# Impact of Asphalt Mixture Design Parameters on Transverse Cracking Performance and Laboratory Testing Results

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# DEDICATION

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# ABSTRACT

In cold climate regions, thermal cracking of asphalt pavements is a primary pavement distress. Cold climates cause thermal contraction within the pavement. When combined with the brittle behavior of asphalt at low temperatures, the thermally induced stresses are relieved by transverse cracks forming in the pavement. This cracking facilitates poor ride quality and premature failure of the pavement. There is currently no asphalt mixture performance test required by a majority of Department of Transportations (DOTs) in the United States to address the issue of thermal cracking. Previous research has indicated that fracture energy of asphalt mixtures is a reliable predictor of transverse cracking performance. This mechanistic property of asphalt mixtures can be found using the disk-shaped compact tension (DCT) test. Based on previous research, a low-temperature cracking performance specification that uses DCT fracture energy has been developed. This project focused on eighteen highways containing twenty-six separate study sections. The projects encompassed different construction techniques, material compositions and climatic zones. The results from field studies, analysis of the mixture parameters for each section and laboratory testing for sections are presented. The results provide validation for previous research that suggests the use of a performance test is vital to accurate projection of roadway transverse cracking performance. Other findings include reaffirmation of common knowledge about various mixture parameters. These findings include various suggestions that relate to positive and negative effects on both cracking amounts and performance testing. In nearly all instances, sections with an overlay construction type performed inferior to reclaimed construction types. For example, reclaimed sections exhibited roughly one-third the average transverse cracking amount of overlay sections. This phenomenon will be monitored in future studies.

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# CHAPTER 1: INTRODUCTION AND MOTIVATION

## 1.1 Introduction

Low-temperature cracking is the primary pavement distress in climates that experience extreme low temperatures and/or high rates of temperature drop. These are the prevalent climatic conditions in the northern states within the United States and all of Canada [1]. The discrete cracking of a material, as in the case of low temperature cracking, is a highly complicated phenomenon, and evaluation of the material beyond the linear response range helps close the gap between experimental results and actual field performance. Restrained by the layers below, the top layer of the pavement structure typically relieves these built up tensile forces by forming a transverse crack on the surface. These cracks lead to poor ride quality, along with expediting moisture related issues and potholes forming at the crack location.

The formation and propagation of low-temperature cracking is controlled by both the mechanical properties of asphalt and climatic conditions imposed on the pavement. Superpave specifications attempt to mitigate thermal cracking by mandating a specific low temperature grade for the asphalt binder (eg. PG XX-28 versus PG XX-34). While this contributes to thermal cracking prevention, specifying a low temperature binder grade does not account for the many variables in an asphalt mixture (aggregate types, gradation, recycled asphalt materials, aging, etc). In addition, not all asphalt binders of the same Superpave low temperature grade have equivalent mechanical properties (modified vs. neat, different sources of crude, etc). Research has shown that the fracture behavior of the asphalt mixture and the mechanical properties of the binder are equally important in terms of low-temperature transverse cracking performance [2], [3], [4]. Findings show that several factors impact the low-temperature cracking performance of asphalt pavements. Modifying the asphalt binder in a mixture is not in itself an adequate method of preventing this distress. A viable method of measuring an asphalt mixture's resistance to low-temperature cracking is through performance testing [5].

### 1.2 Motivation

The current state of practice for pavement design does not adequately address the issue of transverse cracking due to thermal contraction. The current hot mix asphalt specification for Minnesota (MnDOT 2360) focuses heavily on the volumetric properties of the asphalt mixtures [6]. This will lead to varying field results, as the specification can be met using differing materials of which some may be more effective than others. As explained in the introduction, a mandated performance test that correlates well to field cracking amounts would assist in bridging the disconnect between current design practice and field performance.

There is not currently a nationally adopted criterion for performance testing of asphalt mixtures. Previous research has conducted a thorough review of specifications from various State Departments of Transportation (DOTs) [5]. This research revealed that performance tests are required by several State DOTs, but the tests are limited to the prediction of rutting distresses. Two cracking performance tests in particular, DCT and semi-circular bending (SCB) testing, showed great potential for implementation as a performance test predicting low temperature cracking. This study also recommended evaluating the use of the indirect tensile test (IDT) as a performance measure [5]. Essentially, the current processes being used by State DOTs rely too heavily on volumetric properties. This thesis addresses the problems with attempting to correlate field performance and mixture design properties directly. Chapter 4 discusses in detail the correlations between DCT test results and field performance measures. The ability to effectively utilize a performance testing specification could greatly reduce the number of resources required in rehabilitation efforts on roadways.

## 1.3 Literature Review

Climatic conditions in Minnesota can be frigid during the winter months. According to the Minnesota Department of Natural Resources, the average temperature in the northern half of the state during winter months is 6°F (≈ -14°C). This can also include an average of up to 70 inches of snow [7]. Clearly, the climate creates a unique design situation for roadway designers. The extreme low temperature conditions define the controlling pavement distress: low temperature cracking. Previous research and common practice has shown that rutting is the primary pavement distress in many locations. Rutting is the act of permanent deformation in a flexible pavement. This typically occurs in locations where temperatures reach extreme high temperatures, causing the pavement structure to act more fluid and ruts to form in the wheel path of a roadway [8]. However in regions where temperatures are generally cold, rutting is far less concerning than thermal cracking.

As described in the introduction, thermal cracking occurs as a pavement contracts under extreme cold temperatures. Internal tensile stresses are relieved by the formation of transverse cracks on the surface of the asphalt [9]. These cracks accelerate the moisture related deficiencies within the pavement structure, as water and snow fill the crack. Freezing and thawing of this water leads to further roadway damage. Ride quality is also affected dramatically by transverse cracking.

A notable finding within this review was the impact of pavement type on cracking performance. Fatigue cracking for thin pavements has shown a greater tendency to feature bottom up cracking, while thicker pavements have exhibited more top down cracking [10]. This is relevant because thin overlays on a concrete pavement may actually feature fatigue cracking prior to thermal cracking taking place. While this is not covered within the scope of this study, it shows that pavement type must be considered during the design process and one single distress does not necessarily control in all instances.

The current practice for asphalt roadway design is highly dependent on volumetric properties. Very few State DOTs currently require the use of a performance test to predict the field performance of roadways [11]. Previous research has indicated that performance based testing improves the predictability of field cracking performance [12]. The ability to predict pavement performance is essential to reducing current operating costs of highway design. The performance data from various tests has successfully been used as a tool to predict field performance and refine the mix design process [13], [14].

### *1.3.1 Previous Research on Performance Tests*

Based on a study by Dave and Koktan that analyzed the feasibility of various performance measures, numerous performance tests have been studied for the potential for use in the application as a standardized design performance test. While many tests are presently being used in pilot studies, few are being used for thermal cracking performance [5]. Recently a large amount of effort has been exerted working with the asphalt material performance test. This test shows strong correlations for fatigue cracking performance [15], [16]. However, the applicability of this test for low temperature asphalt cracking is not currently available [5]. The Texas Overlay Tester is also a common performance test. This study attempted to evaluate the field performance of asphalt pavements [17]. The results from this were not applicable for the use of this research, however, as findings were related to fatigue and reflective cracking [18]. The evaluation of the indirect tension (IDT) test and fracture based performance tests were recommended [5].

The IDT was considered a prime candidate for a low temperature cracking performance test, as it is already involved in the mixture design process. A study conducted by Dave and Hanson focused on this specific test, which evaluated the potential for such an implementation. This study involved the compilation of thousands of records including cracking measurements and indirect tensile strength (ITS) of specimens. The data came from separate databases; therefore the mixture design information had to first be linked to field performance data. After the "mapping" process was complete, an analysis of a link between ITS and both mix design parameters and field performance was conducted. No useful correlations between the ITS and cracking performance were determined. However, asphalt content and performance grade (PG) spread were two parameters that showed a potential correlation to cracking performance [18]. Based on this study, the IDT is not recommended to be used as a performance measure for low temperature cracking. This recommendation and the mix design recommendations were taken into account during the research process discussed herein.

As a result, fracture mechanic based tests were considered the best indicator of low temperature cracking. The idea of using fracture mechanics for prediction of low temperature cracking began in the 1980's [19]. The two main testing procedures are the disk-shaped compact tension test (DCT) and the semi-circular bending (SCB) test [20]. These two tests are performed with essentially the same general procedure but differing geometries for the test specimens. Consistent with the findings from Dave and Hanson, fracture energy has also been found to be a superior indicator of low temperature cracking performance as opposed to indirect tension testing results [21]. The DCT fracture test has been successfully utilized in recent years for prediction of the low temperature cracking performance of asphalt pavements and overlays [3], [22], [23]. The DCT test measures a mechanistic property known as fracture energy ($G_f$). The test specimen geometry is a circular specimen with a single notch loaded in tension at low temperature. Fracture energy can be used to describe the fracture resistance of an asphalt mixture; mixtures with a high $G_f$ have better low temperature performance and are more desirable in cold climates. The DCT test has been shown to discriminate between asphalt mixtures better than other tests, such as the

Indirect Tensile Test. A study that evaluated State Department of Transportation (DOT) asphalt mixture specifications, as well as conducted a State of the Practice and State of the Art review on the topics of performance based specifications, recommended the use of the DCT test as a suitable performance test for low temperature cracking distress [5]. The repeatability of the DCT test is also superior to other fracture based tests [3]. Based on the research presented, fracture mechanics have the most promise for implementation into mix design methods.

While having a performance indicator is a major goal of this and many other studies, the ability to adjust the testing result is essential. In other words, if a performance indicator is accurate in prediction but cannot be controlled in the mix design phase, implementation of these tests is neither feasible nor beneficial. A study was conducted to determine the effect of temperature, presence of recycled asphalt pavement (RAP), binder modifier and aging on fracture energy results [24]. Another study also investigated the type of PG binder grade and the ability to predict field cracking [25]. There have been multiple studies conducted that looked at the impact of mix parameters on field performance, and others that observed the impact of mix parameters on fracture energy. However, a comprehensive study of the impact mix parameters have on both field performance and fracture energy has not been completed. This study will take all three factors into account when observing in-service roadways and the corresponding mix designs.

### *1.3.2 Application of Previous Studies to Current Research*

This project used an integrated approach of field evaluations and laboratory mixture fracture testing in the form of DCT testing. A well defined method for quantifying field cracking performance measures is a key to consistent and repeatable results. Long Term Pavement Performance (LTPP) databases are used to track field performance over extensive periods of time. This is crucial information for pavement distress models. This data has been used to improve models that predict pavement distresses [13], [14], [26]. According to the study conducted by Dave and Hanson, several studies have been completed that analyze the effect of single or multiple mix parameters in relation to improvement of performance prediction models. However, the only research conducted that developed a cumulative database containing both mix design information and pavement performance data was that of Dave and Hanson [18]. During this study, several databases attempting to analyze mix parameters and field cracking performance were identified as having limited or incomplete datasets [9], [27]. The ability to have an all-encompassing database that contains both cracking performance and mix design records is both brilliant and difficult. It relies on the individuals' surveying to be consistent with each section of roadway they investigate. However, a database that includes all these factors would make research in this field far more consistent and streamline any initial work that must take place prior to testing.

## 1.4 Project Objectives

As noted in the project motivation, a performance testing procedure is currently not required by many DOTs. The focus of this project is to validate that using a performance test, in this case the DCT test, will be advantageous and accurate in the prediction of low temperature transverse cracking in asphalt pavements. Therefore, eighteen field sites were selected as study sections to evaluate transverse cracking performance and laboratory performance of those sections. The efforts of this research project included the following:

1. A field performance evaluation was conducted in various locations to determine in service transverse cracking performance.
2. Laboratory evaluation of field procured samples from each site was conducted using the DCT test.

3. A comparison of field performance, mixture design parameters and laboratory testing was used to determine if any relationships are active between variables of interest.

This procedure allowed several recommendations to be made for future research projects. It also confirms that the use of a single mix design parameter as a performance indicator is neither accurate nor recommended. The conclusions and recommendations from this project provide valuable information for current and future projects related to low temperature asphalt cracking.

### 1.5 Thesis Organization

This thesis is separated into five chapters. Chapter 1 provides background on the basis for this research. Previous projects related to the issues discussed herein are identified and explained to truly understand the relevance of the research being conducted. Chapter 2 introduces the first phase of this research project: field performance evaluation. This discusses the process of field evaluation and cracking performance measures that were used in this analysis. Chapter 3 gives a detailed background on the DCT test and results. Comparisons between roadway section testing results are introduced here. Finally, Chapter 4 details the comparison process for field performance, mix parameters and laboratory testing results. The summary, conclusions and recommendations are presented in Chapter 5. The various appendices provide detailed site visit summaries, raw data, and additional plots not essential for analysis. Figure 1 illustrates the general layout of this thesis, highlighting the key points discussed throughout the entirety of this document.

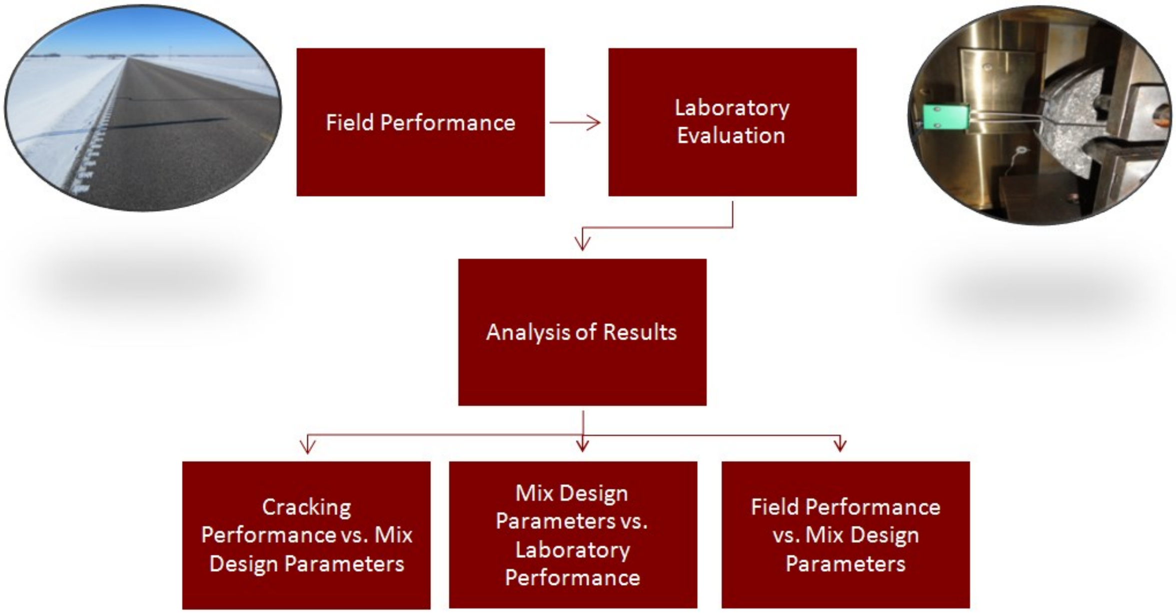

Figure 1: Description of general thesis layout

# CHAPTER 2: FIELD PERFORMANCE

## 2.1 Introduction

The first objective in this research project involved the field evaluation of several roadways across Minnesota. Eighteen asphalt roadways were chosen for this study through interactions with the technical advisory panel (TAP) for the project. The projects were chosen to obtain a wide cross-section of varying asphalt mixture designs and pavement structures. During the course of this task construction plans were evaluated, site visits were conducted, field cracking performance was determined and field sampling plans were developed. This chapter will provide an overview of the individual site visits, cracking performance information for each pavement section and the field sampling plans.

## 2.2 Description of Observed Field Sections

In order to study the effects of the asphalt mix parameters on the field cracking performance, as well as to assess the suitability of laboratory performance tests in predicting cracking performance, a total of 18 highway projects were selected. The field sites were selected through the interactions between the researchers and the technical advisory panel for the project. The design factors for the eighteen sites varied greatly; traffic level, climatic conditions and wear course thickness differed between each of the sites. The location of the sections with respect to MnDOT district layout is shown in Figure 2. The sections are located along the following highways:

- Trunk Highway 1 (District 1)
- Trunk Highway 2 (District 2)
- Trunk Highway 6 (District 3)
- Trunk Highway 6 (District 2)
- Trunk Highway 9 (District 2)

- County State Aid Highway 10 (District 1)
- Trunk Highway 10 (District 3)
- Trunk Highway 10 (District 4)
- Trunk Highway 25 (District 3)
- Trunk Highway 27 (District 3)
- Trunk Highway 28 (District 4)
- County State Aid Highway 30 (Metro)
- Interstate 35 (Metro)
- Trunk Highway 53 (District 1)
- Trunk Highway 113 (District 2)
- Trunk Highway 210 (District 3)
- Trunk Highway 212 (Metro)
- Trunk Highway 220 (District 2)

A = TH 1 (Dist. 1)
B = TH 2 (Dist. 2)
C = TH 6 (Dist. 3)
D = TH 6 (Dist. 2)
E = TH 9 (Dist. 2)
F = CSAH 10 (Dist. 1)
G = TH 10 (Dist. 3)
H = TH 10 (Dist. 4)
I = TH 25 (Dist. 3)
J = TH 27 (Dist. 3)
K = TH 28 (Dist. 4)
L = CSAH 30 (Metro)
M = I-35 (Metro)
N = TH 53 (Dist. 1)
O = TH 113 (Dist. 2)
P = TH 210 (Dist. 3)
Q = TH 212 (Metro)
R = TH 220 (Dist. 2)

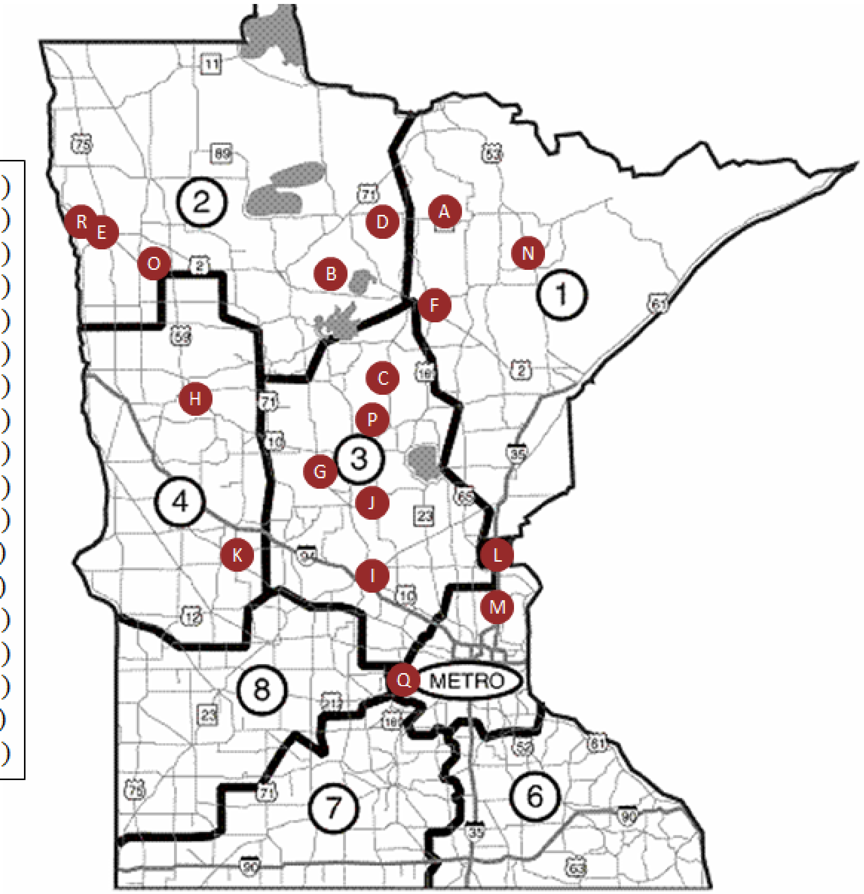

Figure 2: Locations of study sections

## 2.3 Field Evaluation Procedures

The aim of the field visits was twofold: (1) conduct crack surveys to quantify the cracking performance and (2) develop sampling plans for sample procurement. The information available through the MnDOT pavement management system identifies transverse cracking severity along with percent cracking determined through the number of cracks in a pavement section. Researchers in this study conducted the manual cracking surveys and hence were able to measure the actual length of cracking in each study section; however the severity of cracks in terms of crack width was not documented. Section 2.5 explains various cracking measures that were used for data analysis.

When first arriving at a site, the researchers conducted a full length evaluation of the desired roadway by driving the entire length of a project where project is defined as the stretch of highway constructed under same SP number. In most situations, a good performing and poor performing portion of the roadway were identified. These areas were treated as distinct study sections. Among the 18 highways being evaluated, 26 study sections were identified. Where a section featured a driving and passing lane, these were combined into a single section. Also, in instances where two different mixes or pavement structures were used (such as mill and overlay versus reclaim), distinct sections were used. These areas were noted and returned to at the conclusion of the full length review. A reference post (or mile post), herein referred to as RP, or landmark was found in the area(s) of interest and used as a starting point for field evaluation. The evaluation consisted of inspecting a 1000 foot section, measured from the desired RP. Along this 1000 foot section, any transverse cracks were measured in both length of crack and distance from the starting point.

Field cores were taken at these sites for disk-shaped compact tension (DCT) testing. The locations for coring were at 200 foot increments along the 1000 foot inspection corridor. During the field visits the GPS coordinates were recorded at the start, end and each coring location in the survey section. This resulted in five coring locations per inspection site. The total number of cores that were extracted varied relative to wear course depth. Coring plans were then developed for each study section.

## 2.4 Field Visit Summary

The site visit dates for various projects are indicated below. Due to very high traffic levels, a formal site visit was not conducted at I-35 and Trunk Highway 212. The pavement management data from these pavements is available and used in the analysis. The cored samples were also extracted from the roadways and included in the study.

- Trunk Highway 1 (District 1): June 18, 2014
- Trunk Highway 2 (District 2): January 3, 2014
- Trunk Highway 6 (District 3): July 30, 2014
- Trunk Highway 6 (District 2): January 3, 2014
- Trunk Highway 9 (District 2): January 2, 2014
- County State Aid Highway 10 (District 1): July 30, 2014
- Trunk Highway 10 (District 3): October 17, 2013
- Trunk Highway 10 (District 4): July 29, 2014
- Trunk Highway 25 (District 3): January 8, 2014
- Trunk Highway 27 (District 3): April 10, 2014
- Trunk Highway 28 (District 4): April 10, 2014
- County State Aid Highway 30 (Metro): July 28, 2014
- Interstate 35 (Metro): December 12, 2012 (drive through survey)
- Trunk Highway 53 (District 1): June 18, 2014
- Trunk Highway 113 (District 2): January 2, 2014
- Trunk Highway 210 (District 3): January 8, 2014
- Trunk Highway 212 (Metro): No site visit conducted
- Trunk Highway 220 (District 2): July 29, 2014

Table 1 summarizes the data gathered from the site visits along with pertinent information from the construction plans. Please note that the performance measures of “Good” and “Poor” are qualitative and were used to set up distinctly different study sections. The actual performance was determined using crack counts and pavement management data, as discussed later in this report. This information was integral in the analysis segment of research.

Any cells in Table 1 listed as “N/A” are classified as such for one of the following reasons:

- RP/Landmark: no formal site visit was conducted at this site due to high traffic conditions
- Performance: there was no substantial difference throughout the entire section and one survey was sufficient; or no formal site visit was conducted at this site due to high traffic conditions
- Lane: no formal site visit was conducted at this site due to high traffic and no historical data was immediately available

Table 1: Summary of site visits

| Section | SP # | RP / Landmark | Construction Year | Performance | Lane | Construction Type |
|---|---|---|---|---|---|---|
| TH 1 | 8821-103 | RP 235 | 2008 | Poor | D | 1.5" O/L on old AC |
| TH 1 | 8821-103 | RP 230 | 2008 | Good | D | 4" O/L on reclaimed AC |
| TH 2 | 1102-59 | RP 157 | 2003 | N/A | D | 4" O/L on old AC |
| TH 6 | 1103-25 | RP 53 | 2010 | N/A | D | 1.5" M/O |
| TH 6 | 3107-42 | RP 118 | 2004 | Poor | D | 1.5" O/L on old AC |
| TH 6 | 3107-42 | RP 123 | 2004 | Good | D | 4.5" O/L on reclaimed AC |
| TH 9 | 6010-26 | RP 208 | 2011 | Poor | D | 3" O/L on reclaimed AC |
| TH 9 | 6010-26 | RP 214 | 2011 | Good | D | 3" O/L on reclaimed AC |
| CSAH 10 | 031-610-016 | Jct 445B | 2012 | Poor | D | 1.5" O/L on old AC |
| CSAH 10 | 031-610-016 | Jct 446 | 2012 | Good | D | 3" M/O |
| TH 10 | 0502-95 | RP 159 | 2005 | Poor | D/P | 4" M/O (sealed cracks) |
| TH 10 | 0502-95 | RP 161 | 2005 | Good | D/P | 4" M/O (cracks not sealed) |
| TH 10 | 5606-42 | RP 75 | 2013 | N/A | D/P | 3.5" M/O |
| TH 25 | 7104-19 | Jct 17 | 2011 | N/A | D | New BAB |
| TH 27 | 4803-19 | RP 171 | 2010 | Poor | D | 3" M/O |
| TH 27 | 4803-19 | RP 174 | 2010 | Good | D | 3" M/O |
| TH 28 | 6104-11 | RP 81 | 2012 | Poor | D | 4.5" M/O |
| TH 28 | 6104-11 | RP 88 | 2012 | Good | D | 4.5" M/O |
| CSAH 30 | 1306-44 | Jct TH 95 | 2012 | N/A | D | 6” M/O |
| I-35 | 0283-26 | N/A | 2009 | N/A | N/A | 4" M/O on existing concrete |
| TH 53 | 8821-177 | Jct 169 | 2008 | N/A | D/P | 1.5" M/O |
| TH 113 | 4407-12 | RP 10 | 2006 | Poor | D | 1.5" O/L on old AC |
| TH 113 | 5413-10 | RP 5 | 2006 | Good | D | 5" O/L on reclaimed AC |
| TH 210 | 1805-72 | RP 118 | 2010 | N/A | D/P | 2" O/L on existing concrete |
| TH 212 | 1017-12 | N/A | 2008 | N/A | D/P | New BAB |
| TH 220 | 6016-37 | RP 12 | 2012 | N/A | D | 3” M/O |

M/O = Mill and Overlay; O/L = Overlay ; BAB = Bituminous on Aggregate Base
*Where the term “Jct” is referenced as a landmark, a signpost for the specific roadway is being specified.

## 2.5 Cracking Performance

Crack counts from site visits were combined with the MnDOT Pavement Management System (PMS) data to quantify cracking over the service life of the sections. The PMS data source contains all of the field performance (distress) data, specifically cracking performance of different pavement sections. Information pertaining to route types (Interstates, State highways, and US highways) and route numbers are included in this data source which contains 188 unique routes.

The distress information includes transverse cracking, longitudinal cracking, rutting, raveling, patching, and longitudinal joint deterioration. Due to the main focus of this study pertaining to transverse cracking of asphalt pavements, transverse cracking was the only measure included in the analysis phase. Details on the statistical analysis of pavement cracking performance from PMS data against the mix design information were conducted in previous research. The present task focuses on 18 pavement projects and a total of 26 sections. The PMS data for these sections along with cracking performance from field visits is compiled and presented in this section.

### *2.5.1 Cracking Performance Measures*

The transverse cracking data in the PMS data is collected based on the severity of the cracks; low, medium and high. For each severity level the data is reported in terms of percent cracking (% cracking) which is calculated as 2 times the number of cracks per 500 feet length of the survey section. For purposes of conducting a statistical analysis between the amount of cracking and laboratory tests as well as asphalt mix parameters, a number of measures of field cracking performances can be calculated. In this study, the researchers looked at transverse amounts in terms of total cracking. This is the sum total of low, medium and high severity cracks.

The total cracking amounts for a given PMS section for each year of distress survey can be used to calculate additional cracking measures that are representative of field cracking performance. These measures for transverse cracking are described in

Table 2. Please note that all data presented in this report includes the crack counts that researchers collected during the site visits. Thus, the field visit information was incorporated with the PMS data providing the cracking performance information for the pavements from their construction until 2013/2014.

Table 2: Description of transverse cracking measures

| Measure | Description | Unit |
|---|---|---|
| **Maximum Total Transverse Cracking Amount (MTCTotal)** | Maximum transverse cracking amount (low + medium + high) of all survey years for a pavement section normalized against number of years for which pavement section has been in service. | % cracking/year |
| **Maximum Total Transverse Cracking Rate (MTCRTotal)** | Maximum increase in total transverse cracking amounts (low + medium + high) between any two consecutive years of service. | % cracking/year |
| **Average Total Transverse Cracking (ATCTotal)** | Sum of total transverse cracking (low + medium + high) for every survey year of a pavement section normalized against number of years for which pavement section has been in service. | % cracking/year |
| **Weighted Average Total Transverse Cracking (WATCTotal)** | Total transverse cracking (low + medium + high) for every survey year of a pavement section is first normalized against the corresponding survey year. The sum of these values is then normalized against number of years for which pavement section has been in service. | % cracking/year/year |
| **Total Transverse Cracking (TCTotal)** | Sum of the total transverse cracking (low + medium + high) work over the service life. Total area is then normalized against the number of years for which pavement section has been in service. | % cracking |

The primary function behind all five cracking measures is to determine a measure that accurately depicts the cracking performance for a section. A roadway experiencing 0% cracking for the first four years of the service life then cracking to a current amount of 50% is a superior performer to a roadway cracking at 50% in year one and staying at 50% until the current time period. The five measures each portray the transverse cracking in a different fashion, so analyzing all five measures gives merit to each performance. An explanation of the transverse cracking measures follows, along with a graphical representation in Figure 3.

A. Maximum Total Transverse Cracking Amount (MTCTotal): this value is the absolute maximum transverse cracking amount experienced by the section, which is then normalized against the total number of years in service for the roadway. In this instance, 59 percent is the maximum amount of transverse cracking for the pavement over a service life of 11 years. This would result in a maximum total transverse cracking amount of 5.36 percent per year.

B. Maximum Total Transverse Cracking Rate (MTCRTotal): this is simply the greatest increase in transverse cracking between any two consecutive years. For example, Trunk Highway 2 exhibited a 12 percent increase in transverse cracking from the year of construction to the first year in service. Thus, 12 percent is the maximum total transverse cracking rate.

C. Average Total Transverse Cracking (ATCTotal): this measure is not explicitly defined in Figure 3. This value is the sum of all total transverse cracking measurements over the service life of the pavement divided by the total service life. Using the values from Figure 3, the calculation for average total transverse cracking is performed as follows:

$$\text{ATCTotal} = \frac{12 + 19 + 26 + 27 + 28 + 28 + 28 + 33 + 38 + 49 + 59}{11}$$

$$= 31.5\ \%\ \text{cracking/yr}$$

D. Weighted Average Total Transverse Cracking (WATCTotal): this particular measure is not explicitly defined in Figure 3. This value is the sum of all total transverse cracking measurements first normalized against the individual survey years, then divided by the total service life. Using the values from Figure 3, the calculation for average total transverse cracking is performed as follows:

$$\text{WATCTotal} = \frac{\frac{12}{1}+\frac{19}{2}+\frac{26}{3}+\frac{27}{4}+\frac{28}{5}+\frac{28}{6}+\frac{28}{7}+\frac{33}{8}+\frac{38}{9}+\frac{49}{10}+\frac{59}{11}}{11}$$

$$= 6.3\ \%\ \text{cracking/yr/yr}$$

E. <u>Total Transverse Cracking (TCTotal)</u>: this measure is best described in Figure 3. The value is the sum of the area under the percent cracking versus years in service curve (total cracking performance) divided by the total years in service. While the other measures result in percent cracking per year, this measure quantifies the total amount of cracking a roadway experiences. For the values in Figure 3, 28.8 percent is the total transverse cracking amount.

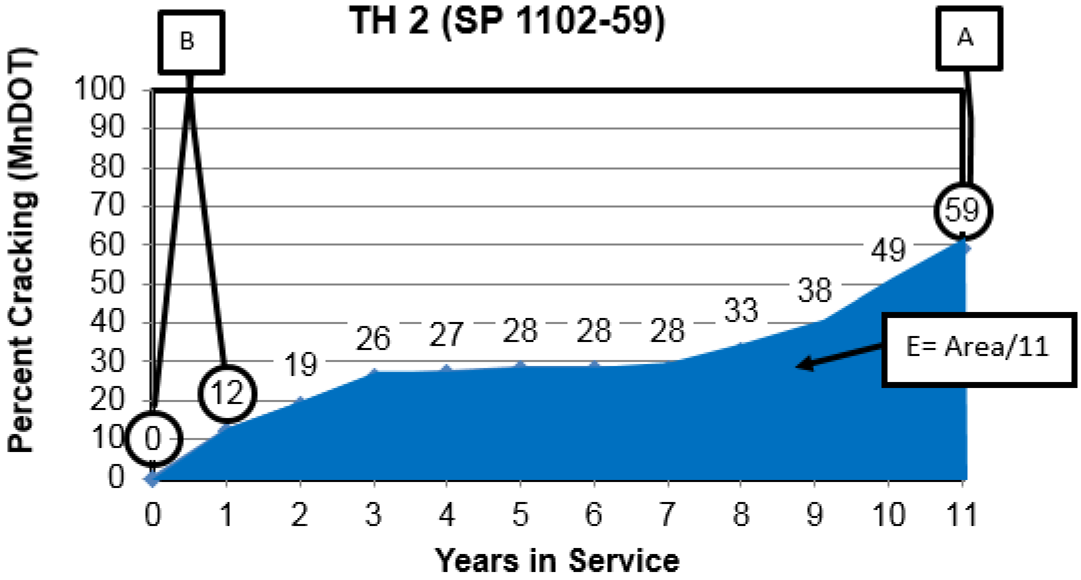

Figure 3: Example of different cracking measures

The amounts of transverse cracking with respect to time for each of the roadways in this study are shown in Figure 4 to Figure 21. Note that all percent cracking measures defined on the y-axis in these figures are designated “(MnDOT)”. This is to identify the use of MnDOT PMS data in the cracking amounts.

The mix parameters of interest for each section in the study can be found in Table 3. Given the limitations of this study, the various roadway sections provide a fair amount of variability for each parameter. This variability provides a sufficient amount of data for determining any preliminary relationships between the individual values and fracture energy. Refer to the following discussions for detailed descriptions on the effect of each parameter.

Table 3: Summary of asphalt mixture parameters by section

| Section | SP # | RP / Landmark | PG Grade | PG Spread | Asphalt Content | Recycled Asphalt Content | Voids in Mineral Aggregate (VMA) | Voids Filled with Asphalt (VFA) |
|---|---|---|---|---|---|---|---|---|
| TH 1 | 8821-103 | RP 235 | 58-34 | 92 | 4.7% | 17.0% | 14.9% | 73.1% |
| TH 1 | 8821-103 | RP 230 | 58-28 | 86 | 4.7% | 17.0% | 14.9% | 73.1% |
| TH 2 | 1102-59 | RP 157 | 58-34 | 92 | 4.6% | 26.1% | 14.0% | 71.4% |
| TH 6 | 1103-25 | RP 53 | 58-28 | 86 | 4.4% | 36.4% | 13.9% | 71.2% |
| TH 6 | 3107-42 | RP 118 | 58-34 | 92 | 5.3% | 17.0% | 14.8% | 73.0% |
| TH 6 | 3107-42 | RP 123 | 58-34 | 92 | 5.3% | 17.0% | 14.8% | 73.0% |
| TH 9 | 6010-26 | RP 208 | 58-34 | 92 | 4.2% | 26.2% | 13.1% | 69.6% |
| TH 9 | 6010-26 | RP 214 | 58-34 | 92 | 4.2% | 26.2% | 13.1% | 69.6% |
| CSAH 10 | 031-610-016 | Jct 445B | 58-28 | 86 | 4.3% | 23.3% | 13.5% | 70.4% |
| CSAH 10 | 031-610-016 | Jct 446 | 58-28 | 86 | 4.3% | 23.3% | 13.5% | 70.4% |
| TH 10 | 0502-95 | RP 159 | 64-28 | 92 | 5.3% | 45.3% | 14.4% | 72.3% |
| TH 10 | 0502-95 | RP 161 | 64-28 | 92 | 5.3% | 45.3% | 14.4% | 72.3% |
| TH 10 | 5606-42 | RP 75 | 58-28 | 86 | 4.3% | 23.3% | 13.7% | 70.8% |
| TH 25 | 7104-19 | Jct 17 | 64-34 | 98 | 4.6% | 17.4% | 13.2% | 69.7% |
| TH 27 | 4803-19 | RP 171 | 58-28 | 86 | 4.3% | 37.2% | 13.6% | 70.6% |
| TH 27 | 4803-19 | RP 174 | 58-28 | 86 | 4.3% | 37.2% | 13.6% | 70.6% |
| TH 28 | 6104-11 | RP 81 | 58-34 | 92 | 4.2% | 23.8% | 12.5% | 68.1% |
| TH 28 | 6104-11 | RP 88 | 58-34 | 92 | 4.2% | 23.8% | 12.5% | 68.1% |
| CSAH 30 | 1306-44 | Jct TH 95 | 64-34 | 98 | 4.4% | 11.4% | 13.4% | 70.2% |
| I-35 | 0283-26 | N/A | 64-28 | 92 | 5.0% | 34.0% | 15.1% | 73.5% |
| TH 53 | 8821-177 | Jct 169 | 58-28 | 86 | 4.7% | 29.8% | 17.6% | 77.2% |
| TH 113 | 4407-12 | RP 10 | 58-28 | 86 | 4.5% | 20.0% | 12.6% | 68.3% |
| TH 113 | 5413-10 | RP 5 | 58-34 | 92 | 4.5% | 20.0% | 12.6% | 68.3% |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| TH 210 | 1805-72 | RP 118 | 58-28 | 86 | 4.4% | 38.6% | 13.5% | 70.4% |
| TH 212 | 1017-12 | RP 147 | 70-34 | 104 | 6.4% | 0.0% | 19.2% | 79.2% |
| TH 220 | 6016-37 | RP 12 | 58-28 | 86 | 4.2% | 23.8% | 13.5% | 70.3% |

### *2.5.2 Individual Roadway Cracking Performance*

Trunk Highway 1 contained two pavement sections within the study domain (Figure 4). The section that is referenced as RP 230 was constructed with a 4” overlay on reclaimed asphalt while the RP 235 had a 1.5” overlay placed onto the old asphalt. The trend in the plot indicates that placing an overlay onto reclaimed asphalt showed a lower amount of cracking for a longer period of time.

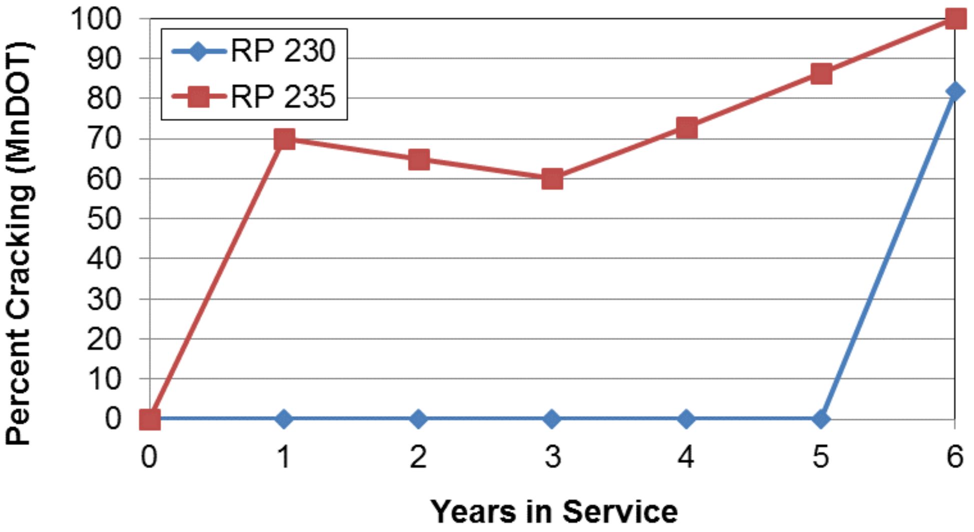

Figure 4: Cracking Performance of TH 1 (SP 8821-103)

The cracking performance of Trunk Highway 2 showed a gradual decline for eleven years. As the plot indicates, the deterioration of the roadway has been consistent over the last four years (Figure 5).

