EVALUATION OF FIELD TRANSVERSE CRACKING OF ASPHALT PAVEMENTS

By

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EVALUATION OF FIELD TRANSVERSE CRACKING OF
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Abstract

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Transverse cracking is one of the major distress types in asphalt pavement. The crack initiates from pavement surface and propagates downward (thermal cracking), or initiates from pavement bottom and propagates upward (reflective cracking). In this study, the mechanism and key factors of a transverse cracking type named Surface-initiated reflective cracking (SIRC) are identified and evaluated. SIRC refers to crack that initiates at the surface of pavement layer in transverse direction and propagates downward to match the existing transverse cracking, and there is no cracking in the interlayer.

The mechanism and key factors of the SIRC are evaluated by using three-dimensional Finite Element Method and laboratory modified Hamburg Wheel Tracking Test. Results indicate that when the HMA surface layer is thick, thermal load associated with traffic load, overlay stiffness and thickness above existing crack/joint are the key factors to initiate SIRC. When the pavement thickness is at certain thickness range (i.e., thick enough to protect the existing crack/joint to move but not too thick which can allow the surface layer bending downward with weak base structure), with relative stiff overlay material and weak base, tensile stress can
develop at the surface of the pavement layer which can ultimate lead to SIRC under repeated traffic load.

The widely used transverse cracking prediction model TCMODEL is evaluated to check its predictive quality on conventional transverse cracking and the SIRC. It is found that the predicted transverse crack from TCMODEL do not match well with the field measurements and therefore, it is necessary to develop new predictive models for transverse cracking including all transverse cracking types.

Statistical methods in conjunction with engineering interpretation is used to develop crack initiation and propagation models for both SIRC and conventional transverse cracking. Hour of low temperature (<15°F), percent passing #200 sieve, IDT strength and service life (age) are found to be the critical indicators for the initiation of the transverse cracking. Material properties (mixture creep compliance, work density, and percent passing #200 sieve), pavement structure (overlay thickness), climate (hour of low temperature), and traffic (AADTT) are found to be critical factors for the propagation of the transverse cracking.
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DEDICATION

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CHAPTER 1 INTRODUCTION

1.1 Overview of Transverse Cracking in Asphalt Pavement

Transverse cracking is one of the major distress types for asphalt pavement. It is perpendicular to traffic direction and often equally spaced (Al-Qadi, 2005). Depending on the initiation mechanism and formation of crack, transverse cracking is usually divided into two types, climate related thermal cracking and load/climate related reflective cracking.

Thermal cracking is caused by sudden dramatic drop of pavement temperature (low temperature thermal) or repeated daily temperature fluctuation (thermal fatigue), which typically is initiated from the surface of pavement and propagate downward (Vinson et al., 1989). Thermal cracking of asphalt pavement is a common distress type in cold regions and has been recognized in the northern United States and Canadian provinces (Marasteanu, et al., 2004). Figure 1.1 shows the typical thermal cracking observed in the field.

Thermal cracking can increase the roughness of pavement and decrease ride quality. It also allows destructive substance, such as air and water to enter the pavement structure. Thermal cracking reduces pavement life and requires frequent maintenance and rehabilitation. It also increases user costs (vehicle repair, tyre, and fuel cost) dramatically. As suggested by Islam and Buttlar (2012), the associated increase of user cost due to the existence of thermal cracking can be over five million dollars per lane mile over a 35-year life cycle. The introducing of water through the cracks further increases the rate of stripping which could result in deterioration of
asphalt concrete. Additionally, it is pointed that the transverse cracking can act as stress focal point from which longitudinal cracks may form (Marasteanu, et al., 2004).

Figure 1.1 Typical thermal cracking in the field pavement.

Reflective cracking is caused by high stress and strain concentration at the bottom of asphalt overlay due to the movement of existing pavement in the vicinity of existing cracks or joints underneath the overlay layer. Such concentrated stress/strain will lead to crack propagation upward in asphalt overlay layers (Lytton et al., 2010). This movement may be induced by bending or shearing due to traffic load and temperature variations, or a combination of the two (Pais and Pereira, 2000; Lytton et al., 2010). Figure 1.2 shows the typical reflective cracking observed in the field. Reflective cracking allows water to penetrate into the pavement structure, weakens the base structure, and leads to many forms of pavement deterioration, including increased roughness and spalling. Therefore, the service life (age) of the pavement can be significantly reduced.
Transverse cracking and its related issues have been studied extensively in the past decades. Generally, these studies can be classified into four groups, including those (1) explore the mechanisms of crack initiation and propagation; (2) find key factors that relate to field transverse cracking; (3) develop empirical or mechanistic models to predict crack initiation and propagation, or (4) discover treatment methods to mitigate crack initiation or propagation. Many advanced theories and techniques have been introduced to explain transverse cracking related problems; some of them are proven. For instance, concepts from fracture mechanics have been adopted to analyze the stress distribution at crack tip, like J-integral and Cohesive Zone Method. In addition, laboratory experiments have been developed to evaluate material properties that could distinguish asphalt/asphalt mixture with different fracture resistance. Some mathematical approaches have been also introduced to obtain prediction models (Fromm and Phang, 1972; Hass et al., 1987). Most of the finished studies share some common assumptions as follows, (1) reflective cracking is reflected from the crack or joint within existing pavement and propagates
upward, and (2) transverse cracking initiates from pavement surface and has nothing to do with the existing crack.

However, existing cracking in underlying pavement layer will not always result in cracking reflected from the bottom of the layers. Many cores taken from the field indicated that some transverse cracks were only observed in the surface of the pavement with a few millimeters in depth. Similarly oriented transverse crack can also be found in the bottom layers. However, no crack can be visually seen from the middle layer (Figure 1.3). It appeared that transverse cracking was initiated at surface and propagates downward to match the existing transverse cracking, and there was no cracking in the interlayer. Obviously, this type of cracking is different from the conventional reflective cracking. Similar cracks were also observed by Nunn and Nesnas (Nunn, 1989; Nesnas and Nunn, 2006) and other researchers (Ros. Et al., 1982; Burt, 1987; Sha, 1993; Bennert et al., 2009).

Figure 1.3 Examples of field cores showing surface initiated cracks and bottom layer cracks without interlayer cracking.
In summary, the transverse crack type described above is different from conventional thermal/reflective cracking in its physical appearance. It could be caused by different mechanisms than conventional transverse cracking. It is named as SIRC in this study to be distinguished from conventional transverse cracking. Nunn, Nesnas and Shen (Nunn, 1989; Nesnas and Nunn, 2006; Shen et al., 2013) reported the wide existence of such crack type in the United Kingdom and the United States. Burt (1987) also found that for the asphalt pavement placed on existing Portland Cement Concrete (PCC) pavement, the crack could initiate from asphalt pavement surface instead of being reflected upward. The author assumed that factors like pavement structure, joint width, and load condition could affect this type of crack. Sha (1993) indicated that this cracking type was observed for Hot Mix Asphalt (HMA) overlay placed directly on cement-treated base layer. The author pointed out that pavement thickness above base layer could be highly correlated to such cracking type. Bennert et al. (2009) suggested that this type of surface crack could initiate and propagate several millimeter away from the existing crack, indicating a potential of shear failure. In the on-going NCHRP 9-49A project, a total of 71 in-service pavement sections were investigated and transverse cracking were observed in 34 pavement sections. Among the 34 sections, 14 (41%) sections had SIRC according to field core observations (Shen et al., 2013).

Although the SIRC has been observed and reported by several studies, very few studies have clearly defined such cracking type or discussed its potential cause and mechanism. In fact, there is very little discussion about the SIRC in the literature.
1.2 Problem Statement

Although SIRC has been observed to a great extent in the field, such distress has not been well documented or specifically studied. Without sufficient knowledge about the distress formation mechanism and influencing factors, it is extremely difficult to select proper pavement maintenance and rehabilitation strategies to address this specific distress. Suggestions on the improvement of current material and structure design concept/methodologies to take into account of such crack types cannot be obtained. Only after clearly determining mechanism and influencing factors, measures could be taken into drive the conventional reflective cracking to SIRC, such that the maintenance and rehabilitation work is more cost-effective.

To fill this knowledge gap, it is needed to perform a comprehensive evaluation and analysis on pavement transverse cracking especially focusing on the SIRC. Particular questions that will be addressed through this study include: “how the different types of transverse cracks should be defined and categorized”, “under which condition the SIRC could happen and how typical it is in the field”, “what are the mechanisms and key factors of the SIRC”, and “can we develop predictive models for field transverse cracking”.

1.3 Research Objectives and Research Tasks

The objectives of this study are: (1) to investigate the mechanisms of surface-initiated reflective cracking (SIRC); (2) to identify and validate the key factors that related to this distress, and (3) to develop predictive models that are able to predict both conventional transverse cracking and SIRC in the field.
To achieve the research objectives, research activities are conducted in six tasks:

1. Quantitatively categorize field transverse cracking based on field performance;
2. Identification of significant factors based on field projects;
3. Investigate the initiation and propagation mechanisms of SIRC and identify key factors that relate to SIRC;
4. Laboratory validation of factors that are highly related to SIRC;
5. Evaluate the predictive quality of current transverse cracking models embedded in AASHTOWare Pavement ME Design program, and
6. Develop predictive models for field transverse cracking include both conventional transverse cracking and SIRC.

Figure 1.4 shows a flowchart of this research.
1.4 Structure of the Dissertation

The dissertation starts with the introduction of the research background, problem statement, and research objectives as described in chapter 1. Chapter 2 includes literature review of asphalt pavement cracking with an emphasis on the mechanism, influencing factor, predictive models, laboratory evaluation methodologies, and crack control methods. Chapter 3 provides detailed project information that is used to determine mechanism, key factors and prediction models of SIRC. In chapter 4, field transverse cracking is firstly categorized into three types: conventional thermal cracking, conventional reflective cracking and SIRC. The potential factors that are correlated with each crack type are also presented. Next, the mechanism of SIRC is evaluated by using three-dimensional (3-D) Finite Element Method (FEM) models. Critical pavement responses under different conditions are evaluated. Both traffic load and thermal load are applied. Field core data are used to validate the modeling results for specific influencing factors. In chapter 5, a laboratory test is presented to evaluate the key factors that may correlate to SIRC based on FEM analysis. In chapter 6, the effectiveness of transverse cracking prediction model in the AASHTOWare Pavement ME Design program is evaluated. In chapter 7, statistical-based predictive models are established for transverse cracking that include both conventional transverse cracking and SIRC. Crack initiation and crack propagation models are developed and validated separately considering different mechanism may dominate at the two different stages. Chapter 8 provides conclusions, recommendations and contributions of this study.
CHAPTER 2 LITERATURE REVIEW

The literature review is focused on the mechanism, prediction models, factors that affect the crack initiation or propagation, laboratory evaluation methodologies, crack control strategies, and other related topics of pavement cracking performance. Specifically, in addition to provide a review for transverse cracking (including both thermal cracking and reflective cracking), the associate topics for longitudinal cracking were also included. It is believed that thermal cracking and longitudinal top-down cracking share some similarities such as both initiated from pavement surface and the climatic effect played a significant role in the development of cracking. Therefore, a review of longitudinal cracking could provide insights into the understanding of various cracking mechanisms.

Here, transverse cracking refers to pavement cracks that are perpendicular to vehicle driving direction. Depending on different forming mechanisms, transverse cracking can initiate from the surface of pavement and propagate downward (thermal cracking) or initiate from the bottom of pavement and propagate upward (reflective cracking).

Longitudinal cracking indicates pavement cracks that are parallel to vehicle driving direction. Typically longitudinal cracking initiates from pavement surface and propagates downward.

2.1 Thermal Cracking
Thermal cracking is one of the most typical transverse cracking types in the field. It is usually distributed approximately uniformly with the natural spacing ranging between a few meters and several hundred meters (Hass et al., 1987; Huang, 1993). Specifically, the crack spacing is usually greater than 100 ft. for new constructed pavement, while is much closer (as short as 10-20 ft.) when the pavement ages and/or more extreme temperature drops occur (Vinson, et al., 1989; Kandhal, 1978). Lytton et al. (1993) states that the maximum amount of thermal cracking can be considered as 200ft./500ft., which is also used in the current AASHTOWare Pavement ME Design program as the limitation of field thermal crack amount. These cracks can grow to approximately 0.8 inch or greater in width after several years (Anderson, et al., 2001). Although the thermal crack spacing could be varied at different pavement sections, it could become a serious pavement distress and reduce pavement service life (age).

2.1.1 Mechanism of Thermal Cracking

The mechanism that relates to low-temperature thermal cracking is the tensile stress (thermal stress) that developed in asphalt overlay for very severe cooling cycles (very low temperature and/or very fast cooling rates) (Vinson et al., 1989). Thermal crack could be initiated when the tensile stress (thermal stress) is equal to or larger than the mixture’s tensile strength. Under additional thermal cooling cycles, the crack propagates downward through the depth of asphalt layer.

Specifically, thermal stress is caused by volumetric contraction under the effect of thermal load (Vinson et al., 1989). If a material is unrestrained, it will shorten as the temperature drops. If a material is restrained, which is the case for asphalt concrete pavement, the tendency to shorten results in the development of a thermal stress.
In addition, thermal stress are not uniform with depth because of a thermal gradient, i.e., the pavement temperatures vary with depth (Huang, 1993). When the temperature on top of the pavement slab is higher than that at the bottom, the top tends to expand with respect to the neutral axis, while the bottom tends to contract. However, the slab is restrained from expansion and contraction; thus, compressive stresses are induced at the top and tensile stresses at the bottom. When the temperature on the top of the slab is lower than that at the bottom, the top tends to contract with respect to the bottom; thus, tensile stresses are induced at the top and compressive stresses at the bottom.

Thermal stress is also significantly affected by asphalt concrete property. At warm temperatures, asphalt concrete acts as a viscoelastic material. Therefore, the thermal stresses that developed at low temperature are dissipated through stress relaxation (viscous flow) when the temperature increases to a warm temperature range. In a low temperature range, the asphalt concrete behaves as an elastic material and the thermal stresses are difficult to dissipate (release) and cracking could occur (Vinson et al., 1989). The higher the ability of asphalt mixture to release stress, the lower the thermal stress buildup will be at a given temperature, and therefore, the pavement can withstand lower temperature before fracture (Tabatabaee et al., 2012). The rate of stress relaxation can be determined by master relaxation modulus curve (Kim et al., 2010a). Once failure has occurred and a crack develops, the stress is relieved.

In summary, three conditions are necessary to initiate low temperature thermal cracking: (a) the restraint force must be high enough to prevent the slab movement and subsequent release of the thermal stress; (b) there should be temperature gradient within pavement structure, and temperature value at pavement surface should be lower than that at pavement bottom to produce
a thermal stress that equals to or greater than the tensile strength of the pavement, and (c) the asphalt concrete does not have the ability to release most thermal stress at a given temperature.

Unlike low temperature thermal cracking, thermal fatigue transverse cracking was firstly noticed in early 1970s in Texas, where the pavements usually do not experience extremely low temperature (Epps, 1999). Thermal fatigue cracking may be related to repeated thermal cycles at moderate temperatures (Vinson et al., 1989; Epps, 2000). The tensile stress in this case is usually much smaller than mixture’s tensile strength. Therefore, the failure does not happen immediately but developed over a period of time. The thermal fatigue cracking appears when the thermal cycling exceeds the fatigue resistance of asphalt mixture (Epps, 2000). It was believed that the mechanism of failure is the same for low temperature cracking as for thermal fatigue cracking; the only difference is in the rate at which cracking occurs (ARA, 2003). Thus one may refer to both phenomena as thermal cracking.

In summary, low temperature thermal cracking could happen under extreme cold conditions and/or rapid cooling rates, while thermal fatigue cracking is believed to occur under milder conditions. However, it is very hard to determine a single fracture temperature at which thermal crack is initiated since it is affected by many factors such as bonding condition, climatic condition, material property, as well as degree of aging.

2.1.2 Factors Affecting Thermal Cracking
Many studies have been conducted to determine significant factors that contribute to thermal cracking. In general, these factors include but not limited to: (1) pavement bonding condition; (2) pavement structure; (3) climatic effect; (4) pavement service life (age); (5) asphalt binder property, and (6) asphalt mixture property.
2.1.2.1 Effect of Bonding Condition

The bonding condition affects the initiation and propagation of thermal cracking from the following aspects, (a) large frictional constraint could delay the formation of open cracks; (b) both the crack spacing and width of cracks decrease as frictional constraint between the asphalt and base layers is increased (Vinson et al., 1989), and (c) when there is high frictional constrain between layers, a greater temperature drop is required for a fictitious crack to develop into an open crack (Kirkner and Shen, 1999).

2.1.2.2 Effect of Pavement Structure

The total pavement thickness can affect the distribution of tensile stress (Hiltunen and Roque; 1994; Lytton et al., 1993). In general, the thicker pavement structure could increase thermal cracking spacing and lead to less incidence of thermal cracking (Hass et al., 1987; Vinson et al., 1989). At the Ste. Anne Test Road, increasing the thickness of the asphalt concrete layer from 4 to 10 in. resulted in one half the cracking frequency when all other variables were the same (Vinson et al., 1989). This may due to the fact that increasing the thickness effectively increases the mass of the asphalt concrete layer, and it is therefore less likely to curl upward due to an increased self-weight acting in the opposite direction (Marasteanu et al., 2004).

A correlation between overlay thickness and 22 field thermal cracking sections indicates that the thermal cracking amount increases with decreased overlay thickness in general, as shown in Figure 2.1 (Shen et al., 2013). Based on finite element analysis, it was proved that the thinner the surface layer, the stronger the interface restraint for the fully bonded surface layer/substrate system, and thermal stress at pavement surface that tends to curl the pavement upward could increase accordingly (Marasteanu et al., 2007).
Figure 2.1 Correlation between overlay thickness and field thermal cracking amount.

It was also confirmed in laboratory that fracture energy increases as specimen thickness increases, indicated a potential of more resistance to thermal cracking (Wagoner et al., 2005).

In the AASHTOWare Pavement ME Design program, within any climate region, the predicted thermal cracking always increases as the asphalt thickness decreases (ARA, 2003).

Kallas (1965) checked the temperature profile along pavement depth direction based on asphalt concrete sections with two different thickness (12 in. and 6 in. thick). The author mentioned that temperatures at the same depths in both the 6 in. and the 12 in. thick asphalt concrete were virtually the same.

2.1.2.3 Effect of Climatic Condition

Among all the factors, climatic condition could be the most critical one (Hass et al., 1987). Climatic condition highly affects the initiation and propagation of thermal cracking in two
aspects: 1) temperature gradient distribution along pavement depth direction, and 2) cooling rate change with time.

One of the conditions of thermal cracking initiation is that the temperature at pavement surface is lower than the temperature at pavement bottom. Therefore, the temperature gradient distribution along pavement depth is critical since a higher temperature difference between pavement surface and bottom tends to induce higher thermal stress at pavement surface. To obtain the distribution of temperature profiles within asphalt concrete pavement, a prediction model named Integrated Climatic Model (ICM) was developed by Larson and Dempsey (1997). The model is able to predict or simulate the changes of environmental conditions that occur over many years of service. In SHRP report (Solaimanian and Bolzan, 1993), the ICM model was calibrated by using field measurements. The model calibration included environmental factors and thermal properties of asphalt pavement. The environmental factors considered are air temperature, percent sunshine and solar radiation. Thermal properties studied in the report were pavement emissivity, absorptivity and thermal conductivity. The model outputs were then compared with field pavement profiles. Results indicate that a good agreement between measured and predicted pavement surface temperatures was observed. A sensitivity analysis was also conducted which indicated that air temperature is the most influential factor regarding the pavement temperature. The difference between air temperature and pavement surface temperature was as low as 6 to 8°C, or as high as 22 to 28°C.

Cooling rate is another factor that greatly affect thermal cracking. Even at low temperatures, if the cooling rate is low enough the asphalt material will have sufficient time to undergo stress relaxation. By contrast, even at temperatures well above the critical cracking
temperature, asphalt concrete may still undergo significant stresses if the cooling rate is very high (Apeagyei et al., 2008). Both analytical solution and finite element simulation show that thermal stress increase as the rate of cooling increases. However, it should be noted that the sensitivity of various asphalt mixture to cooling rate change may different, and various asphalt mixture could response to different magnitude of thermal stress at the same cooling rate (Apeagyei et al., 2008).

2.1.2.4 Effect of Pavement Service Life (age)

It is generally believed that the thermal cracking amount increases with the increase of service year. In addition, the increased pavement service life (age) could lead to greater incident of thermal cracking. This is associated with the increase in stiffness and decrease in fracture energy of the asphalt concrete with age (Marasteanu et al., 2012). Also, with pavement becomes older, the occurrence probability of more extreme low temperature increased as well.

2.1.2.5 Effect of Asphalt Binder Property

Asphalt properties significantly affect the thermal characterization of field pavement and need to be carefully evaluated. So far, several different parameters and test methods have been proposed to determine the low temperature cracking potential of asphalt binders. The most popular ones are creep stiffness $S$ and $m$-value measured from bending beam rheometer (BBR), binder tensile strength and critical cracking temperature tested based on direction tension tester (DTT), fracture toughness ($K_{IC}$) obtained from fracture mechanics-based test, thermal cracking temperature from asphalt binder cracking device (ABCD) test, as well as stiffness from 4mm DSR test.

Bending beam rheometer (BBR) protocol is a standard test as required in AASHTO M320. The detailed test procedure can refer to AASHTO T313. In this test, the specimen is
loaded in three-point bending, and the creep stiffness $S$ and $m$-value are measured simultaneously at 60 second. The asphalt is considered to pass the specification if the creep stiffness $S<300$Mpa and $m$-value$>0.3$ at the test temperature for certain asphalt performance grade (PG). Asphalt with higher stiffness could become brittle at low temperature and is less fracture resistance. The $m$-value reflects the ability of asphalt to relieve thermal stress based on viscous flow mechanisms. The lower the $m$-value, the slower of asphalt binder to relax stress as the temperature decreases, causing thermal shrinkage stress to build more rapidly than asphalt binders with higher $m$-values. It should be noted that thermal cracking evaluated by BBR is associated with a single low-temperature of the pavement temperature to a critical or limiting stiffness temperature. Therefore, it is also called single-event thermal cracking (Anderson, 2001). The creep stiffness $S$ and $m$-value are not necessary related to thermal fatigue cracking (Al-Qadi, 2005).

However, the BBR test may not suitable for modified asphalt binders (Anderson, 2001; Kim et al., 2006). The creep compliance $S$ or $m$-value cannot be used to distinguish fracture resistance between modified and unmodified asphalt, although different field performance are widely observed (Anderson, 2001; Zborowski, 2007). The main effect of modified asphalt is they can significantly improve both fracture temperature and fracture stress of HMA (Aschenbrener, 1995; Raad et al., 1998; Zborowski, 2007). It was also showed that mixture with modified asphalt can subject more thermal cycling compared to unmodified counterparts, and therefore increase thermal fatigue cracking resistance of asphalt mixture (Epp, 1999). Considering the difference between modified and unmodified asphalt, AASHTO MP1a proposed to use both BBR and direction tension tester (DTT) to evaluate asphalt properties at low temperature. In this process, the BBR data are used to calculate thermal shrinkage stresses as temperature drops, and
DTT determines the tensile strength. The temperature at which the thermal shrinkage stresses are equal to the tensile strength of the asphalt binder is used as critical cracking temperature. In addition to DTT, the critical cracking temperature can also be determined by using an asphalt binder cracking device (ABCD) (Kim et al., 2006).

The above BBR method and a combination of BBR and DTT are widely accepted approaches to evaluate the thermal resistance of asphalt binder. The first approach is equi-stiffness temperature approach and the second approach matches thermal shrinkage stresses with tensile strength. Fracture mechanics offers a third and more fundamental approach. It is also a method that is able to distinguish the different fracture properties of modified or un-modified asphalt. The typical used parameter is fracture toughness ($K_{IC}$) which is determined by linear fracture mechanics principles (LFMP). It was found that unmodified asphalt can be well distinguished from modified asphalt at low temperature (Lee et al., 1995) by using $K_{IC}$. The $K_{IC}$ is also sensitive to the content of modifiers (Sabbagh and Lesser, 1998). In addition, fracture mechanics can also be used to determine the binder fracture strength (Roy and Hesp, 2001). Typically, the higher the binder fracture strength, the lower the cracking temperature (Kim et al., 2006).

Anderson et al. (2001) evaluated all the three approaches described above. A total of 14 asphalt binders were considered. The 14 binders were produced from the same base material but modified by different means. A little difference was found among the 14 binders in terms of ranking according to BBR or a combination of BBR and DDT approach. However, the ranking is quite different if fracture toughness ($K_{IC}$) was used.
Besides of the above three approaches, asphalt binder cracking device (ABCD) test and 4mm DSR test have also received wide interests in evaluating low temperature property of asphalt. ABCD test was developed by Kim (2007) in NCHRP IDEA-99 project, and was further refined and evaluated with FHWA’s LIFE program. The major components of ABCD test consist of a silicone mold and an invar ring as shown in Figure 2.2 (a). During testing, asphalt binder is firstly poured into the gap between silicone mold and invar ring. Then the ring with the specimen is put in a cooling chamber and the temperature is steadily decreased. Since the coefficients of thermal expansion/contraction of asphalt binders and metals differs to each other greatly, the binder specimen contracts and compresses the ABCD ring as temperatures goes down. Sensors inside the ABCD ring measure and record the strain throughout the test. When the binder specimen cracks, the strain is relieved abruptly, the temperature at that moment is the ABCD cracking temperature as indicated in Figure 2.2 (b). It was reported that the ABCD test shows similar or better correlation with field cracking index and mixture Thermal Stress Restrained Specimen Test (TSRST) than the AASHTO M320 test (BBR and DTT) (Kim, 2007).

(a)                                                                     (b)

Figure 2.2 (a) Test setup for ABCD test, and (b) typical test results (Kim et al., 2007).
4 mm DSR test was developed by Western Research Institute to measure the low temperature rheological properties of binder (Sui et al., 2010; Farrar et al., 2013). The asphalt specimen is tested by DSR at low temperature (5°C to -40°C) using 4mm diameter parallel plates and a 1.75 mm gap, as shown in Figure 2.3. The 4mm DSR test compares well with BBR tests, but it requires less asphalt sample amount, and is faster and more reliable than BBR test, therefore, the 4mm DSR may replace the BBR test for low temperature evaluation (Farrar et al., 2013).

![Figure 2.3. (a) Test setup for 4mm DSR test, and (b) comparison between 4mm DSR and BBR test (Sui et al., 2010).](image)

Penetration index and penetration-viscosity number were also considered to be the asphalt properties to affect low temperature cracking (Kim et al., 2006). Asphalts with high penetration index or low penetration-viscosity number are considered perform better at low temperature.
2.1.2.6 Effect of Asphalt Mixture Property

The parameters and test methods of asphalt mixture thermal fracture properties in general include tensile strength and creep compliance measured from indirect tensile (IDT) test, critical strain energy release rate ($J_c$) obtained from semi-circular notched bending (SCB) test, fracture energy and fracture work density measured from IDT test, as well as thermal stress tested from thermal stress restrained specimen test (TSRST).

