INVESTIGATION ON THE MECHANISMS OF BLOCK CRACKING IN ASPHALT PAVEMENTS

BY

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DISSERTATION

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ABSTRACT

Block cracking in asphalt pavements is a primary form of surface cracking but has been the subject of very few scientific investigations. The extensive nature of this cracking form often leads to significant maintenance costs and reduces the ride quality and service life of the pavement surface. Although this deterioration mode is covered in many pavement evaluation guides and condition rating systems, the underlying mechanisms of block cracking have not been fully investigated. Therefore, understanding the mechanisms behind block cracking and tailoring preventive solutions merits rigorous investigation.

In this thesis, a three-dimensional analytical elastic model of a two-layer pavement system subjected to constant thermal stresses, a two-dimensional discrete element viscoelastic and heterogeneous micromechanical model, and a three-dimensional discrete element viscoelastic and inhomogeneous micromechanical pavement model subjected to thermal straining were developed. Analytical solutions of displacement and stress fields are presented in equation and graphical form, and the use of the model as a tool for block crack size prediction was demonstrated. A typical PG 64-22, dense-graded Illinois asphalt surface mixture was adopted as the baseline material in the discrete element model because it typically experiences block cracking later in its service life. The mechanisms of block cracking patterns were investigated as a function of the dimension of pavement segments, relaxation capacity and aging state of materials, including spatial gradients, cooling rate and pre-existing crack presence using the aforementioned discrete element models. Discrete element simulations showed that both rectangular and hexagonal shaped cracking could occur under the same assumption, with initial block cracking primarily occurring in the upper one-to-two centimeters of the surface which agreed with field observations. In addition, it was found
that block cracks formed at warmer temperatures than those associated with the onset of traditional thermal (or transverse) cracking. This implied that current test criteria for thermal cracking mitigation may need to be updated or supplemented in order to control block cracking. Finally, possible candidates for preventive maintenance and tailored maintenance techniques were discussed.
To my dad and mom,
for their love, support, and endless patience throughout the doctoral candidate process.
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CHAPTER 1: INTRODUCTION

1.1 Background

Approximately 94% of roads in the United States are surfaced with asphalt materials, for reasons ranging from material availability and cost, driver comfort and safety, rapid construction, and ease of repair (Huang, 2004). Continual exposure to solar radiation, oxidation, thermal shock, and freeze-thaw cycles leads asphalt pavement surfaces to become brittle and vulnerable to cracking after years of service. In addition, due to exposure differences over time with depth from the surface, the asphalt concrete layers become a vertically-graded viscoelastic material system (Dave, 2009). Critically-aged asphalt pavement plus the occurrence of critical low temperature events and/or numerous temperature cycles potentially leads to thermally-induced, low temperature cracking. The formation of such discontinuities in the pavement undermines the integrity of the pavement, provides additional pathways for water and oxygen infiltration, and accelerates damage with time. If not treated promptly or properly, these cracks coalesce to form block cracks, which then propagate rapidly, cover the entire surface of the pavement, and increase in crack density and width with time, as shown in Figure 1.1 (ARA, 2003). This extensive damage to the asphalt pavement leads to the accrual of significant maintenance costs. In the United States (US), $22.81 billion was expended on routine pavement maintenance in 2015 (Fields, 2018).
However, little attention has been devoted in the asphalt literature to explain the precise mechanisms behind block cracking. In particular, little coverage has been devoted to the topic in classic texts in asphalt materials (Brown, 2009) and pavement materials and design (Huang, 2004). Block cracking was originally included in the new mechanistic-empirical pavement design guide evaluated by using an empirical parameter termed ‘distress potential’ but is not included in the current Pavement M-E software (Huang, 2004; MEPDG, 2008; FHWA report, 2008). Therefore, a rigorous research investigation is warranted to better understand and characterize block cracking through analytical and numerical modeling approaches, which in turn, can lead to better design, construction and preventive maintenance techniques for asphalt pavements to reduce the occurrence and severity of block crack formation.

1.2 Problem Statement

Although block cracking is a very common distress in asphalt pavements, its underlying mechanisms have not been fully investigated. For instance, when a pavement system becomes brittle on the surface, why do thermal cracks form in some instances, block cracks form in other

Figure 1.1 Block cracking in asphalt pavement.
instances, and a combination form in yet other instances? What role does aging gradient play in the relative manifestation of these cracking types? How can a deeper understanding of these underlying mechanisms inform strategies for their mitigation? Considering the extensive use of asphalt as a pavement resurfacing materials combined with the increased usage of quasi-brittle, recycled materials in asphalt in recent years, the study and prevention of block cracking has elevated to a very important research priority in the US and abroad.

1.3 Research Objectives and Outline

To obtain a better understanding of block cracking mechanisms and prevention strategies according to the problem statement described above, the major objectives for the current research are identified as follows:

• Develop a 3D analytical model subjected to uniform thermal straining in order to study displacement and stress fields in asphalt pavements, and to verify the developed 3D discrete element model.

• Develop a 2D, viscoelastic, heterogeneous and anisotropic micromechanical model to study block cracking patterns in a planar asphalt pavement, and vertical propagation in a cross section of asphalt pavement.

• Develop a 3D, viscoelastic, inhomogeneous and anisotropic micromechanical model of common flexible pavement system geometries that can be used to simulate and study block cracking mechanisms in a more representative and comprehensive manner.

• Based on newly found block cracking mechanisms, propose recommendations for the prevention of block cracking.

The research outline developed to meet the study objectives is summarized below.
1.4 Organization of Dissertation

This thesis consists of six chapters. The remaining chapters are organized as follows:

- Chapter 2: Background

This chapter introduces the background information of block cracking phenomena in asphalt pavement, the implemented modeling techniques and the utilized material constitutive models.

- Chapter 3: 3D Analytical model for exploration of the block cracking phenomenon
This chapter introduces a 3D analytical model that led to closed-form elastic solutions of displacement and stress fields, and describes new insights towards the mechanisms of block cracking in a graphical manner. An approach that allows for rough estimation of block cracking size is also demonstrated.

- Chapter 4: 2D micromechanical, viscoelastic and heterogeneous model for the study of block cracking patterns in asphalt pavement

This chapter introduces a two-dimensional viscoelastic, heterogeneous and anisotropic microstructure-based pavement model to study the development and propagation of block cracking patterns, and quantifies the effect of cooling rate, material aging and relaxation and pavement geometry on block crack pattern and depth.

- Chapter 5: 3D micromechanical pavement model development for the study of block cracking

This chapter introduces a three-dimensional (3D) viscoelastic, inhomogeneous and vertically-graded microstructure-based pavement model to study the cracking mechanisms behind the development and propagation of block cracking phenomena (extent and depth). The model was used to examine the effects of aging and cooling rate, followed by a discussion of the practical implications of the results and recommendations for future research and application by practitioners to mitigate block cracking

- Chapter 6: Summary, conclusions and future extensions

This chapter summarizes all the findings of the study, concluding remarks are made based on the findings. Finally, potential future extensions are described.
CHAPTER 2: BACKGROUND

2.1 Block Cracking in Asphalt Pavement

Block cracking is generally considered a non-load associated distress, as it is most prevalent in low traffic volume facilities after years of service due to steric hardening (time-dependent, molecular association in the binder, which is relieved by material straining) and oxidative aging, as displayed in Figure 2.1. In asphalt pavements with medium or high traffic volumes, block cracking may be delayed by straining (steric hardening relief) and/or partially healed by traffic action (vertical stressing leading to lateral dilation and closing/re-adhesion of tensile material separation during high temperature events) and thus less pronounced in the wheel paths. Block cracking in high-traffic volume roads, although less common, has been reported in the literature (Brown et al., 2009), and may be partially caused by material factors such as the stiffer asphalt binder systems used on these facilities. In most cases block cracks divide asphalt pavements into rectangular or hexagonal-shaped segments of different sizes ranging from 0.1 to 10 m² (NCHRP 1-37A, 2004).
Similar to thermal cracking, block cracking is also considered to be a thermally-induced form of distress in asphalt pavements (Brown et al., 2009). As pavement temperature decreases, the asphalt pavement layer tends to contract; however, it is restrained from doing so due to bonding and/or frictional contact with the underlying layer. As such, thermal, contractive tensile stresses accumulate in the asphalt pavement layer as the pavement temperature decreases. Due to the viscoelastic nature of asphalt, significant tensile stresses occur only when the rate of temperature decline ‘outpaces’ the material’s inherent ability to relax stress, i.e., during rapid or critical cooling events. These can be caused by rapidly declining asphalt temperatures resulting from sudden air temperature changes, high winds, dark skies, and/or cool rains. Thermally-induced microcracks will begin to develop when thermal stresses exceed the local tensile strength of the asphalt material.
(Anderson, 2005, Fromm et al., 1972, Haas et al., 1987). Depending also on factors such as binder grade, mixture fracture resistance, aging conditions of the pavement, and the thickness of pavement layers, i.e., the boundary conditions, microcracks will either intersect with each other to create block cracks, or propagate through the asphalt layer to become transverse thermal cracking. The combination of both block and thermal cracking may also occur. Other factors, such as low density resulting from poor roller compaction, can accelerate block cracking (Anderson et al., 2011, Brown et al., 2009). This is perhaps why block cracking is sometimes observed on heavy duty pavements – stiffer binders and angular aggregates are used to prevent rutting, but are therefore difficult to compact. In cases where poor density was achieved (high air voids), more rapid oxidative hardening can occur. Although not investigated herein, weakening of the asphalt pavement through moisture damage caused by the porous surface may also be a factor in crack formation in these facilities.

Block cracking is rated as low, medium and high severity levels based on the crack width and area thresholds listed in Table 2.1 (FHWA, 2003). Figure 2.2 plots the block cracking areas of the LTPP pavement sections with cement treated bases as a function of pavement age, as obtained from the LTPP database (ARA, 2003). Block cracks start to initiate after very short or very long service life, e.g., over a range of 2~20 years of service. Once they initiate, block cracks tend to spread over the entire pavement surface exponentially within approximately ten years (ARA, 2003).
Table 2.1 Block cracking severity levels defined by LTPP

<table>
<thead>
<tr>
<th>Severity</th>
<th>Description</th>
</tr>
</thead>
</table>
| Low      | - Cracks with a mean width $\leq$ 6mm  
           - Sealed cracks with sealant materials in good condition and with a width cannot be determined |
| Medium   | - 6mm $\leq$ Mean cracks width $\leq$ 19mm  
           - Any cracks with a mean width $\leq$ 19mm and adjacent low severity random cracking |
| High     | - Cracks with a mean width $>$ 19mm  
           - Any crack with a mean width $\leq$ 19mm and adjacent moderate to high severity random cracking |

Note: An occurrence should be at least 15m long before rating as block cracking.

Figure 2.2 Block Cracking as a Function of Time Expressed as a Percentage of Total Area for Pavements with Cement Treated Base (ARA, 2003).

2.2 Block Crack Patterns in Other Materials

The formation of a repeating block cracking pattern is not unique in asphalt pavements. For instance, it has been widely reported to occur in other materials, as depicted in Figure 2.3. For
example, basalt columns can be found in the ‘Giant’s Causeway’, which is famous for the large formation of interlocking basalt columns. Formed by a volcanic eruption, the resulting columns are hexagonal in shape with very straight edges. The geometry of the block cracking pattern is similar at a given site, with block size (average diameter) evidently determined by the cooling rate of the lava from the volcanic eruption. Mahadevan recreated the size of the columns by pouring water on starch, which cracked as water dried out, which led to the establishment of a relationship between the size of the columns and the speed with which the solidification front moved (University of Toronto, 2008). Inspired by this experiment, Goehring carefully controlled all other factors, and found that the slower the cooling process, the larger the resulting columns (Goehring and Morris, 2008). Interestingly, both basalt columns and starch columns not only exist at the surface, but also extend downwards to the bottom. Rectangular and hexagonal crack patterns are also commonly found in soils. In order to study these crack patterns in soil, Goehring wetted and dried soil samples repeatedly in the laboratory, and found that rectilinear cracks formed first, later evolving into hexagonal cracks after additional cycles of drying and wetting (Goehring, 2013).
Figure 2.3 Hexagonal crack patterns found in: (a) Giant’s Causeway, (b-c) starch, (d-f) soil (Goehring, 2013).

Similar crack patterns are found in asphalt roofing membranes as well. The alligator cracking shown in Figure 2.4a developed due to ultraviolet radiation, and extreme high temperatures in the dark surface leading to volatilization of light ends in the binder. The consequent embrittlement of the membranes combined with daily temperature cycles probably led to the cracking pattern observed (Druert, 2013). The alligator cracks are in different sizes, and some of the edges are curved, which is different from what was observed in basalt columns and mudcracks. This implies that the viscoelasticity of asphalt and the slow UV aging rate might curve the edge of cracks. The relatively thin dimension in the z-direction and interface bonding behavior may also play a role. In an asphalt sample of a relatively small size, spiral cracking has been observed to occur under rapid cooling, as presented in Figure 2.4b (Behnia, 2018).
Figure 2.4 Crack pattern in asphalt materials: (a) alligator cracking in a roofing membrane (Druert, 2013) and (b) spiral cracking in an asphalt sample (Behnia, 2018).

2.3 Discrete Element Modeling

Discrete element modeling (DEM) is a numerical method that is mainly utilized to study the mechanical behavior of engineering materials or structures made of granular or discontinuous materials, since granular particles interact with the adjacent particles only at contact points. Cundall (1971) developed the so-called ‘distinct’ element method, which was capable of modeling different shapes of particles to study rock mechanics and geotechnical problems. Cundall and Strack (1979) verified the discrete element model as a valid tool to simulate the behavior of granular assemblies by comparing the results of photoelastic disc tests with the corresponding numerical results used disc assemblies. During this period, Cundall developed a two-dimensional discrete element model named BALL, which was apparently the prototype of PFC 2D and PFC 3D software, later introduced in 1994. The fundamental particle shapes used in PFC 2D and 3D are discs with unit thickness for PFC 2D and spheres for PFC 3D. Non-spherical morphological features such as aggregates can be approximated by grouping or bonding sets of round DEM particles together (Figure 2.5).
Figure 2.5 Elements in PFC software: (a) basic element: ball; (b) cluster made of multiple balls (Itasca, 2004).

