pavements, which can be defined as pavements with traffic levels of 200 to 500 million ESALs. Based on the performance of section 1 under ATLAS loading, a steel content of 0.7% should be adequate for all other CRC pavements built in Illinois.

8.2.4 Depth of steel

Depth of steel has been found in the past to be a significant factor in the performance of CRCP in Illinois. This conclusion was based on field observations for concrete slabs thinner than 10 inches. Based on the accelerated pavement testing on the 10- and 14-inch sections, the depth of steel didn’t significantly affect the CRCP responses or punchout performance. Furthermore, the proposed M-E PDG CRCP model predicts the initial set temperature of the concrete is significantly more important than the depth of steel for thicker concrete slabs. For standard and extended-life CRCP, the recommended depth of steel is presented in Table 8.1. These depths for thicker slabs are needed to ensure adequate consolidation of the concrete.

<table>
<thead>
<tr>
<th>Slab Thickness, inches</th>
<th>Depth of Steel, inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>3.5</td>
</tr>
<tr>
<td>10</td>
<td>3.5</td>
</tr>
<tr>
<td>12</td>
<td>4</td>
</tr>
<tr>
<td>14</td>
<td>4.5</td>
</tr>
</tbody>
</table>

8.2.5 Crack width

Continuous steel reinforcement is designed to assist in forming transverse cracks and then keeping them tight over time. There are many factors that impact crack width including
pavement temperature, crack spacing, shrinkage, base friction, concrete to steel bond, etc. This research found that with small crack width there is minimal loss of load transfer capacity, even after load-related cracking occurs on the slab. A crack width less than 0.15 mm at the depth of steel will result in the type of punchout failure observed in the CRCP test sections, i.e., large permanent deformations occurring in the support layers. Crack widths exceeding 0.5 mm will likely result in a traditional punchout formation, which is associated with loss in LTE across the transverse cracks. A means of determining the crack width in the field, such as described in Chapter 5, is needed to better assess CRCP performance.

8.2.6 Natural versus induced cracking
Natural cracking of CRC pavements may lead to non-uniform crack patterns and the occurrence of premature punchouts and associated distresses. Induced or active crack control on CRC pavements will help to promote and control early age cracking at the desired crack spacing. Induced cracking does not require a large notch or deformation to initiate the crack. A field implementation study should be conducted by IDOT to verify the findings from the full-scale test sections at ATREL and determine the economical viability of active crack control. Equipment constraints prevented an accelerated loading comparison between passive and active crack control sections.

8.2.7 Construction issues
The zero-stress temperature is defined as the concrete temperature at which the concrete develops tensile stresses or where cracks begin to open. Past research findings in the literature and the current full-scale pavement testing results suggest placement of the concrete mixture at a reduced air temperature leads to better CRCP performance. Lowering the zero-stress temperature through changes in the concrete mixture temperature and proportioning should also improve CRCP performance. Lower zero-stress temperatures will result in more days with cracks fully closed and load transfer efficiencies at their highest. Concrete curing should also be done to minimize other climatic factors such as solar radiation and wind on surface evaporation.
8.3. Future investigations

Service life of CRCP can be extended if permanent deformation under the slab is reduced because it will delay the development of the longitudinal fatigue cracks that result in punchouts. Since the observed CRCP failures in this study originate from loss of support under the concrete slab, it seems reasonable to investigate whether the permanent deformation causing the void comes from compaction or erosion of the subgrade, the aggregate subbase, or the bituminous base and to revisit the potential benefit of cement-treated bases for CRCP.

More research is needed to refine the original M-E PDG formula to predict hourly changes in crack width. Improvements should take into account short-term variation of the internal relative humidity and thermal gradients along with irreversible concrete shrinkage and creep.

More performance data is needed regarding the use of two layers of steel reinforcement and the effect this practice may have on crack width and crack spacing. Collaboration with the Texas Department of Transportation and their affiliates may be advantageous due to their use of two-layer reinforcement for more than 10 years.

IDOT should investigate the implementation feasibility of the recommendations presented in chapter 7 regarding construction of CRCP, particularly the crack induction and the monitoring and reduction of early age temperature development.

A field FWD program should be conducted on CRCP for a stretch of interstate (e.g., 1000 to 2000 ft.) to determine the range, mean, and variation of the transverse crack width. This information would be useful to determine the variability in field crack width and whether this factor is truly a CRCP performance indicator as suggested by many researchers.
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Preface

This appendix presents details on the accelerated load testing of each of the five CRCP sections tested during this study. The sequence of pavement testing with the ATLAS started in August 2002 and ended in August 2004. Section 1 was tested first, followed by section 2. Sections 4 and section 5 followed and finally the testing of section 3 was last. Section 1 and 2 were loaded to failure. Sections 4 and 5 were tested with similar load levels although pavement failure was not achieved. Finally section 3 was tested, resulting in the same failure pattern observed in sections 1 and 2.
1. SECTION ONE

1.1. TESTING DATES AND PAVEMENT CHARACTERISTICS

Much of the time spent on section 1 revolved around troubleshooting the new ATLAS machine and data collection system. Failure of section occurred at three separate locations with similar mechanisms, but different from typical CRCP punchout failure mechanisms referenced in the literature. This report addresses the testing sequence utilized in pavement section 1, followed by a description of crack progression, instrumentation results, and time, location, and mechanisms of the punchout failures.

1.2. TESTING SEQUENCE

Load levels, amount of passes, and ATLAS main events

The ATLAS was positioned on the section in January 2002, but the first passes were applied in late March. The loading was initially applied using the set of dual truck tires until mid-September when the aircraft rim and wheel assembly arrived. The total amount of passes on section 1 was approximately 246,800. As it can be seen in Figure 1-1, the load ranged from 10 to 50 kips. The intended load level for crack formation and deterioration was set to either 30 or 35 kips, and it was later increased to 40 and 50 kips to accelerate the failure.

![Figure 1-1. Load versus passes](image)

Table 1-1 describes the significant events associated with the ATLAS, by month. It includes the maximum load applied during the month and the amount of passes. A pass is defined as one loaded repetition of the wheel in any of the two directions, over the entire section or over the rest of the section after a failure has occurred. The loading is applied in rounds of several hundreds passes, and inspection of the pavement and of the machine are performed between rounds.
Table 1-1. Main events associated with the ATLAS during testing on section 1

<table>
<thead>
<tr>
<th>ATLAS main events</th>
<th>Max load (kips)</th>
<th>Repetitions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan-02 ATLAS placed on section 1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Feb-02 to May-02 Final assembly and development of the data acquisition system</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Jun-02 Breakdown: 06/05/02 the carriage repeatedly impacts the ends 06/20/02</td>
<td>10</td>
<td>9,301</td>
</tr>
<tr>
<td>Jul-02 Normal operation with dual tires</td>
<td>10</td>
<td>73,075</td>
</tr>
<tr>
<td>Aug-02 8/08/02 control board on the vector drive had to be replaced for 2nd time</td>
<td>20</td>
<td>19,438</td>
</tr>
<tr>
<td>Sep-02 09/11/02 A problem with the mechanism that provides tension to the wire</td>
<td>30</td>
<td>10,081</td>
</tr>
<tr>
<td>June 09/12/02 Problem about impacts at the ends is solved. 09/15/02 Aircraft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>wheel is installed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oct-02 Breakdown: 10/07/02 The wheel does not run parallel to the ATLAS frame.</td>
<td>35</td>
<td>22,922</td>
</tr>
<tr>
<td>Decision is made to redesign the swing arm structure. 10/18/02 The axle on the</td>
<td></td>
<td></td>
</tr>
<tr>
<td>aircraft wheel is found to be bent</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nov-02 11/22/02 The new swing arm is installed</td>
<td>30</td>
<td>3,878</td>
</tr>
<tr>
<td>Problems with the lateral position of the carriage were solved and the aircraft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>wheel installed</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dec-02 Breakdown: 12/13/2002 aircraft rim fails, the tire explodes, parts</td>
<td>35</td>
<td>7,105</td>
</tr>
<tr>
<td>of the bearings cases damage the tent and trailer.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan-03 Dual truck tires installed temporarily</td>
<td>10</td>
<td>6,380</td>
</tr>
<tr>
<td>Feb-03 Load applications with the dual truck tires only</td>
<td>10</td>
<td>5,468</td>
</tr>
<tr>
<td>Mar-03 Breakdown: 3/24/2003 lateral position problems after aircraft wheel is</td>
<td>25</td>
<td>17,007</td>
</tr>
<tr>
<td>installed (new rim and carriage arms were installed along with the wheel)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Apr-03 Breakdown: 4/07/2003 one pillow block breaks</td>
<td>40</td>
<td>102</td>
</tr>
<tr>
<td>May-03 Breakdown: 5/16/2003 the wheel does not always lift off the pavement</td>
<td>30</td>
<td>19,115</td>
</tr>
<tr>
<td>at the end of a unidirectional pass</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jun-03 Continuous operation from 6/06/2003 until the end of pavement testing on</td>
<td>50</td>
<td>52,890</td>
</tr>
<tr>
<td>section 1, 6/23/2003</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TOTAL 246,762</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Many “non-relevant passes” were applied on section 1. They are not included in the count of test passes because they corresponded to load values under 10 kips, or close to the center of the lane, or for very short distances. Non-relevant passes were required to verify operation of the machine after replacement of parts, test changes in the data collection program, or during demonstrations.

Figure 1-2 presents ESALs per day and illustrates the times the ATLAS was fully operational (green), partially operational (yellow) and down (red). Out of the 460 days of ATLAS on test section 1, 72 days (16%) were operational, 268 days (58%) were partially operational, and 120 days (26%) the machine was not functional. Most of the effective testing occurred during the last
month, even though there were heavy loading episodes in October and December 2002, and again in April 2003.

![Figure 1-2. Approximate ESAL per day and availability of the testing machine](image)

The amount of ESALs per day is approximate because of the way the data is organized. If a continuous loading test lasts more than one day, the amount of passes is computed on the day the loading finished. In terms of operations, the machine was usually set to run for a few thousand passes and then a detailed inspection of the pavement and the ATLAS was performed before resuming testing. The process of converting passes into ESALs is explained in the following section.

**Conversion of passes into ESALs**

The wheel load applications can be approximately converted into Equivalent Single Axle Loads (ESALs) by using conversion factors. The load repetitions with the ATLAS are applied with a single aircraft wheel or a set of dual wheels, which correspond to half an axle, therefore the equivalent axle load corresponds to twice the specified ATLAS load. An equivalent axle load factor (EALF) defines the damage per pass of an axle of any load relative to the damage to a pavement per pass of a standard axle, usually the 18-kip single-axle load (Huang 1993). The most common method to determine EALF is based on results of the AASHTO road test. For rigid pavements, the factors depend on several parameters such as pavement thickness and terminal serviceability, but they can be approximated according to equation (1), where $L_x$ is the single axle load.
\[ EALF = \left( \frac{L_x}{18} \right)^{4.3} \]  

(1)

Figure 1-3 presents the equivalency factors calculated for a 10-inch pavement, and the fitted exponential approximation. It must be kept in mind the accelerated nature of the loading in this project. Note, the maximum legal load for single axles in Illinois is 20 kips (max gross weight 80 kips for vehicles with 5 or more axles) per Illinois Vehicle Code 625 ILCS sec 5/15-111.