The indirect tensile (IDT) strength and creep compliance tests were developed during the Strategic Highway Research Program (SHRP) to characterize the resistance of HMA to low-temperature cracking (Buttlar and Roque, 1994). Currently, the IDT creep compliance and strength tests are considered the most widely used parameters for predicting the low-temperature performance of asphalt concrete mixtures. The IDT strength is conducted by loading a cylindrical specimen across its vertical diametral plane at a specified rate of deformation (typically 50mm/min.) and test temperature. The peak load at failure is recorded and used to calculate the IDT strength of the specimen. Creep compliance is defined as the time-dependent strain divided by the applied stress, which was obtained by applying a static load of fixed magnitude along the diametral axis of a specimen. The horizontal and vertical deformations measured near the center of the specimen are used to calculate a tensile creep compliance as a function of time. Detailed testing procedure can be referred in ASTM D6931 and AASHTO T322.

Critical strain energy release rate ($J_c$) is another widely accepted parameter that used to describe fracture property of asphalt mixture at low temperature (Little and Mahboub, 1985). The $J_c$ is determined by elastic-plastic fracture mechanics (EPFM) method by using semi-
circular notched bending (SCB) fracture test. Detailed test procedures can be referred in accordance with AASHTO TP 105. The $J_c$ has good correlation with field cracking performance data that obtained from the Louisiana Pavement Management System (Kim et al., 2012). In addition, $J_c$ seems to be related to other low temperature related parameters, like IDT strength (Dongre et al., 1989). The sensitivity of $J_c$ values makes it a good parameter to distinguish asphalt mixtures with different sources at varied low temperatures (Dongre et al., 1989). $J_c$ was also found to be sensitive to mix aggregate gradations, aggregate shape and surface texture, as well as the asphalt binder content and asphalt binder grades (Bayomy et al., 2006).

Fracture energy can also be utilized as a parameter to describe the fracture resistance of asphalt concrete. The fracture energy refers to the energy required to create a unit surface area of a crack. It is particularly useful in the evaluation of mixtures with ductile binders, and has been shown to discriminate between these materials more broadly than the IDT strength (Wagoner et al., 2006). In laboratory, fracture energy can be determined by dividing area under load-CMOD (crack mouth opening displacement) curve by specimen thickness and initial ligament length. Detailed procedure is available in ASTM D7313 or AASHTO TP105.

Work density at low temperature is also considered as a fracture factor. Work density is defined as the fracture work divided by the volume of the specimen. Fracture work is the entire area under the load versus vertical displacement curve until load returns to zero. As a fracture property, work density describes both the strength and the ductility of a material. It was found to have a good agreement with the fatigue performance of Accelerated Loading Facility (ALF) pavements (Wen, 2013) and was also a promising indicator for field cracking performance (Shen et al., 2013).
Thermal stress can be measured by many relaxation experiments in laboratory and the most popular one is thermal stress restrained specimen test (TSRST) ([Monismith et al., 1965; Sugawara and Moriyoshi, 1984; Arand, 1987; Zubeck et al., 1996]). In the TSRST, a thermal stress is induced in a beam or cylindrical asphalt concrete specimen by cooling it at a constant cooling rate. The sample is restrained and the sample length is kept constant. A monotonic decreased temperature was applied until the specimen fractures. The stress recorded at fracture point is called fracture strength, the temperature at fracture point is called fracture temperature. The fracture temperature is highly affected by climatic condition ([Aschenbrener, 1995]). Test results show that thermal stress was build up as the temperature decreased in the restrained specimen ([Tabatabaee et al., 2012]). As the test continued, the stress gradually started to relax. Stress build up is significantly affected by cooling rate. Cooling rate is a critical control parameter when conducting TSRST. The cooling rate in the field seldom exceeds 2.7°C/h ([Jung and Vinson, 1994; Bouldin et al., 2000]), however, TSRST are conducted at cooling rates of 10°C/h or even higher to reduce testing time.

Unlike low temperature thermal cracking, the parameters and laboratory evaluation related to thermal fatigue cracking is relatively limited, and testing results from different research are usually conflicted. Epps (1999) measured the mixture resistance to thermal fatigue in laboratory by using a load-induced slow-cycle beam fatigue test. This equipment was used since the author believed that failure by thermal fatigue is similar to failure to load-induced fatigue, with daily temperature cycles equivalent to traffic-induced loading of the pavement. It was found that the asphalt mixture has limited capacity for large thermal fluctuations at relatively low temperatures, indicating that thermal fatigue could contribute to field transverse cracking. However, it was also believed that thermal fatigue could not lead to enough damage to initiate
and propagation the crack. Based on laboratory, Sugawara and Moriyoshi pointed that fatigue service life (age) of HMA may be much less in a thermal mode than in a load-associated mode (1984). With the aid of direct tensile creep test (DTCT) and TSRST, Jackson and Vinson (1996) concluded that fatigue cracking was not observed in any of the cyclic cooling tests conducted. The authors suggested that thermal fatigue cracking is not a significant mode of distress in the absence of environmental aging. They believed that distress typically attributed to thermal fatigue is actually a special case of low-temperature cracking or transverse cracking in warm areas are base or subgrade related problems.

2.1.2.7 Effect of Other Related Material Properties

Both rapid short-term aging during plant mixing and construction and slower long-term aging in the field stiffen the mixture and increase the stress induced on cooling (Epps, 1999; Epps, 2000). It is also recommended to use long-term oven aging asphalt instead of short-term oven aging asphalt since thermal fatigue is not expected to occur in the first few years of pavement service (Epps, 1999).

Results also indicate that thermal cracking performance of HMA is not sensitive to asphalt content and aggregate quality in terms of low temperature thermal cracking (Aschenbrener, 1995). However, the increased binder content may increase the thermal fatigue resistance of mixture (Epps, 1999).

A summary of factors that could potentially influence thermal cracking performance of asphalt concrete pavement are shown in table 2.1.
Table 2.1 Summary of Potential Factors that Affect Thermal Cracking Performance of Asphalt Concrete Pavement

<table>
<thead>
<tr>
<th>General Factors</th>
<th>Asphalt Binder Factors</th>
<th>Asphalt Mixture Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bonding Condition</td>
<td>$m$-value</td>
<td>Creep Compliance</td>
</tr>
<tr>
<td>Total HMA Thickness</td>
<td>Stiffness</td>
<td>IDT Strength</td>
</tr>
<tr>
<td>Overlay Thickness</td>
<td>Thermal Cracking Temperature</td>
<td>Critical Strain Energy Release Rate</td>
</tr>
<tr>
<td>Climatic Condition</td>
<td>Tensile Strength</td>
<td>Fracture Energy</td>
</tr>
<tr>
<td>Service Life (age)</td>
<td>Fracture Toughness</td>
<td>Work Density</td>
</tr>
<tr>
<td></td>
<td>Penetration Index</td>
<td>Fracture Temperature</td>
</tr>
<tr>
<td></td>
<td>Penetration-Viscosity Number</td>
<td>Aging</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Gradation</td>
</tr>
</tbody>
</table>

2.1.3 Prediction Models for Thermal Cracking

Significant effort has been put forward to establish prediction models to correlate field thermal cracking with material or other properties. The current thermal cracking models may be categorized as empirical model or mechanistically model. Empirical models, which are typically developed by using regression method based on field distress data, are useful in identifying key factors that affecting thermal cracking. However, they are limited to the data set on which are based, and also cannot be used to explain the mechanism of thermal cracking. Conversely, mechanistic-based models can describe the thermal cracking at a more fundamental level. However, most of the current mechanistic-based models have more or less limitations when used to predict field performance.

2.1.3.1 Empirical Model for Thermal Cracking

In general, regression models are easily to be used once they have been developed. Some of the factors are easy to be obtained while some others may require complicated laboratory testing. Therefore, the use of a particular model really depends on the agency’s ability to obtain all the inputs.

Fromm and Phang (1972) conducted field distress survey for 33 pavement sections in Ontario and developed a crack index (CI) to measure the crack severity. Extensive field cores
were taken and laboratory testing were performed on these cores to obtain material properties. Regression models were established between CI and material properties and field information. The key factors that related to CI include viscosity ratio, freezing index, critical temperature, air voids, stripping rating, penetration of asphalt, content of asphaltenes, as well as percent passing #200 for granular base and asphalt concrete.

Carpenter and Lytton (1977) provided a cracking spacing prediction model, and the parameters included are maximum tensile stress, coefficient of thermal contraction, young’s modulus, and temperature drop. The model shows that crack must occur at one-half intervals of the maximum spacing since this is where the maximum tensile stress will occur. This divides the crack spacing curve into distinct intervals which require a definite temperature drop below freezing, ΔT, to produce a new crack spacing.

Based on field data from 26 airport pavements in Canada, Hass et al. (1987) developed regression models that were used to predict average field transverse crack spacing. The regression models were found to relate with climatic effect (minimum temperature, MINTEMP), asphalt property (Pen Vis Number, PVN), mixture property (coefficient of thermal contraction, COEFFX) and pavement structure (thickness of asphalt concrete). The crack spacing was found as increased with increased PVN, minimum temperature and the total pavement thickness, while decreased with the increased coefficient of thermal contraction.

Models with different number of independent variables (one, two, three and four) were developed. MINTEMP was the best one-independent variable while MINTEMP and PVN were the best for the two-independent variable model. For the three-independent variable model,
MINTEMP, PVN and COEFFX were chosen. For the four-independent variable model, total pavement thickness was added.

The alternative models would be applicable if (a) the PVN were not available, and/or (b) the designer does not wish to use the PVN value, and/or c) the designer wishes to use other factors instead. As well, since the coefficient of thermal contraction is largely a function of the aggregate available, over which there may be no control, the coefficient would probably be the same for any bitumen used at a given site. Therefore, the designer might not wish to include it.

The common limitations of the above empirical models are (a) time was not included as a variable, and (b) they were developed primarily based on asphalt properties instead of asphalt mixture properties.

2.1.3.2 Mechanistic Models for Thermal Cracking

In contrast to empirical models, mechanistic models are typically developed at a more fundamental level. However, mechanistic models are usually more complicate and require more mathematical computations.

Monismith et al. (1965) indicated that thermal stress may be a contributing factor to transverse cracking and developed a theoretical model to calculate thermal stress. The model includes parameters such as creep compliance and coefficient of thermal expansion. The prediction results are highly affected by temperature change. The prediction thermal stress has proved to agree well with laboratory measured values.

Hills and Brien (1966) developed theoretical models that can determine at which temperature an asphalt mixture will fracture. However, their models are elastic-based and do not predict the crack spacing or crack amount but only used to determine the temperature at which
cracks may form. The Hills and Brien approach was later implemented by Finn et al. (1986) in the computer program COLD by adding some new features, e.g., use pseudo-elastic to replace elastic property. Also the variability of strength with temperature was accounted, and therefore reliability was included by using different percentile strength values. However, the COLD program still cannot predict the crack spacing or crack amount with time.

Roque and Ruth (1990) developed a physical model to predict thermal crack based on 9 variables including pavement temperature, pavement geometry, asphalt constant power viscosity, layer moduli, Poisson’s ratio, truck load, coefficient of thermal contraction, asphalt content and air voids. But some important parameters that relate to low temperature mixture properties, for instance, tensile strength and fracture energy, were not obtained from testing while determined by asphalt constant power viscosity.

Later in SHRP contract A-005, a mechanics-based performance model, Superpave Thermal Crack Model (TCMODEL), was developed to predict field thermal crack by considering viscoelastic property of asphalt mixture (Hiltunen and Roque, 1994). The TCMODEL consists of three primary parts: the stress intensity factor model, the crack depth model, and the crack amount model. The stress intensity factor model (CRACKTIP) is a two-dimensional finite element (FEM) program that models a single vertical crack in the asphaltic concrete layer via a crack tip element, and predicts the stress at the tip of local vertical crack using the far-field stress. Based upon the stress at the crack tip, a linear elastic fracture mechanics based (Paris’s law) crack depth model was used to predict the amount of crack propagation due to the imposed stress to a given cooling cycle. Finally, by using probability concept, the crack amount model predicts the number (or frequency) of thermal cracks per unit
length of pavement from the depth of the local vertical crack and the assumed crack depth
distribution function (Lytton et al., 1993). The three steps used for thermal cracking prediction is
shown in Figure 2.4.

The model required laboratory tested mixture properties to calculate stress and strain and
to estimate the crack amount with time. Inputs to the TCMODEL include creep compliance and
strength from IDT test, coefficient of expansion/contraction (CTE) of asphalt mixtures, as well
as pavement structure and site-specific weather data. The CTE and relaxation modulus
determined from IDT creep compliance are used to determine how much thermal stress will be
developed at a given temperature. By comparing this thermal stress with strength values, a
cracking temperature can be determined (Kim et al., 2010b). The measured creep compliance and
IDT strength are also used to calculate Pair law fracture parameters A and n. The TCMODEL
was further modified and recalibrated by using more field specimens and thermal crack
observations, and was incorporated into the 2002 MEPDG program (Witczak et al., 2000).
The major assumptions of the TCMODEL include (1) It is assumed that within the surface layer there are potential crack sites uniformly spaced at a distance $S$. At each of these crack sites the induced thermal stresses can potentially cause a crack to propagate through the surface layer, at which time it is assumed that a transverse crack will be visible on the pavement surface, and (2) It is assumed that each of these cracks can propagate at different rates due to spatial variation of the relevant material properties within the surface layer (ARA, 2003).

Although being one of the most popular field thermal cracking prediction models and having been validated by a number of field measurements, the TCMODEL has some limitations. Specifically,
1. The TCMODEL is not purely a mechanistic model but also involved some empiricism. For instance, the parameters $\kappa$, $\beta_1$ and $\sigma$ are determined by comparing prediction results with field observations (Marasteanu et al., 2004).

2. Sometimes the model can give extremely high or low predicted results (crack length) which could be related to one of the assumptions in the model: a crack is not counted as a crack until it propagates through the entire depth of asphalt pavement surface layer.

3. The link between fracture parameters (Pair Law parameters A and n) to IDT strength and m-value from creep compliance are based on unmodified asphalt. Therefore, the model may not suitable for non-conventional asphalt binder like rubber asphalt mixture and asphalt mixture with recycled asphalt (Zborowski, 2007; Dave et al., 2013).

4. TCMODEL assumes that the pavement structure has only one AC layer. If the pavement consists of two or more AC layers the user has to decide which material properties should be input into the model. This is a problem in case when materials used in pavement layers differ significantly (Zborowski, 2007).

5. The pavement may experience no crack in one cooling cycle. Depends on how many sublayers it has, it would require several cooling cycles to propagate the entire pavement surface layer.

6. The modelling of Pair's law approach is based on one dimension.

7. By definition, the mean crack depth (C) can never greater than the pavement thickness (D) and therefore, the analysis will end when $\log C$ equals $\log D$. At this
point, half the cracks are fully open and the other half are not. Hiltunen and Roque (1994) suggest doubling the number of cracks obtained from the analysis to overcome this deficiency.

8. The IDT strength used in the model may not be able to take into account the modified asphalt mixture. It was pointed out that the IDT strength of mixtures with rubber modified asphalt are consequently lower when compared to the conventional HMA mixtures, yet these same asphalt rubber mixtures proved to be superior to HMA mixtures in their resistance to thermal cracking (Zborowski, 2007). A fracture energy-based model could be more appropriate to thermal resistance of rubber modified asphalt mixture and has been validated by field observations (Zborowski, 2007).

Other researchers (Marasteanu et al., 2012; Dave, 2013) made some improvements for the current TCMODEL by using a Cohesive Zone based finite element analysis, and developed a new software IlliTC. Compared with TCMODEL, the IlliTC has some new features in the following situations: (a) the 1D Pairs-law phenomenological modelling approach was replaced with 2D; (b) cohesive zone approach was included to more correctly capture the physics of cracking in a quasi-brittle, and (c) both material strength and “ductility” (fracture energy) were considered in computing crack initiation and propagation. However, the field validation data is very limited for IlliTC and does not match well with prediction results (Dave, 2013).

TCMODEL was also used to evaluate modified asphalt based on 16 field pavement sections selected in Alaska, where both conventional and polymer modified asphalt pavements were considered (Raad et al., 1998). Results show that TCMODEL is very conservative and
predict less crack compared with field measurements. Field data and predictions by using other prediction models (Hass et al., 1987; McHattie et al., 1980; Kanerva, 1993) are also poor related. An improved crack propagation model was developed by the authors using minimum air temperature, TSRST fracture temperature and strength, and pavement age to fit the field data for both conventional and polymer modified sections.

Zubeck and Vinson (1996) developed a probabilistic model to predict the thermal cracking spacing as a function of time using thermal stress restrained specimen test results, pavement thickness, bulk density, pavement restraint condition, and air temperature. The effect of aging was also included in the model. The model can predict crack spacing and its variation with time and yields the reliability of the design with regard to minimum acceptable crack spacing criterion defined by road authorities.

Kirkner and Shen (1999) developed a model called Fictitious Crack Model (FCM) to predict thermal cracking of asphalt pavements. The core of this model is the concept of fictitious crack, which is an imaginary line, governed by a softening type stress crack-opening displacement constitutive relationship. Kirkner and Shen (1999) indicated that the interface friction is very important in terms of thermal cracking development. Specifically, (a) large frictional constraint could delay the formation of open cracks; (b) both the crack spacing and width of cracks decreased as frictional constraint between the asphalt and base layers is increased, and (c) when there is high frictional constrain between layers, a greater temperature drop is required for a fictitious crack to develop into an open crack.

Marasteanu, et al., (2004) developed a thermal cracking model by considering the entire pavement cross section as an integrated engineering system. The interface friction between
layers was also considered and the traffic effects on crack development was taken into account. The heat transfer simulation throughout the pavement structure was also considered. A comparison between four full-scale pavement test sections at Mn/ROAD and prediction results showed that the thermal cracking model works well.

Bouldin et al. (2000) developed a comprehensive semi-empirical mechanistic model to determine the critical cracking temperature of asphalt pavements. The model uses only creep stiffness $S$ from BBR to predict thermal stress in the binder. The advantages of using only binder properties include (a) binder testing is easy and well-standardized; (b) binder properties used in the model are available during routine Superpave specification testing, and (c) the model can be easily implemented in the binder specification. However, without considering mixture properties and other factors (i.e., pavement structure), an extensive calibration is required to binder thermal stress to mixture stress in the pavement. The limited validation use field transverse cracking showed good agreement between field data and prediction results.

Tabatabaee et al. (2012) developed a theoretical approach to calculate thermal stress buildup in mixtures by considering glass transition temperature. If the temperature is close or near glass transition temperature, the asphalt could become more brittle and the time-dependent behavior in this temperature range can have negative effect on actual binder performance. The approach combines relaxation modulus master curves, the Wiliam-Landel-Ferry equation for time-temperature superposition of thermorheological simple material, Boltzmann’s superposition principle, and a sub-model describing the isothermal contraction of asphalt as a continuous function of condition time and temperature. Results indicate that thermal stress calculations can be more accurate accounting for glass transition behavior and the associated time-dependent
strain caused by physical hardening. They also concluded that thermal strain is not always linear related to temperature, instead, thermal strains in mixtures are significantly depend on temperature and isothermal conditioning time.

Unlike the models summarized above for low temperature thermal cracking, there is very limited number of models for prediction of thermal fatigue cracking. Lytton, Shanmugham and Garrett (1982; 1983) worked on thermal fatigue life prediction based on fracture mechanics. They developed a model to predict the thermal fatigue cracking spacing or occurrence caused by temperature cycling below 25°C. The model requires daily temperature data and climatic factors, such as solar radiation, air temperature and wind speed, to calculate the temperature distribution in the surface layer. The aging effect was also included. The principles of viscoelastic fracture mechanics are used to accumulate damage under repeated temperature cycles. Miner’s hypothesis is used to calculate the damage produced by any one thermal cycle. However, the model has not widely validated by field pavement sections.

2.2 Reflective Cracking

The overlay paving of Hot Mix Asphalt (or Warm Mix Asphalt) on existing pavement structure has being a popular strategy for pavement maintenance or rehabilitation for many years. Many agencies place HMA overlays on deteriorating jointed or cracked PCC to improve the smoothness, protect the existing pavement structure, and reduce noise. However, the new overlay may not perform as expected due to the existing cracking, which can reflect through the overlay in 1 to 5 years (Elseifi et al., 2011; Makowski et al., 2005), or even after the first or second winter after rehabilitation (Buttlar and Bozkurt, 2000; Pais and Pereira, 2000). Consequently,
reflective transverse cracking is considered one of the most typical distresses given the nature of existing jointed concrete pavements or cracked asphalt pavements.

2.2.1 Mechanism of Reflective Cracking

In general, the mechanism that induce reflective cracking can be categorized into traffic factor and thermal factor, as shown in Figure 2.5.

For traffic load, it could cause vertical/horizontal pavement movement under mode I (bending) or mode II (shearing) stresses, or a combination of mode I and II. High stress is induced above existing joint/crack and once it exceeds the HMA strength, the crack initiates and propagates through the overlay (De Bondt, 1998). Pais and Pereira (2000) summarized that mode I stress could happen when a wheel load passes over a transverse cracking, and mode II stress could happen when a wheel load is approaches a transverse cracking. Compared with thermal stresses, traffic-induced stresses often occur more rapidly and could be more damaging due to the accumulation of residual stress and the inability of HMA to relax (Baek, 2010).

The second driving force for reflective cracking is daily temperature change result in thermal contractions and expansions in the existing pavement layers. The thermal contractions and expansions then induced concentrated tensile stress or strain in the overlay above the existing crack or joint (Bijsterveld and Bondt, 2004). The thermal related reflective cracking is almost only linked to mode I stress (Kim and Buttlar, 2002) and is considered to be a fatigue process caused by repeated, thermal induced movement of the existing pavement layer (Nunn, 1989).

Burt (1987) and Nunn (1989) believed that for existing HMA overlay placed above existing PCC with joints, the transverse crack could initiate at the overlay surface and propagate
downwards to meet the cracks in the existing PCC. Nunn (1989) indicates that this type of cracking could be caused by a combination of thermal contraction and warping of pavement under cold weather conditions, when the asphalt material is brittle and cannot resist tensile strain caused thermal contraction. However, this assumption has not been validated. Lytton et al. (2010) assumed that if the stress-concentrating effect of the existing joints or cracks has been nullified by some means, a secondary effect of the existing joints or cracks is that a maximum deflection of the pavement under a wheel load will occur at the crack. However, it was not explained how the effects of existing cracks can be nullified.

![Figure 2.5 Mechanisms of reflective cracking (Nunn, 1989).](image)

**Figure 2.5 Mechanisms of reflective cracking (Nunn, 1989).**

### 2.2.2 Factors Affecting Reflective Cracking

Many studies have been conducted to determine key factors that are related to reflective cracking, including bonding condition, pavement structure, asphalt/asphalt mixture properties, climatic condition and other factors.
2.2.2.1 Effect of Bonding Condition

Typically, the propagation rate of reflective cracking becomes slower as the bonding stiffness declined (De Bondt, 1998). When good bonding exists between existing pavement and overlay, the movement tendency of existing pavement is restrained by good bonding and caused high stress concentration at overlay bottom. In contrast, the stress concentration can be highly reduced at overlay bottom if the existing pavement can move freely to some extent under normal/de-bonding condition.

2.2.2.2 Effect of Pavement Structure

Overlay thickness can have a significant effect on both of thermal and traffic reflective cracking. Under traffic load, vertical crack activity is greatly reduced with the increased overly thickness (Pais and Pereira, 2000). A thicker HMA overlay could better insulate the existing pavement from daily temperature changes and can help to prevent thermal reflective cracking (Foulkes and Kennedy, 1986; Bozkurt and Buttlar 2002; Dave and Buttlar 2010). Results indicate that a PCC slab covered by 6 inch thick asphalt concrete overlay can reduce as much as 2.5°C temperature variation at top surface of the PCC slab (Bozkurt and Buttlar 2002). Laboratory test results also showed that the thermal induced fatigue is likely to be the most significant for roads with less than 100mm in thickness (Bijsterveld and Bondt, 2004). Accordingly, with increased overlay thickness, the strain, tensile stress and shear stress at overlay bottom above existing crack decreased (Kim and Buttlar, 2002; Bijsterveld and Bondt, 2004), and the reflective cracking life is greatly increased (Wu et al., 2012).
2.2.2.3 Effect of Asphalt Binder

Based on laboratory test and finite element simulation, it was concluded that thermal reflective crack could be avoided by using softer binders and higher binder contents (Foulkes and Kennedy, 1986; Hu et al., 2011). Increased effective binder content by volume ($V_{be}$) and asphalt film thickness were also found to be able to increase the number of cycles to failure in Texas overlay test (Hu et al., 2011).

In addition, asphalt is a viscoelastic material, its structural responses depend not only on the magnitude of the applied load but on the load duration as well (Khazanovich et al., 2012). The loading duration for traffic loads depends on the vehicle speed. If vehicle speed is approximately 60 mph, then the loading duration ranges between 0.01 and 0.05 seconds. However, the duration of temperature loading is significantly longer.

2.2.2.4 Effect of Asphalt Mixture

Very limited asphalt mixture properties have been reported to well correlate with reflective cracking. The reported parameters consisting of mixture strength, dynamic modulus, fracture properties A and n (Hu et al., 2010), as well as mixture aging.

The reflective cracking initiates when the induced tensile stress directly above joint or crack exceeds the tensile strength of the asphalt overlay. Therefore, asphalt overlay with higher tensile strength is more resistant to reflective cracking.

The dynamic modulus is used to calculate the pavement crack response stress intensity factor (SIF), one of the key parameters in Paris’s law (Lytton et al., 2010). The SIF can be calculated in terms of thermal, shearing and bending separately.
The fracture property parameters $A$ and $n$ are key factors in Paris’s Law-based reflective prediction model, which can be obtained from mixture creep compliance and IDT strength.

For the reflective cracking that initiates from pavement surface, mixture aging could be one of the critical reasons (Von Quintus et al., 2009).

A statistical analysis indicates that for Texas overlay testing, the following factors are not statistical significant to affect the failure cycles: (1) air voids; (2) asphalt absorption, and (3) surface area of aggregate (Hu et al., 2011).

2.2.2.5 Effect of Climatic Condition

The temperature gradient distribution significantly affects the thermal reflective cracking. The positive temperature gradient, that is, when the temperature on the top is higher than that at the bottom, produced the greatest tensile stress at AC bottom (Wang et al., 2013). The negative temperature gradient created the lowest tensile stress at the AC bottom. Therefore, the positive temperature gradient increases the risk for reflective cracking, and the negative temperature gradient increases the potential for thermal cracking. The faster cooling cycle will increase the tensile and shear stress at overlay bottom as well (Kim and Buttlar, 2002).

2.2.2.6 Effect of Other Related Factors

More traffic especially truck load can increase the potential of reflective crack. The lower modulus of base and subgrade could increase stress concentration at overlay bottom and increase the reflective cracking potential (Kim and Buttlar, 2002; Baek and Al-Qadi, 2011).

A summary of factors that could potentially influence reflective cracking of asphalt concrete pavement is shown in table 2.2.
2.2.3 Laboratory/Field/Simulation Evaluation of Reflective Cracking

Many laboratory testing and field evaluation were performed to better understand the initiation and propagation of reflective cracking. The laboratory testing can be divided into traffic load only (i.e., TTI overlay tester) and combined traffic and thermal load (i.e., wheel reflective cracking test). The field evaluation confirms that pavement structure has significant effects of reflective cracking.

