

# Effect of Y-cracking on CRCP Performance

## **ANNUAL REPORT FOR FY 2011**

ODOT SPR ITEM NUMBER 2230

### **Submitted to:**

John Bowman, P.E.  
Planning and Research Division Engineer  
Oklahoma Department of Transportation  
200 N.E. 21<sup>st</sup> Street  
Oklahoma City, Oklahoma 73105

### **Submitted by:**

Tyler Ley and Spencer Woestman  
Oklahoma State University  
Civil and Environmental Engineering  
207 Engineering South  
Stillwater, Oklahoma 74078

Kyle Riding and Wesley Nyberg  
Kansas State University  
Civil and Environmental Engineering  
2118 Fiedler Hall  
Manhattan, Kansas 6606



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# CHAPTER I

## INTRODUCTION

### 1.1 Overview

This document is an update of the progress of the research on ODOT project 2230 “Effect of Y-cracking on CRCP Performance (PS-14)”. This report summarizes the work that was completed at Oklahoma State University between October 1<sup>st</sup>, 2010 and September 30<sup>th</sup>, 2011. The focus of this project is to determine if a correlation exists between Y-cracking and the subsequent performance of continuously reinforced concrete pavements (CRCP) in Oklahoma. Work will also be done to correlate the y-cracking and design and construction variables. It was decided to best accomplish this goal by completing the following tasks:

- A. (OSU and KSU) Literature review of both previous national reports and papers and ODOT reports to determine previous experience with Y-cracking, mitigation methods used, and potential future cost-effective solutions to prevent Y-cracking.
- B. (OSU) Update the ODOT CRCP project database. In this task the investigators will update the ODOT CRCP database for projects constructed since 2003 (McGovern Personal Communication, 2010). The type of information needed for the database includes year constructed, percentage of longitudinal and transverse steel, location, type of shoulder, type of base and subbase, edge drain presence, ODOT standards.
- C. (OSU) Review pavement management condition data to determine current and previous performance levels of CRCP with and without Y-cracking. The pavement system design, construction records, concrete quality control tests, aggregate type used, time and dates of

concrete placement, approximate time of Y-cracking and curing methods used will be gathered with the assistance of ODOT. This information will be used in conjunction with the modeling in Task E.

- D. (OSU) Perform visual inspections as needed of ODOT CRCP projects. Field investigations will document sections with and without Y-cracking, the number of punchouts and patches/mile, average crack spacing, crack standard deviation, punchouts and spalls located at Y-cracking locations. The field investigations will also document patch locations, for possible comparison to past and future patch locations.
- E. (KSU) The early-age stress development and time to first cracking for the pavements evaluated in this study will be modeled using the software package HIPERPAV III by The Transtec Group and FHWA. Figure 4 shows a plot of the tensile stress versus the tensile strength modeled using HIPERPAV III for a concrete pavement in Oklahoma City. The time of first cracking will be modeled and compared to the documented occurrence of early-age Y-cracking in the CRCP evaluations. The software has the ability to then compare the impact of different base type, concrete CoTE, providing additional curing, such as using a double coat of curing compound, or placing at night. However, HIPERPAV III does not output the pavement temperature development from the heat of hydration of the concrete in the pavement. To do this the ConcreteWorks software program developed by Dr. Kyle Riding of Kansas State, a Co-PI on this project, while at the University of Texas in collaboration with Auburn University and the Texas Department of Transportation will be used. Concrete works is capable of calculating the temperature development in the same pavements modeled using HIPERPAV III. This information will allow better investigations into how early curing influenced the cracking

behavior by changing the zero-stress temperature, or temperature at which setting occurs and stresses begin to develop in the pavement. Figure 5 shows the temperature predicted by the ConcreteWorks software for the same concrete pavement modeled with HIPERPAV III.

- F. (KSU) Investigation of correlations between the occurrence of Y-cracking and pavement performance, including increased occurrences of punchouts and spalls, will be developed. The statistical software package SAS will be used to evaluate any correlations between the frequency of Y-cracking, time of increased numbers of distresses from the Y-cracking, and contributing factors such as base type, materials used, and curing conditions.
- G. (OSU and KSU) Evaluate any correlations developed between Y-cracking and design, materials, and construction parameters to determine if any cost effective and timely changes could be made to the ODOT pavement and materials specifications to reduce the occurrence of Y-cracking. The cost effectiveness of pre-notching (grooving or sawcutting) will also be examined. Draft specifications will be developed for the cost effective solutions identified
- H. Complete the Final Report.

During this period tasks A, B and D were completed and are presented in this report. Progress will also be given on tasks C, E, F, G, and H.

## **1.2 Introduction**

Y-cracking in CRCPs has been observed on several Oklahoma pavements. Y-cracking has been associated with spalling and punchouts, increasing maintenance costs, and decreasing ride quality (Kohler and Roesler 2004). Some have suggested that CRCP Y-crack patterns are formed during the early age period, and are influenced by the materials used, percentage of steel, base type and preparation, and curing conditions (Johnston and Surdahl 2008). To help quantify how Y-cracking impacts CRCP pavement failure through punchout and spalling and how to minimize its occurrence, seven tasks were chosen. These tasks are outlined in section 1.1 of this report.

The following sections of this report will either display the completed task or give the status of a task in progress.

## CHAPTER II

### LITERATURE REVIEW

#### **2.1 Continuously Reinforced Concrete Pavement Overview**

Continuously reinforced concrete pavement (CRCP) is a type of rigid pavement that is designed without joints. Instead, the pavement is designed to crack at regular intervals to relieve shrinkage stresses. The cracks are held tight by reinforcing steel. By varying the amount of steel, designers can change the spacing and width of the cracks.

##### **2.1.1 History of CRCP**

The first experimental CRCP was built in 1921 on the Columbia Pike near Washington, D.C. Rigid pavements were thought to be weakest at the joints; therefore, there was an interest in using CRCPs because there were no joints (Huang, 2004). Another advantage was that the pavement thickness could be decreased and no joints had to be saw cut. This helps offset some of the costs associated with the reinforcing steel. Although during the 1940s and 1950s many states began performing studies on CRCPs, CRCPs were not widely used until the 1960s. As of 2005, there are CRCPs in over 35 states covering more than 28,000 lane miles (Choi & Chen, 2005). The states with the highest number of lane-miles of CRCPs are Illinois, Oklahoma, Oregon, South Dakota, Texas, and Virginia (ERES Consultants, Inc., 2001).

##### **2.1.2 Texas History**

For more than fifty years Texas has been the leader in the number of lane-miles and also in designing and monitoring the performance of CRCPs. The Texas Highway Department, which

later became the Texas Department of Transportation (TxDOT) in 1951, saw the performance other states were receiving from their CRCPs and decided to try two CRCP projects around Fort Worth, Texas. Both CRCP sections provided 40 years of excellent performance with the only maintenance being surface texturing for safety reasons. (The Transtec Group, Inc., 2004). After the excellent performance of these CRCPs, TxDOT built many more miles of CRCPs throughout the years. Texas has the most lane miles of CRCPs in the world and designs all high-volume heavy-traffic roads as CRCPs (The Transtec Group, Inc., 2004). A major reason Texas has become a leader in the design of CRCPs is because of their extensive monitoring of their CRCPs since beginning their use.

### **2.1.3 Oklahoma History**

Oklahoma Department of Transportation (ODOT) first started building CRCPs in the early to mid 1970s. As of 1996, there were over 686 lane miles of CRCPs in the state of Oklahoma, with most of these miles being in the eastern half of the state. Roughly 75 percent of these CRCPs were built from 1986 to 1996 (McGovern, Ooten, & Senkowski, 1996).

## **2.2 CRCP Design**

The reason CRCPs have become so popular to construct is because, unlike jointed plain concrete pavements (JPCP) or jointed reinforced concrete pavements (JRCP), the joints do not have to be sawed into the concrete, which then needs to be sealed and maintained. If designed correctly, the crack widths will be small and will keep incompressibles out. The American Association of State Highway Transportation Officials (AASHTO) recommends a minimum transverse crack spacing of 3.5 feet and a maximum of 8 feet. The maximum transverse crack spacing is set to reduce the potential for crack spalling because of excessively wide cracks, while the minimum spacing is set to reduce the chance of punchouts (American Association of State Highway and

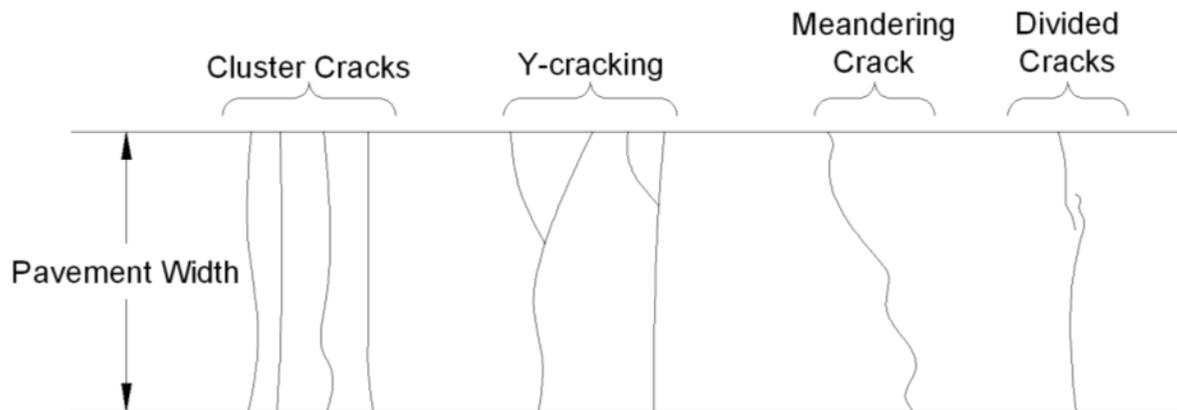
Transportation Officials, 1993). Most transverse cracks develop early in the life of the concrete, usually within the first few months, with incidences of cracking diminishing after about one or two years.

Unlike jointed pavements, CRCP's transverse crack widths are designed to be very small. The increase in longitudinal reinforcing steel causes the transverse cracks to be held together tightly. These tight transverse cracks not only provide a smooth ride to users, but also allows for load transfer by aggregate interlock and protects from water infiltration. These tight transverse cracks do not allow a clear path for the water to infiltrate into the base, which prevents pumping of the base. The crack spacing can be controlled by changing the percent of longitudinal reinforcing steel; the more steel, the smaller the crack widths and crack spacing (McGovern, Ooten, & Senkowski, 1996). Furthermore, with CRCPs, the thickness of the concrete can usually be reduced by 20 to 30 percent as compared to other pavement thicknesses. This usually translates to a pavement that is 1 to 2 inches thinner than a jointed pavement (Huang, 2004).

### **2.3 Cracks Associated with CRCP**

There are four general types of cracks that have been classified pertaining to CRCPs, as seen in Figure 2.12.1, all of which correspond to the designed mean transverse cracks. Cluster cracks happen when the mean crack spacing of the CRCP is reduced and more than three cracks happen in a close spacing of each other. Several studies have indicated possible causes of cluster cracking, "Cluster cracking has been associated with variation in subgrade support, poor concrete consolidation, inadequate drainage, high base friction, and high ambient temperature at time of construction" (McGovern, Ooten, & Senkowski, 1996). The next crack type that happens is Y-cracking. Y-cracking is identified by a single crack that splits off into two other cracks that spread apart. The third type, a meandering crack, is just a transverse crack that does not stay

perpendicular to the edge of the pavement and wanders one way or another. The final crack type that can form is the divided crack. Divided cracks are two cracks that do not crack the full width of the pavement and usually meet around an area and may or may not touch each other. Cluster cracks, Y-cracks, and divided cracks have been suspected to lead to early-age spalling and later punchout distresses (Kohler and Roesler 2004).