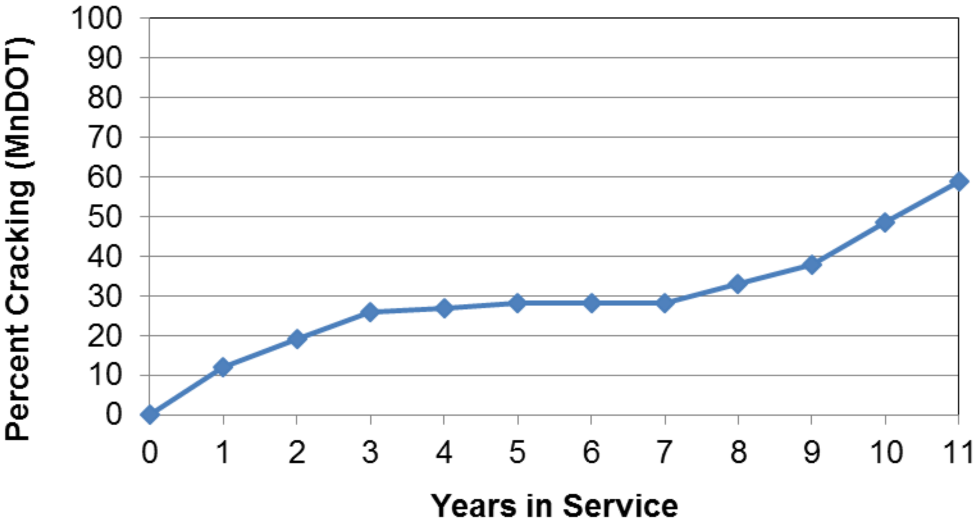

Figure 5: Cracking Performance of TH 2 (SP 1102-59)

The project on Trunk Highway 6 (SP 1103-25) has been in service for four years. During the first year of service, the roadway deteriorated to nearly 20% transverse cracking (Figure 6). Since that time, the cracking rate has tapered off slightly. While the roadway is still experiencing annual increases in transverse cracking amounts, there has not been an overly drastic increase between two years.

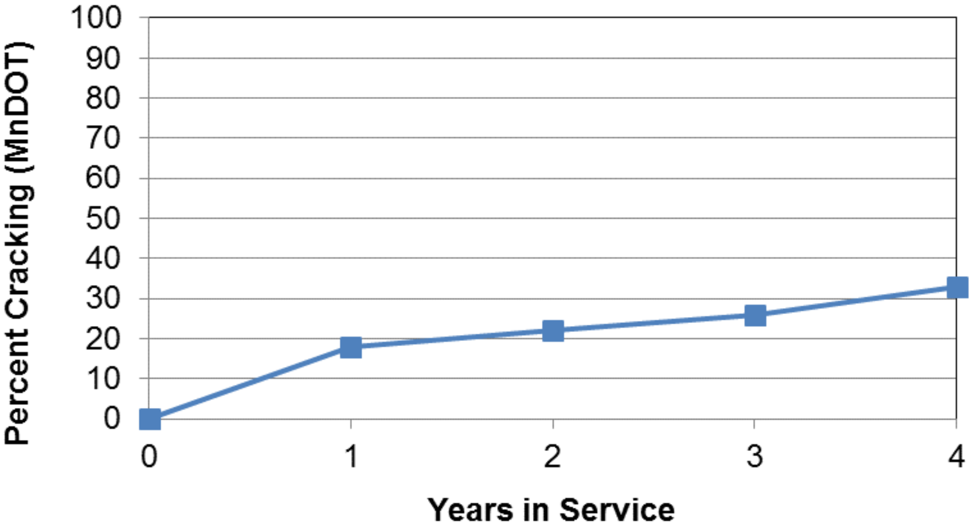

Figure 6: Cracking performance of TH 6 (SP 1103-25)

Trunk Highway 6 (SP 3107-42) had two pavement sections associated with it. There is a noticeable difference in the performance of the two pavements (Figure 7). The section that started at RP 118 showed a large variation in cracking amounts, this may be due to the time of year or extreme temperatures when the automated crack counts were performed. This reinforces the need for site visits on periodic basis and to implement some form of consistency check for automated data collection. RP 118 was constructed with a 1.5" overlay placed on the existing asphalt. The portion referred to as RP 123, the better performer, was constructed with a 4.5" overlay on reclaimed asphalt.

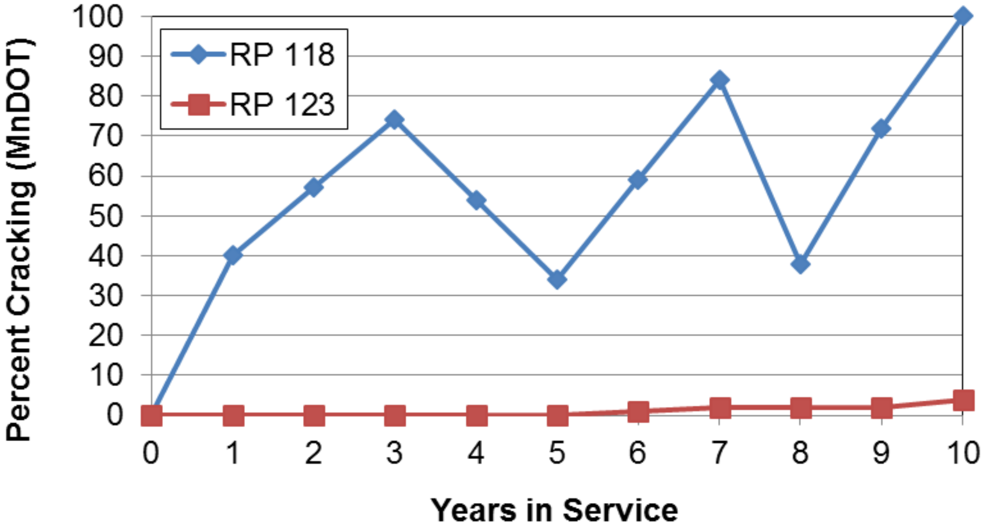

Figure 7: Cracking Performance of TH 6 (SP 3107-42)

Trunk Highway 9 (SP 6010-26) had two study sections. Both of these sections were constructed as 3” overlays on reclaimed asphalt. As can be seen in Figure 8, the section at RP 214 has performed slightly better than the section at RP 208. The main purpose a section was considered poor performing (RP 208) was due to ride quality. Overall, both sections are performing well.

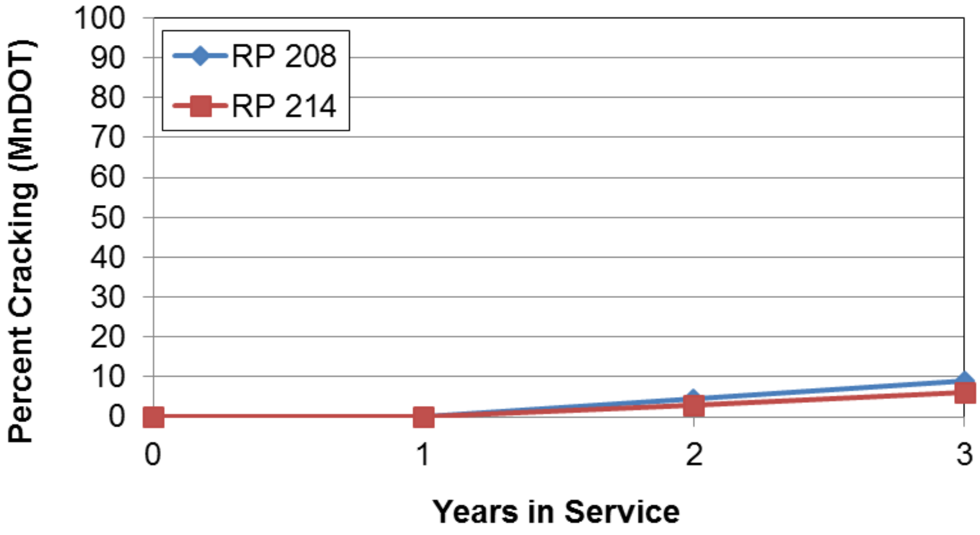

Figure 8: Cracking performance of TH 9 (SP 6010-26)

County State Aid Highway (CSAH) 10 (SAP 031-610-016) has both a poor performing (JCT 445B) and good performing (JCT 446) sections. The performance of each can be seen in Figure 9. The section at JCT 446 is a 3” mill and overlay, while the JCT 445B section is a 1.5” overlay on old asphalt. The service life of two years is short, but the drastic difference between the two sections is apparent.

Figure 9: Cracking performance of CSAH 10 (SAP 031-610-016)

The study area on Trunk Highway 10, a divided four lane highway, contained two different pavement sections. The cracking amounts are separated into driving lane and passing lane data (Figure 10). Both sections, RP 159 and RP 161, were constructed using a 4" mill and overlay. The cracks in the section beginning at RP 159 were sealed at the time of site visit where, as for the section beginning at RP 161 the cracked were not sealed.

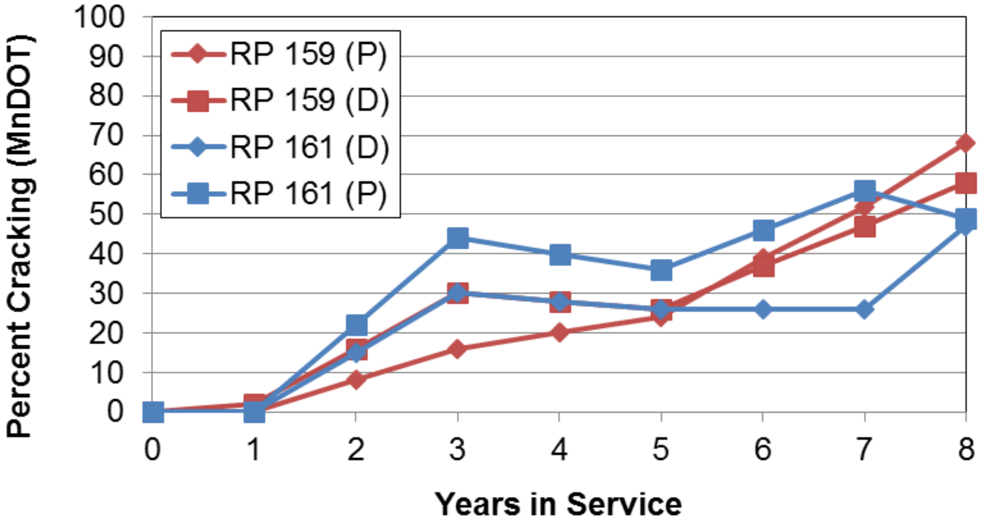

Figure 10: Cracking Performance of TH 10 (SP 0502-95)

Trunk Highway 10 (SP 5606-42) consists of one section and two lanes. The project is a 3.5” mill and overlay. Over the first year of service, this roadway experienced a substantial deterioration (Figure 11). The reason for this is unclear, as most of the mill and overlay sections in this research feature good initial resistance to transverse cracking. The analysis of this project should provide clarity for the extreme cracking experienced by this section.

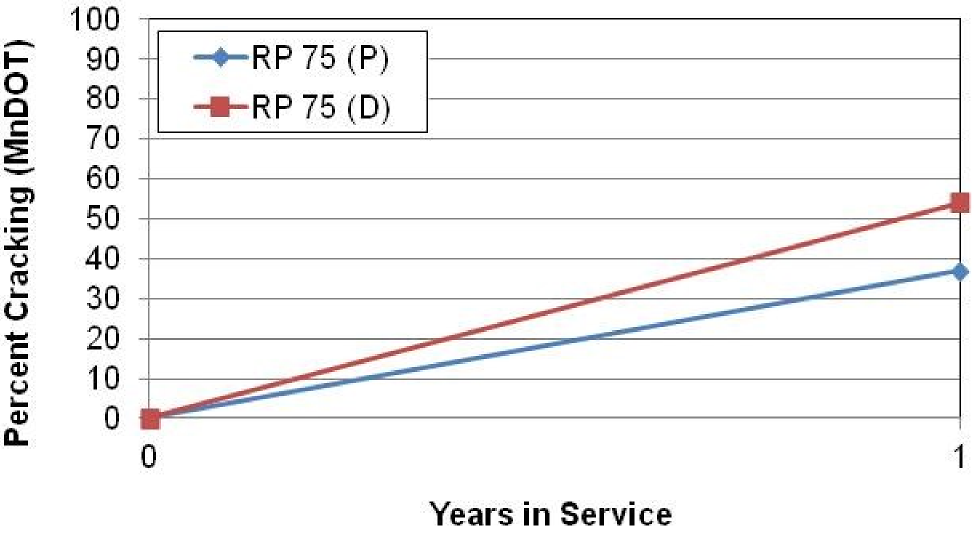

Figure 11: Cracking performance of TH 10 (SP 5606-42)

Trunk Highway 25 (SP 7104-19) is a new bituminous on aggregate base construction project. It has exhibited no thermal cracking over the three year service life (Figure 12). The mix was very open and dry when observed. Monitoring the progress of this roadway in the future will be beneficial to determine if this open mixture can withstand moisture related distresses.

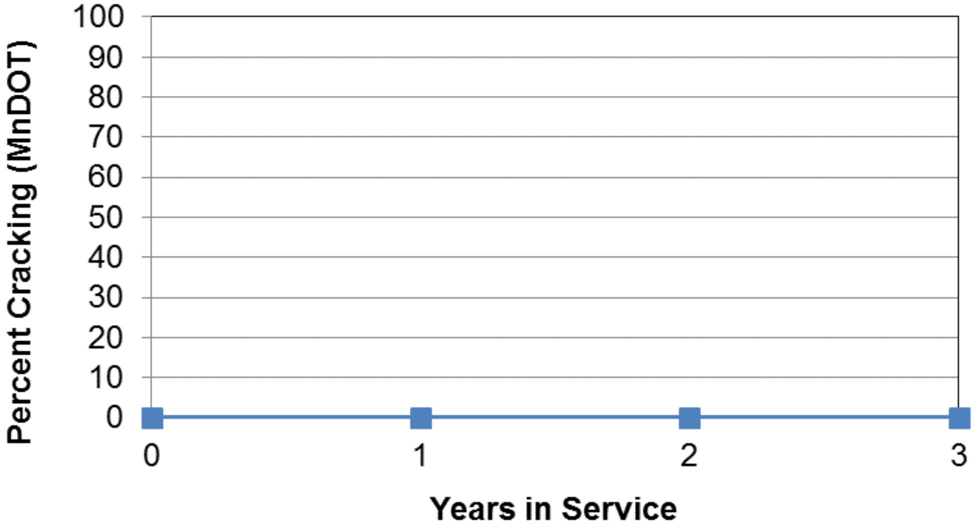

Figure 12: Cracking performance of TH 25 (SP 7104-19)

Trunk Highway 27 (SP 4803-19) currently has a four year service life. Two sections were observed for this project. RP 171 and RP 174 are both 3” mill and overlay construction. The sections feature similar cracking amounts, with both currently exhibiting roughly 35% transverse cracking (Figure 13).

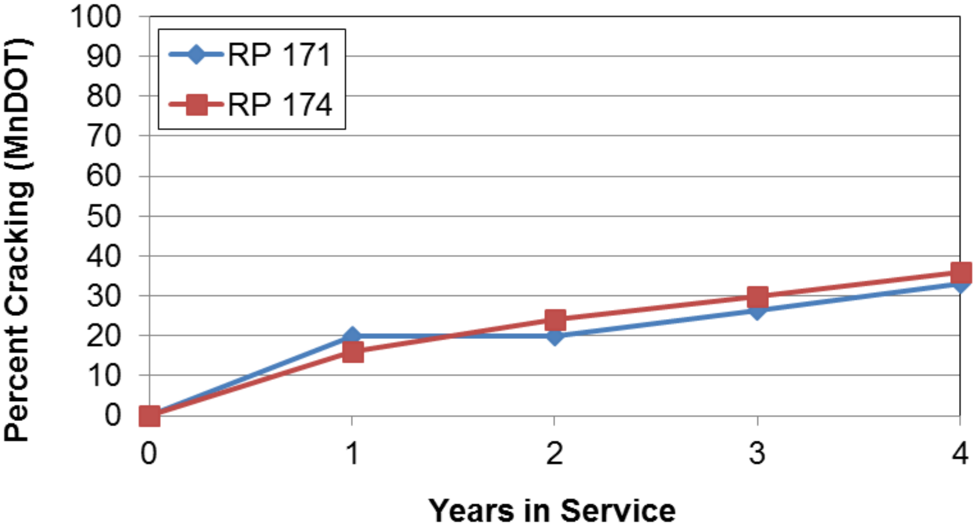

Figure 13: Cracking performance of TH 27 (SP 4803-19)

Trunk Highway 28 (SP 6104-11) performance can be found in Figure 14. Two sections of the same 4.5” mill and overlay construction were observed. Similar to previous sections of same construction types, both study corridors are performing nearly identical. The current transverse cracking levels are at approximately 30% over a two year service life. This is a fairly substantial increase over that time period, especially considering the majority of this deterioration occurred over the second year of the service life.

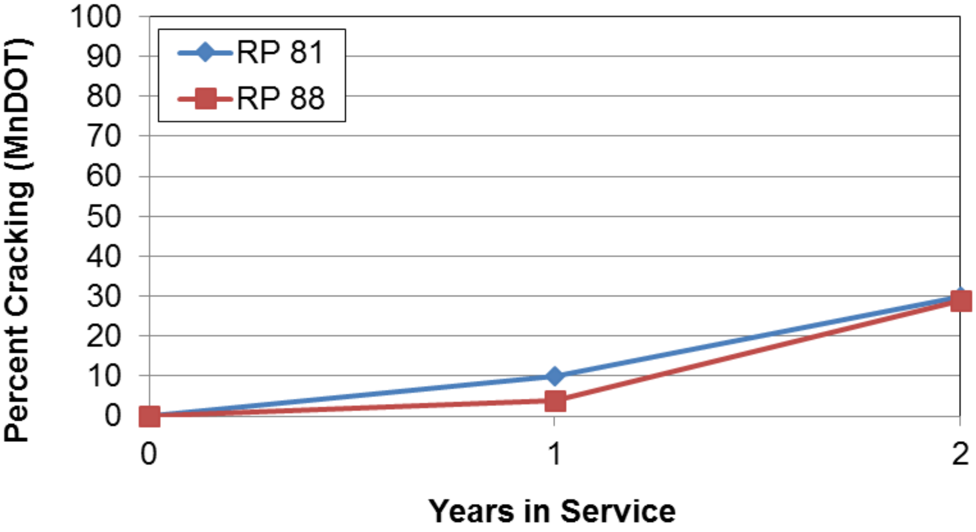

Figure 14: Cracking performance of TH 28 (SP 6104-11)

The service life performance of County State Aid Highway 30 (SP 1306-44) can be seen in Figure 15. Still early in the service life, the roadway has seen a gradual increase in cracking performance since the construction year. Future observation of this roadway should monitor if this gradual trend is maintained.

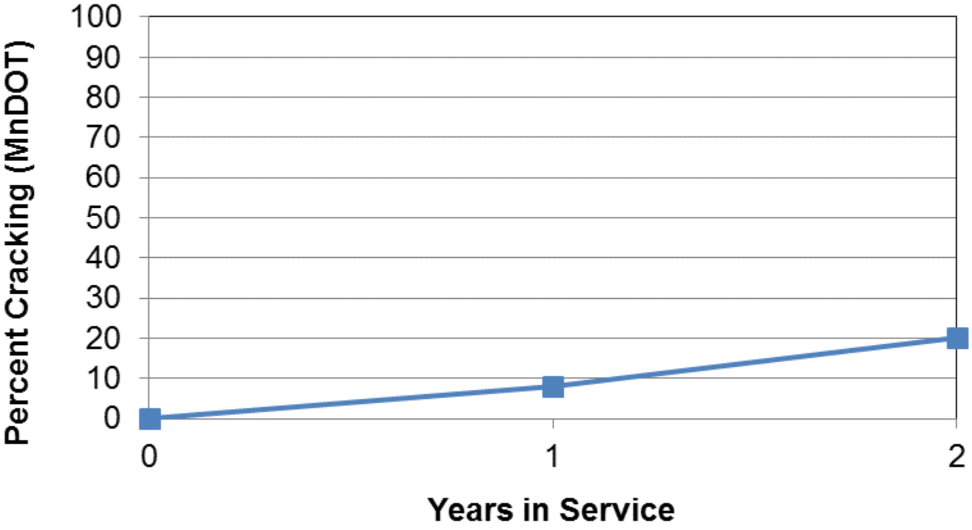

Figure 15: Cracking performance of CSAH 30 (SP 1306-44)

Interstate 35 (SP 0283-26) has shown a consistent amount of cracking over the four year service life. Consisting of a 4” mill and overlay on existing concrete, the roadway has typically served with little transverse cracking present (Figure 16). The decrease from year three to year four can be attributed to the automatic crack count procedure, along with a patching rehabilitation effort made at that time.

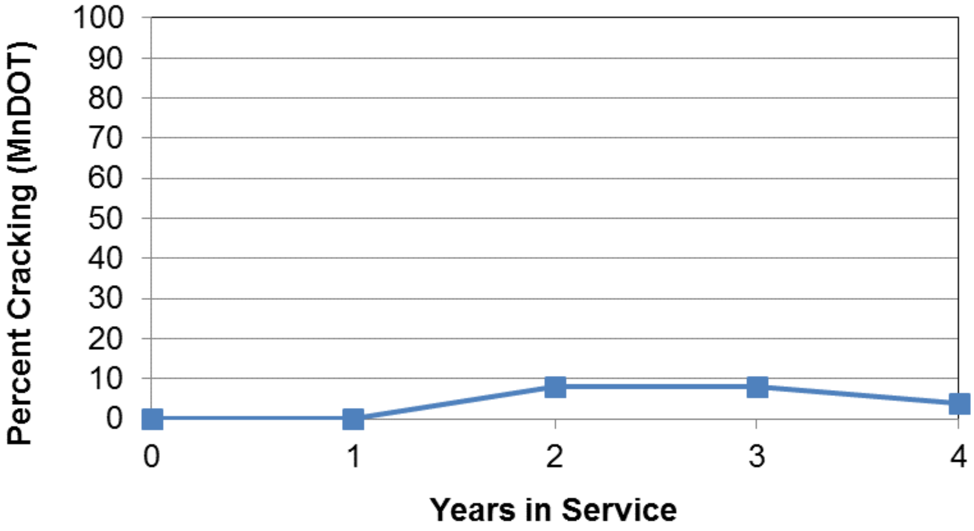

Figure 16: Cracking performance of I-35 (SP 0283-26)

Trunk Highway 53 was rehabilitated using a 1.5” mill and overlay. The cracking amounts vary greatly over time and also show a trend of increasing and decreasing (Figure 17), this is also an inconsistency that most likely resulted from the automated crack counting system. Most of the cracking that was observed during the visual survey appeared to be reflective cracking. As with other sections featuring asphalt overlay on PCC pavement, the majority of reflective cracking occurred during year one of service.

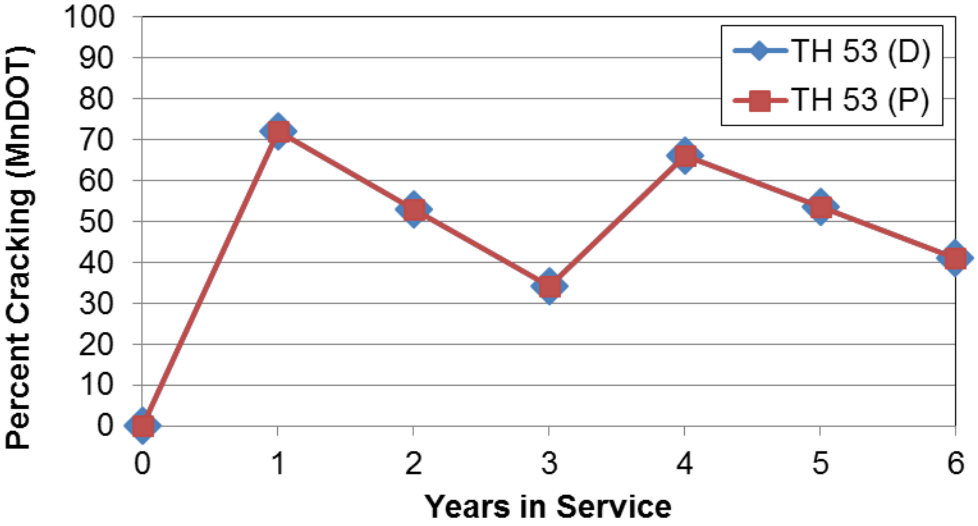

Figure 17: Cracking Performance of TH 53 (SP 8821-177)

The study area of Trunk Highway 113 also contained two differently constructed sections. RP 10 had a 1.5” overlay on existing pavement and RP 5 has a 5” overlay on reclaimed asphalt. Once again as with previous sections, the overlay on reclaimed asphalt performed better than overlays on existing pavement. The plot shows that RP 10 started to crack in year one (Figure 18).

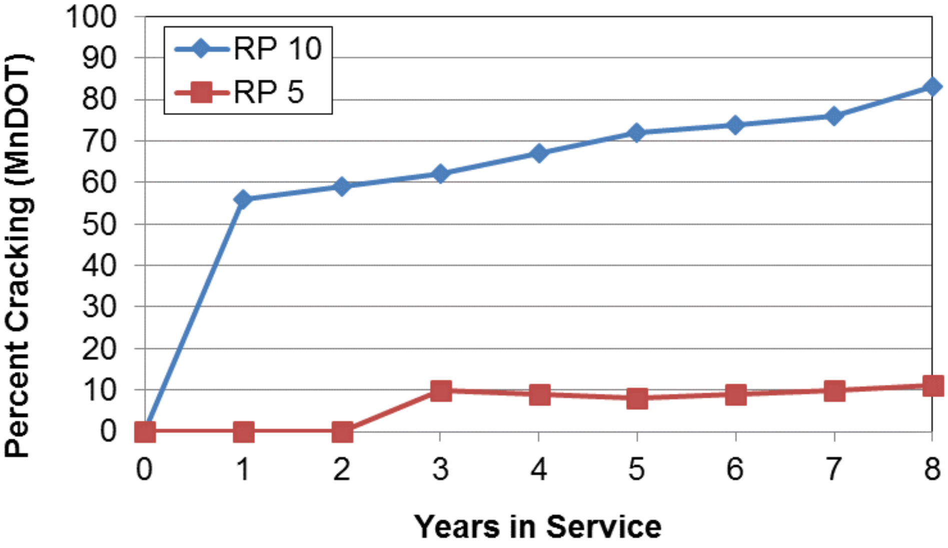

Figure 18: Cracking Performance of TH 113 (SP 4407-12)

A 2” overlay was placed on the existing portland cement concrete pavement of Trunk Highway 210. The amount of cracking, shown in Figure 19, suggests that the transverse cracking is reflected from the underlying concrete. With approximately 30 foot joint spacing of PCC pavement, the amount of transverse cracking comes to approximately 33%. This is another indicator of the reflective cracking since the cracking amount shows little variation with time. Furthermore, the reflective cracks developed within 1 year of service.

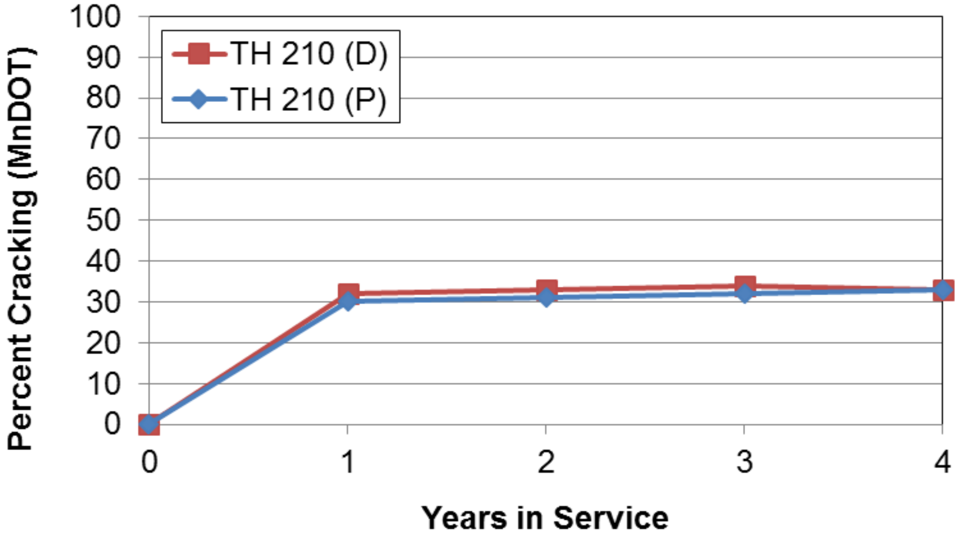

Figure 19: Cracking Performance of TH 210 (SP 1805-72)

Trunk Highway 212 is one of only two pavement sections in the study group that is new bituminous on an aggregate base construction (the other being Trunk Highway 25). Due to high traffic volume, this section was not conducive to a walking visual survey. The data presented in Figure 20 is based on data from the automated crack counts from MnDOT. The section is constructed using SMA mixture and has shown excellent performance.

Figure 20: Cracking Performance of TH 212 (SP 1017-12)

Trunk Highway 220 (SP 6016-37) is a 3" mill and overlay project. As seen in Figure 21, a small amount of transverse cracking has occurred on this roadway, with all of the deterioration occurring after the first year of service. No substantial cracking has occurred on this roadway thus far in the two year service life.

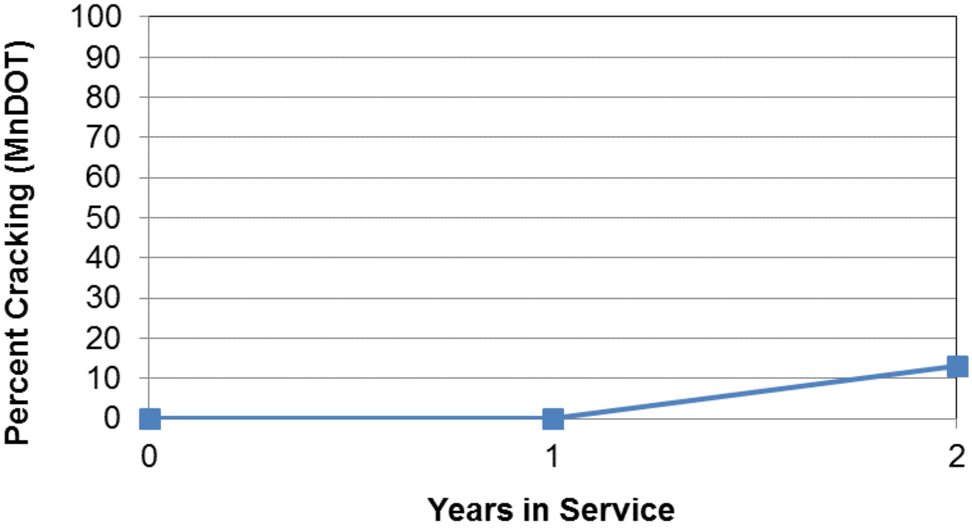

Figure 21: Cracking performance of TH 220 (SP 6016-37)

### *2.5.3 Transverse Cracking Performance of All Study Sections*

The transverse cracking performance of all pavement sections studied in this project is presented here. The performances are presented using the transverse cracking measures described in Chapter 2. Please note that only the cracking performance data is presented herein, the analysis of data is presented Chapter 4.

The maximum transverse cracking (MTCTotal) of each roadway per year is shown in Figure 22. As shown in the plot the worst performing section (TH 10: RP 75) shows approximately 45% cracking per year of service, which translates into 100% cracking within three years of service. Of the pavement sections that were visited, both TH 25 and TH 212 demonstrate the best performance, with 0% cracking per year of service. It should be noted that these are the only two new construction projects in this research.

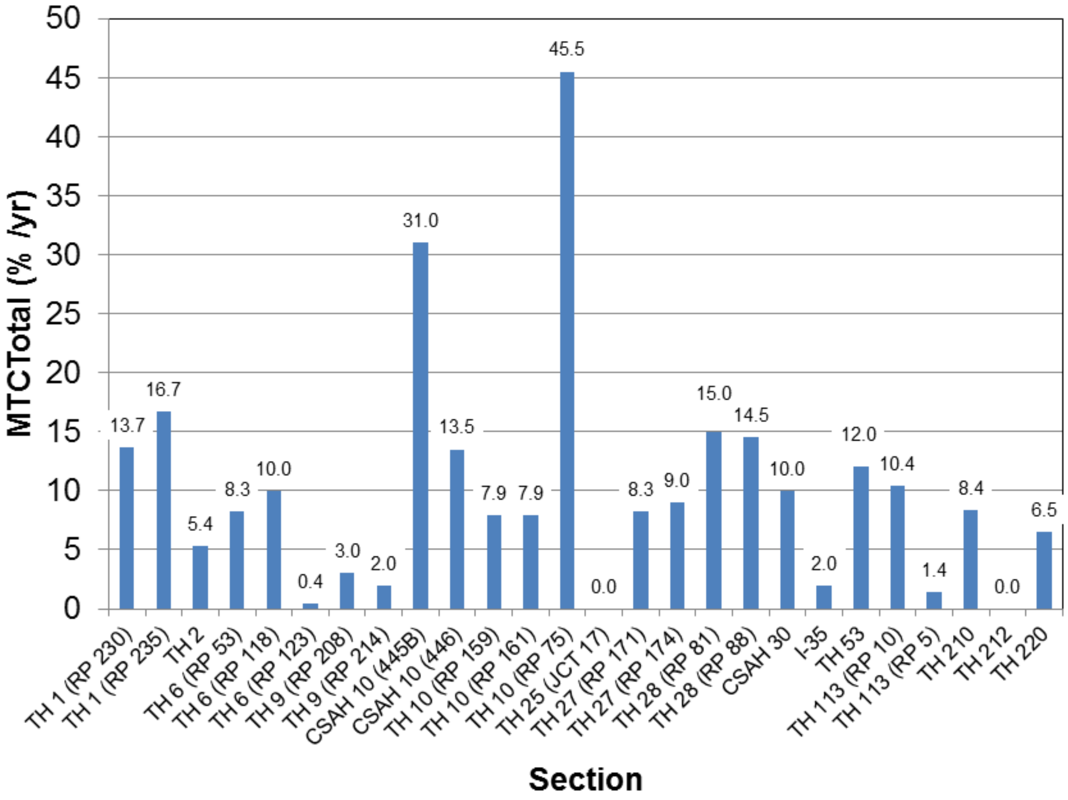

Figure 22: Maximum transverse cracking (MTCTotal) for all study sections

The maximum transverse cracking rate (MTCRTotal) data is presented in Figure 23. Once again this measure represents the maximum cracking increase that a pavement section experienced between two consecutive crack counts. Three of the study sections (TH 1 and TH 53 which are all located in Northern/North Eastern Minnesota) experienced relatively high cracking rates (between 70 and 82% cracking within a year). While more details are presented in the analysis of this data, generally the overlay sections showed higher cracking rates early in service. Alternatively, reclaim sections showed a higher cracking rate later in service.

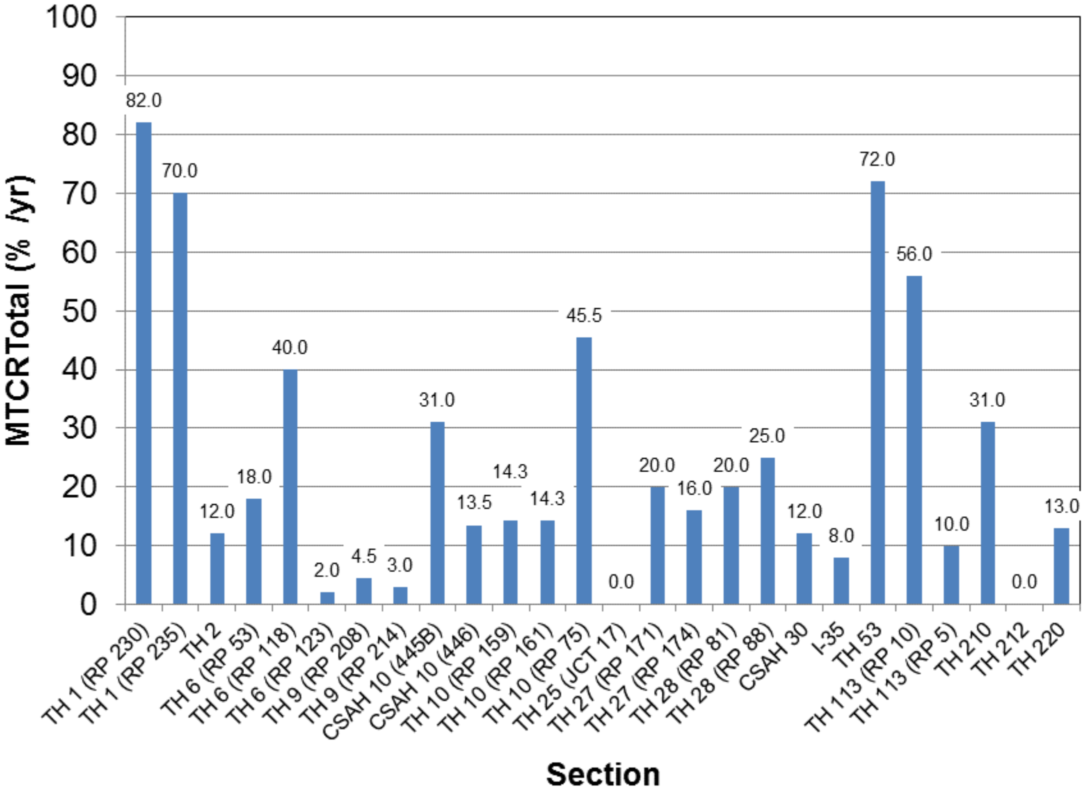

Figure 23: Maximum transverse cracking rates (MTCRTotal) for all study sections

The average transverse cracking (ATCTotal) information for all sections is presented in Figure 24. This measure differs from the previous measures in the sense that it accounts for cracking performance of the pavement section for each service year. Thus, this measure provides credit to pavements that have performed well for several years before cracking over a comparable section that displayed cracking within the first few years of service. The previous measures only focus on the maximum cracking amounts from all available data or maximum rate of cracking. With this measure the TH1 RP 230 and RP 235 sections show significantly different performance as the RP 235 experienced cracking early in the service life where as RP 230 experienced cracking later in the life.

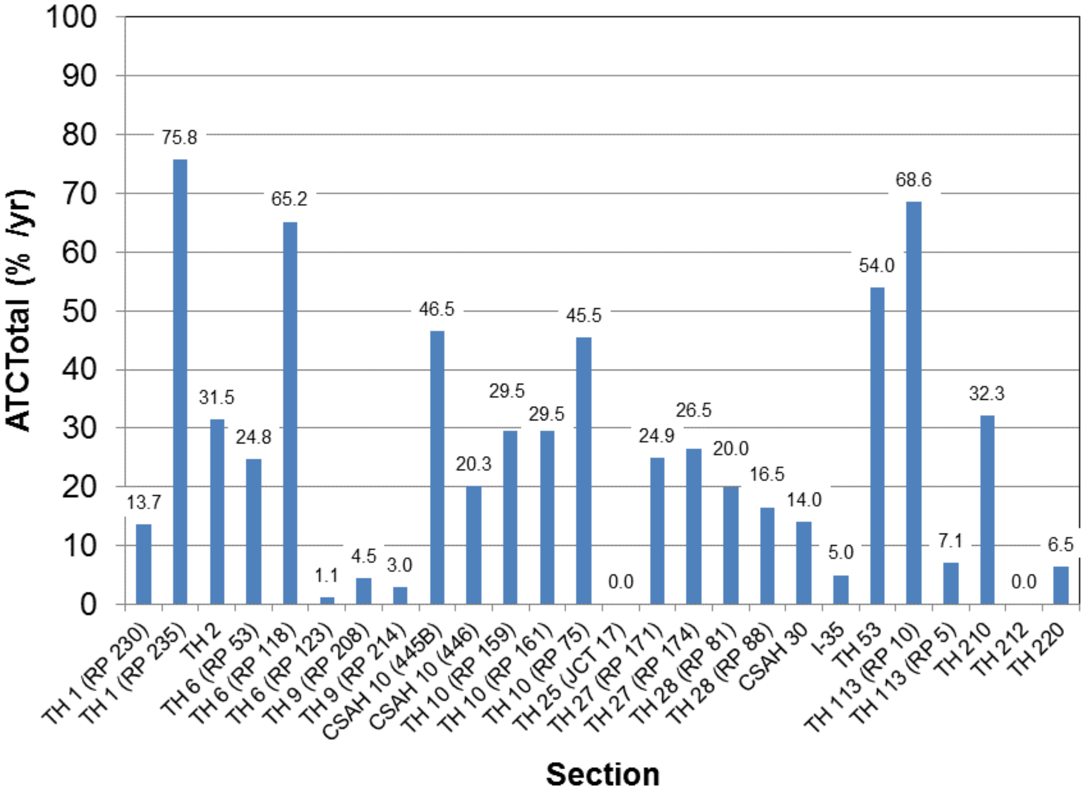

Figure 24: Average transverse cracking (ATCTotal) for all study sections

The weighted average transverse cracking (WATCTotal) data for all sections is presented in Figure 25. This measure attempts to further quantify the historical performance of a roadway. It is similar to ATCTotal, as it factors in the transverse cracking amount for each year, but applies a weight factor to all years. WATCTotal reflects cracking late in the service life positively as compared to early cracking. Similar relationships are shown to ATCTotal; this measure was developed to identify if trends are maintained during comparison from one measure to the other.