The TTI overlay tester was designed by Germann and Lytton (1979) to simulate the opening and closing of joints or cracks, which are the main driving force inducing reflective crack initiation and propagation. The key parts of the apparatus, consist of two steel plates, one fixed and the other movable horizontally to simulate the opening and closing of joints or cracks in the old pavements beneath an overlay. Many applications indicate that overlay testers have the potential to characterize the reflective cracking resistance of asphalt mixtures (Germann and Lytton, 1979; Zhou, 2005). The main advantages of this test include 1) the test is rapid, and poor samples fail in minutes; 2) it characterizes both crack initiation and crack propagation properties of asphalt mixtures; 3) the overlay test is repeatable, and 4) it correlated with the field performance (Zhou, 2005).

<table>
<thead>
<tr>
<th>General Factors</th>
<th>Asphalt Binder Factors</th>
<th>Asphalt Mixture Factors</th>
</tr>
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<tbody>
<tr>
<td>Bonding Condition</td>
<td>Binder Stiffness</td>
<td>Mixture Stiffness</td>
</tr>
<tr>
<td>Overlay Thickness</td>
<td>Asphalt Content</td>
<td>Tensile Strength</td>
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<tr>
<td>Temperature Gradient</td>
<td>Effective Binder Content by Volume</td>
<td>Aging</td>
</tr>
<tr>
<td>Cooling Rate</td>
<td>Asphalt Film Thickness</td>
<td>Dynamic Modulus</td>
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<tr>
<td>Truck Load</td>
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<td>Fracture Parameter A and n</td>
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<td>Modulus of Base and Subgrade</td>
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Gallego and Prieto (2006) reported a piece of laboratory equipment known as wheel reflective cracking (WRC) to assess reflective cracking in asphalt overlays. It is pointed out that the WRC device enables simultaneous laboratory simulation of the mechanisms causing distress leading to reflective cracking, namely, horizontal movements due to thermal contraction and vertical movement due to traffic load. Test results were also found to have adequate consistency with Spanish experience on full-scale test sections. Overlay with the geosynthetic was also evaluated which shows a 1.5 times better strength than the same overlay without the geosynthetic.

A common method of determining the differential vertical movements is by measuring the load transfer efficiency (LTE) at the joints. Lee et al. (2004) conducted a rolling dynamic deflectometer (RDD) to evaluate the movement of joints in concrete pavements. Continuous deflection profiles can be obtained with the RDD to estimate the LTE. It was found that the LTE measured from the RDD is “fairly” consistent with the LTE measured with the FWD. A field project has validated the effectiveness of estimating LTE using RDD (Lee et al., 2004).

Pais and Pereira (2000) measured transverse cracks in 14 cross-sections in the field before placing overlay. The crack activity of pavement after placing overlay was analyzed using finite element method. Both crack activity at horizontal direction (to measure tensile) and vertical direction (to measure shear) were simulated. Results show that after overlay, the horizontal crack activity at overlay bottom was much smaller than that of vertical crack activity (cannot reach 50%). This indicates that shear stress could be more critical compare with tensile stress in terms of reflective cracking. The authors also found that vertical crack activity is greatly reduced with the increased overly thickness.
Bennert et al. (2009) analyzed the causes for early reflection cracking based on a field pavement section from Interstate 495. A 2-in. overlay section and a 5-in. HMA overlay section were constructed on the top of PCC. Reflection cracking was observed within the first 2 months for 2-in. overlay and about 6 months for 5-in. overlay. The field and laboratory testing reveal that for the 2-in. overlay section, a combination of high vertical and horizontal joint movement, along with a mixture that was not suitable for fatigue resistance, resulted in early reflective cracking. For the 5-in. overlay section, the poor fatigue resistance of the leveling and intermediate course mix was not able to withstand the horizontal and vertical movements as the PCC joint and crack area. The authors recommended that a more thorough analysis of the PCC joint movements must be considered before an HMA overlay selection. It is also important that typical HMA may not be suitable for all applications and the designing and selecting a flexible overlay on composite pavements should take into consideration a system approach.

A fully-scale pavement test section was constructed (Yin and Barbagallo, 2013) to represent typical airport overlay structure and evaluate reflective cracking. Both traffic load and thermal load was applied. Results reveal that once reflective cracking initiated, crack developed very fast. When the crack length reached a certain level, the crack propagation slowed.

Cohesive zone method (CZM) was used recently combined with FE analysis to study the reflective cracking. CZM could be better in simulating reflective cracking since it can consider quasi-brittle material, as a finite length scale associated with the fracturing process is considered. (Dave et al., 2007). Baek (2010) studied the behavior of traffic-induced reflective cracking using FEM in terms of HMA overlay placed on existing Joint Concrete Pavement (JCP). Both with and without interlayer systems conditions were evaluated. The fracture behavior of HMA was
simulated use a combination of viscoelastic model and a bilinear cohesive zone model (CZM). Using the bilinear CZM, reflective cracking initiation and propagation were simulated. Various levels of overload were applied to force reflective cracking development by one pass of load application. The study concluded that the sand mix interlayer system extended the service life (age) of the HMA overlay regarding reflective cracking due to its relatively high fracture energy.

2.2.4 Prediction Models for Reflective Cracking

Unlike thermal cracking, predictive models for reflective cracking are relatively limited and it is hard to find one model that is widely accepted. Generally, these models can be divided into traffic-based and thermal-based.

2.2.4.1 Traffic-based Reflective Cracking Predictive Models

Eltahan and Lytton (2000) studied the combined effects of cracked and un-cracked areas in the original pavement beneath the overlay to the prediction of reflective crack propagation. The derivation of crack propagation model based on fracture mechanics was presented. The predicted cracking was comparable with the observed cracking. The crack initiation model is able to predict the first time cracking is visible within the zone around that deterioration. However, field data was limited to Florida area and more data collection is required to validate the models.

Sousa (2002) developed a shear stress based mechanistic-empirical method to design HMA overlays to resist reflective cracking. The model can calculate average shear stresses above the crack zone as well as to model the crack activity before and after the overlay. Field validation showed excellent correlation between the values of the measured and predicted crack activity. A statistical mathematical model was also successfully developed to derive the amount of thickness
needed to control reflective cracking. The statistical model indicated that crack width did not appear to relate to the amount of overlay needed to control a reflective crack.

Hu et al. (2010) developed a fracture mechanics based reflection cracking model that can be integrated into an M-E design framework. The model can accurately model the three mechanisms of reflection cracking: bending, shearing, and thermal loading. The model was calibrated by several field pavement sections and good agreement between prediction results and field measurements were observed. A design tool based on the proposed model was developed which can optimize asphalt overlay thickness for any specific combination of traffic level, climate condition, asphalt overlay property and existing pavement condition.

The current AASHTOWare Pavement ME Design Program predicts reflection cracking using an empirical regression model (AASHTO, 2008). This regression equation estimates the percentage of area of cracks that propagate through the overlay, $RC$, as a function of time, $t$, using a sigmoid function

$$RC = \frac{100}{1 + e^{a + b \cdot C(t)}}$$  \hspace{1cm} (2.1)

Where $a$ and $b$ are regression fitting parameters and are a function of the effective HMA overlay thickness, $H_{eff}$, as shown in equations 2.2 and 2.3. Parameter $c$ equals to 1, and parameter $d$ is defined by the user to describe crack progression. Values for $H_{eff}$ is determined based on overlay thickness and detailed information can be found in 2008 MEPDG manual (AASHTO, 2008).

$$a = 3.5 + 0.75(H_{eff})$$  \hspace{1cm} (2.2)
The above model estimates the percentage of crack areas by using overlay thickness and service age only. The material properties are not accounted for in the model. The prediction ability of this model has received widely suspect since the prediction values cannot well match with field pavement sections.

Lytton et al. (2010) developed mechanistic-based models in NCHRP 1-41 project. The model accounts for the effects of reflection cracking on performance for use in mechanistic-empirical procedures for the analysis and design of HMA overlays. The model was based on the finite element plus fracture mechanics. The key concept in fracture mechanics is Paris and Erdogan’s Law for modeling crack propagation, particularly for fracture-micromechanics applications.

The factors that included are climate, HMA overlay thickness and material properties, PCC layer thickness and material properties, joint load transfer characteristics, and subgrade properties. The design program included a single mechanistic crack growth model with different sets of calibration coefficients for overlay, corresponding to different climatic zones or different pavement structure. Furthermore, the model considers three levels of cracking severity (low, medium, and high), and thus provides flexibility in examining the effects of reflective cracking on a given project/design.

The design program was calibrated using field data from over 400 pavement sections in 28 states and the four climatic zones in the United States. When the sum of bending crack increments reaches the point in the overlay where the bending stresses become compressive, that defines what is termed “Position I”. At crack lengths above the Position I, bending stresses no
longer contribute to the growth of cracks and crack growth is due only to thermal and shearing stresses. S-shaped empirical model was used to describe the amount and severity development of reflection cracking on an asphalt overlay.

Although the NCHRP 1-41 model appears to be theoretically sound and the most sophisticated model currently available to pavement engineers, its immediate implementation depends on the robustness of the NCHRP 1-41 software. The original version of the software produces unrealistic reflection cracking predictions for basic HMA-PCC projects (Khazanovich et al., 2012).

Vandenbossche and Barman (2010) proposed a criteria named relative stiffness which can be used to determine when field reflection cracking (MnROAD in I-94) developed in terms of PCC placed on existing HMA pavement. The relative stiffness is a function of elastic modulus, poisson’s ratio and pavement thickness. It was found that when relative stiffness is smaller than one, reflection cracking develops. The relative stiffness has good correlation with both temperature- and load-related stress. It was also observed that reflection crack develops earlier in the driving lane than in the passing lane, indicating that reflection cracking is influenced by the number of accumulated vehicle loads and that these cracks develop more quickly when the load-related stresses are higher or more frequent.

2.2.4.2 Thermal-based Reflective Cracking Predictive Models

Chang et al. (1976) used linear elastic and viscoelastic fracture mechanics to study the crack propagation in flexible pavement due to thermal contraction and expansion. The crack was assumed to be existing in the base course and the existing pavement layer. To reduce the thermal reflective cracking, the authors recommended (a) use thin overlay with soft asphalt binder and
low modulus of elasticity to server as a stress relieving medium overlay; (b) reduction of thermal expansion coefficient, α, would decrease the reflection cracking substantially.

Bennert and Maher (2008) used the combination of WIM-AVE and FWD to assess vertical joint deflection due to traffic load in the field and CTE test to determine the thermally induced horizontal deflections. The authors proposed a new reflective crack fatigue prediction method uses joint deflection data from FWD testing, axle load spectra data, and laboratory test results from the flexural beam fatigue and dynamic modulus test. Prediction results were found to have good correlation with field observations. The tensile strain used in the model are directly measured instead of being calculated on the basis of elastic theory. The required information (i.e., axle load spectra, FWD testing, dynamic modulus) is also consistent with typical information collected for use with the MEPDG software.

Dave and Buttlar (2010) used cohesive fracture model to capture the property of thermal reflective cracking. The authors indicate that the cohesive zone model can accurately and efficiently account for material response in the crack tip vicinity in the fracture process zone. Results show that the thermal reflective cracking of AC overlays were greatly affected by the opening of PCC joints and the curling of PCC slabs due to temperature differentials within the slabs. It is also pointed out that longer PCC slabs under the AC overlay will increase the thermal reflective cracking potential.

Khazanovich et al. (2013) developed a TPF-5 model for reflective cracking based on CalME model. The model incorporated the effect of temperature variation on reflective cracking growth. The differential energy of subgrade deformation was used characterize the effect of
traffic loading on crack deterioration. The final model is also incorporated into a program that interfaces with the MEPDG easily and quickly.

2.2.5 Treatment Method to Delay Reflection Crack

Compared with thermal cracking or other top-down cracking, reflective cracking is more serious. Once reflected to overlay surface, the pavement structure could be severely damaged and in most cases, the rehabilitation should be performed on the whole pavement structure. It was found that a reduction of transverse cracking spacing from 5 to 20m should result in a 5-year extension of service life (age), with a cost savings of $25,000 per two-lane kilometer, and proper and timely crack treatment can result in an extension of pavement life by 2 years and cost savings of $7,000 per lane kilometer (Tighe et al., 2003). Therefore, the prevention of reflective crack is of high importance. Many methods have been proposed to retard the propagation of reflection crack and the major ones include (1) saw and seal; (2) place interlayer between existing pavement and overlay, and (3) using cold in-place recycling (CIR).

2.2.5.1 Saw and Seal Method

Saw and seal method is one of the most popular used treatment methods to delay reflection crack in composite pavement structure. HMA overlay is firstly sawed at the exact locations of the joints in the concrete pavement. The saw cut portion is then sealed with a rubberized low-modulus sealant. The effectiveness of this treatment method was widely validated (Elseifi et al., 2011; Mallela et al., 2008). It was found that the saw and seal technique offers life extension of 4 years in Louisiana (Elseifi et al., 2011), and 18.5% and 34% life improvement for the 20- and 15-ft joint spacing in New York City (Mallela et al., 2008). The saw and seal treatment method was more effective in sections with low to medium traffic volumes. A finite element analysis
indicate that the saw cut in the HMA overlay allow it to move with the underlying layers and to
dissipate energy generated from expansion and contraction in the concrete layer and from wheel
loading without cracking.

2.2.5.2 Interlayer

The second most popular utilized technology is interlayer between HMA overlay and existing
pavement. The interlayer can be further classified into several types, for instance, interlayer
stress-absorbing composite (ISAC) and sand mix interlayer.

The ISAC system consists of a low-stiffness geotextile as the bottom layers, a
viscoelastic membrane layer as the core, and a very high stiffness geotextile for the upper layer.
The ISAC is able to effectively alleviate or mitigate the reflective cracking both in laboratory test
and field pavement sections (Dempsey, 2002; Baek et al., 2008).

Sand mix interlayer is a thin HMA layer composed of smaller aggregates and rich
polymer-modified asphalt binder. The sand mix interlayer can enhance the reflective cracking
resistance capacity by a factor of 1.17 to 2.45 under traffic load due to the high fracture
toughness of sand mix (Baek and Al-Qadi, 2011). The asphalt-rich HMA interlayer remains
intact and impermeable due to its highly flexible. Therefore, even the crack reflect and
eventually appears in the overlay, the asphalt-rich HMA interlayer can still protect the existing
pavement structure from surface water intrusion (Blankenship et al., 2004). The performance of
sand mix interlayer depends on the fracture energy and tensile strength. The tougher the sand
mix becomes, the better the performance may be (Baek and Al-Qadi, 2011).
It should be noted that the performance effectiveness of the interlayer was more dominant at an early stage of reflective cracking development and gradually decreased as reflective cracking developed (Baek and Al-Qadi, 2011).

Makowski et al. (2005) evaluated different technologies in absorbing joint movement and delaying reflective crack, including fine-aggregate, asphalt-rich and polymer-modified asphalt mix interlay. Four field Wisconsin projects are discussed. Results show that the use of performance-related specifications for flexural beam fatigue to resist cracking makes the use of the interlayer much more effective at delaying reflective cracking. It was also found that the movements in JPCP or JRCP may preclude the effectiveness of the interlay, so projects should be carefully screened or properly patched before the interlay is selected for use. The use of 98% reliable surface binder can further reduce reflective cracking. By using FEM analysis, Kim and Buttlar (2002) indicate that the use of base-isolating interlayer below the HMA overlay can significantly reduce tensile and shear stresses in the overlay bottom.

2.2.5.3 Cold In-place Recycling
The CIR material appears to be typically a stress-sensitive material, providing increased stiffness in response to increased load. This “flexibility” is likely an important factor in the observed good performance of these projects and the delay in the development of reflective cracking (Morian et al., 2004). Loria et al. (2008) evaluated the effectiveness of some popular technologies that used by Nevada DOT to reduce the impact of reflective cracking. Several projects were constructed under each category and the long-term field performance of reflective cracking was monitored. Research indicates that under Nevada’s conditions, CIR was one of the most effective treatments for reflective cracking of HMA overlays over existing HMA pavements.
It should be noted that the performance of reflective cracking treatment is highly
dependent on conditions of pavement before construction of the treatment. If severe alligator
cracking is observed in existing HMA pavement, it is recommended to reconstruction or full-
depth reclamation (Loria et al., 2008).

2.3 Top-down Longitudinal Cracking

Top-down longitudinal cracking is a very common distress type. In Florida, approximately 90
percent cracking type is surface-initiated longitudinal cracking (Myers and Roque, 1998). It is
also a dominated cracking type in European countries (Antunes and Nunn, 1999), Japan
(Matsuno et al., 1992) and Kenya (Wambura, 1999). As shown in Figure 2.6, the longitudinal
top-down cracking appeared to be straight and parallel with vehicle driving direction and of
lower severity than the reflective cracking (Harmelink, 2008), the crack depth is found to range
from several millimeters to the entire depth of pavement overlay (Myers, 1998; Uhlmeyer et al.,
2000; Holewinski et al., 2003). Most longitudinal top-down cracking are within or very close to
longitudinal cracking development into three stages. In stage I, a short single longitudinal crack
was observed. In stage II, short crack grows longer and parallel crack appeared. In stage III,
parallel cracks are connected via short transverse crack.
2.3.1 Mechanism of Top-down Longitudinal Cracking

Typically, the reasons for top-down longitudinal cracking can be attributed to traffic and thermal load. In terms of traffic load, the mechanism can be further classified as two major types (Roque et al., 2010; Roque and Kim, 2009). The first one is called surface tension caused by pavement bending that appeared far away from vehicle tires, which is usually used to explain the cracking initiation for pavement with thin to medium thickness. The second one is called near-surface tension caused by shear stress at the tire edges, which is usually applied to explain cracking initiation for thicker pavement. Compared with thermal stresses, traffic-induced stresses often occur more rapidly and could be more damaging due to the accumulation of residual stress and the inability of HMA to relax (Baek, 2010). The two mechanisms have been validated by several other researchers (Myers et al., 1998; Wang et al., 2003; Wang and Al-Qadi, 2010a). Surface tension usually causes crack initiate from pavement surface, while near-surface tension typically causes crack initiate from some distance below the pavement surface (Myers et al. 1998; Wang and Al-Qadi, 2010a; Zou et al., 2012; Wang et al., 2003).
Thermal load is another possible reason that induce longitudinal top-down cracking. Dauzats and Rampla (1987) estimated that surface initiated cracking was firstly induced by thermal stresses and then further propagated by traffic loads. As indicted by Myers and Roque (1998), the longitudinal cracking may be caused by a combined thermal stress and tire-induced transverse stress. Other researchers (Wang and Al-Qadi, 2010b; Zou, 2009) also pointed out that the potential surface-initiated cracking may increase with the increase of thermal load. Compared with summer time, the high thermal tensile stresses in winter increase top-down longitudinal cracking initiation potential (Svasdisant, 2003). Al-Qadi et al. (2005) quantified the strain amplitude associated with thermal loading in flexible pavement. Results indicated that thermal loading was associated with a very high strain range. The maximum value recorded was as high as 350 μm/m.

2.3.2 Factors Affecting Top-down Longitudinal Cracking

Many studies have been conducted to determine significant factors that contribute to top-down longitudinal cracking. In general, these factors can be divided into eight types: (1) pavement bonding condition; (2) pavement structure; (3) traffic load; (4) service life (age); (5) asphalt/asphalt mixture property; (6) aging effect; (7) effect of base and subgrade, and (8) moisture damage and segregation.

2.3.2.1 Effect of Bonding Condition

It is noted that most field cores with surface initiated cracking are structurally adequate based on field cores (Uhlmeyer et al., 2000). In another word, top-down longitudinal cracking is easier to be initiated for pavement structure with good bonding conditions between different asphalt layers.
2.3.2.2 Effect of Pavement Structure

Pavement structure can affect top-down longitudinal cracking in terms of both initiation and propagation. The initiation of longitudinal cracking could be caused by different stress types for different pavement thickness. Failure at pavement with thin to medium thickness could be caused by surface tension, while pavement with thick thickness could be caused by shear stress (Roque et al., 2010).

The crack propagation is also related to pavement structure but the relationship is more inconclusive. The top-down cracking model embedded in the current AASHTOWare Pavement ME Design program indicates that, the cracking amount decreases with the increase of pavement thickness (AASHTO, 2008; Rao et al., 2013). And it is also pointed out that a decrease of asphalt thickness increases the probability of top-down cracking (Kim and Roque, 2009). However, the assumption that crack increases with thinner pavement may be limited to specific thickness range. As shown in Figure 2.7, the plot indicates the relationship between LTPP field longitudinal cracking data based on 640 sections with asphalt layer thickness (ARA, 2004). As can be seen, the longitudinal cracking is not always decreases with increased asphalt layer thickness. The peak longitudinal cracking is observed with 4-7 inches layer thickness. Based on 98 field cores that were taken from 24 pavement sections with 3-8 service life (age) in State of Washington, Uhlmeyer et al. (2000) concluded that the top-down longitudinal crack depth and amount increases with the increase of asphalt pavement thickness.
2.3.2.3 Effect of Traffic Load

It is generally believed the longitudinal cracking increased with the accumulation of traffic load. It was found that using realistic load spectra during finite element analysis is critical to predict crack growth, as load positioning was found to be overriding contributor to crack propagation. Load wander must be considered for predicting failure and is critical in determining future design conditions (Myers, 2000).

2.3.2.4 Effect of Service Life (age)

The longitudinal crack is initiated at different service life (age) at different location. It was reported that the initiation time is 1-5 years in Japan, 3-5 years in France, 3-8 years in Washington State, 5-10 years in Florida and around 10 years in United Kingdom (Matsuno et al., 1992; Myers, 2000; Uhlmeyer et al., 2000).
2.3.2.5 Effect of Asphalt/Asphalt Mixture Property

Asphalt/asphalt mixture property can significantly affect top-down longitudinal cracking. A higher effective binder content by volume ($V_{be}$) can improve the mixture resistance to crack. Mixture with more fracture resistant ability (high tensile strength or high fracture energy) can reduce surface initiated longitudinal cracking (*Myers et al., 1998; Roque et al., 2010*). In addition, mixtures with higher air voids and coarse gradation result in earlier top-down cracking (*Freitas et al., 2005*). Aggregate gradation is also critical since crack in vertical direction mainly follows course aggregates (*Freitas, 2003*). Based on laboratory test, it was found that top-down cracking initiated at higher temperature (*Freitas, 2005; Wang, 2003*) and the rutted surface appears to contribute significantly to top-down cracking initiation.

It was also indicated that asphalt with better healing property and asphalt mixture with higher creep compliance rate are more resistant to top-down longitudinal cracking (*Roque et al., 2010*).

2.3.2.6 Effect of Material Aging

It is pointed out that the aging of asphalt binder has been attributed to be the major cause of top-down cracking, such as Hugo and Kennedy (*1985*) in South Africa, Wambura et al. (*1999*) in Kenya, and Gerritsen (*1987*) in Netherlands. Wambura (*1999*) indicates that top-down cracking was associated with the oxidation and hardening of the top new millimeters of the AC surface course. Svasdisant (*2002*) concluded that AC layers tend to crack from pavement surface easier with higher modulus due to temperature or aging. The temperature-induced stiffness gradients in the asphalt concrete layer had significant effects on the tensile response of surface cracks, and therefore must be included in the approach to pavement design (*Myers, 2000*).
2.3.2.7 Effect of Base and Subgrade

In general, the increase of base modulus could increase the probability of top-down cracking (Kim and Roque, 2009).

It was also believed that longitudinal crack in asphalt pavement could be reflected from subgrade caused by volumetric change of the expansive subgrade (Prozzi and Luo, 2007). Geogrids and lime are found to be able to prevent reflected longitudinal cracking due to shrinkage of expansive subgrade (Prozzi and Luo, 2007; Luo and Prozzi, 2009). A FEM simulation results showed that compared with pavement without geogrid, the pavement with geogrid can significantly reduce the stress intensity factor at the crack tip in the pavement (Prozzi and Luo, 2007). The FE analysis also indicate that the best place to install the geogrid should be at the interface of the lime-treated layer and the untreated soil, which is close to the initiation location of shrinkage cracks (Luo and Prozzi, 2009). At this location, the geogrid can effectively reduce the stress concentration at the upper crack tip.

2.3.2.8 Effect of Moisture Damage and Segregation

Moisture damage and segregation are found important factors that affect top-down cracking. The moisture damage and segregation of AC mixtures can cause reduction in the tensile strength and increase top-down cracking potential (Baladi et al., 2003). Field research indicates that many field cores taken from top-down cracking sections have visual signs of segregation (Harmelink and Aschenbrener, 2003). The field cores with moisture damage sections also exhibited a much faster reduction in the mixture fracture resistance, in contrast to field cores without moisture damage (Zou et al., 2013). If moisture is allowed into the pavement through the cracks the segregated area will be prone to rapid deterioration. Therefore, it is recommended to seal the
crack as soon as possible to prevent moisture from penetrating the pavement and deteriorating the pavement further (Harmelink and Aschenbrener, 2003).

A summary of factors that could potentially influence top-down longitudinal cracking of asphalt concrete pavement is shown in table 2.3.

<table>
<thead>
<tr>
<th>General Factors</th>
<th>Material Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bonding Condition</td>
<td>Mixture Tensile Strength</td>
</tr>
<tr>
<td>Total HMA Thickness</td>
<td>Mixture Fracture Energy</td>
</tr>
<tr>
<td>Overlay Thickness</td>
<td>Air Voids</td>
</tr>
<tr>
<td>Traffic</td>
<td>Gradation</td>
</tr>
<tr>
<td>Temperature</td>
<td>Base Modulus</td>
</tr>
<tr>
<td>Service Life (age)</td>
<td>Subgrade Modulus</td>
</tr>
<tr>
<td></td>
<td>Moisture Damage</td>
</tr>
<tr>
<td></td>
<td>Aging</td>
</tr>
<tr>
<td></td>
<td>Mixture Creep Compliance Rate</td>
</tr>
<tr>
<td></td>
<td>Asphalt Healing</td>
</tr>
</tbody>
</table>

2.3.3 Field and Laboratory Evaluation of Top-down Longitudinal Cracking

Wheel trafficking devices have been developed to investigate the effects of a rolling wheel on pavement cracking phenomenon. In 1975, Van Dijk (1975) presented a wheel-tracking tester and was successfully developed hairline cracks on asphalt mixture slab. Several full-scale testers have been developed since then, and the most popular ones include the Federal Highway Administration’s ALF (accelerated loading facility), as well as the Heavy Vehicle Simulator (HVS).

Longitudinal crack was observed in the field FHWA ALF test section in Virginia in 1998. The crack was found at the edges of wheel path. Field cores confirm that crack initiates from pavement surface and propagates downward. These longitudinal cracks are hypothesized to be due to permanent transverse tensile strains at the surface, caused by the curvature (Sherwood
et al., 1999). From transverse profiles it appears these cracks occur at the points of maximum curvature. Top-down longitudinal cracking was also observed in FHWA ALF test sections by other researchers (Stuart et al., 2001; Pellinen et al., 2004).

Longitudinal crack was also observed by Rao (2005) by using HVS. Later, this was confirmed by HVS testing at University of California Pavement Research Center, longitudinal cracking was recorded after two million repetitions at room temperatures (20±4°C) (Jones et al., 2006).

The above full-scale testers can simulate field condition very well, however, they also require high initial investment, maintenance cost, and long testing time. Some other scaled-down devices were developed to reduce cost and improve test efficiency, and the major ones are the Model Mobile Load Simulator (MMLS3), and the Asphalt Pavement Analyzer (APA).