**Figure 2.1** - Crack shapes and patterns associated with defective passive cracks.

After (Kohler and Roesler 2004)

### 2.3.1 Y-Cracking

These types of cracks could severely decrease the performance and life of the CRCPs. Figure 2.2 is a picture of Y-cracking in an Oklahoma CRCP. Y-cracking is easily categorized by its very unique shape. There is a single crack, or “trunk” crack, which splits into two other cracks, called the “branch” cracks. It is not known how the crack propagates, whether the branches form and converge into one crack or the trunk forms and branches into the other two cracks.



**Figure 2.2 - Y-cracking example**

## **2.4 Problems Associated with Y-Cracking**

Y-cracking has been speculated to cause punchouts and spalling to occur because the section of the pavement at the location of the crack bifurcation can become a cantilevered section when aggregate interlock is degraded (Kohler and Roesler 2004). These distresses are used to gauge the performance of the CRCP throughout its service life.

### **2.4.1 Spalling**

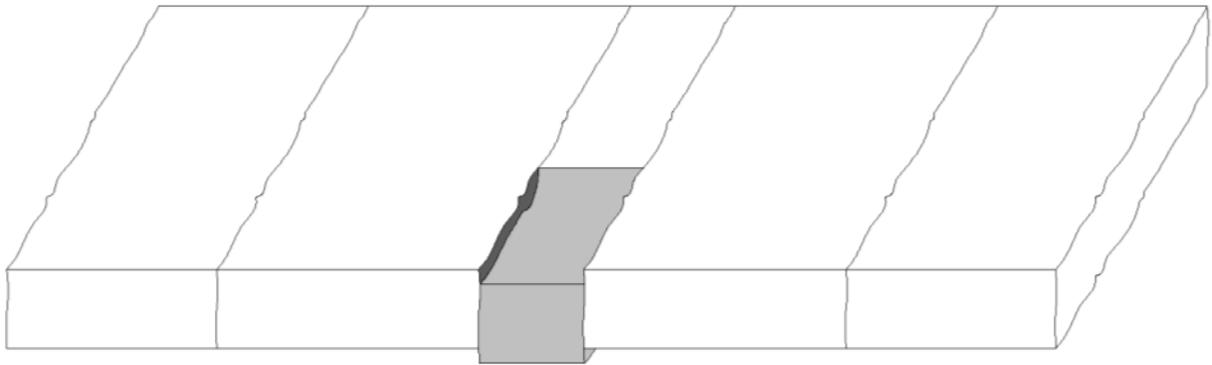
Spalling is defined as “the cracking, breaking, or chipping of the slab edges within 2 feet of the crack” (Choi & Chen, 2005). Normal CRCPs are designed with tight cracks that help resist spalling. Transverse crack spacing and crack width have been associated with controlling the resistance to spalling in CRCPs (Kohler and Roesler 2004). Spalling can be caused by excessive crack widths, which causes excessive stresses at the cracks. These stresses are from water infiltration, expansion of slabs, and traffic loading (Choi & Chen, 2005). An example of spalling in Oklahoma can be seen in Figure 2.3.



**Figure 2.3** - Example of spalling

#### **2.4.2 Punchouts**

The most common major distress associated with CRCPs and its performance life is punchouts. A punchout is defined as “an isolated piece of concrete that settles into depression or void at the edge of the concrete slab” (Beyer & Roesler, 2009). An example of a punchout between two closely spaced transverse cracks can be seen in Figure 2.4.



**Figure 2.4** - Example of punchout between two closely spaced transverse cracks.

After (Kohler, 2005)

A punchout can develop between two transverse cracks or at the branches of a Y-crack. The primary reason for punchouts has been found to be erosion of the subbase and loss of support under the slab (Zollinger & Barenberg, 1990). For this reason, the control of spalling is a key factor in preventing punchouts from happening. When spalling occurs, that means the crack width is wide enough to allow infiltration of water into the subbase of the CRCP. If the subbase is not made of non-erodible material, and there is enough heavy traffic, then pumping of the subbase will occur. Pumping will remove water and soil from the base through the crack and cause loss of support under the slab.

Punchouts usually happen where two transverse cracks are close. This is because the cross sectional area of the concrete to resist the stress is less than when the crack spacing is at its designed interval and uniform. Transverse crack spacing and crack width have been associated with punchouts in CRCPs (Kohler and Roesler 2004).

## **2.5 Possible Causes of Y-Cracking**

There have been several research projects studying the effects of various factors on the transverse crack spacing of CRCPs. However, there have been few that have tried to correlate the effect of these variables on Y-cracking.

### **2.5.1 Transverse Crack Spacing**

Transverse crack spacing is believed to have a major effect on the possibility of Y-cracking in CRCPs. The transverse crack spacing is affected by many factors; the key variables are percent longitudinal steel, depth of steel placement, and climatic condition at time of construction (Zollinger & Barenberg, 1990). Typical desired transverse crack spacing is from 3.5 to 8 feet (Huang, 2004). More closely spaced cracks are believed to increase the probability of cluster cracking and Y-cracking (D. P. Johnston 2008). One reason for the increased possibility of

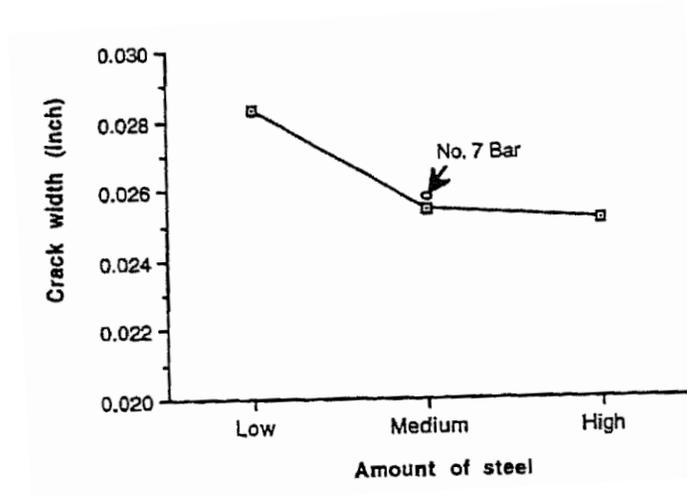
cluster and Y-cracking is that the closeness of these cracks lessens the distance for a crack to form, making it easier for variations in the subgrade restraint and support to cause cluster cracking and for cracks that meander slightly from their normal direction to intersect another crack and form a Y-crack. Large variability in crack spacing increases the probability for punchouts, which could be caused by the formation of Y-cracks or cluster cracks (Kohler and Roesler 2004). A large variability in the crack spacing could also be a sign of poor subgrade and concrete uniformity and poor overall quality control in construction.

### **2.5.2 Crack Width**

A factor that is directly related to the transverse crack spacing is the crack width. It has been seen that the smaller the transverse crack spacing the smaller the crack width, principally because both factors are affected by the amount of steel bridging the crack (The Transtec Group, Inc., 2004). A maximum crack width of 0.04 inches is recommended (Huang, 2004). One reason for this is that as the crack width increases the infiltration of water into the base material also increases. When more water is allowed into the subbase the probability of pumping can increase, which can lead to punchouts of the CRCPs. Another reason for the maximum crack width is that as the crack width increases the load transferred by aggregate interlock decreases. Crack widths are significantly affected by the following factors: time of crack occurrence, ambient temperature, type of coarse aggregate, depth of reinforcement, and percent of longitudinal steel. Cracks that form early in the life of the concrete have been seen to be wider and meander more than cracks that form later. This increase in meandering increases the probability of cracks to intersect and cause Y-cracking (McGovern, Ooten, & Senkowski, 1996). As the depth of reinforcement increases, the crack width decreases.

### 2.5.3 Percent of Longitudinal Steel

The percent of longitudinal steel is a major factor on the transverse crack spacing and crack width. As the percent of longitudinal steel increases the crack width decreases because the stresses and corresponding strains in each bar decrease, holding the cracks tightly together. It should also be noted that even though percent of longitudinal steel has a significant effect on cracking patterns, it cannot be completely controlled if there are other factors (Huang, 2004). An example of this relationship can be seen in a study done by Suh and McCullough, shown in Figure 2.5. In this figure the medium amount of steel refers to the Texas design standard. The high and low refer to about 0.1 percent more and less of the medium amount of steel, respectively (Suh & McCullough, 1994).



**Figure 2.5** - Effect of longitudinal steel design on crack width. From Young-Chan, S., and B. McCullough. Factors Affecting Crack Width of Continuously Reinforced Concrete Pavement. In Transportation Research Record 1449, Figures 8 and 9, p. 138. Copyright, National Academy of Sciences, Washington, D.C., 1994. Reproduced with permission of the Transportation Research Board. None of this material may be presented to imply endorsement by TRB of a product, method, practice, or policy.

#### **2.5.4 Percent of Transverse Steel**

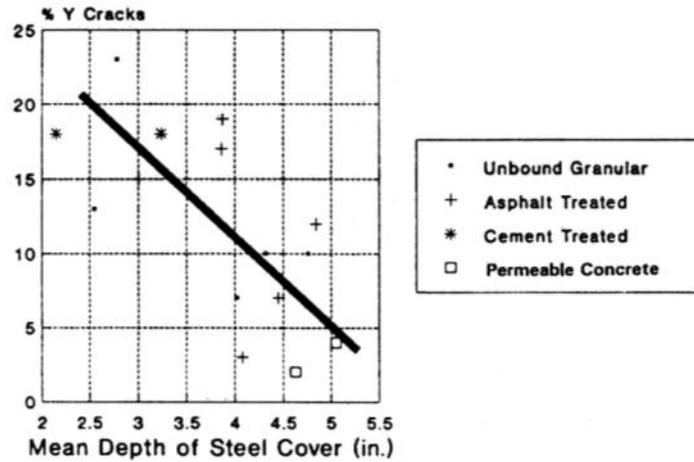
Much less research has focused on the effect of the percent of transverse steel in CRCs than the effect of longitudinal steel. In a study done by Al-Qadi and Elseifi, field test data showed that transverse cracks occurred mostly in the vicinity above transverse bars (60% of transverse cracks within 0.4 inches of transverse bars). But they also noted that transverse cracks did occur away from transverse bars, which led them to conclude that there was a possibly correlation between the transverse bars and the location of transverse cracks. Next they did a thermal stress analysis by creating a Finite Element Model (FEM) and it showed that high longitudinal tensile stress can build up in the concrete above the transverse bars. The model also showed how uniformly distributed compressive longitudinal stresses can build up at the pavement surface in between the transverse bars. This could explain the occurrence of cracks that did not occur over the transverse bars. Since the stress was uniform the crack would probably occur at a weak spot in the concrete, not necessarily at the midpoint between transverse bars (Al-Qadi & Elseifi, 2006).

#### **2.5.5 Depth of Reinforcement**

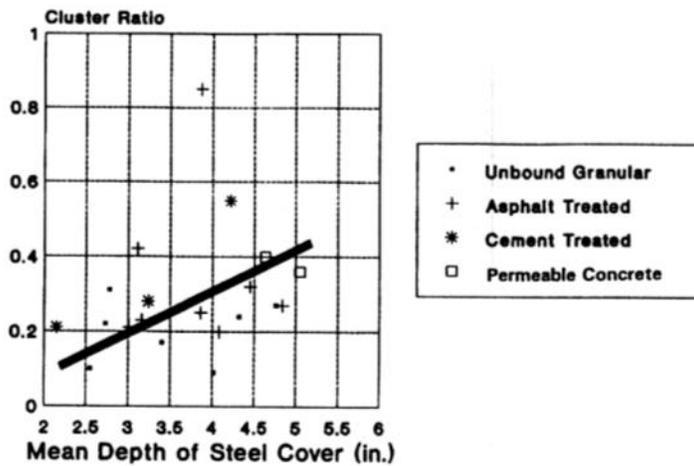
Depth of reinforcement has been seen to increase the uniformity of the cracking pattern of CRCs (Kohler and Roesler 2004). As the variability in crack spacing decreases, the probability of punchouts decreases (Kohler and Roesler, Active Crack Control for Continuously Reinforced Concrete Pavements 2004).