In the DEM system, particle-to-particle interactions are calculated through internal force and moment based on particle-interaction laws, and bonding between particles is calculated according to the assigned contact model (Figure 2.6). PFC2D and 3D do not take the deformation inside each block into account, which are different from the assumptions of some other discrete element models. For example, Ghaboussi (1988) proposed a method where each block was modeled as a single four-node element using the finite element method, and the contact stresses between each block was computed using the discrete element method. Based on this work, large deformation analysis on multiple deformable bodies was accomplished by Barbosa and Ghaboussi (1990), which was an effective method to study the behavior of a system of continuous bodies and discontinuous media. As such, they developed BLOCKS2D and BLOCKS3D, which were two-dimensional and three-dimensional discrete element models for granular materials, which were capable of modeling granular materials over a wide range of conditions, from static and small displacement situations to rapid large movement situations. Nezami et al. (2006) modified BLOCKS3D by introducing a shortest link method, which helped the program to run faster and reduced the computational cost significantly. Huang (2010) improved BLOCKS3D by introducing the application of imaging-based aggregate morphology, which allowed the program to conveniently generate a realistic assembly of ballast aggregates. Qian et al. (2011) demonstrated the effectiveness of BLOCKS3D for modeling ballast reinforced with geogrids.
Asphalt material is essentially a viscoelastic and heterogeneous material, which can be realistically and practically simulated by either the generalized finite element method or discrete element method (Buttlar et al., 2001; Jorge, 2013). However, in terms of micromechanical mechanism analysis of asphalt pavement, the discrete element method has several advantages over the finite element method. Discrete element modeling can account for the grain-level effect on fracture behavior as well as providing continuum analysis throughout the system, whereas the finite element model cannot. Discrete element modeling also allows larger displacements between particles, which makes it easier to conduct fracture analysis of asphalt material as compared to the finite element method. There are numerous numerical studies that have been accomplished on modulus prediction and fracture analysis in pavements using discrete element models. In early studies, asphalt mixture was modeled as a two-phase material composed of asphalt mastic and aggregates to represent its heterogeneous nature. Buttler and You (2001) used PFC2D to simulate the microstructure of asphalt mixture, as shown in Figure 2.7. They demonstrated that PFC 2D software was a promising tool for asphalt concrete microstructure analysis by comparing the numerical results to indirect tension test results. In addition, You and Buttler (2005) used PFC 2D to predict complex modulus in asphalt by simulating the hollow cylinder tensile test, and found that the numerical results were in good agreement with experimental results. You et al. (2008)
extended his study to three-dimensional distinct element method using PFC 3D by stitching a series of two-dimensional images of actual sliced asphalt specimens together, and found that the resulting 3D model provided better dynamic modulus prediction than the previous 2D model (Figure 2.8a). Liu and You (2009) started to simulate asphalt mixtures as a three-phase material, i.e. asphalt mastic, aggregates and air voids (Figure 2.8b). They used PFC3D to generate the microstructure of asphalt mixture randomly to simulate and visualize microstructure and to capture the micromechanical behavior of asphalt mixtures. However, they found that the distribution of aggregate and air voids in this model did not correspond closely with laboratory or field mixes; however, the volumetric properties were reasonably accurate. You et al. (2010) improved the PFC 3D model with randomly generated air voids, and studied air void effects on complex modulus prediction in asphalt mixtures. The third approach is provided by Itasca Consulting Group, which is to generate the model with the use of multi-sized balls. The whole range of aggregates in varied sizes is able to be generated in the model, which provides an accurate skeleton structure of asphalt mixture (Figure 2.8c). However, this generation process takes tremendous computer memory, which makes it only suitable for making a relatively small-size model.

Figure 2.7 Microfabric DEM model of stone matrix mixture specimen using two-phase material by Buttlar and You (2001).
There are several different packing arrangements that can be used to form a DEM mesh, i.e., face-centered, random and hexagonal packing arrangements, as shown in the Figure 2.9. Each has advantages and disadvantages. For example, the face-centered packing arrangement is a very simple method to discretize a material, the material response simulated using this arrangement may be inadequate due to its inability to provide Poison’s effect in the material (Mustoe and Griffiths 1998). A random packing arrangement has an advantage over the face-centered arrangement in terms of simulating more realistic dilatational material behavior, such as Poison’s effect and more random crack paths. However, it might be problematic on assigning material properties to particles of different radius, as particle radius is an important factor when calculating
material contact strength in the discrete element modeling approach. A hexagonal packing arrangement has nearly the same benefits as the random packing arrangement. However, it is more straightforward in terms of assigning identical material properties for each contact of a given type (bulk material, interface).

![Diagram of packing arrangements](image)

**Figure 2.9** Common packing arrangements utilized in discrete element modeling (Kim *et al.*, 2008).

It was also found that both horizontal and vertical hexagonal packing arrangements were sufficient to capture realistic viscoelastic bulk material behavior and fracture behavior in the simulation of asphalt concrete test specimens (Kim *et al.*, 2008). Therefore, a horizontal hexagonal patching arrangement has been selected for the simulations to be performed in this study. Particle size also has a potential influence on simulation results, particularly when a large particle size is used. Kim *et al.* (2008) illustrated that global fracture response became independent of size for particles of diameter 0.6mm and smaller.

Ren *et al.* (2016) developed a more representative asphalt concrete DEM model, which employed elastic contact model for aggregates contact using irregular particles, and generalized Maxwell viscoelastic contact model for asphalt mastic contact. Hill (2016) developed viscoelastic cohesive contact model via PFC 2D, a combination of viscoelastic Burger’s model and linear softening model, which took viscoelastic relaxation into account. He verified the model by simulating Mode-I fracture using SE(B) and DC(T) geometries and found the model performed well to identify local fracture properties of asphalt mixture. Li *et al.* (2016) developed a thermal...
model to simulate thermal cracking of concrete using PFC 2D, which allowed the thermal induced stress developed among the particles by changing the radius of each particle. In the study of Portland cement concrete, Bolander and Berton (2004) simulated shrinkage-induced cracking in cement composite overlays to study stress distribution and potential shrinkage crack development of cement-based composites during drying process. Random lattice coupled with rigid-body-spring networks were used to consider elasticity, creep and fracture properties of the material. The results indicated that the properties of the interfacial zone are very important in determining the ultimate fracture mechanism of the pavement system: either debonding between the overlay and substrate layer or fracture through the thickness of the overlay, as illustrated in Figure 2.10.

![Figure 2.10](image.png)

**Figure 2.10** 1) Debonding along a weak interface: (a) 20days and (b) 50days exposure to drying environment; 2) Crack patterns using a strong/tough interface: (a) 10days and (b) 110days after drying process (Bolander and Berton, 2004).

### 2.4 Contact models utilized in the PFC software

To simulate the viscoelastic and fracture behavior of the asphalt mixture at low temperature, two contact models were utilized: viscoelastic cohesive-softening contact model and thermal contact model.
2.4.1 Viscoelastic Cohesive-Softening Contact Model

To simulate the viscoelastic and fracture behavior of the asphalt mixture in DEM, a viscoelastic cohesive-softening contact model was utilized. The material contact model used was developed and verified by Hill et al. (2016). This model combined a viscoelastic Burger’s model with a linear softening model to describe both viscoelastic (bulk) and local separation behavior of asphalt material as shown in Figure 2.11. The pre-peak portion of the viscoelastic cohesive contact model used Burger’s model as described in Equation (2-1). The Burger’s model was also used in the study by Kim et al. (2009a). After the peak force was reached, the post-peak portion followed a linear displacement softening model as described in Equation (2-2), which first introduced in PFC2D [22],

\[
f_k^{(t+1)} = \pm \frac{1}{C} \left[ u_k^{t+1} - u_k^t + \left( 1 - \frac{B}{A} \right) u_k^t k - \pm f_k^t \right]
\]

\[
A = 1 + \frac{\kappa_k \Delta t}{2C_k}, \quad B = 1 - \frac{\kappa_k \Delta t}{2C_k}, \quad C = \frac{\Delta t}{2C_k A} + \frac{1}{K_m} + \frac{\Delta t}{2C_m}, \quad D = \frac{\Delta t}{2C_k A} - \frac{1}{K_m} + \frac{\Delta t}{2C_m}
\]  

where \( \Delta t, \kappa_k, K_m, C_k, C_m \) represent the time increment, stiffness in the Kelvin and Maxwell units, and the viscosities in the Kelvin and Maxwell units, respectively.

\[
p_k^k \left( \frac{U_p}{U_{p,lim}} \right) = F_{max} \left( 1 - \frac{U_p}{U_{p,lim}} \right) \quad (2-2)
\]

where \( F_k, F_{max} \) and \( U_{p,lim} \) represent the maximum allowable force, accumulated plastic displacement and maximum allowable plastic displacement in the contact respectively.
2.4.2 Thermal Contact Model

A built-in thermal feature in PFC software was adopted to simulate temperature change in DEM. The thermal contact model was developed by Li et al. (2016) to simulate heat conduction. This model allowed thermally-induced stresses to develop among particles in the pavement model by changing the radius of each particle as demonstrated in Equation (2-3). Each particle was considered as a reservoir, and heat flow was accomplished via the imagined heat pipes which connect adjacent reservoirs at the contact points (Figure 2.11). By default, the active heat pipe was only active if two particles overlapped or if a bond existed and each reservoir was temperature-independent. Radiative and convective heat transfer were not considered in the currently provided thermal feature. Thus, the thermal contact model was defined as:

\[ \Delta R = \alpha R \Delta T \]  

(2-3)

where \( \Delta R \) was the change of each particle radius, \( \alpha \) was coefficient of linear thermal expansion, and \( \Delta T \) was temperature change. The change of displacement of the bond was expressed as:

\[ \Delta U^n = \bar{\alpha} L \Delta T \]  

(2-4)

where \( \bar{\alpha} \) was coefficient of thermal expansion of bond material, and \( L \) was length of the
bond which equated the diameter of element in the presented model.

2.5 ILLI-TC Modeling

ILLI-TC is a mechanistic-empirical thermal cracking model developed as part of the low-temperature cracking Pooled Fund study #776 (Marasteanu et al., 2007), described further in Dave et al. (2013). ILLI-TC uses a viscoelastic finite element modeling framework with a built-in 2D, cohesive zone fracture modeling computational engine. In this model, mixture creep compliance, coefficient of thermal contraction, tensile strength and fracture energy are used as material inputs to the model, along with climatic and layer thickness information. In this study, warm and cold temperature conditions for the state of Illinois were selected for comparison, which were considered the closest climatic locations available in the software to the thermal conditions in the DEM simulations. Since the thermal cracking software is not sensitive to pavement structure and the sections had variable layering configurations, a default structure was used. This involved a 3-inch asphalt surface placed on existing pavement, which was selected in order to focus the evaluation on the relative thermal cracking performance of the asphalt overlays.

ILLI-TC was used to estimate the occurrence of any critical thermal cracking events over the design life, along with more detailed information regarding the extent of pavement thickness damaged and cracked based on a 5-year analysis period. The climate data available in the version of the software from the Pooled Fund study dated from 2000 to 2005, thereby producing qualitative crack predictions over the first 5 years of in-service conditions. The critical cracking events are pre-evaluated by the software to determine simulated days where the tensile stress of the surface layer exceeds 80% of the tensile strength of the asphalt mix. In this thesis, mixture tensile strength was estimated from the peak load in the DC(T) test based on a calibrated, empirical equation (Marasteanu et al., 2012). Whenever the 80% threshold is reached, the program then backs up a
couple of days in time to allow viscoelastic history effects to pass the transient state and initialize into an accurate steady state condition before the critical event is simulated. In general, mixtures at the short-term aged level are to be used in the model, as the model was calibrated to take into account the fact that most designers will only have short-term aged sample test results. However, properties of long-term aged material were used to predict the thermal cracking potential of the aged pavement, with the rationale that slightly conservative cracking predictions would be obtained.
CHAPTER 3: 3D ANALYTICAL MODEL FOR EXPLORATION OF THE BLOCK CRACKING PHENOMENON

3.1 Introduction

A review of the literature in asphalt pavements shows that very little research has been conducted in the area of block crack analytical modeling. However, there have been numerous studies directed towards thermal crack modeling in asphalt pavements and Portland cement concrete pavements (Marasteanu, et al., 2007, Velasquez et al., 2013, Shen et al., 2017), which are helpful in the study of block cracking. Wang and Roesler (2012) developed an analytical solution that can be used for predicting the 1D temperature profile in a multi-layered rigid pavement system using Laplace transformation. Chen et al. (2004) developed a one-dimensional (1D) model to investigate the minimum and maximum crack spacing caused by drying shrinkage in concrete pavements. Shen et al. (1999) developed a 1D semi-analytical numerical model for asphalt concrete with inhomogeneous material to simulate distributed thermal cracking in asphalt concrete with shear-spring-type frictional constraint at the interface below the asphalt surface. The simulation results showed that, with small mesoscale inhomogeneities and the presence of a constraining frictional force, micro cracks or mesoscale-localized damage will eventually lead to crack opening in the macroscale. Yin et al. (2007) introduced a two-dimensional elastic model of asphalt overlay/rigid base system, and discussed the displacement field and stress field of the system subjected to constant thermal stress. By comparing the energy release rate and fracture toughness of the surface layer, crack depth and crack initiation for a given temperature can be predicted by this model. The aforementioned models are capable of predicting one dimensional or two dimensional (2D) thermal cracking. However, unlike transverse cracking, they are unable to directly study block cracking in asphalt pavements, which is truly a three-dimensional (3D) phenomenon (cracks channel in all three dimensions).
In this Chapter, a 3D elastic analytical model of a two-layered pavement system was developed to study block cracking. Both the key model framework, assumptions and resulting solutions and detailed intermediate derivation steps are presented. Solutions for displacement and stress fields for a pavement system subjected to uniform thermal stresses are provided, along with illustrative plots. Details regarding how the analytical solution can be used to verify numerical tools such as finite element simulations, is discussed. Illustrative examples are then developed with the model and discussed to provide new insights into the mechanism of block cracking. Preliminary field validation of the analytical model is then performed, by comparing predicted to actual block cracking on a state road located in northern Missouri.