The MMLS3 is a one-third-scale tester that was firstly developed in South Africa for testing rutting resistance of HMA in either the laboratory or field. However, it has been applied in cracking property evaluation of asphalt pavement as well. MMLS3 samples are 1.2 m (47 in.) in length and 240 mm (9.5 in.) in width, with the device applying approximately 7200 single-wheel loads per hour by means of a 300-mm (12-in.) diameter, 80-mm (3-in.) wide tire at inflation pressures up to 800 kPa (116 psi) with a typical value of 690 kPa (100 psi) (Khosla and Sadasivam, 2002; Smit et al., 2003).

Wang (2014) used the MMLS3 to evaluate the top-down longitudinal cracking for in-service pavement. Results show that after loading for 1 million pass under high temperature (around 60°C), top-down longitudinal cracking was observed 1-1.6 inch away from the load side, as shown in Figure 2.8. Testing was also performed at low and room temperatures but no crack
was found. Based on FEM analysis, the author concluded that the longitudinal cracking was initiated due to shear stress and was further propagated by tensile stress. Longitudinal cracking was also observed by other researchers such as Bhattacharjee et al. (2004).

![Figure 2.8 Top-down longitudinal cracking in field MMLS3 testing (Wang, 2014).](image)

The Asphalt Pavement Analyzer (APA) is a loaded wheel tester that was originally designed to evaluate permanent deformation of asphalt mixture. It was later modified to test fatigue cracking of asphalt mixture (Newcomb, 2002).

### 2.3.4 Prediction Models for Top-down Longitudinal Cracking

Many efforts have been put forward to evaluate the top-down longitudinal cracking performance of asphalt pavements and establish prediction models to correlate field cracking with material or other properties. Most of the current top-down longitudinal cracking models are mechanistically based.

Baladi et al. (2003) proposed a crack propagation model as a function of pavement age and distress of segregation based on distress survey data. It was concluded that the main causes
of top-down longitudinal cracking are the tensile stress induced by the tire-pavement interaction and relatively low tensile strength of AC mix.

Zou (2009) developed a HMA fracture mechanistic-based model for predicting top-down cracking performance. The model includes some material property sub-models that account for changes in near-surface mixture properties with aging and healing. The work was focused on bending-induced surface tension away from tire in terms of thin to medium HMA thickness. It was concluded that a thermal response model that predicts transverse thermal stresses that can be an important part of top-down cracking mechanism. A full calibration of field sections indicated that the predictive system adequately represents and accounts for the most significant factors that influence top-down cracking in the field.

Roque et al. (2010) selected two models for integration into a unified predictive system to predict top-down cracking. One is viscoelastic continuum damage (VECD) model to predict crack initiation by modeling damage zones and their effect on response prior to cracking. The second one is an HMA fracture mechanics (HMA-FM) model to predict crack propagation by modeling the presence of macro cracks and their effect on response. Several important material property sub-models, including aging, healing, failure criteria, viscoplasticity, and thermal stress models, were developed, modified, and/or investigated, and then incorporated into the VECD and HMA-FM models. A systematic parametric study showed that the integrated performance model provided reasonable predictions and expected trends for both crack initiation and propagation. A limited calibration/validation using data from field sections indicated that the performance model reasonably represents and accounts for the most significant factors that influence top-down cracking. However, this performance model is not ready or intended for
immediate implementation in the MEPDG because (1) the model should capture damage zone effects, for which the VECD-based model is needed, and (2) further verification of sub-models is needed (Roque et al., 2010).

Later, an enhanced hot mix asphalt fracture mechanics-based pavement performance model (HMA-FM-E) was used by Zou and Roque (2013) to prediction crack initiation time for pavement sections with or without moisture damage. It was found that the predicted crack initiation times by HMA-FM-E for sections without moisture damage were consistent with field observation-based crack initiation times. Furthermore, the predicted crack growth rate and crack initiation time of these sections were found to be inversely related. However, the predicted crack initiation time for sections with moisture damage were generally much longer than field observations. The absence of a mixture property sub-model addressing moisture effects on the change in material properties over time may explain the discrepancy (Zou et al., 2013).

In the current AASHTOWare Pavement ME Design program, top-down longitudinal cracking prediction model is based on the cumulative damage concept given by Miner’s (Huang, 1993). The damage is calculated as the ratio of the predicted number of traffic repetitions to the allowable number of load repetitions. Based on Miner’s concept, a number of models were proposed and the ones developed by Shell Oil (Bonnaure et al., 1980) and the Asphalt Institute (1982) are selected in AASHTOWare. The two equations are exactly the same form with different coefficients. The coefficients finally used in AASHTOWare were calibrated using LTPP field database (ARA, 2004).

In the AASHTOWare model, it is assumed that top-down cracking initiates at points where the critical tensile strains/stresses occur. The location of the critical strain/stress is
dependent upon several factors: (1) tensile strain; (2) dynamic modulus; (3) air voids; (4) effective asphalt content; (5) total HMA thickness; (6) service life (age), and (7) number of axle load. The most important factors are the stiffness of the layer and the load configuration (ARA, 2004). Laboratory regression coefficients and laboratory to field adjustment factor are also included.

The AASHTOWare model calculates the number of cycles to failure, which described the crack initiation stage. The second stage, or vertical crack propagation stage, is accounted for in these models by using the field adjustment factor. Other models in the literature use two different equations to express each stage of the cracking. For example, Lytton et al. (1993) used fracture mechanics based upon the Paris’s law to model the crack propagation stage in his development of the theoretical Superpave Model. Finally, another (third) stage of fracture is associated with the growth in longitudinal area, in which cracking occurs. In general, true field failure is generally associated with a percentage of cracking along the roadway.

However, many studies have shown that the top-down longitudinal cracking model in the current AASHTOWare cannot match well with field pavement sections in terms of HMA overlay placed on existing HMA (Velasquez et al., 2006; Shen et al., 2013; Williams and Shaidur, 2013). In addition, if HMA overlay is placed over existing PCC, the difference between prediction and field measurement getting even larger (Galal and Chehab, 2005; Rao et al., 2013).

2.4 Summary of Literature Review

Based on the literature review conducted in this chapter, it is concluded that:
• Thermal cracking can be caused by either sudden dramatic drop of pavement temperature, which is called low-temperature thermal cracking, or by repeated thermal cycle at model temperature, which is referred as thermal fatigue cracking. The second condition to initiate thermal cracking is pavement much be restrained in longitudinal direction (driving direction) so that the tendency of pavement to shorten under low temperature result in thermal stress. The third condition for initiation of thermal cracking is the asphalt concrete does not have the ability to release most thermal stress at a given temperature. When temperature-induced tensile stress in asphalt pavement equal to or exceeds tensile strength of pavement, thermal crack initiated and propagate under more thermal cycles.

• The thermal property of asphalt pavement is dependent on both characteristics of asphalt binder and asphalt mixture. Specifically, the asphalt properties are stiffness, m-value, ductility, fracture toughness and viscosity. Mixture properties include indirect tensile strength, creep compliance, critical strain energy release rate, fracture energy and work density. It was also found that modified asphalt has the potential to improve thermal resistance of asphalt pavement, however, the evaluation applied on traditional neat asphalt may not be suitable for modified asphalt. Other factors that could contribute to thermal cracking include bonding condition, pavement structure, climatic condition and material aging.

• A number of prediction models have been established to correlate field thermal cracking with material or other properties, and to explain thermal cracking in a fundamental
mathematical way. Among which, the fracture-based TCMODEL is a widely used one by many governmental and research agencies.

- The reflective cracking is usually observed for asphalt overlay placed on existing pavement with transverse joints or cracks. The existing joints or cracks reflect to the asphalt overlay under traffic or thermal load, or a combination of both traffic and thermal load. Both shear and tensile stress can induce the initiation and propagation of reflective cracking. The possible asphalt/asphalt mixture parameters that relating to reflective cracking are binder stiffness, asphalt content, effective binder content by volume, asphalt film thickness, mixture dynamic modulus and fracture parameter A and n. Other factors consisting of bonding condition, overlay thickness, temperature, truck, as well as modulus of base and subgrade. Although many prediction models were proposed, it is hard to find one that is generally accepted to well correlate predicted results and field measured distress. Therefore, is it more critical to prevent or retard the propagation of reflective cracking.

- The top-down longitudinal cracking is caused by traffic- or thermal-induced tensile or shear stress. The possible factors that contribute to top-down longitudinal cracking include bonding condition, pavement structure, traffic and thermal load, service life (age), mixture tensile strength, air voids, aggregate gradation, aging and hardening of asphalt overlay, base and subgrade property, and moisture and segregation. The generally used methodology to evaluate top-down longitudinal cracking is finite element plus fracture mechanics. It is noted that most laboratory evaluation on top-down cracking are using apparatus that applied for rutting property of asphalt mixture, which cannot
realistically replicate the field contact between vehicle and pavement. In terms of prediction model, there is not a generally accepted one. The one proposed by Roque and Kim under support from NCHRP looks like promising but it is too complicate to be used since many sub-models are integrated.

- Several apparatus have been developed to evaluate top-down longitudinal cracking both in field or laboratory, including full-scaled FHWA ALF and HVS. Due to high initial investment and maintenance cost of full-scaled equipment, scaled-down apparatus were developed to reduce cost and testing time. After more than one million traffic load, top-down longitudinal cracking was observed in both full-scaled and scaled-down test sections.
CHAPTER 3 NCHRP 9-49A PROJECT INFORMATION

SUMMARY

A large dataset is needed to categorize transverse cracking types, identify key influencing factors, carry out FEM analysis, and develop prediction models. All the data used in this study are from the NCHRP 9-49A project, and therefore, detailed information of this project is presented in this chapter for further usage.

The NCHRP 9-49A project, “Performance of WMA Technologies: State II--Long-Term Field Performance”, was funded by NCHRP to evaluate the long-term field performance of warm mix asphalt (WMA). Compared to the traditional hot mix asphalt (HMA), WMA technology reduces the mixing and compaction temperatures by approximately 30-50 °F to lower energy consumption and emission of greenhouse or other toxic gases (Corrigan et al., 2010). It can offer many other benefits such as creating a better environment for field workers, helping to achieve densities especially for cold season paving and long-hauling distance, and improving the overall sustainability of transportation industry. There are numerous WMA technologies available in the market. They can be generally categorized into three groups: (1) use of organic additive to decrease viscosity of binder above the melting point of the additive, such as Sasobit®; (2) chemical additives, such as Evotherm®, and Redi-Set WMX; and (3) water-based foaming, such as water injection and AquaBlack®, and water-containing foaming, such as Advera® and Aspha-Min® zeolite.

Many researchers and state Departments of Transportations (DOT’s) have conducted studies on WMA pavements with respect to both laboratory and field performances.
It is generally believed that WMA mixtures are at least equivalent to HMA mixes in terms of laboratory short-term performances (Rubio et al., 2012). However, WMA technologies can also have potential engineering challenges especially when evaluated by long-term field performance. It is not known whether reduced production temperature and foaming technologies could raise addition concerns in the rutting and moisture susceptibility of WMA pavements in the longer term (Rubio et al., 2012). In addition, it is not clear whether any material property differences observed in the laboratory between WMA and HMA could result in practical performance differences in the field. Finally, for the design, implementation and performance evaluation of WMA pavements, it is needed to assess any relationships between laboratory material properties and field performance.

The objectives of the NCHRP 9-49A project are to (1) identify the material and engineering properties of WMA pavements that are significant determinants of their long-term field performance, and (2) propose best practices for the use of WMA technologies.

Five WMA overlay pavement projects constructed in 2011 and 24 in-service projects constructed between 2005 and 2010 were identified and will be continuously monitored for their long-term field performance. In addition to the WMA sections, each project also included one control HMA section which shared the same pavement structure, traffic level, climate conditions, and mixture design as the WMA section. The only difference between the WMA section and the control HMA section was the usage of specific WMA technology in WMA overlay materials.

Figure 3.1 shows a distribution map of these projects. As can be seen, they were located in the four climatic zones of the United States (Dry-Freeze, Dry-No freeze, Wet-Freeze, and
These projects were also varied in terms of traffic level (medium and high), pavement structure (number of asphalt overlays and thickness, etc.), and material properties (with variations in asphalt, aggregate, WMA technologies, base, and subgrade, etc.).

Note: 1. Starts indicate those projects constructed in 2011, dots represent projects paved between 2005 and 2010; 2. Each location may include one HMA and several WMA pavements.

Figure 3.1 Climatic zone distributions of pavement projects.

Specifically, the research methodology for this project mainly included four components: (1) field monitoring and data collection, including field distress survey, FWD testing, sampling of material and field cores, and collection of other information (mix design, traffic, pavement structure, etc.), (2) Laboratory characterization of specimens, (3) Analysis of experiment data in the field, and (4) identification of significant determinants of field performance. Components (1) and (2) will be introduced in detail below.
3.1 Field Monitoring and Data Collection

For each project, three 200-foot research sections were randomly selected for both WMA and HMA pavements. For the five projects constructed in 2011, pre-overlay pavement distress survey, sampling of materials and field cores, and FWD testing after overlay were all conducted on the selected research sections. On-site gyratory compacted specimens were also prepared for laboratory testing. Other collected information included but not limited to: construction temperature, hauling distance, usage of material transfer devices, QC/QA data, and pavement structure. In terms of in-service projects, field distress survey was manually conducted, field cores were taken, and information was collected as much as possible (mix design, pavement structure, etc.).

3.1.1 Pre-overlay Pavement Distress Survey for New Projects

For those projects paved in 2011, in each 200-foot section, a pre-overlay distress survey was conducted using a manual distress survey following LTPP distress survey protocol (Miller and William 2003) to assess the condition of the existing pavements. It is believed that the pre-overlay conditions might contribute to the performance differences between HMA and WMA pavements, which should be differentiated from their material property differences. Figure 3.2 shows how the pre-overlay distress was conducted in the field.
3.1.2 Pavement Distress Survey for In-service Projects

For those in-service projects that have been serviced for 2-7 years (by 2012), distress survey was also conducted within three 200-foot research sections. Manual distress surveys following the distress identification manual of the LTPP (Miller and William 2003) was conducted by Bloom Companies, LLC. Pictures and videos were also taken for all the pavement sections to assist future evaluation. Supplementary cores were taken at the fine crack tip of both the longitudinal and transverse crack to help identify whether the crack was initiated from the surface or the bottom of pavement. Even so, in some cases, engineering judgement had to be used to determine the type of distress base on the best knowledge of the pavement (structure, climate zone, literature information, etc.). Figure 3.3 shows typical distress that obtained from field project.
Figure 3.3 Typical distress observed in the field pavement sections, (a) transverse cracking, WA I-90, and (b) longitudinal cracking, OH 541.

3.1.3 Sampling of Materials and Field Cores

During pavement construction of 2011 projects, materials were sampled for further laboratory testing purpose. The collected material typically include asphalt binder, loose mixture, on-site compacted samples, etc. Field cores were also taken. Figure 3.4 shows how the materials and field cores were taken in the field. Field cores were taken between wheel path within the 200-foot research sections but away from the actual cracked location. This is to ensure the cores have minimal internal distress and the properties of the cores can be representative of the actual pavement sections. In terms of in-service project, only field cores were taken. The cores were taken in the field by Braun Intertec Inc.
3.1.4 FWD Testing

Falling Weight Deflectometer (FWD) is a non-destructive testing device used to detect deflection of each pavement layer in response to a stationary dynamic load. The obtained data are used to evaluate the pavement structural capacity. The device uses a set of weights that are dropped from fixed heights to pavement surface to generate different level of loads. At each drop, the pavement deflection generated by load was measured by sensors and were recorded by software. The load levels of 6000lb, 9000lb and 12000lb are applied. A typical FWD equipment and deflection basin is shown in Figure 3.5. The FWD testing was performed by Braun Intertec Inc.

Figure 3.4 Material sampling in the field (a) loose mixture collection, and (b) field cores taken.
FWD testing results were used to estimate the in situ moduli of existing pavement layers using the back-calculation program Modcomp V6. Using the approach suggested by Irwin (1994), the layer moduli of each layer were back-calculated for each testing spot and averaged over the entire research section to reduce random error. The back-calculated base and subgrade moduli were converted to laboratory test results by multiplying C-values as shown in Table 3.1. It should be noted that many studies (FHWA, 2000; Quintus & Simpson, 2002) have pointed out the limitations of FWD back-calculation and found high variability in the back-calculated layer modulus especially when a PCC layer is used in combination with the AC layers.

Figure 3.5 (a) FWD equipment, and (b) deflection bowl.
Table 3.1 C-Values to Convert the Calculated Layer Modulus Values to an Equivalent Resilient Modulus Measured in the Laboratory (AASHTO 2008)

<table>
<thead>
<tr>
<th>Layer Type</th>
<th>Location</th>
<th>C-Value or Mr/E&lt;sub&gt;FWD&lt;/sub&gt; Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate Base/Sub-base</td>
<td>Between a stabilized and HMA layer</td>
<td>1.43</td>
</tr>
<tr>
<td></td>
<td>Below a PCC layer</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>Below an HMA layer</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>Below a stabilized subgrade/embankment</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>Below an HMA or PCC layer</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>Below an unbound aggregate base</td>
<td>0.35</td>
</tr>
</tbody>
</table>

3.1.5 Collection of Other Information

Other information was also collected and are listed below:

- **Hauling distance**
- **Mixing and compaction temperature**
- **Construction year**
- **Sampling date**
- **Pavement structure**
- **Traffic (AADT, % of truck)**
- **Aggregate type**
- **Aggregate gradation**
- **Asphalt PG**
- **Contents of asphalt binder, warm mix additives, anti-stripping agent (if any), modifier (if any), RAP/RAS (if any)**
- **Theoretical maximum specific gravity (G<sub>mm</sub>)**
3.2 Experimental Testing and Analysis

Laboratory testing obtained mechanical/engineering properties of WMA and HMA materials. It also provided inputs for Pavement ME analysis and model development. A summary of all the laboratory tests conducted in this study for both mixtures and asphalt binders is shown in Table 3.2. In addition to conventional standard tests, some research based tests such as indirect tensile fracture tests at room temperatures and low temperatures (Wen and Kim, 2002; Wen and Bhusal, 2012) were also conducted. They provided additional material properties that could potentially correlate well with the predicted field performance.

Experimental testing for laboratory characterization of plant-mixed field compacted (PMFC) samples (field cores) and plant-mixed laboratory compacted (PMLC) samples, as well as recovered binder and original binder were conducted. The analysis in this study used test results mostly from field cores and recovered binder since they were most representative to the pavement condition in the field.

| Table 3.2 Summary of Laboratory Testing for New Pavement Projects

<table>
<thead>
<tr>
<th>Mixture Test</th>
<th>AMPT Dynamic Modulus</th>
<th>IDT Dynamic Modulus</th>
<th>IDT Creep Compliance</th>
<th>IDT Fatigue</th>
<th>IDT Thermal Cracking</th>
<th>AMPT Flow Number</th>
<th>Thermal Cracking</th>
<th>Hamburg Rut Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMLC</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>PMFC (cores)</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Binder Test</th>
<th>PG Grading</th>
<th>MSCR</th>
<th>Fatigue</th>
<th>Thermal Cracking</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Extracted &amp; Recovered</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Specification (Or reference)</td>
<td>AASHTO MP 1</td>
<td>AASHTO TP 70, MP 19</td>
<td>(Wen et al., 2010)</td>
<td>(Wen and Bhusal, 2012)</td>
</tr>
</tbody>
</table>
Note: 1. The testing were performed at WCAT at WSU majorly by Mr. Shenghua Wu and Mr. Weiguang Zhang, many other graduate and undergraduate students were also involved. The Hamburg rut depth was tested by Louisiana State University.

2. AMPT dynamic modulus was used as input in AASHTOWare Pavement ME Design program.

3. The evaluation of Hamburg test results is based on TxDOT criteria (Tex-242-F method).

4. Numbers in the parentis provide the reference of the specific testing method.

5. For regular binder, a shear strain rate of 0.075 s\(^{-1}\) is used. For highly oxidized binders from in-service pavements, a lower shear rate of 0.01 s\(^{-1}\) is used to ensure the stress-strain curve with an obvious peak stress for conducting further analysis.

3.2.1 Laboratory Testing on Asphalt Mixture

The performance of asphalt mixture was evaluated in terms of resistance to low temperature cracking, fatigue and rutting. A number of tests that are identified by the team as most promising performance tests to characterize the field performance of asphalt concrete pavements were conducted (Shen et al., 2011).

3.2.1.1 Volumetric and General Properties

The volumetric and other general properties of asphalt mixture were determined in laboratory, including air voids, theoretical maximum specific gravity, voids in mineral aggregate (VMA), void filled by asphalt (VFA), asphalt type and content, aggregate type and gradation.

3.2.1.2 Low Temperature Thermal Cracking Resistance

Fracture energy, work density, IDT strength and creep compliance were carried out to determine the asphalt mixture properties at low temperature. Fracture energy is a standard IDT test, higher fracture energy indicates higher resistance to thermal cracking in the field. As a fracture property, work density describes both the strength and the ductility of a material and has been
found correlate well with field fatigue property of asphalt pavement (Wen, 2013). IDT strength and creep compliance are essential parameters in the thermal cracking prediction model in the AASHTOWare Pavement ME Design program.

The test procedure was followed AASHTO T322, “Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt (HMA) Using the Indirect Tensile Test Device”. The sample size is 1.5 inch in thickness and 4 inch in diameter. The temperature selected for fracture energy, work density and IDT strength depend on the low temperature PG grade of asphalt binder. In this study, 14°F is the appropriate testing temperature. Creep compliance is tested at -20, -10, and 0°C respectively. The loading rate selected is 0.1 in./min. Figure 3.6 shows the test set up for low temperature thermal cracking testing.

Figure 3.6 Low temperature thermal cracking testing setup.
The fracture energy was calculated as the area under the load-vertical deformation curve. Work density is the area of the stress-strain curve from a constant shear rate IDT test divided by the volume of specimen to eliminate the effect of geometry. The IDT strength was determined based on peak load at failure and specimen geometry. Creep compliance is the time-dependent strain divided by the applied stress. Typical testing curves for fracture energy and work density are shown in Figure 3.7.

![Testing curves](image)

**Figure 3.7 Low temperature asphalt mixture properties (a) fracture energy, and (b) work density.**

3.2.1.3 Fatigue Resistance

Many fatigue testing methods have been proposed to evaluate resistance of asphalt mixture to cracking at room temperature. However, these testing either requires complicate sample dimension (for instance, flexural beam tests) or have not been reported to well correlate with field performance. Based on the past and existing research by the team member (*Wen and Kim 2002; Wen 2009*), the fracture property obtained from an indirect tensile test at room temperature
was applied. The test procedures of the IDT fracture test followed the AASHTO T322. The test condition was controlled at 20°C. The loading rate is 2 in./min in accordance with the AASHTO T322.

3.2.1.4 Rutting Resistance

In terms of laboratory-fabricated samples, flow number was used to determine the rutting resistance ability of asphalt mixture since it has good correlation with field rut depth (Kaloush and Witczak, 2002). The test was conducted according to AASHTO TP79, “Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester”. The test condition was determined by the recommendation in NCHRP 9-43, with an unconfined deviator stress of 600kPa at the design temperature at 50% reliability as obtained from LTPP Binder Version 3.1. Typical result of flow number testing is presented in Figure 3.8(a).

Due to the size limitation, field cores were not conducted by flow number, which requires a specimen of 150mm in height and 100mm in diameter. Therefore, Hamburg Wheel Tracking Test was used to evaluate the rutting resistance ability of field cores. Besides, it has been widely proved that this test has good agreement with field rut depth (Yildirim and Kennedy, 2001; Lu and Harvey, 2006). The Hamburg Wheel Tracking Test was conducted by AASHTO T324, “Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot-Mix Asphalt (HMA)” and tested by Louisiana State University. Specimens were emerged in water and were tested at 50°C. The test speed is 52 passes per minute. Test stopped either 20000 passes was reached or 12.5mm rut depth was observed. Typical result of Hamburg Wheel-Track Testing is shown in Figure 3.8(b).
Figure 3.8 Typical asphalt mixture rutting test results (a) flow number, and (b) Hamburg rut depth.

3.2.1.5 Dynamic Modulus Test

Dynamic modulus is an essential factor in the fatigue cracking prediction models in the AASHTOWare Pavement ME Design program. Two types of dynamic modulus was conducted. For laboratory compacted gyratory samples, the dynamic modulus was characterized following AASHTO TP79. In terms of field cores, dynamic modulus was tested in the IDT mode due to the sample size limitation.

In terms of laboratory compacted gyratory samples, dynamic modulus was tested using the IPC AMPT (Asphalt Mixture Performance Tester) testing system, as shown in Figure 3.9(a). Nine frequencies and four target temperatures were used. The frequencies 0.1, 0.2, 0.5, 1, 2, 5, 10, 20 and 25 Hz were applied under testing temperatures of 4.4, 21.1, 37.8 and 54.4°C. Test specimens were conditioned at an environmental chamber to reach desired temperatures. Specimens were tested from the lowest temperature to the highest temperature. At the beginning, testing was...
conducted at the highest frequency. Then, all frequencies in descending order were tested. After finishing testing for each of the temperature, testing data are exported to develop a dynamic modulus master curve. The strain level during testing were controlled between 75 and 125 μstrain to ensure the samples are not damaged during testing procedure.

For field cores, dynamic modulus was tested in IDT loading mode in the GCTS MTS testing equipment, as shown in Figure 3.9(b). Both horizontal and vertical deformation of specimen were measured using LVDTs. Six temperatures (-20, -10, 0, 10, 20 and 30°C) and 5 frequencies (0.01, 0.1, 1, 5, 10 and 20Hz) were used in the testing program. It was found that the dynamic modulus master curves developed from the IDT test are generally in good agreement with those determined from axial compression test.

After testing, dynamic modulus master curve can be drawn based on tested data. Dynamic modulus master curve is a composite curve constructed at a reference temperature by shifting dynamic modulus data from various temperatures along the “log frequency” axis. In this case, dynamic modulus at any frequency and temperature can be obtained to assist mixture evaluation and characterizing. Detailed procedure for master curve plotting can be referred in AASHTO PP 61.

Figure 3.10 shows typical master curves of dynamic modulus under both AMPT and IDT modes. The reference temperature selected is 20°C. As seen, by using the same material (i.e. same mix design, asphalt type, aging condition), the dynamic modulus of AMPT in general has good agreement with IDT results. The difference shown in the figure could be induced by the varied air voids.
3.2.2 Laboratory Testing on Asphalt Binder

The performance of asphalt mixture and field asphalt concrete pavement are greatly affected by asphalt binder properties. Two types of asphalt binder were evaluated, original asphalt binder

3.2.2.1 Low Temperature Binder Property

Bending beam rheometer (BBR) was used to determine asphalt properties to resistance thermal cracking. A standard test was used, as required in AASHTO T313, “Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)”, and AASHTO PP42, “Determination of Low-Temperature Performance Grade (PG) of Asphalt Binders”. In this test, the specimen is loaded in three-point bending, and the creep stiffness $S$ and slope of stiffness master curve $m$-value are measured simultaneously at 60 second. Asphalt with higher stiffness could become brittle at low temperature and is less fracture resistance. The $m$-value reflects the ability of asphalt to relieve thermal stress based on viscous flow mechanisms. The lower the $m$-value, the slower of asphalt binder to relax stress as the temperature decreases, causing thermal shrinkage stress to build more rapidly than asphalt binders with higher $m$-values. Typical test results are shown in Figure 3.11(a).