Tayabji et. al. (1998) showed that as the depth of reinforcement increases, the percent of Y-cracking decreases. Figure 2.6 shows the relationship between steel cover depth and Y-cracking, while Figure 2.7 shows the relationship between the steel cover depth and cluster cracking. Tayabji et al. provided a trendline showing a relationship between cluster cracking and steel

cover depth, however it appears from Figure 2.7 that this trend is very weak and somewhat suspect.



**Figure 2.6** - Y-cracking versus the mean depth of steel cover. (Tayabji, Zollinger, Vederey, & Gagnon, 1998)



**Figure 2.7** - Cluster ratio versus the mean depth of steel cover. (Tayabji, Zollinger, Vederey, & Gagnon, 1998)

### 2.5.6 Base Type

The effect of base type on cracking patterns is not as clear as other factors. It can be seen in Figure 2.6 and Figure 2.7 that the relationship between the type of subbase and cluster and Y-

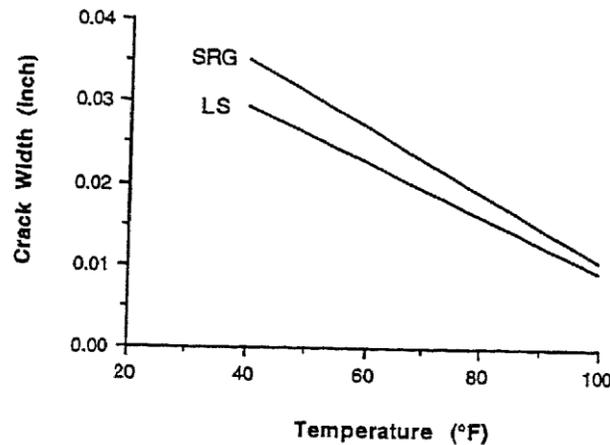
cracking is still not clear (Tayabji, Zollinger, Vederey, & Gagnon, 1998). Other studies have shown that the base type matters because the amount of restraint provided by the base affects how many cracks occur, the transverse crack spacing, and when the cracks develop.

Early cracking can lead to spalling, wide cracks, and meandering because the concrete strength is not high enough at the early ages to resist these stresses (D. P. Johnston 2008). TxDOT has made it mandatory that bases be stabilized with either cement or asphalt. They did this to help prevent pumping and also to help with high stresses if support was lost at pavement edges in order to prevent punchouts (The Transtec Group, Inc., 2004). It was later discovered by Texas that the cement-stabilized bases, while providing more support, started excessively cracking. The stiffer bases provide more restraint for the concrete pavement. Cement-stabilized bases are also vulnerable to shrinkage cracking. The cracks in the base can reflect through in the pavement, giving more cracking. Currently in Texas, if a cement-stabilized base is used, a bond-breaker is required to reduce the subbase restraint and reduce excessive cracking in the CRCP (McGovern, Ooten, & Senkowski, 1996).

### **2.5.7 Coarse Aggregate**

One material parameter that may be important with CRCPs is the concrete coefficient of thermal expansion (CoTE). The CoTE of the concrete is dependent on the water-cement ratio, concrete age, richness of the mix, relative humidity, and type of aggregate in the concrete mix. However, the most important of these parameters is the type of coarse aggregate used because of the high volume of aggregates used in concrete (Huang, 2004). Suh and McCullough studied the difference in crack width between siliceous river gravel and limestone at various slab temperatures at the time of measurements. Their results can be seen in Figure 2.8. From this figure it can be seen that the siliceous river gravel led to greater crack widths at all slab

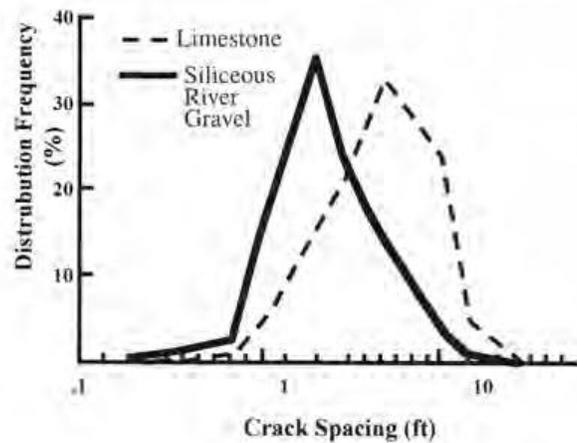
temperatures. This greater crack width is caused by the siliceous river gravel having a higher CoTE than the limestone aggregates studied, and hence more thermal movement. It also can be seen that at lower temperatures the difference in crack width is greater than at higher temperatures (Suh & McCullough, 1994).



**Figure 2.8** - The effect of coarse aggregate type and slab temperature on crack width. From Young-Chan, S., and B. McCullough. Factors Affecting Crack Width of Continuously Reinforced Concrete Pavement. In Transportation Research Record 1449, Figures 8 and 9, p. 138. Copyright, National Academy of Sciences, Washington, D.C., 1994. Reproduced with permission of the Transportation Research Board. None of this material may be presented to imply endorsement by TRB of a product, method, practice, or policy.

TxDOT has studied the effects of coarse aggregate type on CRCP performance. Those studies have shown that CRCPs constructed with siliceous river gravel do not last as long as those constructed with limestone. The limestone CRCPs have, on average, lasted 10 years longer than siliceous river gravel CRCPs. The reason for this is the effect that the coarse aggregate type has on the transverse crack spacing. The crack spacing for siliceous river gravel and limestone were 2 to 3 feet and 6 feet, respectively. The higher CoTE of siliceous river gravel causes more

thermal stresses in the concrete compared to the lower CoTE of limestone. This results in more cracking of the concrete and a lower mean crack spacing. The frequency distribution of cracks can be seen in Figure 2.9. The closer crack spacing, such as seen with the siliceous river gravel, can lead to more punchouts in CRCPs.



**Figure 2.9** - Influence of aggregate type on crack spacing. (The Transtec Group, Inc., 2004)

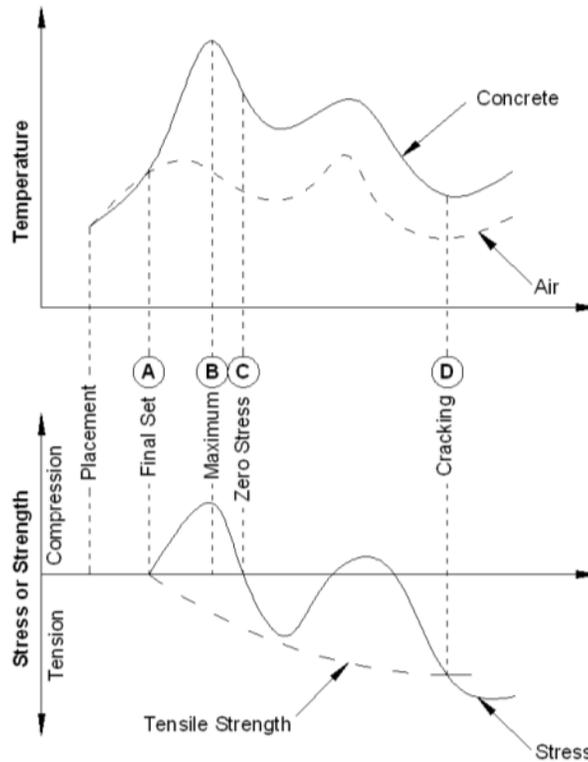
The coarse aggregate is also believed to affect the amount of cracks that meander. Studies have shown that pavements with limestone and lightweight aggregates tend to have cracks that are straighter with less meandering than pavements with siliceous river gravel. The lightweight and limestone aggregates are a weaker aggregate and have a stronger bond with the paste than siliceous river gravel. This causes the cracks to be able to propagate through the lightweight and limestone aggregates but not through the siliceous river gravel. Since it is difficult for the cracks to propagate through the siliceous river gravel, they will meander through the paste because it is weaker at early ages (Du & Lukefahr, 2007).

Another factor of coarse aggregate that has been shown to effect the performance of CRCPs is the aggregate size. In a study in South Dakota, a maximum coarse aggregate size of 1 inch and 1½ inches were used with no other factors adjusted. The crack spacing was on average 3.09 feet

and 2.19 feet for the 1½ inches and 1 inch aggregate, respectively. Further inspection showed that the cracks from the larger aggregate were more uniform and tighter. Also, the larger aggregate reduced the amount of cluster cracks and Y-cracking. The increase in aggregate size causes an increase in steel bond strength and concrete fracture resistance (D. P. Johnston 2008).

### **2.5.8 Construction Environment**

The two main construction environment factors that affect the performance of CRCs are the concrete temperature and air temperature at the time of curing. Thermal stresses are driven by the concrete pavement temperature change. Figure 2.10 shows this relationship between the ambient temperature and cracking for an example pavement. At point A, the concrete sets and is in compression as the temperature of the concrete increases from the heat of hydration. At point B, the maximum temperature and compression of the concrete is achieved. At point C, the concrete temperature decreased which leads to a decrease in concrete compression until temperature decreases and autogenous shrinkage changes the compression in the concrete to tension. After this point, the stresses vary as the temperature difference in the concrete and air change. At point D, the tensile stresses have exceeded the tensile strength of the concrete and cracking occurs (Schindler & McCullough, 2002).



**Figure 2.10** - Relationship between temperature and concrete cracking. After (Schindler & McCullough, 2002)

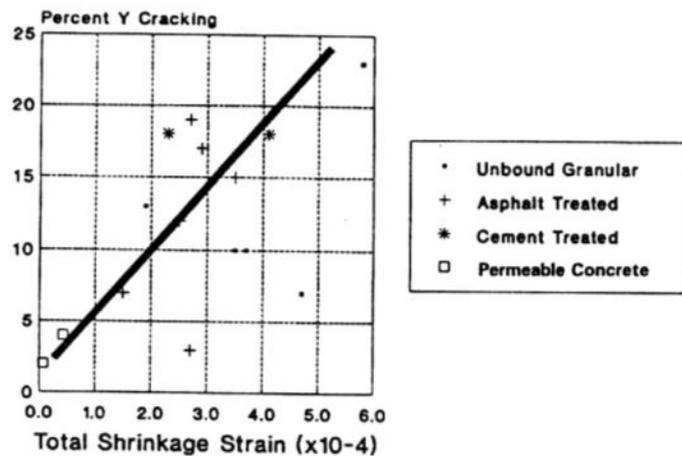
Because of the relationship between the difference in concrete temperature and air temperature and the subsequent temperature decrease of the concrete with time, there have been many problems with construction of CRCPs in hot weather. In Texas, CRCPs placed in hot weather have exhibited non-uniform crack spacing and Y-cracking. Because of this, TxDOT has determined the air temperature and concrete temperature must be monitored during construction (The Transtec Group, Inc., 2004). Texas currently has a maximum concrete temperature at placement of 95°F to help reduce this risk (Texas Department of Transportation, 2004).

A major factor that affects the air temperature and concrete temperature is the placement season. Crack widths have been seen to be more than two times greater in CRCPs placed in the summer months than CRCPs placed in the winter months (Suh & McCullough, 1994). The crack spacing

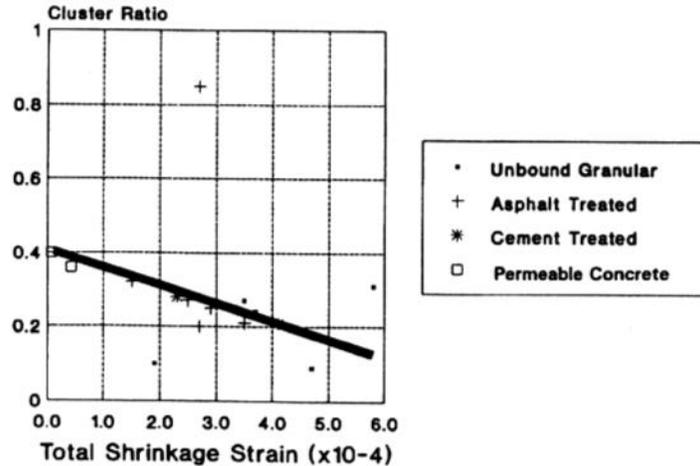
has been found to be narrower for CRCs placed in warmer weather because the higher the concrete second zero stress temperature, the more total temperature decrease and strain that will result during the cold winter months (The Transtec Group, Inc., 2004). This decrease in crack spacing caused by placement and curing in warm weather could increase the potential for punchouts in CRCs.