3.2 Physical Problem and Formulation

A two-layer pavement system has been adopted in this study to assess stress distributions resulting from temperature changes in an asphalt pavement for the rectangular-shaped block cracking phenomenon. Consider an asphalt layer resting on top of a rigid substrate and subjected to constant thermal stress, i.e., that occurring from a temperature drop (Figure 3.1a). Thermally-induced stresses in the pavement layer develop due to differential thermal contraction coefficients between asphalt concrete and that of the underlying layer. Due to its high thermal contraction coefficient, the asphalt layer will contract more than the substrate as the temperature decreases, leading to tensile stress development. In the development of this initial baseline solution, the temperature profile through the pavement thickness was assumed to be constant. This would most accurately portray a pavement with a relatively thin asphalt surface layer. The asphalt layer was also assumed to be fully bonded to the underlying substrate prior to and after thermal loading. Block cracking may be prevalent at very low temperatures, when asphalt concrete is brittle and viscoelastic effects are less pronounced. To develop baseline, closed-form analytical solutions for
block cracking, the behavior of asphalt concrete and substrate materials are considered to be linear elastic, homogeneous, and isotropic at low temperature condition. More discussion regarding the uses and limitations resulting from this assumption is provided in a following section.

The model shown in Figure 3.1a depicts block cracks subdividing the pavement surface into smaller, rectangular-shaped fragments with periodic spacing as a result of temperature cycling. A typical rectangular-shaped fragment of asphalt pavement is schematically shown in Figure 3.1b, which is continuous in the base layer but with cracked surface layer due to block cracking. This physical representation is the basis of the analytical solutions developed herein. The use of a pre-cracked structure does not limit the ability of the solution to predict the onset of initial cracking in a practical sense, as the planar dimensions are variables in the model and large values can be used to represent the pavement at the onset of block crack development.

The model in Figure 3.1b can be decomposed and represented by the models shown in Figure 3.1c and Figure 3.1d. For instance, ‘Model Figure 3.1c’ depicts a pavement structure subjected to thermal stresses in the both x-direction(\(\sigma_{xx}^0\)) and y-direction(\(\sigma_{yy}^0\)) throughout the thickness at boundaries, which is a relatively simple 3D elastic problem to solve analytically. ‘Model Figure 3.1d’ represents a pavement structure where only the surface layer is subjected to the same value of external thermal stress in both the x-direction(-\(\sigma_{xx}^0\)) and y-direction(-\(\sigma_{yy}^0\)) but in the opposite direction at the boundaries, where there is no external thermal stress normal to the vertical edge of the substrate layer. In this way, the solutions for the baseline geometry presented in Figure 3.1b can be obtained through the superposition of solutions developed for the models shown in Figure 3.1c and Figure 3.1d, which was undertaken to simplify the solution of the governing equations. In addition, the fully bonded interface of ‘Model Figure 3.1d’ can be simulated as a frictional interface assuming the substrate layer is a Winkler-type foundation, which
is achieved by introducing linear springs as the lower boundary condition. Finally, Figure 3.1d represents the reduced problem to be solved, i.e., a new 3D baseline analytical model for the study of block cracking. Thus, the new model considers a rectangular-shaped asphalt overlay that is $2\alpha$ in length, $2\beta$ in width and $h$ in thickness, and subjected to thermal stresses in both the $x$-direction($\sigma_{xx}^0$) and $y$-direction($\sigma_{yy}^0$) throughout the thickness. The material of the asphalt overlay is assumed to be elastic with an elastic modulus of $E$ and Poisson’s ratio of $\mu$. Since only thermal stresses are considered in this problem, there is no in-plane shear strain in the $xy$-plane (equation (3-1)):

\[ u_{x,y} = 0 \quad (3-1) \]

![Figure 3.1. Schematic model of a two-layered pavement system subjected to constant thermal stress.](image)

To solve the proposed problem, the following field equations were used:

Constitutive law:

\[ \sigma_{xx} = \frac{E}{(1 + v)(1 - 2v)} \left[ (1 + v)u_{x,x} + vu_{y,y} + vu_{z,z} \right] \quad (3-2) \]

\[ \sigma_{yy} = \frac{E}{(1 + v)(1 - 2v)} \left[ vu_{x,x} + (1 + v)u_{y,y} + vu_{z,z} \right] \quad (3-3) \]

\[ \sigma_{yy} = \frac{E}{(1 + v)(1 - 2v)} \left[ vu_{x,x} + vu_{y,y} + (1 - v)u_{z,z} \right] \quad (3-4) \]
\[ \sigma_{xz} = \frac{E}{2(1 + v)} u_{x,z} \]  
\[ \sigma_{yz} = \frac{E}{2(1 + v)} u_{y,z} \]  

Equilibrium equations:

\[ \sigma_{xx,x} + \sigma_{zx,z} = 0 \]  
\[ \sigma_{yy,y} + \sigma_{zy,z} = 0 \]  
\[ \sigma_{xz,x} + \sigma_{yz,y} + \sigma_{zz,z} = 0 \]

The first step is to obtain general equations of \( u_x, u_y \). Inserting kinematic equations into equilibrium equations (3-7) and (3-8), we have,

\[ \frac{1 - v}{1 - 2v} u_{x,xx} + \frac{1}{2} u_{x,zz} = 0 \]  
\[ \frac{1 - v}{1 - 2v} u_{y,yy} + \frac{1}{2} u_{x,zz} = 0 \]

Both equations (3-10) and (3-11) are second differential equations of two variables \( u_x \) and \( u_y \) and accordingly, general solutions of \( u_x, u_y \) can be obtained through separation of variables, as follows:

\[ u_x(x,z) = \left( C_1 e^{\frac{a}{h}x} + C_2 e^{-\frac{a}{h}x} \right) \left( C_3 \sin \frac{b}{h}z + C_4 \cos \frac{b}{h}z \right) \quad (b = a \sqrt{\frac{2(1 - v)}{1 - 2v}}) \]  
\[ u_y(y,z) = \left( C_5 e^{\frac{c}{h}x} + C_6 e^{-\frac{c}{h}x} \right) \left( C_7 \sin \frac{d}{h}z + C_8 \cos \frac{d}{h}z \right) \quad (d = c \sqrt{\frac{2(1 - v)}{1 - 2v}}) \]

Where \( C_1, C_2, C_3, C_4, C_5, C_6, C_7, C_8, a, b, c, d \) are unknown variables to be solved.

In order to simplify equations (3-12) and (3-13), several boundary conditions are utilized. Since the displacement of the layer is symmetric on the \( z \)-axis due to symmetry, the variables of equations (3-12) and (3-13) can be further simplified. Because \( u_x(x,z) = u_x(-x,z) \) and \( u_y(y,z) = u_y(-y,z) \), thus \( C_1 = -C_2 \) and \( C_5 = -C_6 \).
In addition, shear stress is zero on the free upper surface,

\[ u_{x,z}(x, h) = 0, u_{y,z}(y, h) = 0 \] (3-14)

In this way, equations (3-12 and (3-13 can be simplified as follows,

\[ u_x(x, z) = C_1 \left( e^{a \pi x} - e^{-a \pi x} \right) \cdot C_3 \cos \frac{b}{h} (h - z) \quad (b = a \sqrt{2(1 - v) \over 1 - 2v}) \] (3-15)

\[ u_y(y, z) = C_5 \left( e^{c \pi y} - e^{-c \pi y} \right) \cdot C_8 \cos \frac{d}{h} (h - z) \quad (d = c \sqrt{2(1 - v) \over 1 - 2v}) \] (3-16)

In summary, displacement \( u_x, u_y \) are expressed as:

\[ u_x(x, z) = A_1 \sinh \left( a \frac{x}{h} \right) \left[ \cos \frac{b}{h} (h - z) \right] \quad (b = a \sqrt{2(1 - v) \over 1 - 2v}) \] (3-17)

\[ u_y(y, z) = A_2 \sinh \left( c \frac{y}{h} \right) \left[ \cos \frac{d}{h} (h - z) \right] \quad (d = c \sqrt{2(1 - v) \over 1 - 2v}) \] (3-18)

Where, \( A_1, A_2, a, b, c, d \) are still unknown.

In order to solve unknown parameters \( b \) and \( d \), a frictional interface is used to create responses equivalent to those that would be present in a fully-bonded interface, by specifying that displacements change proportionally with shear stress \( \sigma_{zx}, \sigma_{zy} \) along the interface (Xia and Hutchinson 2000; Timm et al., 2003), i.e.,

\[ \sigma_{zx}(x, y, 0) = k u_x(x, z) \] (3-19)

\[ \sigma_{zy}(x, y, 0) = k u_y(y, z) \] (3-20)

where \( k \) is a spring constant used to estimate the effects of the vertical stiffness of the underlying layers.

Inserting stress-strain relations and equations (3-17/(3-18 into equations (3-19/(3-20, we obtain,
\[
b \tan b = \frac{2(1 + v)kh}{E} \quad b \in [0, \frac{\pi}{2}]
\]
\[
d \tan d = \frac{2(1 + v)kh}{E} \quad d \in [0, \frac{\pi}{2}]
\]

As indicated from equations (3-21)/(3-22), b and d are material parameters. In this case, b and d are the same for a specific material, namely, c = a and d = b for a particular material in all cases.

Due to the zero in-plane shear stress assumption made for this model, there is no expression for \(u_{z,z}\) that can satisfy the boundary condition everywhere. Therefore, a weak form boundary condition is used to obtain the general solution of \(u_{z,z}\).

Inserting equation (3-4) into equation (3-9 and simplifying, we get:

\[
\frac{E}{(1 + v)(1 - 2v)} \left[ \frac{1}{2} u_{x,xz} + \frac{1}{2} u_{y,yz} + (1 - v)u_{z,zz} \right] = 0
\]

As shown in equation (3-23), the derivative of all parameters with z is zero. Integrating equation (3-23) on z, the right-hand side should be a function of all variables other than z.

\[
\frac{E}{(1 + v)(1 - 2v)} \left[ \frac{1}{2} u_{x,x} + \frac{1}{2} u_{y,y} + (1 - v)u_{z,z} \right] = F(x, y)
\]

Then, equation (3-24) can be rewritten as,

\[
\sigma_{zz} + \frac{E}{2(1 + v)}(u_{x,x} + u_{y,y}) = F(x, y)
\]

Integrating both sides of equation (3-25 w.r.t. x, y and z, accordingly, and using the total stress to substitute the resultant force of \(F(x, y)\). We have,

\[
\int_{z=0}^{h} \int_{y=-\beta}^{\beta} \int_{x=-\alpha}^{\alpha} \left[ \sigma_{zz} + \frac{E}{2(1 + v)}(u_{x,x} + u_{y,y}) \right] = F \cdot 2\alpha \cdot 2\beta \cdot h
\]

Because there is no external loading in the z-direction, the integral of stress \(\sigma_{zz}\) in the fractured layer is zero. Therefore,
\[\int_{x=0}^{h} \int_{y=-\beta}^{\beta} \int_{x=-\alpha}^{\alpha} \left[ \frac{E}{2(1+v)} (u_{xx} + u_{yy}) \right] = \bar{F} \cdot 2\alpha \cdot 2\beta \cdot h \]  

(3-27)

By inserting the derivative of equations (3-17 and 3-18 and then integrating the left side, we obtain,

\[\bar{F} = \frac{E \sin b}{2b(1+v)} \left[ \frac{A_1}{\alpha} \sinh \left( \frac{\alpha}{h} \right) + \frac{A_2}{\beta} \sinh \left( \frac{\beta}{h} \right) \right] \]  

(3-28)

Inserting equation (3-28 back to equation (3-24, we have,

\[u_{zz} = \frac{(1 - 2v) \sin b}{2b(1-v)} \left[ \frac{A_1}{\alpha} \sinh \left( \frac{\alpha}{h} \right) + \frac{A_2}{\beta} \sinh \left( \frac{\beta}{h} \right) \right] - \frac{1}{2(1-v)} u_{xx} \]  

(3-29)

Inserting equation (3-29 into equations (3-2 and (3-3, we have

\[\sigma_{xx} = \frac{E(2-v)}{2(1+v)(1-v)} u_{xx} + \frac{Ev}{2(1+v)(1-v)} u_{yy} + \frac{Ev \sin b}{2b(1+v)(1-v)} \left[ \frac{A_1}{\alpha} \sinh \left( \frac{\alpha}{h} \right) + \frac{A_2}{\beta} \sinh \left( \frac{\beta}{h} \right) \right] \]  

(3-30)

\[\sigma_{yy} = \frac{E(2-v)}{2(1+v)(1-v)} u_{xx} + \frac{E(2-v)}{2(1+v)(1-v)} u_{yy} + \frac{Ev \sin b}{2b(1+v)(1-v)} \left[ \frac{A_1}{\alpha} \sinh \left( \frac{\alpha}{h} \right) + \frac{A_2}{\beta} \sinh \left( \frac{\beta}{h} \right) \right] \]  

(3-31)

In this way, all the expressions of stress and displacement field have been developed, and only the expressions of \(A_1\) and \(A_2\) need to be solved. The assumption that the total stress of problem d equals that of problem c is another boundary condition that can be used to solve for \(A_1\) and \(A_2\).