<table>
<thead>
<tr>
<th>ATLaS Load (kips)</th>
<th>EALF</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.081</td>
</tr>
<tr>
<td>9</td>
<td>1.00</td>
</tr>
<tr>
<td>10</td>
<td>1.58</td>
</tr>
<tr>
<td>20</td>
<td>27.91</td>
</tr>
<tr>
<td>30</td>
<td>152.6</td>
</tr>
<tr>
<td>40</td>
<td>551.1</td>
</tr>
<tr>
<td>50</td>
<td>1518.9</td>
</tr>
<tr>
<td>60</td>
<td>3493.9</td>
</tr>
</tbody>
</table>

Wheel loads of 50 kips (which would correspond to a single axle load of 100 kips) are unrealistic for vehicles and therefore the relevancy of load equivalency factors is questionable.

Another factor that has to be applied is related to the fact that, with the ATLAS, most of the traffic was applied close to the edge. Results from the PCA design method (1984) established that when 6% of the heavy traffic is driven at the edge, it causes an equal amount of damage to 100% of the traffic distributed in the wheel path (for the cases when the edge has no tied shoulder). The equivalent damage ratio (EDR), developed as part of the IDOT Mechanistic-Empirical Design Procedure for JPCP (Zollinger and Barenberg-1989), has the same concept as the PCA edge damage factor. For a 10-inch slab, the EDR for a JPCP is 0.05 or 20 times worse damage at the edge versus loading in the wheel path.

After applying the appropriate load equivalency factor and equivalent damage ratio for edge loading of 20, the amount of ESALS applied with the ATLAS can be calculated. The daily results were presented in Figure 1-2 and the accumulated ESALs are presented in Figure 1-4 and Figure 1-5, versus time and versus passes, respectively.
The total approximate ESALs applied at the end of testing in section 1, calculated as described above, was 911 million. As described later in this document, the first failure was noted when the approximate ESALs was at 511 million, the second punchout failure occurred at 687 million ESALs, and the third failure occurred at 834 million ESALs.
1.3.Crack Progression and Failure

Before load testing commenced, there were only three visible cracks present on section 1. The initial loading on section 1 was bi-directional in order to generate transverse cracks that could later be deteriorated. After significant transverse cracks were developed, unidirectional loading was employed to better simulate highway traffic conditions.

Crack surveys were performed regularly to determine the appearance of new cracks and the growth of the existing ones. Initially the crack surveys included only the pavement surface, but later the importance of crack mapping at the edge was realized. Each crack was marked and dated on the pavement, and the results were translated to paper and then into electronic files.

The first transverse cracks were identified on the section in September 2002, before the heavy loading started. More transverse cracks were detected over time as summarized in Figure 1-6, which includes crack surveys until just before the first punchout failure. The section is 85 feet long (the upper part of the image shows the distances), and its beginning is on the right hand side, because of the orientation of the ATLAS. The unidirectional loading took place from right to left.

The cracks maps presented in Figure 1-6 correspond to the pavement condition after the heavy load episodes. There were 13 transverse cracks by October 2002, and 19 by December. In April 2003, after 30 hours of continuous loading at 35 kips, a longitudinal crack appeared at the end of the section, extending across several transverse cracks. The crack had its beginning and ending at the edge of the pavement, with a half-moon shape, and it extended 4.5 feet into the lane. The crack pattern in the rest of the section did not change. With all the loading that took place after April no new transverse cracks were created. The last crack map shows some cracking occurring inside the half-moon crack. Since April, all new cracking was related to punchout failures. Figure 1-7 shows the cracks and punchouts at the end of the testing on section 1.
<table>
<thead>
<tr>
<th>Date</th>
<th>Crack Progression</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/25/02</td>
<td></td>
</tr>
<tr>
<td>10/28/02</td>
<td></td>
</tr>
<tr>
<td>12/12/02</td>
<td></td>
</tr>
<tr>
<td>4/7/03</td>
<td></td>
</tr>
<tr>
<td>6/5/03</td>
<td></td>
</tr>
<tr>
<td>6/12/03</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1-6. Crack progression
Failure in the section occurred in the form of half-moon punchouts. The first of these punchouts (see Figure 1-8) happened on June 13, and it was 18 ft long (measured at the edge), located toward the end of the section. The second punchout took place as an extension of the first one, on June 19, and it was about 9 feet long. After the first failure the wheel traffic length was reduced to prevent further loading on the failed segment, consequently the loading was set from the 0 to the 64 feet (station). After the second failure the traffic was reduced again to load only from the 0 to the 46 feet. On June 21 the third failure occurred, this time as a 30 feet long punchout. A summary of the most relevant information regarding the punchouts is presented in Table 1-2.

<table>
<thead>
<tr>
<th>Failure</th>
<th>Date</th>
<th>Approx. load repetitions (passes)</th>
<th>Accumulated ESALs (millions)</th>
<th>Wheel load at time of failure (kips)</th>
<th>Punchout Area (sq feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6/13</td>
<td>220,900</td>
<td>511</td>
<td>40</td>
<td>26</td>
</tr>
<tr>
<td>2</td>
<td>6/19</td>
<td>235,750</td>
<td>687</td>
<td>40</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>6/21</td>
<td>244,260</td>
<td>834</td>
<td>50</td>
<td>70</td>
</tr>
</tbody>
</table>
Regarding crack shape, cracks on the surface were mostly transversal (not skewed) and propagated across the entire lane width. The exceptions to this are the crack at station 20 and the half-moon cracks. The crack that appeared during the heavy loading in April is shown in Figure 1-9, digitally remarked on top of the pavement.

Figure 1-9. Half-moon crack that formed in April (enhanced photo)
1.4.SENSORS RESULTS AND ANALYSIS

Sensor setup
Pavement responses were measured with vertical LVDTs, horizontal LVDTs, and embedded strain gages. Temperature profiles were recorded with multi-depth thermocouples. The purpose of the vertical LVDTs is to measure deflections at the loaded edge at certain cracks locations; the horizontal LVDTs measure crack opening; and the embedded gages indicate the strain in the pavement close to the surface as a result of the slab bending with the load. A detailed description of the sensors can be found in the construction report (Kohler et al.-2002).

Nomenclature for the sensors and location
The four embedded gages were located along the section at 12 ft. intervals and transversally at 4.5 feet from the edge. The exact location is presented on the first column of Table 1-3. The numbers in Table 1-3 correspond to distances that are referenced to the 0 to 85 –foot length of the section and not to the general 0 to 500 feet marking used in the construction report. The initial letter identifies the type of sensor, and the suffix refers to the location of the sensor relative to the crack, East (e) or West (w), or to top (t), middepth (m) or bottom (b), depending on whether is a vertical or horizontal LVDT. The LVDTs were moved to different locations during the testing in an attempt to capture responses at the most significant places. At the beginning of the test, when there were no cracks, only the vertical LVDTs were used. They were initially installed at arbitrary locations and later moved to the initial transverse cracks. Two vertical LVDTs were used to monitor deflections at a crack, one at each side, so that the Load Transfer Efficiency (LTE) could be calculated. Two sensors at a crack share the same location which is the reason for using the east or west suffix. The different locations at which these sensors were installed, at least for time during the testing, is listed in the second column of Table 1-3. Horizontal LVDTs were sometimes installed at different depths on the edge of the pavement, which is why the sensors were labeled as top, mid-depth, or bottom.

Data collection, storage and processing
Signals coming from the sensors are collected in a synchronized manner with the passage of the loading wheel. For every pass of the wheel a complete set of responses is scanned from the sensors in order to determine the maximum, minimum and unloaded responses. This scanning is performed every 0.5 inch along the zone of interest in the section. The unloaded value corresponds to the sensor reading that is taken at the beginning of the pass, before the load has been applied. This allows for the determination of the rebound values and permanent deformation. Rebound values are defined as the difference between extreme responses (maximum or minimum) and the unloaded value, and represent the effect of the load in one pass. The maximum, minimum and unloaded values from all sensors are saved in every pass. A time-history response is recorded for each sensor, as the wheel rolls over the section, every 10 or 20 passes.
### Table 1-3. Location of sensors and nomenclature

<table>
<thead>
<tr>
<th>Strain gages (s)</th>
<th>Deflection sensors (D)</th>
<th>Crack opening sensors (O)</th>
</tr>
</thead>
<tbody>
<tr>
<td>s-22.4</td>
<td>D-22.4e</td>
<td>O-19.8t</td>
</tr>
<tr>
<td>s-34.4</td>
<td>D-22.4w</td>
<td>O-24.4m</td>
</tr>
<tr>
<td>s-46.3</td>
<td>D-24.4e</td>
<td>O-24.4t</td>
</tr>
<tr>
<td>s-58.3</td>
<td>D-24.4w</td>
<td>O-51.4b</td>
</tr>
<tr>
<td></td>
<td>D-35.5</td>
<td>O-51.4t</td>
</tr>
<tr>
<td></td>
<td>D-36.3</td>
<td>O-61.9b</td>
</tr>
<tr>
<td></td>
<td>D-48.5</td>
<td>O-61.9m</td>
</tr>
<tr>
<td></td>
<td>D-51.4e</td>
<td>O-61.9t</td>
</tr>
<tr>
<td></td>
<td>D-51.4w</td>
<td>O-72.2t</td>
</tr>
<tr>
<td></td>
<td>D-61.9e</td>
<td>O-78.1b</td>
</tr>
<tr>
<td></td>
<td>D-61.9w</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D-72.2e</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D-72.2w</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D-78.1e</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D-78.1w</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1-10 is an example of a strain time history response for four gages on section 1. It shows the response of the pavement as perceived in each one of strain gages during a single pass of the wheel. Figure 1-11 and Figure 1-12 show deflections and crack opening during the same pass.

![Figure 1-10. Example of strains collected during a pass.](image-url)
Vertical deflections measured on both sides of a crack are very similar to each other, meaning good shear transfer efficiency. In the crack opening plot, when the wheel approach the crack, the top sensor shows a small opening, but the crack closes when the wheel is very near and over the crack. The opposite behavior is true for sensors at the bottom of the slab. The response at the mid-depth may sometimes tend to show more opening and sometimes more closing, depending on temperature. More discussion about this issue is presented later.

In terms of the evolution of responses over time, it is important to mention again that different load levels were applied during the testing of section 1, and that the loadings were not continuous, but spaced over time according to the availability of the ATLAS. This second issue.
makes the role of temperature become more important than it would be in a case with a whole section tested in few weeks.

**Deflection results**

Since the pavement responses depend on the load, and the load applied was not constant along the passes, it is confusing to simply show responses versus passes. Figure 1-13 presents deflections from one sensor concurrently with the applied load levels. Naturally higher deflections are obtained with higher loads, but the point is that load repetitions also increase deflections, even if load is kept at the same load level.

![Figure 1-13. Rebound deflection from sensor D-24.4e](image)

A normalized view of deflections permits to overcome the difficulty of various loads, and shows the change in pavement responses caused by repetitions. The normalization consists of dividing the deflection by the load that causes it, and then multiplying the result by a standard load. The standard load selected is 9-kips. Figure 1-14 and Figure 1-15 show normalized deflections at different locations. The vertical line denotes the occurrence of punchouts.
Figure 1-14. Normalized rebound deflections from sensors at 24.4, 35.5, and 48.5
Figure 1-15. Normalized rebound deflections from sensors at 51.4, 61.9, and 72.2

It can be seen how the deflections increased during the test. In most cases the deflections increased considerably at the time of failure.