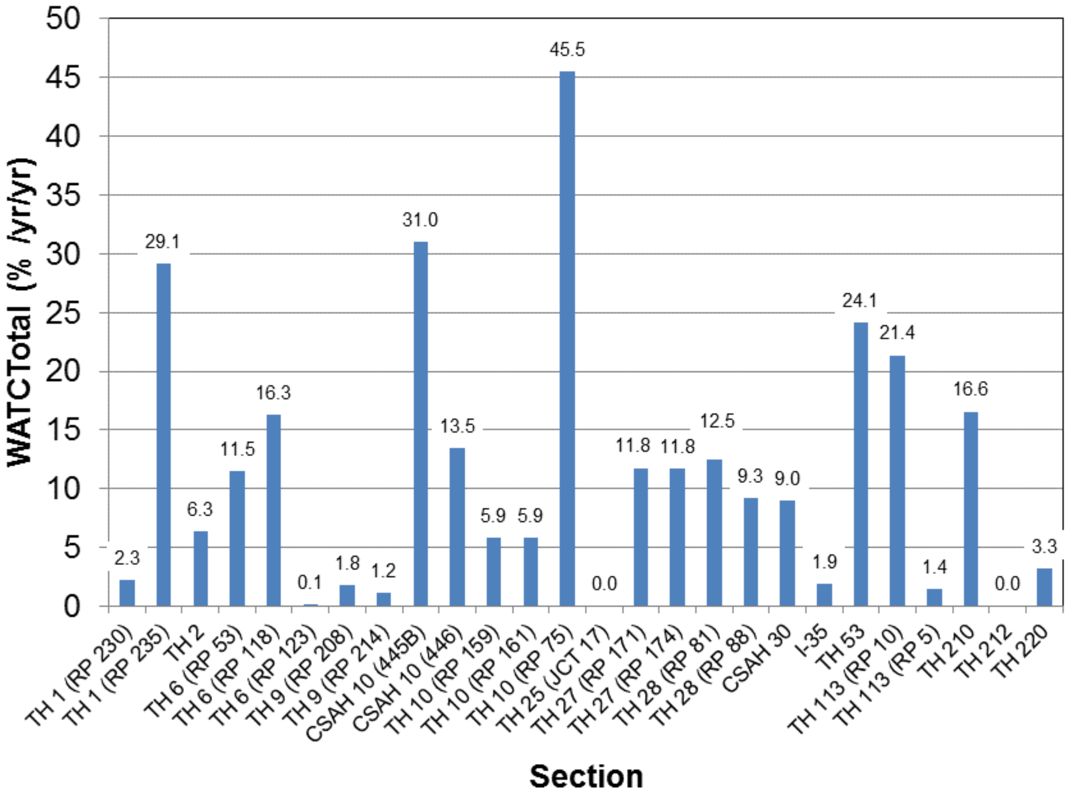

Figure 25: Weighted average transverse cracking (WATCTotal) for all study sections

Total transverse cracking (TCTotal) for all sections can be found in Figure 26. While previous measures are calculated using only the annual cracking amounts, TCTotal evaluates the sum of the area under the percent cracking versus years in service curve (total cracking performance). This is conducted to essentially quantify all the cracking experienced by a roadway. Again, the different measures are used to determine if a different perspective of cracking disproves any preliminary conclusions made during the analysis process.

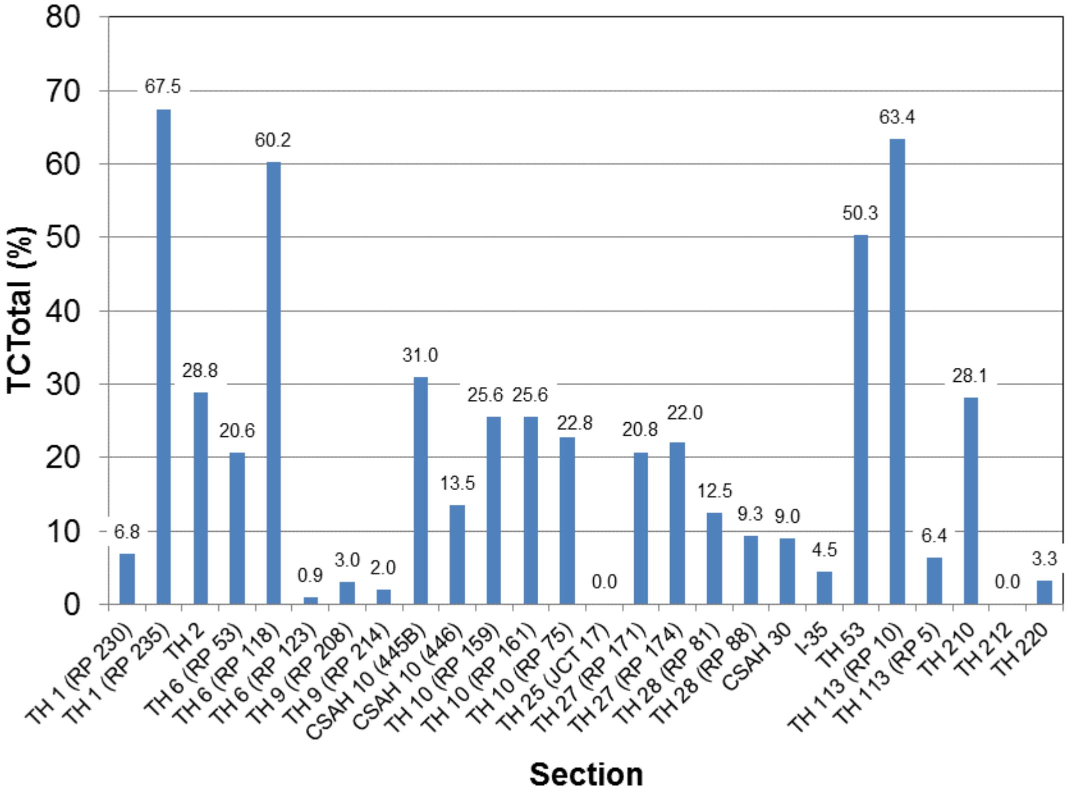

Figure 26: Total transverse cracking (TCTotal) for all study sections

## 2.6 Summary of Findings from Field Performance Analysis

The initial phase of this research focused on the field evaluation of eighteen highways over 26 analysis sections. During this task the highway sections were visited and using a uniform site visit format, a number of pavement study sections were identified. The pavement study sections were evaluated to conduct crack counts as well as a visual distress survey. The data collected during the site visits is summarized throughout Chapter 2. The raw crack count data is compiled and stored for use during analysis. Furthermore, the locations for obtaining cored samples for performance testing were also identified. Using the sample collection information and on the basis of the construction drawings, field sampling plans were developed and delivered to MnDOT staff.

Two cracking performance measures were developed through earlier research (MTCTotal and MTCRTotal) [18]. The three other performance measures (ATCTotal, WATCTotal and TCTotal) were developed during the analysis process of these studies. Those five performance measures were utilized in conjunction with the pavement management data and information from field visits to quantify the cracking performance of pavement sections. The information collected and processed through this task is being utilized to make comparisons between field cracking performance and asphalt mix attributes as well as disk-shaped compact tension (DCT) fracture energy measurements in Chapter 3.

While detailed analysis of the data is being conducted and described in Chapters 3 and 4, some general observations from the cracking performance and sites visits are as follows:

- The average of the maximum cracking amount (MTCTotal) of all 26 study sections is approximately 10.1% per year of service. This information can be used to determine the number of years of service at which the pavement is expected to reach the state of 100% transverse cracking. On an averaged basis, using data from 26 pavement sections studied herein, approximately 10 years of service to reach 100% transverse cracking is obtained. The shortest life as seen from the study sections is expected to be three years.

- For the sections studied in this project, the maximum cracking rate (MTCRTotal) is observed to be as high as 82% per year with an average of 24.4% per year.
- The average of the average transverse cracking amounts (ATCTotal) for all 26 sections is approximately 25.6%. This measure indicates the average amount of cracking that would be present on any section during the course of its service life. Related to this, the average of the weighted average transverse cracking amounts (WATCTotal) is 11.3%. This is essentially providing the same information as the ATCTotal. The difference in the values is due to the weight applied during the calculation of WATCTotal.
- Total transverse cracking (TCTotal) for all 26 sections results in an average of 20.2%. This indicates the actual amount of transverse cracking a section has undergone over the service life relative to the potential cracking amount.
- The asphalt layers on reclaimed sections show lower amounts of cracking and delayed cracking as compared to mill and overlay sections on the same stretches of highways. It should be noted though that the reclaim sections consist of greater asphalt layer thicknesses (3" – 4") as compared to mill and overlay sections (1.5" – 2.5").
- The pavement sections consisting of asphalt overlay on PCC pavements showed significant reflective cracking within the first year of service. Once all joints/cracks have reflected into the overlay, minimal additional cracking was observed.

# CHAPTER 3: LABORATORY EVALUATION

The second phase of this research project involved laboratory testing of samples from the study sections. During the course of this task, field samples were tested using the disk-shaped compact tension (DCT) test. Chapter 3 provides a detailed description of the DCT test and how to interpret the results. Laboratory performance test results for the study sections will also be provided. Due to circumstances outside the control of the research team, some sections were not available for DCT testing. These specific sections will be identified in the DCT results.

## 3.1 Overview of the Disk-Shaped Compact Tension (DCT) Test

### *3.1.1 DCT Test Description*

The DCT test is standardized by ASTM D7313-13 [20]. The primary function of the test is to quantify the resistance an asphalt mixture will have to low temperature cracking. Low temperature cracking is the primary pavement distress in climates that experience extreme low temperatures and/or high rates of temperature drop. The discrete cracking of a material, as in the case of low temperature cracking, is a highly complicated phenomenon, and evaluation of the material beyond the linear response range helps close the gap between experimental results and actual field performance. All sections in this study, along with the majority of the State of Minnesota, undergo extensive low temperature climatic conditions. This study uses the DCT test on field cored samples to determine if any trends are found for use in future research projects.

Specimens for the DCT test can come from gyratory compacted pills or field cores. In the case of this study, all specimens came from field cored samples. Sample preparation involves sawing the pills or cores into 50 mm thick disks. Generally, both faces (top and bottom) of the disk are saw cut. The flat face, 25 mm diameter loading holes, and notch are then cut. See Figure 27 for a schematic of the typical DCT specimen and an actual prepared sample.

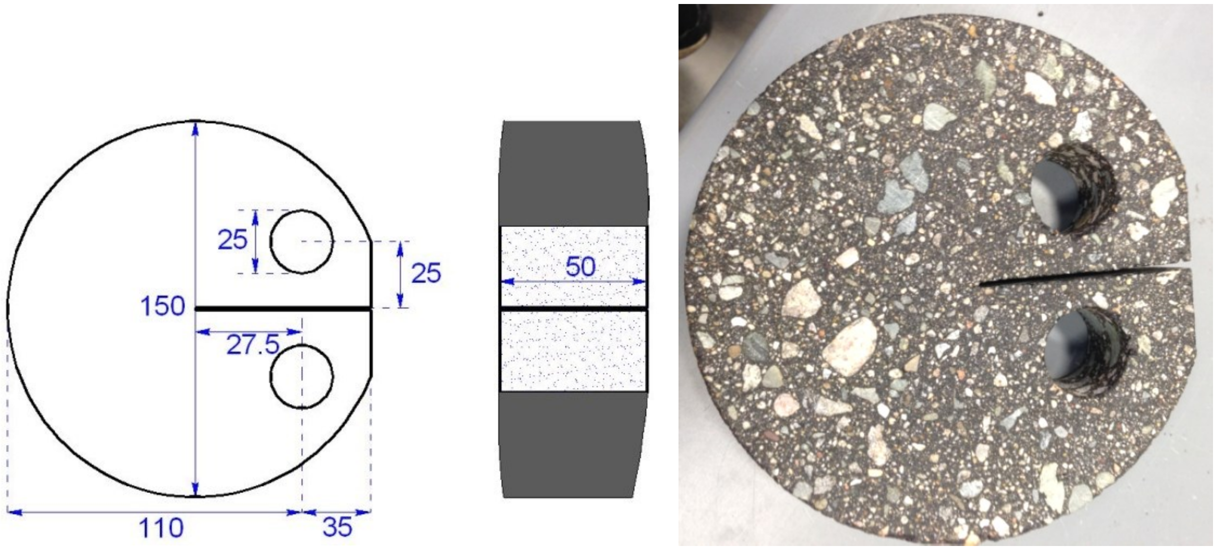

Figure 27: (a) Disk-shaped compact tension specimen geometry (dimensions in mm);
(b) prepared DCT specimen

Prior to testing, gage points are first applied above and below the notch on the flat surface. These act as anchor points for the measuring device. This device is referred to as the crack mouth opening displacement (CMOD) gage. Once each specimen has been cut and gage points are attached, each specimen is carefully measured. The measurements (recorded by hand using a caliper) are averaged from several areas on the specimen to account for any variance. The thickness is recorded at quarter points around the perimeter of the sample, and the ligament length (length between the inside of the notch and exterior edge of the sample) is recorded on both sides of the specimen. Both the thickness and ligament length are vitally important to the accuracy of the results (see Section 3.1.2). The averages of these results allow for the calculation of the sample area, the relevance of which will be explained later in this document. After completion of preparation and measurement, specimens must undergo temperature conditioning. DCT results are highly dependent on the temperature of the chamber. The ASTM specification for the DCT test (D7313-13) recommends conducting testing at a temperature 10°C greater than the low temperature Superpave performance grade of the binder in the asphalt mixture. While this may be applicable for quantifying the general resistance of a mixture to low temperature cracking, the low temperature performance grade is not always indicative of the environmental temperature to which a mixture is exposed.

For this study, specimens were loaded into the testing chamber at a temperature 10°C greater than the 98% reliability environmental low temperature using Superpave specifications. For example, instead of testing a PG XX-34 at -24°C, temperature data shows (with 98% reliability) that this roadway will only experience -31°C. Therefore, DCT test conditioning for the corresponding specimens will target -21°C. This eliminates the unnecessary “penalization” for a binder in this scenario, as it will likely never see the extreme temperature recommended by the ASTM standard. Alternatively, a PG XX-28 binder tested at -18°C will not provide accurate DCT results for an environment experiencing temperatures colder than -28°C. Location is a primary function of this study. This required the research team to provide site-specific temperature conditioning data. In order to achieve this, historical temperature data was required to accurately predict this 98% reliability. LTPPBind was utilized to determine these values based on the specific location of each section.

Once the temperature has been determined and DCT specimens have been placed in the testing chamber, the DCT testing procedure can begin. The temperature conditioning process is the first step. In an effort to accurately model in-service conditions, the test temperature is achieved by ramping down the internal temperature of the chamber over a period of two hours. This is performed to avoid “shocking” the sample, as it is highly unlikely a roadway environmental temperature would drop from room temperature (≈20°C) to desired test temperature instantaneously. At the completion of the two hour ramping period, specimens are “soaked” at the target temperature for a minimum of two hours prior to the beginning of testing. Additional investigations on the impact of "shocking" samples and the accuracy of the soaking period are ongoing.

A specimen is then mounted onto the testing apparatus (Figure 28). As can be seen in Figure 28, pins are inserted into the two 25 mm loading holes. The pins facilitate the application of load via the loading clevis. The CMOD gage, as mentioned earlier, is clipped onto the gage points attached to the specimen (see Figure 29). The chamber is then allowed to cool back to the target temperature. At this time, the DCT testing can begin.

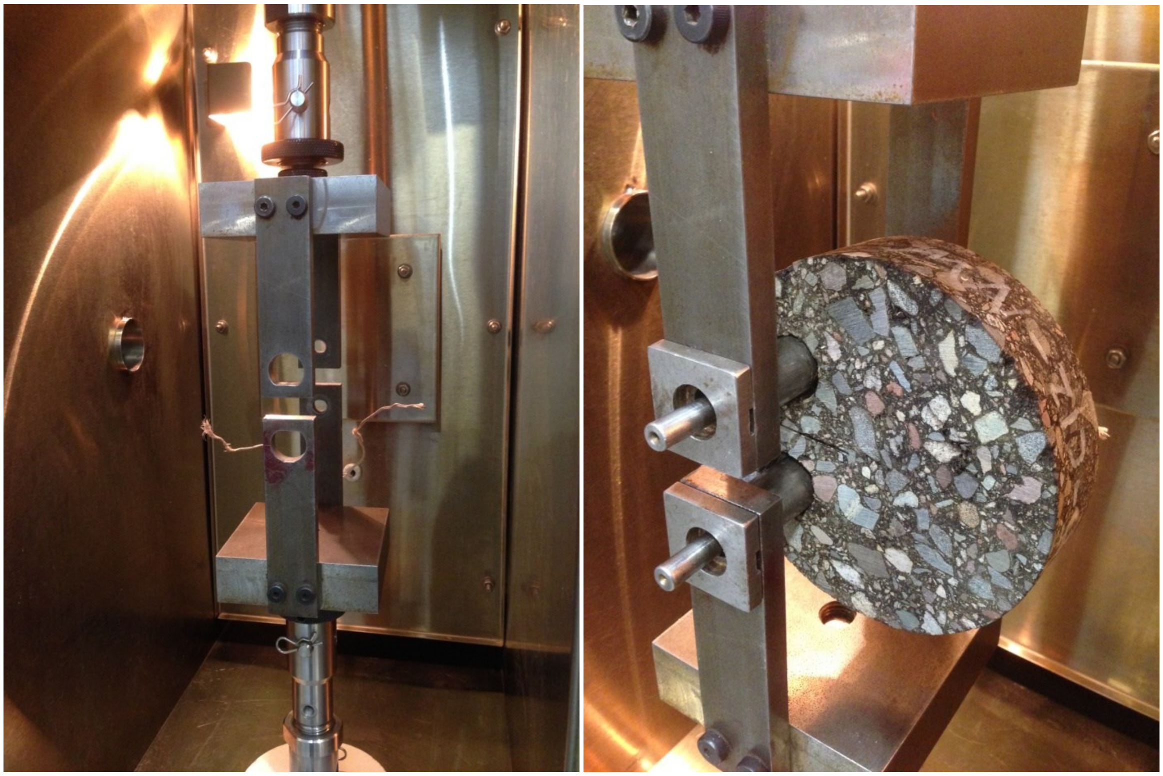

Figure 28: (a) DCT testing apparatus; (b) DCT specimen mounted onto apparatus

Figure 29: CMOD gage clipped to DCT specimen

A seating load of 100 N is applied prior to the beginning of testing. This reduces any potential loading "shock" that may cause a premature failure. Once preloading is complete, the upper loading clevis moves at a controlled rate; imposing a load on the DCT specimen. This rate is specified as 1 mm/minute and is dictated by the CMOD gage. The upper clevis increases the load applied to the sample, attempting to achieve this CMOD rate per minute. The load applied to the specimen will increase with little variation in the rate until initial failure of the sample takes place. Upon failure, the rate dramatically spikes. The upper clevis often has to retract slightly in order to keep the rate consistent with 1 mm/minute. The frame continues to add or remove load from the sample, until the load drops below 100 N (seating load value). At this time, the testing is stopped and the specimen is removed from the loading clevis. For this study, all samples from a section were tested during the same session. This reduced any potential variability from samples of the same section being tested on different days.

### *3.1.2 DCT Results Description*

Fracture energy is the work required to fracture the DCT specimen normalized against the area of the specimen. Previous research has indicated that fracture energy of 400 J/m$^2$ is a desirable threshold for DCT testing [3]. Mixtures testing above 400 J/m$^2$ exhibit little to no transverse cracking, while those below 400 J/m$^2$ feature higher levels of transverse cracking. In order to determine the fracture energy for each specimen, an extensive amount of information must be collected. The software controlling the rate at which the specimen is loaded stores 25 data points per minute. This data lists the time, load, CMOD displacement and chamber temperature over the course of each test. As mentioned earlier, the CMOD gage controls the rate of loading for the DCT test. When coupled with the software, it serves the additional purpose of recording the displacement of the mouth opening over the time span of the test. The relationship between load and CMOD is the most important in terms of calculating the fracture energy of a specimen. Figure 30 illustrates a sample of the load versus CMOD plot. Note that CMOD and time are essentially a direct relationship; i.e. as CMOD increases, time generally increases. As can be seen during the initial portion of the test, the DCT specimen resists a significant amount of load while exhibiting very little crack mouth displacement.

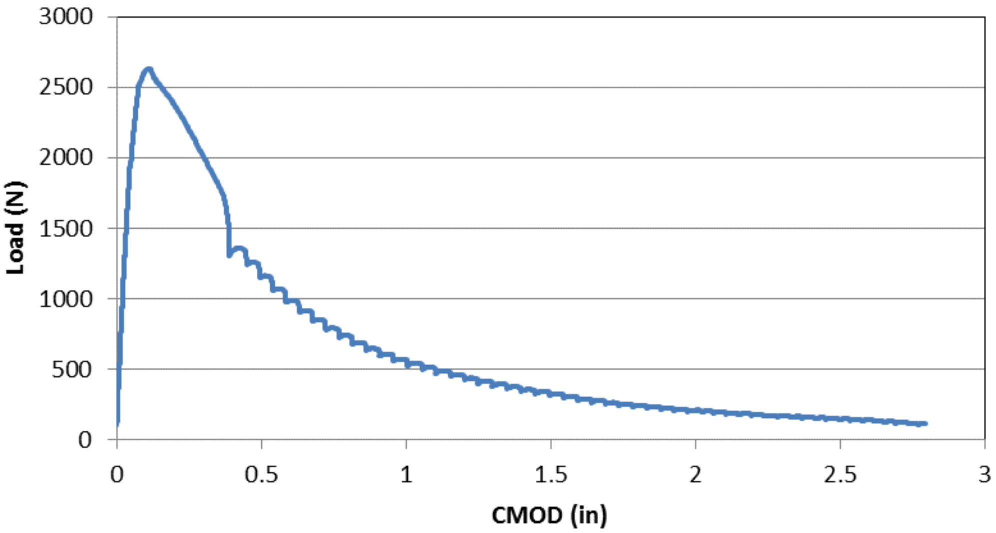

Figure 30: Sample DCT test output

At peak load the specimen experiences a quasi-brittle fracture, where a crack forms in a brittle manner at the inside of the notch of the DCT specimen. However, instead of forming a crack across the entire diameter of the specimen, softening can occur and the specimen continues to resist load as the crack mouth displacement increases. This phenomenon can be seen in Figure 30, as the load gradually decreases and CMOD continues to increase. For quasi-brittle materials, such as asphalt mixtures, the formation of a discrete crack is preceded by region of damage that is present ahead of the crack tip. This region is known as the fracture process zone.

Figure 31 clarifies the discussion on the fracture in asphalt materials. Both the green and red areas represent two separate specimens each having the same geometric specimen sizes and exhibiting the same peak load, thus the same tensile strength. However, the green specimen behaves in a more ductile manner (the specimen exhibits a greater amount of softening) than the red sample. This results in a significantly higher fracture energy, as the fracture work is much greater for the green specimen as opposed to the red specimen.

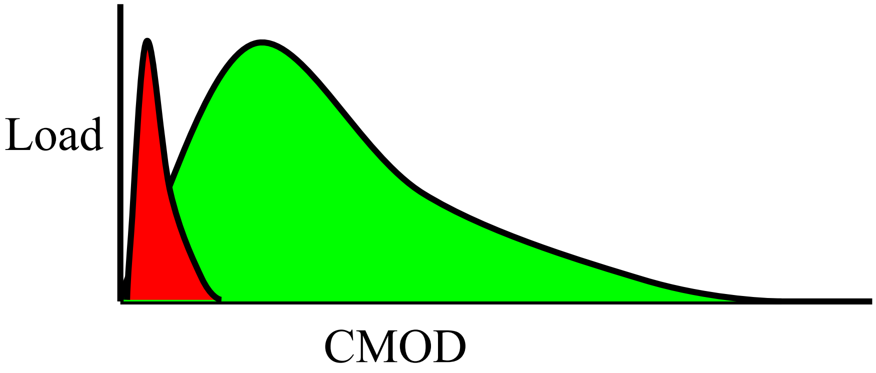

Figure 31: Comparison of brittle and ductile failure results

As mentioned before, an extensive number of data points are used to compile the plot shown in Figure 30. Using these discrete points, the area under the curve between any two consecutive points can be calculated. The sum of all of the individual areas is considered the <u>fracture work</u>. This work is then divided by the area of the fracture zone, which is the thickness of the sample multiplied by the ligament length. The result of this calculation is the <u>fracture energy</u> for an individual sample.

It should be noted that the normalization of samples for this research was especially important. Field cored specimens can vary greatly in thickness. This study incorporated thin overlays (≈1.5" or 37.5 mm). While the ASTM D7313 specification advises all DCT specimens to have a 2" (50 mm) thickness, this is obviously not feasible for the thin overlay sections. As a result, thin overlays do not result in high fracture work values, but the normalization of the area allows for a fair comparison between the 1.5" (37.5 mm) disks and typical 2" (50 mm) field cored samples.

### 3.2 Laboratory Testing Results

The results from each highway project and the individual study sections that were established can be found herein.

Table 4 provides DCT results for each tested section in the study. This table includes the average fracture energies, standard deviations and coefficients of variance (COV) for each section from DCT testing of the field cores. Generally a COV of 15% is considered high variability for a set of samples. However, this only applies to a set of samples produced under controlled laboratory conditions. Being that the samples for this study are field cored, there are many potential variations that could have occurred from one sample to another; the most notable being a core coming from two different days of paving. The COV values from the DCT testing for this study are provided primarily as a reference to show the potential variation in field cores. The values do not indicate any unnatural variability.

A notable challenge with this study is the influence of binder aging. Each individual section will feature a variable (and unknown) amount of aging in the corresponding binder. An attempt to mitigate the impact of this factor is accounted for in the cracking measures by normalizing each measure over the service life. However, each binder does not necessarily age at the same rate. The sections all see different climatic conditions. Fracture energy will be influenced by the age of the binder, with brittle binders providing a lower fracture energy than a ductile binder. Therefore, the age of binder in field cores can have an unpredictable effect on fracture energy performance. This can lead to additional uncertainty when comparing fracture energy between sections.

As can be seen in

Table 4, multiple sections were not available for DCT testing. These sections will be used for comparison purposes between field performance and mix design parameters, but will not be referenced further in discussions related to laboratory testing. The sections lacking laboratory testing results are not displayed in

Table 4 for brevity.

Table 4: Summary of DCT testing results

| Section | RP / Landmark | Performance | Test Temperature (°C) | Average Fracture Energy (J/m$^2$) | Standard Deviation (J/m$^2$) | Coefficient of Variance (COV) |
|---|---|---|---|---|---|---|
| TH 1 | RP 235 | Poor | -26.3 | 342 | 130 | 38%[1] |
| TH 1 | RP 230 | Good | -26.3 | 408 | 45 | 11% |
| TH 2 | RP 157 | N/A | -24.4 | 449 | 104 | 23% |
| TH 6 | RP 118 | Poor | -24.2 | 311 | 109 | 35%[1] |
| TH 6 | RP 123 | Good | -24.2 | 352 | 95 | 27% |
| TH 10 | RP 159 | Poor | -24.2 | 317 | 78 | 25% |
| TH 10 | RP 161 | Good | -24.2 | 365 | 66 | 18% |
| I-35 | N/A | N/A | -20.8 | 379 | 50 | 13% |
| TH 53 | 169 to Ely | N/A | -25.7 | 397 | 130 | 38% |
| TH 113 | RP 10 | Poor | -23.7 | 182 | 17 | 9% |
| TH 113 | RP 5 | Good | -23.7 | 326 | 54 | 17% |
| TH 210 | RP 118 | N/A | -24.8 | 293 | 76 | 26% |
| TH 212 | N/A | N/A | -20.7 | 1040 | 148 | 14% |

TH 212 has an exceptionally high fracture energy. This could be due to a number of factors, including (but not limited to) one of the following: new construction project, higher quality binder (PG 70-34), or stone matrix asphalt (SMA) mix design. The relatively low COV for TH 212 also shows that this value is not likely an anomaly. The field cracking results for this section also validate this number, as no transverse cracking has been observed on this roadway over the six year service life of the pavement.

TH 1 (RP 235) was a fairly poor performing section. It featured a high amount of transverse cracking throughout the service life, and currently exhibits 100% transverse cracking. The construction documents specify a 1.5” wear course for this section. However upon receiving field cores, the wear course was found to be approximately 1.25” and in very poor condition. Fabrication of DCT specimens was difficult, and two samples broke prematurely during testing. This section

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[1] The specimen thicknesses were relatively low due to thin lifts, this is anticipated to be primary reason for high COV in fracture energy results.

appears to have the potential for high fracture energy variability due to the poor quality of the roadway.

Figures illustrating the individual DCT test results used to generate the averages in

Table 4 can be found in the Appendix. These plots include the individual replicates, as well as the average fracture energies. Representative photos of DCT specimens for each section are supplied with these plots.

### *3.2.1 Comparison of Fracture Energy*

The plots in this section provide visual comparisons between good and poor performing sections (Figure 32 through Figure 35). In all instances, the good performing (GP) section exhibited a higher average fracture energy than that of the poor performing (PP) section. This provides further validation that fracture energy can differentiate between inferior and superior sections on a roadway.

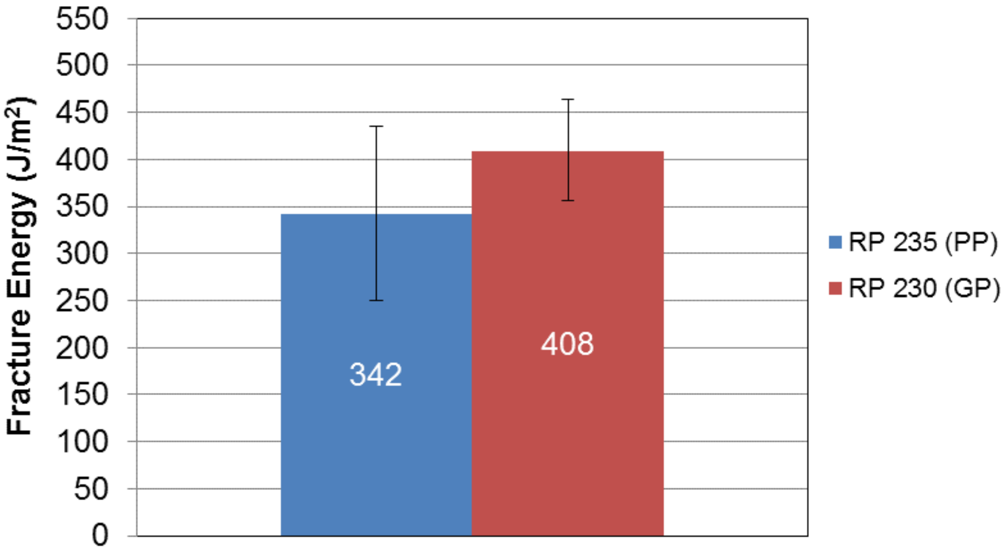

Figure 32: TH 1-poor performer versus good performer

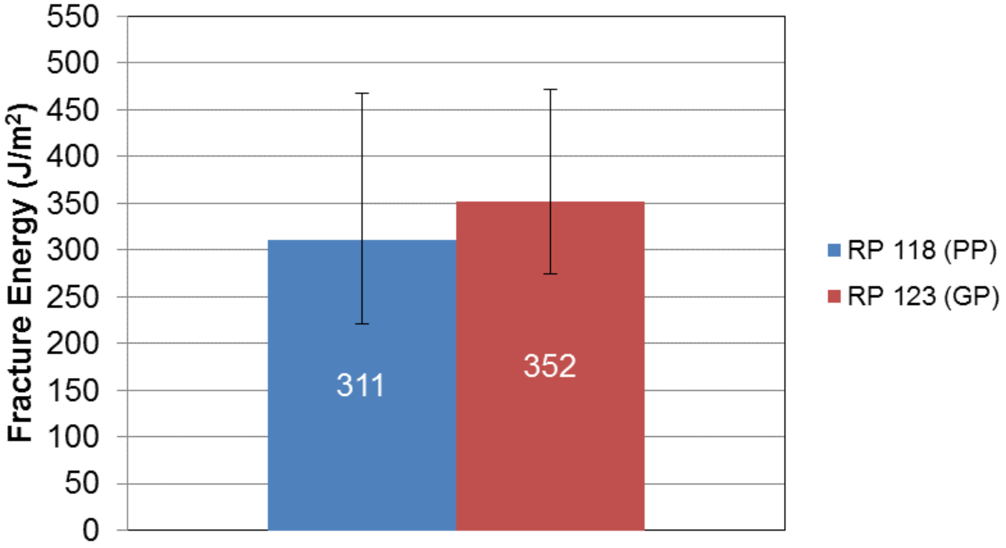

Figure 33: TH 6-poor performer versus good performer

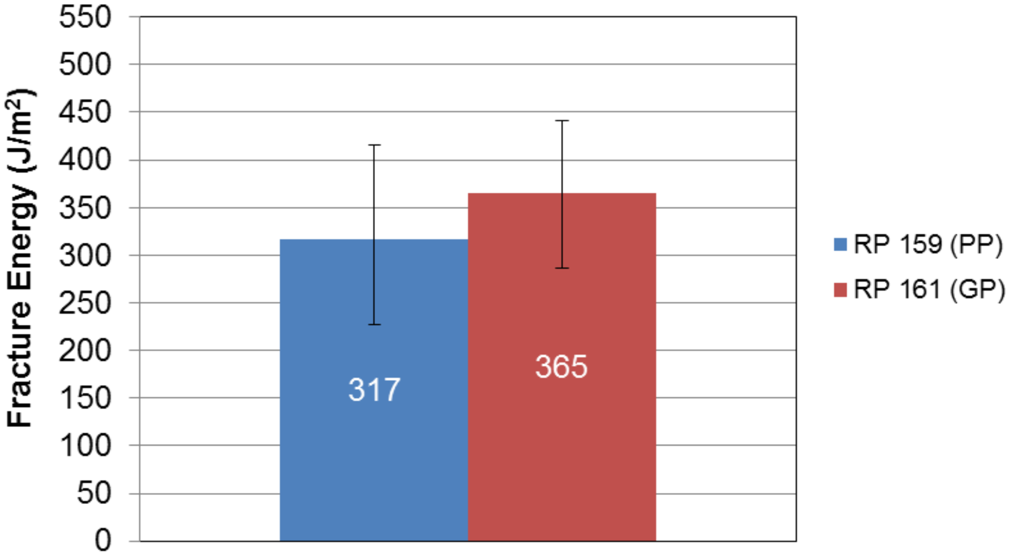

Figure 34: TH 10-poor performer versus good performer

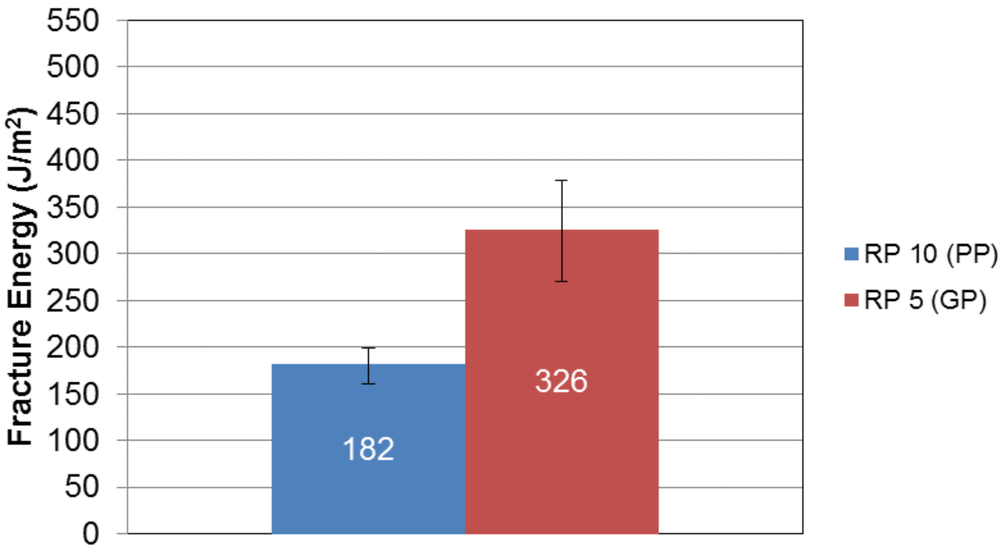

Figure 35: TH 113 poor performer versus good performer

### 3.3 Summary of Findings from Laboratory Evaluation

This phase consisted of conducting DCT testing on nine roadways with a total of 13 different sections. The DCT test results in a value expressed as fracture energy. This value is the summation of work required to fracture the sample over the duration of the test normalized against the specimen area. At least four specimens were attempted to be tested for each roadway. Due to the effort and funding required to extract field samples, a small number of specimens were available for testing. As a result of routine test procedures, samples would occasionally fracture prior to the test being conducted. This would result in less than four specimens being tested for a section. The research team feels that the number of tested specimens is still adequate in terms of providing a representative value for the corresponding section.

The discussion from this chapter evaluated the fracture energy differences between sections from the same roadway that were qualitatively assigned a “poor” or “good” performance designation. In all four instances, the poor performer exhibited a lower fracture energy than the good performing section. This provides further validation that the DCT test is accurate in quantifying the potential a roadway may have for transverse cracking.

# CHAPTER 4: COMPARISON OF FIELD PERFORMANCE, MIXTURE DESIGN PARAMETERS AND LABORATORY TESTING

## 4.1 Introduction

The comparisons discussed herein will evaluate the impact of mix design parameters on laboratory testing results. Previous research concluded that such a relationship may exist on the basis of a statistical analysis of mix design records and pavement management data. The main purpose of this phase is to determine if any correlation exists between mix design properties and laboratory performance testing. Thus, this task serves as a check or validation for the general findings made through field observations and previous research [18]. By conducting disk-shaped compact tension (DCT) testing on field cored samples of which mix design properties are known, any preliminary correlations between performance testing and mix parameters can be observed. Since this research involves using actual field sections and testing of field procured materials, only a small number of sections could be tested. Therefore, any results presented herein should only be used to validate previous findings or for purposes of designing future research. Note that

Table 4 should be referenced for any inquiries related to fracture energy values in the following material. Table 5 provides values of all cracking measures for each section, and is provided here as a reference for upcoming information. These values are discussed in depth and provided in relevant plots within the following subsections.

Table 5: Summary of transverse cracking performance

| Section | RP / Landmark | Performance | MTCTotal | MTCRTotal | ATCTotal | WATCTotal | TCTotal |
|---|---|---|---|---|---|---|---|
| TH 1 | RP 235 | Poor | 16.7 | 70.0 | 75.8 | 29.1 | 67.5 |
| TH 1 | RP 230 | Good | 13.7 | 82.0 | 13.7 | 2.3 | 6.8 |
| TH 2 | RP 157 | N/A | 5.4 | 12.0 | 31.5 | 6.3 | 28.8 |
| TH 6 | RP 53 | N/A | 8.3 | 18.0 | 24.8 | 11.5 | 20.6 |
| TH 6 | RP 118 | Poor | 10.0 | 40.0 | 65.2 | 16.3 | 60.2 |
| TH 6 | RP 123 | Good | 0.4 | 2.0 | 1.1 | 0.1 | 0.9 |
| TH 9 | RP 208 | Poor | 3.0 | 4.5 | 4.5 | 1.8 | 3.0 |
| TH 9 | RP 214 | Good | 2.0 | 3.0 | 3.0 | 1.2 | 2.0 |
| CSAH 10 | Jct 445B | Poor | 31.0 | 31.0 | 46.5 | 31.0 | 31.0 |
| CSAH 10 | Jct 446 | Good | 13.5 | 13.5 | 20.3 | 13.5 | 13.5 |
| TH 10 | RP 159 | Poor | 7.9 | 14.3 | 29.5 | 5.9 | 25.6 |
| TH 10 | RP 161 | Good | 6.4 | 21.5 | 30.7 | 6.8 | 27.7 |
| TH 10 | RP 75 | N/A | 45.5 | 45.5 | 45.5 | 45.5 | 22.8 |
| TH 25 | Jct 17 | N/A | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| TH 27 | RP 171 | Poor | 8.3 | 20.0 | 24.9 | 11.8 | 20.8 |
| TH 27 | RP 174 | Good | 0.4 | 2.0 | 1.1 | 0.1 | 0.9 |
| TH 28 | RP 81 | Poor | 15.0 | 20.0 | 20.0 | 12.5 | 12.5 |
| TH 28 | RP 88 | Good | 14.5 | 25.0 | 16.5 | 9.3 | 9.3 |
| CSAH 30 | Jct TH 95 | N/A | 10.0 | 12.0 | 14.0 | 9.0 | 9.0 |
| I-35 | N/A | N/A | 2.0 | 8.0 | 5.0 | 3.0 | 4.5 |
| TH 53 | Jct 169 | N/A | 12.0 | 72.0 | 54.0 | 22.9 | 50.3 |
| TH 113 | RP 10 | Poor | 10.4 | 56.0 | 68.6 | 21.4 | 63.4 |
| TH 113 | RP 5 | Good | 1.4 | 10.0 | 7.1 | 1.4 | 6.4 |
| TH 210 | RP 118 | N/A | 8.4 | 31.0 | 32.3 | 13.8 | 28.1 |
| TH 212 | N/A | N/A | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| TH 220 | RP 12 | N/A | 6.5 | 13.0 | 6.5 | 3.3 | 3.3 |

## 4.2 Comparison of Cracking Performance and Laboratory Testing

The relationship between average fracture energy for each section and the various measures described in Chapter 2 are illustrated in Figure 36 to Figure 40. Each measure shows a correlation between decreasing fracture energy and increasing transverse cracking. While some of the trends are minor, note that the number of data points is relatively insufficient to validate substantial relationships. Due to the variations between field cores (as mentioned earlier), it will take a significant amount of data to create trends that can be used as predictive functions. The trends in Figure 36 to Figure 40 are provided to suggest general guidance for future research. Considering this, the plots show encouraging trends for the following reasons; the function of DCT testing is to correlate potential cracking amounts to a corresponding high or low fracture energy. For all measures, higher fracture energies result in lower cracking amounts.