For problem c, at the end of the asphalt overlay, the normal stress satisfies,

\[\sigma_{xx}^0(\alpha, z) = \frac{E}{(1+v)(1-2v)} \left[ (1-v)\varepsilon_{xx}^0 + v\varepsilon_{yy}^0 \right] \]  

(3-32)

\[\sigma_{yy}^0(\beta, z) = \frac{E}{(1+v)(1-2v)} \left[ v\varepsilon_{xx}^0 + (1-v)\varepsilon_{yy}^0 \right] \]  

(3-33)

The resultant normal force of problem d is equivalent to the total stress of problem c, by using the weak form stress boundary condition we have:
\[
\int_{y=-\beta}^{\beta} \int_{z=0}^{h} \sigma_{xx}(\alpha, y, z) \, dy \, dz = -\sigma_{xx}^0 (\alpha, z) \cdot h \cdot 2\beta \\
\int_{x=-\alpha}^{\alpha} \int_{z=0}^{h} \sigma_{yy}(x, \beta, z) \, dx \, dz = -\sigma_{yy}^0 (\beta, z) \cdot h \cdot 2\alpha
\]}

(3-34)

(3-35)

In this way, \(A_1\) and \(A_2\) can be expressed as:

\[
A_1 = \frac{-G(\beta)[v \varepsilon_{xx}^0 + (1 - v) \varepsilon_{yy}^0] + H[(1 - v) \varepsilon_{xx}^0 + v \varepsilon_{yy}^0]}{K} \\
A_2 = \frac{-G(\alpha)[(1 - v) \varepsilon_{xx}^0 + v \varepsilon_{yy}^0] + J[v \varepsilon_{xx}^0 + (1 - v) \varepsilon_{yy}^0]}{K}
\]}

(3-36)

(3-37)

Where,

\[
G(x) = \frac{4h^2 b \nu (1 - v)}{\beta \sin(b (1 - 2v))} \sinh(a \frac{x}{h})
\]

\[
H = \frac{2hb (1 - v)}{\sin(b (1 - 2v))} \left[\cosh\left(a \frac{\beta}{h}\right) + \frac{hv}{\beta} \sinh(a \frac{\beta}{h})\right]
\]

\[
J = \frac{2hb (1 - v)}{\sin(b (1 - 2v))} \left[a(2 - v) \cosh\left(a \frac{\alpha}{h}\right) + \frac{hv}{\alpha} \sinh(a \frac{\alpha}{h})\right]
\]

\[
K = \frac{4h^2 v^2}{\alpha \beta} \sinh\left(a \frac{\beta}{h}\right) \sinh\left(a \frac{\alpha}{h}\right) - [a(2 - v) \cosh\left(a \frac{\alpha}{h}\right) \frac{hv}{\alpha} \sinh(a \frac{\alpha}{h})]
\]

Since the total stress of the aforementioned problem b is the resultant force of problem c and d, solutions of problem b are provided as follows,

\[
\sigma_{xx}^T = \sigma_{xx}^0 + \sigma_{xx}
\]

(3-38)

\[
\sigma_{yy}^T = \sigma_{yy}^0 + \sigma_{yy}
\]

(3-39)

\[
\sigma_{xx}^T/\sigma_{xx}^0 = 1 + j \frac{u_{xx}}{e_{xx}^0} + k \frac{u_{yy}}{e_{xx}^0} + Q
\]

(3-40)

\[
\sigma_{yy}^T/\sigma_{xx}^0 = q + k \frac{u_{xx}}{e_{xx}^0} + j \frac{u_{yy}}{e_{xx}^0} + Q
\]

(3-41)

Where,

\[
b = \sqrt{\frac{2(1+\nu)kh}{E}}, \quad a = b \sqrt{\frac{1-2\nu}{2(1-\nu)}}, \quad n = e_{yy}^0/e_{xx}^0, \quad q = \frac{v+n-n\nu}{1-v+n\nu}, \quad j = \frac{(2-\nu)(1-2\nu)}{2(1-\nu)(1-\nu+\nu)}
\]

\[
k = \frac{\nu(1-2\nu)}{2(1-\nu)(1-\nu+n)}
\]

\[
Q = \frac{ksinb}{b} \left[\frac{A_1}{e_{xx}^0} \frac{A_{1/2}}{e_{xx}^0} \frac{A_{1/2}}{\alpha} \sinh\left(a \frac{\alpha}{h}\right) + \frac{A_{2/2}}{\beta} \sinh\left(a \frac{\beta}{h}\right)\right]
\]
\[ A_1 = \frac{-G(\beta)\left[v\varepsilon_{xx}^0 + (1 - v)\varepsilon_{yy}^0\right] + H[(1 - v)\varepsilon_{xx}^0 + v\varepsilon_{yy}^0]}{K} \]

\[ A_2 = \frac{-G(\alpha)\left[(1 - v)\varepsilon_{xx}^0 + v\varepsilon_{yy}^0\right] + J[v\varepsilon_{xx}^0 + (1 - v)\varepsilon_{yy}^0]}{K} \]

\[ G(x) = \frac{4h^2bv(1 - v)}{\beta \sin b(1 - 2v)} \sinh \left(\frac{x}{h}\right) \]

\[ H = \frac{2hb(1 - v)}{\sin b(1 - 2v)} \left[ \cosh \left(\frac{\beta}{h}\right) + \frac{hv}{\beta} \sinh \left(\frac{\beta}{h}\right) \right] \]

\[ J = \frac{2hb(1 - v)}{\sin b(1 - 2v)} \left[ a(2 - v) \cosh \left(\frac{\alpha}{h}\right) + \frac{hv}{\alpha} \sinh \left(\frac{\alpha}{h}\right) \right] \]

\[ K = \frac{4h^2v^2}{\alpha \beta} \sinh \left(\frac{\beta}{h}\right) \sinh \left(\frac{\alpha}{h}\right) - \left[ a(2 - v) \cosh \left(\frac{\alpha}{h}\right) + \frac{hv}{\alpha} \sinh \left(\frac{\alpha}{h}\right) \right] \]

Using this set of equations, the normal stress field of a given asphalt pavement structure (length 2\(a\), width 2\(\beta\) and thickness \(h\)) can be solved as a function of \(E\) and \(\mu\) in the surface layer, the spring constant (\(k\)) between the surface and base layer and the applied thermal stress (\(\sigma_{xx}^0, \sigma_{yy}^0\)) /thermal strain (\(\varepsilon_{xx}^0, \varepsilon_{yy}^0\)). Similarly, the maximum normal stresses of a specific asphalt pavement structure can be obtained as a function of the average temperature change within the asphalt layer. Conditions for crack initiation can be approximated by comparing the maximum normal stress with tensile strength of asphalt material. Following this simplifying assumption, if the maximum calculated normal stress is larger than tensile strength of the asphalt mixture, cracks will begin to initiate and propagate through the asphalt layer.

### 3.3 Results and Discussion

#### 3.3.1 Pavement Response to Thermal Loading

In addition to an asphalt-over-substrate pavement structure, the proposed model is applicable to any two-layer elastic system with frictional interface subjected to constant normal
stresses in both the x- and y-directions. Displacement fields, shear stress fields and normal stress fields can be obtained for various planar dimensions, thicknesses, and by varying other model constants. A sensitivity analysis of crack spacing, spring coefficient, elastic modulus and Poisson’s ratio on the normal stress of the top layer was conducted.

The displacement field along the x-axis at the top and bottom of the asphalt layer is illustrated in Figure 3.2. As expected, the contraction at the top is always larger than that of the bottom, which is constrained by the underlying layer. As the distance from the center of an unbroken block increases, lateral displacements along the top and bottom pavement surfaces increase. The effect of pavement layers with differing elastic moduli ($E_1$, $E_2$, $E_3$ at three different conditions) were modeled by changing the relative stiffness parameter of the pavement surface versus substrate $kh/E$, i.e. by changing the elastic modulus while keeping the spring coefficient constant. As such, a stronger pavement surface yields smaller overall displacement magnitude at the top and bottom of the asphalt layer. In addition, an increase in the elastic modulus of the asphalt layer results in a more localized, higher curvature pattern near the cracked boundary at the top and bottom of the asphalt layer.

![Figure 3.2](image)

**Figure 3.2** Displacement field along x-axis at the top and bottom of the overlay where solid lines represent results at the surface and dash lines represent results at the bottom.

The shear stress distribution along the x-axis of the bottom of the overlay was also analyzed.
Figure 3.3. Shear stress distribution along x-axis at the bottom of the overlay with different spring coefficients/elastic modulus.

Normal stress response in the asphalt layer along the x-axis for different crack spacings are displayed for two length scales in Figure 3.4 and Figure 3.5, where the widths of uncracked pavement segments investigated in Figure 3.4 are larger than the range investigated in Figure 6. Similarly, conversely to shear stresses trends, the normal (tensile) stresses in the asphalt layer presented in Figure 3.4 and Figure 3.5 are maximum at the center of the uncracked block segments, and decrease gradually to approximately zero at the edge. Since a ‘weak form’ style boundary condition was used as a simplifying assumption in the derivation of the model, the normal stress at the boundary is not perfectly accurate, and therefore does not converge to exactly zero as it should. This will be apparent when comparing to finite element results in a later section. As indicated by Figure 3.4, when the uncracked segment width is relatively large ($\beta/h=10$), the normal stress at the center of the uncracked block segments is highly concentrated over a wider range of

\[
\frac{\epsilon_{xx} - \epsilon_{yy}}{\epsilon_{xx}} = \frac{1}{\sqrt{1+\nu}} \left( \frac{k}{E} \right)
\]

\[
\alpha/h = \beta/h = 10, \quad k/h/E = 10, \quad k/h/E = 1, \quad k/h/E = 0.1
\]
crack spacings. When the uncracked segments are smaller (Figure 3.5), the overall normal stress is higher with larger crack spacing, which indicates that normal stress magnitudes are relieved as more cracking forms and that cracks will eventually stagnate until a larger thermal cycle is experienced and/or when the pavement increases stiffness and/or decreases fracture resistance as it ages with time in the field.

![Figure 3.4](image)

**Figure 3.4** Normal stress distribution along x-axis at different crack spacing over a range of large uncracked segment widths \((\alpha/h)\) where solid lines represent results at the surface and dash lines represent results at the bottom.

![Figure 3.5](image)

**Figure 3.5** Normal stress distribution along x-axis at different crack spacing over a range of relatively small uncracked segment widths \((\alpha/h)\) where solid lines represent results at the surface and dash lines represent results at the bottom.

As shown in Figure 3.6, the normal stress distribution along the x-axis at different interface
conditions/elastic moduli were also studied. It was found that the normal stress at the center of the overlay was higher for stiffer interface conditions and/or with stiffer asphalt layers.

![Normal stress distribution along x-axis at different spring coefficients/elastic modulus](image)

**Figure 3.6** Normal stress distribution along x-axis at different spring coefficients/elastic modulus where solid lines represent results at the surface and dash lines represent results at the bottom.

Poisson’s ratio of asphalt mixture at low temperature varies from 0.15 to 0.35, considering different mixtures, aging levels and loading rate (NCHRP 1-37A, 2004; Camarena, 2016). The effect of Poisson’s ratio on the normal stress distribution along the x-axis at the top of asphalt layer is presented in Figure 3.7. As shown in the figure, the overall trends of normal stress at three cases were almost the same. When Poisson’s ratio changes from 0.15 to 0.35, slightly smaller normal stress was obtained.
3.3.2 Use and Limitations of Model

In practice, asphalt concrete will be viscoelastic over its service range, which cannot be directly accounted for in the present model. However, the model provides a very useful first step in better understanding block cracking for several reasons, including: (1) through engineering judgment and a reasonable choice of low temperature asphalt modulus and Poisson’s ratio, the analyst can obtain useful information regarding general trends in block crack formation at very low temperatures, where viscoelastic stress relaxation is less dominant; (2) through use of the elastic-viscoelastic correspondence principle and the help of a spreadsheet or program such as Matlab, viscoelastic moduli can be substituted for elastic moduli to obtain a closer approximation of pavement responses resulting from temperature changes, and; (3) the solutions can be used in the verification of more complex numerical simulations, such as 3D finite element (FE) analyses, which can directly account for material viscoelasticity and fracture. Without reference elastic analytical solutions, it is difficult or impossible to know if complex FE models are providing trustworthy results, and thus, the onus is on the model developer to verify their approximate numerical solutions. The finite element method is by nature an approximation of the system it
seeks to model, with solution accuracy that depends on mesh size and quality, element formulation, solution framework and convergence, and on human errors in model development, data entry, and interpretation of results. By setting properties as elastic in candidate FE models, the solution presented herein provides a comprehensive tool for model verification. This should be done prior to turning on advanced, highly non-linear features in the FE model, such as viscoelasticity, damage, fracture, and/or interface sliding. When turning on these features, reference solutions will typically not be available. Thus, model verification against closed-form analytical solutions such as provided herein are an essential step in developing a trustworthy numerical solution.

3.3.3 **Comparison of Present Analytical Solution with FEM Simulations**

In order to verify the proposed analytical model, FEM simulations were performed using the commercial finite element code ABAQUS. To simplify the verification procedure, the overlay was assumed to be broken into periodically-arranged square block crack segments, i.e., $\alpha=\beta$. Since the overlay is symmetrical about the x- and y-axes, with a choice of proper boundary conditions, only a quarter of the model domain was simulated. Four-node quadrilateral elements were used in the FEM simulation. Since neither analytical nor numerical solutions were available in the literature for this problem, the approach taken was to simultaneously verify both the analytical and numerical model by comparing solutions and evaluating expected similarities and differences, especially due to differences in boundary conditions and other slight differences in model formulations.