In addition, fracture energy from monotonic test at low temperature (5°C) has been found to correlate well with field thermal cracking performance (Shen et al., 2011), and was used as a characterization factor as well. Typical test results are shown in Figure 3.11(b).
3.2.2.2 Fatigue Property of Asphalt Binder

Wen et al. (2009) found that the fracture property of asphalt binder obtained from monotonic test at room temperature has good agreement with field performance at FHWA ALF sections. Therefore, monotonic test was included as a parameter. The test was performed at 20°C with a constant shear rate of 0.05 per second.

3.2.2.3 Rutting Property of Asphalt Binder

The multiple stress creep and recovery (MSCR) test was used to evaluate the rutting susceptibility of asphalt binders. The test was performed in accordance with AASHTO TP70, “Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt binder Using a Dynamic Shear Rheometer (DSR)”, and AASHTO Designation MP19-10, “Performance-graded Asphalt Binder Using Multiple Stress Creep Recovery(MSCR) Test”. The asphalt sample should be RTFO aged before testing. A 25mm parallel plate geometry was used with 1mm gap setting. The sample was tested at creep stress level of 100Pa and 3200Pa. The

Figure 3.11 Low temperature properties of asphalt binder (a) stiffness and m-value of BBR test, and (b) definition of fracture energy.
creep portion of the test lasted for 1s followed by a 9s recovery. The first stress level was
repeated for 10 cycles and then increased to the next level. All the binder samples were
performed at a high temperature which was determined based on the environmental high
pavement temperature from LTPPBind Version 3.1 software at 98% reliability. Figure 3.12
shows typical MSCR testing results.

![Figure 3.12 MSCR test result at two stress levels (a) 100Pa, and (b) 3200Pa.]

In summary, the field collected information and laboratory tested material properties
from the NCHRP 9-49A project are used in the following respects:

1. They will be used to determine if there are common features of each type of
   transverse cracking. Specifically, the properties can help people to know at what
   conditions which transverse cracking type could be initiated.

2. They will be used to construct 3-D FEM simulation models, which will be used to
   find the mechanism of SIRC, as well as to determine the key factors that could be
correlated to such crack type.
(3) They will be used to run AASHTOWare Pavement ME Design program, in order to evaluate if the applicability of current default transverse cracking prediction model to predict field transvers cracking values.

(4) They will be used to develop statistical-based transverse cracking models that include all the transverse cracking types.
CHAPTER 4 MECHANISM OF SURFACE-INITIATED REFLECTIVE CRACKING

In this chapter, the dissertation study will start with categorization of field transverse cracking into three types: conventional thermal cracking, conventional reflective cracking and surface-initiated reflective cracking (SIRC). Out of in total 69 investigated pavement sections with transverse cracking, the percentage of each crack type observed is summarized. The potential factors that are correlated with each crack type are also presented. Next, the mechanism of SIRC is evaluated by using three-dimensional (3-D) Finite Element Method (FEM) models. Critical pavement responses under different conditions were evaluated. Both traffic load and combined traffic and thermal loads are applied. Field core data are used to validate the modeling results for specific influencing factors.

4.1 Categorization & Definition of Field Transverse Cracking

As discussed in Chapter 1, there are more than two types of transverse cracking in the field and some types of transverse cracking are less recognized nor studied. As the starting point of this study, it is necessary to review the types of transverse cracking and provide a clear definition of specific type. All future research such as determining the mechanism of specific transverse type and identifying influencing factors will be based on a clear and correct categorization of specific cracking types according to their appearance and forming mechanism. In addition, the percentage of observed pavement projects that fall in a specific crack type category will be quantified. It will
allow to develop a practical snapshot on the frequency (how common) of a particular transverse cracking happened in the field.

The data (field manual distress survey and cores) of this study were collected as part of the NCHRP 9-49A project, “Performance of WMA Technologies: State II-Long-Term Field Performance”. A total of 69 pavement sections from 24 projects across the United States were included for field performance monitoring. Detailed project information can be referred in Chapter 3.

Based on the field distress survey data collected in year 2012, 35 out of 69 pavements (50.7%) showed no transverse cracking. The remaining pavements developed different types of transverse cracking, which can be categorized into three groups.

- Conventional thermal cracking: transverse crack initiated from the surface of pavement and propagates downward due to the extreme cold climate or the fluctuation of thermal cycles. Out of 69 pavements investigated, there are in total 19 pavements (27.5 %) in this category.

- Conventional reflective cracking: transverse cracking initiated from the bottom of asphalt overlay and propagates upward due to high stress or strain concentration. There is a total 1 pavement (1.4%) in this category.

- Surface-initiated reflective cracking (SIRC): transverse cracking initiated from the overlay surface and propagates downward to potentially connect with the existing cracks or joints in the existing pavement layers. There are a total of 14 pavements (20.4%) in this category.
It is noticed that both the conventional thermal cracking and SIRC are dominating transverse crack types. Therefore, the SIRC is a widely seen cracking type and cannot be neglected.

After categorizing field transverse crack types, it is important to determine if there are common features of each type of transverse cracking. Properties considered here include bonding condition, pavement structure, material properties and other information (i.e., traffic, climate). The properties included here can provide a general idea to determine which type of transverse cracking could be initiated under fixed conditions. A more fundamental analysis of the property effects on crack type will be evaluated later in this chapter.

4.1.1 Bonding Condition

Bonding conditions are classified into two types, good bonding and de-bonding. Based on the cores taken from the field pavement sections, if different HMA layers are fully bonded without any deterioration or damage, these cores can be referred as good bonding. Otherwise, if HMA layers are loosely connected or even totally detached from each other, these cores can be referred as de-bonding. A summary of the bonding conditions from different transverse crack types are shown in Table 4.1. In general, it is observed that except for reflective cracking, most of the field cores from the other two transverse crack types (cracks initiated from pavement surface) are good bonded between layers.

<table>
<thead>
<tr>
<th>Crack Type</th>
<th>Number of Good Bonding Sections</th>
<th>Number of De-bonding Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Thermal Cracking</td>
<td>17</td>
<td>2</td>
</tr>
<tr>
<td>Conventional Reflective Cracking</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>SIRC</td>
<td>12</td>
<td>2</td>
</tr>
<tr>
<td>No Transverse Cracking</td>
<td>35</td>
<td>0</td>
</tr>
</tbody>
</table>
4.1.2 Layer Thickness

A total pavement thickness was measured and averaged according to field cores. A summary is presented in Table 4.2. Generally, pavements with thicker structure are more prone to conventional thermal cracking and SIRC. It is hard to come to conclusion in terms of reflective cracking only based on one pavement section. Pavement thickness above existing transverse crack is also averaged and shown in the table for SIRC. It is found that the thickness above the existing crack is relatively thick, which could be an important factor that initiate SIRC.

<table>
<thead>
<tr>
<th>Crack Type</th>
<th>Total Pavement Thickness, in.</th>
<th>Thickness Above Existing Transverse Crack, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Thermal Cracking</td>
<td>8.1</td>
<td>N/A</td>
</tr>
<tr>
<td>Conventional Reflective Cracking</td>
<td>10.0</td>
<td>N/A</td>
</tr>
<tr>
<td>SIRC</td>
<td>9.9</td>
<td>4.7</td>
</tr>
<tr>
<td>No Transverse Cracking</td>
<td>6.4</td>
<td>N/A</td>
</tr>
</tbody>
</table>

4.1.3 Material Properties and Other Properties

Statistical analyses is conducted to determine if there is significant difference in material or other properties among pavement sections based on different transverse crack types. Conventional reflective cracking is not considered due to the very limited data. It was found that the SIRC could be caused by a combination of good bonding condition among layers, relatively thick pavement structure, stiffer material and longer service life (age).

Specifically, material/other properties are grouped based on different crack types and then averaged. The statistical methods used are ANOVA combined with Tukey’s HSD (Honestly Significant Difference) or Games-Howell Pairwise Comparisons. The ANOVA was carried out to find if there is any significant difference among the three types of crack, the Tukey’s HSD or
Games-Howell was further conducted to determine the statistical same or different between any two of the groups. The Tukey’s HSD assumes equal variance exists between groups, while Games-Howell assumes unequal variance between groups (Timothy, 2005). The null hypothesis for ANOVA is all means are equal, the alternative hypothesis is at least one mean is different from the other. The null hypothesis for Tukey’s HSD/Games-Howell is both means are equal, the alternative hypothesis is the two means is different from each other. A significance level of 0.05 (95% confidence) was used for both ANOVA and Tukey’s HSD/Games-Howell. Material properties along with p-value are summarized from Tables 4.3 to 4.5. The first column in the table shows difference of means between two different cracking types. The second column in the tables presents p-value. Properties that are statistically different between two groups are highlighted in the table, as shown, the following findings can be observed:

(1) In terms of asphalt binder properties, asphalt from SIRC projects are stiffer compared with these projects have no transverse cracking. Specifically, the asphalt from SIRC sections have higher PG grade and higher BBR stiffness. No differences between SIRC and conventional thermal cracking sections are observed except for BBR m-value, the SIRC sections have lower m-value.

(2) In terms of mixture properties, mixtures from SIRC sections have the highest E*, indicating that stiffness could be a critical factor for such cracking type. No statistical difference was found between conventional thermal cracking sections and pavements with no crack.
(3) The hour of low temperature from conventional thermal cracking sections is the highest, which is within expectation. No statistical difference was found between SIRC sections and pavements with no crack.

(4) The SIRC sections have the highest total HMA pavement thickness, service year and AADTT. Therefore, it is expected that these factors could be critical to initiate SIRC.

(5) Other properties were also checked but no statistical difference were observed and therefore are not shown here. These properties include E* at 21.1°C, mixture work density, mixture IDT strength, mixture creep compliance, air voids, percent passing #200 sieve, m-value obtained from creep compliance master curve, asphalt content, effective binder content, as well as overlay thickness.

| Table 4.3 Asphalt Property Comparison between Different Cracking Types Group |
|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|
| Asphalt Property            | Crack Type                  | High PG Grade               | Low PG Grade               | m-value, BBR, -6°C           | Stiffness, BBR, -6°C, Pa     |
|                             | Difference of Means         | P-Value                     | Difference of Means         | P-Value                     | Difference of Means          | P-Value                     |
| SIRC                        | Conventional Thermal Cracking | 3.1                         | 0.443                      | 1.6                         | 0.56                        | -0.023                      | 0.042                        | 24.3                         | 0.438                        |
| Conventional Thermal Cracking | No Transverse Crack         | 6.4                         | 0.019                      | 3.7                         | 0.023                      | -0.034                      | 0.087                        | 52.9                         | 0.011                        |
| SIRC                        | SIRC                        | -3.1                        | 0.443                      | -1.6                        | 0.56                       | 0.023                       | 0.042                        | -24.3                        | 0.438                        |
| Conventional Thermal Cracking | No Transverse Crack         | 3.3                         | 0.26                       | 2.1                         | 0.198                      | -0.011                      | 0.713                        | 28.6                         | 0.179                        |
| SIRC                        | SIRC                        | -6.4                        | 0.019                      | -3.7                        | 0.023                      | 0.034                       | 0.087                        | -52.9                        | 0.011                        |
| Conventional Thermal Cracking | No Transverse Crack         | -3.3                        | 0.26                       | -2.1                        | 0.198                      | 0.011                       | 0.713                        | -28.6                        | 0.179                        |

| Table 4.4 Mixture Property Comparison between Different Cracking Types Group |
|-----------------------------|-----------------------------|-----------------------------|-----------------------------|-----------------------------|
| Mixture Property            | Crack Type                  | E*, -10°C, 1Hz, Mpa         | E*, 4.4°C, 1Hz, Mpa         |
|                             | Difference of Means         | P-Value                     | Difference of Means         | P-Value                     |
| SIRC                        | Conventional Thermal Cracking | 4355.9                      | 0.000                      | 2793.8                      | 0.009                      |
| Conventional Thermal Cracking | No Transverse Crack         | 2468.6                      | 0.026                      | 1348.6                      | 0.237                      |
| SIRC                        | SIRC                        | -4355.9                     | 0.000                      | -2793.8                     | 0.009                      |
| Conventional Thermal Cracking | No Transverse Crack         | -1887.3                     | 0.070                      | -1445.2                     | 0.133                      |
| SIRC                        | SIRC                        | -2468.6                     | 0.026                      | -1348.6                     | 0.237                      |
| Conventional Thermal Cracking | No Transverse Crack         | 1887.3                      | 0.070                      | 1445.2                      | 0.133                      |
### Table 4.5 Other Property Comparison between Different Cracking Types Group

<table>
<thead>
<tr>
<th>Other Property</th>
<th>Crack Type</th>
<th>Hour of low temperature</th>
<th>Total HMA Thickness, in.</th>
<th>Service Year</th>
<th>AADTT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Difference of Means</td>
<td>P-Value</td>
<td>Difference of Means</td>
<td>P-Value</td>
</tr>
<tr>
<td>SIRC</td>
<td>Conventional Thermal Cracking</td>
<td>-292.2</td>
<td>0.027</td>
<td>1.7</td>
<td>0.235</td>
</tr>
<tr>
<td></td>
<td>No Transverse Crack</td>
<td>-42.2</td>
<td>0.905</td>
<td>3.5</td>
<td>0.001</td>
</tr>
<tr>
<td>Conventional Thermal Cracking</td>
<td>SIRC</td>
<td>292.2</td>
<td>0.027</td>
<td>-1.7</td>
<td>0.235</td>
</tr>
<tr>
<td></td>
<td>No Transverse Crack</td>
<td>250.0</td>
<td>0.018</td>
<td>1.8</td>
<td>0.086</td>
</tr>
<tr>
<td>No Transverse Crack</td>
<td>SIRC</td>
<td>42.2</td>
<td>0.905</td>
<td>-3.5</td>
<td>0.001</td>
</tr>
<tr>
<td></td>
<td>Conventional Thermal Cracking</td>
<td>-250.0</td>
<td>0.018</td>
<td>-1.8</td>
<td>0.086</td>
</tr>
</tbody>
</table>

### 4.2 Mechanism of SIRC

This section describes the dissertation work to (1) evaluate the mechanism of SIRC, and (2) identify the potential factors that contribute to such cracking type. Three-dimensional (3-D) Finite Element Method (FEM) is used to evaluate the critical pavement responses (both magnitude and location) under different conditions. The models are constructed based on WA I-90 pavement project. An extensive parametric analysis is further performed to investigate the effects of different parameters (for instance, material properties and pavement structure) on SIRC. By identifying the potential conditions that could lead to specific types of transverse cracking, this study could make the field categorization of transverse crack more clear so that appropriate research methodology and maintenance strategies can be selected.
4.2.1 FE Model Construction

In this section, specific case (WA I-90) which showed SIRC is used for constructing FEM models. The specific material property, pavement structure and other conditions of each pavement project are applied.

4.2.1.1 Material Properties

Asphalt overlay is simulated as linear viscoelastic material to consider its time and temperature related properties. Superpave indirect tensile (IDT) tests were conducted according to AASHTO T322-2012 using field cores taken from WA I-90 project. Creep compliance was measured at three temperatures, -20°C, -10°C and 0°C with 100 second loading time. Results were used to generate creep compliance master curve and determine time and temperature dependent relaxation modulus.

The relationship between relaxation modulus and creep compliance of viscoelastic material is shown in the following equation,

\[
L[D(t)] * L[E(t)] = \frac{1}{s^2}
\]

where:

\[L[D(t)]\] = the Laplace transformation of the creep compliance, \(D(t)\);

\[L[E(t)]\] = the Laplace transformation of the relaxation modulus, \(E(t)\);

\(s\) = the Laplace parameter;

\(t\) = time (or reduced time, \(\xi\)).

The Prony series representation of creep compliance is shown in the following form,

\[
D(t) = D_0 + \frac{1}{\eta} t + \sum_{i=1}^{N} D_i (1 - e^{-t/\xi_i})
\]

(4.2)
where $D_0$, $D_i$, and $\eta$=Prony series parameters; and $\tau_i$=retardation times.

Performing the Laplace transform to Eq. (4.2) and inserting the results into Eq. (4.1) yields the following form (Kim et al., 2010a):

$$L[E(t)] = \frac{1}{s \left(D_0 + \frac{1}{\eta \cdot s} + \sum_{i=1}^{N} \frac{D_i}{\tau_i \cdot s + 1} \right)}$$

(4.3)

Performing the inverse Laplace transformation for Eq. (4.3) and finally yields the generalized Maxwell model in parallel (relaxation modulus).

It should be noted that the inverse of creep compliance can be directed used as relaxation modulus, only under low temperatures and low loading times with hard materials (Lytton et al., 1993). Otherwise, the relaxation modulus should be calculated from the procedures described above. The relaxation modulus for Washington I-90 project of HMA mixture is shown in Figure 4.1.

Figure 4.1 Master relaxation modulus curve for Washington I-90 project.
The viscoelastic property of HMA is expressed with Prony series parameters, which can be obtained by fitting master relaxation modulus master curve. Several methods are available for fitting Prony-series function to given data. Among which, the collocation method used by Schapery (1961) and a least-squares scheme introduced by Cost and Becker (1970) are widely applied due to their simplicity. The least-squares method is employed in this study since the number of observations is greater than the number of unknown coefficients in the model (Park and Kim 2001). Table 4.6 shows the shift factor and Power Law parameters used to construct creep compliance, as well as the Prony series parameters.

Table 4.6 Viscoelastic Properties of HMA Overlay for WA I-90 Project

<table>
<thead>
<tr>
<th>Shift Factor</th>
<th>Power Law Parameter for Creep Compliance</th>
<th>Prony Series Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Spring constant, MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Relaxation Time, sec</td>
</tr>
<tr>
<td>Log (1/shift factor), -20°C</td>
<td>D₀, (1/psi) 2.962E-07</td>
<td>Log τ₀, (sec) 1.23</td>
</tr>
<tr>
<td>Log (1/shift factor), -10°C</td>
<td>D₁, (1/psi) 5.712E-08</td>
<td>Log τ₁, (sec) 2.46</td>
</tr>
<tr>
<td>Log (1/shift factor), 0°C</td>
<td>D₂, (1/psi) 7.045E-08</td>
<td>Log τ₂, (sec) 3.69</td>
</tr>
<tr>
<td>Log (1/shift factor), 0°C</td>
<td>D₃, (1/psi) 1.240E-07</td>
<td>Log τ₃, (sec) 4.92</td>
</tr>
<tr>
<td>Log (1/shift factor), 0°C</td>
<td>D₄, (1/psi) 3.119E-07</td>
<td>Log ηᵥ, (psi·sec) 12.44</td>
</tr>
</tbody>
</table>

Note: -20°C was used as reference temperature.

Most FE programs require shear and bulk moduli as inputs in terms of viscoelastic properties. Since the performing of shear and bulk modulus tests are costly and complex, many researchers used mathematical approach to calculate the two parameters from relaxation modulus. The most widely used equations are shown as following:

\[
G(t) = \frac{E(t)}{2(1 + \mu)}
\]  

(4.4)
where, $E(t)$=relaxation modulus; $G(t)$ and $K(t)$=shear and bulk modulus; $\mu$=Poisson’s ratio. The relaxation modulus has already determined and shear and bulk modulus can therefore calculated from Eq. (4.4) and (4.5). It should be noted that the equations are used based on two assumptions: (1) materials are homogeneous and isotropic; (2) Poisson’s ratio is time-dependent constant. The first one has been widely accepted as a primary assumption for engineering materials, and it has been reviewed by many researchers and proved to be validate (Elseifi et al., 2006; Wang et al., 2006). In terms of Poisson’s ratio, a value of 0.35 was used as recommended (Kim et al., 2010a). Table 4.7 shows the shear and bulk relaxation modulus ratio for HMA mixture of WA I-90 project.

<table>
<thead>
<tr>
<th>Relaxed Time, sec.</th>
<th>Shear Relaxation Modulus Ratio</th>
<th>Bulk Relaxation Modulus Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.2</td>
<td>0.165</td>
<td>0.165</td>
</tr>
<tr>
<td>240.1</td>
<td>0.141</td>
<td>0.141</td>
</tr>
<tr>
<td>3768.3</td>
<td>0.163</td>
<td>0.163</td>
</tr>
<tr>
<td>52727.4</td>
<td>0.195</td>
<td>0.195</td>
</tr>
<tr>
<td>2375900.8</td>
<td>0.335</td>
<td>0.335</td>
</tr>
</tbody>
</table>

For other layers of the pavement such as existing asphalt layers, base and subgrade, linear elastic behavior was assumed for simplicity purpose. The elastic modulus of each layer was back-calculated from deflections measured by Falling Weight Deflectometer (FWD) using Modecomp V6 program. Table 4.8 gives a summary of the material properties. The Poisson's ratio for HMA overlay and other layers were selected based on recommendations from literature (Kim et al., 2010a; Kim and Buttlar, 2002).
### Table 4.8 Elastic Properties Summary for WA I-90 Project

<table>
<thead>
<tr>
<th>Layer</th>
<th>Elastic Modulus, Mpa</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overlay</td>
<td>N/A</td>
<td>0.35</td>
</tr>
<tr>
<td>Existing HMA</td>
<td>4573.6</td>
<td>0.35</td>
</tr>
<tr>
<td>Base</td>
<td>72.1</td>
<td>0.4</td>
</tr>
<tr>
<td>Subgrade</td>
<td>131.1</td>
<td>0.4</td>
</tr>
</tbody>
</table>

4.2.1.2 Model Structure and Mesh

The dimensions of the pavement model is selected as 100 in. × 100 in. × 100 in. for the x, y and z directions. Because of the symmetry, only half model is applied in this study. For WA I-90 project, the existing crack was simulated as a gap between two HMA slabs. The crack size is approximately 0.4 inch in width and 8.3 inch in depth which is determined from field cores.

The selection of mesh density needs to ensure the simulation accuracy but maintain sufficient computational efficiency. In this study, a fine mesh is used in the cracking vicinity and loading area. A coarse mesh is used farther from the crack and the loading area. The mesh between fine and coarse are connected by graded mesh. A sensitivity analysis is performed to determine the proper mesh density at all three directions (X, Y and Z-axis directions). The element along thickness direction is found to be the most critical, and the thickness of single element used is 0.25 inch for overlay, 0.5 inch for existing HMA, 1.6 inch for base and 3.85 inch for subgrade. Figure 4.2 shows the FEM model used in this study based on WA I-90 project with analysis locations identified. Table 4.9 shows the field pavement structure for WA I-90 project.
4.2.1.3 Bonding and Boundary Condition

For bonding condition, the tangential behavior between pavement interfaces is controlled by the Coulomb Friction Model. Both fully bonded and de-bonding conditions are considered. Separation is not allowed once the layers are contacted.

In terms of boundary condition, considering the symmetrical model, the boundary conditions of the four sides (faces) in the model are fixed in horizontal directions while free in vertical direction, with the bottom part fixed in all directions.
4.2.1.4 Loading Conditions

A static traffic load, as well as a combination of traffic and thermal load are applied. In terms of traffic load, the load footprint with a rectangular shape is used since it matches with field tire print better than circular or ellipse shape (Lytton et al., 2010). A load magnitude of 0.69 MPa is used. Two loading positions are considered, one is located directly above the crack in the center of the model and the other one is located several inches away from the center of the crack.

The thermal load is specified for the entire asphalt overlay and existing HMA layer. The magnitude of thermal load uses critical cooling event in George, WA, where the WA I-90 project located. The Enhanced Integrated Climatic Model (EICM) was applied to determine pavement temperature distribution along the depth (Zapata and Houston, 2008). The critical cooling event started at 11AM, January 4th, 2004 and ended at 8AM in the following day. The total temperature dropped 13.7°C during the 21-hour period. Figure 4.3(a) shows the temperature profiles used in this study at the top of pavement overlay and at the bottom of existing HMA layer. As seen, the temperature profiles can be grouped into two categories: the temperature at the beginning and at the end of cooling cycle changes slowly, while the temperature in the middle of cooling cycle dropped sharply. The analysis focused on the period with significant temperature variations, which was critical for thermal stress accumulation. Typical temperature gradient between the top of overlay and the bottom of existing HMA layer is presented in Figure 4.3(b). As seen, the temperature difference is relatively large due to the thick pavement structure. The coefficient of thermal expansion of asphalt layers is set as 2.5E-05 mm/mm/°C (Solaimanian & Bolzan, 1993; Myers, Roque, & Ruth, 1998; Wang & Al-Qadi, 2010a).
4.2.2 Failure Criteria

Two failure criteria were used to determine the magnitude and location of maximum stress, including tensile stress in the longitudinal direction (tensile stress), and Von Mises stress (or maximum octahedral shear stress). Von Mises stress can take into account the multiaxial stress state including the shear when the potential failure location is near the pavement surface (Collop et al., 2003; Mo et al., 2007; Wang & Al-Qadi, 2009; Wang & Al-Qadi, 2010a; Wang & Al-Qadi, 2010b; Wang et al., 2013). It is a good indicator for evaluating quasi-ductile material (Wang & Al-Qadi, 2010b). It can be expressed as either in Equation (4.6) or (4.7), using either principal stresses or normal and shear stresses in the expression.

\[
\sigma = \frac{1}{\sqrt{2}} \left[ (\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 \right]^{1/2}
\]  
(4.6)

\[
\sigma = \frac{1}{\sqrt{2}} \left[ (\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + 6(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2) \right]^{1/2}
\]  
(4.7)

Figure 4.3 Pavement temperature at George, WA (a) temperature distribution with cooling cycle, and (b) temperature distribution along pavement depth.
Where $\sigma_1$, $\sigma_2$, and $\sigma_3$ are principal stresses; $\sigma_x$, $\sigma_y$, and $\sigma_z$ are normal stresses; $\tau_{xy}$, $\tau_{yz}$, and $\tau_{zx}$ are shear stresses.

It should be noted that although the above criteria are used in this study, other criteria may also be applied to evaluate the stress magnitude and location. Especially under thermal load, the asphalt material becomes more brittle at low temperature and these criteria that can consider such material property may be applied, such as Griffith’s criterion.

4.2.3 Simulation Results

An existing transverse crack is preset to evaluate whether the crack could have any impact on the surface initiated transverse crack. Traffic load and a combined traffic and thermal load were applied. As shown in Figure 4.4 (a), when only traffic load is applied, the maximum stress value is Von Mises that observed at pavement surface. Although the magnitude of Von Mises is not very high, but crack may initiate under repeated traffic load. When combining traffic and thermal loads (Figure 4.4b), Von Mises and tensile stresses at the surface increase greatly with temperature drop. The magnitude of the stresses can even be greater than the IDT strength of WA I-90 field cores (5.6MPa, tested at -10°C) measured in the laboratory. Results indicated a potential failure from the surface related to a combined effect of tension and shear. The negative thermal load (temperature drop) significantly increased the magnitude of Von Mises and tensile stresses at the surface, and therefore increased the potential of surface-initiated cracking.
Figure 4.4 Simulation results of WA I-90 case with existing transverse crack, (a) traffic load; and (b) combined traffic load and thermal load.