**Strain results**

Strains are measured close to the pavement surface (1-inch), at an approximate distance from the edge such that the highest tensile strains in the transverse direction are captured (4.5 ft.). To account for the maximum strain, which is experienced at the surface, the measured strain is amplified by a factor of 1.25 (4 inches from the neutral axis to the strain gage, but 5 inches to the surface). As with deflections, when the results are normalized by load, a better picture of
damage is obtained. However, since strain is measured to predict stress, the actual values are of particular interest. Figure 1-16 presents strain and normalized strain for each of the four strain gages.

To obtain stress out of strain, the results have to be multiplied by the elastic modulus of concrete, which is about 7 millions psi. This makes a value of 100 microstrains approximately equivalent to a 700 psi stress. The beam flexural strength is approximately 750 psi.

**Load transfer efficiency**

Two values of load transfer efficiency are obtained in each loading pass, the LTE on the approach side and the LTE on the leave side. LTE calculated with a rolling wheel is different from the LTE calculated with load at a fix position as in the case of falling weight deflectometer. When loading is applied bi-directionally, there are four LTE values from each crack. An algorithm was programmed to obtain LTE from the deflection on both sides of a crack, ensuring that calculation is performed at the instant when the load is only at one side of the crack. The results of LTE are presented in Figure 1-17. The behavior of the crack at station 61.9 was not expected and no plausible explanation for the LTE increasing with load repetitions could be found.
Temperatures
Temperature in the pavement was also measured with thermocouples at different depths. Figure 1-18 shows top and bottom pavement temperatures on section 1 along the months of testing. Figure 1-19 shows the temperature difference, top minus bottom. Figure 1-20 is a close up of the data in Figure 1-19 but only for the last month of testing.
Figure 1-18. Pavement temperatures at top and bottom from September to June

Figure 1-19. Temperature difference in the pavement depth
Temperatures shown here were collected only during the time the ATLAS was in use, but it is worthy to mention that continuous record of pavement temperature is available from the static instrumentation system. The ATLAS was stopped every day to do the inspections and crack surveys.
1.5. FAILURE MECHANISM

Based on the geometry of the pavement failures and from the results of the instrumentation, the punchout fracture occurred as a result of permanent deformation of the support layers and occurred rapidly without much change in the rebound deformation before the failure. This punchout was a result of repeated loading but more related to the support layer deformations than the fatigue of the concrete. In a typical punchout, the concrete segment that pushes down is delimited by two transverse cracks, a longitudinal crack, and the edge of the pavement. The punchouts observed in section 1 are delimited by a single crack and the edge of the pavement.

The half moon crack failures on section 1 were still tight with additional loading. However, the transverse cracks inside the failure area began exhibiting compression failure at the edge of the slab due to the high deflections. The location of the steel at 3.5 inches from the concrete surface caused the concrete to delaminate (horizontally) from the steel bars and began spalling at the transverse cracks.

**Crack geometry at punchouts**

Figure 1-21 offers a close up on the damaged pavement from station 53 to 86, with the cracks as seen from the surface as well as from the edge. Scale problems make difficult to appreciate the importance of cracks as seen from the edge, hence Figure 1-22 depicts edge cracks in more detail, covering only the zone from station 75 to 83, and its evolution from June 5 to June 13. Figure 1-23 shows pictures of the cracks as seen from the top and from the edge of the pavement on punchout number one. Figure 1-24 shows the ends of the crack at punchout number three.

Cracks on the edge were only in the vertical direction before the beginning of failure. There are some inclined cracks on the upper part of the slab, as a result of secondary compression failures above the rebar due to high deflections. The half-moon cracks appear at the edge very inclined, tending to be horizontal toward the bottom.

The cracks that form the boundary of the punchouts were not detected previous to the failures. There was no indication of the cracks propagating on the surface. Instead the cracks formed entirely during a few passes.

The cracks as seen on the edge reveal signs of the high deflections the slab experienced. The original cracks were practically vertical, but before the failure they started to show diagonal cracks on the upper part of the slab, as part of spalling cracks. These cracks are the result of compression occurring at the top of the slab.
Figure 1-21. Top and edge view of cracking pattern

Figure 1-22. Crack deterioration at the edge (on June 5 and then on June 13, 2003)
Figure 1-23. Cracks and deformation at punchout number one
Failure in terms of deflections

Maximum and unloaded deflections are measured during each wheel pass, and its difference is defined as rebound deflection. Theoretically, the unloaded deflection should remain unaltered if no damage occurs to the pavement, reflecting the fact that after each load application the slab

Figure 1-24. End of crack at punchout number three
returns to a flat condition. The maximum deflection represents the vertical deformation of the pavement system when subjected to load, and again, it should theoretically remain constant without permanent deformation. Reality first deviates from theory when temperature curling is included in the analysis, because it induces slab deformations that vary the contact condition between the slab and the base layer. The second and most important deviation from theory results when permanent or plastic deformation is considered under application of heavy loads.

The following analysis addresses the damage process in CRCP, neglecting temperature effects. Consider the following scenarios, represented in Figure 1-25.

i) Elastic deformation in slab and supporting layers: both maximum and unloaded deflections remain the same pass after pass. Low load levels are being applied. No damage to the pavement system.

ii) Elastic deformation in the slab and permanent deformation in the subbase: maximum deflections increase as more heavy-load repetitions are applied, but unloaded deflections remain unchanged because the slab returns to be flat after each load repetition. The slab is progressively subjected to higher flexural stresses, and receives less contribution from the subbase to distribute the load.

iii) Permanent deformation in both slab and subbase: when the bending capacity of the slab is exceeded, the concrete fractures and seats, with the unloaded and loaded deflection increasing.
If the entire accelerated testing could be done continuously, real magnitude of permanent deformation would be obtained on the asphalt layer under the concrete. However, practical limitations prevent a continuous loading and data recording. As mentioned before, the load repetitions are applied in rounds of several hundreds at a time, and an inspection of both the pavement and the machine is performed before the next round of load passes is launched. In terms of rebound deflection this is not a problem because a true result is obtained as long as the unloaded and maximum deflections are collected. However, the sensors are reset every time a new round of passes starts.

To illustrate how the collected data supports the aforementioned mechanism, next are presented the results from the load round in which the third punchout failure occurred. Figure 1-26 shows maximum and unloaded deflection from sensor D-24.4e during the entire load round, which consisted of 3,000 passes. Deflections increased suddenly at a point between passes 244,000 and 245,500.
Figure 1-26. Maximum and unload deflection, sensor D-24.4e

Figure 1-27 presents details of data in Figure 1-26 for a shorter span of passes, along with the rebound deflection. Unloaded deflection increased 0.5mm in less than 50 passes. This is the exact instant of concrete fracture, because the sensor indicates that the edge of the slab did not return to its unloaded position. During this 50 passes the maximum deflection also increased, but less than the unloaded deflection, causing a reduction in rebound deflection.
The same behavior observed in sensor D-24.4e was observed in sensor D-24.4w, and even at sensor D-36.3, which was located also inside of punchout area number 3. Identical mechanisms developed also on punchouts 1 and 2. For punchout number 2 the results from sensor D-61.9e are presented in Figure 1-28 and Figure 1-29. For punchout number 1 the data from sensor 72.2 and 78.1 could be used, but unfortunately the instant of exact fracture was not available.
Figure 1-28. Maximum and unload deflection, sensor D-61.9e
Figure 1-29. Detail of maximum, unloaded, and rebound deflection at time of failure at punchout 2

**Voids**

Loading on top of punchout number 1 continued after punchout formation, while in the case of punchouts 2 and 3 the load was stopped shortly after failure formation. Data from sensor D-72.2e reveals that the unloaded deflection reached more than 2 millimeters, on top of a rebound deflection of more than 4. Based on deflection information, there has to be a void under the slab of 6 to 7 mm in the case of punchout number one, while in punchouts number two and three the void is believed to be probably 5 to 6 mm.

The voids formation may relate to deformation in the asphalt layer and to subgrade compaction and erosion. There was rainwater standing over the level of asphalt layer during development of punchout number one, and the load caused visible pumping of fines.

**Forensic of punchout number one**

At location of punchout number one the slab was extensively damaged such that concrete pieces could easily be removed with hand tools. The rebars were in excellent conditions, with the
epoxy coating intact, even at the location of concrete cracks that had been identified several months before. There appeared to be indications of slippage between the interface of concrete and steel rebars. There was also abundant “white powder”, believed to have come from aggregate-paste disintegration as a consequence of relative displacement at the spalled cracks. Figure 1-30, Figure 1-31, and Figure 1-32 show concrete removal and evidence of delamination.

Figure 1-30. Concrete removal at stations 80.1 and 81.7
Figure 1-31. Concrete delamination at station 80.1
Figure 1-32. Concrete removal at station 78.1
2. SECTION TWO

2.1. TESTING DATES AND PAVEMENT CHARACTERISTICS

Testing on section 2 was performed between June 30 and September 13, 2003. There were again some problems with the ATLAS that delayed the test, but in general much of the experience learned in section 1 was applied and better test data resulted. Failure in section 2 was almost identical to section 1, which consisted of extended punchouts with half-moon shaped cracks.

This report follows the same structure as the previous one, starting with the testing sequence, followed by a description of crack progression, instrumentation results, and time, location, and mechanisms of the punchout failures.

2.2. TESTING SEQUENCE

Overview
The ATLAS was positioned on the section during the last week of June. The protective tent was not moved from section 1, therefore all the testing on section 2 was performed under conditions open to the environment. Figure 2-1 shows the ATLAS machine located on section 2.

Figure 2-1. ATLAS on section 2
Load levels, amount of passes, and ATLAS main events

Pavement responses resulting from temperature changes were recorded 36 hours before applying the trafficking load. The loading began on June 30 with a 48-hour period at 10-kips. This stage was intended to capture the elastic responses of the section, without damaging the pavement section. When the 10-kip test was finished, the load was then increased to 30-kips, and kept at that level for approximately 40,000 passes, and then the load was increased to 35-kips. The change from 30 to 35-kips was made based on the following facts: no new cracking was being observed, no increase in permanent deformation, and no decrease in load transfer efficiency. After approximately 50 thousand passes at 35-kips the pavement section failed and the load was increased to 50-kips to extend the damage. It remained at 50 kips for an additional 8 thousand passes. The 35-kip loading was done in uni-directional mode, while the rest of the loading was done with bi-directional traffic. The aircraft wheel was the only one used in this section.

Figure 2-2 shows load levels as passes accumulated in section 2. A pass is defined as one loaded repetition of the wheel in any of the two directions. The loading had to be interrupted two times, as shown in Figure 2-3. These interruptions took place during the time the loading was set at 30 kips.

![Figure 2-2. Load versus passes](image-url)
These interruptions were caused by problems with the ATLAS, as described in Table 2-1.