### 2.5.9 Shrinkage

Tayabji et. al. showed that as the total shrinkage strain in the concrete increases, Y-cracking increases and the cluster ratio decreases, as seen in Figure 2.11 and Figure 2.12. There appears to be more of a trend between the shrinkage strain and cluster cracking, than with the shrinkage strain and the Y-cracking (Tayabji, Zollinger, Vederey, & Gagnon, 1998).



**Figure 2.11** - Y-cracking versus total shrinkage strain for different subbase types. (Tayabji, Zollinger, Vederey, & Gagnon, 1998)



**Figure 2.12** - Cluster ratio versus total shrinkage for different subbase types. (Tayabji, Zollinger, Vederey, & Gagnon, 1998)

Shrinkage strain is one of the key factors that affect the development of early-age cracking in CRCPs (Kohler & Roesler, 2006). When cracks develop in the first few days, they have a higher tendency to meander. This increases the probability of cracks that can result in Y-cracking (McGovern, Ooten, & Senkowski, 1996). To control the amount of total shrinkage in concrete, the correct actions must be taken in design and construction to control autogenous and drying shrinkage. This can be accomplished by not using excessive amounts of cement and using a moderate water-to-cement ratio.

## 2.6 Historical Issues with Y-Cracking

No studies have previously been performed exclusively focusing on Y-cracking. However, a few studies have included Y-cracking in an overall investigation into CRCP performance. *Tayabji et al.*, The study entitled "Performance of Continuously Reinforced Concrete Pavements" by Tayabji et al. was completed in October 1998 with the purpose of the study being to update the design, construction, maintenance, and rehabilitation of CRCP for better performance. The study was funded by the states of Arizona, Arkansas, Connecticut, Delaware,

Illinois, Louisiana, Oklahoma, Oregon, Pennsylvania, South Dakota, and Texas. One of the focuses of the study was to conduct field and laboratory testing on existing CRCPs. The field studies were all conducted in the fall of 1991 (Tayabji, Zollinger, Vederey, & Gagnon, 1998). A total of 23 CRCP sites were selected; five from Illinois, three from Iowa, five from Oklahoma, three from Oregon, two from Pennsylvania, and five from Wisconsin. Sites were selected to include a wide variety of design and construction attributes of the CRCPs. Field investigations at each site included the performance of a representative 1000 foot length section by the following: visual condition surveys, profile measurements, falling weight deflectometer, and corrosion testing. Laboratory testing included: concrete core testing for strength, stiffness, and CoTE, base, subbase, and subgrade material characterization. The study also gathered design, construction, maintenance, performance, and traffic data. The authors attempted to correlate the actual crack spacing to the performance of the CRCPs. The performance was judged by the extent of different types of structural distresses such as Y-cracking and cluster cracking observed in the pavement. The results have been discussed in previous sections (Tayabji, Zollinger, Vederey, & Gagnon, 1998).

## **2.7 McGovern et al. Study**

The second study, "Performance of Continuously Reinforced Concrete Pavements in Oklahoma - 1996" by McGovern et al. (1996), was completed in August 1996 under the direction of the Oklahoma Department of Transportation (ODOT). The purpose of this study was to inspect the performance of Oklahoma CRCPs and concentrate on crack spacing, cluster cracking, Y-cracking, and overall condition. The report also compares the CRCP design and construction methods between ODOT and TxDOT. ODOT summarizes various studies that have been conducted on their CRCPs since 1988, including the study by Tayabji et al. Based on these

previous surveys, ODOT conducted field surveys of 44 projects in June of 1996 and investigated the number of punchouts per mile. After this, ODOT did a CRCP tour in July and August of 1996, performed by ODOT Pavement Engineer Mr. Tim Borg. The CRCP tour focused on cracking patterns such as cluster cracking, Y-cracking, and long crack spacing (McGovern, Ooten, & Senkowski, 1996).

Based on previous studies, the 1996 ODOT field studies and CRCP tour, and comparisons between ODOT and TxDOT design and construction methods, ODOT made recommendations for future design and construction of CRCPs in Oklahoma. Although the study was not inclusive of all CRCPs in Oklahoma, the data from the CRCPs studied showed that Oklahoma had fewer punchouts per mile than the average for all of the sites in the Tayabji study. The major concern for the Oklahoma CRCPs was the large crack spacing and cluster cracking. The following recommendations were made:

- Continue to investigate the projects with 0.61 percent longitudinal steel
- Use an asphalt bond breaker between the CRCP and cement treated base
- Decrease the amount of cement used in cement treated base
- Longitudinal construction joints should be sealed and sawed between the outside lane and PCC shoulder

ODOT also decided to keep investigating the effect that ambient conditions during construction, depth of steel, swelling potential of soil, coarse aggregate type, and rate of strength gain had on the performance of CRCPs (McGovern, Ooten, & Senkowski, 1996).

## CHAPTER III

### FIELD INSPECTION

#### 3.1 Overview

##### 3.1.1 Visual Inspection

Visual inspection of continuously reinforced concrete pavement is the best way to determine the condition of the structure. For this research project a visual inspection was conducted by slowly driving the shoulder of the roadway. While traveling the shoulder, extreme caution was used to avoid self-endangerment while inspecting the entire pavement. Stretches of roadway where there was either no shoulder to traverse or the danger was too high were documented and taken into account while collecting data.

The visual inspection of this project consisted of several parts including counting Y-cracks, counting patches, and photographing all subjects of interest. During the inspection a hand held Global Positioning System (GPS) was used to document the location of each patch and picture for future reference. The GPS was used to measure the distance in miles, to the nearest hundredth (0.01), from the beginning of the project to the patch or photograph. This distance is commonly referred to as a chainage.

For a few of the sites, mainly the ones studied in previous inspections, measurements were taken so that mean spacing, standard deviation, coefficient of variation, and extent of Y-cracks could be determined. These sites are noted in the following sections.

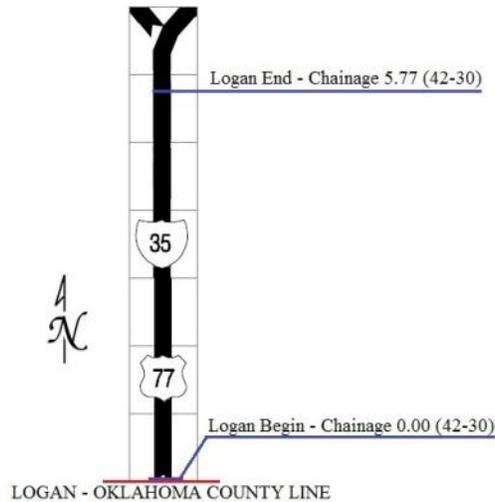
## **3.2 Logan County**

### **3.2.1 Overview**

The continuously reinforced concrete pavement (CRCP) studied in Logan county is located on Interstate 35 just south of the town of Guthrie. As shown in figure 3.1, the pavement begins at the Oklahoma-Logan County line and extends north nearly six miles. Each square in figure 3.1 represents one square mile. Information provided by ODOT shows that the 2011 Annual Average Daily Traffic (AADT) for the project was estimated at 35,200 vehicles per day (VPD); meanwhile, the plan-sets provided by ODOT for the project show that the design average daily traffic (ADT) was 42,000 VPD.

The CRC pavement was completed in 1988 by Koss. It is 10 inches thick with 0.51% longitudinal and 0.11% transverse steel, according to the specifications for the project provided by ODOT. The pavement sits atop three inches of Type-A asphalt concrete and eight inches of select sub-base, with jointed plain concrete shoulders. Type-A asphalt concrete has a  $\frac{3}{4}$  inch nominal maximum aggregate size (NMS).

A visual inspection was conducted on July 5 and 26, 2011. The visual inspection consisted of counting Y-cracks and patches, while photographing subjects of interest. The chainage of each patch and photograph taken was documented for future reference. A thousand foot section was also inspected for results comparable to Federal Highway Administration (FHWA) Report Number: FHWA-RD-94-174. This study consisted of 10.56 lane miles, consisting of 5.36 miles in the north bound direction and 5.20 miles in the south bound direction.



**Figure 3.1** – Layout of the Logan County CRCP visited during inspection. The pavement is marked by county name, begin and end points, along with chainage (the length in miles from the beginning of the control section to a specified point) and the control section number.

### 3.2.2 Inspection Details

The south bound lane of his project had several more distresses and failures than the north bound lane. The south bound lane had several large areas of map cracking as seen in figure 3.2. The lane also exhibited multiple Y-cracks, or transverse cracks with several branching cracks, shown in figure 3.3 below.



**Figure 3.2** – Photo shows transverse cracks with inter-connecting longitudinal cracks.



**Figure 3.3** – Photo shows Y-cracks with multiple branches

The overall crack spacing in the south bound lane was fairly irregular. Cracks seemed to range in spacing from two to ten feet. There were several instances of close transverse cracks (less than one foot) starting to break up, by way of longitudinal cracks, shown in figure 3.4. During the south bound lane inspection it was noted that areas with deeper tines made it more difficult to count Y-cracks, but seemed to make the cracks meander less.



**Figure 3.4** – Photo showing closely spaced transverse cracks developing into punch-outs

The north bound lane also exhibited irregular crack spacing, with cracks ranging from three to eight feet in spacing. There were a few developing punch-outs that had not been patched (figure 3.5); however, these appeared to be close transverse cracks and not Y-cracks. Many of the Y-cracks had spalling along their length. It was noted during the inspection that Y-cracks seemed to be spaced far apart and in clusters. In areas of increased crack spacing, such as eight to ten foot, Y-cracks were scarce.



**Figure 3.5** – Developing punch-out between two close transverse cracks

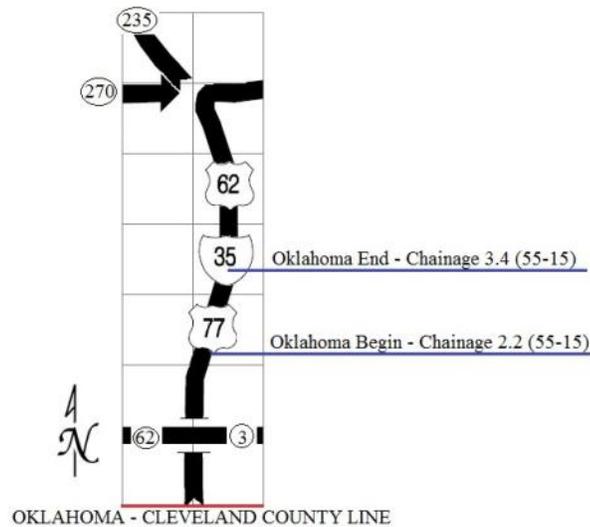
### **3.3 Oklahoma County**

#### **3.3.1 Overview**

The CRC pavement studied in Oklahoma County is located on Interstate 35. The project begins two miles north of the Oklahoma-Cleveland County line and extends north for just over a mile. Figure 3.6 shows the location of the project with each square representing one square mile of area. The 2011 AADT data provided by ODOT estimated 120,000 VPD for the pavement, while the plan-sets showed the design ADT to be 94,000 VPD. The inspection consisted of 0.73 miles in the north bound lane and 1.01 miles in the south bound lane, for a combined total length of 1.74 miles.