Figure 4.5 shows the simulation results without transverse cracking in existing pavement layers to evaluate the effects of existing crack. By comparing Figure 4.4 with Figure 4.5, it is found that existing crack/joint in the pavement layer could induce approximately 112% (for traffic load only) and 20% (for combined traffic and thermal load) higher maximum Von Mises stress at the pavement surface compared to pavements without existing crack/joint. Therefore, the existing crack/joint increases the magnitude of tensile stress at pavement surface which could be critical for SIRC.
In summary, the findings here indicated that the transverse crack could initiate from overlay surface even with the existing transverse crack/joint. The Von Mises stress at the top of overlay was found to be critical stress, which described a combined effect of tension and shear. In the next sections, parametric studies were conducted to evaluate the influencing factors that could result in SIRC.

4.2.4 Parametric Analysis-Traffic Load only

A total number of eight factors were evaluated for their influences on SIRC, which are shown in Table 4.10. The WA I-90 project was used as the base case so that only one factor was changed for each analysis. A total of 16 FEM runs were carried out. Below shows the results from traffic load only.

Figure 4.5 Simulation results of WA I-90 case without existing transverse crack, (a) traffic load; and (b) combined traffic load and thermal load.
Table 4.10 Parameters Used for Parametric Analysis

<table>
<thead>
<tr>
<th>Factors</th>
<th>Control (WA I-90)</th>
<th>Variation in Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bonding Condition</td>
<td>Fully bonded</td>
<td>Friction coefficient=0.1, 1.0</td>
</tr>
<tr>
<td>Thickness above Existing Crack, cm</td>
<td>15.2</td>
<td>7.6, 11.4</td>
</tr>
<tr>
<td>Viscoelastic Material of HMA Overlay</td>
<td>WA I-90 (PG 76-16, instant. Young’s Modulus=23277MPa)</td>
<td>Softer material (MN 169): PG70-22, instantaneous Young’s Modulus=19195MPa</td>
</tr>
<tr>
<td>Elastic Modulus of Existing HMA, MPa</td>
<td>4573.6</td>
<td>2000, 6000</td>
</tr>
<tr>
<td>Elastic Modulus of Base, MPa</td>
<td>72.1</td>
<td>50, 300</td>
</tr>
<tr>
<td>Elastic Modulus of Subgrade, MPa</td>
<td>131.1</td>
<td>50, 300</td>
</tr>
<tr>
<td>Loading Location</td>
<td>Right above the center of crack</td>
<td>Several inches away from the center of the crack</td>
</tr>
<tr>
<td>Tire Pressure, MPa</td>
<td>0.69</td>
<td>0.41, 0.83</td>
</tr>
</tbody>
</table>

4.2.4.1 Effect of Thickness above Existing Crack

Figure 4.6 shows the effect of pavement thickness above existing crack on stress responses. As shown, tensile stresses at the existing HMA bottom is dominant when the thickness above existing crack is relatively thin (3 inch). However, Von Mises at the top of overlay become dominant when pavements have thicker layer above the existing crack (≥4.5 inch), which could induce surface-initiated cracking. When pavement overlay is thin, the existing pavement layer with crack may move in vertical direction under traffic load, thereby resulting in stress concentration at the bottom of existing HMA bottom. If pavement is thick, the existing pavement layer with crack is well protected and has less potential to experience high vertical movement. Therefore, the bottom of overlay is protected and critical stress location can be transferred to the surface.
4.2.4.2 Effect of Other Factors

Figure 4.7 demonstrates the effect of the other seven factors. Although these factors affect the magnitude of observed stresses, they do not change the location of maximum stress. The critical stresses are always Von Mises located at the top of overlay. As shown, the magnitude of Von Mises stress at pavement surface increases under the following conditions (1) the bonding condition becomes worse; (2) stiffer material is used; (3) decrease of elastic modulus of existing HMA, base and subgrade; (4) traffic load moves directly above the existing joint, and (5) higher load pressure is applied.
(a) Stress, Mpa
-6, -2, 2, 6
- Fully Bonded, COF=1.0
- Stiffer Material, Control
- Bonding Condition
- Existing HMA Bottom
- Tensile Stress, Overlay Top

(b) Stress, Mpa
-6, -2, 2, 6
- Stiffer Material, Control
- Overlay Stiffness
- Softer Material
- Existing HMA Bottom
- Tensile Stress, Overlay Top

(c) Stress, Mpa
-6, -2, 2, 6
- 2000, 4573.6, 6000
- Modulus of Existing HMA, MPa

(d) Stress, Mpa
-6, -2, 2, 6
- 50, 72.1, 300
- Modulus of Base, MPa

(e) Stress, Mpa
-6, -2, 2, 6
- 50, 131.1, 300
- Modulus of Subgrade, MPa

(f) Stress, Mpa
-6, -2, 2, 6
- Central Load, Side Load
- Load Location
- Existing HMA Bottom
- Tensile Stress, Overlay Top
- Tensile Stress, Existing HMA Bottom
Figure 4.7 Critical responses of SIRC under change of different conditions (a) bonding condition; (b) overlay stiffness; (c) modulus of existing HMA; (d) modulus of base; (e) modulus of subgrade; (f) load location, and (g) tire pressure.

4.2.5 Parametric Analysis-Combined Traffic and Thermal Load

A total number of eight factors were evaluated for their influences on SIRC. In addition to all the factors that shown in Table 4.10, cooling rate at pavement surface is also included. The three cooling rates used are 0.3, 0.65 and 1.0ºC/h. Side load location was not used since thermal load is evenly distributed in the model. The WA I-90 project was used as the base case so that only one factor was changed for each analysis. A total of 17 FEM runs were carried out. Below summarizes the results from combined traffic and thermal load.

4.2.5.1 Effect of Thickness above Existing Crack

Figure 4.8(a) shows the effect of pavement thickness above existing crack on stress responses. As shown, Von Mises and tensile stresses at the bottom of overlay are dominant when the thickness above existing crack is relatively thin (≤4.5inch). However, Von Mises and tensile stresses at the top of overlay become dominant when pavements have thicker layer above the
existing crack (≥6inch), which could induce surface-initiated cracking. When pavement overlay is thin, the existing pavement layer with crack may suffer significant temperature variation and lead to large contraction and expansion, thereby resulting in stress concentration at the bottom of overlay. If pavement is thick, the existing pavement layer with crack is well protected and has less potential to experience high temperature variation as shown in Figure 4.3(a). Therefore, the bottom of overlay is protected and critical stress location can be transferred to the surface.

4.2.5.2 Effect of Overlay Stiffness

The effect of overlay stiffness was considered by using asphalt mixture with different binder PG and instantaneous Young’s Modulus. The instantaneous Young’s Modulus is a parameter required in ABAQUS for viscoelastic property that determined from mixture creep compliance. It is assumed that the asphalt mixture with higher binder PG and higher instantaneous Young’s Modulus is generally stiffer. As a variation from the control (WA I-90 project), the two stiffness levels (softer and stiffer) were obtained from material properties of field projects MN 169 and SC 178 respectively. Figure 4.8(b) shows the effect of overlay stiffness on stress responses. As shown, Von Mises and tensile stresses at the top of overlay are greater than the stresses at the bottom of existing HMA if the stiffer overlay material was used. Conversely, Von Mises and tensile stresses at the top of overlay are smaller than the stresses at the existing HMA bottom if the softer overlay material was used. It is noted that no modified asphalt was used among the three projects evaluated here; hence higher stiffness could be related to higher degree of aging and brittleness. It is suggested that aged material, with less capability to relax stresses, could be more prone to SIRC.
Figure 4.9 shows the field cores taken from SC US178 project and MN169 project. It was confirmed that only surface-initiated cracking was observed in SC US178 project although the pavement has cement treated base structure. In MN169 project the existing transverse cracking reflected through the entire pavement structure.

![Figure 4.8 Critical responses of SIRC under change of different conditions](image)

(a) Thickness above existing crack, and (b) overlay stiffness.

![Figure 4.9 Field observation of transverse cracking](image)

(a) Field core of SC178, and (b) field core of MN169.
4.2.5.3 Effect of Other Factors

Figure 4.10 demonstrates the effect of the other six factors except for thickness above existing crack and overlay stiffness. As shown, in terms of modulus of existing HMA and cooling rate, although they affect the magnitude of observed stresses, they do not change the location of maximum stress. The stress values are almost insensitive to the other four factors (bonding condition, modulus of base and subgrade, and tire pressure).
4.2.6 Mechanism for Surface-initiated Reflective Cracking (SIRC)

Based on the FEM simulations summarized above, it is proposed that the possible mechanisms that contribute to SIRC include:

1) The crack in the existing HMA layer is protected from movement. The critical condition to initiate a reflective crack is the movement of existing pavement. This movement can be caused by either thermal-induced expansion and contraction, or traffic-induced shear or bend stress. However, if the pavement thickness above existing crack is very thick, which is the case for WA I-90 project (Figure 4.2(b)), then the temperature fluctuation at bottom of existing HMA layer tends to be minimized (Figure 4.3(a) ) and may not be able to cause movement of the existing crack. In addition, large shear/bend stresses may not be induced in existing HMA layer with crack/joint because of the thick pavement overlay as well as the relatively higher stiffness of the overlay material.
(2) The critical stress location is driven to pavement surface under traffic load. Based on the simulation results, it is very possible that under traffic load in the field, influenced by the existence of underneath crack/joint, large stress could develop at the surface of pavement. Such stress could be close to or greater than the tensile strength of asphalt mixture, contributing to crack failure at the pavement surface especially under the following conditions (a) the thickness above existing joint increases; (b) the bonding condition becomes worse; (c) the overlay stiffness increases; (d) modulus of existing HMA, base and subgrade decreases; (e) the traffic load drives directly above existing joint, and (f) higher tire pressure is used.

(3) The critical stress location is driven to pavement surface under combined traffic and thermal load. Based on the simulation results, it is also possible that under thermal load in the field, influenced by the existence of underneath crack/joint, large stress could develop at the surface and excess than the asphalt mixture strength, resulting in crack failure at pavement surface. Such failure could be accelerated at higher cooling rate and lower modulus of existing HMA layers. The tensile stress or von Mises Stress at the top of overlay becomes more critical under thicker pavement and stiff overlay material.
CHAPTER 5 LABORATORY EVALUATION OF SURFACE-INITIATED REFLECTIVE CRACKING

Laboratory testing is performed to evaluate the key factors that contribute to the initiation and propagation of the surface-initiated reflective cracking (SIRC). As suggested by the FEM analysis results obtained in Chapter 4, the following factors are investigated: layer thickness, aging, existing joint, and base/subgrade support conditions.

5.1 Overview of the Experiment

Hamburg Wheel Tracking Device (HWTD) is used in this study to evaluate the cracking potential of the material/structure under various conditions.

The HWTD was firstly developed in Germany and is now extensively used by many US state DOTs and industries to evaluate the rutting and moisture damage potential of asphalt mixture. It applies repeated loading on asphalt mixture specimens submerged in water at relatively high temperature (typically at 50°C) and uses rut depth and stripping inflection point at a specific loading cycle to describe the rutting and moisture resistance of the mixture. As specified in AASHTO T324, Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA), the standard load on the wheel is around 705 N (158 lb). The wheel reciprocates over the specimen at 52±2 passes across the specimen per minute, with the position varying sinusoidally over time. The wheel-tracking device shall shut off when 20,000 passes have occurred or when the test has achieved the maximum rut depth (i.e., 0.5 inch). Literatures
(Aschenbrener, 1995; Zhou et al., 2003) indicate that the rut depth from the HWTD can correlate well with field rutting and moisture damage performance; it thus becomes a popular equipment at both research institutes and industry laboratories.

As a repeated loading device that simulates cyclic traffic loading in the field, it is hypothesized that HWTD can also be utilized for evaluating the cracking resistance of the pavement. However, considering the fundamental difference in failure mechanism between rutting and cracking, the HWTD needs to be modified and improved in the following four areas in order to make it potentially a cracking evaluation device as well. They include:

- Test conditions need to be changed to ensure that cracking instead of permanent deformation is the dominant failure mechanism. Typically, asphalt mixture reacts as viscoelastic-plastic material under standard HWTD loading condition since the test is performed at relative high temperature (i.e., 50°C) and low loading rate (52 passes per minute). To obtain asphalt material that responds as more elastic – brittle solid, the test temperature should be decreased to room level (i.e., 20-25°C) or even lower (close to zero degree), or the wheel load speed needs to be increased to a higher value that can simulate fast moving vehicle.

- Long-term aging of specimen should be included since it is a critical factor that affect both fatigue cracking and low-temperature thermal cracking. The aging of asphalt mixture increases the stiffness of the material, reduce its ductility, and could negatively affect its resistance to fatigue induced cracking (Glover et al., 2005). It is also found that the aging decreased the low-temperature cracking resistance of asphalt concrete pavement and increases crack frequency (Kliewer et al., 1996).
• The base (or support) stiffness of test specimen should be adjustable to match with field pavement condition. Such adjustment can be achieved by placing rubber with different stiffness under specimen. Typically, the potential of fatigue cracking initiation increases with the decrease of base modulus (Hu et al., 2008).

• Boundary condition and temperature effects may be included. By including proper boundary condition and temperature change capability, the HWTD effect of thermal loads can be evaluated in addition to the traffic load. This is useful when combined thermal and traffic load are used to evaluate low-temperature resistance of asphalt mixture.

Based on these considerations, the research team worked closely with the James Cox & Sons, Inc company to make some modifications to their standard Hamburg Wheel Track Device (HWTD) so that cracking properties of the material can be evaluated. These changes include:

• Wider temperature range: 5°C to 60°C. An extra chiller is connected to the machine to keep the test temperature as low as 5°C. The chiller can provide a cooling rate as high as 5°C/hour and is suitable for simulating field cooling cycle. The high temperature level can be controlled by water.

• Wider loading rate range: maximum 70 passes per minute, which is faster than standard wheel load speed (52±2).

• Capable of include rubber base to adjust support conditions.

Figure 5.1 shows a picture of HWDT from James Cox & Sons, Inc.
5.2 Design of Test Condition

Table 5.1 presents a summary of all the tested specimens and test conditions. The conditions that SIRC is observed are also pointed out. All the experiments were carried out by using the James Cox & Sons HWDT equipment. There are in total 9 specimens tested. The reasons to include these conditions are:

- **Layer thickness**: layer thickness has been proved to affect top-down longitudinal cracking significantly. Also as seen in Chapter 4, pavement thickness could be a critical factor that correlate to SIRC. Therefore, it is expected that initiation of crack can be observed at different wheel load repetitions by using varied specimen thickness. Therefore, HWT samples with different thickness are used, consisting of 1 inch, 1.5 inch, and 2.4 inch.

- **Aging condition**: specimen with different degree of aging (5-day aging and 10-day aging) are applied. 5-day aging is considered to be equivalent to long-term aging in the field.

![Hamburg wheel tracking device](image)
pavement in accordance with AAASHTO R30. 10-day aging is used to obtain asphalt mixture with even higher stiffness (more brittleness), which is critical for surface-initiated crack as suggested in chapter 4 in this dissertation.

- **With/without existing joint**: both specimens with and without existing joint are used to evaluate the effects of existing joint on the property of SIRC.

- **Weak base/strong base**: As seen in chapter 4, for pavement under traffic load, the modulus of base can greatly affect the stress magnitude at pavement surface. Specifically, the higher the base modulus, the smaller Von Mises stress value at pavement surface. In contrast, the lower the base modulus, the higher Von Mises stress value at pavement surface. This has been confirmed by other researchers as well (Hu et al., 2008; Schwartz et al., 2011). Specifically, the sensitivity analysis of AASHTOWare Pavement ME program shows that the top-down longitudinal cracking is very sensitive to base resilient modulus (Schwartz et al., 2011), more amount of longitudinal cracking is predicted if lower base modulus is used. It is also pointed out that pavement with lower base modulus is more prone to initiate top-down fatigue cracking (Hu et al., 2008). Therefore, with flexible base, it is expected that SIRC is easier to be simulated in laboratory.
Table 5.1 Design of Test Condition

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Layer Thickness, in.</th>
<th>Aging Condition</th>
<th>With/Without Existing Joint</th>
<th>Strong Base/Weak Base</th>
<th>Observed SIRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0</td>
<td>10-day</td>
<td>With</td>
<td>Weak Base</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>1.0</td>
<td>10-day</td>
<td>With</td>
<td>Weak Base</td>
<td>Yes</td>
</tr>
<tr>
<td>3</td>
<td>1.5</td>
<td>10-day</td>
<td>With</td>
<td>Weak Base</td>
<td>Yes</td>
</tr>
<tr>
<td>4</td>
<td>1.0</td>
<td>10-day</td>
<td>Without</td>
<td>Weak Base</td>
<td>No</td>
</tr>
<tr>
<td>5</td>
<td>1.0</td>
<td>5-day</td>
<td>With</td>
<td>Weak Base</td>
<td>No</td>
</tr>
<tr>
<td>6</td>
<td>1.0</td>
<td>5-day</td>
<td>With</td>
<td>Weak Base</td>
<td>No</td>
</tr>
<tr>
<td>7</td>
<td>2.4</td>
<td>10-day</td>
<td>With</td>
<td>Strong Base</td>
<td>No</td>
</tr>
<tr>
<td>8</td>
<td>2.4</td>
<td>10-day</td>
<td>With</td>
<td>Strong Base</td>
<td>No</td>
</tr>
<tr>
<td>9</td>
<td>2.4</td>
<td>10-day</td>
<td>With</td>
<td>Strong Base</td>
<td>No</td>
</tr>
</tbody>
</table>

5.3 Test Results and Analysis

Testing results are shown in accordance with the specimen sequence that presented in Table 5.1. The specimens (#1 to #3) with observed SIRC are firstly introduced and analyzed, followed by these conditions that no crack are found (specimens #4 to #9).

5.3.1 Testing with Observed SIRC

Specimen #1 has 1 inch in thickness, 10-day aged, with existing joint at specimen bottom, and with rubber base placed below sample to simulate soft base in the field. The placement of rubber and specimen #1 in HWTD mold is shown in Figure 5.2.
It should be noted that the specimen thickness used in laboratory may not be equivalent to field pavement thickness directly. The reduced specimen size has been widely used to simulate field pavement condition in the laboratory. A commonly used reduced-scale accelerated trafficking device is the one third scale Model Mobile Load Simulator (MMLS3). Research studies have shown that application of MMLS3 loading creates stresses and strains in the pavement slab trafficked that are similar to those created in pavements subjected to field traffic if conditions of similitude are met (Kumar and Chehab, 2014).
Such scaled down is based on dimensional analysis (Kim et al., 1995; Ven et al., 1998) which shows that: if N is the scaling factor applied between prototype and model, to get the same strains and stress in the model for inertial effect, it is necessary to scale down the length by 1: N and load magnitude by 1:N², while keeping 1:1 scale for material properties and stress on surface. For the MMLS3, these conditions entail scaling down of the pavement layer thickness and traffic velocity to 1/3rd (N=3) and applied load to 1/9th of those experienced in the field respectively. The tire inflation pressure of MMLS3 is kept as a constant of 600 kPa.

The scale down factor N is not a fixed number and typically is determined by users (Kim et al., 1995). In this study, a same scale down factor as MMLS3 is applied to HWTD (N=3) to take into account of the mold geometry size of the existing HWTD test. Then the specimen size is equivalent to 3 to 4.5 inch in thickness in field. The HWTD provides a pressure of 0.69 MPa.

In this test, 40,000 passes is applied daily with a testing period of 12-13 hours. After test is ended on each day, specimens are taken out to check if there is any crack initiated at either the surface or the bottom of the specimen.

As can be seen in Figure 5.3, it is found that after 40,000 wheel passes, a minor crack appeared at specimen surface. The crack is perpendicular to wheel loading direction and therefore can be considered as a transverse crack. In addition, the crack location matches well with existing crack/joint at specimen bottom, indicating that the existing crack/joint may be a critical reason for the initiation of such crack type. It is also seen that there is recordable rut depth.
Figure 5.3 Specimen with surface-initiate reflective crack, (a) specimen within in mold, and (b) specimen taken out.

The specimen was continued testing for additional 30,000 passes. As seen in Figure 5.4 (a), after in total 70,000 passes, the original crack further propagates to the outside of the specimen, and at the same time, the crack initiates at the other side of the specimen (figure 5.4 (b)). As shown in Figure 5.4 (c), the surface transverse crack matches well with existing transverse crack/joint. It is also seen that the rut depth increases in the end of test compared with that at 40,000 cycles.

It is interesting to note that the specimen bottom is squeezed from outside toward to the specimen middle in Figure 5.4 (d). By seeing Figure 5.4 (c) and (d) together, it is reasonable to assume that when wheel load moves on one side of the existing transverse cracking (the near side), the specimen on this side tends to bend downward under wheel load especially when the support is relatively soft. If without constraint, the other side of the specimen (the far side) should consequently be tilted upward. However, the bonding with base layer and the self-weight of the specimen restrict the upward movement of the far side specimen. Therefore, tensile stress at the surface of the specimen on top of the bottom crack is created. At the same time, the bottom
crack has the potential of being squeezed and pushed together. A schematic to explain the hypothesized causes of such crack is shown in Figure 5.5.

Figure 5.4 Specimen with SIRC, (a) specimen with propagated transverse crack; (b) transverse crack initiates at the other side; (c) SIRC matches well with existing joint, and (d) specimen bottom.
A repetition specimen (#2 in Table 5.1) uses the same conditions (specimen thickness of 1.0 inch, 10-day aging, with existing joint and with weak base) is tested and specimen after 80,000 passes is shown in Figure 5.6. As seen, the transverse crack initiates from surface as well and the specimen bottom tends to squeeze to the middle. Similar to the first specimen shown in Figure 5.4, the repetition specimen has the same crack direction (perpendicular to wheel load) and location (at pavement surface) and matches well with existing joint. The repetition test proves that the appearance of SIRC is not happened by accident in the laboratory test, it should appear as long as specific conditions are satisfied (weak base, long-term aged material, proper layer thickness, with existing joint). Recordable rut depth is seen as well for this specimen.

Figure 5.5 Mechanism sketch for SIRC.

Figure 5.6 Repetition specimen with SIRC, (a) specimen top, and (b) specimen bottom.
Specimen of #3 that listed in Table 5.1 also showed SIRC. Specifically, the specimen is 1.5 inch in thickness, 10-day aged, with existing joint at specimen bottom and with weak base placed below sample. Testing result shows that it takes in total 180,000 passes before surface transverse crack was observed. As seen in Figure 5.7, there is a transverse crack in the middle of the specimen under wheel load, and the specimen bottom severely squeezes to the middle. Test was continued for additional 60,000 passes, but the crack was not further developed. Recordable rut depth is seen as well for this specimen.

Considering that the only difference between #3 and specimens #1 and #2 is layer thickness, it can be concluded that the layer thickness is a critical factor for SIRC. It is possible that with the increase of layer thickness, the traffic load-induced bend on specimen end decreases as well. This has been confirmed by the FEM analysis in Chapter 4 that the magnitude of the Von Mises stress at pavement surface decreases with the increase of HMA layer thickness. More traffic repetitions are required to induce the same stress level for thicker pavement compared with thinner pavement. Also, unlike the specimens that shown in Figures 5.4 and 5.6, the crack is seen within wheel load while no crack is found on the two sides, the reason for such crack location needs to be further evaluated.
5.3.2 Testing without Observed SIRC

Other test conditions are also evaluated while no SIRC is found. Detailed testing of each specimen and analysis is listed as follows.

Specimen #4 in Table 5.1 has 1 inch in thickness, 10-day aged, without existing joint at specimen bottom and with weak base placed below sample. Test result shows that after 100,000 passes, no crack is observed on either specimen surface or bottom, as seen in Figure 5.8. Considering that the only difference between #4 and specimens #1 and #2 (test ends with crack) is with or without existing joint, it can be concluded that the existing joint is a critical pre-condition for such cracking type.
Figure 5.8 Specimen without existing joint after testing, (a) specimen top, and (b) specimen bottom.

In addition, specimen #5 and #6 that listed in Table 5.1 are evaluated. Specifically, they are both 1 inch in thickness, 5-day aged, with existing joint at specimen bottom, and weak base is placed below both samples. Test result shows that after 100,000 passes, no crack was observed on either specimen surface or bottom, as seen in Figure 5.9. Considering that the only difference between #5, #6 and specimens #1 and #2 (test ends with crack) is material aging (5-day and 10-day aging respectively), it can be concluded that the aging (stiffness or brittleness) is a critical pre-condition for SIRC.
Specimens #7, #8, and #9 are also evaluated. Specifically, they are 2.4 inch in thickness, 10-day aged, with existing joint at specimen bottom. Since 2.4 inch is the maximum specimen thickness for HWTD, no rubber layers are used in the test. In such case, the base of specimen is the metal model, which is very strong in terms of stiffness and can be considered as strong base. The three specimens are different from each other in terms of existing joint depth. The joint depth is 0.3 inch, 0.6 inch, and 1.2 inch for #7, #8 and #9 respectively. This is designed to evaluate the effects of varied joint depths on SIRC. Specimens and test setup are shown in Figure 5.10.
Figure 5.10 Specimen tested with full thickness of 2.4 inch, (a) specimen with varied joint depth, and (b) specimen placed in molds.

No crack is observed after in total 1,500,000 passes, as seen in Figure 5.11. There is no material squeezing at specimen bottom as well. The phenomenon that no crack was observed under such conditions could attribute to the following reasons: (1) the ratio of specimen thickness to length (0.46) is too high to induce specimen bending, and (2) the strong base is used, which could reduce the stress level at pavement surface and not allow specimen bending.

Figure 5.11 Specimen tested with full thickness of 2.4 inch, (a) specimen top, and (b) specimen bottom.

Table 5.2 summarizes the cycle at crack initiation, the rut depth at crack initiation, cycles of the final test, as well as final rut depth. As shown, the surface crack appears very fast in terms of specimens #1 and #2 with 1 inch in thick, while it takes more passes to initiate crack when thicker layer is used (#3). It is also seen that crack is always observed with different levels of rut depth.
## Table 5.2 Summary of Crack Initiation Cycles and Rut Depth

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Observed SIRC</th>
<th>Cycle at Crack Initiation</th>
<th>Rut Depth at Crack Initiation, mm</th>
<th>Total Test Cycle</th>
<th>Final Rut Depth, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Yes</td>
<td>40,000</td>
<td>5.0</td>
<td>70,000</td>
<td>7.0</td>
</tr>
<tr>
<td>2</td>
<td>Yes</td>
<td>40,000</td>
<td>5.5</td>
<td>80,000</td>
<td>7.0</td>
</tr>
<tr>
<td>3</td>
<td>Yes</td>
<td>180,000</td>
<td>2.0</td>
<td>240,000</td>
<td>3.0</td>
</tr>
<tr>
<td>4</td>
<td>No</td>
<td>N/A</td>
<td>N/A</td>
<td>100,000</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>No</td>
<td>N/A</td>
<td>N/A</td>
<td>100,000</td>
<td>10.0</td>
</tr>
<tr>
<td>6</td>
<td>No</td>
<td>N/A</td>
<td>N/A</td>
<td>100,000</td>
<td>10.0</td>
</tr>
<tr>
<td>7</td>
<td>No</td>
<td>N/A</td>
<td>N/A</td>
<td>1,500,000</td>
<td>6.0</td>
</tr>
<tr>
<td>8</td>
<td>No</td>
<td>N/A</td>
<td>N/A</td>
<td>1,500,000</td>
<td>6.0</td>
</tr>
<tr>
<td>9</td>
<td>No</td>
<td>N/A</td>
<td>N/A</td>
<td>1,500,000</td>
<td>6.0</td>
</tr>
</tbody>
</table>

### 5.4 Summary of Laboratory Validation

This chapter evaluates the key factors that are correlated to SIRC that found from FEM simulation. Based on both FEM simulation and laboratory test, it is concluded that

1. The specimen thickness, base stiffness, material stiffness (aging effect) and existing joint are critical factors for SIRC. Specifically, both FEM simulation and laboratory test indicate that such crack type is seen easier for softer base, stiffer asphalt mixture with existing transverse joint.