The normal stress ($\sigma_{xx}$) distribution along the x-axis of the surface of the asphalt layer is presented in Figure 3.8. The analytical solution of horizontal thermal stress distribution along the x-axis matched well with FEM results for values of $x/\alpha$ smaller than 0.80. However, when the
boundary of an existing crack is approached, i.e., $x/\alpha$ is greater than 0.80, the analytical solution predicted lower normal stress as compared to the FEM solution. This trend was expected due to the weak-form boundary solution used in the analytical model. However, recall that the intended use of the analytical model was to study tensile stresses in the center of relatively large un-cracked pavement segments, where $x/\alpha$ is small and the analytical solution is quite accurate. Thus, the accuracy of stress/strain results close to the center of the unbroken segment are much more important than those at the boundary.

![Figure 3.8](image)

**Figure 3.8** $\sigma_{xx}$ distribution along x-axis on the surface of the top layer where solid lines represent analytical results and dash lines represent FEM results.

In the proposed analytical model, the use of a frictional interface and a superposition technique to negate shear sliding at the interface was assumed to be an accurate and efficient method to develop analytical expressions for the response of a fully-bonded interface. In order to verify this assumption, comparison to FEM results was also pursued. Numerical FE models with two different interface conditions, a frictional interface and a fully-bonded interface, were studied using ABAQUS models in order to verify if the interface assumption used in the analytical model is valid, as shown in Figure 3.9. Spring elements were used in the x- and y-direction to simulate a frictional interface condition, while nodal displacements were pinned (prohibited) in all directions.
for the fully-bonded interface condition. The thickness of the base is ten times that of the overlay for the model with the fully-bonded interface in this example. As shown in Figure 3.10, the model with a frictional interface yielded a higher overall tensile stress prediction than the one with a fully-bonded interface, due to the smaller allowable deformation in the fully-bonded interface condition. Combined with the effects of ignoring viscoelastic stress relaxation effects, the proposed analytical model in this chapter will provide conservative results for block cracking initiation predictions.

Figure 3.9. Finite element (FE) model with frictional interface (left) versus FE model with fully bonded interface (right).
3.4 Application Examples Using the 3D Analytical Block Cracking Model

To further explore how the new 3D analytical solution can be used to understand block-cracking mechanisms, additional results and discussion are now presented.

3.4.1 Application Example #1: Predicting Block Crack Initiation and Size

To study how a rectangular block cracking pattern might form, a schematic figure depicting stress fields and resulting crack patterns was developed as shown in Figure 3.11. In addition to the simplifying assumption of elastic material properties, it was also assumed that once conditions for crack initiation are reached, the crack will eventually propagate through the thickness. Note that normal stress $\sigma_{xx}$ along the y-axis is the driving force for transverse cracking, and that normal stress $\sigma_{yy}$ along the x-axis is the driving force for longitudinal cracking. In this case, we assume that transverse cracks always initiate at the center of the unbroken block crack segment. Thus, rectangles first form, but eventually a square block cracking will begin to emerge. Then, once a square-shaped pattern is reached, transverse cracking and longitudinal cracking are developed at the same rate until a saturated (equilibrated) crack spacing pattern is reached for the given loading conditions. Further cracking into smaller blocks will require larger driving forces (larger
temperature change or material stiffening) or less resistance, i.e., lowering of tensile strength (or in a more complex analysis, lowering of fracture energy or other material fracture parameters).

Figure 3.11 Hypothesized formation of block cracking based on new analytical solutions where dash lines represent \( \sigma_{xx}/\sigma_{xx}^0 \) and solid lines represent \( \sigma_{yy}/\sigma_{xx}^0 \).

Although quite idealized and simplified, the current 3D model can be used to approximate the point of crack initiation for a given asphalt pavement under critical temperature cycles, or can be used to predict the critical thickness and saturated block crack dimensions for a given asphalt material. Moreover, the solutions can be used to verify more sophisticated finite element or discrete
element models, which after verification against the elastic, strength-of-materials type solution, can be used to study the effects of more complex and realistic conditions, such as material viscoelasticity, damage, 3D crack propagation, temperature and material gradients and interface cracking and sliding.

3.4.2 Application Example #2: Surface Thickness Effect on Block Crack Formation

A second example is now provided using the 3D analytical model, this time using realistic ranges of elastic moduli estimates, pavement temperature cycling magnitudes, thicknesses and modulus ratios with the underlying layer for different pavement configurations. The resulting solutions are then discussed in terms of conditions necessary for block crack formation. Suppose the following conditions exist: asphalt material elastic modulus of 20GPa (low temperatures prevail), a spring coefficient of 2GPa, Poisson’s ratio of 0.2, a coefficient of thermal expansion of 3E-5/°C, asphalt tensile strength of 3.5MPa, and a base layer with a thermal expansion coefficient of 1E-5/°C. As shown earlier, thinner asphalt layers will have larger stress concentrations, and therefore a critical thickness of the asphalt layer will exist for a given material system and given average temperature fluctuation in the asphalt layer. For this parametric study, it is assumed that when the thickness of the asphalt layer is smaller than some critical thickness, the computed stresses will be lower than the material strength and further block cracks will not develop. For example, if a pavement already having a 1m×1m asphalt block cracking pattern experiences an average temperature change through the thickness of 20°C during the winter, the critical asphalt thickness for the development of additional block cracks would be 90mm. Asphalt surfaces thinner than 90mm would be vulnerable to further block cracking under these conditions. To gauge the sensitivity of the pavement thickness parameter, the saturated crack area for a slightly thinner 80 mm-thick asphalt layer predicted by the 3D model would be 0.85m×0.85m. Because an assumption
of material elasticity is used, the results presented can be viewed as conservative.

3.4.3 Application Example #3: Field Case Study

A field validation example is now presented, evaluating the ability of the analytical model to estimate approximate block crack size using laboratory results obtained from field cores. In July 2017, a dense block cracking pattern was observed on US63 near La Plata, Missouri after nine years of service life, as shown in Figure 3.12. The surface mix was placed as a rehabilitative overlay in 2008 with a thickness of 1.75in. The surface mixture was comprised of a PG 64-22 binder, in a 12.5 mm nominal maximum aggregate size Superpave mixture, featuring 26.8% asphalt binder replacement achieved by using 18% reclaimed asphalt pavement (RAP) and 2% reclaimed asphalt shingles (RAS). In order to examine the relaxation behavior and low-temperature fracture properties of the surface mix, creep compliance testing and DC(T) fracture energy testing was conducted in accordance with AASHTO T-322 and ASTM D7313, respectively. Due to the elastic assumption of the analytical model, the instantaneous relaxation modulus was taken as the elastic modulus of the surface mix, which was obtained through interconversion of the creep compliance master curve at a reference temperature of -24°C. The tensile strength of the surface mixture was estimated from the peak load measured in the DC(T) test using Equation (3-42), which was obtained from ASTM E399-90 (1997) and applied by Dave et al. (2013).

\[ S_t = \frac{2P(2W + a)}{B(W - a)^2} \]  

(3-42)

Where, \( S_t \) is material tensile strength in MPa and \( P \) is the maximum load sustained by sample with the unit of N, \( B \) is the thickness of sample, \( W \) and \( a \) are geometric parameters of DC(T) specimen with recommended values of 110mm and 27.5mm, accordingly (Wagoner et al., 2005). Due to normal in-situ, top-down aging processes, the material properties in the surface layer change through the layer thickness in a highly graded manner, whereas the obtained laboratory
results are the averaged results of the material properties in the top 50 mm of the cores. To obtain graded properties, results from the studies of (Apeagyei, 2006, Braham et al., 2009, Buttlar et al., 2006) were examined and applied. In summary, based on these test results along with engineering judgment derived from previous studies, material properties (elastic modulus and tensile strength of the surface layer) and material grading parameters across the 50 mm thickness were established, as shown in Table 3.1. The method of other parameter assignment was similar to the previous example; i.e., the value of the spring coefficient between the surface layer and the underlying layer is assumed to be one tenth of the elastic modulus of the surface layer, and all other parameters are assumed to be the same with the previous example.

**Figure 3.12.** Block cracks at US63 at La Plata, MO in July 2017, after 9 years of service.
Table 3.1 Block crack areas of US63 at La Plata, MO predicted via analytical solution

<table>
<thead>
<tr>
<th>Crack depth</th>
<th>Elastic modulus multiplier</th>
<th>Elastic Modulus (MPa)</th>
<th>Tensile strength multiplier</th>
<th>Tensile strength (MPa)</th>
<th>Crack area (m²)</th>
<th>Observed block crack dimension range (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>near-surface</td>
<td>1.55</td>
<td>2.15E+04</td>
<td>1.05</td>
<td>3.28</td>
<td>1.67×1.67</td>
<td>0.12×0.15&lt;sup&gt;1&lt;/sup&gt; 1.54×1.76</td>
</tr>
<tr>
<td>mid-depth</td>
<td>1.20</td>
<td>1.66E+04</td>
<td>1.01</td>
<td>3.16</td>
<td>1.98×1.98</td>
<td>-</td>
</tr>
<tr>
<td>full-depth</td>
<td>1.00</td>
<td>1.38E+04</td>
<td>1.00</td>
<td>3.13</td>
<td>2.34×2.34</td>
<td>-</td>
</tr>
</tbody>
</table>

<sup>1</sup>For example, 0.12m by 0.15m, rectangular-shaped block cracks were observed (minor cracks), residing within 1.54 m by 1.76 m rectangular-shaped block crack patterns (major cracks), see also Figure 3.12.

Examining the temperature history of La Plata, MO from December 2016 to January 2017, the average daily winter temperature change was about -15°C. Therefore, for full-depth, mid-depth and near-surface cracks, the corresponding saturated crack block dimensions were found to be 2.34m*2.34m, 1.98m*1.98m, and 1.67m*1.67m, respectively. A visual observation of the field section (Figure 3.12) revealed that the sizes of block cracks vary from 0.12m*0.15m to 1.54*1.76m. The ‘major’ block crack size observed in the field (larger blocks with notably wider cracks – likely the block cracks that formed first and have penetrated the deepest) was similar to the near-surface crack size predicted by the analytical model. The presence of smaller block cracks in the field suggests the strong near-surface cracking vulnerability of the highly-aged mixture containing RAP and RAS. Therefore, the present analytical model is capable of providing a reasonable prediction of the primary, larger block crack pattern observed in the chosen field section. The smaller block cracking pattern perhaps could be captured in the future with more precise materials characterization, and moreover, with a 3D numerical modeling scheme to capture additional physical quantities such as bulk viscoelasticity and fracture property gradients, temperature cycling, and material morphological representation (aggregates, mastic, air voids).
A number of other parametric studies can be generated using the model, but are not included herein for brevity. Again, the primary utility of the model is to provide a benchmark for the development and verification of more complex 3D numerical models. Work is currently underway to develop 3D block cracking models in both the finite element method and the discrete element modeling (DEM) method, where it is anticipated that the phenomenon of hexagonal block cracks can be better understood by allowing steep material gradients (highly oxidized, brittle asphalt surface) and full 3-D crack propagation to be simulated.

3.5 Summary

A comprehensive new closed-form, 3D analytical model of an elastic two-layer pavement structure under constant temperature change through the thickness was established to explore the mechanisms underlying block cracking. Displacement fields, along with shear and normal stress distributions were presented graphically. A schematic was developed and presented to illustrate how transverse and longitudinal block crack patterns systematically develop due to prevailing high tensile fields and combine to eventually form block cracking patterns. It was also demonstrated how the solution can be used for rough estimation of block cracking trends as a function of temperature change, layer thickness, existing block crack size, and layer properties. The model was shown to provide a reasonable estimate for the major rectangular block cracking pattern observed on a field validation section in La Plata, Missouri. However, smaller, near-surface hexagonal crack patterns were not captured with the analytical model. To capture the true fractal, 3D nature of block cracking, a true 3D model with graded viscoelastic and fracture properties will be required, which will be introduced in Chapter 5.
CHAPTER 4: 2D MICROMECHANICAL, VISCOELASTIC AND HETEROGENEOUS MODEL FOR THE STUDY OF BLOCK CRACKING PATTERNS IN ASPHALT PAVEMENT

4.1 Introduction

Figure 4.1 shows two major types of block cracking patterns observed in the field. The most common one is a rectangular pattern, which represents a superposition of longitudinal and transverse cracks in the surface of the asphalt layer, as shown in Figure 4.1a. Of these two crack orientations, the transverse direction is clearly related to the mechanisms behind traditional thermal cracks. That is, tension develops in the ‘long’ or ‘longitudinal’ direction, resulting in a mode I or pure tensile crack that propagates orthogonal to the maximum stress, or in the transverse direction. In the case of block cracking, when the crack penetrates vertically, it encounters warmer (and thus tougher), less aged (and thus less brittle) asphalt concrete. This, combined with the fact that temperature gradients and therefore strain levels are less critical with depth, means that cracks have a much higher propensity towards channeling around the surface as compared to penetrating deeper. In terms of the longitudinal crack orientation, this pattern may be preferential in cases where the transverse cracks developed first, relieving longitudinal strain in the vicinity of cracks, and leaving transverse strain as maximum. In addition, longitudinal material non-uniformity and damage patterns, caused by pavers (mix segregation at paver conveyor edges, mix flowing around gear boxes, auger extensions, etc.), rollers (check marks and other permanent damage and inhomogeneities), and traffic induced damage (top-down cracking damage patterns leading to vulnerability to thermal damage) are all expected to facilitate preferential rectangular block cracking patterns.
The other common type of block cracking is hexagonal in shape, as depicted in Figure 4.1(b). This is similar to the pattern formed when soil dries below the shrinkage limit. When thermally-induced macro cracks with different orientations propagate and grow in size; they eventually coalesce with each other to form a hexagonal type of crack. A hexagonal-shaped block cracking pattern might be attributable to situations where transverse cracks do not develop and propagate significantly before other orientations, perhaps due to the nature of the gradient of aging and therefore properties in the vertical directions. A high brittleness gradient combined with a critical contraction event could create conditions for rapid channeling of cracks in random orientations. Also, the effect of construction or traffic-related longitudinal distresses may be less prevalent in situations where this type of cracking occurs. Regardless of the type of block cracking pattern that develops, its manifestation is that of a widespread surface deterioration pattern, often leading to significant damage and high maintenance costs. Therefore, it is important to develop a better understanding of the mechanisms behind block cracking in order to establish effective techniques to prevent or delay its damaging effects.