Maximum total transverse cracking (MTCTotal) is a simplistic way to evaluate cracking performance. It does not apply any value to a roadway that performed at near 0% cracking for the majority of the service life. It is a quickly calculated measure that provides users with a general sense of roadway performance. Figure 36 illustrates the results for MTCTotal versus fracture energy.

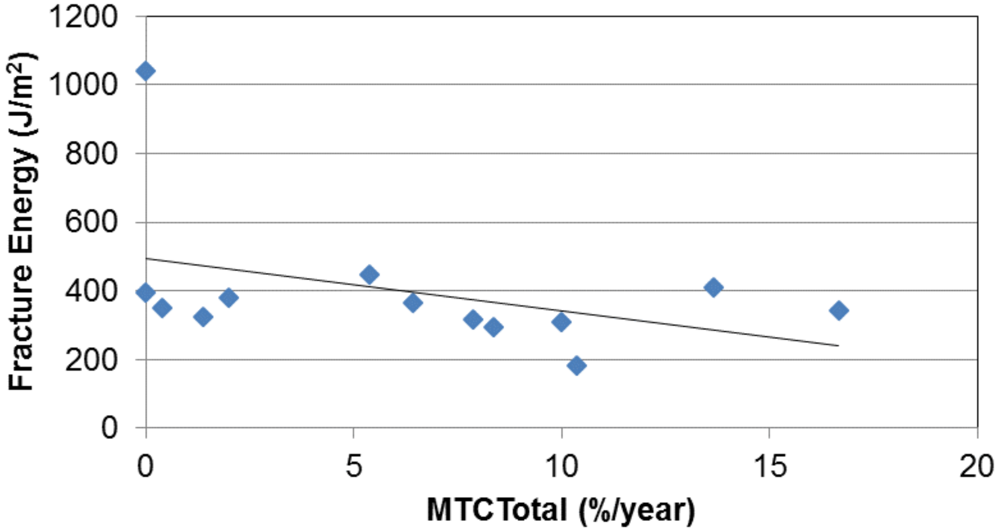

Figure 36: Fracture energy versus maximum total transverse cracking (FE vs. MTCTotal)

Maximum total transverse cracking rate (MTCRTotal) evaluates the maximum increase from two consecutive years. It provides a refined analysis, in comparison to MTCTotal, for a roadway. This is because gradual failure is generally more desirable than a quick, drastic failure. Figure 37 shows results for MTCRTotal versus fracture energy.

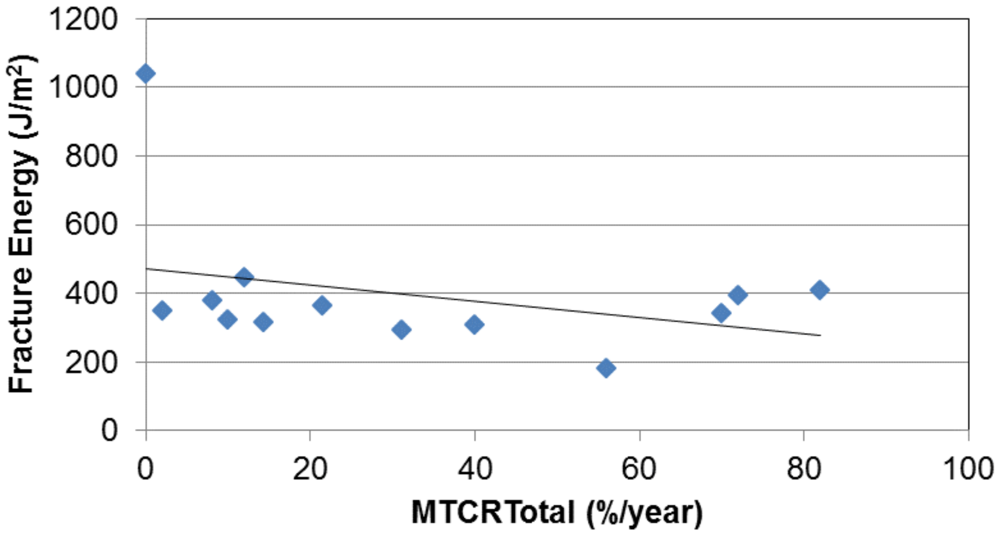

Figure 37: Fracture energy versus maximum total transverse cracking rate (FE vs. MTCRTotal)

Average total transverse cracking (ATCTotal) accounts for annual cracking rates, and is slightly more complex than MTCTotal and MTCRTotal, thus requiring more data to calculate. This is the first measure that takes into account annual cracking amounts. ATCTotal positively credits sections of roadways that exhibit lower levels of transverse cracking over the service life, and penalizes sections that crack early in service. Figure 38 illustrates results for ATCTotal versus fracture energy.

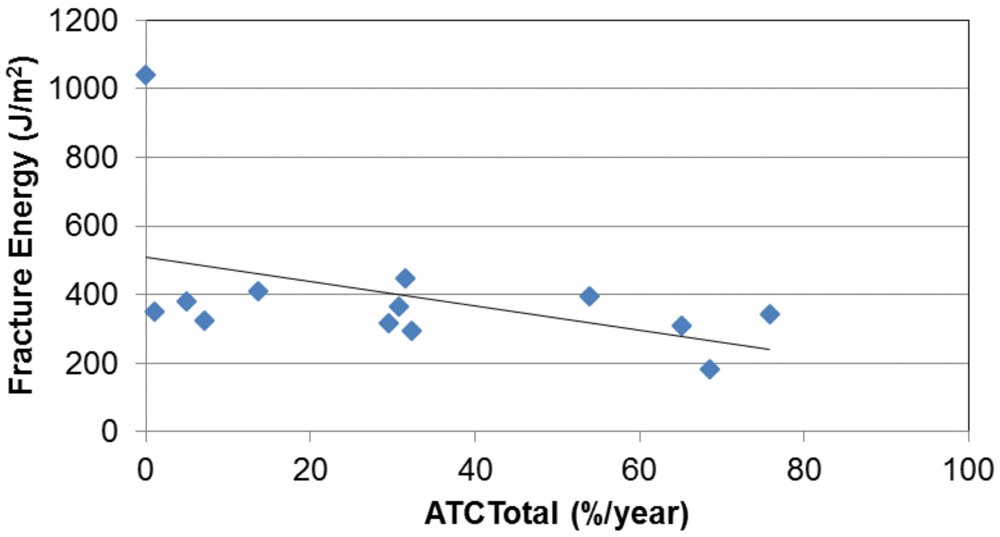

Figure 38: Fracture energy versus average total transverse cracking (FE vs. ATCTotal)

In comparison to ATCTotal, weighted average total transverse cracking (WATCTotal) provides further positive credit for sections that maintain low cracking levels throughout the service life. Building on the idea of evaluating annual cracking performance, each year is evaluated individually over the corresponding year(s) of service. The sum of these individual performance measures is then normalized for the total service life. WATCTotal versus fracture energy results can be seen in Figure 39.

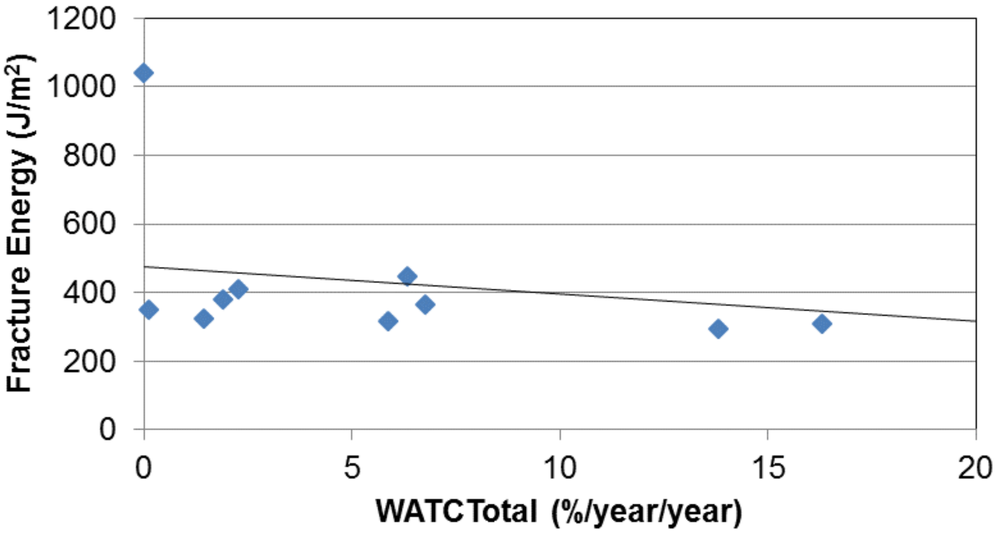

Figure 39: Fracture energy versus weighted average total transverse cracking (FE vs. WATCTotal)

Total transverse cracking (TCTotal) is the most complex measure presented in this report. Similar to the calculation of fracture energy, TCTotal is the sum of the transverse cracking performance exhibited by the roadway over the entire life of the pavement section (given by area under percent cracking and service life curve). This value is then normalized by dividing by the total service life. The results of TCTotal versus fracture energy can be found in Figure 40.

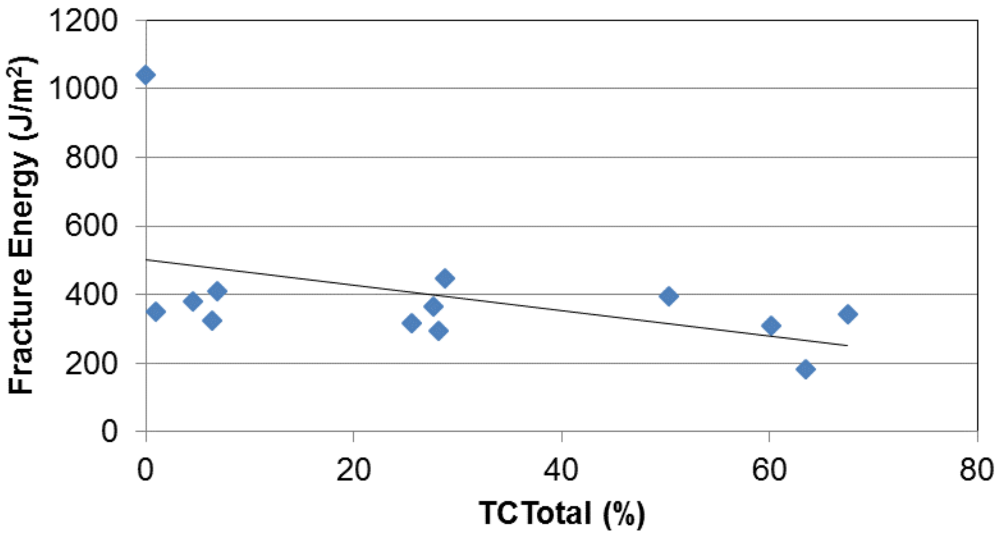

Figure 40: Fracture energy versus total transverse cracking (FE vs. TCTotal)

### *4.2.1 Normalization of Transverse Cracking Amounts against Traffic*

A typical relationship between fracture energy and transverse cracking data normalized for traffic level is illustrated in Figure 41. Total annual traffic, total annual truck traffic and daily traffic rates were all considered for this topic. All plots produced using this method resulted in a cluster of data near the origin and several points straying from this location, resulting in no true relationship. Upon removal of the "stray data", no relationship was found as the trend was essentially nonexistent. The data from this effort appears to validate that no strong correlation exists between traffic levels and fracture energy. Additional plots related to this topic can be found in the Appendix to this document.

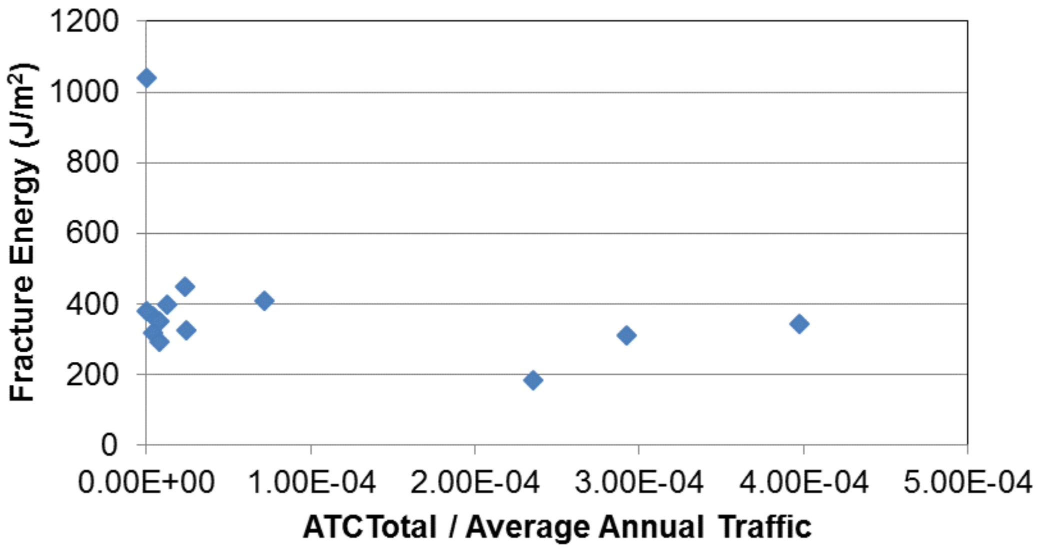

Figure 41: Fracture energy versus ATCTotal normalized for average annual traffic

### *4.2.2 Summary of Findings from Comparison of Cracking Performance and Laboratory Testing*

This phase of the study focused on the performance testing and comparison to field data for nine highways. During this task the field cores from each highway section, 13 sections in total, were tested using the disk-shaped compact tension test. Data was compared to field performance using various transverse cracking measures, in an effort to reduce a potentially misleading measure. Those performance measures were utilized in conjunction with the pavement management data and information from field visits to quantify the cracking performance of pavement sections. This data is presented in several fashions, considering both traffic level of the section and pavement construction type. The detailed analysis of the data conducted throughout this phase led to the following observations and potential recommendations for future research:

- A relationship between decreasing fracture energy and increasing transverse cracking amounts is apparent for various measures of cracking performance. This reaffirms the potential for using the DCT test as a performance indicator.

    - The impacts of binder age and core location have an unknown influence on the fracture energy discussed in this report. A core from the beginning of the project and a core from the end of the project can have significant differences. Similarly, a binder from a project in 2010 will have aged differently than a project in 2003. Both of these factors have unknown impacts, as it is virtually impossible to track. Future research should recognize these components when using field procured samples.
- Traffic levels do not appear to heavily influence cracking amounts.
- TH 212 performed at an exceptional level during testing, exhibiting an average fracture energy of 1,040 J/m$^2$. This is far greater than any other section in this study and well above the 400 J/m$^2$ threshold. Being that this section has experienced zero transverse cracking over the six year service life, it would appear to further validate the use of this threshold.
    - This section is the only SMA mixture and new construction project in this study, making any comparisons with other mixtures practically impossible. There are a multitude of factors that could contribute to the success of this mix and these factors will continue to be monitored in future studies.
- The small amount of data makes it difficult to confidently conclude any trend in this study. This study provides validation for the DCT test, cracking measures and contributing factors for fracture energy, but does not provide predictive function that can be used to determine the extent of field cracking on basis of measured DCT fracture energy. The use of simulation models (such as, IlliTC) is recommended for that purpose.

### 4.3 Comparison of Mix Design Parameters and Laboratory Performance

Early phases of this research established several correlations regarding the influence of mix design parameters on transverse cracking amounts. Based on the findings from previous studies as well as results from the comparison of field performance and laboratory testing, the fracture energy of the mix affects the transverse cracking performance. Therefore as transverse cracking increases or decreases, fracture energy shows similar trends. A review of key findings from earlier research [18] that correlate mix design parameters to transverse cracking, and fracture energy, are as follows:

- A higher percentage of crack free pavements were represented by asphalt mixes that have lower adjusted asphalt film thickness (AFT) and higher voids in mineral aggregates (VMA). For pavements that have cracks present in them, neither adjusted AFT nor VMA showed consistent trends.
- Asphalt binder grade has a significant impact on the pavement cracking performance. Mixes containing asphalt binders with low temperature grades of -34 have a greater amount of crack-free pavements as compared to mixes containing -28 binders. A lower percentage of pavements with significant amounts of transverse cracking are represented by mixes with -34 binder grades as compared to those with -28 binder grades.
- The amount of asphalt binder has an effect on field cracking performance. The mixes with higher asphalt content showed lower amounts of cracking.
- Very few pavements constructed with all virgin materials were present during initial analysis, thus limited data was available to draw any final conclusions regarding recycled materials.

These correlations provided guidance as to which mix design parameters required further inspection during this study. Using this information, the following mix design parameters were considered as potentially having an impact on field cracking performance:

- PG Grade

- PG Spread
- Asphalt Content
- Recycled Asphalt Content
- Voids in Mineral Aggregate (VMA)
- Voids Filled with Asphalt (VFA)
- Adjusted Asphalt Film Thickness (AFT)

Each of these parameters will be compared to fracture energy for the pavement sections listed in

Table 4. It should be noted that the objective here is not to develop correlations (or predictive equations) between mix design parameters and fracture energy but rather to determine if any of the mix design parameters show potential for such correlations to be developed through future research.

For any plots featuring a best fit regression line, it should be noted that the intention of this is not to show linear uniformity. The placement of this linear regression line is to simply show the approximate trend for the data being presented. In the data that follows, the blue markers represent all sections except for Trunk Highway 25 and Trunk Highway 212 which are represented with red markers. This is the typical condition unless explicitly defined in an alternative manner. In several instances multiple lanes (passing, driving etc.) were surveyed for same pavement section. Throughout the analysis the cracking performance for such sections are presented as average values for all lanes.

It should also be noted that the mixes in this study were developed using different material specifications. Some mixes were products of the VMA driven specification, while others were established using the AFT specification. These specifications have inherent differences. Some of these differences will be covered within this report (AFT, VMA). However, other unknown contributing factors from the use of these specifications may influence the results presented herein.

### *4.3.1 Effects of Asphalt Binder on Fracture Energy*

#### *4.3.1.1 PG Grade*

Figure 42 presents the plot of PG grade against fracture energy. This plot incorporates a “box-and-whisker” design, where the average is the “box” with the maximum and minimum values the “whiskers”. The average values are for all pavement sections that were constructed using same grade of the virgin binder in the mix. The plots are generated in the order of increasing average fracture energies with the PG grades. From these plots a loose trend of increasing fracture energy is observed as the binder grade goes in the order of PG 58-28, PG 64-28, PG 58-34 and PG 70-34.

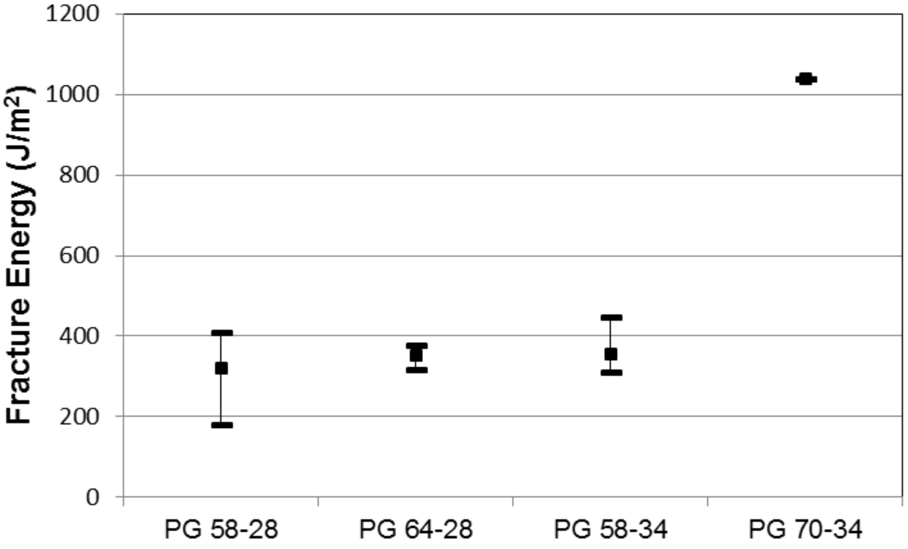

Figure 42: Effect of PG grade on fracture energy

*4.3.1.2 PG Low Temperature (LT) Grade*

Figure 43 presents the plot of PG low temperature (LT) grade against average fracture energies. As with Figure 42, this plot also incorporates a box-and-whisker design. The average values are for all pavement sections that were constructed using same PG LT grade. The plots are generated in the order of increasing fracture energy with the PG LT grades. TH 212 consisted of a PG 70-34 binder, the only representative of that grade in the study. Thus two data sets are provided for PG LT XX-34: one with TH 212 results and one without. From these plots a loose trend of increasing average fracture energy is observed as the low temperature binder grade goes in the order of PG XX-28, PG XX-34 (without TH 212) and PG XX-34 (all sections).

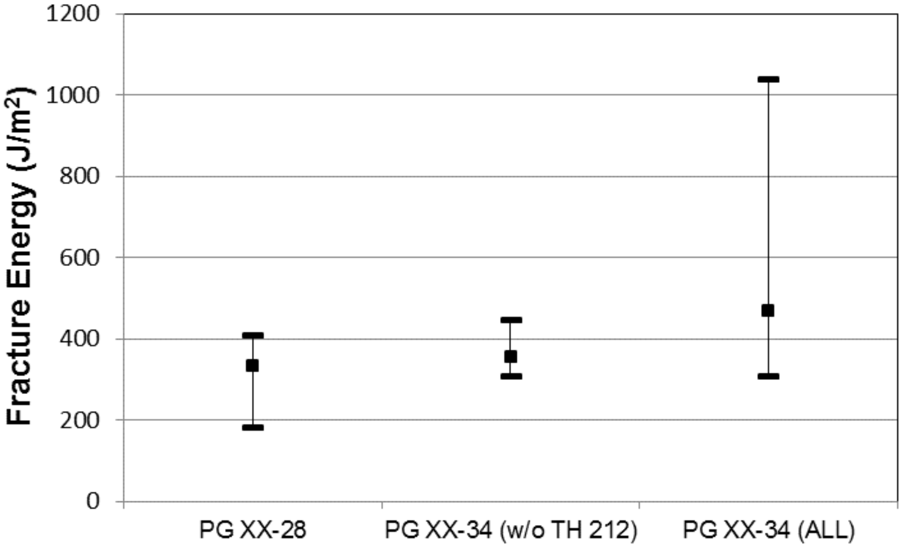

Figure 43: Effect of PG LT on fracture energy

*4.3.1.3 PG Spread*

The PG spread of the binder is defined as the total spread between the high and low performance grade temperatures for a binder. For example, the PG spread for PG 58-28 binder would be 86 (58 + 28). Figure 44 represents a similar trend to the PG grade plots. As the spread between the high temperature and low temperature of a binder increases, the average fracture energy of the study group increases accordingly. This appears to suggest that as the PG spread increases, the fracture energy also increases. Trunk Highway 212 was the only PG 70-34 binder in the study. Therefore, the spread of 104 only applies to one average fracture energy. The results from this mix are exceptional, with an average fracture energy of 1040 J/m$^2$. In the future, it would be beneficial to survey additional sections with PG 70-34 binder and/or SMA mix types to determine if the findings presented here in context of TH 212 are applicable to similar asphalt mixes.

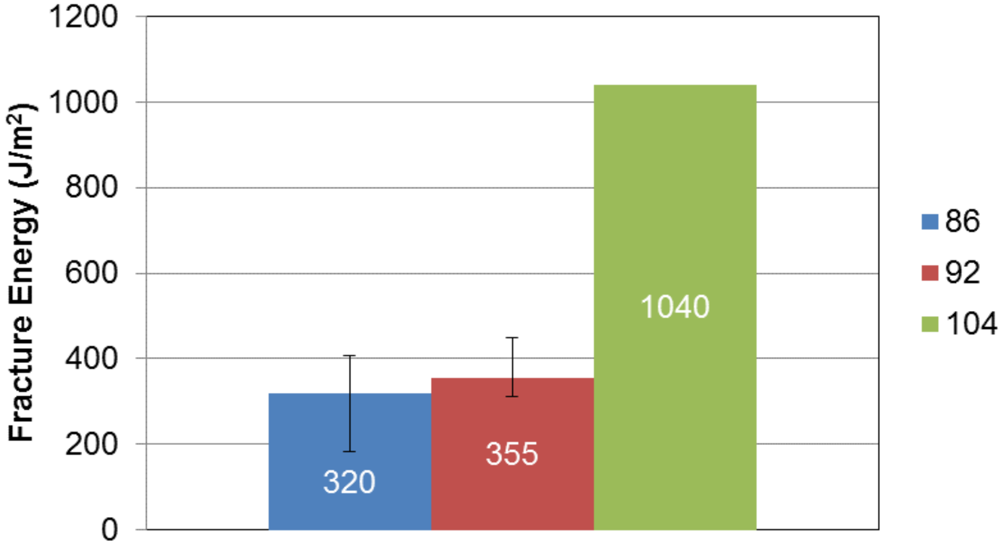

Figure 44: Effect of PG spread on fracture energy

### *4.3.2 Total Asphalt Binder Content in the Mix*

The plots analyzing the percent of asphalt content (Figure 45 and Figure 46) exhibit a general upward trend indicating superior DCT testing performance for mixes with higher asphalt binder contents. Figure 46 is provided to facilitate the observation of the typical "cluster" of data, as TH 212 results make this observation difficult. The upward trend concurs with the previous conclusions on asphalt mix design, claiming mixes with increased amounts of binder showed lower levels of transverse cracking, thus a higher fracture energy. If more asphalt is available to act as a medium for this "ductile straining" that occurs within the pavement system during low temperatures, it is reasonable that such a pavement would be more resistant to transverse cracking that occurs during DCT testing.

An additional trend is that the amount of scatter between data points appears to decrease as the amount of binder increases. This would seem reasonable as an asphalt mix with a higher binder content could rely less on the aggregate structure and more on the elastic properties of the binder during DCT testing. This would result in more consistent results with mixes of a higher binder content, as aggregate types and configurations can vary greatly between DCT specimens, even those of the same mix type.

It should be noted that while the averaged trend shows improving cracking performance with increased asphalt binder content, there is significant scatter in the data, indicating that other factors may also be important and asphalt binder content alone cannot be used as an independent performance measure.

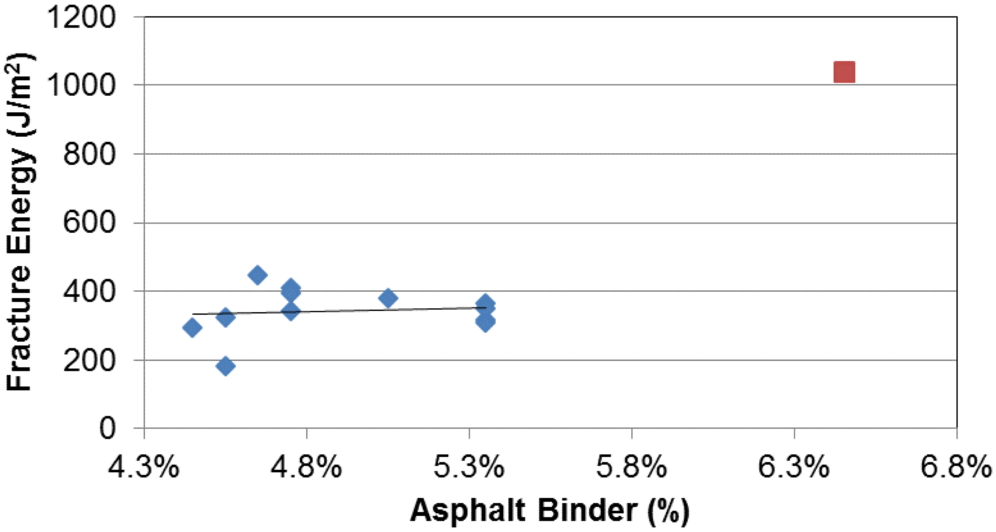

Figure 45: Effect of asphalt binder content (%) on fracture energy

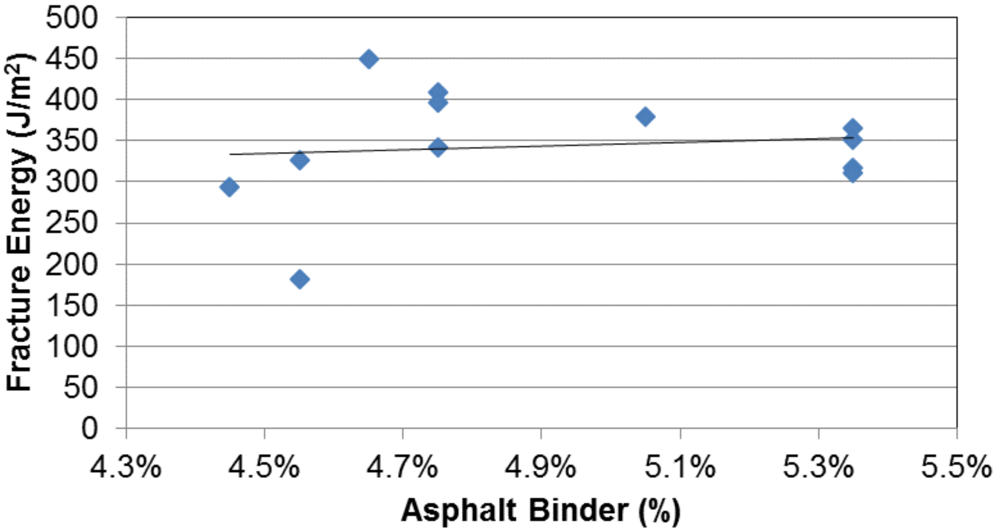

Figure 46: Effect of asphalt binder content (%) on fracture energy--excluding TH 212

### *4.3.3 Effect of Amount of Recycled Asphalt Content on Fracture Energy*

The impact of recycled asphalt content on transverse cracking is a multifaceted issue. It is difficult to draw any consistent conclusions since the amount of recycled asphalt binder is tied with many other variables such as: type and age of recycled asphalt pavement (RAP), type and amount of recycled asphalt shingles (RAS) and original grade of binder in recycled products. Additionally, Figure 47 does not appear to suggest any strong relationship exists. In this instance the scatter in the data seems to agree with presence of other variables. This once again supports the need for using laboratory testing based performance measures, such as DCT fracture energy, as opposed to using a mix design parameter as a performance control parameter. The fracture energies of the asphalt mixes studied herein have been determined through DCT testing of field sampled materials, and their relationship to transverse cracking can be found in Chapter 4.4.

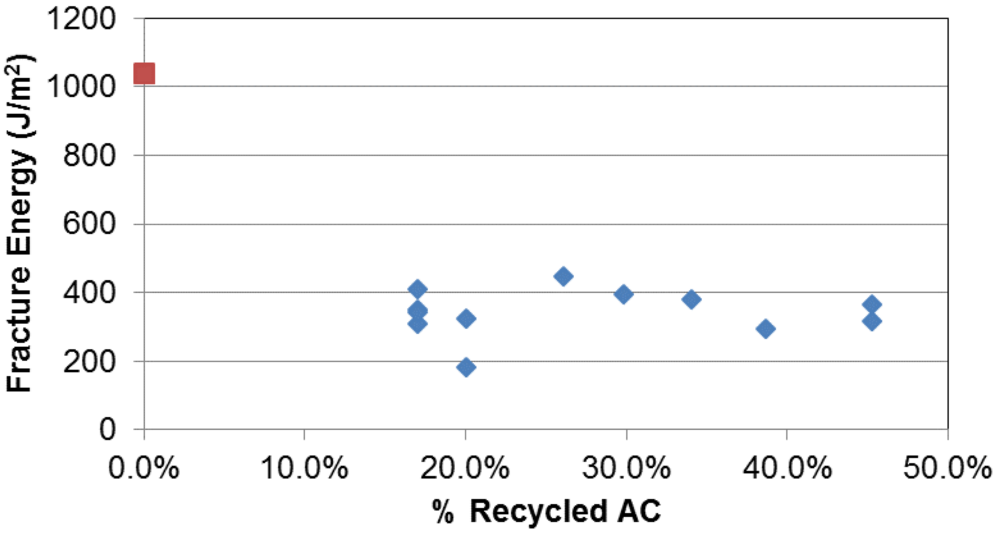

Figure 47: Effect of recycled asphalt content (%) on fracture energy

### *4.3.4 Effect of Voids in Mineral Aggregate (VMA) on Fracture Energy*

The comparison between fracture energy and the voids in mineral aggregate (VMA) of each mix can be found in Figure 48. All the mixes in this study are three-quarter inch maximum aggregate size; therefore normalizing for recommended VMA values is not beneficial. It should be noted that except for two projects (TH 113 and TH 210), the remaining mixes tested in this study were all designed and constructed using the older version of MnDOT 2360 specifications that required a minimum VMA amount. Both of the newer designs that utilized adjusted asphalt film thickness (AFT) specifications had significantly lower VMA amounts. Figure 48 does show a slight upward trend in fracture energy as VMA increases. The addition of different mix sizes to future studies may be advantageous when investigating the effect of VMA.

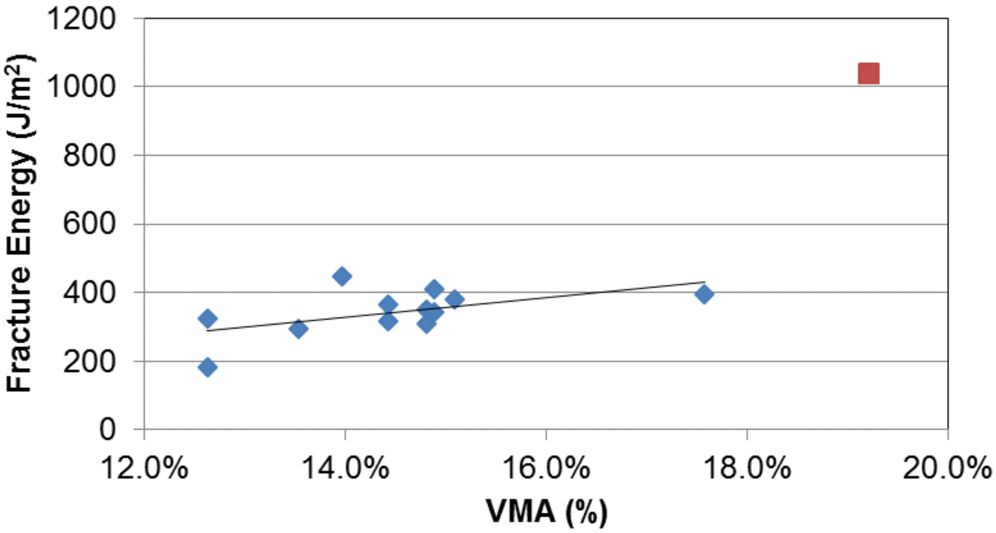

Figure 48: Effect of voids in mineral aggregate (VMA) on fracture energy

### *4.3.5 Effect of Voids Filled with Asphalt (VFA) on Fracture Energy*

Figure 49 shows the trend that results from the analysis of the voids filled with asphalt (VFA) of each mix. The data in this portion of the study has an identical relationship to that of fracture energy and VMA. While all the mixes in this study do not have the same design traffic level (basis for Superpave VFA recommendations), all of the mixes meet the suggested VFA range for the corresponding traffic level. Further studies will be required to validate if any relationship exists between fracture energy and VFA.

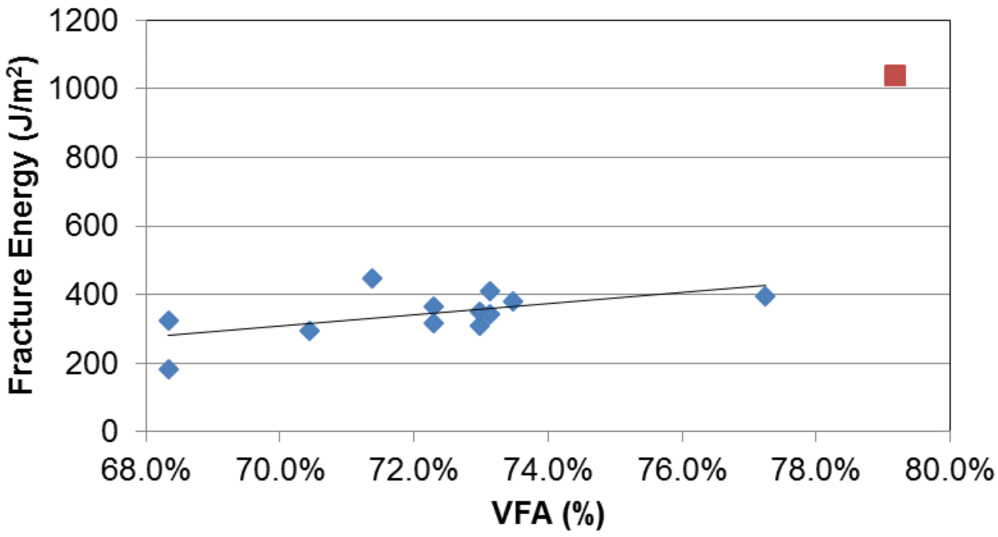

Figure 49: Effect of voids filled with asphalt (VFA) on fracture energy

### *4.3.6 Effect of Adjusted Asphalt Film Thickness (AFT) on Fracture Energy*

The adjusted asphalt film thickness (AFT) for various mixes are plotted against the fracture energies from laboratory testing in Figure 50. For the mixes designed and produced using the older MnDOT 2360 specifications the adjusted AFT values were calculated using the information from MDRs and the mix test summary sheets (TSS). It should be noted that the TH 212 results were not included in Figure 50, as the TSS were not available for this section.

The plot does not appear to indicate any trend relating fracture energy and adjusted AFT. As with other parameters the data is still prone to significant scatter and this information should not be used for the purposes of drawing conclusions.

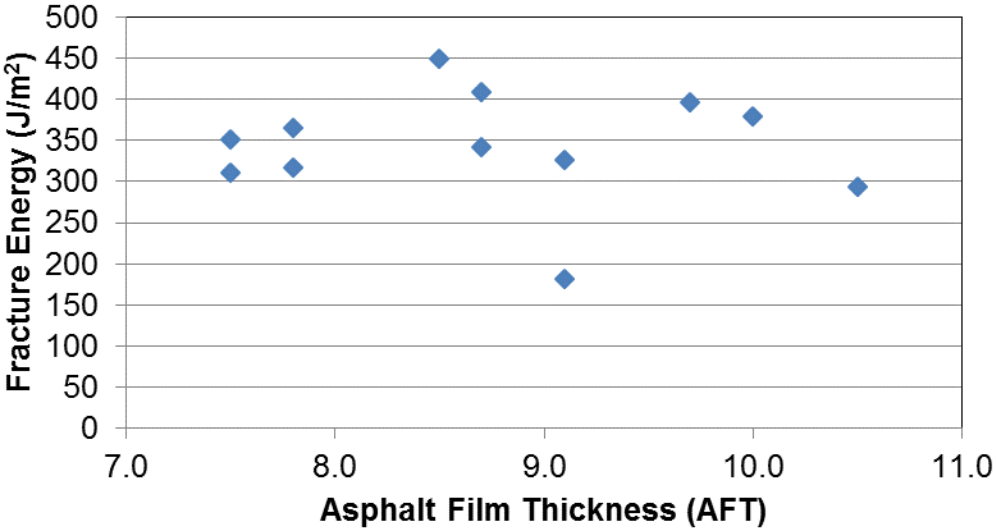

Figure 50: Effect of asphalt film thickness (AFT) on fracture energy

### *4.3.7 Summary of Findings Relating Mix Design Parameters and Laboratory Testing*

The analysis of this data focused on the laboratory testing from field samples of nine highway projects and thirteen pavement sections. These represent four virgin binder types, three pavement section types and two design traffic levels. This task had a primary goal of validating aforementioned recommendations and determining if any mix design parameters had potential for use as an indicator of fracture energy performance.