2. In terms of pavement thickness, FEM simulation shows SIRC under traffic load appears if thickness above existing joint is thick enough ($\geq$4.5 inch). While laboratory test indicates that specimen with thinner thickness (1 inch) takes less wheel load passes before crack was observed. Two reasons could be used to explain this difference, (a) thin specimen cannot be directly equal to field thin HMA layers, scale down factor needs to be considered in laboratory test, and (b) in the FEM simulation, both 4.5-inch and 6-inch thickness above existing joint can lead to the maximum stress at the pavement surface.
However, as shown in Chapter 4, the maximum stress level of 6 inch pavement thickness is lower than that of 4 inch pavement thickness. This indicates that sufficient thickness (such as a threshold level of 4.5 inch as found in this study) is important to shift the critical distress position to the surface of the pavement and lead to surface initiated cracking. However, above the threshold value, the pavement with thinner thickness is more prone to develop SIRC and crack may appear with less traffic repetitions.

(3) SIRC can be induced under traffic load. Although thermal load gradient (cooling cycles) could accelerate the development of the surface-initiated transverse cracking, both FEM simulation and laboratory evaluation proved that such crack type can be caused by repeated traffic load at intermediate temperature.

(4) The possible mechanism that induce the SIRC is sketched in Figure 5.5. As seen, with existence of transverse joint at pavement bottom, it is very possible that when traffic load tracks back and forth in the middle of the road, a bending is induced in upward direction near the load area. Therefore, tensile stress is caused on pavement surface and tears the pavement apart under repeated wheel load. Compression is caused on pavement bottom and leads to asphalt mixture squeeze together, which has been validated by the laboratory tests as seen in Figure 5.4 (d). Such bending may only occur when the base structure is relatively weak. If strong base is applied, no bending is expected and SIRC is not seen any more. When the tensile stress is higher than strength of material, the crack initiates. The asphalt material with higher stiffness or brittleness may crack easier than soft material since soft asphalt mixture can relax stress faster.
CHAPTER 6 VALIDATION OF TRANSVERSE CRACKING MODEL EMBEDDED IN AASHTOWARE PAVEMENT ME DESIGN PROGRAM

The transverse cracking prediction model in AASHTOWare Pavement ME Design program is widely accepted and used. In this chapter, prediction model from AASHTOWare will be evaluated using field distress data. It is hypothesized that the model embedded in the AASHTOWare cannot fully capture/predict the transverse cracking performance of asphalt pavement (Shen et al., 2013), especially for the new cracking type, SIRC, as described in this dissertation. This analysis will thus confirm the need to develop new predictive models for transverse cracking that can consider all three types of transverse cracking.

6.1 Introduction to AASHTOWare Pavement ME Design

The Mechanistic-Empirical Pavement Design Guide (MEPDG) was developed by NCHRP Project 1-37A under funding from the AASHTO. The purpose of the MEPDG is “to provide the highway community with the state-of-the-practice for the design of new and rehabilitated pavement structure, based on mechanistic-empirical principles” (NCHRP, 2004). The newest version of the software is DARWin-ME, currently named AASHTOWare Pavement ME Design program, which was released by AASHTO in April 2011.

The design and analysis procedure calculates pavement responses (stresses, strains, and deflections) and uses those responses to compute incremental damage over time. The procedure
empirically relates the cumulative damage to observed pavement distress. The software program is able to predict major flexible pavement distress, including IRI, permanent deformation, fatigue cracking, and thermal cracking by using nationally calibrated performance models considering traffic, climate, pavement structure, and material properties.

There are three hierarchical levels in AASHTOWare Pavement ME Design program, Level 1, Level 2, and Level 3, with the accuracy of prediction increasing from Level 3 to Level 1 (AASHTO, 2008). The use of each level will depend on the available data and the most accurate level cannot always be ensured. In this chapter, a combination of the three levels was used due to limited information, especially for existing asphalt concrete (AC) and Portland cement concrete (PCC) layers, as well as base and subgrade layers.

6.2 Data Collection for AASHTOWare Pavement ME Design Program

A total of 16 projects from 12 states located at different climatic zones of the United States are used to validate prediction model in the ME program. A total of 43 pavement sections including 15 HMA sections and 28 WMA sections were included.

The input data required in the program are traffic, climate, material properties and pavement structure. Because slight variation between the HMA and WMA pavements usually exist, the actual pavement thickness, density, base, and subgrade material properties for each HMA/WMA section obtained from specific project data and FWD tests were used as inputs.

6.2.1 Field Performance Data

For each field HMA and WMA pavement section, three 200-foot research sections were randomly selected. The visual inspections were conducted to identify and classify any distresses
in accordance with the LTPP manual (*Miller and William 2003*). Pictures and videos were also taken for validation purpose.

FWD test was also conducted in general accordance with FHWA-LTPP test protocol, *Manual for Falling Weight Deflectometer Measurements (LTPP manual 2000)*, in the outside wheelpath. The measured results were used to estimate the in situ moduli of existing pavement layers using the back-calculation program Modcomp V6. Using the approach suggested by Irwin (Irwin 1994), the layer moduli of each layer were back-calculated for each testing spot and averaged over the entire research section to reduce random error. The back-calculated base and subgrade moduli were converted to laboratory test results by multiplying C-Value (*AASHTO 2008*).

### 6.2.2 Laboratory Testing of Material Property

Laboratory testing obtains mechanical/engineering properties of WMA and HMA materials. It also provides inputs for Pavement ME analysis. A summary of laboratory tests on field cores conducted in this study for both mixtures and asphalt binders are shown in Table 6.1. It should be noted that other testing, for instance, mixture and binder fatigue testing based on field cores were also performed while only testing related to Pavement ME analysis inputs are presented here. For conciseness, detailed descriptions about laboratory testing can be found elsewhere (*Shen et al., 2013*).
Table 6.1 Summary of Laboratory Testing Performed

<table>
<thead>
<tr>
<th>Laboratory Test</th>
<th>Specification (Or reference)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixture IDT Dynamic Modulus</td>
<td>*(Wen and Kim, 2002)*³</td>
</tr>
<tr>
<td>Mixture IDT Creep Compliance</td>
<td>AASHTO T 322</td>
</tr>
<tr>
<td>Mixture IDT Thermal Cracking²</td>
<td>AASHTO T 322, <em>(Wen and Bhusal, 2012)</em></td>
</tr>
<tr>
<td>Mixture Hamburg Rut Depth</td>
<td>AASHTO T 324</td>
</tr>
<tr>
<td>Asphalt Binder Performance Grade (PG) Test</td>
<td>AASHTO MP 1</td>
</tr>
</tbody>
</table>

Note: 1. Most of the test were performed by Mr. Shenghua Wu and Mr. Weiguang Zhang at Washington State University.
2. The temperature selected for thermal cracking depends on the low temperature PG grade of asphalt binder (AASHTO T 322). In this study, 14 °F is the appropriate testing temperature.
3. Numbers in the parentis provide the reference of the specific testing method.

6.2.3 Traffic Information and Other Information Collected

Table 6.2 provides a summary of traffic input, operational speed and number of lane in design direction for the in-service pavement projects used in this study. A traffic growth factor of 3% was used for all the projects.

Other information (construction year, pavement structure and volumetric properties) that required in AASHTOWare Pavement ME program are listed in Table 6.3.

Table 6.2 Traffic Information Summary

<table>
<thead>
<tr>
<th>Project Name</th>
<th>AADT</th>
<th>Percent of Truck, %</th>
<th>Initial AADT</th>
<th>Operational Speed, mile/hour</th>
<th>Number of Lane in Design Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>OH 541</td>
<td>650</td>
<td>1.54</td>
<td>10</td>
<td>35</td>
<td>1</td>
</tr>
<tr>
<td>MD 925</td>
<td>10,480</td>
<td>3.6</td>
<td>377</td>
<td>35</td>
<td>1</td>
</tr>
<tr>
<td>MN 169</td>
<td>12,600</td>
<td>11.24</td>
<td>1,416</td>
<td>35</td>
<td>1</td>
</tr>
<tr>
<td>CO I-70</td>
<td>30,000</td>
<td>10</td>
<td>3,000</td>
<td>65</td>
<td>2</td>
</tr>
<tr>
<td>WA SR 12</td>
<td>6,550</td>
<td>16</td>
<td>1,048</td>
<td>60</td>
<td>2</td>
</tr>
<tr>
<td>PA SR 2012</td>
<td>254</td>
<td>3</td>
<td>8</td>
<td>45</td>
<td>1</td>
</tr>
<tr>
<td>WA I-90</td>
<td>13,000</td>
<td>26</td>
<td>3,380</td>
<td>60</td>
<td>2</td>
</tr>
<tr>
<td>Project Name</td>
<td>Pavement Structure</td>
<td>Construction Year</td>
<td>Air Voids, %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------------</td>
<td>--------------------</td>
<td>--------------------</td>
<td>--------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OH 541</td>
<td>1.25'' overlay+6.75''HMA+9''base</td>
<td>2006</td>
<td>4.7(HMA) 6.6(Sasobit)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MD 925</td>
<td>2'' overlay+5''HMA+8''base</td>
<td>2005</td>
<td>6.1(HMA) 6.0(Sasobit)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MN 169</td>
<td>2'' overlay+8''HMA+6''base</td>
<td>2010</td>
<td>8.6(HMA) 8.7(Evotherm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CO I-70</td>
<td>2.5'' overlay+10.5''HMA</td>
<td>2007</td>
<td>3.0(HMA) 3.6(Sasobit)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WA SR 12</td>
<td>3'' overlay+7.8''HMA+9.8''base</td>
<td>2010</td>
<td>1.8(HMA) 3.1(AQB)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PA SR 2012</td>
<td>1.5'' overlay+5''HMA+4''base</td>
<td>2009</td>
<td>7.9(HMA) 5.3(Gencor) 5.4(LEA)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WA I-90</td>
<td>3'' overlay+11.3''HMA+5.8''base</td>
<td>2008</td>
<td>3.0(HMA) 3.0(Sasobit)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NV Bravo</td>
<td>6'' overlay+9''base</td>
<td>2008</td>
<td>2.2(HMA) 2.9(Foaming)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>IL 147</td>
<td>1.5'' overlay+7.5''HMA</td>
<td>2010</td>
<td>6.4(HMA) 8.4(DBG)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC 178</td>
<td>2'' overlay+5.7''HMA+7''base</td>
<td>2007</td>
<td>7.7(HMA) 6.9(Evotherm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MO CC</td>
<td>3.75'' overlay+7''PCC+6''base</td>
<td>2007</td>
<td>4.7(HMA) 4.9(Evotherm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MO Hall</td>
<td>1.75'' overlay+12''PCC+1.5''base</td>
<td>2006</td>
<td>4.6(HMA) 4.1(Sasobit)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PA SR 2006</td>
<td>1.5'' overlay+5''HMA+4''base</td>
<td>2009</td>
<td>4.0(Evotherm) 3.3(Aspha-min)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TX 251</td>
<td>2'' overlay+4.3''HMA</td>
<td>2008</td>
<td>5.1(HMA) 6.3(Sasobit)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CA 3a</td>
<td>2.4'' overlay+2.4''HMA+15.7''base</td>
<td>2010</td>
<td>8.8(Advera)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CA 3b</td>
<td>2.4'' overlay+2.4''HMA+15.7''base</td>
<td>2010</td>
<td>4.4(HMA) 4.5(DBG)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note 1, 2: Projects CA 3a and 3b were evaluated using Heavy Vehicle Simulator (HVS). The repetitions were ranged from 74,000 to 224,000 for CA 3a, and ranged between 128,500 and 450,000 for CA 3b.
6.2.4 Climatic Information

For climate data, the AASHTOWare Pavement ME Design program contains 851 stations covering four climatic zones nationwide. The closest station to each project site was used for prediction to ensure the similar climate conditions.

6.3 Correlation between Field Transverse Cracking and Predicted Thermal Cracking

Figure 6.1 shows the relationship between field measured transverse cracking and ME predicted thermal cracking, with HMA and WMA results separated. As shown, the field values range from 0 to 12685.2 ft./mile, while the ME predicted thermal cracking located between 83.4 to 2592.7 ft./mile. The coefficient of determination value ($R^2$) equals to as low as 0.21, indicating the ME prediction cannot predict field measurements well. It is also shown that no significant prediction quality difference is observed between HMA and WMA.

Among all the 43 sections evaluated, 28 has predicted values close to 83.4 ft./mile and 10 has predicted values equal to 2592.7 ft./mile, only 5 sections have other values between. In the ME program, 83.4 and 2592.7 ft./mile are the minimum and the maximum prediction values respectively based on 95% reliability level. Since most predicted thermal cracking either have no predicted values or reach to maximum values, while field measurements located at different severity levels, a bad correlation between field and predicted values is observed. The reason that causes so many predictions located to either minimum or maximum values could be due to the two assumptions that applied in the current thermal cracking prediction TCMODEL (Hiltunen and Roque, 1994):
1. A cracking is not counted as a crack until the local vertical crack propagates through the entire depth of asphaltic concrete surface layer, and

2. As long as the crack depth reach the maximum value (the depth of total HMA layers), the amount of thermal cracking will be maximized accordingly. And the value will be a constant and will not change any more.

![Figure 6.1 Correlation between field transverse cracking and predicted thermal cracking.](image)

In the recently completed NCHRP 9-47A project, the short-term field performance indicates that transverse cracking is the most common type of cracking out of all the 14 projects. However, the ME predicted zero thermal cracking in most cases, which confirms the above conclusion (West et al., 2014).

In addition, Spearman’s rank correlation coefficient is used to statistically rank the rut depth for both field measured and predicted values, and provides an evaluation of the relative correlation between the two. Rank correlation coefficient measures the degree of similarity
between two rankings, and neglects the effect of magnitude. For instance, if the rank of prediction matched well with field rank, then the prediction can still be used to distinguish performance differences among different technologies, even though the ME program could over-predicts/under-predicts field values.

Spearman’s rank correlation coefficient can be used to measure the statistical dependence between two groups of variable. It assesses how well the relationship between two groups of variable can be described using a same function (Cabilio and Tilley, 1999). The Sperman’s rank correlation coefficient ranges from -1 to 1. The sign of the coefficient indicates the direction of association between two variables. Specifically, if ME predicted values tend to increase when field measurements increase, the coefficient is positive. Alternatively, the coefficient is negative if ME predicted values tend to decrease when field measurements increase. The higher absolute value of coefficient indicates higher correlation between two ranks. A coefficient of zero means that the rankings are completely independent.

The equation to calculate Spearman’s rank correlation coefficient is shown in Equation (6.1).

\[
\rho = 1 - \frac{6\sum_{i=1}^{n} d_i^2}{n(n^2 - 1)}
\]  

(6.1)

Where \(d_i\) indicates the difference between ranks and \(n\) equals to number of variable.

Table 6.4 shows how the \(d_i\) in equation (6.1) was obtained based on transverse cracking. As shown, both field measured and ME predicted cracking are ranked from maximum rut depth (assigned to 1) to minimum rut depth (assigned to 43/22 due to several equivalent values). The
Spearman’s rank correlation coefficient can then be calculated based on equation (6.1) and \( \rho \) equals to 0.4. The \( \rho \) value (0.4) shows that it is hard to conclude that the ranking of ME predicted thermal cracking matches well with the ranking of field transverse cracking. It also indicates that the ME predicted values tend to increase when field measurements increase.

### Table 6.4 Spearman’s Rank Correlation Coefficient for Transverse Crack

<table>
<thead>
<tr>
<th>Project Name</th>
<th>Technology</th>
<th>Predicted Thermal Crack, ft./mile</th>
<th>Field Transverse Crack, in.</th>
<th>Ranking of Prediction (1)</th>
<th>Ranking of Field (2)</th>
<th>( d [(1)-(2)] )</th>
<th>( d^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>OH 541</td>
<td>HMA</td>
<td>2592.7</td>
<td>12685.2</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Sasobit</td>
<td>2592.7</td>
<td>5005.4</td>
<td>1</td>
<td>5</td>
<td>-4</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>Evotherm</td>
<td>2592.7</td>
<td>5121.6</td>
<td>1</td>
<td>4</td>
<td>-3</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>Aspha-min</td>
<td>2592.7</td>
<td>4881.4</td>
<td>1</td>
<td>6</td>
<td>-5</td>
<td>25</td>
</tr>
<tr>
<td>MD 925</td>
<td>HMA</td>
<td>83.4</td>
<td>237.6</td>
<td>40</td>
<td>19</td>
<td>21</td>
<td>441</td>
</tr>
<tr>
<td></td>
<td>Sasobit</td>
<td>85.3</td>
<td>831.6</td>
<td>17</td>
<td>15</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Evotherm</td>
<td>2225.5</td>
<td>11014.1</td>
<td>11</td>
<td>2</td>
<td>9</td>
<td>81</td>
</tr>
<tr>
<td>MN 169</td>
<td>HMA</td>
<td>1078.2</td>
<td>6528.7</td>
<td>14</td>
<td>3</td>
<td>11</td>
<td>121</td>
</tr>
<tr>
<td></td>
<td>Evotherm</td>
<td>83.8</td>
<td>1557.6</td>
<td>36</td>
<td>9</td>
<td>27</td>
<td>729</td>
</tr>
<tr>
<td>CO I-70</td>
<td>HMA</td>
<td>83.4</td>
<td>245.5</td>
<td>41</td>
<td>18</td>
<td>23</td>
<td>529</td>
</tr>
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<td>21</td>
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<td></td>
<td>AQB</td>
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<tr>
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<tr>
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<td>1444.1</td>
<td>1</td>
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<td>-9</td>
<td>81</td>
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<td>942.5</td>
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<td>660.0</td>
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<td>-15</td>
<td>225</td>
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</table>
In this chapter, to determine the capability of Pavement ME Design program in predicting transverse cracking of flexible pavement, a correlation between predicted performance and field measured performance was analyzed for 43 pavement sections. Furthermore, Spearman’s rank correlation coefficient was used to evaluate if the Pavement ME Design program could provide consistent trend (ranking) with field measurement. Based on the comparison and analysis, the following findings are obtained:

(1) It was found that the Pavement ME predicted thermal cracking does not match well with the field transverse cracking measurements for both HMA and WMA pavements. Most ME predictions have either minimum or maximum values, which could be partially attributed to the assumptions that are used in the TCMODEL;

(2) Based on Spearman’s rank correlation coefficient, it is found that the ranking of ME predicted thermal cracking does not match well with the ranking of field transverse cracking;
(3) A new model is expected that can correlate individual factors with field transverse cracking and can take into account different types of transverse cracking that described in Chapter 1.
As stated in Chapters 2 and 6, despite the prediction quality, most existing models have some common deficiencies for predicting field transverse cracking, such as:

(1) Most existing models only focused on one mechanism such as thermal cracking. However, field transverse cracking may be thermal or reflective cracking and could be caused by a combination effect of thermal and traffic variables. Clear separation among different crack types in the field is extremely difficult.

(2) Most existing models targeted only one of two situations: 1) on crack initiation to predict when, or at what temperature, pavement will crack, or 2) on crack propagation to estimate the amount of crack in the pavement. Very few models considered both crack initiation and propagation together. In fact, it is very possible that pavement may follow different mechanisms and trends for crack initiation and propagation.

(3) The confounding effect of field conditions such as construction and material variability could make a mechanistic-based or physical-based prediction model extremely challenging.

Therefore, the predictive models that can consider various transverse cracking types should be developed, and the models should be able to predict both crack initiation and propagation. A statistical method offers a systematic approach to investigate causality and identify the effect of changes in the values of predictors or independent variables on response(s) or dependent variables (Henseler et al., 2009). Through meaningful and reasonable data collection, organization, analysis,
and interpretation, statistics provide unique opportunities to analyze comprehensive engineering problems like pavement performance prediction.

Therefore, the objective of this chapter is to develop predictive models for field transverse cracking using statistical methods. The models should integrate all types of transverse cracking in the field and take into account the variability of field conditions. Two different models for crack initiation and crack propagation are established to determine whether they are governed by the same mechanisms (predictor variables). A Binary Logistic (BL) Regression model is used for crack initiation model to allow prediction of the probability of crack initiation under different scenarios. A Partial Least Squares (PLS) Regression model is used to predict the amount of crack propagation. Finally, this chapter identifies critical indicators to predict transverse cracking in pavement, and provides a ranking of their degree of influence. Because warm mix asphalt (WMA) has been widely used recently in asphalt pavement industry, both hot mix asphalt (HMA) and WMA are included in the model development.

7.1 Methodology

Multiple Linear Regression (MLR) is widely used in pavement engineering to determine the relationship between predictor variables and responses. When the factors are few in number, or not significantly collinear, or have a well-understood relationship to the responses, MLR can be a good method to use. However, if any of these three conditions fail, the MLR method can be inappropriate and the regression models could be misleading. Some researchers found that if collinearity exists between variables, the MLR can yield high coefficient of determination values.
(\(R^2\)) when some less significant predictor variables are involved, and even worse, can give incorrect signs for the parameters \((Yeniay and G\ddot{A}oktas, 2002)\). Here, the collinearity is defined as a high level of correlation between two predictor variables, and when more than two parameters are involved, this could be called multicollinearity. For a model with a small number of data but a relatively large number of predictor variables, MLR could obtain a good fitting model but the model may not be able to predict new responses well. This is called over-fitting \((Hawkins, 2004)\).

Statistically, a number of methods can be used to solve these problems (i.e., small dataset and high collinearity). The two most widely employed methods are the Partial Least Squares (PLS) and the Principal Components Regression (PCR) due to their simplicity, reliability and versatility \((Wentzell and Montoto, 2003)\). The PLS Regression method is selected for this study because: (a) it can make better predictions for regression models than the PCR method \((Yeniay and G\ddot{A}oktas, 2002)\); (b) it is time- and cost-effective because fewer latent factors are required \((Yeniay and G\ddot{A}oktas, 2002; Wentzell and Montoto, 2003)\), and (c) PLS Regression is most appropriate when sample sizes are relatively small \((Peng and Lai, 2012)\). Studies performed by Wold \((Wold et al., 1984)\) and Otto \((Otto and Wegscheider, 1985)\) showed that by using PLS, the model coefficients (both magnitude and sign) do not change substantially when new calibration responses are introduced.

The working principle of PLS can be understood together with MLR. When MLR is used, if the number of variables becomes too large, a model that fits the response data perfectly can be obtained. However, this model may not be able to predict new responses well (over-fitting). By contrast, PLS can help to select these latent factors that are responsible for most of the variation.
It is able to solve collinearity problems and to ensure only parameters that highly contribute to the prediction model are used.

In this study, a crack propagation model is developed using the PLS Regression method. For crack initiation, the Binary Logistic (BL) Regression method is used in conjunction with the PLS Regression method to develop a probabilistic based predictive model. BL Regression is a widely used statistical method for analyzing data with two categorical responses. By comparison, linear regression techniques (MLR, PLS Regression) are used with continuous response. The BL crack initiation model is unique however, in that instead of simply providing a binary result of “yes” or “no” for crack initiation, the BL model estimates the probability that the pavement will initiate a crack under the current condition. A higher probability will indicate a higher chance of crack initiation, just like a weather forecast.

The development of transverse crack propagation model is conducted using statistical software Minitab. LOOCV repeatedly divides data into two sets, one called a training dataset and the other a testing dataset. The training dataset is used to construct the model and the testing dataset is used to validate the model. Similar to a stepwise Multiple Linear Regression (MLR), PLS regression iterates the regression process by including different variables (different number of variables and different types of variables) but taking into account of collinearity. During each regression, PRESS (the prediction sum of squares between predicted responses and validated responses) is calculated. Smaller PRESS values indicate a better predictive quality. At the end of the analysis, Minitab will generate a relationship between PRESS versus number of variables. The number of variable that gives the minimum PRESS should be selected (Bulut and Alma 2012). By selecting the optimum number of predictor variables, Minitab will at the same time output the
preferred model using variables that have the highest standardized coefficients. The model with the optimum variable number will be validated by using LOOCV at the same time. This preferred model is thus optimized for variable number, variable type, and model coefficients.

Validation of the effectiveness of the prediction model is another critical component in the model development. There are many ways to validate the prediction model statistically, like k-fold Cross Validation (CV) and Leave One out Cross Validation (LOOCV). LOOCV is used in this study since it is always complete and every single data can be used. By using PLS (or MLR) method in conjunction with LOOCV, the model will decide the optimum variable number of independents, determine the coefficients of model parameters, and validate the model’s capability of predicting new performance at the same time.

The development of statistical based cracking prediction models mainly involves four steps:

- Data collection. Potential data that can be included into the analysis include but not limited to:
  - Dependents: pavement performance data such as crack area and crack length.
  - Independents: potential influencing factors for particular cracking distress such as climate, material properties, traffic, pavement structures, etc. These data should be included based on engineering experience and could vary due to data availability.

- Data preprocessing. The existence of collinearity, the randomness of data distribution, and other assumptions for the statistical analysis must be firstly checked to determine
the type of regression method to be used, the needs of data transformation, and other data preprocessing needs.

- Model development. Statistical analysis will be performed to select optimum number of independents, develop model parameters, and calibrate the models. This step usually can be performed using a commercial statistical software such as SPSS, Minitab. It is suggested that two different models, crack initiation models and crack propagation models being constructed respectively.

- Review of model structure. The reasonableness of the models must be reviewed according to their engineering meanings (influencing factors, trend of the relation, etc.). Sensitivity analysis should also be performed to determine the effect of individual factors on the overall performance.

Depending on the data input as dependents and independents, models can be developed either for local application or as a global performance model. It should be noted, however, statistical analysis must be accompanied by engineering judgment based on practical experiences during the entire process of model development. Relying purely on statistical models could sometimes give unreasonable results, which must be avoided (Sousa et al. 2014).

7.2 Data Collection

The development of a statistical model requires a large database. In this study, a total of 61 asphalt pavement projects (both HMA and WMA pavements) were investigated including 22 pavements with transverse cracking and 39 pavements without transverse cracking. These projects and the collected data were part of NCHRP 9-49A, a study investigating the long-term field performance
of WMA pavements. Details of these projects and testing results can be found in chapter 3. Data in five categories were collected for transverse cracking model development: pavement performance (transverse cracking), material properties (both mixtures and binders), pavement structure, climate, and traffic.

7.2.1 Field Performance Data

The amount of transverse crack was quantified according to LTPP manual (Miller and Bellinger, 2003) using crack mapping method along with pictures and videos for validation purpose. Cores taken at the crack tips confirmed that transverse crack could initiate either from the surface of pavement or from the bottom of the overlay; a mixed effect from traffic and climate can contribute to the transverse crack in the field. The distress was averaged from three randomly selected test sections. Each is 200 feet long. Weight factors, 1, 3.4, and 7.7, were used to take into account of the severity level of crack (crack width) in low, medium, and high levels respectively (WSDOT, 1999). Figure 7.1 shows a summary of the quantified transverse crack length using a weighted average method which ranged from 9 to 484.1 ft./200ft.