Table 2-1. Main events associated with the ATLAS during testing on section 2

<table>
<thead>
<tr>
<th>ATLAS main events</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jun 25</td>
</tr>
<tr>
<td>ATLAS placed on section 2</td>
</tr>
<tr>
<td>Jun 30</td>
</tr>
<tr>
<td>Response test at 10 kips</td>
</tr>
<tr>
<td>July 3</td>
</tr>
<tr>
<td>Load set to 30 kips</td>
</tr>
<tr>
<td>July 6</td>
</tr>
<tr>
<td>Breakdown: electronic failure in mechanical trailer, during lightning storm</td>
</tr>
<tr>
<td>Aug 5</td>
</tr>
<tr>
<td>Loading resumed after replacing parts</td>
</tr>
<tr>
<td>Aug 8</td>
</tr>
<tr>
<td>Breakdown: wire rope breaks (no longitudinal mobility)</td>
</tr>
<tr>
<td>Aug 21</td>
</tr>
<tr>
<td>Loading resumed after replacing wire rope. Load set to 35 kips, and changed from bi-directional to unidirectional</td>
</tr>
<tr>
<td>Aug 29</td>
</tr>
<tr>
<td>Test suspended because of rainfall</td>
</tr>
<tr>
<td>Sept 3</td>
</tr>
<tr>
<td>Test resumed at 35 kips, unidirectional</td>
</tr>
<tr>
<td>Sep 8</td>
</tr>
<tr>
<td>Load set to 50 kips and back to bi-directional, until end of testing</td>
</tr>
</tbody>
</table>

*Conversion of passes into ESALs*

The conversion factors used to transform from wheel passes into Equivalent Single Axle Loads (ESALs) are the same utilized in section 1. These factors take into account the load levels and where load is applied at the edge instead of at the wheel path. The load factor is $(Lx/18)^{4.3}$, where $Lx$ is the equivalent axle load. A wander magnification factor has to be used because of channelized edge loading. A factor of 20 is used when the wheel is right at the edge, and no factor is used when the wheel traffics at the wheel path. Since the wheel load in section 2 was applied at 2 inches from the edge, then a magnification factor somewhat different than the one
used in section 1 was applied. When the load is at the edge, an “edge damage factor” of 20 is used (see Technical Memo on Failure of Section One for more details), but there exists no information on what factor has to be use at different offsets from the edge. A value of 17 was decided assuming a linear decrease from the edge to wheelpath. This value is believed to be reasonable, for small offsets as in the case of section 2. Figure 2-4 shows accumulation of calculated ESALs over time for section 2.

![Figure 2-4. Accumulated ESALs versus time](image)

2.3.CRACK PROGRESSION AND FAILURE

Frequent crack surveys were performed during the testing of the section in order to have a record of progression of cracking. Transverse and longitudinal cracks were identified on the surface of the pavement as well as the cracks that formed on the vertical plane along the loaded edge. The progression of crack on the surface is presented in Figure 2-5.

**Transverse cracks**

Approximately 25 transverse cracks were present on section 2 at the time the loading started. These cracks were not uniformly distributed, exhibiting decreased density toward section 1.
(west). On the first 15 feet of the section there were 9 cracks, which were particularly disconnected, meaning that they had segments that were not visible on the surface. The approximate crack spacing was 1.5 feet. From station 20 to 35 the average spacing was about 2.5 feet, and from station 40 to the end of the section this value is greater than 3.5 feet. No new transverse cracks appeared as a consequence of the ATLAS loading, nor did disconnected cracks intersect. Some transverse cracks were only visible near the edges, and they remained the same despite the loading. The CRCP compression due to summertime temperatures is believed to be the reason for no new transverse cracks developing during the testing.

**Longitudinal cracks**

The first load-related cracks started to appear at the end of August, after approximately 20,000 passes at 35 kips had been applied, when the accumulated ESALs were about 230 million. The first two longitudinal cracks eventually developed into half-moon cracks, and began at station 30 feet and 70 feet. The cracks advanced longitudinally and then turned toward the pavement edge. The places where the first longitudinal crack extended to the edge coincide with longer panels. The presence of other transverse cracks apparently allowed the fracture to continue advancing longitudinally. The crack that started at about station 70, grew as an extended half-moon crack and extended 6 feet from the edge at one point.

A cascade effect created failures associated with the original punchouts, finally affecting the entire length of the section. Secondary longitudinal cracks formed closer to the edge roughly from stations 20 to 30, and from stations 40 to 70. The secondary longitudinal cracks formed when the load was at the 50 kips level.

**Edge cracks**

The cracks seen from the edge were mostly vertical with little to moderate meandering. Oblique cracks appeared later at points corresponding to the extremes of punchouts. Spalling cracks were generated in more than one half of all the vertical cracks, and they are the result of compression of the concrete above the neutral axis. A diagram of the cracks along the edge is presented in Figure 2-6. Spalling cracks were less frequent toward the end of the section, where deflections were smaller.
Figure 2-5. Crack progression
Out of the 36 cracks that were vertical (as opposed to the oblique cracks), 16 developed spalling cracks on one side and seven developed spalling cracks on both sides. They started as fissures that separated from the vertical crack at an average depth of 3.9 inches, but ranging from 2 to 7 inches from the top surface. These cracks propagate upward diagonally forming a triangle. The length of the horizontal side of the triangle reached up to 10 inches, but the average was about 7 to 8 inches. Several cracks have longer developments than 10 inches, however they are not spall-related but are part of the half-moon surface cracks.

For the purposes of clarification, failure of section 2 has been broken down into 5 parts, according with the sequence of cracking or punchout formation, and shown in Figure 2-7 and detailed in Table 2-2. The time of occurrence of each part is considered when a punchout is completed, which is different from the time the crack appeared.
Table 2-2. Punchout information

<table>
<thead>
<tr>
<th>Failure</th>
<th>Date</th>
<th>Approx. load repetitions (passes)</th>
<th>Accumulated ESALs (millions)</th>
<th>Wheel load at time of failure (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8/27</td>
<td>75,000</td>
<td>230</td>
<td>35</td>
</tr>
<tr>
<td>2</td>
<td>9/4</td>
<td>83,800</td>
<td>282</td>
<td>35</td>
</tr>
<tr>
<td>3</td>
<td>9/8</td>
<td>100,000</td>
<td>379</td>
<td>35</td>
</tr>
<tr>
<td>4</td>
<td>9/9</td>
<td>106,500</td>
<td>431</td>
<td>50</td>
</tr>
<tr>
<td>5</td>
<td>9/13</td>
<td>118,600</td>
<td>778</td>
<td>50</td>
</tr>
</tbody>
</table>

Figure 2-8 shows punchout number one. The rubber marks from the tire are clearly visible near the edge. The cracks were marked with permanent paint to track their growth, and temporarily enhanced with water to make them visible from a distance.

Figure 2-8. Failure number one. a) general view, b) detail of crack marking methods
2.4. SENSORS RESULTS AND ANALYSIS

Sensor setup

Pavement responses were measured with vertical LVDTs, horizontal LVDTs, and embedded strain gages. Temperature profiles were recorded with multi-depth thermocouples. The purpose of the vertical LVDTs were to measure deflections at the loaded edge at certain crack locations; the horizontal LVDTs measured crack opening; and the embedded gages indicated the strain in the pavement close to the surface as a result of the slab bending with the load. A detailed description of the sensors can be found in the construction report (Kohler et al.-2002).

Nomenclature for the sensors and location

The four embedded gages were located along the section at 12 ft. intervals and transversally at 4.5 feet from the edge. The exact location is presented in the first column of Table 2-3. The numbers in Table 2-3 correspond to the section distances, i.e., 0 to 85 feet markings. The initial letter identifies the type of sensor, and the suffix refers to the location of the sensor relative to the crack, East (e) or West (w), or to top (t), middepth (m) or bottom (b), depending on whether it’s a vertical or horizontal LVDT. Two vertical LVDTs were used to monitor deflections on each side of the crack in order to calculate the Load Transfer Efficiency (LTE). Horizontal LVDTs were installed across transverse cracks at different depths (top, middle, bottom) on the edge of the pavement.

<table>
<thead>
<tr>
<th>Strain gages (s)</th>
<th>Deflection sensors (D)</th>
<th>Crack opening sensors (O)</th>
</tr>
</thead>
<tbody>
<tr>
<td>s-24.6</td>
<td>D-21.6e</td>
<td>O-21.6t</td>
</tr>
<tr>
<td>s-37.2</td>
<td>D-21.6w</td>
<td>O-21.6m</td>
</tr>
<tr>
<td>s-53.0</td>
<td>D-29.0e</td>
<td>O-21.6b</td>
</tr>
<tr>
<td>s-63.1</td>
<td>D-29.0w</td>
<td>O-29.0t</td>
</tr>
<tr>
<td></td>
<td>D-34.0e</td>
<td>O-34.0t</td>
</tr>
<tr>
<td></td>
<td>D-34.0w</td>
<td>O-48.2t</td>
</tr>
<tr>
<td></td>
<td>D-57.1e</td>
<td>O-57.1t</td>
</tr>
<tr>
<td></td>
<td>D-57.1w</td>
<td>O-57.1b</td>
</tr>
</tbody>
</table>

Data collection, storage and processing

Signals coming from the sensors were collected in a synchronized manner with the passage of the loading wheel. For every pass of the wheel a complete set of responses was scanned from the sensors in order to determine the maximum, minimum and
unloaded responses. This scanning was performed every 0.5 inch along the section. The unloaded value corresponds to the sensor reading taken at the beginning of the pass, before the load had been applied. This allows for the determination of the rebound values and permanent deformation. Rebound values are defined as the difference between extreme responses (maximum or minimum) and the unloaded value, and represent the effect of the load during one pass. The maximum, minimum and unloaded values from all sensors were saved for each every pass. A time-history response was recorded for each sensor, as the wheel rolls over the section, every 10 to 20 passes.

Figure 2-9 shows the deflections measured with all the vertical sensors during a wheel pass. The wheel pass shown here occurred on August 27, when the load was 35 kips. As the test progressed, the deflections increased, as seen in Figure 2-10. The sensors depicted in Figure 2-11 were located at station 29.0.

![Figure 2-9. Deflections at different locations during a wheel pass](image)

![Figure 2-10. Increase in deflection during test](image)
The vertical deflections measured on both sides of the crack at station 29 were very similar, meaning good shear transfer efficiency.

Similarly to deflections, rebound strains increased as the test progressed, as shown in Figure 2-11. The different curves presented in Figure 2-12 are from the same set of passes presented in Figure 2-10, specifically they are from 6/30, 8/21, and 8/27. One of the strain gages embedded in section 2 did not work, and a second one worked intermittently.

![Figure 2-11. Strains collected as the test progressed.](image)

In the crack opening plot, when the wheel approaches the crack, the top sensor shows a small opening, but the crack closes when the wheel is near and on top of the crack. The opposite behavior is true for sensors at the bottom of the slab. Figure 2-12a shows crack opening from a sensor located near the top of the slab and Figure 2-12b is from a sensor near the bottom. A negative opening means actually closing of the crack, and it can be seen that the opening at the bottom can be an order of magnitude higher than the movements at the top. The different curves are again from the same set of passes presented above.
Figure 2-12. Crack opening along the test, a) top sensor, b) bottom sensor.