The influence due to binder aging and/or different material specifications should be noted again. The impact of these factors is difficult to quantify. Binder age is not necessarily related to service life, while the use of different material specifications has many implications that cannot be easily identified. The issues with these should be considered in any future studies involving field procured specimens. Overall, analysis of mix design parameters revealed the following correlations and conclusions:

- PG grade had a slight correlation to higher fracture energy as the performance grade of the binder progressed in the order of PG 58-28, PG 64-28, PG 58-34 and PG 70-34. It should be noted that the study did not look at the type of modification for manufacture of PG XX-34 binders. A separate analysis is presently underway at UMD to look at effects of binder modification on field cracking performance.
- As PG spread increased, it appeared to correlate with a higher fracture energy. However, only one section was available for the PG spread of 104. This finding should be further validated in future studies.
- PG low temperature grade showed a loose trend of improved fracture energy for PG XX-34 as compared to PG XX-28.
- Asphalt content showed a general increase in fracture energy as the amount of binder increased.

- Asphalt film thickness did not feature a significant trend. If anything, a lower fracture energy correlated to an increase in adjusted AFT. This is an indication that this parameter has a negligible impact on fracture energy, and thus transverse cracking performance.
- All other mix parameters showed minimal to no correlation with laboratory testing.

In general, the results indicate that the use of mix design parameters as an independent fracture energy performance predictor is not recommended. These findings are not entirely surprising. Earlier analysis showed that some parameters have potential to be performance predictors but none showed a very strong correlation. The findings from this phase reinforce these recommendations of using laboratory testing based performance parameter. While some parameters indicate slight trends with increased fracture energy, none are definitive. PG grade, PG spread and asphalt content all show encouraging trends. These will be observed closely in future studies.

### 4.4 Comparison of Field Performance and Mix Design Parameters

Similar to the comparison of mix design parameters and laboratory testing performance, this section will discuss the comparison of field performance and mix design parameters. As with previous comparisons, cracking performance measures of interest are described in detail within Chapter 2 and pertinent values can be found in Table 3. It is recognized that transverse cracking performance of certain mix design parameters may be altered by other factors. The initial review of data led to the conclusion that traffic level and asphalt layer thickness were two potentially significant variables. Effects of these variables were accounted for during the analysis procedure through normalization. Traffic level was taken into account by dividing the corresponding transverse cracking measure with the average daily truck traffic for each individual section. Similarly, asphalt layer thickness was normalized by multiplying the corresponding transverse cracking measure with the total asphalt layer thickness. Indirectly this also accounts for the potential added cost of a thicker asphalt layer.

After normalizing for either traffic level or asphalt layer thickness the resulting correlations between majority of cracking performance measures and asphalt mix parameters did not exhibit any recognizable trends as compared to before normalizing. Only selected plots with normalized data are provided in the report for brevity, these are data sets where some observable trends were noticed. The remaining plots from this effort may be found in the Appendix of this report.

### *4.4.1 Effects of Asphalt Binder on Transverse Cracking Performance*

#### *4.4.1.1 PG Grade*

The plots presented in Figure 51 to Figure 55 compare the results of PG grade against maximum total transverse cracking amount. These plots incorporate a box-and-whisker design, where the average is the "box" with the maximum and minimum values as the "whiskers". The average values are for all pavement sections that were constructed using same grade of the virgin binder in the mix. The plots are generated in the order of decreasing averaged cracking measures with the PG grades. From these plots a loose trend of decreasing transverse cracking performance is observed as the binder grade goes in the order of PG 58-28, PG 58-34, PG 64-28, PG 64-34 and PG 70-34.

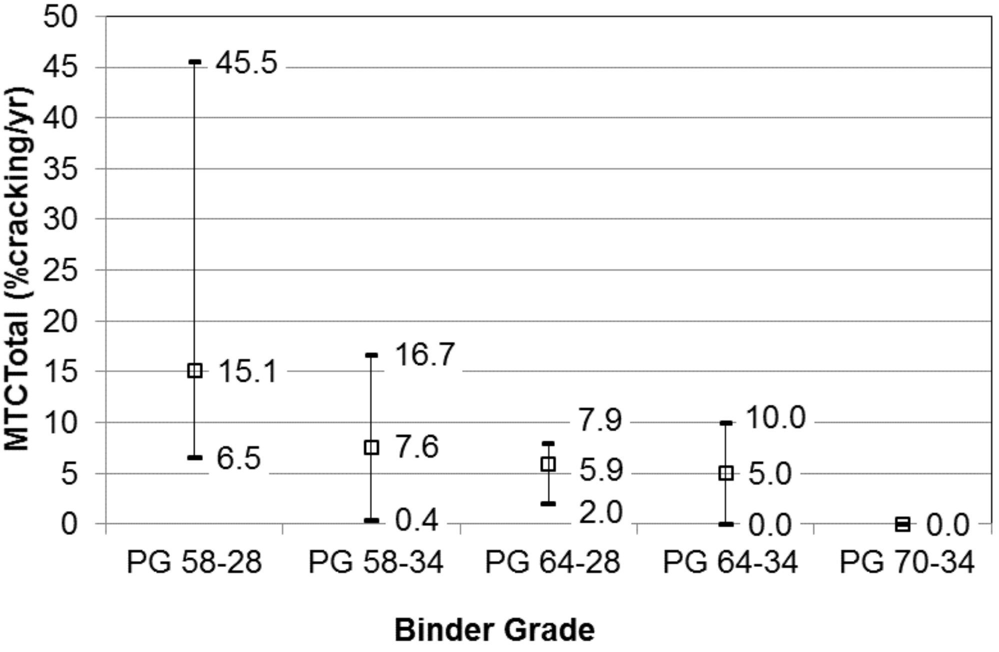

Figure 51: Effect of asphalt binder grade on the maximum total transverse cracking amount (MTCTotal)

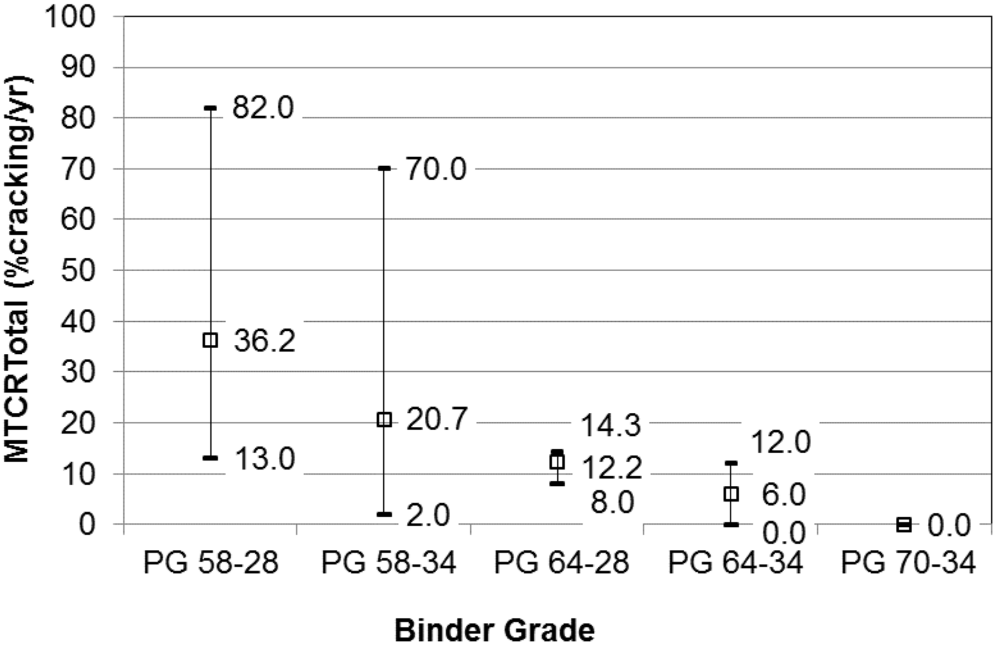

Figure 52: Effect of asphalt binder grade on the maximum total transverse cracking rate (MTCRTotal)

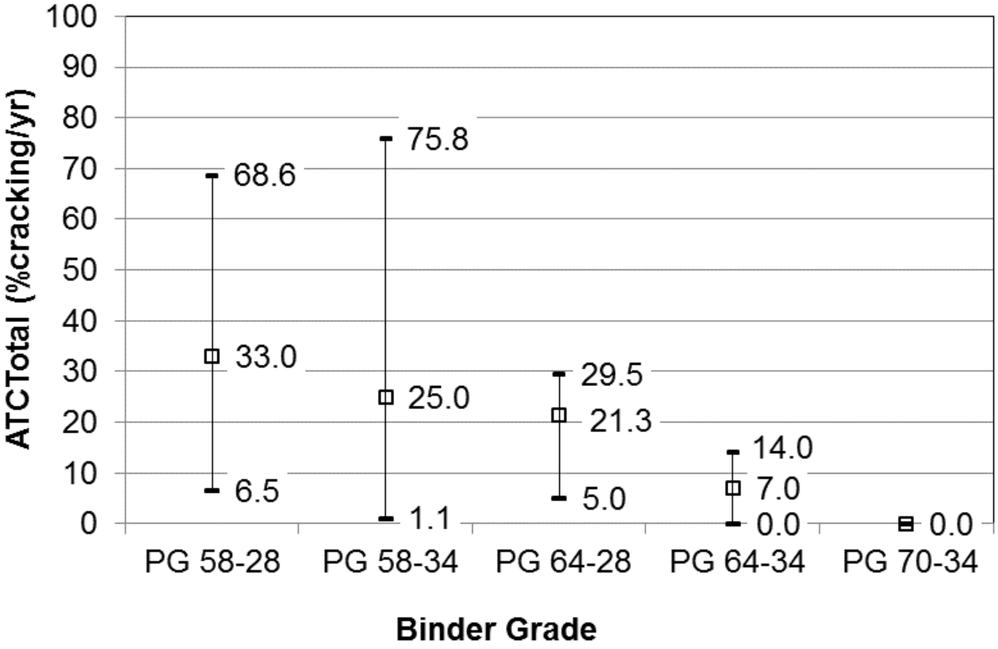

Figure 53: Effect of asphalt binder grade on the average total transverse cracking amount (ATCTotal)

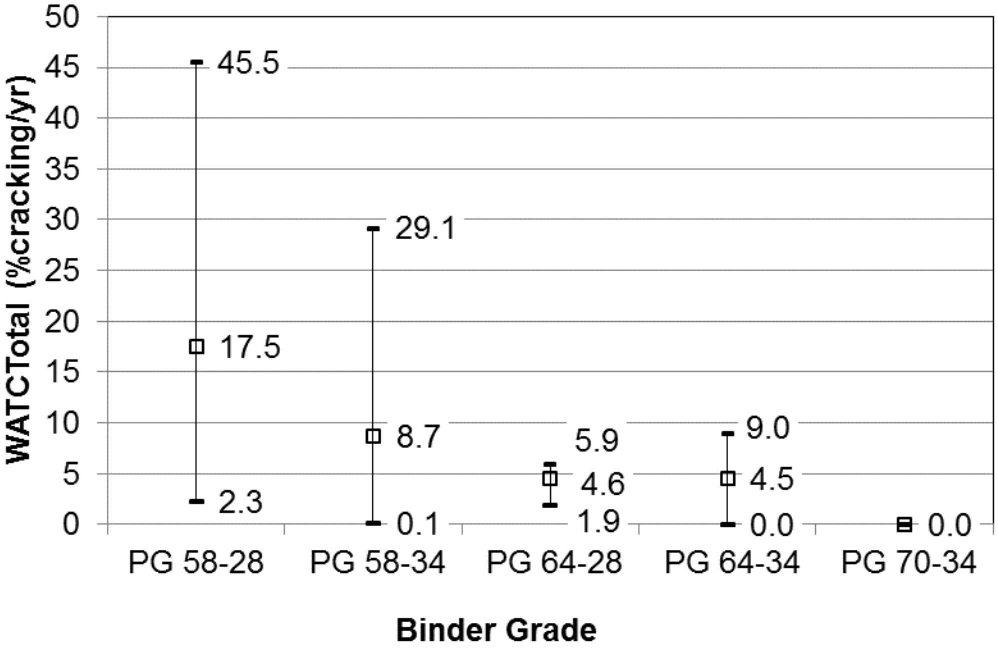

Figure 54: Effect of asphalt binder grade on the weighted average total transverse cracking amount (WATCTotal)

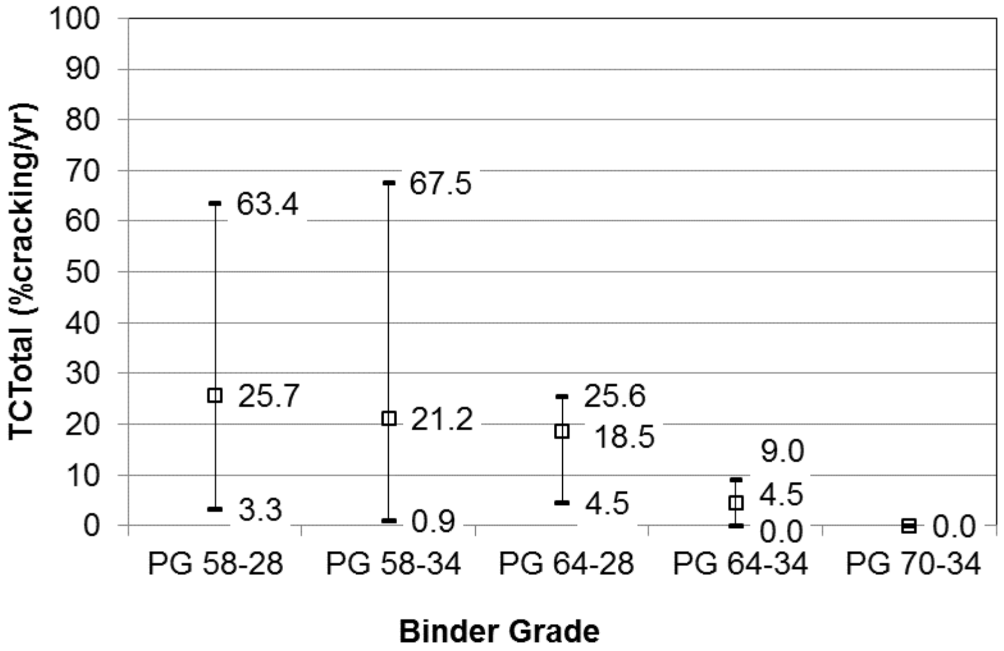

Figure 55: Effect of asphalt binder grade on the total transverse cracking amount (TCTotal)

*4.4.1.2 PG Spread*

The PG spread of the binder is defined as the total spread between the high and low performance grade temperatures for a binder. For example, the PG spread for PG 58-28 binder would be 86 (58 + 28). Figure 56 and Figure 60 exhibit similar trends to the PG grade plots. As the spread between the high temperature and low temperature of a binder increases, the transverse cracking performance of pavement deteriorates. However, most of the measures feature a significant amount of scatter at each PG spread. This suggests that while a downward trend is present, PG spread alone will not sufficiently predict transverse cracking performance. It is interesting that both Trunk Highway 212 and Trunk Highway 25 exhibit zero transverse cracking, but also feature the two largest PG spreads. However as mentioned previously, these projects are the only two projects with new bituminous on aggregate base construction. In the future, it would be beneficial to survey additional sections with PG 70-34 or PG 64-34 binder and/or SMA mix types to determine if the findings presented here in context of TH 212 and TH 25 are applicable to other highways.

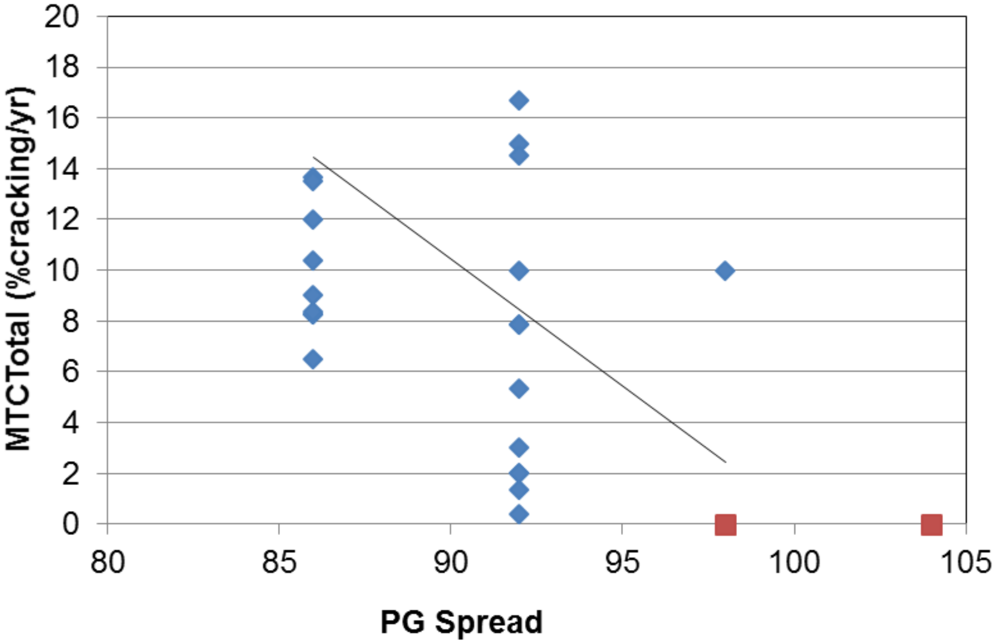

Figure 56: Effect of Performance Grade spread of asphalt binder on the maximum total transverse cracking amount (MTCTotal)

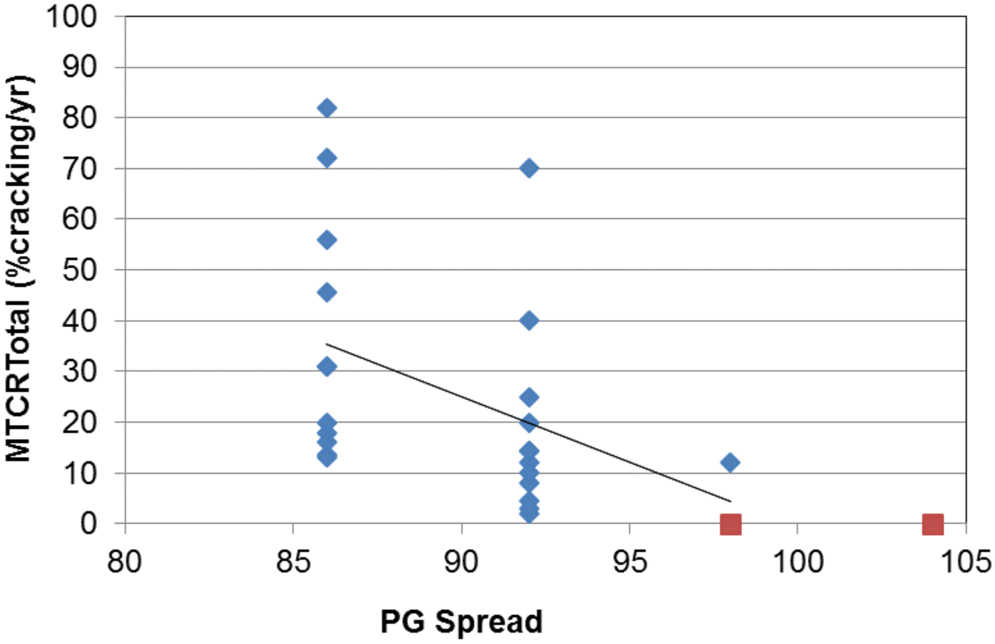

Figure 57: Effect of Performance Grade spread of asphalt binder on the maximum total transverse cracking rate (MTCRTotal)

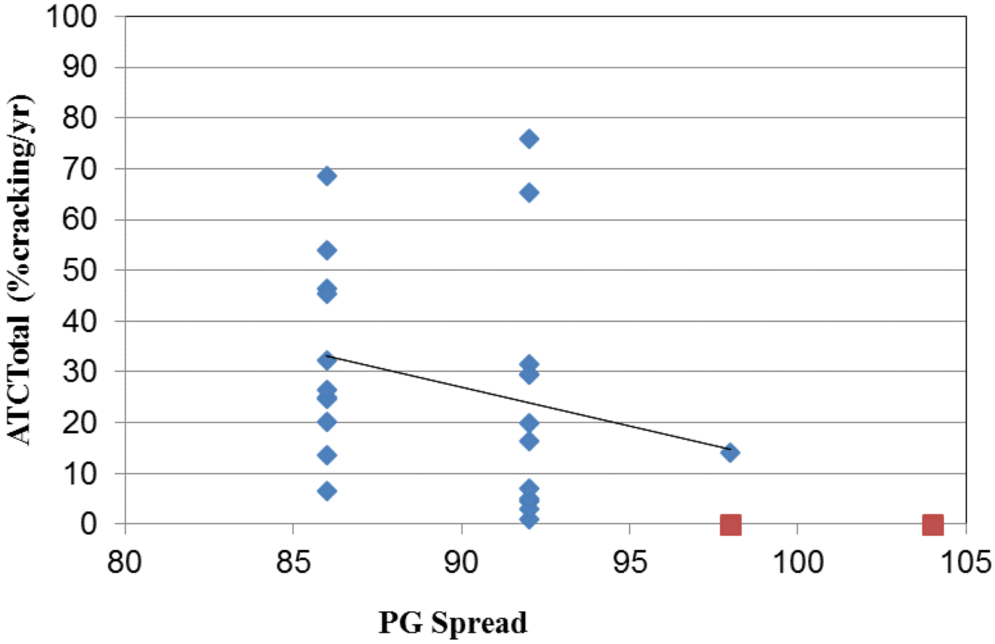

Figure 58: Effect of performance grade spread of asphalt binder on the average total transverse cracking amount (ATCTotal)

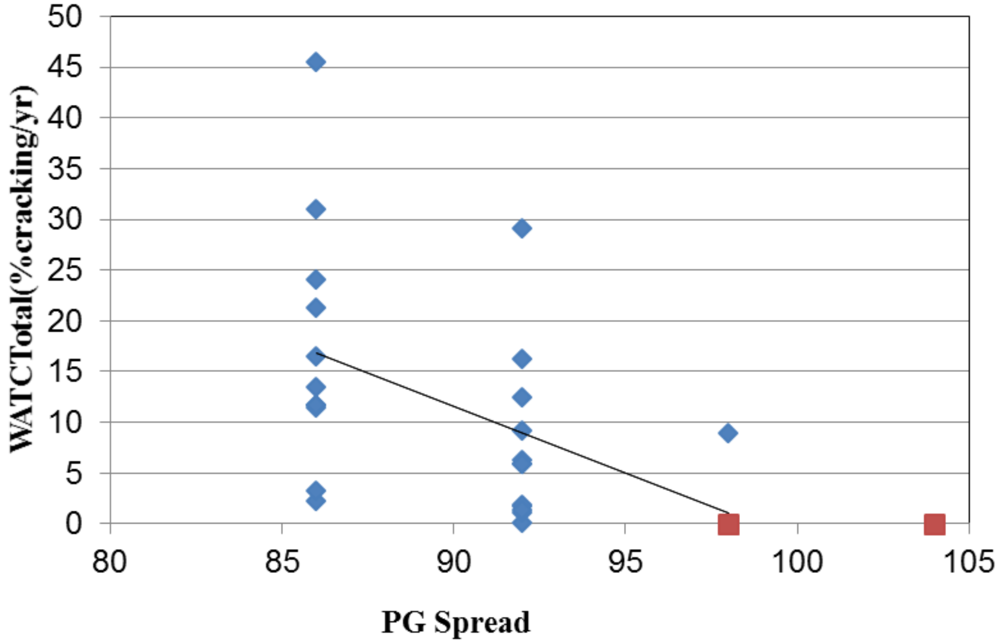

Figure 59: Effect of Performance Grade spread of asphalt binder on the weighted average total transverse cracking amount (WATCTotal)

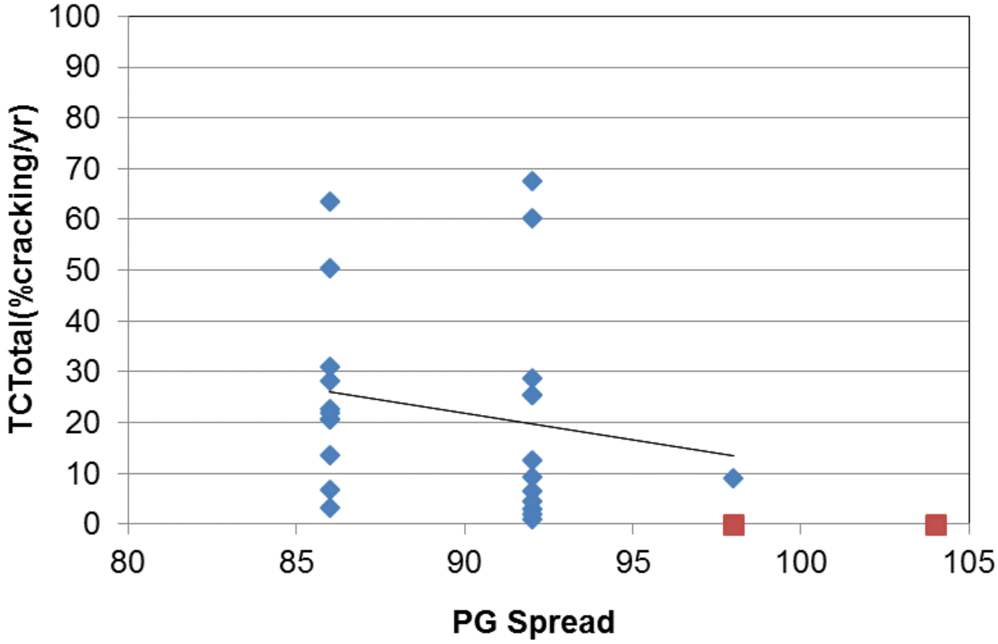

Figure 60: Effect of Performance Grade spread of asphalt binder on the total transverse cracking amount (WATCTotal)

#### *4.4.1.3 Total Asphalt Binder Content in the Mix*

The plots analyzing the percent of asphalt content (Figure 61 to Figure 65) exhibit varying trends between performance measures. A general downward trend indicating superior transverse cracking performance for mixes with higher asphalt binder contents occurs when observing MTCTotal and WATCTotal. Transverse cracking is the product of an asphalt pavement contracting under extreme low temperatures. If more asphalt is available to act as a medium for this “ductile straining” that occurs within the pavement system, it would seem reasonable that such a pavement would be more resistant to transverse cracking.

However, the other measures show slight upward trends suggesting cracking performance decreases as binder content increases. It should be noted that a large amount of scatter is present in these plots. This indicates that other factors may also be important and asphalt binder content alone cannot be used as an independent performance measure. While a general trend may exist that concurs with the recommendations from earlier research, other factors (possibly binder aging) have an impact on performance. This strongly illustrates the need for a performance test, as many factors have an impact on cracking performance.

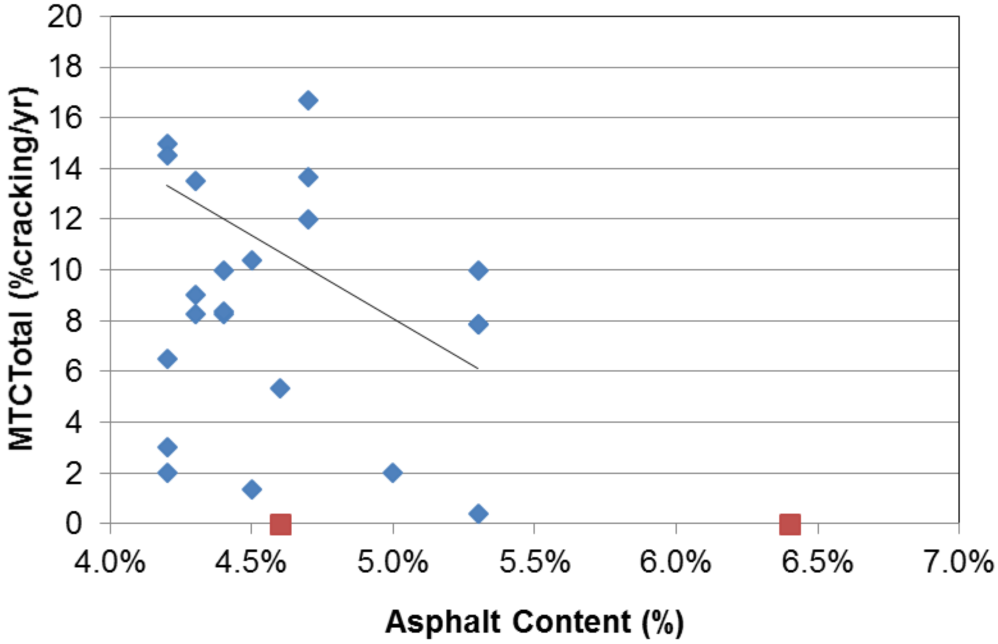

Figure 61: Effect of percentage of asphalt content on the maximum total transverse cracking amount (MTCTotal)

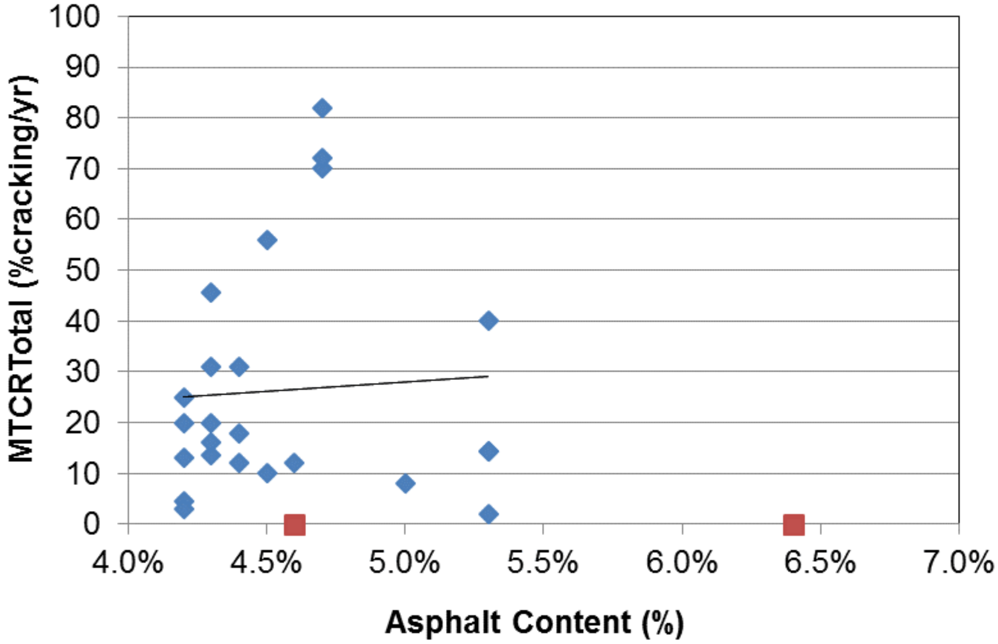

Figure 62: Effect of percentage of asphalt content on the maximum total transverse cracking rate (MTCRTotal)

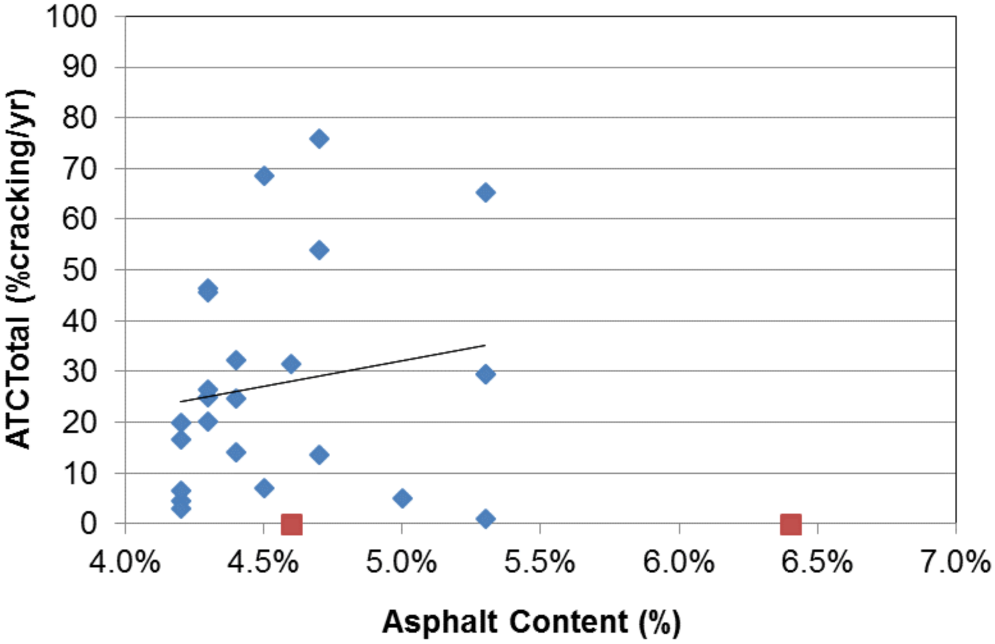

Figure 63: Effect of percentage of asphalt content on the average total transverse cracking amount (ATCTotal)

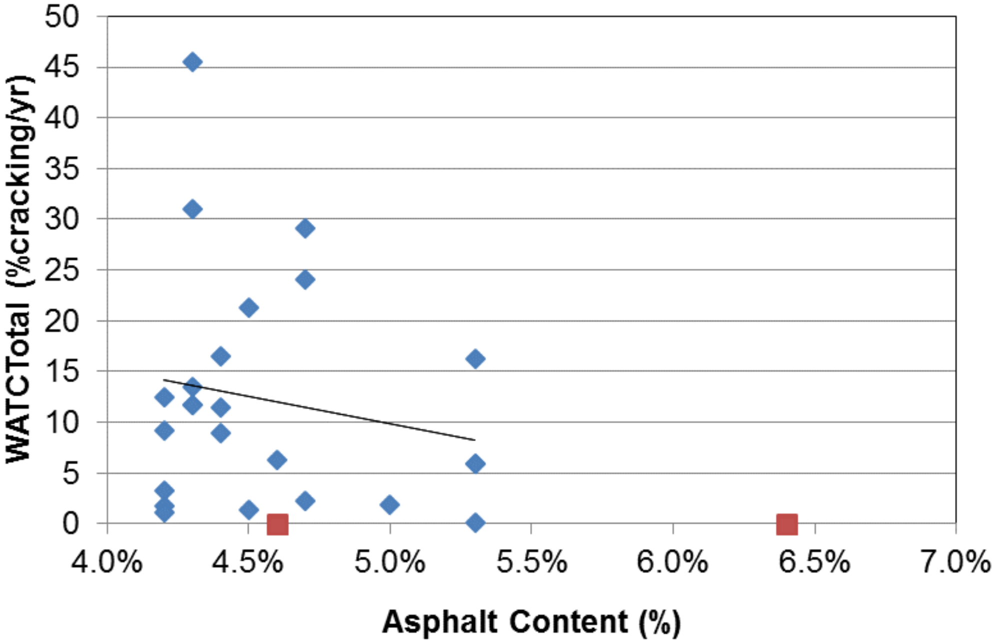

Figure 64: Effect of percentage of asphalt content on the weighted average total transverse cracking amount (WATCTotal)

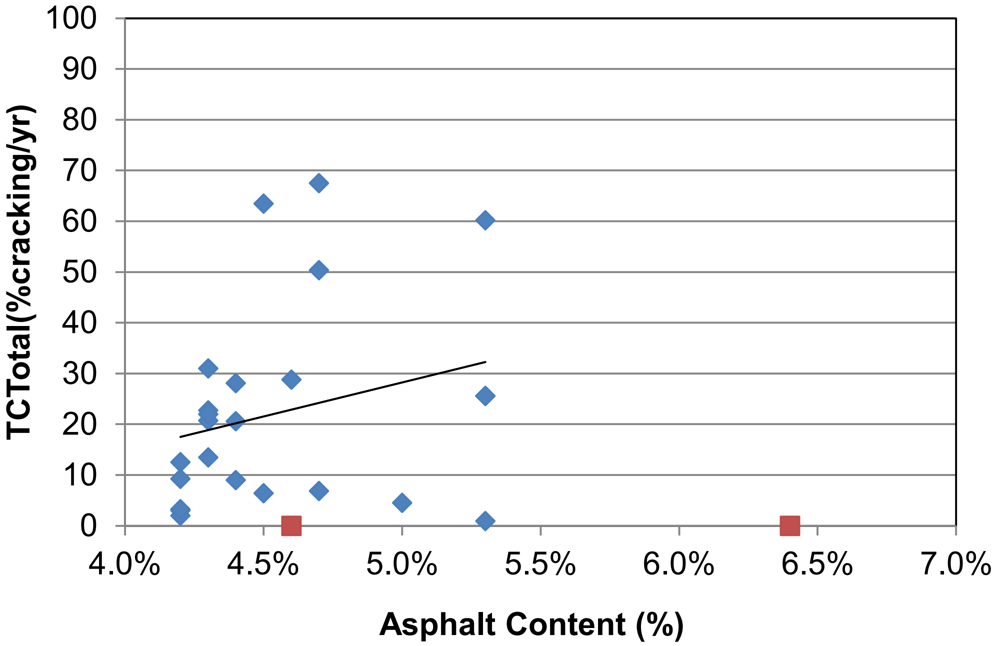

Figure 65: Effect of percentage of asphalt content on the total transverse cracking amount (TCTotal)

### *4.4.2 Effect of Amount of Recycled Asphalt Content on Transverse Cracking Performance*

The impact of recycled asphalt content on transverse cracking is a multifaceted issue. While Figure 66 to Figure 70 show a slight downward trend as the amount of recycled asphalt binder increases, there are four sections near zero transverse cracking and less than 20% recycled binder. These two values would appear to follow the common philosophy that virgin binder results in a better performing pavement. It is difficult to draw any consistent conclusions since the amount of recycled asphalt binder is tied with many other variables such as: type and age of recycled asphalt pavement (RAP), type and amount of recycled asphalt shingles (RAS) and original grade of binder in recycled products. In this instance the scatter in the data seems to agree with presence of other variables. This once again supports the need for using a laboratory testing based performance measure, such as DCT fracture energy, as opposed to using a mix design parameter as a performance control parameter.