The pavement project was completed in 2001 by Neilson Inc. It is ten inches thick with 0.61% and 0.07% longitudinal and transverse steel respectively according to ODOT plan sets. The pavement is sitting atop four inches of open graded bituminous base and twelve inches of aggregate base with CRCP shoulders.

A visual inspection was conducted on July 5, 2011. The visual inspection consisted of counting Y-cracks and patches, while photographing subjects of interest. The chainage of each patch and photograph was also noted for future reference.



**Figure 3.6** – Layout of the Oklahoma County CRCP visited during inspection. The pavement is marked by county name, begin and end points, along with the chainage and control section number.

### 3.3.2 Inspection Details

The north bound lane of this project had isolated areas of heavy Y-cracking and closely spaced transverse cracks connected by longitudinal cracks (see figure 3.7). It was noted that the crack width seemed to be very small, with a crack spacing of eight to ten feet between clusters of cracks.



**Figure 3.7** – Closely spaced transverse connected by longitudinal cracks

Before the project was inspected, ODOT commented that they had a particular concern within mileposts 124.1 and 124.2. Figures 3.8 and 3.9 show the distresses in this particular region. The transverse steel in this region appears to be high in the pavement, leaving only  $\frac{3}{4}$ -1 inch of concrete cover over the bar. It was also noted that the transverse steel in this region does not appear to be perpendicular to the direction of traffic. Chris Westlund, an ODOT employee, said that the steel on this project was placed with a machine during the paving process and he recalled that the machine was having problems. Nevertheless, the distresses in this section of pavement seemed to be caused by inadequate steel cover and not related to Y-cracking, or any other cracking patterns. None of the distresses caused by the transverse steel have been patched in the north bound lane. The north bound lane also exhibited several stretches where the road surface was extremely worn. The tines in the wheel paths were barely visible. Several stretches of pavement had to be skipped while performing the visual inspection due to exits, entrances, and barrier walls not providing a shoulder to conduct the inspection.



**Figure 3.8** – Distresses in ODOT’s region of concern



**Figure 3.9** – More distresses in ODOT’s region of concern

The south bound lane appeared to be in slightly better condition than the north bound lane. The south bound lane still had distress caused by steel placement but significantly fewer than the other direction. Only one of the steel placement distresses had been patched, see figure 3.10. Figure 3.11 shows images where it appears as though a Y-crack occurred over the shallow transverse steel. The steel in this area appeared to be less than an inch below the surface and is

more likely to be the cause of the distress, not the Y-crack. Ignoring the steel placement issues, the pavement appeared to be in good shape with a low number of Y-cracks observed and no Y-crack related distresses.



**Figure 3.10** – AC patch over a distress with exposed transverse steel



**Figure 3.11** – Possible Y-crack over shallow transverse steel

## **3.4 Cleveland County**

### **3.4.1 Overview**

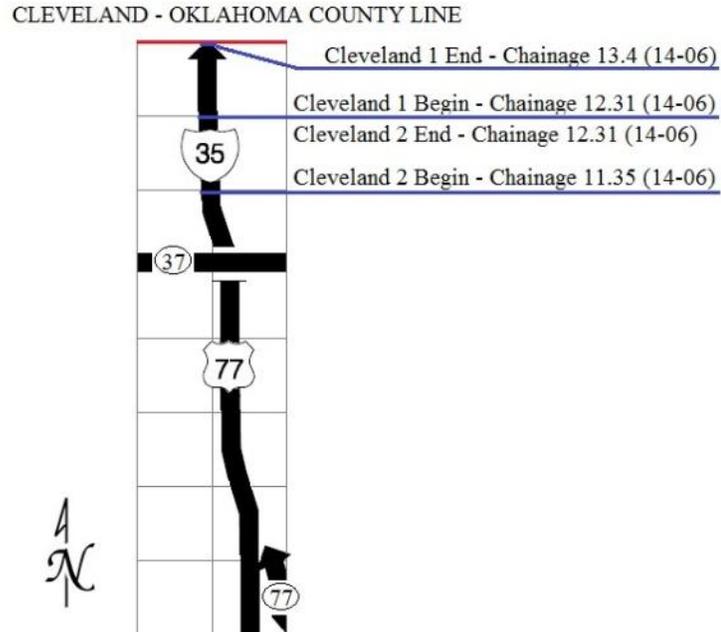
The CRC pavement studied in Cleveland County consisted of two adjacent projects located on Interstate 35 near the town of Moore. The pavements begin at the Oklahoma-Cleveland county line and extend south for two miles, see figure 3.12. Each square in figure 3.12 represents one square mile. The CRCP projects visited in Cleveland County displayed slight differences in AADT data, according to 2011 data provided by ODOT. Cleveland 1 AADT was estimated at 126,600 VPD, while Cleveland 2 was 113,500 VPD; meanwhile, the design ADT values were 94,000 and 110,000 VPD respectively.

The two projects of interest (Cleveland 1 and Cleveland 2) have many similar characteristics with only a few minor differences. Cleveland 1 was completed in 2002 by Haskell Lemon. The pavement is ten inches thick with 0.61% and 0.07% longitudinal and transverse steel respectively, according to the specifications for the project. The pavement sits atop four inches of open graded bituminous base and twelve inches of aggregate base, with CRCP shoulders.

Cleveland 2 was completed in 2005 also by Haskell Lemon. The pavement is ten inches thick with 0.71% and 0.07% longitudinal and transverse steel respectively, according to the specifications for the project. The pavement also sits atop four inches of open graded bituminous base and twelve inches of aggregate base, with CRCP shoulders. Therefore these two projects are similar in contractor, thickness, base, and sub-base. However, they differ in year constructed and percent longitudinal steel content.

A visual inspection was executed on July 5, 2011. The inspection consisted of counting Y-cracks and patches, while photographing subjects of interest along the pavement. The chainage

of each patch and photograph was also noted for future reference. Together the two projects totaled 1.80 miles north bound and 1.52 miles in the south bound direction.



**Figure 3.12** – Layout of the Cleveland county CRCP sites visited during inspection. The pavements are marked by designated county number, begin and end points, along with chainage and control section number.

### 3.4.2 Cleveland 1 Inspection Details

Inspection began at the Oklahoma-Cleveland County line with the pavement being inspected for two hundred feet, up to a bridge and then the bridge was skipped. On the south side of the bridge, in the south bound lane it was noted that the crack spacing was approximately three to four feet. A ½ mile south of the county line a construction joint was found, after the construction joint the crack spacing increased from three to four feet, to eight to ten. The south bound lane of

the project was in great shape. There were no patches or punch-outs observed and Y-cracking was minimal.

The north bound lane of the project was also in great shape. There were no punch-outs or patches observed in the north bound lane. It was noted, though, that the north bound lane did have a lot more Y-cracks than the south bound lane, nearly triple. However, no failures were found.

### **3.4.3 Cleveland 2 Inspection Details**

The inspection began in the south bound lane just north of exit 118 at a bridge joint. The length of pavement that was inspected was less than the full length of the project because of entrance ramps, heavy traffic, and lack of a shoulder to drive. Overall, the south bound lane was in great shape and contained no patches or punch-outs.

The north bound lane of the project was more accessible which allowed the full length of the pavement to be viewed. The inspection began at the joint on the southern end of the project (figure 3.13). It was noted that the pavement had no cracks for the first forty-two feet, and then displayed a long crack spacing of approximately fifteen feet. Then cracks slowly became closer, reaching a spacing of three to four feet. Overall, the north bound lane, like the south bound lane, was in good shape with no patches or punch-outs recorded.



**Figure 3.13** – Joint at the Southern end of North bound lane

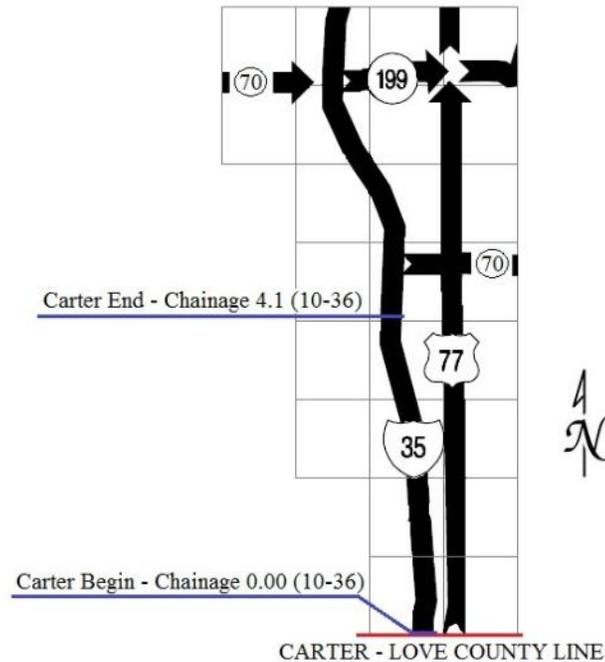
### **3.5 Carter County**

#### **3.5.1 Overview**

The CRC pavement studied in Carter County is located along Interstate 35 near the town of Ardmore. The project begins at the Carter-Love County line and extends north for just over four miles. Figure 3.14 below shows the location of the project with each square representing one square mile of area. The 2011 AADT data provided by ODOT estimated 32,700 VPD for the pavement, while the plan-sets showed the design ADT to be 52,700 VPD.

The pavement project was completed in 2007 by Koss. The CRCP is twelve inches thick with 0.73% and 0.06% longitudinal and transverse steel respectively, according to the plan sets provided by ODOT. That makes this section not only one of the thickest but also the most heavily reinforced. The pavement is sitting atop four inches of open graded bituminous base, eight inches of aggregate base, followed by eight inches of lime treated sub-base. The pavement has jointed plain concrete shoulders.

A visual inspection was conducted on July 6, 2011. The inspection consisted of counting Y-cracks and patches, while photographing subjects of interest along the project. The chainage of each patch and photograph was also noted for future reference.



**Figure 3.14** – Layout of the Carter county CRCP visited during inspection. The pavement is marked by county name, begin and end points, along with chainage and control section number.

### 3.5.2 Inspection Details

The north bound lane inspection began approximately two-hundred feet north of the county line due to a bridge. The first thing that was noted was that the pavement has its white line moved over two feet from the joint between the outside lane and the shoulder. The pavement contained a higher than usual Y-crack count for the first mile and a half but the count decreased afterward. However, the cracks for the first mile and a half displayed a regular crack spacing of approximately eight feet. Figure 3.15 shows a typical Y-crack with spalling for the region on the

left and spalled circular area on the right. The Y-crack shown in figure 3.15 seemed to be the most common, it was noted during inspection that there were multiple Y-cracks at the pavement edge with several of them being outside the white line (approximately 40%). This was especially noticed between mileposts 125 and 126. Figure 3.16 shows two construction joints that display signs of raveling. Despite the occurrence of Y-cracks (see figure 3.17 left), more potential for distresses to develop were found by closely spaced transverse cracks (see figure 3.17 right). These cracks displayed spalling and in several instances were connected by longitudinal cracks.



**Figure 3.15** – (Left) Typical Y-crack for the region, (right) circular spalled area



**Figure 3.16** – Two construction joints displaying signs of spalling



**Figure 3.17** – (Left) Y-crack located in the region; (right) two close transverse cracks showing distress

There was a construction joint 4.15 miles north of the Carter-Love County line. According to the 2008-2009 PMS Database and the CRCP Database, the project ended at this location along with the CRCP in the area. Meanwhile, the 2009 Interstate Structural Pavement History lists the roadway as project number IMY-35-1(145)029 and simply states future construction.

Due to curiosity, approximately ½ mile of this project was inspected in both the north and south bound directions. Figure 3.18 shows images of the pavement. The roadway displays a grey/silver color, possibly due to curing compound, while the surface appears to be in good shape with virtually no wear.



**Figure 3.18** – Two images showing the glossy pavement surface with little wear

The south bound inspection began at a bridge approach joint 0.16 miles north of mile post 29. The first 0.64 miles inspected were part of the project IMY-35-1(145)029. This area of pavement displayed a low number of Y-cracks. The south bound lane appeared to be more worn than the north bound lane, and its appearance was not as glossy. The south bound lane of the original project of interest displayed various forms of Y-cracking as seen in figure 3.19. However no patches or punch-outs were documented.