*Deflection results*

Figure 2-13 shows the crack rebound deflections as measured with the sensor located at station 21.6 for different load levels during the trafficking. The vertical lines denote the changes in load level. Figure 2-14 to Figure 2-19 show rebound deflections at the other 3 instrumented crack locations.
Deflections increased the most when the load was at 35 kips. At 30 kips the pavement was able to sustain repeated loading without major damage accumulation.
Sensors at station 34.0 feet (Figure 2-15) were installed later during the experiment, therefore their results are not available until after pass 60,000. In the case of the sensors at station 57.1, they seem to have had problems during the second half of the test, but they were fixed at approximately pass 116,000 (they are believed to have gone out of range and did not record the true increase around pass 70,000 to 116,000).
**Strain results**

Strains are measured close to the pavement surface (1 inch), at an approximate distance from the edge to capture the highest tensile strains in the transverse direction (4.5 ft.). To account for the maximum strain, which is experienced at the surface, the measured strain was amplified by a factor of 1.25 (4 inches from the neutral axis to the strain gage, but 5 inches to the surface). Figure 2-17 to Figure 2-19 present rebound strain for each of the working strain gages.

![Figure 2-17. Rebound strain at sensor s-37.2](image-url)
To obtain stress out of strain, the results have to be multiplied by the elastic modulus of concrete, which is about 7 million psi. This makes a value of 100 microstrains approximately equivalent to a 700 psi stress. The beam flexural strength is approximately 750 psi.
**Crack opening**

Crack opening was affected by changes in temperature, but more important by the deformations caused by the ATLAS wheel loading. A higher vertical deflection caused more horizontal compression near the slab surface, and a wider opening at the bottom of the slab. Figure 2-20 shows the results of crack opening at station 21.6 feet. The top sensor shows little movement, and it is more on the closing side than on the opening side of movement. The sensor at the middle shows opening and closing, and the sensor at the bottom indicates considerable openings.

Figure 2-20. Crack opening at station 21.6. a) top, b) middepth, and c) bottom
Station 21.6 is the only location with three horizontal sensors installed. Figure 2-21 presents the results from the two sensors at station 57.1, and Figure 2-22 shows results at the other three stations where there was only one sensor installed on each, at the top of the slab.

Figure 2-21. Crack opening at station 57.1. a) top, and b) bottom
Figure 2-22. Crack opening at top at station: a) 29.0. b) 34.0, and c) 48.2

**Load transfer efficiency**

Two values of load transfer efficiency are obtained in each wheel pass, the LTE on the approach side and the LTE on the leave side. LTE calculated with a rolling wheel is different from the LTE calculated with load at a fixed position as in the case of falling weight deflectometer. When loading is applied bi-directionally, there are four LTE values for each crack. An algorithm was programmed to obtain LTE from the deflection on both
sides of a crack, ensuring that calculation is performed at the instant when the load is only at one side of the crack. Figure 2-23 shows the average LTE values, which are high, for each of the four instrumented cracks.

![LTE Values Graphs](image)

**Figure 2-23. Load transfer efficiency**

**Temperatures**

Temperature in the pavement was measured with thermocouples at different depths and at different locations. Figure 2-24 shows top and bottom pavement temperatures on section 2 during the weeks of testing (measured one inch from top surface and one inch from bottom). The average pavement temperature during the test was approximately 85°F, with temperatures near the top exceeding 100°F during most days, and it was around 75°F during nights. Figure 2-25 shows the temperature difference in the depth of the pavement, which in general ranged from −5 to +25°F.
Temperatures presented in Figure 2-25 and Figure 2-26 were recorded with the static data collection system, with the sensors located at midpoint in the lane. Because of the absence of the protective tent, the shade of the ATLAS caused the pavement temperatures to vary along the width of the section. The machine covered one half of the lane, and the temperatures were always higher on the uncovered half because of the solar radiation. To
assess the differences, temperature was measured with the dynamic system at both sides, open and in-the-shade. The results are presented in Figure 2-26a (measured at one inch from the surface). The side exposed to direct sunlight was generally more than 30°F hotter during the daylight time, but during nighttime the difference decreased considerably. There is an appreciable trend as the summer was ending, and the heat from sunlight decreased. Figure 2-26b shows an hourly detail of the difference between sides, taken during the last week of August.

Figure 2-26. Temperature difference in the pavement caused by direct sunlight. a) along the weeks of testing, b) detail by hour of the day
2.5. FAILURE MECHANISM

Failure mechanism in section 2 was the same reported for section 1. Based on the geometry of the pavement failures and from the results of the instrumentation, the punchout fracture occurred as a result of permanent deformation of the support layers. In a typical punchout, the concrete segment that pushes down is delimited by two transverse cracks, a longitudinal crack, and the edge of the pavement. The punchouts observed in section 2, as the ones in section 1, are delimited by a single large crack and the edge of the pavement.

Section 2 presented more transverse cracks before test began which might have led to a possible different failure mechanism than section 1. Many of these transverse cracks, created by shrinkage, were clearly visible, and extended both horizontally and vertically across the slab. Load transfer efficiency did not appreciable decrease during loading, and therefore the segments in between transverse cracks continued working together.

One important factor observed in section 2 is the initiation of the half-moon cracks and their longitudinal advancement. As theory predicts, the longitudinal cracks seemed to have begun at a transverse crack, then they propagated to the next transverse cracks, and finally reached the edge. Figure 2-27 depicts this process.

The half moon cracks in section 2 opened more with loading than the cracks in section 1. The widening is an indication that the transverse bending of the slab was causing vertical propagation of the crack. The longitudinal cracks seem to have formed more slowly than in section 1. The longitudinal cracks sometimes were offset or mismatched when crossing a transverse crack. One last aspect in the formation of longitudinal cracks was the presence of “false start”, or longitudinal cracks that appeared early during the failure,
but then did not grow wider. Figure 2-28 illustrates the “false start”, the mismatch, and the widening phenomena.

**Figure 2-28. Development of longitudinal cracks.**

a) photo taken on Sept. 8, b) photo taken on Sept. 9.

**False starting:** the crack presented on the upper part in Figure 2-28 was the first to appear in the region, however it remained practically unchanged during the rest of the test. This crack was detected on August 27, and as shown in Figure 2-29, its width was about 0.08mm. The crack marked on red color in Figure 2-28 was the last one to be visible.
Mismatch: as it can be seen in Figure 2-28, the longitudinal crack is discontinuous across the transverse crack. This separation at other places ranged from a few millimeters to a couple of inches.

Widening: Figure 2-30 shows a detail of the pictures presented in Figure 2-28. Both photos were taken under unloaded conditions, and are evidence of a change in width from 0.5mm to 1.5mm caused by load repetition.

Transverse cracks remained tight even after the failure of the section. The presence of reinforcement keeps them tight, as opposed to longitudinal cracks where there is minimal transverse reinforcement. Figure 2-31 exemplifies this by showing a wide longitudinal crack and the transverse crack at station 29.0.
Figure 2-31. Example of difference in crack width between longitudinal and transverse cracks

The transverse cracks inside the failure area began exhibiting compression failure at the edge of the slab due to the high deflections. The location of the steel at 3.5 inches from the concrete surface caused the concrete to begin to spall at the transverse cracks. The occurrence of these spalling cracks along the section was presented in Figure 2-6, and a picture is shown in Figure 2-32.
Distressed zone

The most deteriorated region of the section was at around station 2.0ft, shown in Figure 2-33, where the surface of concrete loosened. This happened close to the beginning of the section, inside one of the punchouts. No forensic study was done, but the damage seems to be the consequence of cracks that cause horizontal delamination within the depth of the pavement.
Failure in terms of deflections

The sequence of events that explained failure of section 1 holds true for section 2. Basically the elastic deformation experienced by the slab, at the edge, and especially in front of transverse cracks, increased to a point where the fracture of the concrete occurred. In order for these high deflections to take place, a permanent deformation in the support layers have to occur. Even though the instrumentation was not set up to measure the permanent deformation, an estimation can be made from the unloaded deflection, and is presented in Figure 2-34.
Figure 2-34. Estimated permanent vertical deformation
3. SECTION THREE

3.1. TESTING DATES AND PAVEMENT CHARACTERISTICS

Testing on section 3 was performed between May 27 and August 4, 2004, after sections 4 and 5. Section 3 failed in a similar manner to sections 1 and 2, with extended punchouts. Section 3 consisted of a 10-in thick slab with a single layer of reinforcement located at 3.5 in below the surface. The amount of steel was 1.09% (the highest of all sections), which was achieved with #7 bars at 5.5 in spacing.

3.2. TESTING SEQUENCE

Load levels, amount of passes, and ATLAS main events

Environmental responses were collected for four days during the last week of May, before any loading was applied. Loading in the elastic range occurred on June 1st, for approximately 24 hours. The load level for testing at the elastic range was 10 kips and lasted for almost 8,000 wheel passes. The load was then increased to 30 kips, and kept at that level for approximately 154,000 passes. On August 3rd the load was increased to 55 kips to accelerate punchout failure, the first of which happened about 1,000 passes later. The 55-kip load was selected to try and impart fatigue damage to the section in a short amount of time since the 30-kip load used on the previous sections was not causing new cracking. The test stopped when the total number of passes was about 163,400.

At the end of July the hard drive started to fail in the computer that collects sensor data. It took several days to have the problem solved and the testing resumed on August 2nd.

The timeline of environmental and load testing in section 3 are shown in Figure 3-1. Figure 3-2 shows load levels versus accumulated passes. The loading is not continuous but completed over the 2.5 months of testing.

![Figure 3-1. Environmental and load testing versus time](image-url)
Several important load spectra tests were performed on this section, similar to sections 4 and 5. A load spectrum test consists of the application of a few passes at increasing load levels with the intention of determining the crack width at a given temperature. Higher loads cause greater closing movement of the CRCP cracks on the upper part of the slab, and the point where no more closing is observed is referred to as the true crack width. The load spectra test method needs to minimize the effect of changes in temperature, so it is performed within a 15 minute time period. Results of crack width calculated with this method were documented in a journal paper authored by Kohler and Roesler (2004).

**Conversion of passes into ESALs**

The conversion factors used to transform wheel passes into Equivalent Single Axle Loads (ESALs) are the same utilized in previous sections (Kohler and Roesler, 2005). These factors take into account the load levels and the fact that the load is applied near the edge instead of at the wheel path. The load factor is \((Lx/18)^{4.3}\), where \(Lx\) is the equivalent axle load. A wander magnification factor of 20 is used because of channelized edge loading. Figure 3-3 shows accumulation of calculated ESALs over time in section 3. The total calculated number of ESALs applied on section 3 was 626.8 millions.
3.3. CRACK PROGRESSION

Frequent crack surveys were performed during the testing of the section in order to capture any progression of cracking. Cracks were identified on the surface of the pavement as well as on the vertical plane along the loaded edge. No new transverse cracks developed during the testing of section 3. The longitudinal and half-moon shaped cracks did not appear until after the load was increased to 55 kips. Cracks are presented in Figure 3-4 as they became visible on the surface of the section. From the point where the number of ESALs was 590.1 M to the end of test (ESALs = 626.8 M) no new cracks developed on the surface.
Transverse cracks

There were 33 transverse cracks on section 3 at the time of testing. The average spacing was 2.6 feet. No new transverse cracks were created as consequence of traffic. In general the cracks were straight with little meandering, although some cracks presented sectors in which the crack was divided (see Figure 3-5). No traffic circulated over the divided areas and no crack deterioration was observed.
**Longitudinal cracks**

As presented in Figure 3-4, the longitudinal cracking started approximately between station 10 and 20 feet. That first longitudinal crack reached the edge at station 26 feet, but soon after it continued advancing, to form a second failed area. The longitudinal cracks are located at about 4 feet from edge. At the end of testing the first failure extended from station 0 to 26 feet and the second from about station 20 to 55 feet. No longitudinal crack developed in the last part of the section.