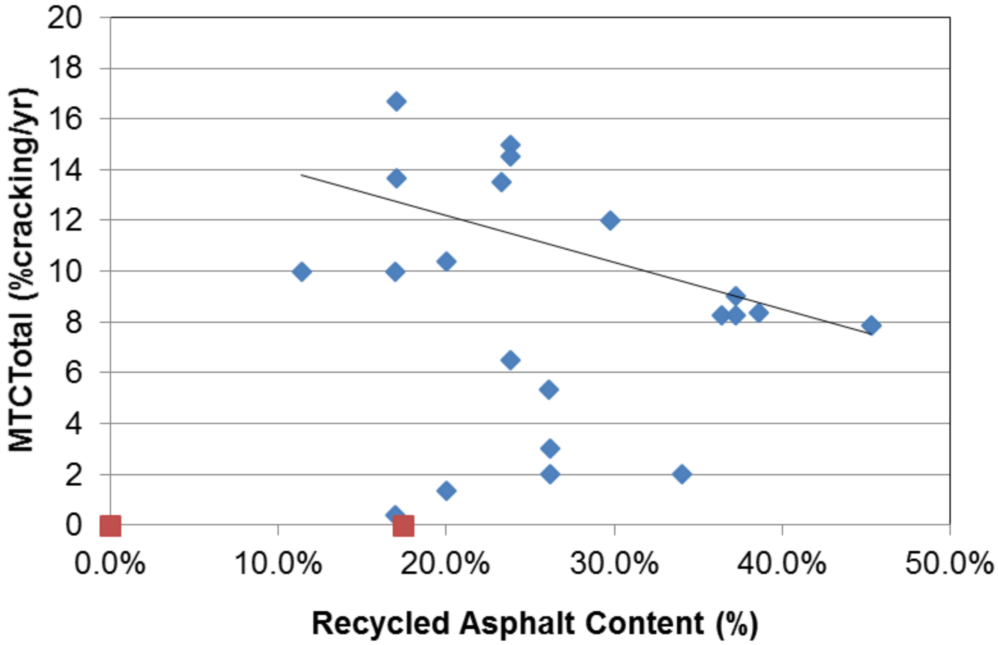

Figure 66: Effect of recycled asphalt content on the maximum total transverse cracking amount (MTCTotal)

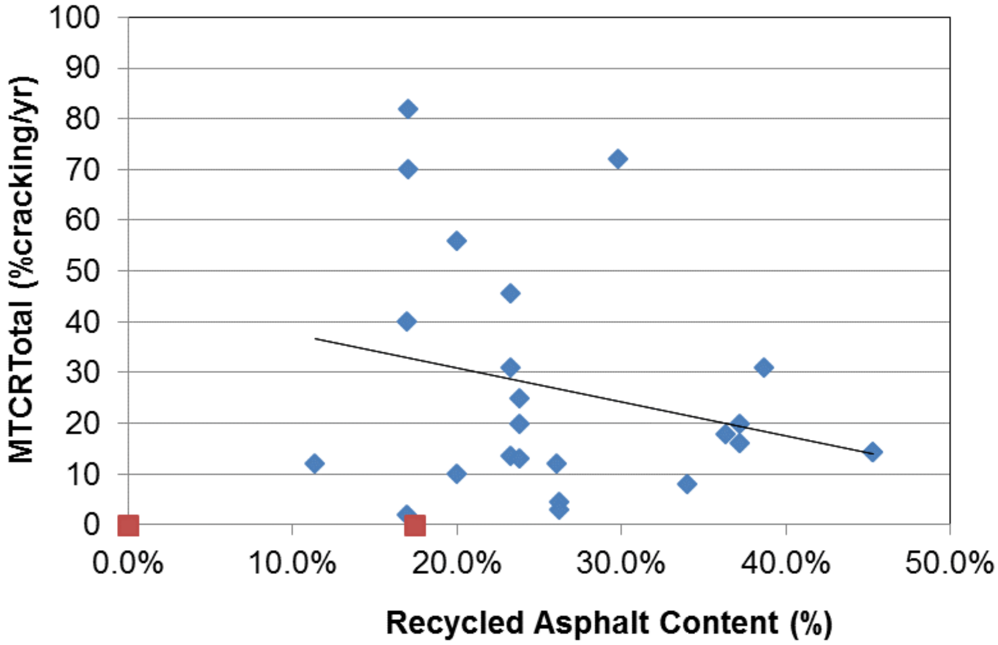

Figure 67: Effect of recycled asphalt content on the maximum total transverse cracking rate (MTCRTotal)

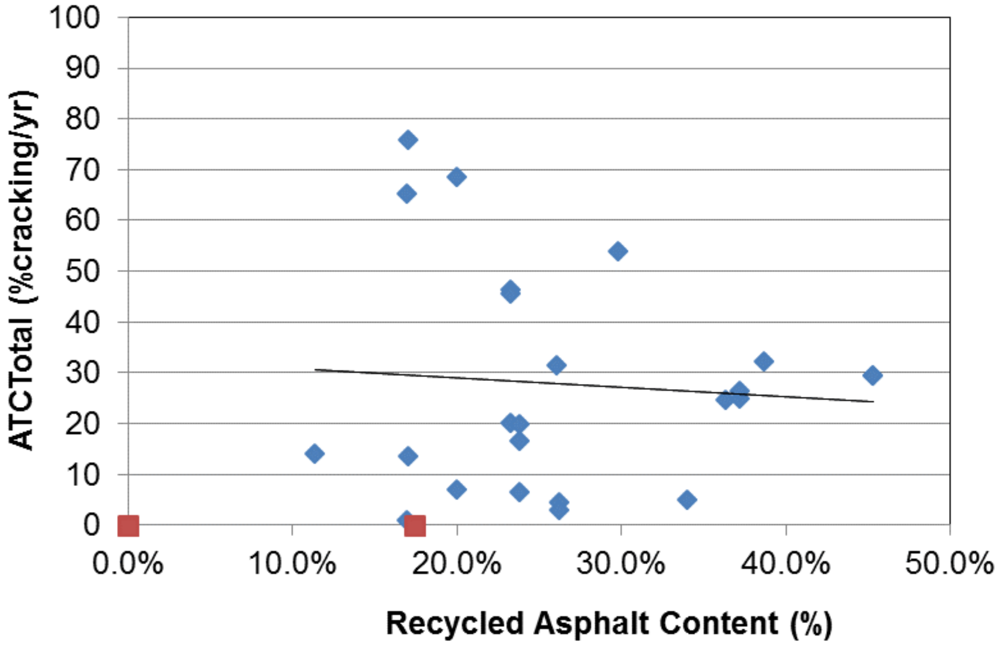

Figure 68: Effect of recycled asphalt content on the average total transverse cracking amount (ATCTotal)

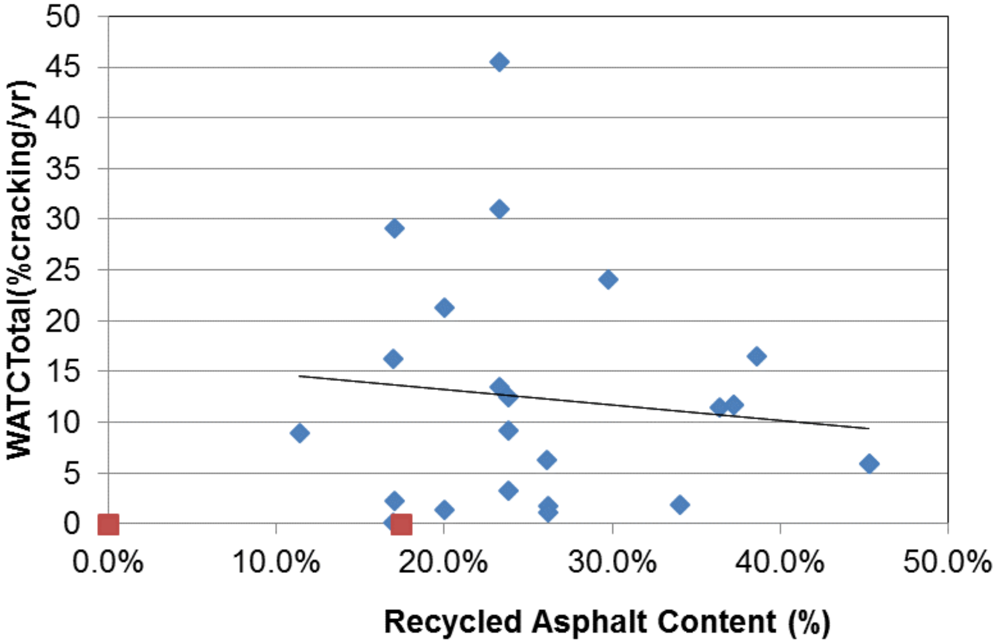

Figure 69: Effect of recycled asphalt content on the weighted average total transverse cracking amount (WATCTotal)

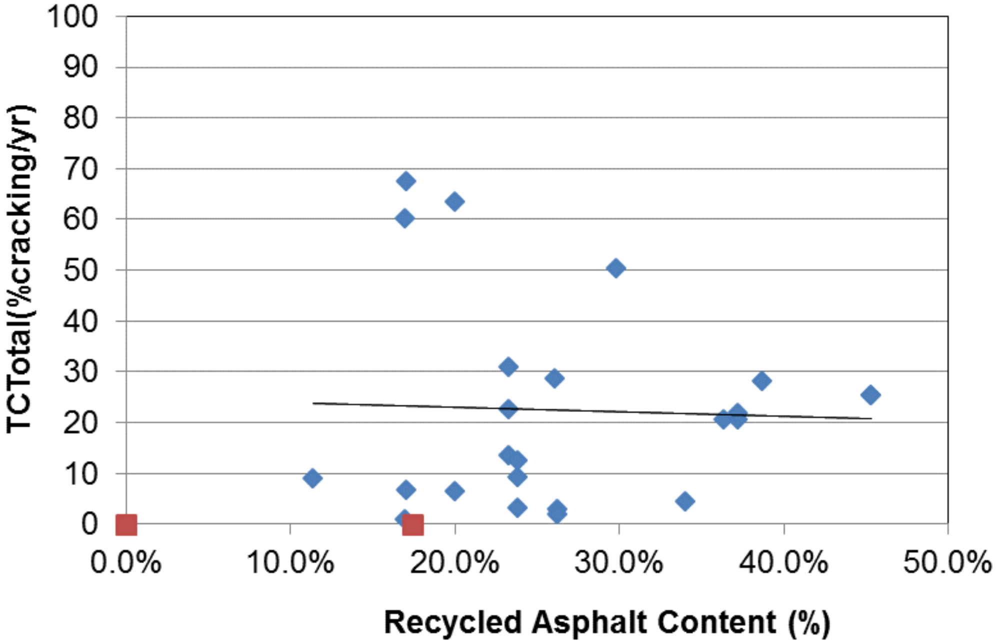

Figure 70: Effect of recycled asphalt content on the total transverse cracking amount (TCTotal)

### *4.4.3 Effect of Voids in Mineral Aggregate (VMA) on Transverse Cracking Performance*

Figure 71 to Figure 75 show the comparison between transverse cracking measures and the voids in mineral aggregate (VMA) of each mix. Contrary to earlier recommendations, the data in this portion of the study does not appear to have any type of trend in relation to VMA and transverse cracking. It should be reiterated that the projects incorporated in this research study were all designed and constructed using two different versions of MnDOT 2360 specifications that required a minimum VMA amount. The newer pavements were constructed with a minimum required asphalt film thickness (AFT). Note that the newer designs utilizing the adjusted asphalt film thickness (AFT) specification have significantly lower VMA amounts. This specification variability may also provide some unknown material discrepancies.

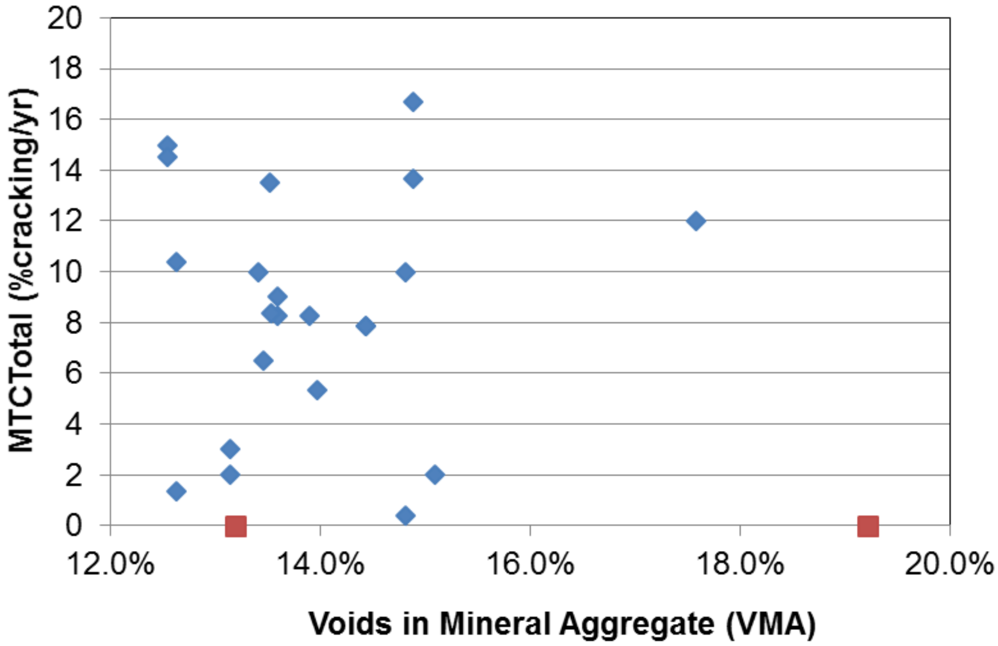

Figure 71: Effect of voids in mineral aggregate (VMA) on the maximum total transverse cracking amount (MTCTotal)

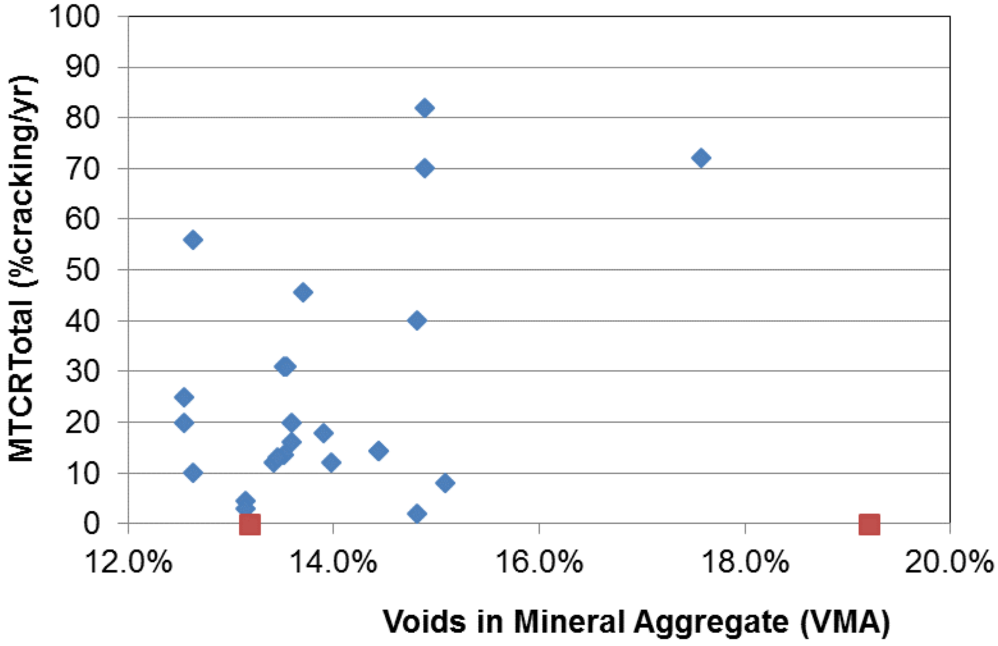

Figure 72: Effect of Voids in Mineral Aggregate (VMA) on the Maximum Total Transverse Cracking Rate (MTCRTotal)

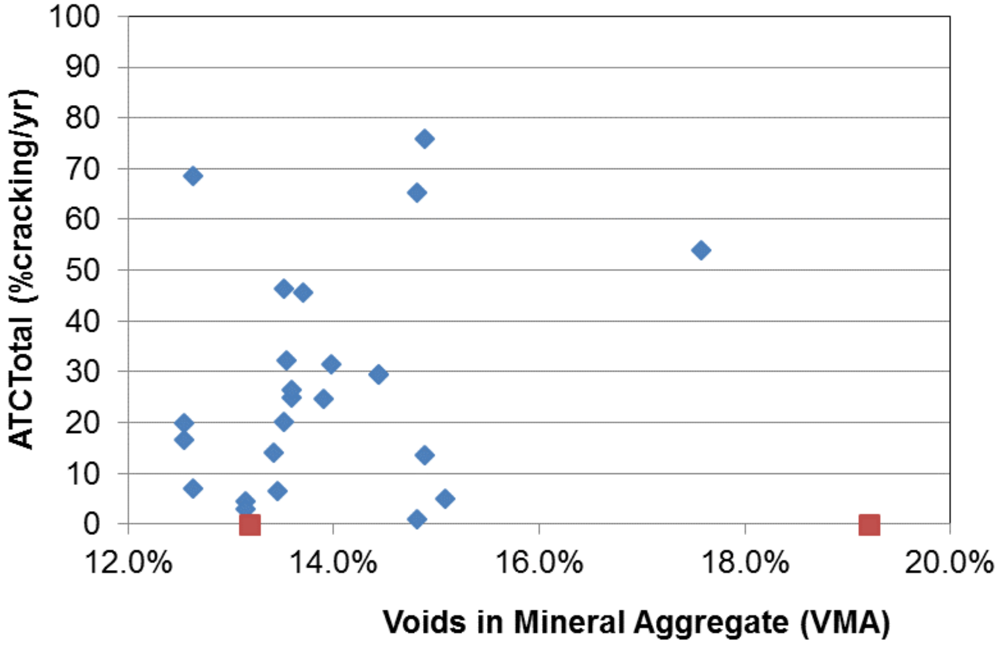

Figure 73: Effect of voids in mineral aggregate (VMA) on the average total transverse cracking amount (ATCTotal)

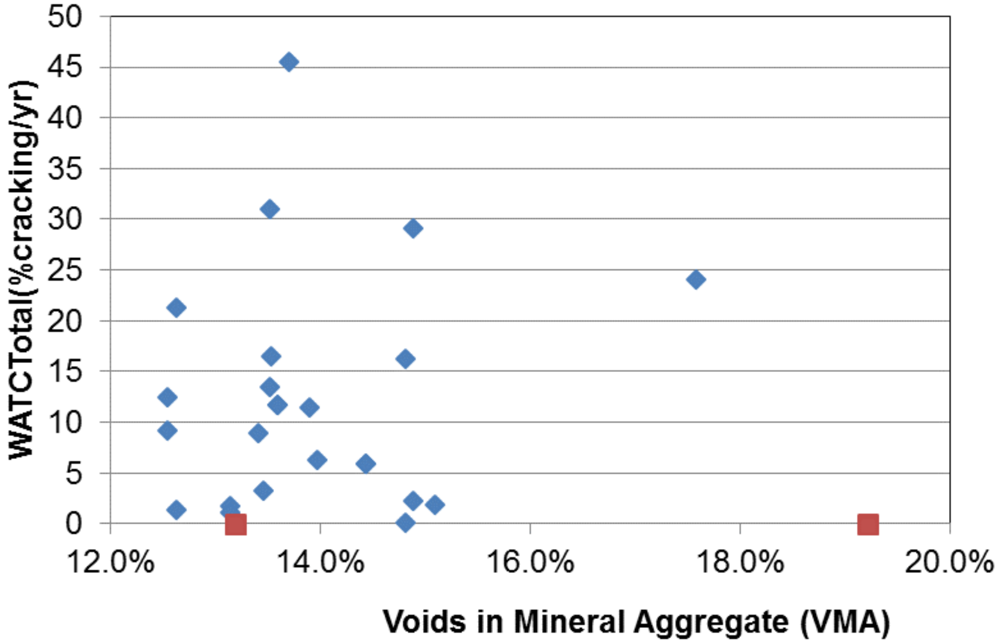

Figure 74: Effect of voids in mineral aggregate (VMA) on the weighted average total transverse cracking amount (WATCTotal)

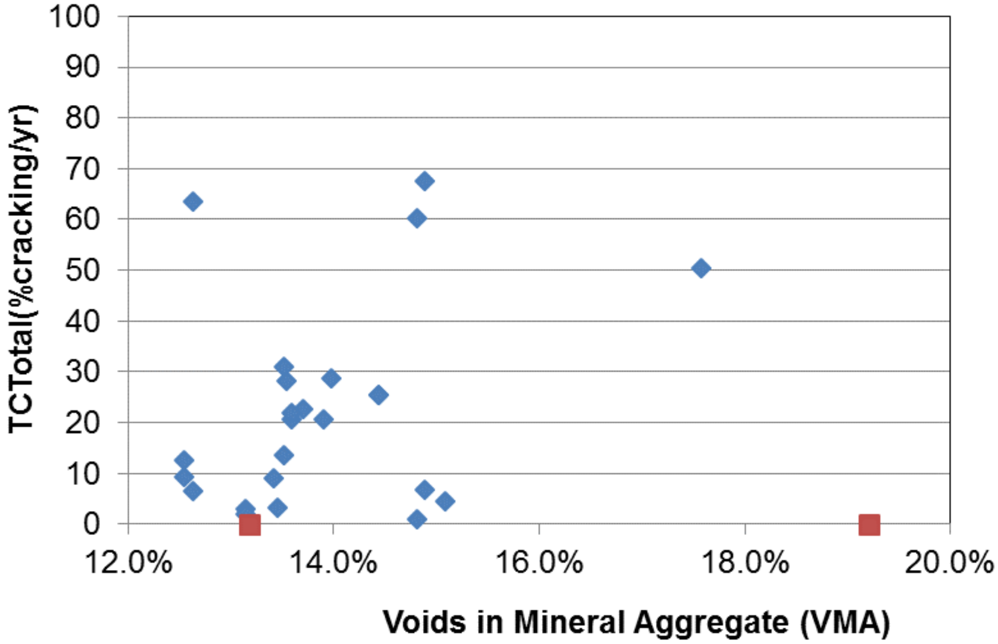

Figure 75: Effect of voids in mineral aggregate (VMA) on the total transverse cracking amount (WATCTotal)

### *4.4.4 Effect of Voids Filled with Asphalt (VFA) on Transverse Cracking Performance*

A single MTCTotal versus VFA plot has been provided in this section (Figure 76), as no unique trends were present in VFA comparisons. The remaining plots that result from the analysis of the voids filled with asphalt (VFA) of each mix can be found in the Appendix. The data in this portion of the study does not appear to have any type of trend in relation to VFA and transverse cracking. While all the mixes in this study do not have the same design traffic level (basis for Superpave VFA recommendations), all of the mixes meet the suggested VFA range for the corresponding traffic level.

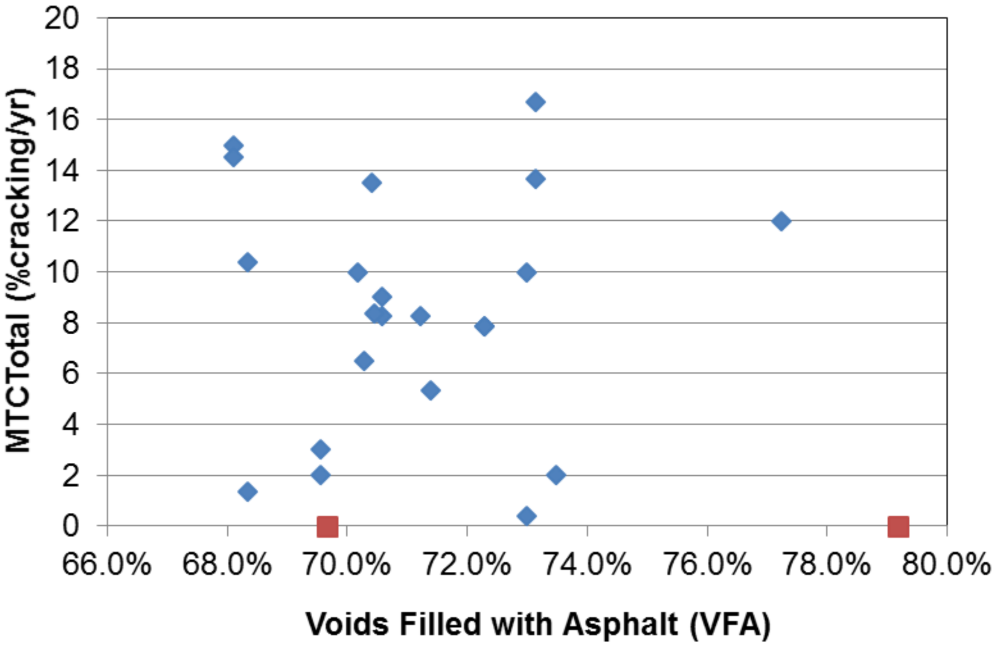

Figure 76: Effect of voids filled with asphalt (VFA) on the maximum total transverse cracking amount (MTCTotal)

### *4.4.5 Effect of Adjusted Asphalt Film Thickness (AFT) on Transverse Cracking Performance*

The adjusted asphalt film thickness (AFT) for various mixes are plotted against the transverse cracking performance measures in Figure 77 through Figure 81. For the mixes designed and produced using the older MnDOT 2360 specifications the adjusted AFT values were calculated using the information from MDRs and the mix test summary sheets (TSS). Note that TH 212 is omitted, as the TSS was not available for this section. Furthermore, SMA mixtures do not correlate well to the calculation of AFT.

Three of the five plots (MTCTotal, MTCRTotal and WATCTotal) indicate a general trend of slightly deteriorating transverse cracking performance with increasing values of adjusted AFT. The other two measures indicate a slight decrease in cracking as AFT increases. As with other parameters, and similar to asphalt content, the data is still prone to significant scatter. The trends indicated here should not be used for the purposes of drawing conclusions.

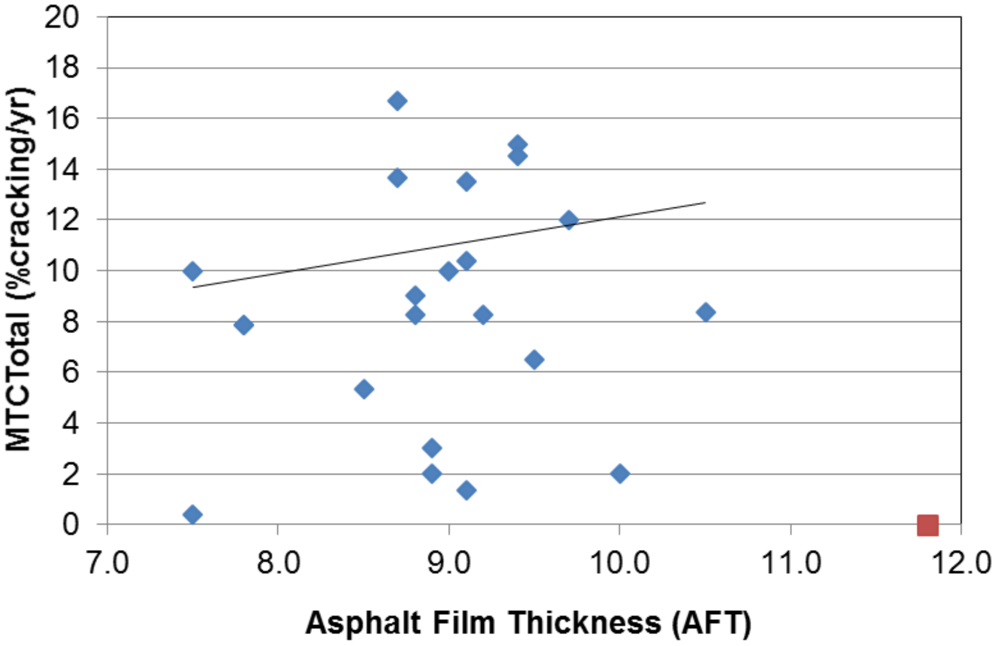

Figure 77: Effect of adjusted asphalt film thickness (AFT) on the maximum total transverse cracking amount (MTCTotal)

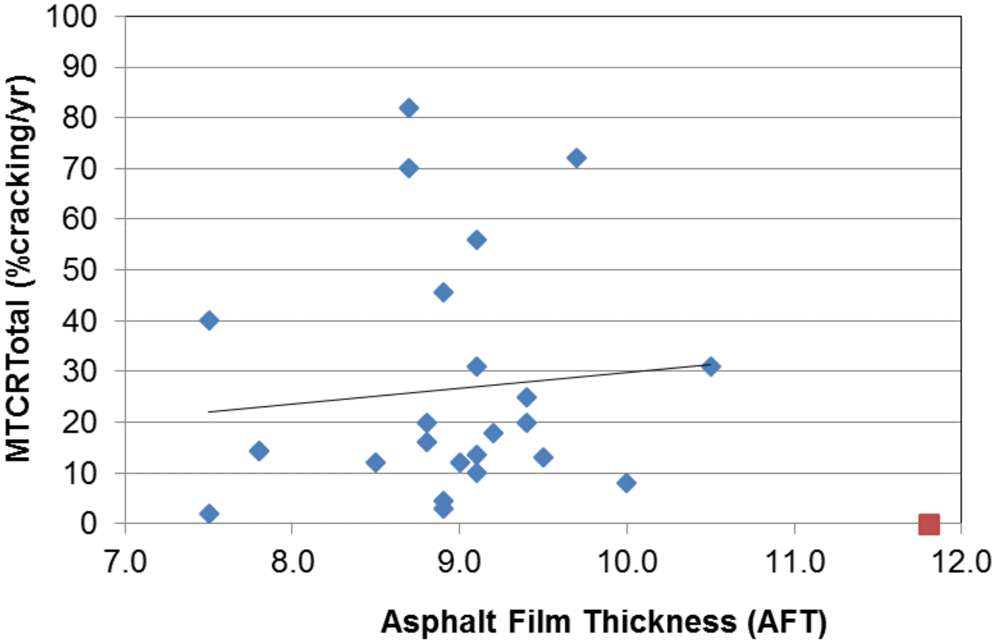

Figure 78: Effect of adjusted asphalt film thickness (AFT) on the maximum total transverse cracking rate (MTCRTotal)

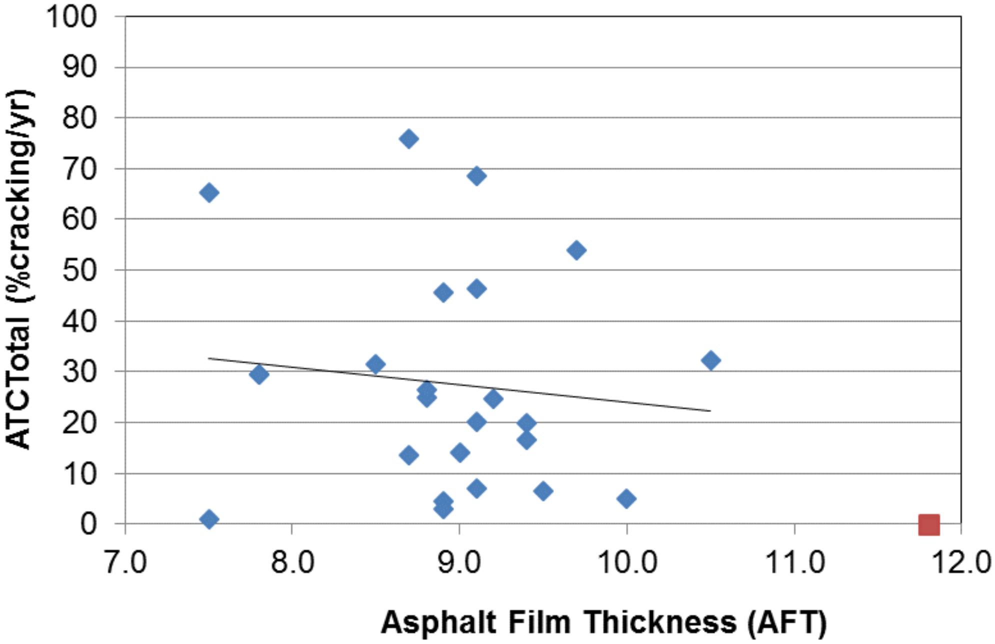

Figure 79: Effect of adjusted asphalt film thickness (AFT) on the average total transverse cracking amount (ATCTotal)

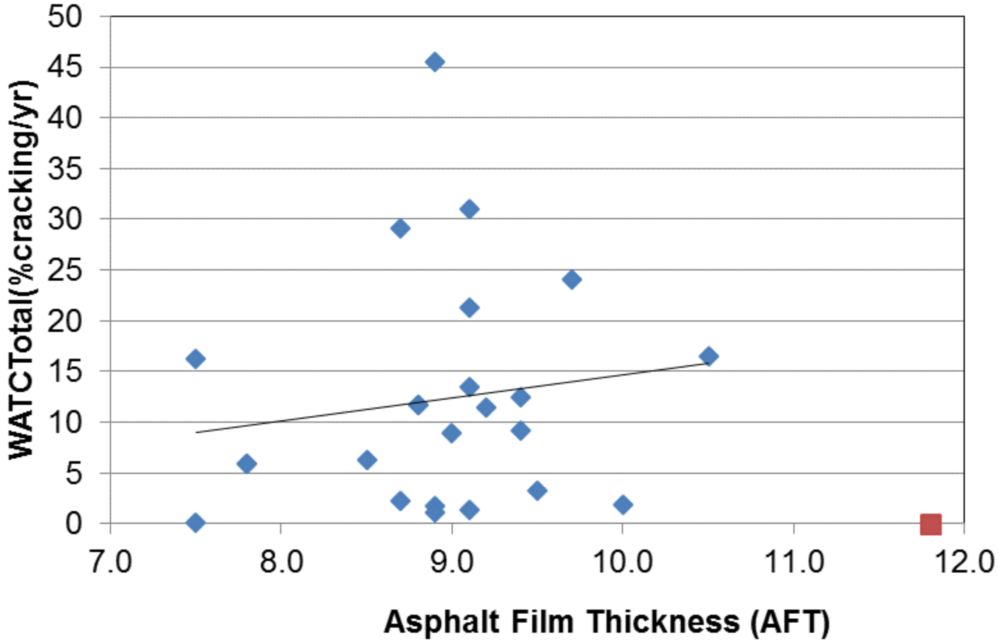

Figure 80: Effect of adjusted asphalt film thickness (AFT) on the weighted average total transverse cracking amount (WATCTotal)

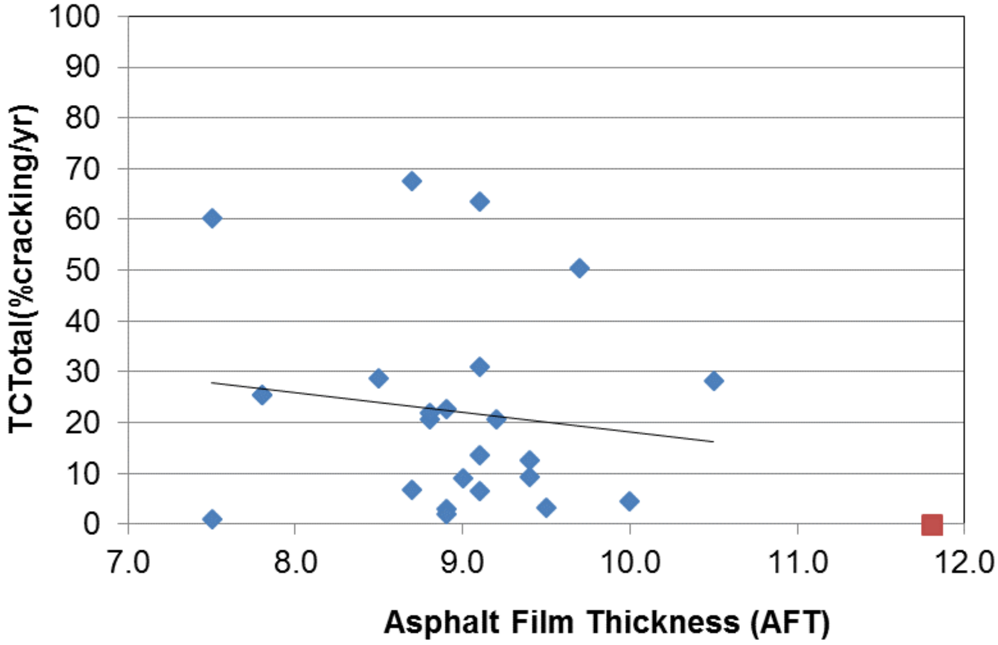

Figure 81: Effect of adjusted asphalt film thickness (AFT) on the total transverse cracking amount (TCTotal)

### *4.4.6 Comparison of Pavement Construction Type with Transverse Cracking Performance*

#### *4.4.6.1 Introduction*

During the analysis of mix design parameters, it became apparent that construction type may have an impact on transverse cracking performance. In this study there were several construction methods used with various asphalt layer depths. It is not practical to attempt and relate each variation to cracking performance. For analysis purposes, three primary construction types were identified: overlays, reclaimed asphalt and new construction. Any section with an asphalt wear course on an existing roadway is herein referred to as an "overlay". Sections with overlay on a reclaimed asphalt layer will be considered a "reclaim". Historical records show no cracking on either of the new construction sections (TH 25 and TH 212). These data points are incorporated in the following plots, but the lack of sufficient data for new construction cracking is recognized.

#### *4.4.6.2 Effect of Mix Parameters on Cracking Performance for Different Construction Types*

Generally, mix design parameters did not show a strong trend relating construction type and transverse cracking performance. Figure 82 through Figure 84 show two parameters that did exhibit a potential relationship between construction type and field performance. In Figure 82 as PG spread increases from 86 to 92, transverse cracking in reclaim sections shows a significant improvement while overlays exhibit less improvement. To further verify this trend, TCTotal was also examined to see if this trend continued. As shown in Figure 83, this exact relationship is also present. Figure 84 shows a looser version of the same trend as asphalt content increases. The preliminary trend in this instance is that it appears to be advantageous to use higher asphalt content or larger PG spread in reclaim sections as opposed to overlays. This trend should continue to be monitored in future studies.

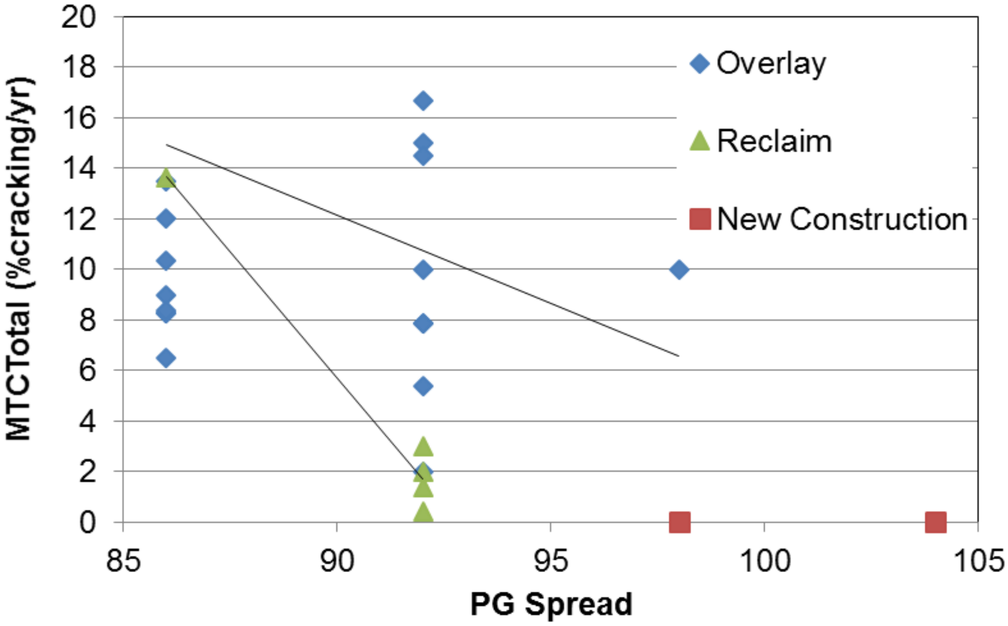

Figure 82: Effect of Performance Grade spread on the maximum total transverse cracking amount (MTCTotal) categorized by construction type

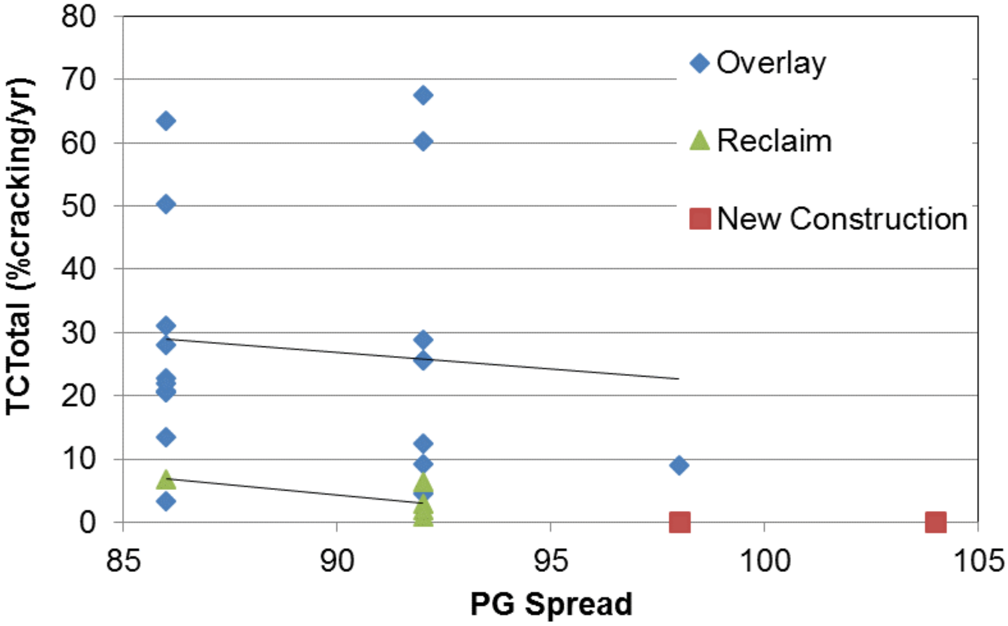

Figure 83: Effect of Performance Grade spread on the total transverse cracking amount (TCTotal) categorized by construction type

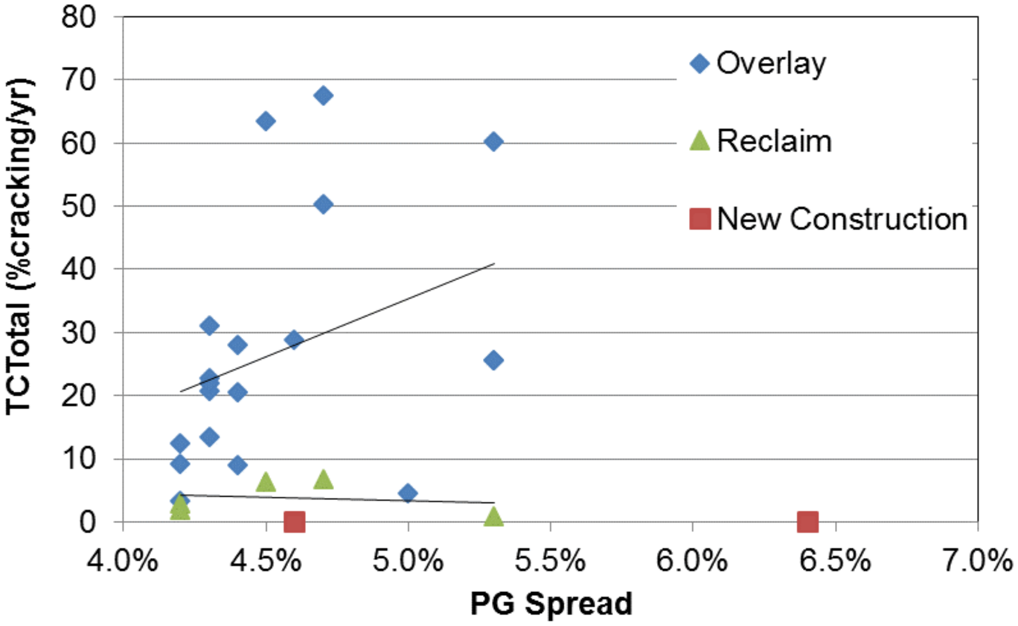

Figure 84: Effect of percentage of asphalt content on the total transverse cracking amount (TCTotal) categorized by construction type

### *4.4.6.3 Effect of Pavement Section Type on Cracking Performance*

Observing the data in Figure 85 thru Figure 89, the construction methods are arranged in the following order: overlay, reclaim and new construction. In these plots, there is a fairly pronounced trend of decreasing transverse cracking as the plot progresses left to right. Figure 86 is best understood viewed with Figure 87. While asphalt reclamation projects appear to result in greater rates of transverse cracking (Figure 86), the average amount of transverse cracking present on a yearly basis for the reclaim sections is significantly lower than overlay projects (Figure 87). In other words, reclaim sections often see a significant increase in the amount of cracking over certain year during their life, but this usually happens later in the service life as opposed to overlays where the high cracking rate occurs early in the service life. For example, comparisons between the overlay and reclaim sections from TH 1 (Figure 90) show that the overlay section experienced

70% cracking in first year of service, whereas, the reclaim section did not experience any significant cracking until year 6.