**Figure 3.19** – Two different forms of Y-cracking

The CRCP pavement in Carter County seemed to be in good overall condition. There were no Y-crack related distresses observed. The 2008-2009 PMS database provided by ODOT showed that the north bound lane contained four patches, while the south bound lane contained five patches. During the inspection the chainages of the patches, according to the PMS Database, were specifically checked and no patches could be found within two tenths of a mile in either direction of the specified location.

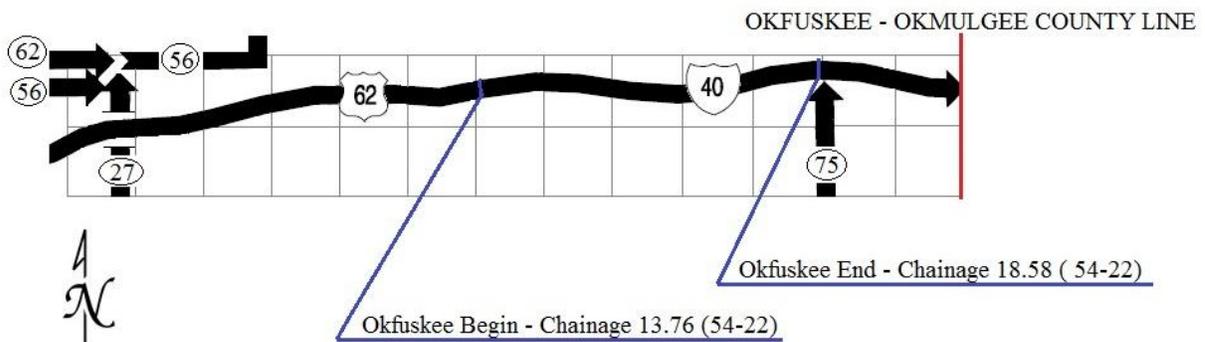
### **3.6 Okfuskee County**

#### **3.6.1 Overview**

The CRC pavement studied in Okfuskee County is located on Interstate 40 near the town of Okemah. The project is located five miles east of Okemah near interstate mile post 227. Figure 3.20 shows the location of the pavement with each square in the figure representing one square mile of area. The investigation consisted of 4.71 miles east bound and 4.69 miles in the west bound direction. The 2011 AADT data provided by ODOT estimated 16,200 VPD for the pavement, while the plan-sets showed the design ADT to be 21,400 VPD.

The project was completed in 1985 by Koss. The CRCP is nine inches thick with 0.50% and 0.08% longitudinal and transverse steel respectively, according to the plan sets provided by ODOT. That makes this pavement not only the thinnest, but also the lowest steel content of all of the pavements studied. The pavement is supported by four inches of course aggregate bituminous base and twelve inches of method b sub-base with jointed plain concrete shoulders.

A visual inspection was conducted on July 7, 2011. The visual inspection consisted of counting Y-cracks and patches, while photographing subjects of interest along the pavement. The chainage of each patch and photograph was also noted for future reference. A thousand foot section was also inspected for results comparable to Federal Highway Administration (FHWA) Report Number: FHWA-RD-94-174. An additional five-hundred foot crack spacing survey was also performed on the first five-hundred feet of the thousand foot section studied in the FHWA report.



**Figure 3.20** – Layout of the Okfuskee county CRCP visited during inspection. The pavement is marked by county name, begin and end points, along with chainage and control section number.

### 3.6.2 Inspection Details

The inspection began in the east bound direction; figure 3.21 shows the terminal joint at the project beginning. The terminal joint has some signs of deterioration. The east bound direction contained several types of Y-cracks (figure 3.22), but seemed to have many places where the transverse cracks were close together. Figure 3.23 shows a place where five transverse cracks all occurred within an eight foot section of pavement. Figure 3.22 right does contain a Y-crack on the far left, but in most cases the closely spaced transverse cracks did not consist of Y-cracks, and in most cases the close transverse cracks did not show signs of punch-out or failure. However, a few did, as shown in figure 3.23.



**Figure 3.21** – Deteriorated terminal joint at project beginning



**Figure 3.22** – Two different types of Y-cracks found in the area



**Figure 3.23** – Closely spaced transverse cracks

This pavement showed the first signs of a Y-crack leading to a punch-out (see figure 3.24). However, the majority of failures that were not patched seemed to be caused by closely spaced transverse cracks with intersecting longitudinal cracks, as shown in figure 3.25. Overall the east bound lane was in poor shape with thirty-three patches in the outside lane alone over the length of the project.



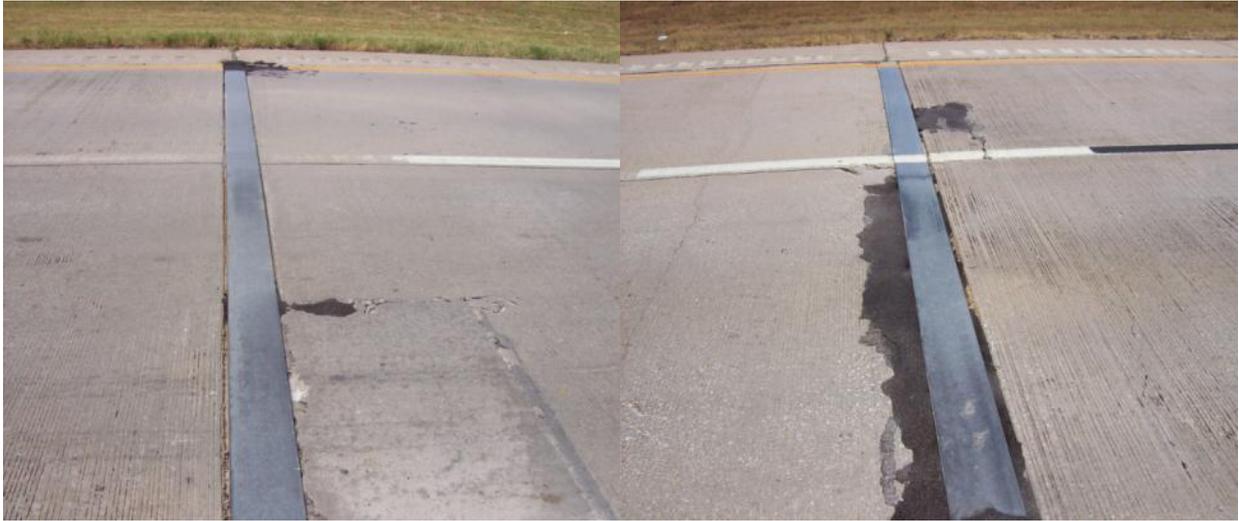
**Figure 3.24** – Two instances where a Y-crack seemed to be developing into a punch-out



**Figure 3.25** – Punch-outs forming between closely spaced transverse cracks

Figure 3.26 shows the terminal joint at the beginning (left) and the end (right) of the west bound inspection. The joints appeared to be in poor shape with several patches. The west bound lane had many patches of both asphalt and Portland cement. Some of the patches filled with asphalt left an outline that seemed to look like a Y-crack, see figure 3.27, while many of the patches seemed to be caused by closely spaced transverse cracks with longitudinal connecting cracks, see

figure 3.28. By observing the west bound lane it was clear that along with the numerous patches, asphalt joint sealant had been applied to many of the transverse cracks along the roadway, see figure 3.29.



**Figure 3.26** – (left) Terminal joint at west bound beginning, (right) terminal joint at west bound end



**Figure 3.27** – Y-shaped patches



**Figure 3.28** – Patches caused by closely spaced transverse cracks



**Figure 3.29** – Transverse cracks sealed with an asphalt joint sealant

While several Y-cracks showed signs of breaking up, figure 3.30 (left), most of the observed distresses appeared to be caused by closely spaced transverse cracks, see figure 3.30 (right). It

was also noted that the image in figure 3.30 (right) showed a longitudinal steel splice. The steel appeared to be approximately two inches from the concrete surface.



**Figure 3.30** – (left) Y-crack breaking up, (right) distress caused by close transverse cracking

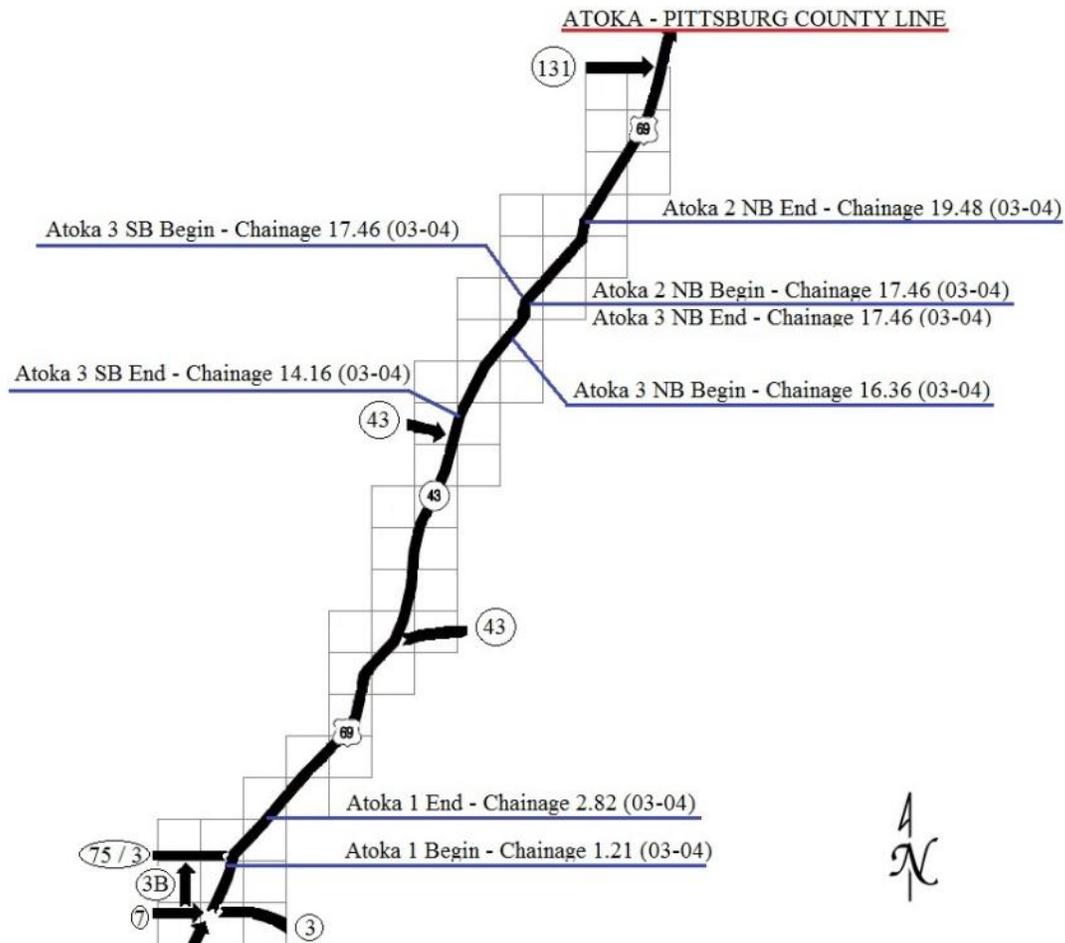
As mentioned previously, the east bound lane was in poor shape, with thirty-three patches in the outside lane alone; however, the east bound lane was in significantly better shape than the west bound lane, which recorded two-hundred seventy-seven patches in the outside lane alone. One factor that could be helping provide these failures is the pavements thickness. This pavement matches Atoka County for the thinnest CRCP pavement in the state at nine inches.