**Edge cracks**

The cracks at the edge were mostly vertical with little to moderate inclination. Spalling cracks were generated in about one third of all the vertical cracks, and they are the result of compression of the concrete above the neutral axis. A diagram of the cracks along the edge is presented in Figure 3-6. The inclined cracks at station 26.0 and 56.2 are where the half-moon cracks reach the edge. The ones at station 38.0 and 62.0 developed near the end of testing.

![Figure 3-6. Side view of cracks on the slab edge.](image)

Figure 3-7 shows a panoramic view of section 3, including crack instrumentation at the four locations. Figure 3-8 shows the longitudinal cracks as viewed from three different points in the section. Crack visibility was enhanced with water.
Figure 3-7. Panoramic view of section 3

Figure 3-8. Longitudinal cracks. a) picture taken from station 22.0, b) from station 34.0, c) from station 53.0
3.4 SENSOR RESULTS AND ANALYSES

Sensor setup

Pavement responses were measured with vertical LVDTs, horizontal LVDTs, and embedded strain gages. Temperature profiles were recorded with multi-depth thermocouples. The purpose of the vertical LVDTs was to measure deflections at the loaded edge at certain crack locations; the horizontal LVDTs measured crack opening; and the embedded gages indicated the strain in the pavement close to the surface as a result of the slab bending with the load. A detailed description of the sensors can be found in the construction report (Kohler et al., 2002). Crack instrumentation in section 3 used the same amount of sensors used in section 4, which is higher than the other sections.

Nomenclature for the sensors and location

The four embedded strain gages were located along the section at 12 ft. intervals and transversally at 4.5 feet from the edge. The exact location is presented in the first column of Table 3-1. The numbers in Table 3-1 correspond to the section distances, i.e., 0 to 85 feet markings. Three of the four strain gages were not functioning (readings could be obtained only from sensor s-60.6). Four cracks were instrumented with each crack containing two vertical LVDTs, one on each side of the crack, in order to calculate the Load Transfer Efficiency (LTE) and four horizontal LVDTs at different depths (top, mid-top, mid-bottom, and bottom) on the edge of the pavement. The initial letter identifies the type of sensor, and the suffix refers to the location of the sensor relative to the crack, East (e) or West (w), or top (t), mid-top (mt), mid-bottom (mb) or bottom (b), depending on whether it’s a vertical or horizontal LVDT.

<table>
<thead>
<tr>
<th>Strain gages (s)</th>
<th>Deflection sensors (D)</th>
<th>Crack opening sensors (O)</th>
</tr>
</thead>
<tbody>
<tr>
<td>s-23.4</td>
<td>D-14.9-e, D-14.9-w</td>
<td>O-14.9-t, O-14.9-mt, O-14.9-mb, O-14.9-b</td>
</tr>
<tr>
<td>s-36.8</td>
<td>D-23.4-e, D-23.4-w</td>
<td>O-23.4-t, O-23.4-mt, O-23.4-mb, O-23.4-b</td>
</tr>
<tr>
<td>s-48.7</td>
<td>D-47.3-e, D-47.3-w</td>
<td>O-47.3-t, O-47.3-mt, O-47.3-mb, O-47.3-b</td>
</tr>
<tr>
<td>s-60.6</td>
<td>D-57.2-e, D-57.2-w</td>
<td>O-57.2-t, O-57.2-mt, O-57.2-mb, O-57.2-b</td>
</tr>
</tbody>
</table>

Data collection, storage and processing

Signals coming from the sensors were collected in a synchronized manner with the passage of the loading wheel. For every pass of the wheel a complete set of responses
was scanned from the sensors in order to determine maximum, minimum and unloaded responses. This scanning was performed every one inch along the section. The unloaded value corresponds to the sensor reading taken at the beginning of the pass, before the load had been applied. This allows for the determination of the rebound values and permanent deformation. Rebound values are defined as the difference between extreme responses (maximum or minimum) and the unloaded value, and represent the effect of the load during one pass. The maximum, minimum and unloaded values from all sensors were saved for each pass. A time-history response was recorded for each sensor, as the wheel rolled over the section, every 10 to 20 passes.

Figure 3-9 shows the deflections measured with all the vertical sensors during a wheel pass. The wheel pass shown here occurred on June 23, when the load was 30 kips. Similar influence lines were obtained from the strain gages and the horizontal displacement sensors. Figure 3-10 shows the opening and closing movement at the top and bottom of the slab under a 30-kip load at the cracks located at station 23.4 and 57.2 feet.

![Figure 3-9. Deflections at different locations during a wheel pass](image-url)
**Deflection results**

Figure 3-11 shows the crack rebound deflections as measured with the sensor located at station 14.9 for different load levels during the trafficking. The vertical lines denote the changes in load level. Figure 3-12 to Figure 3-14 show rebound deflections at the other 3 instrumented crack locations.
Figure 3-11. Rebound deflections sensor D-14.9

Figure 3-12. Rebound deflections sensor D-23.4
Deflections at the crack located at 14.9 were always higher than at the other instrumented cracks. Deflection at cracks 23.4 and 47.3 were always similar to each other. Deflection at crack 57.2 was in between the other but became the smallest at the end of the test, when the extended punchouts affected the results in the other three cracks. The difference between crack 14.9 and the other increased as the loading progressed, as shown in Table 3-2. A comparison of deflection at all the cracks is presented in Figure 3-15.
Deflections measured in the morning were generally between 0.1 and 0.2mm higher than in the afternoon hours (maximum difference), and it is attributed to curling. The temperature on the surface of the slab were similar in the early morning to temperature at the bottom, but were about 7°F higher at 5 pm.

**Strain results**

Strains are measured 1 inch from the pavement surface, and at 4.5 feet from the edge to capture the highest tensile strains in the transverse direction. To account for the maximum strain, which is experienced at the surface, the measured strain was amplified by a factor of 1.25 (5 inches from the neutral axis to the surface, but 4 inches to the strain gage,). Figure 3-16 presents rebound strain for the strain gages.
When the load was increased from 10 to 30 kips, the strains responded accordingly, but that was not the case when the load was increased to 55 kips. The strain remained about the same, between 60 and 80 microstrain. The fact that the strain decreased when the test was stopped could be an indication of propagation of the longitudinal crack, given that the second punchout extended to about station 55 and the gage was at station 60.6. To estimate stress from the strain, the results (in strains) have to be multiplied by the elastic modulus of concrete, which is about 7 million psi.

**Crack opening**

Figure 3-17 and Figure 3-18 show the opening and closing movement at each sensor. The following observations are important.

- Horizontal crack movements were affected by average pavement temperature and temperature differential, and that is especially clear over the long period of constant loading at 30 kips.
- The major responses to load are crack closing near the surface (“min” lines in the upper most plots) and crack opening near the bottom of the slab (“max” lines in the lowest plots).
Figure 3-17. Crack opening at various depth at stations 14.9 and 23.4
Figure 3-18. Crack opening at various depth at stations 47.3 and 57.2

A 83
Load transfer efficiency

Two values of load transfer efficiency are obtained in each wheel pass, the LTE on the approach side and the LTE on the leave side of cracks. LTE calculated with a rolling wheel is different from the LTE calculated from a fixed position loading like FWD. When loading is applied bi-directionally, there are four LTE values for each crack. An algorithm was programmed to obtain LTE from the deflection on both sides of a crack, ensuring that the calculation is performed at the instant when the load is only at one side of the crack. Figure 3-19 shows the average LTE values for each of the four instrumented cracks. LTE is higher than 95% in all the cracks.

![LTE Graphs](image)

Figure 3-19. Load transfer efficiency

Temperatures

Pavement temperatures were measured with thermocouples at different depths and at different locations. Figure 3-20 shows top and bottom pavement temperatures on section 3 recorded with the static data collection system during the weeks of testing (measured one inch from top surface and three inches from bottom). The average pavement
temperature during the test was approximately 73°F, with temperatures at the top oscillating between 70 and 80°F most of the days.

![Graph showing pavement temperatures at top and bottom.](image)

**Figure 3-20.** Pavement temperatures at top and bottom

### 3.5. FAILURE MECHANISM

Failure mechanism in section three was the same reported for sections 1 and 2. Based on the geometry of the pavement failures and from the results of the instrumentation, the punchout fracture occurred as a result of permanent deformation of the support layers. In a typical punchout, the concrete segment that pushes down is delimited by two transverse cracks, a longitudinal crack, and the edge of the pavement. The punchouts observed in
section 3, as the ones before, are delimited by a single large crack and the edge of the pavement.

Figure 3-21 shows that there is some discontinuity on the surface at the longitudinal crack when it crosses the transverse cracks.

![Figure 3-21. Longitudinal crack in front of instrumented crack at station 14.9.](image)

Load transfer efficiency did not appreciably decrease during loading, and therefore the segments in between transverse cracks continued working together. Even at the surface of the pavement the transverse cracks looked very tight, 0.2 mm or less (see Figure 3-22)
Figure 3-23 shows that the transverse cracks did not present signs of damage on the surface caused by loading, although spalling cracks appeared on the edge. Spalling cracks at station 30.0 and 31.5 are shown in Figure 3-24 and the diagram with the spalling cracks along the entire edge was presented in Figure 3-6.
Figure 3-23. Expanded view of crack at 23.4.

Figure 3-24. Spalling cracks at station 30.0 and 31.5.
4. SECTION FOUR

4.1. INTRODUCTION

Testing on section 4 was performed between January 17 and March 6, 2004. Compared with the two previously loaded sections (1 and 2), section 4 did not structurally fail. The test was terminated after the number of accumulated load repetitions and ESALs was considered enough to compare performance with the other sections and to balance the interest of further loading in section 4 with the need to continue testing the remaining sections. The concrete slab thickness in section 4 is 14 in. as compared with the 10-in thick slab tested in sections 1 and 2. The protective tent was moved over section 4 and allowed for testing during the winter months. Section 4 consists of a single layer of reinforcement steel bars located at 4.5 inches below the surface, totaling 0.78% of steel in the cross section (#7 bars at 5.5-in. spacing).