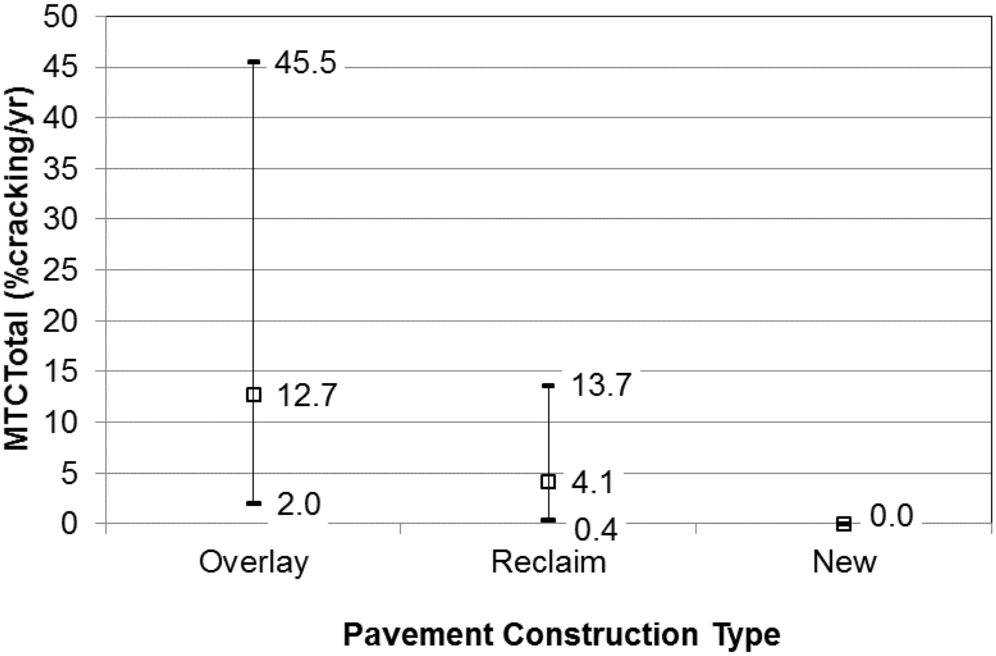

Figure 85: Comparison of maximum total transverse cracking amounts (MTCTotal) between construction types

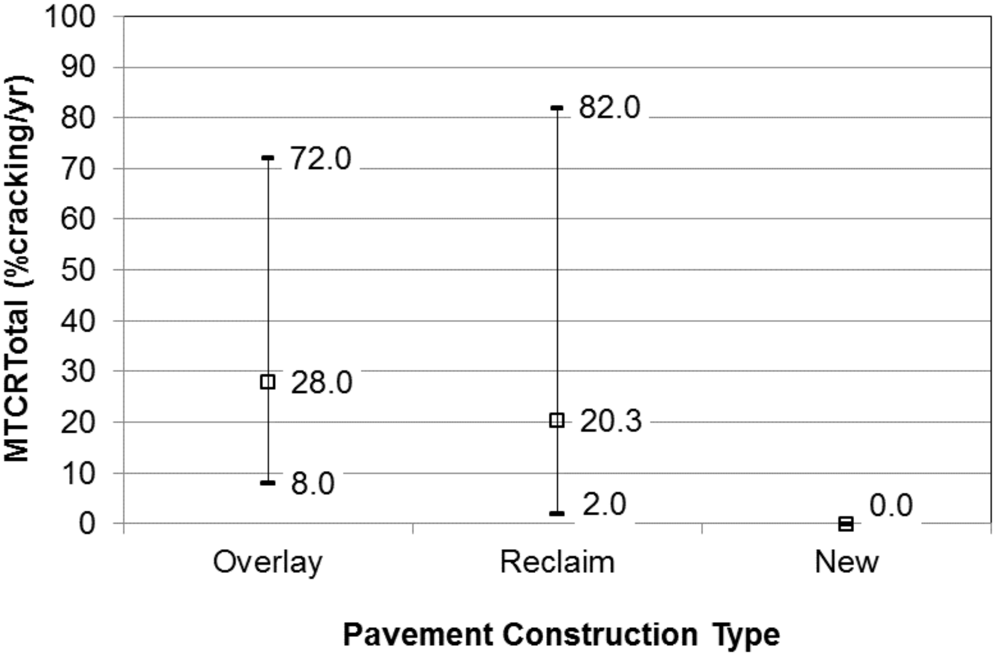

Figure 86: Comparison of maximum total transverse cracking rates (MTCRTotal) between construction types

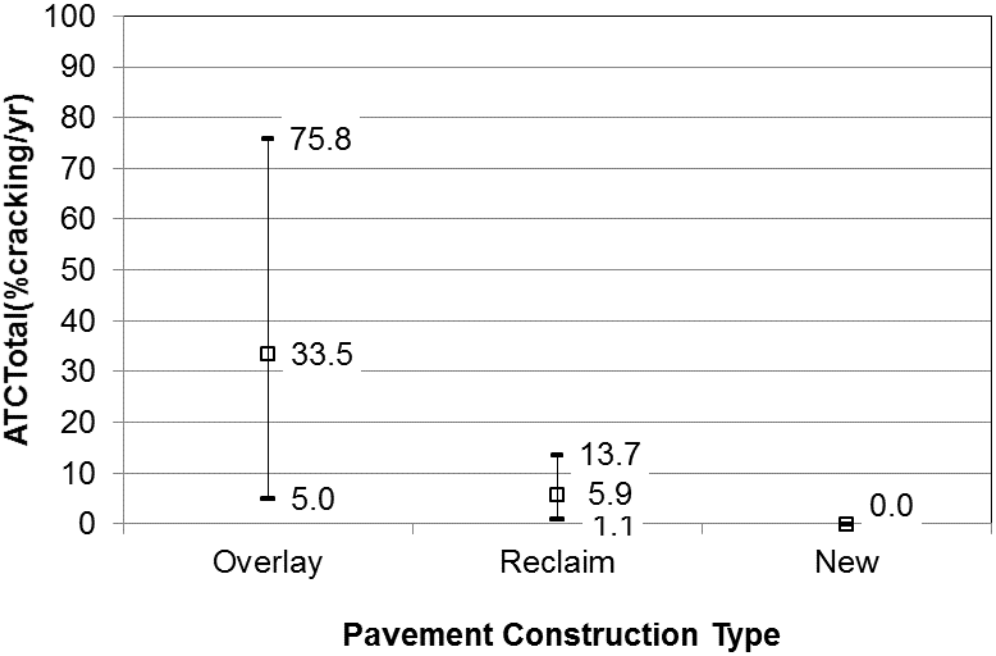

Figure 87: Comparison of average total transverse cracking amounts (ATCTotal) between construction types

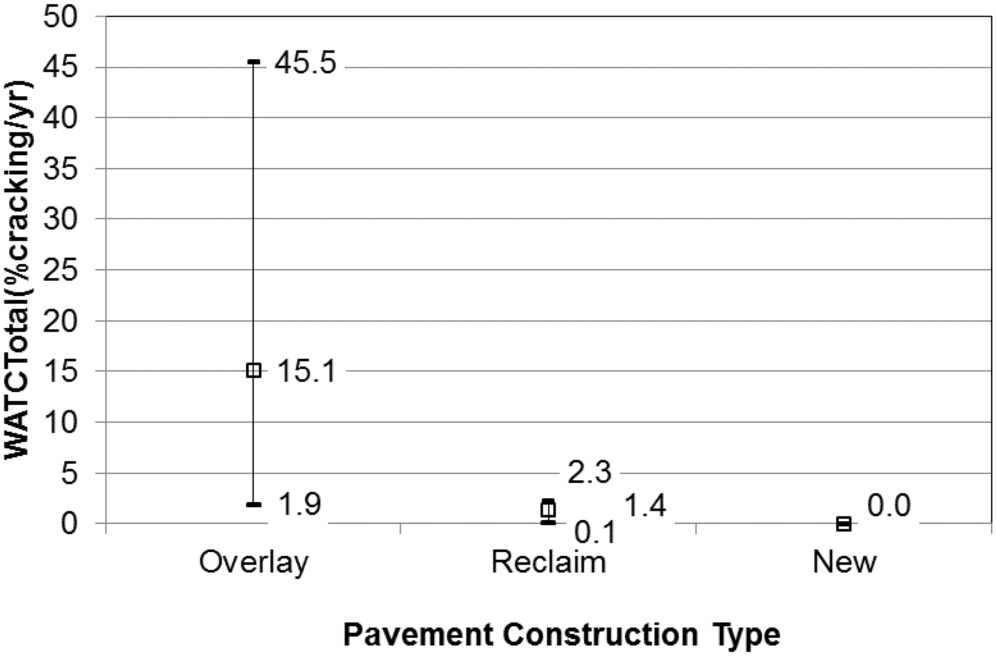

Figure 88: Comparison of weighted average total transverse cracking amounts (WATCTotal) between construction types

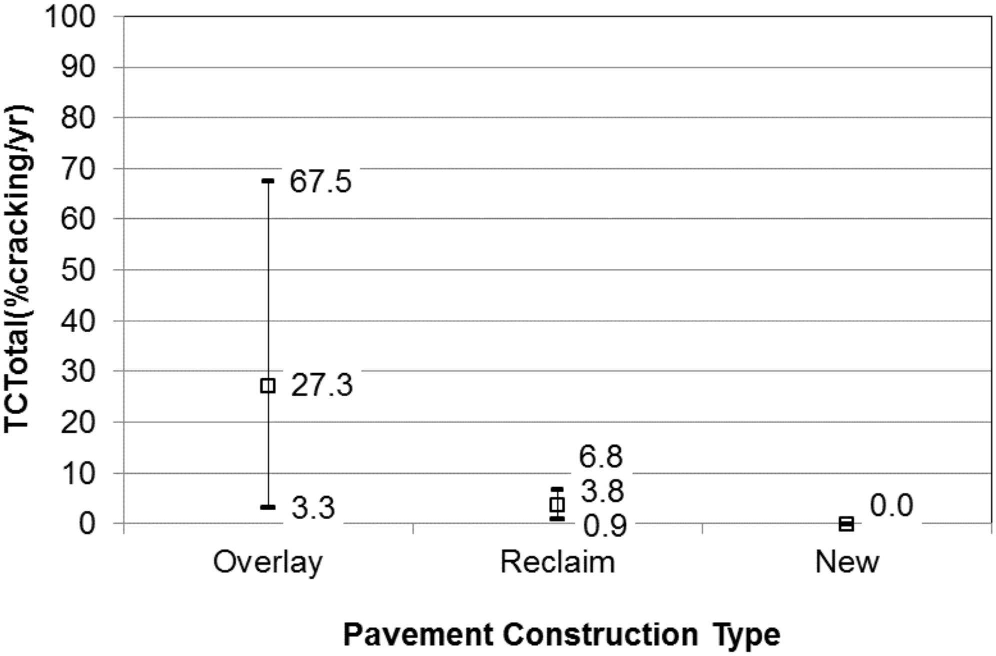

Figure 89: Comparison of total transverse cracking amounts (TCTotal) between construction types

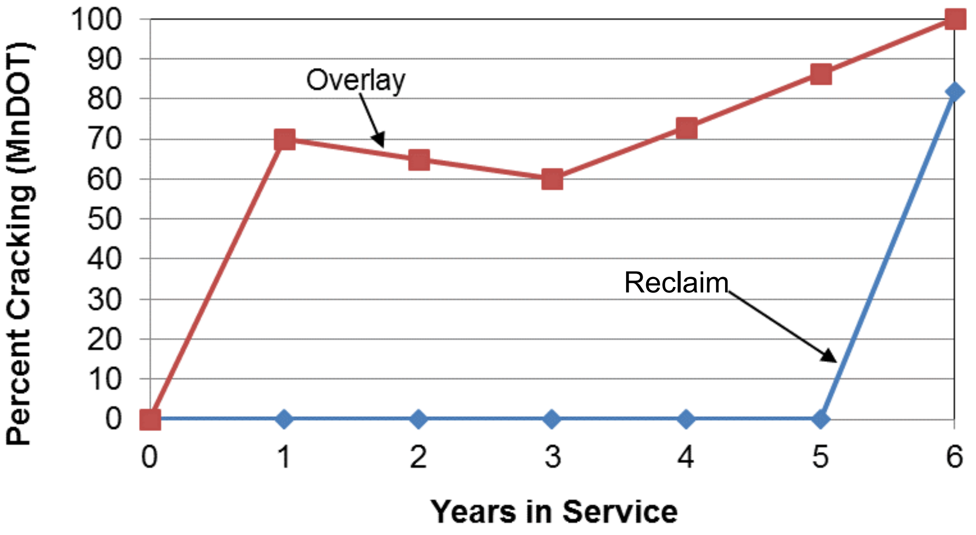

Figure 90: Cracking Performance of TH 1 (SP 8821-103)

In addition, Figure 91 has been provided to reinforce the point that the reclaim sections from this project exhibit a greater resistance to transverse cracking. Figure 91 has been normalized against the asphalt layer thickness. Normalization of asphalt layer thickness is conducted by multiplying the transverse cracking amounts with the total asphalt layer wear course thickness. In a general sense reclaim projects tend to feature a thicker asphalt layer than overlays. After normalizing for the asphalt layer thickness, reclaim sections still exhibit a superior transverse cracking resistance to overlay sections. Thus, even when the added cost of thicker asphalt layer is accounted for in the analysis the cracking performance is still superior with reclaim sections. It should be noted again that this task features a small amount of sections and these trends are not identified with a high level of confidence. These are initial observations and will continue to be observed as more data is acquired.

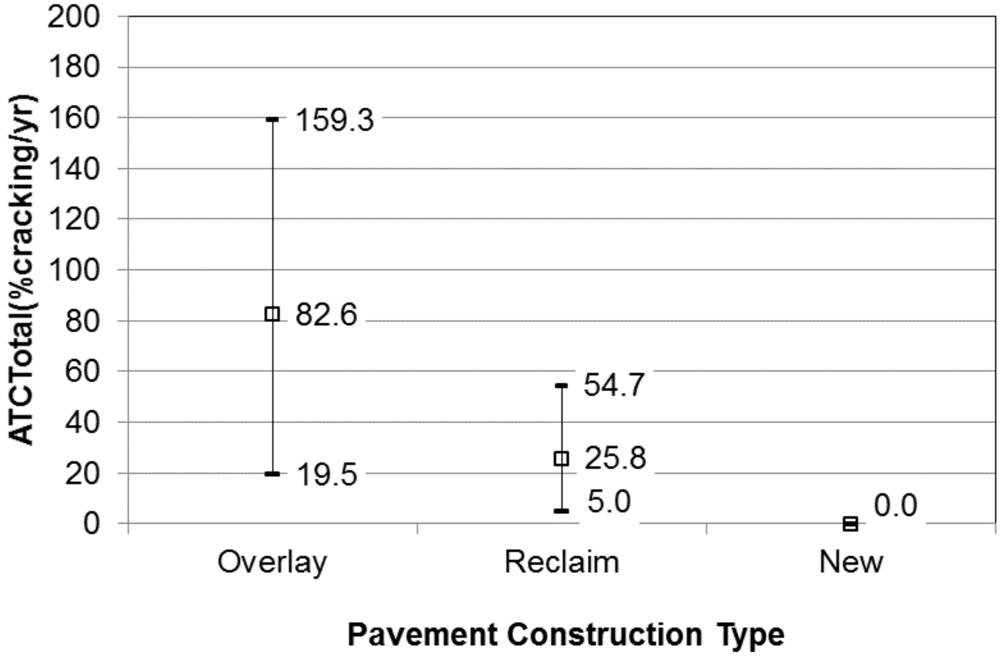

Figure 91: Comparison of average total transverse cracking amounts (ATCTotal) normalized against wear course thickness between construction types

### *4.4.7 Summary of Findings from Comparison of Field Performance and Mix Design Parameters*

This phase of the research project analyzed the field evaluation results of 18 highway projects and 26 pavement sections. These represent five virgin binder types, three pavement section types and two design traffic levels. The focus of this task was to further validate the initial research recommendations and determine if any correlations exist between mix design parameters and transverse cracking performance in the field. The field evaluation results consisted of crack counts conducted by the researchers and historical pavement distress information from the MnDOT Pavement Management System (PMS). The compilation of this data provided researchers with a timeline of transverse cracking performance over the service life of the sections. The analysis of mix design parameters revealed the following correlations and conclusions:

- PG Grade:
    - All performance measures exhibit a loose trend of improved performance as binder grade goes in the order of PG 58-28, PG 58-34, PG 64-28, PG 64-34 and PG 70-34.
    - Note that there are some PG grades with a small number of sections (PG 64-34 consists of two sections and PG 70-34 is a single data point). Therefore additional studies are required before any conclusions can be drawn for these binder grades.
- PG Spread:
    - Transverse cracking amounts from all measures appear to decrease as the spread between high and low binder grade increases.
    - The results are encouraging as trends appear convincing. However, the large amount of scatter and variability between equal PG spreads suggests that it would not be an accurate standalone performance indicator.
- Asphalt Content (%):

    - Performance measures indicate a discrepancy between transverse cracking amounts and binder content, with some measures indicating improvement in performance as content increases and vice versa.
    - A large amount of scatter is present in the plots related to asphalt content indicating other factors are influencing field performance and asphalt content cannot be used as an independent performance measure.
- Recycled Asphalt Content (%):
    - A slight downward trend in cracking exists as amount of recycled asphalt content increases. However, four pavements exhibit near zero transverse cracking with less than 20% recycled asphalt content.
    - A large amount of variability exists with the presence of recycled asphalt pavement (RAP) or recycled asphalt shingles (RAS)
        - Type and age of RAP and RAS
        - Original grade of binder in recycled products
    - This issue reiterates the need for using a laboratory testing based performance measure, namely DCT fracture energy.
- Voids in Mineral Aggregate (VMA)
    - No consistent trends are present in any plots.
    - Two different specifications were used on these projects. Older projects required a minimum VMA amount, while newer construction utilized a minimum AFT.
- Voids Filled with Asphalt (VFA):
    - Does not appear to have any type of trend in relation to transverse cracking.
    - All mixes do not have same traffic level, but all meet suggested VFA ranges.

- Adjusted Asphalt Film Thickness (AFT):
    - Three plots indicate a trend of slightly deteriorating transverse cracking performance as adjusted AFT increases.
    - Also features significant amount of scatter, suggesting other factors influence transverse cracking amounts.
- Normalization for traffic level and asphalt layer thickness had minimal impact on correlations between transverse cracking and mix parameters.

Overall, mix design parameters exhibited a large amount of scatter when plotted against the five performance measures. This indicates that the parameters would not be strong independent performance indicators for transverse cracking. Of all the parameters analyzed, PG grade, PG spread and binder content appear to have the strongest correlation to transverse cracking performance.

Analysis of construction type versus transverse cracking amounts yielded intriguing results. When observing mix parameters arranged by construction type against cracking performance, two parameters showed a correlation: PG spread and asphalt binder content. The following relationships were observed:

- As PG spread increases, reclaim projects experience significantly better transverse cracking resistance as compared to overlays.
- As asphalt binder content increases, reclaim projects exhibit greater transverse cracking resistance as compared to overlays.

In general, the results indicate that the use of mix design parameters as an independent transverse cracking performance predictor is not recommended. These findings are not entirely surprising. Earlier studies showed that some parameters have potential to be performance predictors but none showed a very strong correlation. The findings from this phase strongly support the recommendation of using a laboratory testing based performance parameter. The findings also reinforce the need for using superior asphalt binder grade in the reclaim sections. Finally, it should be noted that the conclusions regarding the mix parameters in context of different pavement section types (reclaim versus mill and overlay) should be treated preliminary as the number of sections were limited.

# CHAPTER 5: SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

## 5.1 Summary

This research project focused on the field evaluation, laboratory testing and analysis of mix design parameters for eighteen highway sections. Due to extenuating circumstances related to acquisition of field samples, nine highways were not available for laboratory testing. Initially, all highway sections were visited and twenty-six pavement study sections were identified. The locations for obtaining cored samples for performance testing were also identified at this time. Five cracking performance measures were used during the analysis process (MTCTotal, MTCRTotal, ATCTotal, WATCTotal and TCTotal). The five performance measures were utilized to quantify the cracking performance of pavement sections. This phase also involved acquiring mix design records (MDRs) and pavement management system (PMS) data from the Minnesota Department of Transportation (MnDOT). MDR data was used to correlate mix constitution related parameters (asphalt binder amount, binder type, design traffic level etc.) of interest to roadway performance. The PMS data provided historical cracking records for each roadway, which allowed the use of the cracking performance measures.

The second phase of this research project consisted of conducting the disk-shaped compact tension (DCT) testing on nine roadways with a total of 13 different sections. Evaluation of the fracture energy differences between sections from the same roadway that were assigned a "poor" or "good" performance designation resulted in the poor performers exhibiting a lower fracture energy than the good performing sections. Roadways that featured a single study section were also tested and the results reported. Minimal comparisons for these sections were conducted during this phase due to lack of "fair" comparison possibility in absence of companion section.

The final phase of the study focused on the comparison of field performance, mix testing parameters and performance testing results. The data was compared to field performance using various transverse cracking measures. This data is presented in several fashions, considering both traffic level of the section and pavement construction type. The analysis of this data resulted in several conclusions and recommendations for current and future research.

## 5.2 Assumptions and Implications

Several assumptions were made in this research in order to make the process feasible. Each parameter leads to particular limitations within the research process. The use of field cored specimens posed some unique challenges within the research process. The major assumptions and corresponding implications are listed as follows:

- Climatic conditions
    - The discrete behaviors that take place at each site are unquantifiable. Levels of precipitation, rates of cooling and number of freeze-thaw cycles are just three of the many potential factors that can impact the low temperature cracking performance. The normalization of service life was an attempt to mitigate the effects of the climatic conditions. However, each site will see various freezing cycles and this number could differ greatly in comparison between two locations. This is an inherent issue with field procured samples and one that is not easily remedied. This factor will inevitably lead to some of the variability seen in field sample performance testing.
- Binder aging
    - The influence due to binder aging is difficult to quantify. Binder age may not be necessarily related to service life as the chemical composition of binders vary significantly and can play a major role in the oxidative aging process. Typically an older binder can lead to a much more brittle material

    than that of a new binder. This will impact the results of any performance testing conducted on the samples. As with climate, it is impractical to characterize it within the realm of present study. The issues with these should be considered in any future studies involving field procured specimens. Natural variability will occur with any field samples due to this factor.

- Material specifications
    - This research project involved construction using two significantly different types of material specifications. The first, a specification requiring a minimum void in mineral aggregate (VMA) amount was in practice until mid-2000s, and the second, a minimum adjusted asphalt film thickness (AFT) specification has been adopted by MnDOT. Based on the analysis in this project, the AFT specification generally leads to a lower VMA. The reasoning for this is unclear, but it suggests that these mixes may have underlying differences in the final product. This could be related to many factors of the aggregates being used in this mixes (different sources have varying properties). This may or may not have any bearing on results. If comparing results between different specifications, the analyst should understand that additional variability should be expected.
- Construction quality
    - As with any field procured samples, the final product that is tested is in direct relation to the construction quality. An identical mix on an identical project could have completely different performance testing results. This

could be due to the specimens coming from different days of construction. As a result, the project could have different individuals constructing the roadway on that day. This leads to variability in worker experience and attention to the project. Clearly, these are inherent risks when testing field samples and should lead to expectations for additional variability when testing field acquired samples.

### 5.3 Conclusions

The findings from this study during the detailed analysis of the data resulted in several conclusions on the effect of mix design parameters on both field performance and DCT testing results. While lot of trends were observed between various mix constitution parameters and field cracking performance as well as between the measured DCT fracture energy of field samples and field cracking performances the trends were not discernable enough to develop definitive relationships. The results presented herein are for the purposes of further validating the need of a performance test along with guiding future research efforts. The key conclusions drawn from this study are as follows:

- All five transverse cracking performance measures were found to adequately quantify the actual cracking performance. The measures are best utilized in a group, as each measure values cracking amounts differently. Therefore, a parameter of interest can be validated for all potential cracking performance concerns.
- Field performance correlated well with fracture energy results. As fracture energy increased, transverse cracking generally decreased.

- PG grade had a slight correlation to both higher fracture energy and higher transverse cracking amounts as the performance grade of the binder progressed in the order of PG 58-28, PG 58-34, PG 64-28, PG 64-34 and PG 70-34.
- As PG spread increased, it appeared to correlate with a higher fracture energy and lower transverse cracking amounts.
- Asphalt content showed a general increase in fracture energy as the amount of binder increased. However, binder content was inconclusive in comparison to field performance.
- Asphalt film thickness did not feature a strong trend using any of the measures.
- All other mix parameters showed minimal to no correlation with laboratory testing.
- Normalization of results for traffic levels illustrated that there does not appear to be a substantial impact by traffic on transverse cracking amounts.
- TH 212 (the sole SMA mixture in the study) exhibited a transverse cracking level of zero over a six year service life, with an average fracture energy of 1,040 J/m$^2$. This is by far the best performer in this study and a notable finding.
- The asphalt layers on reclaimed sections show lower amount of cracking and delayed cracking as compared to mill and overlay sections on the same stretches of highways. As PG spread increases, reclaim projects experience significantly better transverse cracking resistance as compared to overlays. The same trend exists when asphalt binder increases.
- The DCT fracture energy test continues to show promise as a practical and reliable procedure for screening good and poor performing asphalt mixtures from perspective of transverse cracking.

## 5.4 Recommendations

The findings from this study have shown many reasons why a performance test is vital to the accurate prediction of transverse cracking performance. It is clear that a single mix parameter is not sufficient for the prediction of field performance or laboratory results. The potential variability with each parameter is too great to accurately project the cracking performance. The following recommendations are key aspects determined during the course of this research:

- Further testing must be conducted on mixes containing PG 64-34 and PG 70-34 binder grades. When used in a BAB new construction project, both of these binder types have exhibited zero transverse cracking over their respective service life. PG 64-34 was used in a mill and overlay section and has shown a fair amount of cracking (roughly 14% over two years). However, the lack of information on these projects makes it difficult to develop any definitive conclusions. The projects are also unique conditions (only new construction) skewing their results from the pool of data. The testing of mill and overlay sections containing these binder grades is highly recommended.
    - Trunk Highway 212 was the only stone matrix asphalt (SMA) mix in the pool of study sections. It would also be beneficial to further evaluate the impact SMA mixes have on fracture energy performance.
- Binder modification (i.e. polymer modification) can potentially impact the transverse cracking performance of a mixture. However, the ability to track these parameters has not been accounted for in the past. As a result, any polymer modified binders in this study are unknown. The current Superpave specifications were designed primarily using neat, or unmodified, binders. A polymer modified binder can have a significantly different performance than that of a standard neat binder [28]. The testing results and field performance for a polymer modified mixture could have significantly different results than

an identical mixture with a neat binder. The ability to track the presence of a modified binder, as well as the type of modification, would allow for the analysis of this variable. The exact impact is unknown, but tracking the modification would decrease some potential variability in future studies.

- The key focus of this research project was to attempt and validate laboratory testing for the purposes of predicting field performance. A major challenge throughout this study was the coordination of efforts to obtain field samples. It required extensive amounts of time and funding. The analysis of the results proved, while many preliminary trends exist, the lack of data did not provide statistically significant results. The expenses and time associated with performing testing on field samples is not practical for work on a large scale. The research conducted here was necessary to validate that fracture energy results are applicable to field cracking, but the process must change for any relationships to be truly confirmed. The use of IlliTC and other simulation models is recommended for this purpose. This would allow for a number of simulations to be conducted, allowing research teams to potentially develop links between field cracking, mixture parameters and laboratory performance results.

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## APPENDIX A: SECTION SUMMARIES FROM FIELD VISITS

### *Trunk Highway 1 (SP 8821-103)*

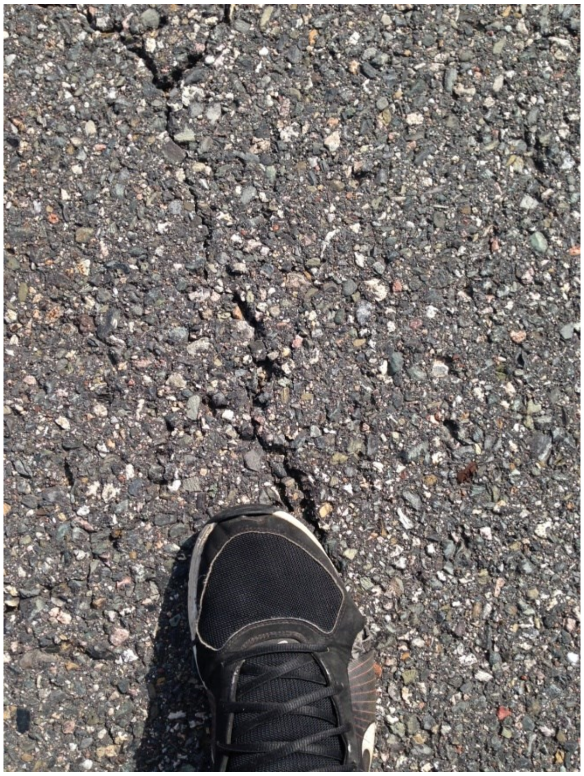
- Location: Northern Minnesota west of Cook
- Construction Year: 2008
- Construction Type(s):
  - 1-1/2 inch overlay on old asphalt concrete (poor performer)
  - 4 inch overlay on reclaimed asphalt concrete (good performer)
- Section Length: Nearly 21 miles
- Site Notes (poor performer):
  - Section Start: RP 235
  - 1-1/2 inch overlay on old asphalt
  - Very poor ride
  - Appears to have extensive amount of high severity cracking
  - Large amount of wheel path cracking
  - Alligator cracking prevalent
  - Severe centerline joint segregation
  - Areas of overlay have completely failed
  - Mix appears to be very dry

TH 1 poor performer-section start

TH 1 poor performer-overview

TH 1 poor performer-surface profile

TH 1 poor performer-typical crack configuration



TH 1 poor performer-typical crack profile

TH 1 poor performer-overlay failure

- Site Notes (good performer):
    - Section Start: RP 230
    - 4 inch overlay on reclaimed asphalt concrete
    - Reclaimed section exhibits much smoother ride
    - Mix appears very dry
    - Significant amount of alligator cracking
    - Centerline joint segregation

TH 1 good performer-section start

TH 1 good performer-overview

TH 1 good performer-surface profile

TH 1 good performer-typical crack configuration

TH 1 good performer-typical crack profile

***Trunk Highway 2 (SP 1102-59)***

- Location: Northern Minnesota stretching through Bena
- Construction Year: 2003
- Construction Type(s):
  - 4 inch overlay on old asphalt concrete
  - Section Start: RP 157
- Section Length: Approximately 16 miles
- Site Notes:
  - Dry mix with a large amount of distributed cracking
  - Substantial fatigue and alligator cracking in wheel path
  - High amounts of medium to low severity transverse cracks
  - Centerline joint cracking throughout
  - Shoulder cracked both longitudinally and transversely throughout
  - Mix looks similar to TH 113

TH 2-section start

TH 2-overview

TH 2-surface profile

TH 2-typical crack configuration

TH 2-typical crack profile

***Trunk Highway 6 (SP 1103-25)***

- Location: Spans between Remer and Outing
- Construction Year: 2010

- Construction Type(s):
    - 1-1/2 inch mill and overlay
    - Section Start: RP 53
- Section Length: 17.33 miles
- Site Notes:
    - Ride is generally smooth with a little uniform roughness due to thermal cracking
    - Majority of cracks have been sealed
    - Same construction type throughout project
    - Section has large amount of incline changes throughout

TH 6-section start

TH 6-overview

TH 6-surface profile

TH 6-typical crack configuration

TH 6-typical crack profile

***Trunk Highway 6 (SP 3107-42)***

- Location: North from Talmoon to the junction at TH 1
- Construction Year: 2004
- Construction Type(s):
  - 1-1/2 inch overlay on old asphalt concrete (poor performer)
  - 4-1/2 inch overlay on reclaimed asphalt concrete (good performer)
- Section Length: Nearly 19 miles
- Site Notes (Poor Performer):
  - Section Start: RP 118
  - 1-1/2 inch overlay on old asphalt concrete
  - Approximately 100 cracks per 1000 feet

TH 6 poor performer-section start

TH 6 poor performer-overview

TH 6 poor performer-surface profile

TH 6 poor performer-typical crack configuration

TH 6 poor performer-typical crack profile

- Site Notes (Good Performer):
    - Section Start: RP 123
    - 4-1/2 inch overlay on reclaimed asphalt concrete
    - Approximately 5 to 8 cracks per mile

TH 6 good performer-section start

TH 6 good performer-overview

TH 6 good performer-surface profile

TH 6 good performer: typical crack configuration

TH 6 good performer-typical crack profile

***Trunk Highway 9 (SP 6010-26)***

- Location: South of Crookston to Beltrami
- Construction Year: 2011
- Construction Type(s):
  - 3 inch mill and overlay on reclaimed asphalt concrete (good and poor performers)
- Section Length: Roughly 18 miles
- Site Notes (Poor Performer):
  - Section Start: RP 208
  - Approximately 15 cracks per mile
- Site Notes (Good Performer):
  - Section Start: RP 214
  - Approximately 11 cracks per mile
  - Smoother ride than RP 208 section

***County State Aid Highway 10 (SAP 031-610-016)***

- Location: South of Bovey to Warba
- Construction Year: 2012
- Construction Type(s):
    - 1-1/2 inch overlay on old asphalt concrete (poor performer)
    - 3 inch mill and overlay (good performer)
- Section Length: Nearly 14.5 miles
- Site Notes (Poor Performer):
    - Section Start: JCT 445B sign
    - Visually more cracking than good performer
    - Centerline joint segregation

CSAH 10 poor performer-section start

CSAH 10 poor performer-overview

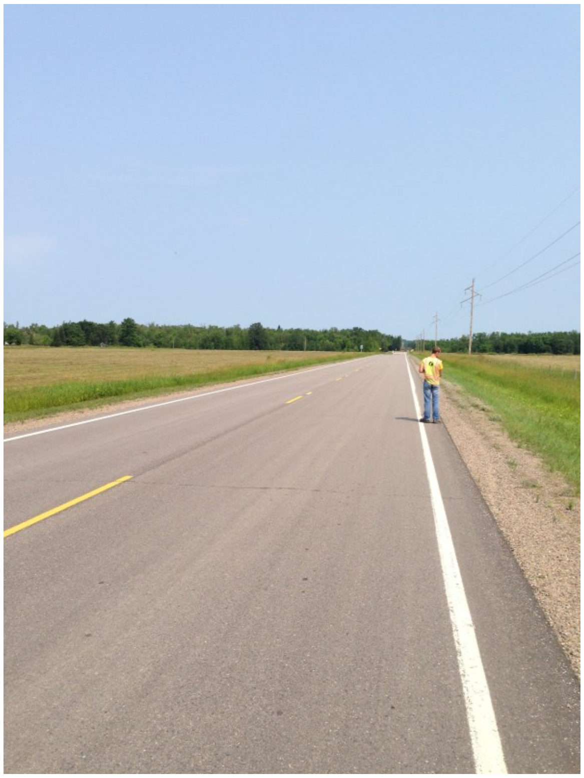
CSAH 10 poor performer-surface profile

CSAH 10 poor performer: typical crack configuration

CSAH 10 poor performer-typical crack profile

- Site Notes (Good Performer):
    - Section Start: JCT 446 sign
    - Smooth ride
    - Centerline joint segregation

CSAH 10 good performer-section start



CSAH 10 good performer-overview

CSAH 10 good performer-surface profile

CSAH 10 good performer: typical crack configuration

CSAH 10 good performer-typical crack profile

***Trunk Highway 10 (SP 0502-95)***

- Location: South of Little Falls, just outside Sartell
- Construction Year: 2005
- Construction Type(s):
    - 4 inch mill and overlay (good and poor performers)
    - Placed in two lifts 1-1/2 inch and 2-1/2 inch
    - Same mixture for both lifts
- Section Length: Slightly over 13 miles
- Site Notes (Poor Performer):
    - Section Start: RP 159
    - Cracks recently sealed
    - Inferior ride to RP 161

TH 10 poor performer-overview

TH 10 poor performer-typical crack configuration

TH 10 poor performer-typical crack profile and surface profile

- Site Notes (Good Performer):
  - Section Start: RP 161
  - Cracks are not sealed
  - Rides better than RP 159

TH 10 good performer-overview

TH 10 good performer-typical crack configuration

TH 10 good performer-typical crack profile and surface profile

***Trunk Highway 10 (SP 5606-42)***

- Location: Spans through New York Mills
- Construction Year: 2013
- Construction Type(s):
    - 3-1/2 inch mill and overlay
- Section Length: Roughly 7 miles
- Site Notes:
    - Section Start: RP 75
    - Extensive shoulder cracking both longitudinal and transverse
    - Good ride quality
    - Centerline joint segregation apparent throughout most of section

TH 10-section start

TH 10-overview

TH 10-surface profile

TH 10-typical crack configuration

TH 10-typical crack profile

***Trunk Highway 25 (SP 7104-19)***

- Location: Between Monticello and Big Lake
- Construction Year: 2011
- Construction Type(s):
  - New construction-bituminous on aggregate base (BAB)
- Section Length: Nearly 1 mile
- Site Notes:
  - Section Start: Junction 17 sign
  - Zero thermal cracking
  - Very open surface

  - Poor construction joints
  - Extremely dry and coarse mix

***Trunk Highway 27 (SP 4803-19)***

- Location: Starts in Onamia and spans west
- Construction Year: 2010
- Construction Type(s):
  - 3 inch mill and overlay (good and poor performers)
- Section Length: Roughly 7.5 miles
- Site Notes (Poor Performer):
  - Section Start: RP 171
  - Rides significantly worse than 174 section
  - Chip seal applied to surface
  - Poor base in this location—swamp to both sides
  - Significant settlement in some areas
  - Some severe longitudinal cracking
- Site Notes (Good Performer):
  - Section Start: RP 174
  - Much improved ride as compared to RP 171
  - Chip seal applied to surface
  - Also has large amount of cracking