Environmental and material-related factors that cause thermal or transverse cracking are
also thought to drive block cracking. The environmental factors, particularly near the surface after years of service, include: critical low air temperature, sharp temperature changes, and aging mechanisms. Other material and construction-related factors may influence thermal and block cracking formation and the specific pattern of block cracking observed. An example of the combination of both block and thermal cracking is shown in Figure 1c. Factors such as binder grade, aggregate strength, binder and additive chemistry, and thickness of pavement layers can play a significant role in block crack formation.

Asphalt mixture is assumed to be a viscoelastic material at low temperatures, i.e., the material properties change with time and temperature. It is assumed that the formation of block cracking phenomenon can be divided into two steps: (1) initiation of microcracks, because of material insufficiency in stress relaxation capacity or fracture resistance property, and (2) channeling of microcracks in arbitrary orientation, which is highly dependent on local material properties. Considering such complex requirements, a microstructure-based discrete element model is one of the simulation methods that can meet all of these requirements. Further, Particle Flow Code (PFC), developed by Cundall, has proven to have this capability, based on the work of previous researchers, including those studying cracking in asphalt pavement surfaces (Cundall, 1971; Kim et al., 2008; Kim et al., 2009a; Hill, 2016).

Asphalt mixture is multi-phase material that includes asphalt, multi-size aggregates, fillers and/or other additives. If all the materials are simulated in the model, then the model will become very complicated and computationally expensive or prohibitive to run. Along these lines, You and Buttlar (2005) pointed out that mastic, interface and aggregate are sufficient to represent the skeleton structure of asphalt mixtures, and a model comprised of elements smaller than 0.6mm in radius which filled in hexagonal close packing will be sufficient to simulate fracture properties of
asphalt concrete test specimens (Kim et al., 2008). In addition, they also illustrated that the viscoelastic and fracture properties of asphalt mixture can be represented with a creep compliance master curve and disk-shaped compact tension test, or DC(T) based fracture energy and maximum displacement value as minimum model requirements. In this portion of the study, a typical PG 64-22 asphalt mixture was utilized as the baseline material, all bulk and local material properties of which are provided by Hill (2016). Hill obtained the local fracture parameters by minimizing the difference between the measured displacement field from digital image correlation and the simulated displacement field from the discrete element model via a nonlinear optimization method.

The primary objective of this chapter is to develop a two-dimensional viscoelastic, heterogeneous and anisotropic microstructure-based pavement model to study the cracking mechanisms behind the development and propagation of block cracking patterns, and to examine the effect of cooling rate, material aging and relaxation and the model dimension on the crack patterns.

4.2 Two-dimensional Microstructure Model

In the ensuing simulation results, it is assumed that the block cracking pattern can be simulated in a 2D large-scale microstructure-based model with a viscoelastic constitutive model, heterogeneous material morphology representation and thermal contact model. The heterogeneous material morphology was obtained from high-resolution image and achieved by considering asphalt mixture as a three-phase material comprised of mastics, aggregates and air voids, along with interfaces between mastics and aggregates. In order to simulate a block cracking pattern, an approximate square-shaped domain extent in plan view is desired, as illustrated from the analytical model presented in Chapter 3. The close-hexagonal packing arrangement with particles of radius 0.55mm were selected to assure accuracy in the simulation (Kim et al., 2009b). Considering the
computational power of a server-type personal computer, a 0.44m*0.38m model with 160,000 elements and 478,401 contacts subjected to planner thermal straining was constructed to simulate the asphalt pavement surface. Supercomputer simulations were considered but were ruled out as it was desired for other researchers and practitioners to be able to verify and extend the results of this study. Similarly, a 0.89m*0.77m and a 0.89m*0.06m models were also built. In terms of the square-shaped pavement model, it was assumed that the pavement system was symmetrical in all directions, one section in the middle was selected as the area of interest for the simulation. Even with this simplifying approach, a number of practical simulation thrusts were possible. For instance, when the model was used to simulate a square-shaped pavement surface, it was assumed to be fully-restrained by the surrounding materials; when the model was used to simulate a rectangular-shaped pavement model, then it was assumed to be only restrained at the short ends, and finally; when the model was used to simulate the pavement cross section, then it was assumed to be fully bonded to the underlying pavement layer.

4.2.1 Model Establishment Approach

The approach of making a representative 2D microstructure model, and schematic diagrams of the utilized thermal and viscoelastic cohesive-softening contact model are introduced in Figure 4.2. Because it is difficult to obtain a 0.44m*0.38m asphalt mixture specimen, the actual particle arrangement was obtained from duplicating the images of multiple smaller samples that were arranged in a random order. Next, the mastic and aggregates were recognized via MATLAB by detecting color intensities, and the coordinates of all the representative points were outputted to PFC software to generate the heterogeneous geometry. As such, a 0.44m*0.38m pavement model was built, and likewise, the other models investigated were built using the same approach.
Step 1: The specimen was sliced into 13 pieces and high-resolution images of each slice were taken.

Step 2: Images of each slice were cropped into rectangular shapes, which were randomly arranged and stitched together to create a model for a larger section.

Step 3: The specimen image was converted into binary form, and mastics and aggregates were identified based on color intensities. After that, mesh coordinates according to regular hexagonal packing arrangement were used to determine if each point was mastic or aggregate and then output locations and material group information for further processing.

Step 4: Generate elements in PFC based on the coordinates and material group information outputted from MATLAB.

**Figure 4.2** Five-step generation process of a 2D representative pavement model.
Step 5: Generate all contacts in the PFC software, detect mastic-mastic contacts and output their IDs. Calculate contact strength of the mastic contacts based on a normal distribution, and assign them back to the contacts randomly.

4.2.2 Materials

To represent the viscoelastic behavior of asphalt mixture, a viscoelastic cohesive-softening contact model was utilized. This model combined a viscoelastic Burger’s model with a linear softening model to describe viscoelastic (bulk) and local separation behavior of asphalt concrete. A schematic model was provided in Figure 4.2, Step 4, with additional details introduced in Chapter 2.
A typical PG64-22 asphalt mixture without any additive or recycled materials was utilized, the local viscoelastic and fracture properties of mastic, interface and aggregates were provided by Hill (2016). Hill identified the local fracture properties through a DIC-DEM optimization approach by minimizing the difference between the measured displacement from DIC and the simulated displacement from the PFC software, and reported that the optimum local fracture parameters for a PG 64-22 mixture was \((f_{max}^{opt}, \delta_{max}^{opt}) = (83N, 1.2 \times 10^{-4}m)\), which was used in the following simulations. The interface properties were assigned as the ratios of the interface strength and fracture energy to mastic strength and fracture energy following the method introduced by Kim et al. (2008), and the aggregate properties were also assumed to be the same with those used by Kim et al. (2008). Detailed material properties used in the model are provided in Table 4.1. In terms of material properties after aging, these were established by assuming previously measured properties of recycled asphalt pavement (RAP) material from Illinois, as reported by Hill, where the optimum local fracture parameters were \((f_{max}^{opt}, \delta_{max}^{opt}) = (100N, 3.3 \times 10^{-5}m)\).
Table 4.1 Model input parameters for mastic, interface, aggregates and Portland cement concrete base

<table>
<thead>
<tr>
<th>Mastic - Viscoelastic cohesive-softening contact model</th>
<th>Burger’s model parameters (pre-peak portion)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maxwell</td>
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<tr>
<td></td>
<td>Stiffness (N/m)</td>
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<td>3.82e8</td>
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<td>Viscosity (N·s/m)</td>
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<td></td>
<td>Kelvin</td>
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<tr>
<td></td>
<td>Stiffness (N/m)</td>
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<td>Interface - Viscoelastic cohesive-softening contact model</td>
<td>Burger’s model parameters (pre-peak portion)</td>
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<td>Strength (N)</td>
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In general, identical material properties were assigned for each contact of a given type, i.e. mastic, interface or aggregate, which was usually not an issue in terms of crack path determination for simulations that have a pre-defined notch to guide the crack path. However, in simulations intended to study randomly-initiated and randomly-oriented cracks, it was not desired for the crack path to be pre-defined or biased in terms of propagation direction. It is known that the propagation direction of a crack depends on the local fracture properties of the materials around the crack front, along with the local state of stress. In general, crack tortuosity will be reduced, due to the regularity of the mesh used, and regularity of the material assignment, particularly when a homogeneous material modeling approach is used. Therefore, in order to create a more realistic heterogeneous...
material simulation, the mastic portion was simulated by using properties that randomly varied according to a normal-type distribution in terms of contact strength. The mean of the distribution was set to be equal to the local contact strength presented by Hill (2016), while a standard deviation of 8 J/m² was used, which corresponds to a coefficient of variation of 9.6%. The distribution of the contact strength of mastics can also be seen by referring back to Figure 4.2, step 5. The other material properties associated with the mastic (e.g., stiffness) and all other material properties associated with aggregates and interfaces were set using the properties in Table 4.1 without introducing statistical randomness, for the sake of simplicity.

4.2.3 Thermal Properties

The thermal loading was applied by using a built-in thermal feature provided in the PFC software. This contact model was developed by Li et al. (2016) to simulate the heat conduction between the particles, which allowed thermally-induced stresses to develop among particles in the DEM model by changing the radius of each particle, where details were introduced in Chapter 2.

The simulation was assumed to start from -12°C, as previous simulation results had shown that there was no significant stress build up prior this temperature due to the significant material relaxation that existed. This simplification also served to reduce simulation times dramatically, which was desirable given the computational expense associated with many of the simulations conducted in this study (i.e., some simulations taking more than one day to complete). The coefficients of thermal contraction of mastic and aggregates were selected as 4.0×10⁻⁴/°C and 2.0×10⁻⁶/°C, accordingly, as found in the literature (Islam and Tarefder, 2015; Marasteanu, 2012). A relatively harsh pavement surface cooling rate found in cold climates in the United States is -2°C/h, which corresponds to a cooling rate of -0.03°C/step in the simulation. This correspondence in rate is based on the assumption that the viscoelastic model reaches suitable convergence within
60 seconds. This was further validated by checking the average computation time of each step in the simulations ran in this study, noting that the computation time associated with microcrack location searching was not considered.

4.3 Simulation Results and Discussion

4.3.1 Formation of Block Cracking Pattern

In the simulation, it was assumed that block cracking formation could be separated into two steps: (1) initiation of microcracks as the result of insufficient stress relaxation capacity and/or fracture resistance of the material, and; (2) channeling of cracks in the pavement surface, the path of which is highly dominated by local material properties and boundary conditions. To understand how displacement and force distribute, and how cracks initiate and propagate in the model throughout the simulation, full simulation results of one sample run are provided in Appendix B, while selected simulation results are presented herein. In discrete element modeling, displacements of particles and force at contacts are easily obtained from the simulation results, while strain and stress must be computed from displacement and force using a selected averaging technique. Thus, most of the results presented herein are in terms of local force, displacement and breakage/separation of bonds.

4.3.1.1 Initiation of Microcracks

As temperature decreases in a simulation, mismatches in local thermal strain between particles increases and resulting local thermal stress begins to accumulate. Although dominated by tension, model heterogeneity in the DEM simulation will in fact create localized areas of light compression. As shown in Figure 4.3, locations with larger element displacement were mainly located along the interfaces between the asphalt mastic and aggregates, due to the relative mismatch in coefficient of thermal contraction. When the energy stored in the contact exceeded
the assigned fracture energy, the contact broke and a local microcrack developed.

**Figure 4.3** Locations with larger x-/y-displacement were observed along the interface between the asphalt mastic and aggregates as a result of mismatch coefficient of thermal shrinkage, where the x-/y-components of displacements are positive when pointing in the directions of the positive global x-, y-coordinate, the microcrack are marked as either red short line or black short line.
In general, microcracks first developed in the contacts which were assigned lower fracture resistance. In the simulation (Figure 4.4), microcracks were mostly located in the mastic or along the interface because of their significantly lower strength than that of aggregates and because of the smaller thermal coefficient in aggregates. In some cases, the resultant force in some of the aggregates was larger due to their higher stiffness but the force did not surpass their contact strength, and thus no cracking was detected within aggregates at early simulation cycles. Therefore, to emphasize the force distribution in the mastics and interfaces, responses in aggregates were suppressed from plots starting from -12.9°C.

![Image](image_url)

**Figure 4.4** Example of crack initiation locations in the model at -12.9°C, the parameter bb_tenf in the figure represents contact strength of the contacts, both red and black lines represent broken contacts/microcracks.

When cracks formed, the thermal stress of the surrounding materials was released significantly, leaving the remaining thermal stress mainly concentrated at the crack tip. For example, in Figure 4.5, the magnitude of the thermally-induced force at the contacts was reduced significantly after a few cracks formed, as shown in the left region of the figure, whereas that of
the contacts in right region of the figure (which had fewer cracks) were relatively higher. A decomposition of the force along horizontal and vertical directions was also provided in the same figure. It was found that the component of force in the horizontal direction was larger, which indicated that the cracks were more likely to propagate vertically in the next calculation step. On the other hand, the component of force in the vertical direction was smaller, which indicated that the existing cracks released the majority of the forces in this direction such that the cracks would not propagate further until at least the next calculation step. This was confirmed by the simulation results at -13.2°C, when the cracks propagated either upward or downward.