4.2. TESTING SEQUENCE

Load levels, amount of passes, and ATLAS main events

The ATLAS was run for the first time on section 4 during the first week of January, two weeks after the tent was moved into place. This time was used to prepare the section by assembling the machine and installing the required instrumentation. Some initial passes were run at 10 kips to verify the machine operation and pavement sensors. Before mechanical loading commenced, environmental responses were collected for three days (66 hours). Approximately 10,000 passes were then applied at 10 kips to capture the elastic responses of the section without damaging the pavement section. When the 10-kip test was finished, the load was then increased to 35 kips, and kept at that level for approximately 25,000 passes, after which it was increased to 45 kips. The 45-kip load was selected to try and impart fatigue damage to the section in a reasonable amount of time since a 30-kip load used on the previous sections would likely not fail the 14-in test section. The 45-kip loading did not produce significant fatigue damage on the pavement section, but caused wearing problems on the machine. The loading was changed to unidirectional and maintained at 45 kips, but the excessive wearing on one of the guide rails and corresponding wheels prevented more loading. The last two weeks of February were spent in fabricating and installing a machine retrofit, and the 45-kip load was resumed on March 3. A total of more than 25,000 passes were applied at the 45-kip level, and the load was then increased to 55 kips. The retrofit did not solve the wearing problem, which was exacerbated by the new higher load. Less than 4,000 passes at 55 kips had been applied at the time the testing was suspended, of which one half was done with unidirectional loading. The aircraft wheel was used throughout the testing on section 4 and had to be replaced at the cessation of testing due to excessive tire wear.

The timeline of environmental and load testing in section 4 is shown in Figure 4-1. Figure 4-2 shows load levels versus accumulated passes. The loading is not continuous but completed over the 2.5 months of testing.
Several important load spectra tests were performed on this section. A load spectrum test consists of the application of a few passes at increasing load levels with the intention of determining the crack width at a given temperature. Higher loads cause greater closing movement of the CRCP cracks on the upper part of the slab, and the point where no more closing is observed is referred to as the true crack width. The load spectra test method needs to minimize the effect of changes in temperature, so it is performed within a 15-minute time period. Results of crack width calculated with this method are documented in a journal paper authored by Kohler and Roesler (2004).
Conversion of passes into ESALs

The conversion factors used to transform from wheel passes into Equivalent Single Axle Loads (ESALs) are the same utilized in sections 1 to 3. These factors take into account the load levels and the fact that the load is applied near the edge instead of at the wheel path. The load factor is \((\frac{Lx}{18})^{4.3}\), where \(Lx\) is the equivalent axle load. A wander magnification factor of 20 is used because of channelized edge loading. Figure 4-3 shows accumulation of calculated ESALs over time in section 4.

![Figure 4-3. Accumulated ESALs versus time](image)

4.3.Crack Progression

Frequent crack surveys were performed during the testing of the section in order to capture any progression of cracking. Cracks were identified on the surface of the pavement as well as on the vertical plane along the loaded edge. No new cracks were developed during the testing of section 4. Cracks as they appeared on the surface of the section are presented in Figure 4-4.

![Figure 4-4. Transverse Cracks on section 4](image)
**Transverse cracks**

There were approximately 15 transverse cracks on section 4 at the time of testing. Seven of these cracks had occurred between station 22 and 33 feet, forming a cluster of cracks with an average spacing of less than 1 foot. It is not clear why cluster cracks form, but they are associated with construction related variability (i.e. depth of steel cover and concrete strength). Many cracks were discontinuous along the width of the slab and they remained discontinuous despite the aggressive loading.

**Longitudinal cracks**

No longitudinal cracks developed in section 4.

**Edge cracks**

The cracks at the edge were mostly vertical with little to moderate meandering. There was no spalling nor oblique cracks as seen in previous sections. A diagram of the cracks along the edge is presented in Figure 4-5.

![Figure 4-5. Side view of cracks on the slab edge.](image-url)
Figure 4-6 shows the region of cluster cracks as seen from the south side (the deflection measurement support system can be seen at crack 24.1 ft. and 28.8 ft.). The cracks were enhanced with water to make them visible from a distance. Figure 4-7 shows the instrumented crack at station 63.1 and the crack at station 65.1. The surface at the center of the slab became colored due to rusting of the metal dust that fell from the ATLAS’s rail because of wearing.
Figure 4-7. Instrumented crack at station 63.1 and crack at station 65.1.
4.4 SENSOR RESULTS AND ANALYSES

Sensor setup

Pavement responses were measured with vertical LVDTs, horizontal LVDTs, and embedded strain gages. Temperature profiles were recorded with multi-depth thermocouples. The purpose of the vertical LVDTs was to measure deflections at the loaded edge at certain crack locations; the horizontal LVDTs measured crack opening; and the embedded gages indicated the strain in the pavement close to the surface as a result of the slab bending with the load. A detailed description of the sensors can be found in the construction report (Kohler et al., 2002). One difference between the instrumentation on section 4 with respect to the other sections is that more horizontal LVDTs were used, allowing the study of crack opening at four instead of three depths in the slab. An extra datalogger module was acquired for that purpose in order to expand the number of available channels.

Nomenclature for the sensors and location

The four embedded strain gages were located along the section at 12-ft. intervals and transversally at 4.5 feet from the edge. The exact location is presented in the first column of Table 4-1. The numbers in Table 4-1 correspond to the section distances, i.e., 0 to 85 feet markings. Only three of the four strain gages were functioning (no readings could be obtained from sensor s-51.0). Four cracks were instrumented with each crack containing two vertical LVDTs, one on each side of the crack, in order to calculate the Load Transfer Efficiency (LTE) and four horizontal LVDTs at different depths (top, mid-top, mid-bottom, and bottom) on the edge of the pavement. The initial letter identifies the type of sensor, and the suffix refers to the location of the sensor relative to the crack, East (e) or West (w), or top (t), mid-top (mt), mid-bottom (mb) or bottom (b), depending on whether it’s a vertical or horizontal LVDT.

<table>
<thead>
<tr>
<th>Strain gages (s)</th>
<th>Deflection sensors (D)</th>
<th>Crack opening sensors (O)</th>
</tr>
</thead>
<tbody>
<tr>
<td>s-51.0</td>
<td>D-44.0-e, D-44.0-w</td>
<td>O-44.0-t, O-44.0-mt, O-44.0-mb, O-44.0-b</td>
</tr>
<tr>
<td>s-63.4</td>
<td>D-63.1-e, D-63.1-w</td>
<td>O-63.1-t, O-63.1-mt, O-63.1-mb, O-63.1-b</td>
</tr>
</tbody>
</table>
Data collection, storage and processing

Signals coming from the sensors were collected in a synchronized manner with the passage of the loading wheel. For every pass of the wheel a complete set of responses was scanned from the sensors in order to determine maximum, minimum and unloaded responses. This scanning was performed every one inch along the section. The unloaded value corresponds to the sensor reading taken at the beginning of the pass, before the load had been applied. This allows for the determination of the rebound values and permanent deformation. Rebound values are defined as the difference between extreme responses (maximum or minimum) and the unloaded value, and represent the effect of the load during one pass. The maximum, minimum and unloaded values from all sensors were saved for each pass. A time-history response was recorded for each sensor, as the wheel rolled over the section, every 10 to 20 passes.

Figure 4-8 shows the deflections measured with all the vertical sensors during a wheel pass. The wheel pass shown here occurred on February 11, when the load was 45 kips. Similar influence lines were obtained from the strain gages and the horizontal displacement sensors. Figure 4-9 shows the opening and closing movement at the top and bottom of the slab under a 45-kip load at the crack located at station 44.0 feet.

![Figure 4-8. Deflections at different locations during a wheel pass](image-url)
**Deflection results**

Figure 4-10 shows the crack rebound deflections as measured with the sensor located at station 24.1 for different load levels during the trafficking. The vertical lines denote the changes in load level. Figure 4-11 to Figure 4-13 show rebound deflections at the other 3 instrumented crack locations. A comparison between deflections in each crack is presented in Figure 4-14. It can be seen that deflections were similar at all locations when the loading was at 10 kips, then deflections at 24.1 were higher than at the other locations during loading at 35 kips. At the end of the test deflection values at the two cracks in the zone of cluster cracking (24.1 and 28.8) were similar to each other and higher than at the other cracks.
Figure 4-10. Rebound deflections sensor D-24.1

Figure 4-11. Rebound deflections sensor D-28.8
Figure 4-12. Rebound deflections sensor D-44.0

Figure 4-13. Rebound deflections sensor D-63.1
**Strain results**

Strains are measured 1 inch from the pavement surface, and at 4.5 feet from the edge to capture the highest tensile strains in the transverse direction. To account for the maximum strain, which is experienced at the surface, the measured strain was amplified by a factor of 1.167 (7 inches from the neutral axis to the surface, but 6 inches to the strain gage,). Figure 4-15 to Figure 4-17 present rebound strain for each of the working strain gages.
The strain levels recorded at sensor s-25.7 were higher than at sensor s-38.9 because the first one was located at about 4 inches from a transverse crack while the second one was a few feet from a crack. Sensor s-63.4 was located very close to a crack (2 or 3 inches) and presented higher scatter. To estimate stress from the strain, the results (in strains) have to be multiplied by the elastic modulus of concrete, which is about 7 million psi.
Crack opening

Figure 4-18 and Figure 4-19 show the opening and closing movement at each sensor. The following observations are important.

- Horizontal crack movements were strongly affected by average pavement temperature and temperature differential which fluctuated significantly during the January to March testing time.
- The major responses to load are crack closing near the surface and crack opening near the bottom of the slab.
- Figure 4-20 for crack movements at station 44.0 gives a clear example how the increase in both the average pavement temperature and temperature differential affected the crack closing at the top and the opening at the bottom of the slab.
Figure 4-18. Crack opening at various depth at stations 24.1 and 28.8
Figure 4-19. Crack opening at various depth at stations 44.0 and 63.1
Figure 4-20. Crack opening at time of temperature increase

Load transfer efficiency

Two values of load transfer efficiency are obtained in each wheel pass, the LTE on the approach side and the LTE on the leave side of cracks. LTE calculated with a rolling wheel is different from the LTE calculated from a fixed position loading like FWD.
When loading is applied bi-directionally, there are four LTE values for each crack. An algorithm was programmed to obtain LTE from the deflection on both sides of a crack, ensuring that the calculation is performed at the instant when the load is only at one side of the crack. Figure 4-21 shows the average LTE values for each of the four instrumented cracks. LTE is approximately 90% in each of the first three cracks, and 95% in the crack at station 63.1. An increase in LTE is seen at about pass 55,000 as a consequence of higher pavement temperatures.

Figure 4-21. Load transfer efficiency

**Temperatures**

Pavement temperatures were measured with thermocouples at different depths and at different locations. Figure 4-22 shows top and bottom pavement temperatures on section 4 recorded with the static data collection system during the weeks of testing (measured one inch from top surface and three inches from bottom). The average pavement temperature during the test was approximately 35°F, with temperatures at the top reaching below 30°F some days, and exceeding 50°F at the end of the test.
4.5. FAILURE MECHANISM

Failure was not attained during testing of section 4.
5. SECTION FIVE

5.1. INTRODUCTION
Section 5 was tested for elastic responses from wheel and environmental loading only. No trafficking by the ATLAS was completed at load levels that would cause fatigue damage to the CRC pavement. Fatigue damage load levels were not used because of excessive wearing occurring on the ATLAS with loads higher than 35 kips. The response testing on section 5 was completed from April 12-15, 2004.

Section 5 consisted of a 14-inch concrete slab, with two layers of reinforcement steel bars located at 3.5 and 7.0 inches below the surface, totaling 0.78% of steel in the cross section (#7 bars at 5.5 in. spacing).