***Trunk Highway 28 (SP 6104-11)***

- Location: Spans from Glenwood to West Port
- Construction Year: 2012

- Construction Type(s):
  - 4-1/2 inch mill and overlay (good and poor performers)
- Section Length: Roughly 13 miles
- Site Notes (Poor Performer):
  - Section Start: RP 81
  - Rides well
  - Thermal cracking straight across
  - Centerline segregation
  - Significant shoulder cracking
  - Slightly more cracking than RP 88
- Site Notes (Good Performer):
  - Section Start: RP 88
  - Rides well
  - Thermal cracking straight across
  - Centerline segregation
  - Significant shoulder cracking

***County State Aid Highway 30 (SP 1306-44)***

- Location: In North Branch city limits
- Construction Year: 2012
- Construction Type(s):
  - 6 inch mill and overlay
- Section Length: ¼ of a mile
- Site Notes:

- Section Start: Intersection with TH 95
- Very short section
- Complex geometry with large number of intersections
- Performing well, good ride

CSAH 30-section start

CSAH 30-overview

CSAH 30-surface profile

CSAH 30-typical crack configuration

CSAH 30-typical crack profile

### *2.4.5 Interstate 35 (SP 0283-26)*

- Location: Section begins in Forest Lake and stretches south
- Construction Year: 2009
- Construction Type(s):
    - 4 inch mill and overlay on existing concrete
- Section Length: Approximately 8 miles
- Site Notes: (Based on drive through survey)
    - Four sections were surveyed, two in the northbound direction and two in the southbound direction
    - First section of northbound direction featured the greatest amount of cracking. Cracks were not full width, but were rougher than rest
    - Second section of northbound direction had relatively uniform crack spacing. Most cracks were full width across all three lanes.
    - First section of southbound showed the least amount of cracking, with all cracks being full width
    - Second section of southbound was very comparable to the second section of the northbound direction. Cracks were of relatively uniform spacing and full width.
- Due to the high traffic level of this roadway, no relevant pictures could be taken

### *Trunk Highway 53 (SP 8821-177)*

- Location: North of Virginia
- Construction Year: 2008
- Construction Type(s):

    - 1-1/2 inch mill and overlay
- Section Length: 6 miles
- Section Start: Sign saying "TH 169 to Ely" (exit ¾ mile)
- Site Notes:
    - Moderate ride quality
    - Consistent amount of transverse cracking
    - Raveling in some locations
    - Shoulder cracking is not sealed
    - Cracks on primary driving areas sealed
    - Shoulder cracking 2:1 ratio in comparison to cracking in driving area

TH 53-section start

TH 53-overview

TH 53-surface profile

TH 53-typical crack configuration

TH 53-typical crack profile

***Trunk Highway 113 (SP 4407-12)***

- Location: Spans between Syre and Waubun
  - Project is split between two SP numbers
    - SP 4407-12 extends west from Waubun for approximately 6 miles
    - SP 5413-10 spans the remaining 9 miles to Syre
- Construction Year: 2006
- Construction Type(s):
  - 1-1/2 inch overlay on old asphalt concrete (poor performer)
  - 5 inch overlay on reclaimed asphalt concrete (good performer)
- Section Length: Slightly under 15 miles
- Site Notes (Poor Performer):
  - SP 4407-12
  - Section Start: RP 10
  - 1-1/2 inch overlay on old asphalt concrete
  - Near Waubun
  - Some transverse cracking meanders into longitudinal cracks
  - Potential reflective cracking

TH 113 poor performer-section start

TH 113 poor performer-overview

TH 113 poor performer-surface profile

TH 113 poor performer-typical crack configuration

TH 113 poor performer-meandering transverse cracks

- Site Notes (Good Performer):
  - SP 5413-10
  - Section Start: RP 5
  - 5 inch mill and overlay on reclaimed asphalt
  - Near Syre
  - Good ride
  - Traditional transverse cracking

TH 113 good performer-section start

TH 113 good performer-overview

TH 113 good performer-surface profile

TH 113 good performer-typical crack configuration

TH 113 good performer-typical crack profile

***Trunk Highway 210 (SP 1805-72)***

- Location: Spans through Baxter
- Construction Year: 2010
- Construction Type(s):
  - 2 inch overlay on existing concrete
- Section Length: Roughly 4.5 miles
- Section Start: RP 118
- Site Notes:
  - Mix is quite coarse
  - Longitudinal joint is 100 percent cracked
  - Section exhibits transverse cracking roughly every 30 feet
  - 2 inch overlay over existing concrete
  - All transverse cracking is 100 percent reflective cracking

- Raveling in various areas of the section

TH 210-section start

TH 210-surface profile

TH 210-raveling

TH 210-typical crack configuration

TH 210-typical crack profile

***Trunk Highway 212 (SP 1017-12)***

- Location: Spans through Chaska
- Construction Year: 2008
- Construction Type(s):
  - Bituminous over aggregate base
  - SMA mix design
- Section Length: Approximately 3 miles
- Due to high traffic levels, this site could not be surveyed
- Historical data on this section was available and will be presented

***Trunk Highway 220 (SP 6016-37)***

- Location: Spans between Climax and East Grand Forks
- Construction Year: 2012
- Construction Type(s):

    - 3 inch mill and overlay
- Section Length: 23.5 miles
- Site Notes:
    - Section Start: RP 12
    - Good ride
    - Extremely small amount of cracking, but cracks are large where they occur
    - Open surface
    - Small amount of raveling on surface

TH 220-section start

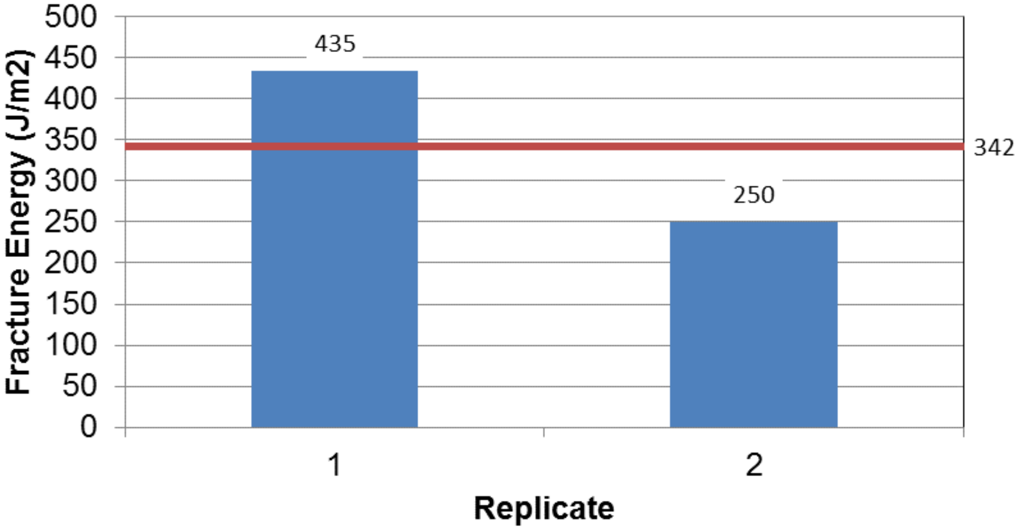
TH 220-overview

TH 220-surface profile

TH 220-typical crack configuration

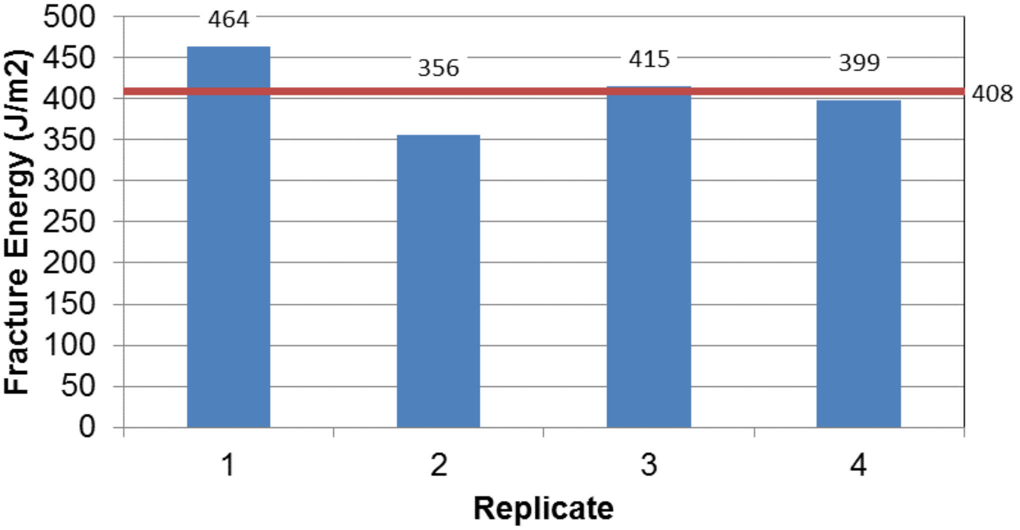
TH 220-typical crack profile

## APPENDIX B: ADDITIONAL DCT TEST PLOTS AND REPRESENTATIVE PHOTOS

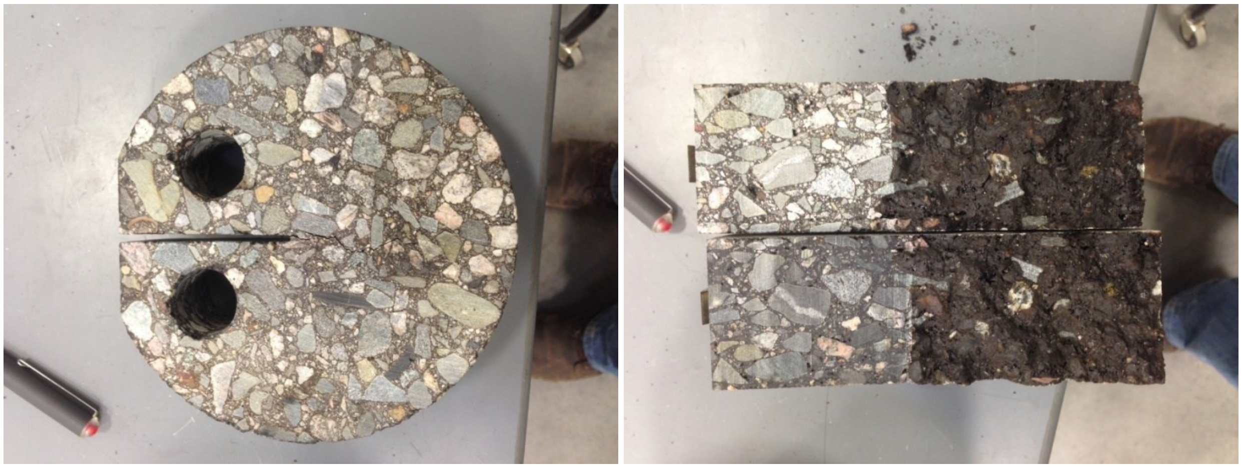


TH 1-RP 235 (poor performer) DCT results

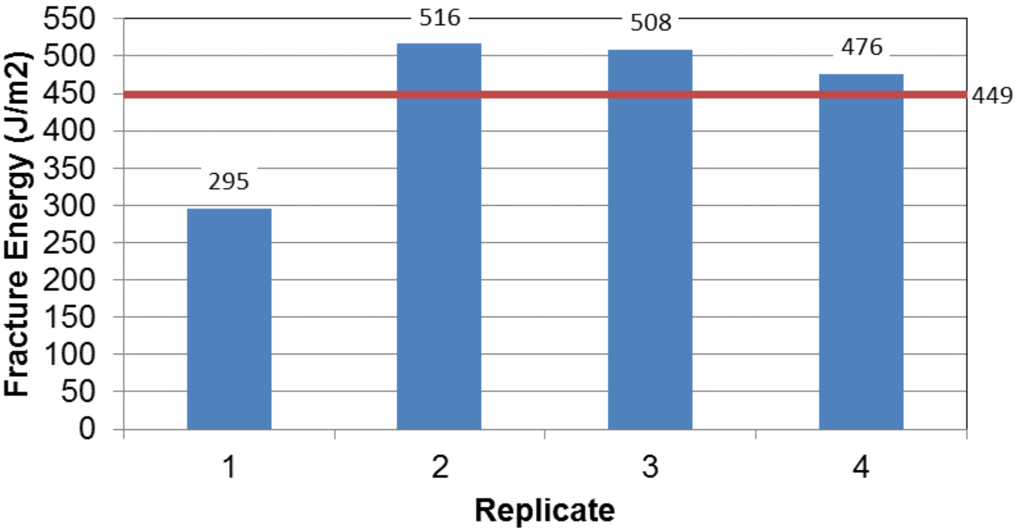
TH 1-RP 235 (poor performer)--(a) specimen profile; (b) fractured cross-section



TH 1-RP 230 (good performer) DCT results

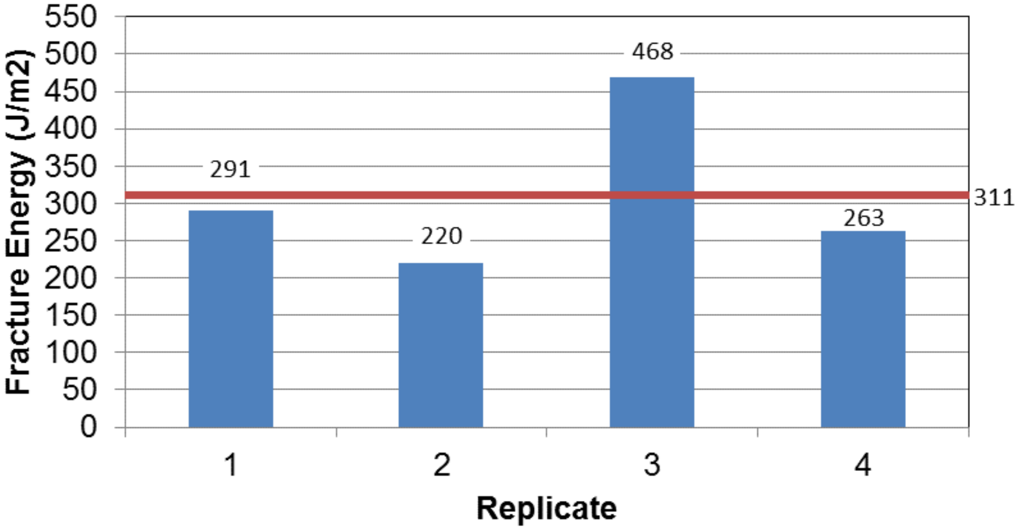


TH 1-RP 230 (good performer)--(a) specimen profile; (b) fractured cross-section

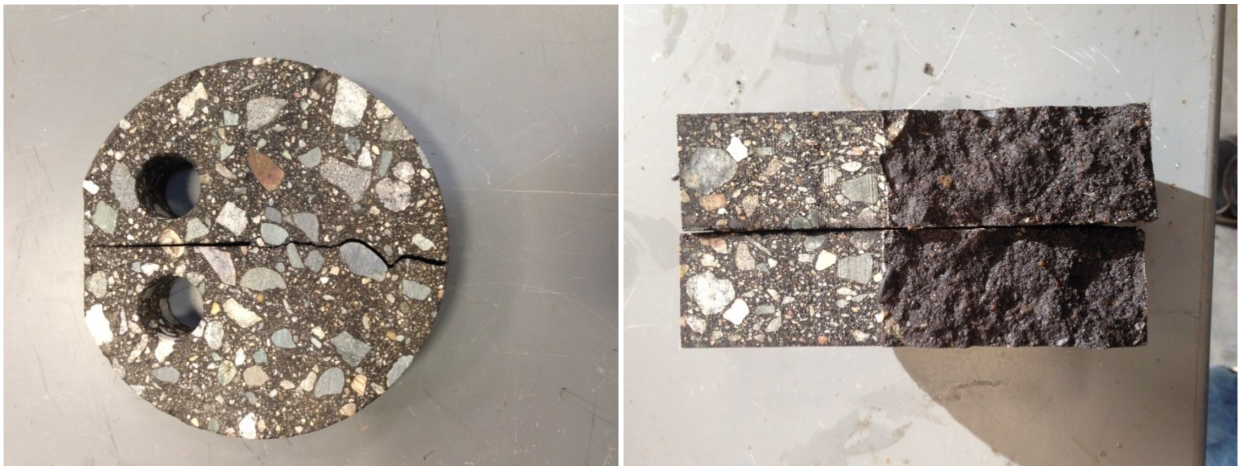


TH 2 DCT results

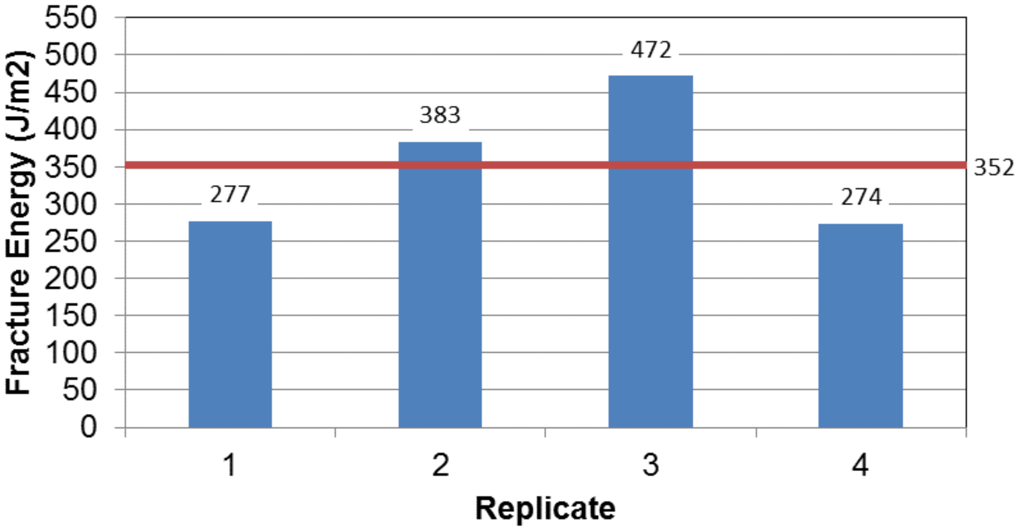
TH 2--(a) specimen profile; (b) fractured cross-section



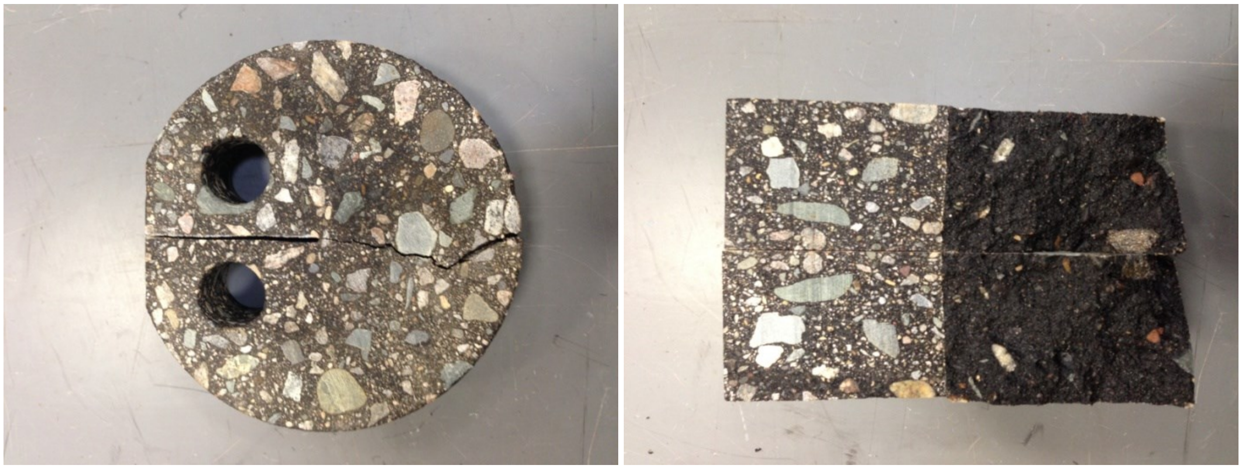
TH 6-RP 118 (poor performer) DCT results



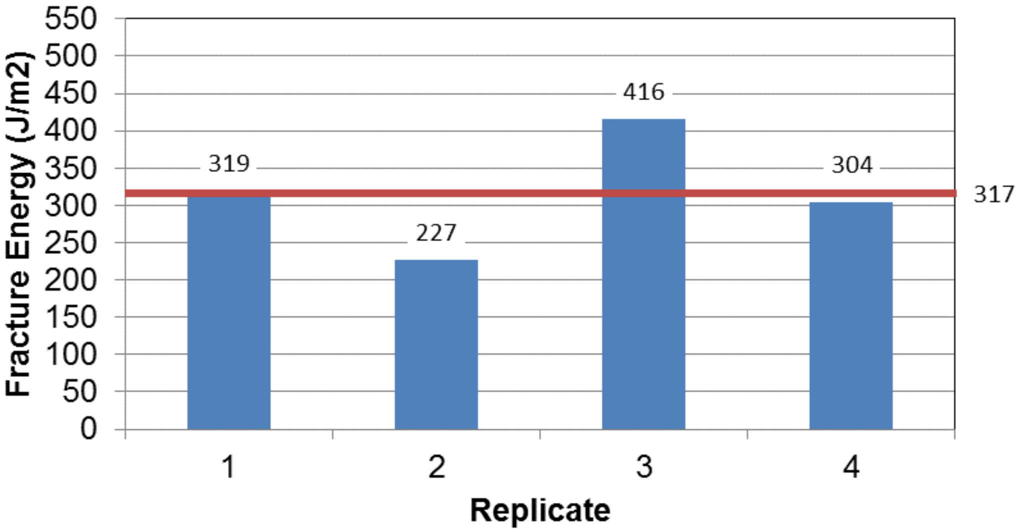
TH 6-RP 118 (poor performer)--(a) specimen profile; (b) fractured cross-section



TH 6-RP 123 (good performer) DCT results

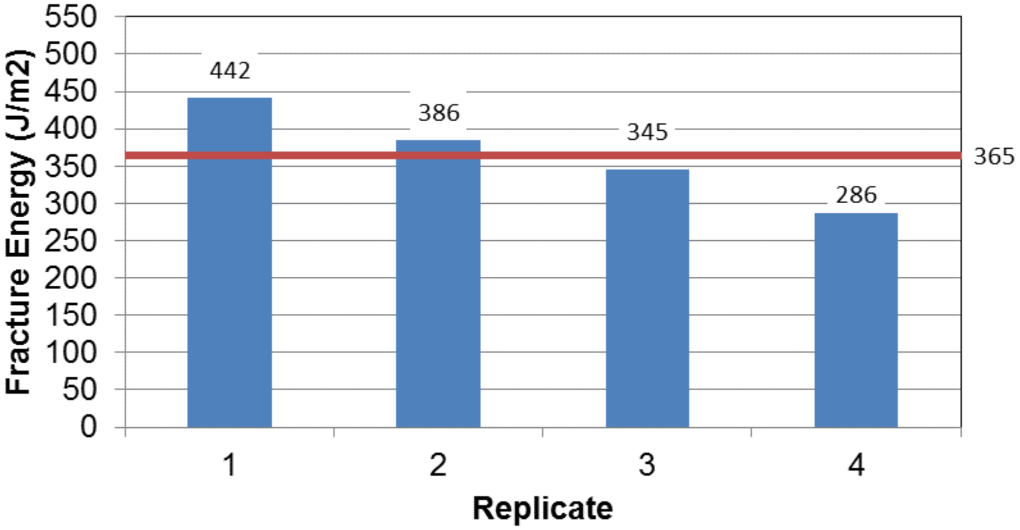


TH 6-RP 123 (good performer)--(a) specimen profile; (b) fractured cross-section



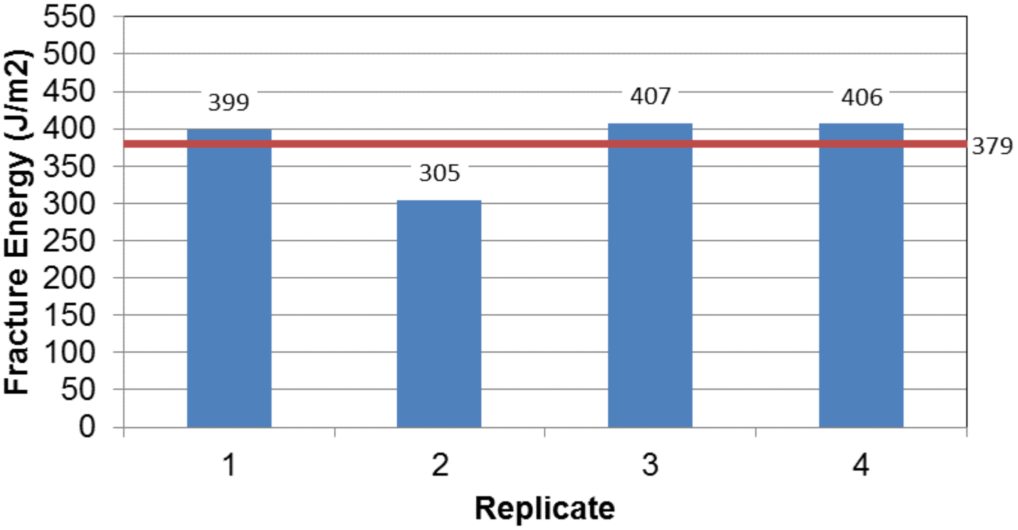
TH 10-RP 159 (poor performer) DCT results

TH 10-RP 159 (poor performer)--(a) specimen profile; (b) fractured cross-section

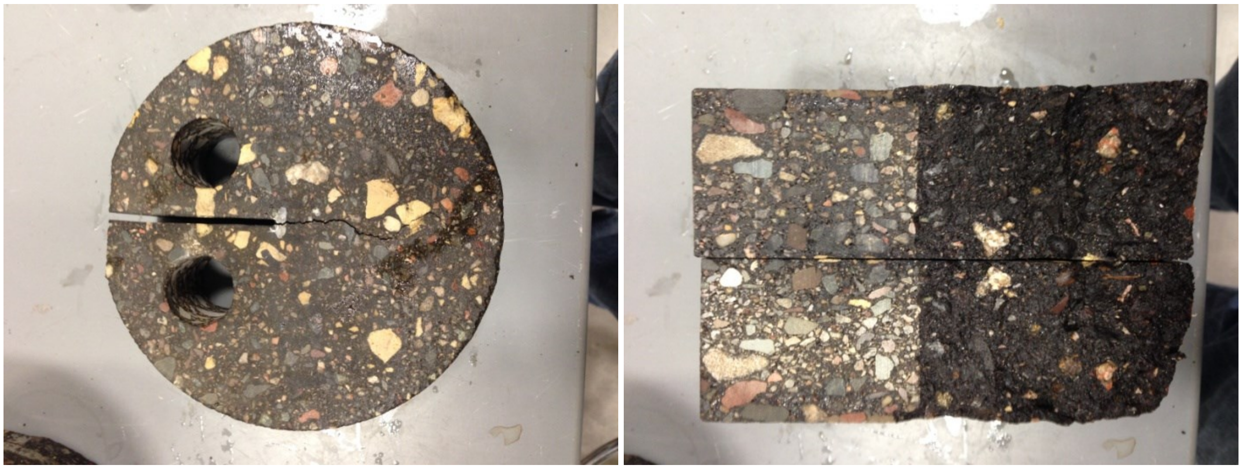


TH 10-RP 161 (good performer) DCT results

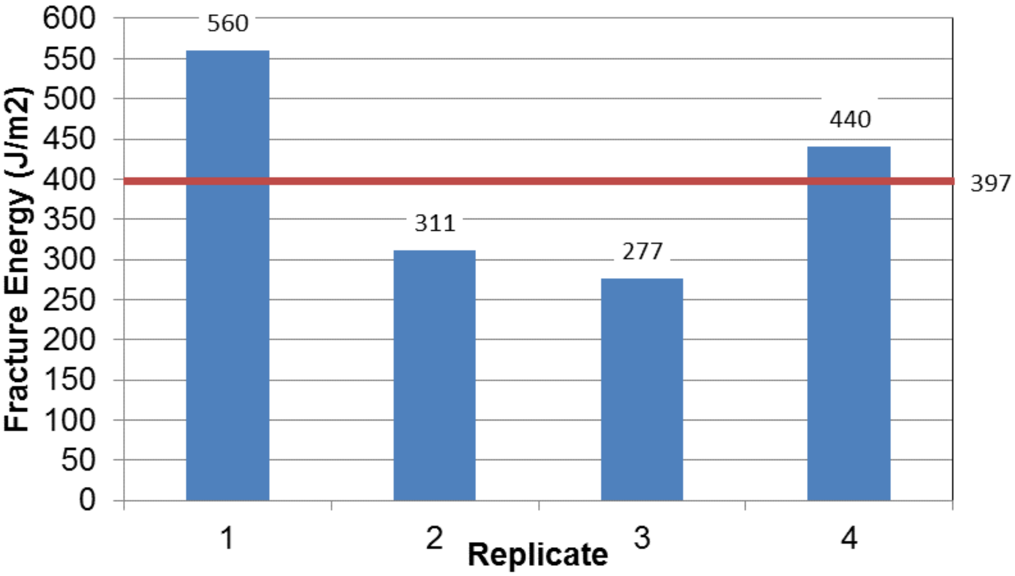
TH 10-RP 161 (good performer)--(a) specimen profile; (b) fractured cross-section



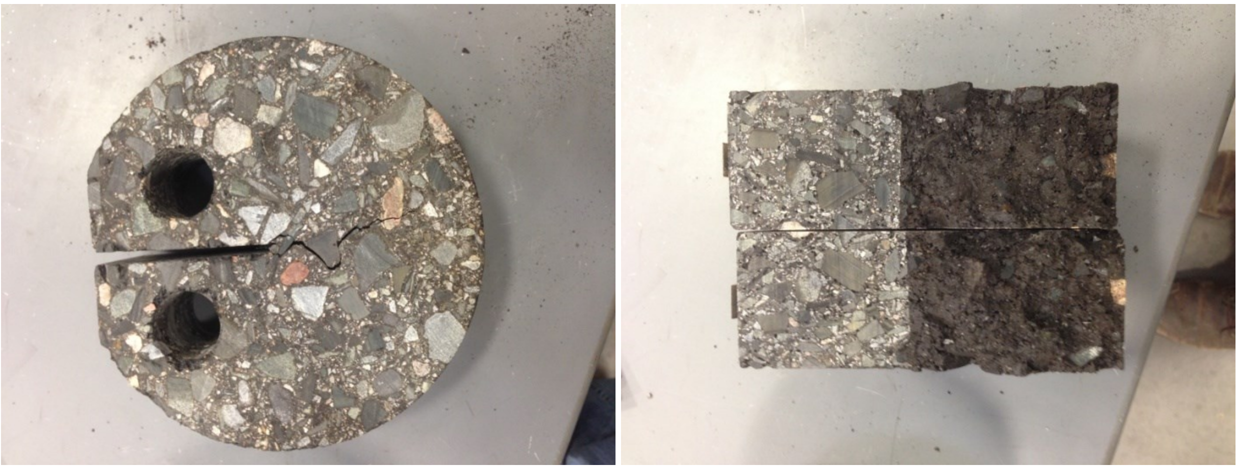
I-35 DCT results



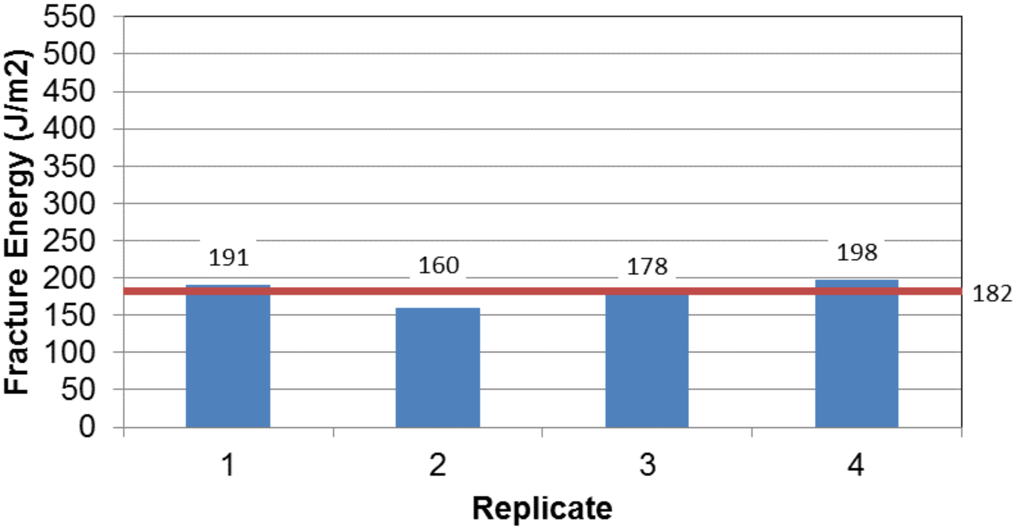
I-35--(a) specimen profile; (b) fractured cross-section



TH 53 DCT results

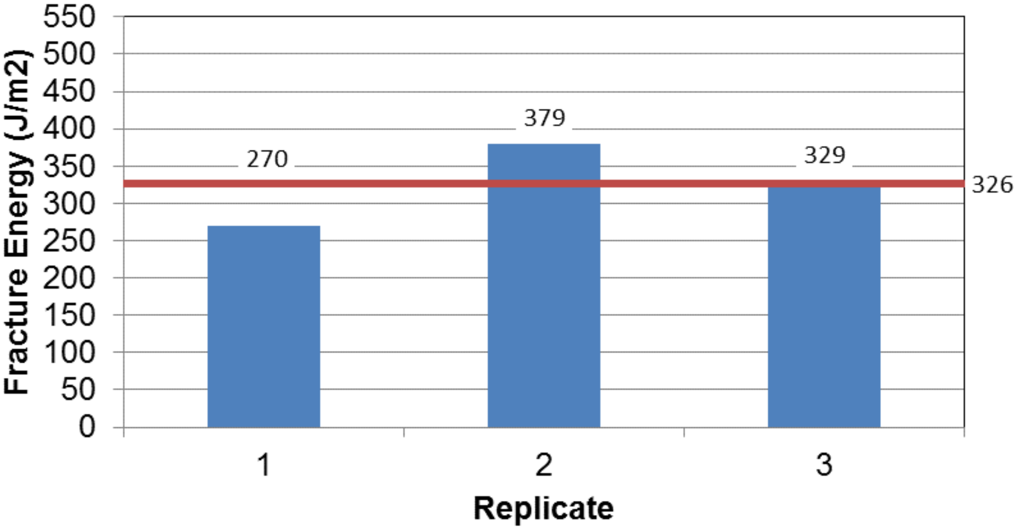


TH 53--(a) specimen profile; (b) fractured cross-section



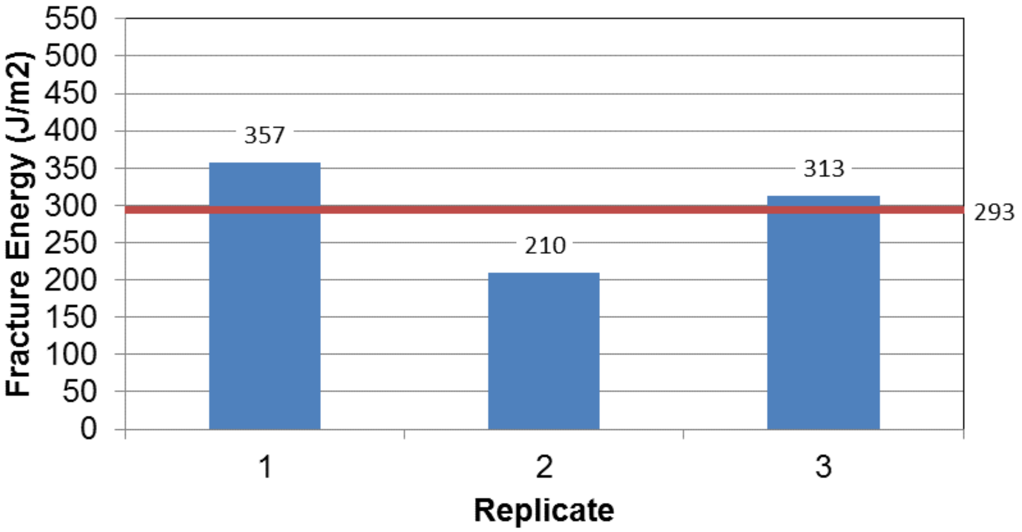
TH 113-RP 10 (poor performer) DCT results

TH 113-RP 10 (poor performer)--(a) specimen profile; (b) fractured cross-section



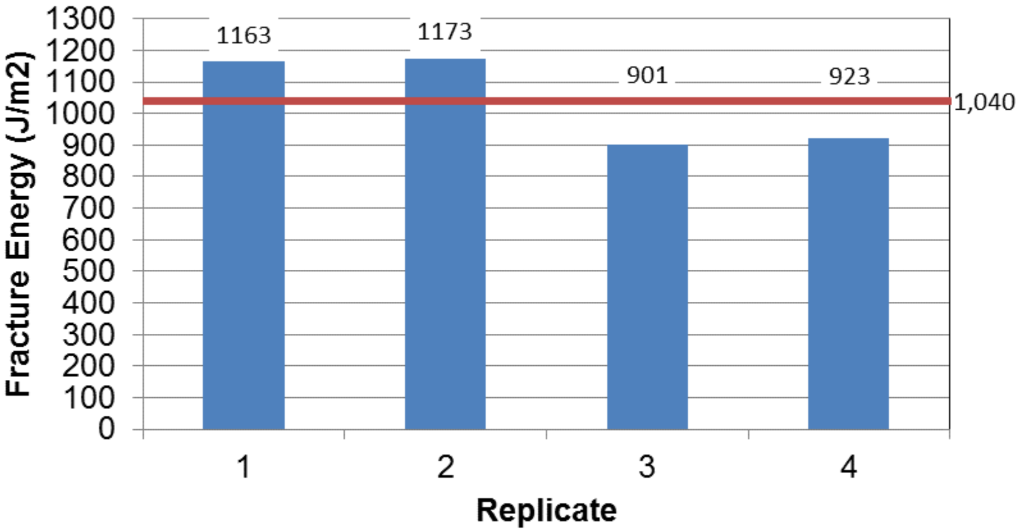
TH 113-RP 5 (good performer) DCT results

TH 113-RP 5 (good performer)--(a) specimen profile; (b) fractured cross-section

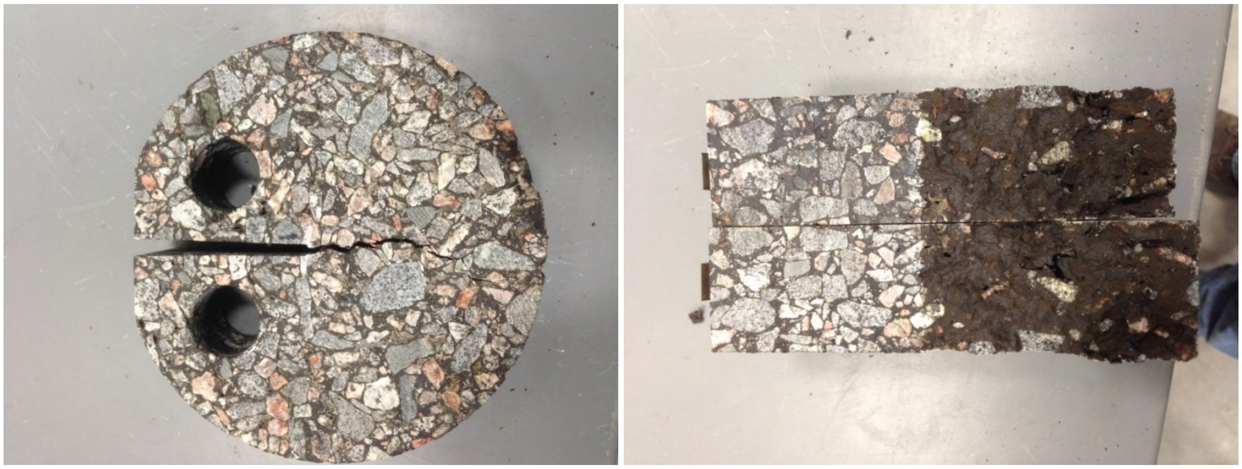


TH 210 DCT results

TH 210--(a) specimen profile; (b) fractured cross-section



TH 212 DCT results



TH 212--(a) specimen profile; (b) fractured cross-section