**Figure 4.5** Example of the force distribution in the model at -12.9°C, where the bur_force_mag denoted represents the magnitude of contact force, the bur_force_x denoted represents contact force along horizontal direction and the bur_force_y denoted represents contact force along vertical direction, positive forces indicate compression and negative forces indicate tension, all black lines represent broken contacts/cracks.
4.3.1.2 Channeling of cracks

As temperature continues to decrease, cracks will continue to nucleate randomly at sites of local stress intensity or propagate in a relatively straight path until reaching a boundary, an aggregate, other tougher materials or other cracks. Recall that cracks tend to grow in a direction that minimizes the required energy to nucleate a new crack surface. In simulations of asphalt concrete, cracks tend to propagate orthogonal to the direction of maximum tensile stress (termed herein as ‘case A’) until they meet an aggregate (‘case B’), or zones of tougher mastics (‘case C’). All three cases are presented in Figure 4.6. In Figure 4.6(a), it was clearly seen that cracks propagate in a straight line within the mastic before reaching a stronger material. It is noted that this case is not true all the time, i.e., in some instances cracks might propagate directly through aggregates continuing along the same straight path if the loading rate is fast or if aggregates are very brittle. This phenomenon is observed in mixture fracture testing at low temperature (Kim, 2007). However, this case holds true in the present simulations because of the much higher strength of the aggregates assigned as compared to that of mastics or interfaces. One example of case C crack behavior was provided in Figure 4.6(b), where the crack was deflected, traveling around the tougher material. In general, this case is more common when the cooling rate is relatively small, such that the thermal stress develops slowly, providing time for cracks to ‘locate’ paths that minimize the required energy to create a new crack surface. This complex, rate-dependent fracture phenomenon is certainly a function of material viscoelasticity and dynamic response, both of which are present in the current DEM simulations.

Similar to case A and B, departure from case C is observed at times, not only in the field (and in laboratory testing), but also in the simulations. Because the difference between the fracture energy of mastic and that of another mastic is significantly smaller than that between mastic and
aggregates, stored energy, when released is sometimes enough to break through slightly tougher mastics zones. Therefore, sometimes cracks were seen to propagate through the tougher mastics in the simulation. Generally, when this case happened, a few microcracks developed at the same time such that a clear propagation path could not be distinguished. A good example can be found near the middle of Figure 4.6(a), close to the bottom region (not highlighted). This helps justify why varied contact strength values were assigned to mastic contacts instead of the uniform property assignment; i.e., the statistical variance helps produce richer and presumably more realistic local fracture behavior.

Figure 4.6 Examples of varying crack propagations; case A: cracks propagate straight, with examples highlighted in yellow rectangular; case B: cracks turn when aggregates are met, with examples highlighted in green circles; case C: cracks turn when encountering tougher material, with examples highlighted in black semi-eclipses. Note: these figures were selected from the simulation results obtained at -14.1°C, where all red short lines denoted in the figure represent broken contacts/cracks.

In general, cracks tend to grow along the direction that maximizes the difference between the energy released and the fracture energy consumed during the crack growth. The overview of crack patterns at the end of the simulation is shown in Figure 4.7. In this global view, cracks tend to join at 90° or 120° which implies that both rectangular and hexagonal shaped cracks can form in the same simulation. Noting that this conclusion is based on the assumption that there is no material segregation and no other types of existing cracks dominating in the model, details about
how material segregation or other types of cracks affect the formation of block cracking pattern will be discussed later. Although cases where cracks intersected at other angles were also observed in the simulation results, the cases where cracks joined at 90° or 120° are far more frequent. Examples of each are presented in Figure 4.8.

![Figure 4.7 Overview of crack patterns at point of crack saturation.](image)

![Figure 4.8 Cracks joining at different angles, dominated by 90° and 120° intersections.](image)
T-junction cracks, or the case when the angle between cracks is 90°, are very common in the field, as shown in Figure 4.9a. When multiple cracks coalesce, a growing crack will tend to curve to intersect another crack at right angles. This is because cracks relieve stress perpendicular to its direction of orientation, as Goehring (2013) illustrated. An evolved version of T-junction cracks was also observed in the simulations. When two adjacent parallel cracks propagate, they tend to turn and join each other at right angles if there is a local material heterogeneity present to induce crack deviation (Figure 4.10). Similar phenomena have been observed in the field (Figure 4.9b, c).

Figure 4.9 T-junction cracks observed in the asphalt pavements: (a) traditional T-junction cracks, (b) evolved T-junction cracks, photos were taken from a parking lot in Champaign, Illinois.
Y-junction cracks, or the case when the angle between cracks is 120°, has been observed in other materials as well including the crack pattern in Figure 2.3 and 2.4. Cracks in concrete and brick walls of this nature are shown in Figure 4.11. In the present DEM simulations, the hexagonal packing arrangement of elements might be a biasing factor for the relatively high occurrence of hexagonally-shaped cracks. In the field, uniform stress states and non-biased material randomness may produce a higher occurrence of Y-junction cracks, which is one of the important steps in the formation of hexagonal cracks. Goehring (2013) observed the phenomenon that a rectilinear mud crack will turn into a hexagonal pattern after a cyclic drying and wetting process. This might be due to some heterogeneity effect of muds developed during the drying and wetting cycles. It is known that asphalt mixture is heterogeneous, which implies that the rectilinear and hexagonal crack pattern may occur under the same conditions.

Figure 4.11 Y-junction cracks: (a) in Portland cement concrete (sb LENDING, 2018), and (b) in a brick wall (House Check, 2018).

All in all, rectangular and block crack patterns form in a similar way with other materials,
especially soils. However, after careful examination of field cracking, it was found that the edges of block cracking patterns in asphalt pavements are more curved as compared to others, and the size of block patterns varies widely. A possible reason for this might be the time- and rate-dependent viscoelasticity properties and heterogeneous nature of asphalt mixture. From the simulation results, it was found that, once initiated, cracks grew very fast and would reach a plateau (point of crack saturation) at some stage. Therefore, most of remaining results presented herein will focus on the equilibrated, final crack patterns reached in each simulation.

4.3.2 Factors of Block Crack Patterns

Literature shows that thermal and/or moisture effects are the leading factors behind observed crack patterns in materials such as starch columns, basalt columns and mudcrack fields. It is therefore assumed that, at least some extent, these factors also apply to asphalt pavement surfaces. These factors, plus the viscoelastic nature of asphalt materials, and the specific three-dimensional geometry of pavements are believed to underlie the major forces behind the observed block cracking patterns in asphalt pavement. Moisture effects were not considered in this study, and thus, the current study was limited to factors including the effects of model geometry (3D domain extent and boundary conditions), viscoelastic properties, modulus and thermal coefficient variations between component materials, cooling rate, aging gradients, and existence of pre-existing crack. These factors are now examined by isolating their effects in the following simulations.

4.3.2.1 Dimension

In order to investigate the phenomenon of why block cracking patterns are often observed in the field to be square- or rectangularly-shaped, a model with a large length-to-width ratio (8:1) was produced, the simulation result of which is shown in the Figure 4.12. It was found that a small
degree of predominantly straight, transverse cracks developed in the rectangular-shaped model. This supports the concept that for a pavement with large length-to-width ratio, thermal stress is concentrated in the direction perpendicular to the long dimension of the model, which results in the classic thermal cracking pattern, as presented in Chapter 3. In addition, it was found that if the length-to-width ratio is even more pronounced, only thermal cracking might occur at the beginning of the simulation, while block cracking will not occur until after the model has first segmented into a number of blocks, with similar aspect ratios (squares).

![Figure 4.12 Saturated crack patterns of models of different sizes and aspect ratios.](image)

In order to further study the dimensional effect on block crack patterns with the present 2D DEM model, a model with the same length-to-width ratio but four times as large as the previous square-shaped model was built, with the results presented in Figure 4.12. Similar to what was
observed in the previous simulations, cracks tended to grow in a random direction. Unfortunately, a full block cracking pattern was not realized before the simulation was terminated due to a memory overflow error. That notwithstanding, it is clear that Y-junctions cracks are located throughout the model, indicating the early stages of block crack development. In the following sections, the 0.44m*0.38m model was used to study the effect of additional factors.

4.3.2.2 Relaxation time

The effect of relaxation time on the crack pattern in the model was achieved by setting different equilibrium criteria. The term in the PFC code called ‘Aratio’, literally means ‘average ratio’, which is an equilibrium parameter set in the discrete element modeling code to determine when the equilibrium condition is met (so that the next loading cycle can be started). Aratio is the ratio of the average value of the unbalanced force magnitude (i.e., magnitude of the sum of the contact forces, body forces, and applied forces) over all bodies to the average value of the sum of the magnitudes of the contact forces, body forces and applied forces over all bodies. The model will cycle until the average ratio is less than the assigned value, so models set with a smaller Aratio tends to produce more accurate results, but take longer to reach the equilibrium condition.

In this study, two common Aratio values were selected to study their effect on the saturated crack pattern, with simulation results shown in Figure 4.13. As expected, fewer microcracks were predicted in the model with smaller Aratio. This is because when the Aratio was smaller, longer calculation times allowed the viscoelastic material to relax the accumulated stress. It is important to note that the software assumes a constant temperature when the model is running through its iterative calculations to reach equilibrium. The model with smaller Aratio predicted a perfect Y-junction crack, which also provided evidence that when a microcrack propagates, it tends to propagate along the directions that can even out the angle, if the model and loading conditions are
symmetric. In this study, the main goal was to study the saturated crack patterns with a larger number of cracks developing in a relatively short amount of simulation time, and thus, larger Aratio values were generally taken in order to save on computational time. For the current model, each simulation generally took 2–11 days to complete depending on the complexity of the model, not including post-processing time. Because of the arbitrary, but important effect of Aratio on cracking rate, it can be concluded that the simulations conducted herein were qualitative, aimed at studying block cracking development, geometrical patterns, and relative importance of driving factors. Stated otherwise, the focus was on attaining a better understanding of the mechanisms behind block cracking. In order to arrive at more quantitative results, i.e., determining precise aging, temperature, and loading cycle conditions to produce a given amount of block cracking for a given material (in a given climate), additional model calibration and validation would be required. For instance, simulation of standard asphalt mixture fracture tests, or better yet, thermally restrained tension type tests, could lead to proper calibration of the Aratio parameter and other local fracture parameters to achieve more quantitative predictions.

![Figure 4.13 Saturated crack patterns models at different Aratio (average ratio) values.](image)

4.3.2.3 Cooling rate

When the pavement temperature decreased at different cooling rates, the trend of the
number of microcracks with temperature and the saturated crack patterns of the pavement model was found to vary, at times significantly.

The relationship between the number of cracks with temperature at three different cooling rates is presented in Figure 4.14. It was found that cracks initiated at a warmer temperature when the cooling rate was faster, and the saturated number of microcracks was also higher. As shown in the figure, under a faster cooling rate, the saturated number of cracks was larger. Similar trends were observed by Behnia (2013) in acoustic emission testing, where more critical cracking events were obtained from samples cooled at a faster rate (Behnia, 2013). This is likely because the cooling rate outpaced the speed at which the asphalt binder can relax stress, leading to more rapid tensile stress accumulation, and a higher microcracking rate. As expected, more block cracks were observed when the cooling rate was faster, as more channeling occurred at existing microcrack sites.
Figure 4.14 No. of microcracks versus temperature, and the corresponding saturated crack patterns of models with cooling rates of -0.01°C/step, -0.02°C/step, -0.03°C/step.

Saturated crack patterns of the pavement model at cooling rates of -0.01°C/step, -0.02°C/step, -0.03°C/step are also presented in Figure 4.14. By comparing the crack patterns between figures, different cooling rates led to different cracking patterns. For instance, when the cooling rate was -0.02°C per calculation step, the block crack observed at the top-right region of the figure was more curved than that observed for a cooling rate of -0.03°C/step. A possible reason is that when the cooling rate is slower, there is enough time for the crack at the crack front to ‘seek out’ a path where the contacts possess minimized fracture energy, which resulted as a curved, more tortuous crack. When the cooling rate was faster, multiple microcracks coalesced more rapidly.
such that a dominant, less tortuous (and less curved) macro crack developed.

### 4.3.2.4 Aging

As expected, the aging of the asphalt mixture also played an important role in determining the resulting block crack patterns, producing surprising results at times. The assigned contact strength of mastics with advanced aging was larger than those of the unaged material, and thus fewer microcracks developed. The crack patterns in the ‘aged’ case were formed mainly through the channeling of dominant, highly coalesced macro cracks. Although fewer microcracks initiated at the beginning, the speed of crack propagation in the aged modeling case was faster, such that the number of cracks of this case reached a plateau at a warmer temperature and therefore as a result of less temperature-induced strain energy. In addition, this simulation result indicated that slight aging might deter the initial formation of local microcracks, but once the cracks initiated, they propagated rapidly, which agreed with the field observation. This is likely because the assumed fracture properties possessed higher local material strength, but also increased post-peak softening slope (brittleness). Thus, higher local force was required to initiate particle separation, but once the crack began to form, the brittleness led to a fast macro crack propagation rate.

![Figure 4.15 Saturated temperature and crack pattern of models with or without long-term aging.](image-url)
4.3.2.5 Pre-existing crack

In the field, hexagonally-shaped crack patterns are primarily found in the pavements where there is no obvious existing bias towards transverse or longitudinal cracking. To study this phenomenon, a linear crack was inserted by assigning very small contact strength (20N) to the contacts highlighted by a red line in Figure 4.16, so as to simulate the case such as material segregation owing to construction (paver gear box segregation, or other linear segregation or built-in linear inhomogeneities (such as roller marks)), pre-existing transverse cracking and so on. It was found that the existing crack would release most of the applied thermal loading, so that no additional cracks were developed in the vicinity of the crack. The stress was primarily concentrated at the crack tip and cracks channeled outward from the end of the pre-existing crack, eventually interacting with model boundaries. The simulated results agreed with the field observation that rectangular-shaped crack patterns tend to form when existing forms of orthogonal (transverse or longitudinal) cracks exist (Figure 4.17).