The protective tent remained on section 4, knowing that the response testing would not take long. Since sections 4 and 5 were contiguous, the ATLAS was partially covered by the tent as shown in Figure 5-1 and Figure 5-2.
5.2. TESTING SEQUENCE
After the instrumentation was installed, environmental responses were collected for seven
days. After that, a total of about 1,800 passes were accumulated with the ATLAS. The
initial 50 passes were applied with a 10-kip load, and the rest was applied with a 35-kip
load. Other load levels were also used, but for only a very limited number of passes, as
part of load spectra tests. Similar to section 4, a load spectra test sequence was used and
consisted of applying a few passes at increasing load levels over the test section for the
purpose of determining the crack width of the instrumented transverse cracks.

Response testing was stopped occasionally in order to inspect the machine regarding the
ATLAS wearing problem. From pass number 320 to pass 1,800 uninterrupted constant
load level was applied from early morning hours until approximately 3pm.

Figure 5-3 and Figure 5-4 present a general description of the load levels and test
sequence versus time and passes, respectively.
Figure 5-3. Environmental and load testing versus time

Figure 5-4. Load versus passes

Conversion of passes into ESALs

The conversion factors used to transform from wheel passes into Equivalent Single Axle Loads (ESALs) are the same utilized in previous sections. These factors take into account the load levels and the fact that the load is applied near the edge instead of at the
wheel path. The load factor is $(Lx/18)^{4.3}$, where $Lx$ is the equivalent axle load. A wander magnification factor of 20 is used because of channelized edge loading. Figure 5-5 shows accumulation of calculated ESALs over time in section 5, which reached almost 12.5 millions.

![Figure 5-5. Accumulated ESALs versus time](image)

**5.3.Crack Progression**
Crack surveys were performed during the testing of the section in order to capture any progression of cracking. Cracks were identified on the surface of the pavement as well as on the vertical plane along the loaded edge. No new cracks were developed during the testing of section 5. Cracks as they appeared on the surface before and after testing are presented in Figure 5-6. Section 5 is at the end of the lane and therefore it is expected a limited number of transverse cracks should exist due to the lugs not being perfectly fixed.

![Figure 5-6. Cracks on section 5](image)
**Transverse cracks**

There was only one transverse crack, located at station 14.5 in the 85-foot section, plus partial surface cracks at stations 27.9 and 76.5, and several other locations on the unloaded edge. These cracks are likely from concrete surface shrinkage.

**Longitudinal cracks**

No longitudinal cracks developed in section 5.

**Edge cracks**

The only crack was not perfectly vertical on the edge. A diagram of the crack along the edge is presented in Figure 5-7.

![Figure 5-7. Crack along the edge.](image)

Figure 5-8 and Figure 5-9 show the transverse crack at station 14.5 on the top of the slab and along the slab edge.
Figure 5-8. Transverse crack on pavement surface (enhanced photo)

Figure 5-9. Crack as seen on the edge (enhanced photo)
5.4. SENSORS RESULTS AND ANALYSIS

Sensor setup

Pavement responses were measured with vertical LVDTs, horizontal LVDTs, and embedded strain gages. Temperature profiles were recorded with multi-depth thermocouples. The purpose of the vertical LVDTs was to measure deflections at the loaded edge at certain crack locations; the horizontal LVDTs measured crack opening; and the embedded gages indicated the strain in the pavement close to the surface as a result of the slab bending with the load. A detailed description of the sensors can be found in the construction report (Kohler et al., 2002).

Nomenclature for the sensors and location

The numeric part of the sensor names in Table 3-1 corresponds to their location within the section, i.e., 0 to 85 feet markings. Only one of the four embedded strain gages was functioning (no readings could be obtained from the other sensors). Although there is only one full-width transverse crack in section 5, two locations were instrumented. Location 14.5 is at the only transverse crack that extends across the pavement width, while location 27.9 correspond to a spot where a crack only extends partially across the slab. As always, two vertical LVDTs were used to monitor deflections, one on each side of the crack, in order to calculate the Load Transfer Efficiency (LTE). Four horizontal LVDTs were installed at each location at different depths (top, mid-top, mid-bottom, and bottom) on the edge of the pavement. The initial letter identifies the type of sensor, and the suffix refers to the location of the sensor relative to the crack, East (e) or West (w), or top (t), mid-top (mt), mid-bottom (mb) or bottom (b), depending on whether it is a vertical or horizontal LVDT.

Table 5-1. Location of sensors and nomenclature

<table>
<thead>
<tr>
<th>Strain gages (s)</th>
<th>Deflection sensors (D)</th>
<th>Crack opening sensors (O)</th>
</tr>
</thead>
<tbody>
<tr>
<td>s-61.7</td>
<td>D-14.5-e, D-14.5-w</td>
<td>O-14.5-t, O-14.5-mt, O-14.5-mb, O-14.5-b</td>
</tr>
<tr>
<td></td>
<td>D-27.9-e, D-27.9-w</td>
<td>O-27.9-t, O-27.9-mt, O-27.9-mb, O-27.9-b</td>
</tr>
</tbody>
</table>

Data collection, storage and processing

Signals coming from the sensors were collected in a synchronized manner with the passage of the loading wheel. For every pass of the wheel a complete set of responses was scanned from the sensors in order to determine maximum, minimum and unloaded responses. This scanning was performed every one inch along the section. The unloaded
value corresponds to the sensor reading taken at the beginning of the pass, before the load had been applied. This allows for the determination of the rebound values and permanent deformation. Rebound values are defined as the difference between extreme responses (maximum or minimum) and the unloaded value, and represent the effect of the load during one pass. The maximum, minimum and unloaded values from all sensors were saved for each pass. A time-history response was recorded for each sensor, as the wheel rolled over the section, every 10 to 20 passes.

Figure 5-10 shows the deflections measured with all the vertical sensors during wheel passes at the two different load levels. One of the wheel passes shown here occurred at 9:00am on April 13, when the load was 10 kips. The other one was at 6:00am on April 15, with a 35-kip load. Similar influence lines were obtained from the strain gage and the horizontal displacement sensors.

![Figure 5-10. Deflections at different locations during wheel passes at 10 and 35-kip load](image)

**Deflection results**

Figure 5-11 shows rebound deflections measured at the crack during the trafficking. The vertical line denotes the change in load level from 10 to 35 kips. The reduction in deflection after pass 1200 is probably caused by the increasing temperature that curls the slab downward and an increase in load transfer efficiency. Figure 5-12 shows rebound deflections at the other location.
Figure 5-11. Rebound deflections sensor D-14.5

Figure 5-12. Rebound deflections sensor D-27.9
Strain results

Strains are measured 1 inch under the pavement surface, and at 4.5 feet from the edge to capture the highest tensile strains in the transverse direction. To account for the maximum tensile strain, which is experienced at the surface, the measured strain was amplified by a factor of 1.167 (7 inches from the neutral axis to the surface, but 6 inches to the strain gage). Figure 5-13 presents rebound strain.

To estimate stress from the strain, the results (in strains) have to be multiplied by the elastic modulus of concrete, which is about 7 million psi.

Crack opening

Horizontal movement was detected at the four instrumented depths. Figure 5-14 shows that bottom and mid-bottom sensors reveal positive movement (opening), while top and mid-top sensors reveal negative movement (closing) when the load is directly over the crack. This particular data set is from wheel pass 850, which occurred at 6:00am on April 15, with a 35-kip load. The curves in Figure 5-15 come from another pass with the same load level, but that took place later that day when the pavement temperature was higher and temperature difference was greater. It can be seen that closing at top of the crack

Figure 5-13. Rebound strain at sensor s-61.7
was reduced considerably. The other observation is the behavior of mid-top sensor, which changed from closing under load to slight opening under load.

Figure 5-14. Horizontal crack movement at four sensors, early morning

Figure 5-15. Horizontal crack movement at four sensors, afternoon
Figure 5-16. Horizontal movement at various depth at station 14.5 and 27.9
Figure 5-16 presents the maximum opening and closing movement versus pass number at for each sensor. The horizontal sensor at station 27.9 revealed the absence of a crack even though the top sensor picked up movement. That movement is attributed to tension/compression strain in the concrete and a surface crack caused by shrinkage.

**Load transfer efficiency**

Figure 5-17 shows the LTE values calculated as the average of LTE on the approach and leave side of the crack. LTE on the other instrumented location was 100%.

![Figure 5-17. Load transfer efficiency](image-url)
Temperatures

Figure 5-18 shows top and bottom pavement temperatures on section 5 recorded with the static data collection system during the days of testing (measured one inch from top surface and three inches from bottom of the slab). The average pavement temperature during the test was approximately 57°F, with temperatures at the top ranging from 40 to 80°F.

5.5. FAILURE MECHANISM

Failure was not attained during testing of section 5.


6. SUMMARY OF TESTED SECTIONS

This part of the document is aimed to summarize and compare the conditions and main results from pavement sections subjected to accelerated loading testing. Sections 1, 2 and 3 failed in almost identical form, which can be described as extended punchouts, connected to each other. Section 4 did not fail under accelerated testing with similar total traffic loading as the other sections. Total ESALs applied, pavement temperature, and other pertinent testing information is presented in Table 6-1.

Table 6-1. Summary of tested sections

<table>
<thead>
<tr>
<th></th>
<th>Section 1</th>
<th>Section 2</th>
<th>Section 3</th>
<th>Section 4</th>
<th>Section 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Duration of testing</td>
<td>12 months (6/16/02-23/6/03)</td>
<td>11 weeks (6/30/03-9/13/03)</td>
<td>10 weeks (5/26/04-8/4/04)</td>
<td>9 weeks (1/3/04-3/6/04)</td>
<td>2 weeks (4/6/04-4/15/04)</td>
</tr>
<tr>
<td>Total load repetitions</td>
<td>246,800</td>
<td>118,600</td>
<td>163,400</td>
<td>64,300</td>
<td>1,800</td>
</tr>
<tr>
<td>Total ESALs</td>
<td>911 M</td>
<td>778 M</td>
<td>627 M</td>
<td>764 M</td>
<td>12.5 M</td>
</tr>
<tr>
<td>Approx ESALs at first failure</td>
<td>511 M</td>
<td>230 M</td>
<td>548 M</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Maximum load applied</td>
<td>50 Kips</td>
<td>50 Kips</td>
<td>55 Kips</td>
<td>55 Kips</td>
<td>35 Kips (3)</td>
</tr>
<tr>
<td>Pavement temperature range</td>
<td>34-80 °F (2)</td>
<td>75-95°F</td>
<td>64-80°F</td>
<td>25-50°F</td>
<td>40-65°F</td>
</tr>
<tr>
<td>Maximum rebound deflection</td>
<td>3.2mm (at 50kips)</td>
<td>4.4mm (at 50kips)</td>
<td>4.2mm (at 55kips)</td>
<td>1.1mm (at 55kips)</td>
<td>0.5mm (at 35kips)</td>
</tr>
<tr>
<td>Failure description</td>
<td>Extended punchouts</td>
<td>Extended punchouts</td>
<td>Extended punchouts</td>
<td>Section did not fail</td>
<td>Response loading only</td>
</tr>
</tbody>
</table>

(1) Includes time when no load was being applied.
(2) Most of the effective test was done during June 2003, when temperature was 60-80 °F
(3) Loads up to 55 kips were used, but less than 10 passes were applied at each load level higher than 35 kips

A map of the cracks as at the end of each test is shown in Figure 6-1.
Figure 6-1. Final crack mapping of test sections