![Figure 7.1 Summary of Field Measured Transverse Crack (Weighted Crack Length).](image-url)
7.2.2 Material Properties

Field cores were used to conduct a suite of laboratory tests to determine the material physical and engineering properties. Specifically, the following material properties are considered as potential influencing variables for transverse cracking. The rationality for the selection of these parameters is also listed below.

- **IDT strength and creep compliance** are critical factors for both thermal and reflective crack prediction (*Hiltunen and Roque, 1994; Lytton et al., 2010*).

- **Mixture m-value calculated according to creep compliance** correlates with field thermal cracking (*Lytton et al., 1993*), and is also a material property considered for reflective cracking prediction (*Lytton et al., 2010*).

- **Work density at 14°F** is the area of the stress-strain curve from a constant shear rate IDT test divided by the volume of specimen to eliminate the effect of geometry. As a fracture property, work density describes both the strength and the ductility of a material. It was found to have a good agreement with the fatigue performance of Accelerated Loading Facility (ALF) pavements (*Wen, 2013*) and was also a promising indicator for field cracking performance (*Shen et al., 2013*).

- **Air voids** could affect the thermal coefficient of the contraction (*Lytton et al., 1993*) and fatigue performance of asphalt mixtures (*ARA 2004*).

- **The binder PG** is believed to correlate with thermal crack (*Schwartz, 2011*).

- **Binder stiffness and m-value from BBR test** correlates with field thermal crack (*Lytton et al., 1993*).
• **Asphalt content** has an important effect on mixture fatigue property (*ARA* 2004), although its effect on thermal crack is inconclusive (*Clyne et al.*, 2006).

• **The effective binder content** is a potential variable that could affect thermal crack (*Fugro Consultants Inc.*, 2011), and is used to predict fatigue cracking of HMA pavement as well (*ARA* 2004).

• **Percent passing #200 sieve** was selected for its effect on field thermal crack number (*Fromm and Phang*, 1972).

*7.2.3 Other Data Collected*

Total pavement thickness can affect the tensile stress distribution (*Hiltunen and Roque*, 1994; *Lytton et al.*, 1993). Asphalt overlay thickness was found to be a critical factor for reflective cracking (*Jongeun*, 2010). Both factors are considered in this study for their possible effect on transverse cracking. The pavement structure information was obtained by contacting local DOTs and agencies, and was verified by field cores.

The climatic data such as average yearly hour of low temperature was obtained from outputs of AASHTOWare Pavement ME Design program. The hour of low temperature used in this study is the number of hours when pavement surface temperatures are lower than 15°F as recommended (*ARA*, 2003).

Traffic load and service life (age) are highly related to reflective cracking (*Lytton et al.*, 2010) and could also accelerate thermal cracking damage (*Waldhoff et al.*, 2000). Service life (age) is defined as the number of years the pavement had been in service when the field cores were taken for this study. In this study, traffic data such as Average Annual Daily Truck Traffic (AADTT) as
well as other project related information (i.e., QC/QA, construction, mix design, etc.) were also obtained from local DOTs and agencies.

A summary of all collected data to be considered as predictor variables for model development is shown in Table 7.1. A total number of eighteen variables and sixty one HMA/WMA responses (twenty two with measured transverse crack, and thirty nine with zero measured transverse crack) were finally selected to perform the regression. In this case, there were more responses than variables for both crack propagation and crack initiation cases, which is the assumption for the “least-squares method”. It should be pointed out that when the number of responses (m) is less than the variables (n), there is an infinite number of solutions fit the equation, and then PLS can be used continually by deleting some variables to make sure m>n (Geladi and Kowalski, 1986).

<table>
<thead>
<tr>
<th>Predictor Variables</th>
<th>Unit</th>
<th>Range</th>
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<tr>
<td>IDT strength, 14°F</td>
<td>Mpa</td>
<td>2.17-5.56</td>
</tr>
<tr>
<td>Creep compliance (D1), -4°F</td>
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<td>0.04-0.16</td>
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</tr>
<tr>
<td>Work density, 14°F</td>
<td>Mpa</td>
<td>0.02-0.13</td>
</tr>
<tr>
<td>Air voids</td>
<td>%</td>
<td>1.8-9.1</td>
</tr>
<tr>
<td>Binder low PG</td>
<td>°C</td>
<td>-6.5 to -28.4</td>
</tr>
<tr>
<td>Binder stiffness, BBR</td>
<td>Pa</td>
<td>21.5-414.2</td>
</tr>
<tr>
<td>m-value, BBR, -6°C</td>
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</tr>
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<td>Asphalt content</td>
<td>%</td>
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</tr>
<tr>
<td>Effective binder content</td>
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<td>6.7-14.4</td>
</tr>
<tr>
<td>Passing #200 sieve</td>
<td>%</td>
<td>2.8-11.9</td>
</tr>
<tr>
<td>Service life (age)</td>
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<tr>
<td>Hour of low temperature</td>
<td>hour</td>
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<tr>
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<tr>
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<td>1.0-6.0</td>
</tr>
<tr>
<td>AADTT</td>
<td>N/A</td>
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</table>

Note: 1. Numbers follow by predictor variables indicate testing temperature.
7.3 Data Preprocessing

7.3.1 Check for Collinearity

A Variance Inflection Factor (VIF) was firstly used to check if collinearity exists between predictor variables. The VIF provides an index that measures how much the variance of an estimated regression coefficient is increased because of collinearity. The equation used to calculate VIF for the $i^{th}$ predictor variable $X_i$ is:

$$VIF_i = \frac{1}{1-R_i^2}$$  \hspace{1cm} (7.1)

where,

$R_i^2$ = coefficient of multiple determination when $X_i$ is regressed on the other predictor variables.

A threshold value of 10 is typically used for VIF (Henseler et al., 2009). That is to say, when $VIF_i$ is greater than 10, the $i^{th}$ predictor variable is considered to be highly correlated (multicollinear) with the other predictor variables. The VIF values calculated in this study showed that many predictor variables hold high VIF values (greater than 10). For example, Figure 7.2 shows multicollinearity between creep compliance and hour of low temperature, as well as work density and m-value (BBR). Therefore, MLR is not suitable for this analysis. A method like PLS Regression that can take care of high multicollinearity must be applied.
Figure 7.2 Examples of multicollinearity (a) creep compliance and hour of low temperature (<15°F); (b) work density and m-value, BBR.

7.3.2 Check for Collinearity

Randomness of data must be checked prior to any statistical modeling to determine if any transformations of the response or variables are necessary. If the model can fit the data well, the residual would approximate randomly distributed (Urbano-Marquez et al. 1989). A plot of standardized residuals versus fitted values (predicted response) can be used for this purpose. As seen in Figure 7.3 (a), a non-random structure is evident in the residuals, which is a clear sign of poor fitting. A number of data transformation methods were tried and finally natural logarithmic transformation was selected. The transformed data are shown in Figure 7.3 (b). As seen, the transformed data are randomly distributed, indicating a well-behaved standardized residuals versus fitted values. Therefore, natural logarithmic transformed data will be further used for model development.
7.4 Identification of Number of Predictor Variables

One of the most critical steps for PLS regression is to determine the number of predictor variables. The number of predictor variables controls the complexity of the model and its predictive ability (McWilliams and Montana, 2010). Specifically, (1) if more than required number of predictor variables is selected, more noise would be added to the data and will result in over-fitting; (2) if the number of predictor variables is too small, meaningful data for model calibration might be discarded (Darwish et al., 2013).

Cross-validation (CV) method is used in this study in order to determine proper number of predictor variables. It is a widely used method to predict the optimum number of predictor variables of PLS regression method (Darwish et al., 2013; Stone, 1974; Geladi, 1986; Niazi and Azizi, 2008). CV repeatedly divides data into two sets, one called training dataset and the other one called testing dataset. Training dataset is used to construct the model and testing dataset is
further used to validate the model. The most popular methods are leave one out (LOO) CV and k-fold CV. The LOOCV method is used in this study since it is always complete and every single data can be used.

The CV selection criterion of predictor variable used in this study is Prediction Sum of Squares (PRESS), which was proposed by Wold et al. (1984). The equation for PRESS is shown below,

\[ PRESS = \sum_{i=1}^{n} (y_i - \hat{y}_{(i)})^2 \]  

(7.2)

Where,

\( y_i \) = field transverse crack for the \( i^{th} \) observation, and

\( \hat{y}_{(i)} \) = predicted responses when the regression model is fitted to a sample of \( n-1 \) observations with the \( i^{th} \) observation omitted.

The smaller PRESS values indicate a better model to predict. Usually the PRESS reaches a minimum value and then rise again. Therefore, the number of variables that gives the minimum PRESS is selected (Geladi, 1986; Bulut and Alma, 2012). Figure 7.4 shows the PRESS calculation results for crack propagation based on the data. As shown, with six predictor variables, model reaches the lowest PRESS, and thus the number of predictor variables is determined as six for crack propagation model. It was also addressed that the variable number can also be selected as a range since sometimes the PRESS values are very close to each other (Wold et al., 1984).
Figure 7.4 Summary of PRESS results at different number of predictor variable for crack propagation.

7.5 Determination of Model Coefficients

By using Minitab software to perform PLS Regression analysis, standardized coefficients for each predictor variable are determined. Standardized coefficients identify the importance of each predictor variable in the model. The higher the standardized absolute value of the coefficients, the greater their roles in the prediction model. For the crack propagation model based on 22 data points (22 pavements with transverse cracking), 6 predictor variables with the highest standardized coefficients (absolute values) are selected and are shown in Table 2 with their corresponding trend. The positive trend indicates high values correlate with high transverse crack distress and vice versa. The negative trend indicates that high values correlates with low transverse crack distress and vice versa.
It can be noticed that the trend of service life (age) is negative, which does not follow engineering judgment. The reason could be the crack amounts used for analysis are from different projects. Some relatively new projects have very high amount of cracks (e.g., MN 169 project, with service life (age) of 2 years) due to cold climate and other possible reasons (poor mix design, etc.), while some projects have very few cracks even after a long time of service (e.g., MD 925 project, with service life (age) of 7 years) because of warmer climate, lower traffic, or proper mix design, etc. It was also confirmed that there was almost no relationship between transverse cracking length and service life (age). In fact, as can be seen later from the crack initiation model, the service life (age), combined with other factors like climate, has more influence on the crack initiation than on crack propagation. Once the crack has started, other factors like material properties and pavement thickness have more impact on the development of crack length. Therefore, the service life (age) was excluded from the crack propagation model consideration.

Then the PRESS is calculated again and the optimum number of predictor variables is still six. PLS Regression is performed without service life (age) at this time. The crack propagation model with six highest standardized coefficients predictor variables is shown in Equation (7.3). This model has a modified $R^2$ of 0.82, standard error of estimate of 0.47, and a Mallow’s Cp of 7.0 (Riani and Anthony, 2010), all of which indicate good prediction quality.

$$ Y = e^{(8.825-11.665 X_1 -9.587 X_2 +0.0033 X_3 -0.267 X_4 -1.047 X_5 +0.0006 X_6)} $$

(7.3)

where,

$Y = \text{weighted transverse crack length, ft./200ft.}$

$e = \text{base of the natural logarithm approximately equals to 2.718}$
\[X_1 = \text{work density, Mpa}\]
\[X_2 = \text{creep compliance (D_3), 1/Gpa}\]
\[X_3 = \text{hour of low temperature, hour}\]
\[X_4 = \text{percent passing #200 sieve, \%}\]
\[X_5 = \text{overlay thickness, in.}\]
\[X_6 = \text{Average Annual Daily Truck Traffic (AADTT)}\]

The effect of individual predictor variables are in good agreement with literature findings: (a) The mixture with a higher work density and creep compliance may be more fracture-resistant \((\text{Hiltunen and Roque, 1994; Wen, 2013})\); (b) more hours at a low temperature (<15°F) result in more transverse crack length \((\text{ARA, 2003})\); (c) within a specific range, a higher percent passing #200 sieve would introduce more fine aggregates and result in a more flexible mixture \((\text{Schwartz et al., 2011})\); (d) the thicker overlay thickness can help to reduce thermal/reflective crack \((\text{Hiltunen and Roque, 1994; Jongeun, 2010})\), and (e) it is generally agreed that higher traffic load (AADTT) can lead to more crack propagation.

Further, scaling of the data was used to determine the standardized coefficients of the six predictor variables. This helps eliminate the influence from individual variable’s unit, and is usually done to determine which of the predictor variables have a greater effect on the response \((\text{Yuan and Chan, 2011})\). Based on the standardized coefficients, creep compliance and hour of low temperature have the most significant effect on crack propagation among the six predictor variables, while the work density and percent passing #200 sieve have the least effect. The effects of overlay thickness and AADTT on crack propagation are close to each other.
Figure 7.5 shows the relationship between field-measured and predicted transverse crack according to Equation (7.3), with HMA and WMA results separated. As can be seen, most of the dots are distributed close to the line of equality, indicating that the prediction model has very good agreement with field measurements. It also appears that there is no significant differences between HMA and WMA, and the regression model has the capacity to predict transverse crack for both HMA and WMA pavements.

![Figure 7.5. Relationship between predicted and field measured transverse crack (crack propagation).](image)

**7.6 Model Validation**

Figure 7.6 indicates the validation of crack propagation model by using LOOCV method. As shown, most of the validated data located fairly close to the prediction data, indicating a good...
predictive power of the regression model given in Equation (7.3). In other words, it is expected that when the new validation data were introduced, the model should still work well.

![Figure 7.6. Validation of transverse crack propagation model.](image)

Figure 7.6 indicates the validation of crack propagation model by using pavements with existing PCC (8 sections in total), which were not included during the process of model development. As shown, most of the validated data located fairly close to the prediction data, meaning that the model can be used for such case as well.
7.7 Sensitivity Analysis

Sensitivity analysis is performed to test the robustness of the model prediction results considering the possible presence of uncertainty. The prediction results (not extremely high or low) will be evaluated when variables are varying in specific range. Many methods have been developed to carry out sensitivity analysis and the one used in this study is called one-at-a-time (OAT). For this method, one factor is being varied at each time and the others are kept at their baseline values.

A 10% increase and decrease of input variables (±10%) was applied as recommended (Schwartz et al. 2011). Sensitivity analysis results for crack propagation prediction model are shown in Figure 7.8. It indicates that the changes of transverse crack amount are acceptable with
the variation (±10%) of work density, creep compliance, hour of low temperature, percent passing #200 sieve, thickness of overlay, and AADTT.
Figure 7.8 Sensitivity of field transverse crack amount to variable changes (a) work density; (b) creep compliance; (c) hour of low temperature; (d) percent passing #200 sieve; (e) thickness of overlay, and (f) AADTT.

7.8 Development of Crack Initiation Model

The same procedure that is used for developing crack propagation model is applied to construct a crack initiation model. A total number of eighteen predictor variables and sixty one responses (twenty two with field crack, and thirty nine with zero field crack) are used for model regression. The optimum number of variables is determined as four by PRESS plot. The four predictor variables are further selected based on PLS regression analysis with the highest standardized coefficients (absolute values). The four predictor variables are IDT strength (+), hour of low temperature (+), percent passing #200 sieve (-) and service year (+). The sign in the parentheses indicate the trend between predictor variables and responses.

The BL Regression method is further used to develop a probabilistic model for crack initiation with the identified four predictor variables. Arbitrary values were assigned to two groups of pavement responses: a very small number of 0.02 for pavement sections without crack ($y_i=0.02$), and 1.0 for pavement sections with recorded field crack ($y_i=1$). The probability model is shown in Equation (7.4), where $P$ indicates the probability of initiation of transverse crack and therefore $1-P$ means the probability of no crack initiation.

$$
\hat{P}(y_i = 1) = \frac{1}{1 + e^{-(8.42+0.0019x_1-0.284x_4+1.521x_8+0.867x_9)}}
$$

(7.4)

where,
\[ \hat{P} \] = probability of initiation of transverse crack, %

\[ y_i = i^{th} \] pavement section

\[ e \] = base of the natural logarithm approximately equals to 2.718

\[ X_3 = \] hour of low temperature, hour

\[ X_4 = \] percent passing #200 sieve, %

\[ X_7 = \] IDT strength, Mpa

\[ X_8 = \] service life (age), year.

The effect of individual predictor variables are in good agreement with literature findings: (a) it is generally agreed that the more hour of low temperature and longer service life (age) can lead to higher probability of crack initiation; (b) within a specific range, a higher percent passing #200 sieve would result in a more flexible and less crack susceptible mixture (Schwartz et al. 2011). Overlay mixtures with high IDT strength values could be related to high stiffness and high brittleness, and would therefore be more prone to transverse crack.

The standardized coefficients of the four variables are calculated and compared. Results show that service life (age) and IDT strength have the most significant effect on crack initiation, followed by hour of low temperature, while the percent passing #200 sieve has the least effect.

Using the crack initiation model, the probability of cracking for each project is estimated, and the results are shown in Figure 7.9 in dark color. Among the projects “with transverse crack” (Figure 7.9a), 15 out of 22 pavement projects are predicted to have higher than 50% probability for crack initiation. In the “without transverse crack” group (Figure 7.9b), 36 out of 39 pavement projects are predicted to have less than 50% probability for crack initiation. That indicates the
prediction results matched the field condition well. To validate the model, LOOCV method was used. Each sample data was validated exactly once during LOOCV and the results are shown in Figure 6.9 in grid. As can be seen, the cross validated data have almost the same probability range distribution as predicted data, indicating a good prediction power of the model. In other words, it is expected that when the new validation data were introduced, the model should still work well.

![Figure 7.9. Validation of transverse crack initiation model (a) with transverse crack, and (b) without transverse crack.](image)

Figure 7.9. Validation of transverse crack initiation model (a) with transverse crack, and (b) without transverse crack.

Figure 7.10 indicates the validation of crack propagation model by using pavements with existing PCC. It is noted that the 8 sections all have field transverse cracking and no zero cracking section was observed in the field in such case. As shown, 7 out of 8 sections have probability range greater than 50%, indicating a good prediction power of the model and the crack initiation model should work for such case (asphalt concrete placed on existing PCC) as well.
Figure 7.10 Validation of transverse crack propagation model by using pavements with existing PCC.

Sensitivity analysis was conducted for crack initiation model as well and results are shown in Figure 7.11. It appears that for both sections with and without crack, the probability changes of transverse crack are acceptable with the variation (±10%) of IDT strength, hour of low temperature, percent passing #200 sieve, and service life (age).
Figure 7.11 Sensitivity of field transverse crack probability to variable changes (a) IDT strength; (b) hour of low temperature; (c) percent passing #200 Sieve, and (d) service life (age).

In summary, discrepancies could result from some common issues for these mechanistic-based prediction models, for example: (1) typically a performance model only focus on a specific type of cracking mechanism due to the nature of the mechanistic based models, (2) a mechanistic based model usually has limited ability to take into account of the construction/climate variability in the field, and (3) using a single model could have poor prediction quality for field performance as field distress conditions are sophisticated and clear separation of a single damage mechanism is very difficult. Statistical methods are generally used in engineering fields to develop performance prediction models. They also provide different techniques to allow users to identify the effect of variables on responses based on varied scenarios (Henseler et al. 2009), for instance large/small sample size, with/without collinear data and different types of variable. Through meaningful and reasonable data collection, organization, analysis, and interpretation,
statistics provide prediction models that works well for both the selected response and the new introduced response.

Most importantly, the statistical based framework for transverse cracking performance prediction introduced in this chapter can be modified and implemented by local agencies based on specific needs and requirement. For instance, it can also be used for model development of longitudinal cracking and rut depth.
CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

Transverse cracking is one of the most widely seen distress types in asphalt pavement. The mechanism and key factors related to conventional thermal cracking and reflective cracking have been well studied. However, the researches relating to surface-initiated reflective cracking (SIRC) are very limited. SIRC is the cracking type that the transverse cracking initiates at pavement surface and propagates downward to match the existing transverse cracking, and there is no cracking in the interlayer. Such cracking type has been widely observed, but no mechanism was proposed to explain the phenomenon. The factors that could correlate to such crack type are not identified. Researching on the mechanism of the SIRC, the influencing factors, and the predictive models that can take into account of such cracking type is very important since it will:

1. help to categorize different types of transverse cracking with different forming mechanisms;
2. help to the pavement structural design and material design to produce more durable pavement;
3. help to properly select pavement maintenance and rehabilitation strategies based on different type/mechanisms of pavement cracking; and
4. help designer to consider more cost-effective and sustainable design options by shifting the distress to the surface of the pavement (i.e., transfer bottom initiated reflective cracking to surface initiated reflective cracking).

As the first attempt to study the SIRC comprehensively, this research started with the categorization of transverse crack, and then investigated the mechanism of SIRC. Factors that are closely related to this crack type were found based on FEM analysis. Laboratory testing were performed to evaluate the key factors that contribute to initiation and propagation of the SIRC. Because existing predictive models cannot reasonably predict the pavement transverse cracking
including the SIRC, new predictive models were developed to describe the initiation and propagation of field transverse cracking. Although based on relatively limited amount of field projects and laboratory testing, some important findings were concluded as follows:

**Mechanism and Key Factors of SIRC**

1. By comparing distress types observed in the field projects and evaluating their related pavement properties using statistical methods, it was found that the stiffness of asphalt binder and mixture are significantly higher in SIRC sections than in the conventional thermal cracking and no transverse cracking pavement sections. This indicated that stiffness could be a critical factor for such cracking type. The SIRC sections also have significantly higher total HMA pavement thickness and longer service year. Therefore, it is expected that these factors could be critical to initiate SIRC.

2. FEM analysis results indicate that when the HMA surface layer is thick, thermal load associated with traffic load, as well as overlay stiffness and thickness above existing crack/joint are the key factors to initiate SIRC. Both FEM and the laboratory test results indicate that when the pavement thickness is at certain thickness range (i.e., thick enough to protect the existing crack/joint to move but not too thick which can allow the surface layer bending downward with weak base structure), with relative stiff overlay material and weak base, tensile stress can develop at the surface of the pavement layer which can ultimate lead to SIRC under repeated traffic load.
3. With proper improvements, Hamburg Wheel Tracking Device (HWTD) can be effectively used to evaluate the cracking potential of the material/structure under various conditions.

Model Development

1. It is found that the TCMODEL embedded in the AASHTOWare Pavement ME program does not predict thermal cracking properly when comparing with field transverse cracking measurements for both HMA and WMA pavements. Most ME predictions have either minimum or maximum values, which could be partially attributed to the assumptions that are used in the TCMODEL. In addition, Spearman’s rank correlation coefficient indicates that the ranking of ME predicted thermal cracking does not match well with the ranking of field transverse cracking.

2. A statistical based framework for transverse cracking performance prediction is proposed. The framework uses Partial Least Squares (PLS) Regression method, Binary Logistic (BL) Regression method, Leave One out Cross Validation (LOOCV) and Prediction Sum of Squares (PRESS) jointly to determine optimum variable number, coefficients of variables, as well as validation results of prediction models.

3. Hour of low temperature (<15°F), percent passing #200 sieve, IDT strength and service life (age) are critical indicators for the initiation of transverse crack. Service life (age) and IDT strength have the most significant effect on crack initiation among the four identified key factors.

4. Material properties (mixture creep compliance, work density, and percent passing #200 sieve), pavement structure (overlay thickness), climate (hour of low temperature), and
traffic (AADTT) can all have important effect on transverse crack propagation. Creep compliance and hour of low temperature (<15°F) have the most significant effect on crack propagation among the identified six key predictor variables.

5. Both transverse crack initiation and propagation models have good predictability and are well validated.

6. No significant differences were observed between prediction results of HMA and WMA mixtures, and the regression models can be applied for both.

**Recommendations for Future Study**

1. As an exploratory study, this research offers some initial investigation on the SIRC which should not be overlooked in the pavement engineering. However, it is also admitted that the research findings are based on limited experimental testing, FEM modeling, and field investigation. More research should be conducted to validate the proposed mechanisms so that SIRC distress type can be reasonably considered in the pavement structural design, material design, and maintenance practice.

2. Other criteria may need to be used to further evaluate the stress status at pavement surface especially under combined thermal and traffic load. Considering asphalt mixture becomes more brittle at low temperature, failure criteria that can consider such property should be used, such as Griffith’s criterion.

3. More FEM analysis is recommended to include more factors and combinations (i.e., layer thickness, base stiffness, aging effect) in terms of evaluating their effects on SIRC.

4. In terms of laboratory test, the stiffness of base layer needs to be determined to quantitatively evaluate its effect on the SIRC. In addition, more laboratory evaluations
with varied types of asphalt, aggregate, mix design and additives need to be carried out to evaluate their resistance to SIRC. Beam specimens may be used in the future laboratory tests to better simulate field pavement condition.

5. A laboratory test procedure using the modified HWTD should be standardized to evaluate the cracking properties of asphalt mixture. Key factors that determined based on combined traffic and thermal load from FEM analysis.

6. More field transverse cracking data from different climate zones can be collected to validate the transverse cracking initiation and propagation models that proposed in this study.

7. The statistical based framework proposed in this study may be modified and implemented and used for development of other distress models, such as longitudinal cracking and rut depth.

**Main Contributions of This Study**

1. This study attempted to fill the gap of transverse cracking types by systematically analyzing the SIRC for the first time.

2. This study explains the mechanism of SIRC through both FEM simulation and laboratory experiment. The possible mechanism for SIRC is also proposed.

3. This study develops a laboratory wheel tracking testing method to simulate cracking conditions in the field. This laboratory testing method can help to further evaluate the characteristics of SIRC and provide a way to evaluate the material resistance to SIRC.
4. This study provides innovative models for both crack initiation and propagation. The parameters determined in the models can be used to guide mixture and structural design and consequently help to improve maintenance and rehabilitation strategies.
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LIST OF ABBREVIATIONS

AASHTO-The American Association of State Highway and Transportation Officials
AADT-Annual Average Daily Traffic
AADTT-Average Annual Daily Truck Traffic
AC-Asphalt Concrete
ASTM-The American Society for Testing and Materials
BBR-Bending Beam Rheometer
BL-Binary Logistic
COF-Coefficient of Friction
CV-Cross Validation
CZM-Cohesive Zone Method
DOE-Design of Experiment
DSR-Dynamic Shear Rheometer
ESAL-Equivalent Single Axle Load
FEM-Finite Element Method
FWD-Falling Weight Deflectometer
HMA-Hot Mix Asphalt
HWTD-Hamburg Wheel Tracking Device
ICM-Integrated Climatic Model
EICM-Enhanced Integrated Climatic Model
IDT-Indirect Tensile
LOOCV-Leave One Out Cross Validation
LTPP-Long-term Pavement Performance
MEPDG-Mechanistic-Empirical Pavement Design Guide
MLR-Multiple Linear Regression
MMLS3-Model Mobile Load Simulator
MSCR-Multiple Stress Creep Recovery
NCHRP-National Cooperative Highway Research Program
RTFO-Rolling Thin-Film Oven
OAT-One-at-a-Time
PCC-Portland Cement Concrete
PG-Performance Grade
PLS-Partial Least Squares
PRESS-Prediction Sum of Squares
QA/QC-Quality Assurance/Quality Control
SCB-Semi-circular Notched Bending
SHRP-Strategic Highway Research Program
SIF-Stress Intensity Factor
TCMODEL-Thermal Crack Model
TSRST-Thermal Stress Restrained Specimen Test
TTI-Texas A&M Transportation Institute
WMA-Warm Mix Asphalt
VIF-Variance Inflection Factor