## **3.7 Atoka County**

### **3.7.1 Overview**

The series of CRC pavements studied in Atoka County lie along US 69, just north of the town Atoka. Several of the CRC pavements have been overlaid either by asphalt or concrete white-topping; therefore, the entire pavement was not available for inspection. Figure 3.31 shows the pavements that were a part of the inspection, each square in the figure represents one square mile of area.

The inspection consisted of three CRCP projects. The ODOT project numbers for the sites are F-299(99), F-299(45), and F-299(35). The following sections will discuss the construction details of each project referring to them as Atoka 1, Atoka 2, and Atoka 3 respectively. The 2011 AADT data shows that the pavements transport 16,300, 14,700, and 14,700 VPD; while the plan-sets show that the pavements were designed to carry 9,000, 10,800, and 7,500 VPD correspondingly.



**Figure 3.31** – Layout of the Atoka county CRCP’s visited during inspection. The pavements are marked by county name, begin and end points, along with chainage and control section number.

### 3.7.2 Atoka 1

Atoka 1 was completed in 1989 by Wittwer. The pavement is ten inches thick with 0.61% and 0.07% longitudinal and transverse steel, according to the specifications provided by ODOT. The pavement is sitting on top of three inches of type A- ac and twelve inches of aggregate base with jointed plain concrete shoulders.

A visual inspection was performed on July 8, 2011. The inspection of Atoka 1 consisted of 1.25 miles north bound and 1.37 miles in the south bound direction; this is noticeably less than the total project length because the stretch had two bridges. The visual inspection consisted of counting Y-cracks and patches, while photographing subjects of interest. The chainage of each patch and photograph was also noted for future reference. While inspecting this particular pavement another test was also performed; it consisted of measuring the distance between cracks for a random five-hundred foot section.

### 3.7.3 Atoka 1 Inspection Details

The pavement in the north bound lane inspection contained several patches. Many of these patches were not full lane patches and punch-outs had developed near the patch (figure 3.32 right), while some patches showed signs of distress (figure 3.32 left). Most of the w-shaped beams used at terminal joints showed signs of deterioration. Signs of deterioration were spalling of the CRCP, AC patching, and missing sections of the w-shape, see figures 3.33 and 3.34.



**Figure 3.32** – Small patch with distress developing close by



**Figure 3.33** – Terminal joints showing signs of spalling



**Figure 3.34** – Terminal Joints with different kinds of distress

The pavement did exhibit Y-cracks; however, the first Y-crack in the north bound lane did not occur until 0.13 miles into the project. Figure 3.35 shows a typical Y-crack for the area. It was noted that a few of the Y-cracks appeared to be leading to punch-outs, mainly because of longitudinal cracks connecting the two legs of the “Y”, see figure 3.36.



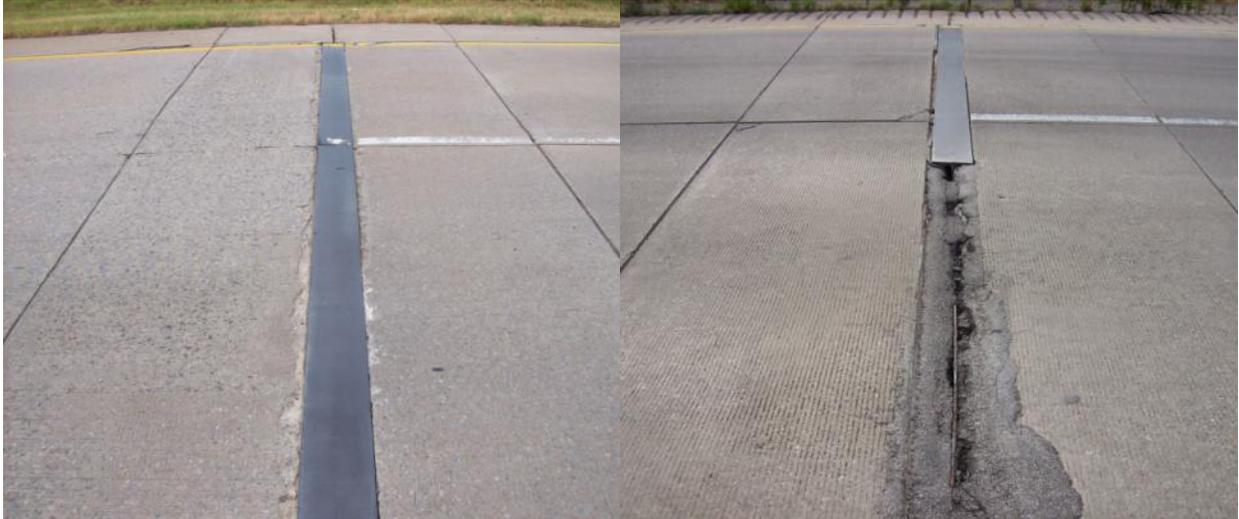
**Figure 3.35** – Typical Y-crack for the area



**Figure 3.36** – Y-cracks leading to punch-out and eventually failure

The south bound lane showed many of the same characteristics as the north bound lane. Several of the w-shaped beams at the terminal joints had deterioration, see figure 3.37. Figure 3.38 (left) shows a patch starting to break up and possibly develop into a punch-out; meanwhile, figure 3.38 (right) shows the stretch of roadway in the south bound lane where the crack spacing measurements were taken. Overall, both lanes seemed to be in similar condition, and both

recorded the same number of patches; however, the south bound lane contained one-hundred more Y-cracks than the north bound lane.



**Figure 3.37** – Terminal joints showing distress



**Figure 3.38** – (left) patch showing signs of distress (right) area of road where crack survey was taken

### **3.7.4 Atoka 2**

Atoka 2 was completed in 1986 by Koss. The pavement is nine inches thick with 0.50% and 0.08% longitudinal and transverse steel respectively, according to the specifications provided by ODOT. The pavement is sitting atop three inches of type C ac with jointed plain concrete shoulders. Type C ac is equivalent to a super pave S5 mix which has a 3/8 inch NMS.

A visual inspection was performed on July 8, 2011. The inspection of Atoka 2 consisted of 1.37 miles in the north bound lane only; the south bound lane was not CRCP and appeared to be a concrete overlay. The visual inspection consisted of counting Y-cracks and patches per mile, while photographing subjects of interest. The chainage from the beginning of the project to the location of every patch and photograph was also documented for future reference.

### **3.7.5 Atoka 2 Inspection Detail**

Atoka 2 north bound showed wear throughout the length of the pavement. The roadway surface tines had been greatly worn down in most places; this can be seen in figure 3.39. The construction and terminal joints also showed signs of deterioration, mainly spalling of the pavement edge along the joint, as seen in figure 3.40. Several large patches displayed map cracking as seen in figure 3.40 (right); however, these sections had not led to any punch-out or other type of failure. The project recorded four patches over its length, and none of the patches showed any clues as to what may have caused them. The roadway also contained many Y-cracks, with none of them showing signs of developing into punch-outs.



**Figure 3.39** – Spalled terminal joint with worn roadway surface



**Figure 3.40** – Spalled terminal joints, with the (right) displaying map-cracking

### **3.7.6 Atoka 3**

Atoka 3 was completed in 1986 by Northern Improvements. The pavement is nine inches thick with 0.50% and 0.08%, longitudinal and transverse steel respectively. The thickness and steel contents were taken from the specifications provided by ODOT. The pavement is sitting atop three inches of type C ac (3/8 Nominal Maximum Aggregate (NMS)), with jointed plain concrete shoulders.

A visual inspection was conducted on July 8, 2011. The inspection of Atoka 3 consisted of 1.03 miles in the north bound lane and 3.15 miles in the south bound lane. The visual inspection consisted of counting Y-crack and patches, while photographing subjects of interest. The chainage of each patch and photograph was also documented for future reference. A crack spacing measurement procedure was also implemented to determine the mean spacing, standard deviation, and coefficient of variation for a five-hundred foot section of pavement in both directions.

### **3.7.7 Atoka 3 Inspection Details**

The north bound lane of Atoka 3 had many patches, especially for the length of roadway inspected. The pavement had twenty-two total patches over a 1.08 mile stretch. The majority of the patches seemed to be caused by closely spaced transverse cracks intersected by longitudinal cracks, as seen in figures 3.41 and 3.42. However, a few of the patches seemed to have been caused by the breakup of Y-cracks. The Y-cracks seemed to have longitudinal cracks intersecting the two legs of the “Y”, as seen in figure 3.43 (left). Several of the patches left no evidence of the cause of the patch, as seen in figure 3.43 (right). It was noted that in figure 3.43 (right) the longitudinal steel was protruding through the patch. Despite the fact that a few of the Y-cracks

appeared to have caused failures, the majority of the distress seemed to be the result of closely spaced transverse cracks being intersected by longitudinal cracks, see figure 3.44.



**Figure 3.41** – Patches caused by closely spaced transverse cracks



**Figure 3.42** – More patches cause by closely spaced transverse cracks



**Figure 3.43** – (left) Patch caused by a Y-crack (right) patch with steel protruding through



**Figure 3.44** – Two transverse cracks intersected by a longitudinal crack

The south bound lane appeared to be in worse condition than the north bound lane; it contained more patches per mile and showed promise for future patches. Like the Okfuskee County project, several of the transverse cracks had been sealed with an asphalt crack sealant material. The majority of the visible patches seemed to be caused by transverse cracks being intersected by longitudinal cracks, see figures 3.45 – 3.47. Several future and developing punch-outs were

also noted and appeared to be caused by the same phenomenon, see figure 3.48. The south bound lane displayed several instances where multiple transverse cracks were intersected by longitudinal cracks, see figure 3.49. Despite the majority of the distresses appearing to be caused by transverse cracks being intersected by longitudinal cracks, a few patches seemed to display evidence of a Y-crack causing the patch, see figure 3.50. However, the instances were very miniscule compared to the other cause.



**Figure 3.45** – Transverse cracks intersected by longitudinal cracks



**Figure 3.46** – Transverse cracks intersected by longitudinal cracks



**Figure 3.47** – Transverse cracks intersected by longitudinal cracks



**Figure 3.48** – Two transverse cracks intersected by a longitudinal crack



**Figure 3.49** – Multiple transverse cracks intersected by a longitudinal crack

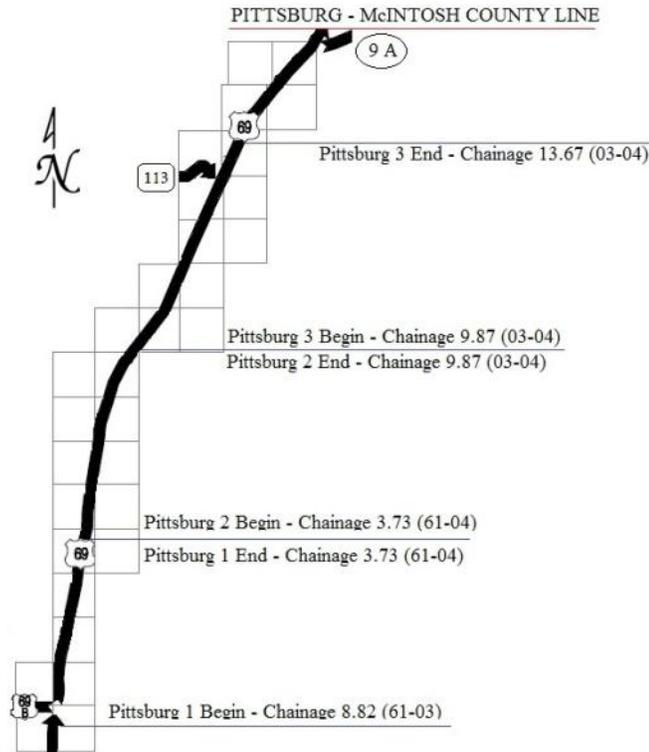


**Figure 3.50** – Patches that appear to be the result of a failed Y-crack

## **3.8 Pittsburg County**

### **3.8.1 Overview**

The series of CRC pavements studied in Pittsburg County lie along US 69, between the town of McAlester and the McIntosh-Pittsburg County line. Figure 3.51 shows the location of each project for the region, where one square represents one square mile of area.



**Figure 3.51** – Layout of the Pittsburg county CRCPs visited during inspection. The pavements are marked by county name, begin and end points, along with chainage and control section number.

Three CRCP projects were inspected in Pittsburg County. The ODOT project numbers for these projects are MAF-186(183), MAF-186(185), and DPIY-204(001). These projects will be referred to as Pittsburg 1, Pittsburg 2, and Pittsburg 3, respectively. According to the 2011 AADT provided by ODOT, Pittsburg 1 and 2 were estimated to handle 16,100 VPD, while Pittsburg 3 was assessed to carry 15,200 VPD; meanwhile, the plan-sets for the projects showed the design ADT to be 20,000 for Pittsburg 1 and 2 and 9,000 for Pittsburg 3.

All three of the Pittsburg County projects have the same characteristics; the only differing factor between the projects is the year of completion, with Pittsburg 1 completed in 1991, Pittsburg 2

completed in 1989, and Pittsburg 3 completed in 1994. Otherwise, the pavements were all constructed by Koss and are ten inches thick, with 0.61% and 0.07% longitudinal and transverse steel respectively. The steel contents and thicknesses were taken from the specifications provided by ODOT. The pavements are supported by four inches of open graded concrete base and twelve inches of stabilized aggregate, with jointed plain concrete shoulders.

A visual inspection for Pittsburg County was conducted over a course of two days with the north bound direction being inspected July 9, 2011 and the south bound direction inspected on July 10, 2011. The visual inspections consisted of counting Y-cracks and patches, while photographing subjects of interest. The chainage of each patch and photograph was also noted for future reference.

### **3.8.2 Pittsburg 1 Inspection Details**

Overall Pittsburg 1 north bound was in good shape, but it showed signs of future deterioration. The terminal joints at the project were in good shape overall, displaying only a small amount of spalling, see figure 3.52. The pavement contained a few distresses (figure 3.53 (left)) that seemed to be random and unrelated to any cracking. It was also noted that many cracks, both Y-cracks and transverse cracks had spalling, see figure 3.53 (right). There were also a few Y-cracks showing signs of beginning to develop into punch-outs, see figure 3.53 (right). There were several other places along the pavement where closely spaced transverse cracks were being intersected by longitudinal cracks and beginning to form punch-outs, see figure 3.54. The North bound lane also contained a few patches of map-cracking, see figure 3.55. Despite all of this the north bound lane only contained two patches over its four mile length, see figure 3.56.



**Figure 3.52** – Terminal joints at north bound beginning and end



**Figure 3.53** – (Left) Large circular distress, (right) Spalled Y-crack beginning to break up



**Figure 3.54** – Two developing punch-outs



**Figure 3.55** – Stretch of pavement showing signs of map-cracking



**Figure 3.56** – The two patches in the north bound lane

The north bound lane also contained several places where past testing had been done, see figure 3.57. These testing sites consisted of several cores. Some of the cores had been taken from along the outside edge of the pavement and even in the shoulder. A few cores were also taken out of the roadway, and showed signs of sensors of some type being implemented, see figure 3.58.



**Figure 3.57** – (right) road test sign found beside roadway, (left) one of several large cores taken



**Figure 3.58** – (left) cores taken from shoulder, (right) core with sensor wire extending to shoulder

The south bound lane was in a similar condition to the north bound lane. The south bound lane did show more deterioration at the terminal joints, figure 3.59. Like the north bound lane, the south bound lane seemed to have several instances where closely spaced transverse cracks led to distress, see figure 3.60 (left). The south bound lane also contained a Y-crack beginning to develop into a punch-out, but this was a rare case when compared to the closely spaced transverse cracks leading to distresses, see figure 3.60 (right).



**Figure 3.59** – Terminal joints at the beginning and end of the project



**Figure 3.60** – (left) Two closely spaced transverse cracks, (right) Y-crack developing into a punch-out

Overall both directions of CRCP had very similar characteristics. Both contained a high Y-crack per mile average and a low number of patches, while showing signs of future deterioration. They were also similar in that the main cause of deterioration seemed to be closely spaced transverse cracks being intersected by longitudinal cracks.

### 3.8.3 Pittsburg 2 Inspection Details

The northbound lane of Pittsburg 2 was in overall good shape. Figure 3.61 shows the terminal joints on each end of the project; both appear to be in good shape, displaying only a minor amount of spalling. The pavement recorded just two patches over its six mile length, despite the occurrence of Y-cracking. The patches did not leave any evidence of what may have caused them; however, there was one Y-crack that appeared to be developing into a punch-out, see figure 3.62. Even though one Y-crack seemed to be developing into a punch-out, none of the other Y-cracks showed this.



**Figure 3.61** – Terminal joints on each end of the project



**Figure 3.62** – Y-crack developing into a punch-out

The south bound lane of Pittsburg 2 was in worse condition than the north bound lane. Figure 3.63 shows the terminal joints at the project beginning and end, which like the north bound lanes, were in good condition, displaying only minor signs of spalling. The south bound lane contained eight patches over its six mile length, which is four times more than the north bound lane. All of the patches seemed to be the result of closely spaced transverse cracks, see figures 3.64. It was noted during the inspection that the third mile of the project contained a heavy amount of Y-cracking -- two-hundred and eight cracks within the mile. However, this stretch of pavement only contained one patch. Despite the occurrence of patches and Y-cracks, there was no evidence of a Y-crack leading to a punch-out or patch over the six miles of pavement in the southbound direction.



**Figure 3.63** – Terminal joints at the project beginning and end



**Figure 3.64** – Patches formed near closely spaced transverse cracks

### 3.8.4 Pittsburgh 3 Inspection Details

Pittsburg 3 north bound was in good shape as far as patches, recording zero over its 3.66 mile length. However, most of the project contained map-cracking, but no punch-outs or patches had developed as a result of the map-cracking. The sections of roadway that did have map-cracking reported slightly lower numbers of Y-cracks. Figure 3.65 shows the map-cracking. Figure 3.66 shows the w-shaped beams used at the joints; these joints show signs of spalling, which could possibly be enhanced by the map-cracking.



**Figure 3.65** – Close ups showing map-cracking

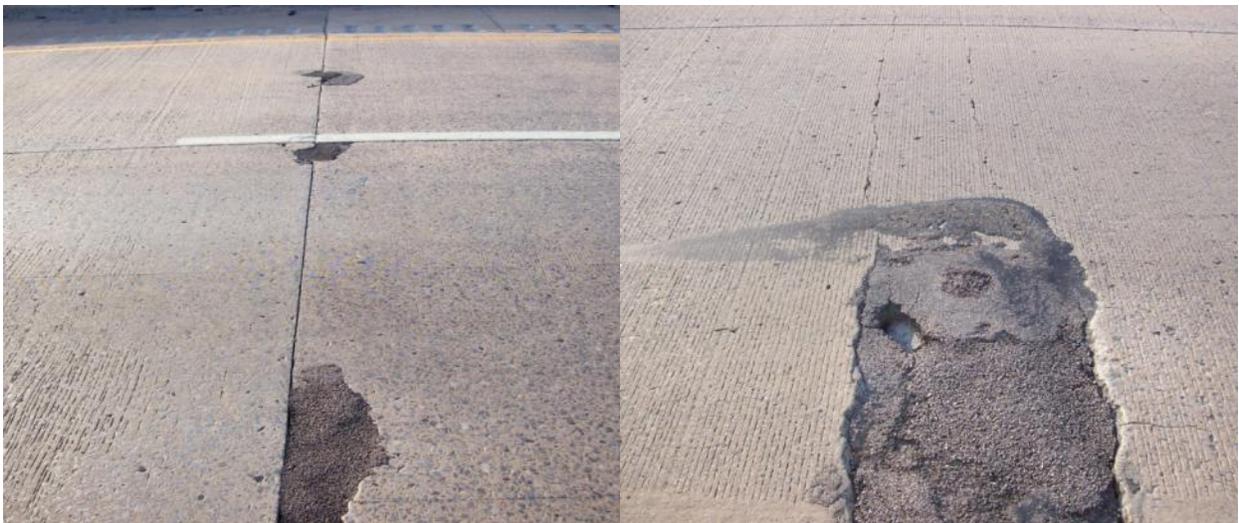


**Figure 3.66** – Terminal joints at project beginning and end

Pittsburg 3 south bound displayed many of the same issues as its northbound counterpart. The south bound lane contained map-cracking throughout its entire length, see figure 3.67. Despite having one of the lowest amounts of Y-cracks per mile, the project did have three patches over its 3.62 mile length. None of the patches seemed to be related to the Y-crack phenomenon, figure 3.68. Figure 3.69 shows the terminal joints at the projects beginning and end, both of which show deterioration.



**Figure 3.67** – Map cracking in the south bound lane



**Figure 3.68** – Patches in the south bound lane



**Figure 3.69** – Terminal joints at the beginning and end of the project

Overall the CRCP at Pittsburg 3 appeared to be in good shape, containing only three patches over its 7.28 mile length. However, the presence of map-cracking and close transverse cracking interconnected by longitudinal cracks seems to hint at future deterioration.

## CHAPTER IV

### ODOT CRCP DATABASE

#### **4.1 Overview**

The CRCP database contains information such as year constructed, percentage of longitudinal and transverse steel, location, type of shoulder, type of base and subbase, edge drain presence, and ODOT standards. The database contains this information for all of the CRCPs in the state of Oklahoma.

#### **4.2 CRCP Database Update**

Updating the CRCP database consisted of three main steps. Step one was to complete all entries contained in the original database. The next step was to locate and add all CRCP projects not in the original database. The final step was to complete all the new entries in the database.

Steps one and three were completed using standards, plan sets, and other information obtained through requests from ODOT. Step two was completed using historical research sites list, 2009 Interstate Highway Pavement History, Interstates Database, and Appendix B of McGovern et al. 1996. Once these steps were completed the database was formatted and finalized. Due to size and clarity constraints the CRCP database has been included as a separate Access database file.

## CHAPTER V

### TASK UPDATES

#### **5.1 Task C – Review of pavement management condition data**

During the 6 day trip around Oklahoma looking at CRCP projects, patches were counted, and their location was recorded using a hand held GPS unit. For each patch the GPS unit was used to measure the distance (in 0.01 mile increments) from the beginning of the project to the patch. These distances were then translated into chainages.

Work is currently in progress to correlate the patches observed during the 6 day trip, to data contained in the 2001, 2006, and 2008 Pavement Management Systems Databases. The intent of the correlation is to find out how patches increase over time and try to pin-point a few patches before they became patches. If this can be done, it would be possible to pin-point a few patches before they became patches, then look back at the downward images and see the distress before it was patched, and ultimately see what caused it.

#### **5.2 Task E – HYPERPAV III Modeling**

Closer crack spacing is thought to increase the occurrence of meandering or Y-cracks in CRCP. In order to investigate the effects of construction, material, and design choices on the crack spacing, a sensitivity analysis was performed using the HIPERPAV III software package. The sensitivity analysis was performed for five Oklahoma sites that were also included in a six-state field investigation of CRCPs conducted in 1991 (Tayabji, Zollinger, et al., Performance of CRC

Pavements - Volumes 2 & 3 1998). Three of these sites (OK-1, OK-2, and OK-3) were also investigated in the field. The analysis was performed using a CRCP Early-Age Module.

Future work related to the HIPERPAVE III modeling, includes possibly running more analysis for different CRCP projects, while compiling and analyzing the results obtained from the first set of analysis.

### **5.3 Task F – Correlations between Y-cracking and pavement performance**

No current progress has been made on this task. Once task C is closer to completion this task will begin.

### **5.4 Task G – Correlations between Y-cracking, design, materials, and construction**

No current progress has been made on this task. Once task E is closer to completion this task will begin.

### **5.5 Task H- Final Report**

No current progress has been made on this task. Once all tasks are closer to completion this task will begin.

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