![Figure 4.16 Crack pattern of models at -15.6°C with or without a pre-existing crack.](image)
4.3.3 Crack Depth

To simulate how cracking propagates along the thickness direction, a pavement cross-section was modeled. Figure 4.18 shows the simulation results of the model with the control PG 64-22 mixture under uniform thermal straining (uniPROP_uniTEMP) and Figure 4.19 shows the simulation results of model with the aged PG64-22 mixture under graded thermal straining (gradPROP_gradTEMP). As expected, debonding happened in both cases because of the difference in thermal contraction coefficient of the asphalt overlay and the underlying concrete base. In the case uniPROP_uniTEMP, microcracks developed randomly in the model, and then followed by crack channeling. Both top-down and bottom-up cracking were found, as a result of relatively uniform material and temperature distribution. In the case gradPROP_gradTEMP, microcracks started developing in the pavement surface and then propagated downward to the bottom. Some of the cracks stopped at the top which formed partial-depth cracks, where the crack
depth was about $1/6$–$1/4$ of the pavement thickness. However, two macrocracks were observed as forming full-depth cracks. Afterwards, these two cracks then propagated along the interface which resulted in further debonding. This simulation results explained why the block cracking has been mainly found to be a partial-depth cracking form (Buttlar et al., 2010), and sometimes mixed with thermal cracking.

**Figure 4.18** Full simulation results of the model of pavement cross section made of PG 64-22 mixture subjected to uniform temperature change.
Figure 4.18 cont.

<table>
<thead>
<tr>
<th>Temperature</th>
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<tbody>
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<tr>
<td>-13.2°C</td>
<td>722</td>
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</tbody>
</table>

Figure 4.19 Full simulation results of the model of pavement cross section made of aged PG 64-22 mixture subjected to graded temperature change.
4.4 Summary

In this Chapter, a two-dimensional microstructure-based model was built to study block cracking patterns in asphalt pavement. The formation mechanism of block cracking was extensively studied, and the effects of the model size, relaxation time of asphalt materials, aging and pre-existing crack effect on block cracking pattern was investigated. The following
conclusions were drawn according to the presented simulation results:

(1) Block cracking patterns occur in pavement segments that are symmetric or infinitely large as compared to crack size, and when other types of cracks (i.e. longitudinal or transverse cracks) are not dominate; whereas transverse thermal cracking patterns tend to occur in a pavement segment with a large, plan-view aspect ratio;

(2) The formation process of block cracking can be separated into two steps: the initiation of microcracks, followed by the coalescence of microcracks and channeling of macro cracks, the path of which is mainly determined by the fracture properties of the local materials;

(3) When microcracks propagate in a symmetric or infinitely large pavement segment, they tend to develop in arbitrary directions to form a Y-junction pattern. When growing cracks intersect, they tend to join at right angles;

(4) The dimension of the pavement segment, relaxation behavior and aging susceptibility of asphalt materials and cooling rate are all factors contributing to block cracking patterns. Because the path of cracks is highly influenced by local material properties, no unique crack pattern was found in simulations where a degree of random property assignment was made;

(5) Due to the effect of material brittleness and the larger temperature change at the pavement surface, block cracking tends to occur predominantly within the top several centimeters of the pavement surface. This has clear implications on preventive and rehabilitative maintenance, which will be discussed later in this thesis.
CHAPTER 5: 3D MICROMECHANICAL PAVEMENT MODEL DEVELOPMENT FOR THE STUDY OF BLOCK CRACKING

5.1 Introduction

Asphalt paving mixtures are viscoelastic and heterogeneous materials with time- and loading rate- dependency. The formation of block cracking is assumed to be separated into two processes: 1) initiation of macro-sized cracks due to insufficient stress relaxation capacity and/or fracture resistance of the material, and; 2) channeling of cracks, the path of which is highly dominated by local material properties. Considering such complex behavior, microstructure-based discrete element modeling is one of the few modeling tools that is capable of considering viscoelastic and fracture properties in the simulation of thousands of randomly-oriented, coalescing and intersecting cracks in a layered structure. As mentioned earlier in the thesis, Particle Flow Code, or more specifically PFC-3D, is a 3D microstructure-based discrete element model developed by Cundall that has been shown to capture such behavior (Cundall, 1971; Kim et al., 2008; Kim et al., 2008; Kim et al., 2009; Hill, 2016; Hill et al. 2016).

A typical DEM fracture analysis of asphalt mixtures requires input parameters defining bulk material behavior as well as localized fracture properties. According to Kim et al. (2009) and Hill et al. (2016), creep compliance testing such as AASHTO T-322 (2004) is sufficient to characterize bulk and local material linear viscoelastic properties at low temperatures. However, the determination of local fracture properties from standard fracture energy tests is more complicated due to viscoelastic energy dissipation (Kim et al., 2005). Ideally, modeling or correction factors should be employed to convert the total fracture energy measured in standard tests such as the disk-shaped compact tension test (ASTM D7313, 2007), or DC(T), into local fracture energy by isolating the work of fracture. This process is completed by subtracting out the
creep dissipation contribution to CMOD gage displacement in the tests. The creep dissipation accounts for 5 to 50% of the measured total fracture energy in most cases and depends on material and test temperature (Braham, 2008; Song et al., 2006). Applying these principles, Kim et al. simulated experimental load-CMOD curves to extract calibrated, local fracture parameters, through iterative, inverse analysis techniques (guess-and-check). This included local fracture energy, maximum cohesive force and maximum contact displacement (Kim et al., 2008; Kim et al., 2009) for the mastic and for aggregate-mastic interfaces. Hill et al. (2016) provided a more rigorous nonlinear optimization approach to inversely obtain local fracture parameters, by minimizing the differences between measured displacement fields obtained from digital image correlation and simulated displacement fields obtained from DEM.

The primary objective of this chapter was to develop a three-dimensional (3D) viscoelastic, inhomogeneous and anisotropic microstructure-based pavement model to examine the effects of aging and cooling rate on block cracking phenomena (extent and depth) in asphalt pavement. Cracking mechanisms behind the development and propagation of hexagonal-shaped block cracking were then discussed, followed by a discussion of the practical implications of the results and recommendations for future research and applications by practitioners to mitigate block cracking.

5.2 Three-dimensional Microstructure Model

Asphalt pavement layers are viscoelastic, heterogeneous combinations of asphalt binder, aggregates, and oftentimes other liquid, solid, and even fibrous additives. In this study, it is assumed that the primary factors underlying top-down, non-load associated cracks can be effectively simulated using a 3D discrete element model with a viscoelastic constitutive model, thermal contact model and inhomogeneous material morphology representation. The
inhomogeneous material morphology was achieved by adding air voids to the homogeneous model instead of individual mastic and aggregate elements. This simplifying assumption was made in this initial modeling phase in order to avoid the computational cost of a heterogeneous mastic and aggregate geometry. For example, it generally takes about 20 to 30 days to complete a full block cracking simulation of a three-dimensional viscoelastic material system with inhomogeneity subjected to thermal loading. This length of time includes: model assembly, material properties assignment, simulation processing in PFC software, and post-processing via MATLAB and AutoCAD.

5.2.1 Model Establishment Approach

![Figure 5.1](image.png)

**Figure 5.1.** A representative pavement model in discrete element modeling, along with packing arrangement, thermal contact and viscoelastic cohesive-softening contact model.

A representative pavement model and schematic diagrams of the utilized thermal and
viscoelastic cohesive-softening contact model are provided in Figure 5.1. The thickness of the pavement model was selected to be 0.078m (approximately 3in), which is within the typical range of thickness of the typical asphalt overlays. By utilizing the analytical solutions presented in Chapter 3, it was found that the major block cracks (larger blocks with significantly wider cracks) would only occur when the lateral pavement extent was longer than ten times the given thickness. Additionally, the dimension of the pavement model needed to be approximately square-shaped to obtain a block cracking pattern. In this chapter, the model was desired to study both thermal and block cracking of asphalt pavement, so a rectangular-shaped pavement model was used instead of a perfect square-shaped pavement model. As a result, the dimension of the pavement model was selected to be 2.00m*0.86m*0.078m. The radius of particles was initially chosen to approximately 0.6mm based on the suggestion by Kim et al. (2008) in his simulation of fracture in the DC(T) testing arrangement (approximately 150mm in diameter). As such, the approximate number of particles required to build a 2.00m*0.86m*0.078m model was twelve million, which far exceeded the computational capacity of a personal desktop. However, the Kim et al. (2008) results demonstrated that the overall trend of applied load versus crack mouth opening displacement in a DC(T) fracture simulation comprised of larger particles was reasonably close to that of the 0.6mm method. Therefore, particle radius was selected to be 5mm in this study, which is approximately the maximum diameter of particles found in fine aggregate stockpiles in Illinois, and one-to-three sieve sizes finer than the typical coarse aggregate stockpile maximum particle sizes. This particle size was still capable of capturing the crack propagation trends in an asphalt pavement, and at the same time served to reduce the computational cost tremendously. A 2.00m*0.86m*0.078m pavement model with 186,489 elements and 983,264 contacts was constructed to simulate an asphalt overlay pavement system. The asphalt overlay was assumed to be fully bonded with the
underlying layer, i.e. the boundary condition of elements at the bottom of the model was restrained in all directions.

5.2.2 Materials

A typical asphalt mixture in Illinois was utilized in this study. The mixture was composed of PG64-22 binder and a 9.5mm nominal maximum aggregate size Superpave aggregate blend without any additives or recycled materials. A seven percent air voids condition (post-paving) was selected because it is consistent with most density criteria set by state departments of transportation. This air void level was achieved in the model by randomly deleting seven percent of the elements in a fully populated rectangular mesh, i.e., a 3D, close-packed tetrahedral arrangement. The relaxation behavior and low-temperature fracture properties of the mixture were provided by Hill et al. (2016). The data was obtained from IDT creep compliance and DC(T) test in accordance with AASHTO T-322 and ASTM D7313, respectively (AASHTO T-322, 2004; ASTM D7313, 2007). Relaxation modulus parameters were obtained through interconversion of the creep compliance master curve at a reference temperature of -12°C, as shown in Figure 5.2. Fracture constitutive laws, i.e., load-displacement and fracture behavior in the connections between discrete elements, were input to the model based on DC(T) fracture energy test results (Figure 5.3). It is known that the numerical material fracture energy will be overestimated if the experimental fracture energy value of the mixture is directly used in the simulation (Wagoner et al., 2005). As a result, Kim et al. suggested the use of a fracture multiplier of 0.74 to calibrate the local fracture energy value in the homogeneous model without requiring the calibration of material strength and viscoelastic properties (Kim et al., 2009b). Therefore, with the consideration of the differences of material behavior at the macro and micro level, both the creep compliance and fracture properties were converted to local viscoelastic and fracture properties accordingly and
used as material property inputs in the discrete element model, as shown in Table 5.1 (Kim et al., 2005).

![Figure 5.2 Creep compliance of PG 64-22 asphalt mixture at -12°C (Hill, et al., 2016).](image1)

![Figure 5.3 DC(T) load versus CMOD of PG 64-22 asphalt mixture at -12°C (Hill, et al., 2016).](image2)
Table 5.1 Viscoelastic cohesive-softening model parameters

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Softening model parameters (post-peak portion)

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To simulate the viscoelastic and fracture behavior of the asphalt mixture in DEM, a viscoelastic cohesive-softening contact model was utilized. The material contact model used was developed and verified by Hill et al. (2016). This model combined a viscoelastic Burger’s model with a linear softening model to describe both viscoelastic (bulk) and local separation behavior of asphalt material as shown in Figure 5.1. Details about this model were provided in Chapter 2.

A vertically-graded material fabric was used to capture the effects of asphalt aging. Aging effects are most severe on the surface, and exponentially diminish with depth from the surface (Dave, 2009). Braham et al. (2009) demonstrated the change in fracture behavior with material oxidative aging. As such, aging created a vertical gradient in fracture resistance with time in addition to a viscoelastic stiffness gradient. These gradients in material properties of a given pavement structure arise mainly because of two reasons: oxidative aging and temperature change with depth. Lytton et al. (2018) developed a kinetics-based modeling of long-term field aging in asphalt pavement that considered both of the aforementioned factors regarding the aging process in asphalt materials and its effect on top-down cracking in asphalt pavement layers. Buttlar et al. (2006) illustrated that the dynamic modulus of a particular asphalt material used on Interstate I-155 at the pavement surface was about two times more than that of the material at 100mm in depth after eight service years. This prediction was based on the global aging system of Mirza and Witczak (1995) which considered aging and temperature gradients.
To simulate block crack formation due to severe aging at the late stage of service life of an asphalt surface layer, vertically-graded material assignment was made in the discrete element model using a parametric study approach. The relaxation modulus at the top of the surface layer was assumed to be four times higher than that of the bottom, following an exponential grading distribution. In this study, a total of two different aging levels were considered—short-term aging and long-term aging. Short-term aging referred to uniform material viscoelasticity experienced with 2 hours of aging at the mixing temperature. Long-term aging referred to the aforementioned graded material viscoelasticity in a more extreme aging condition. To simulate these stiffness gradients, a relaxation modulus multiplier was introduced following previously reported data (Apeagyei, 2006; Braham et al., 2009; Buttlar et al., 2006) as presented in Figure 5.4.

![Relaxation modulus multiplier of asphalt mixtures at the surface layer of asphalt pavement under short-term aging condition (1:1) and extreme aging condition (1:5).](image)

**Figure 5.4.** Relaxation modulus multiplier of asphalt mixtures at the surface layer of asphalt pavement under short-term aging condition (1:1) and extreme aging condition (1:5).

Aged asphalt mixture becomes brittle at low temperatures and at advanced aging levels, both of which tend to reduce mixture fracture energy. Apeagyei indicated that the peak load measured in a fracture test of an asphalt mixture was about 1.5 times greater than that of an unaged asphalt mixture. Furthermore, Apeagyei showed that a 25% reduction in fracture energy would
result after approximately 6 years of field aging (Apeagyei, 2006). Therefore, an increase of 50% in local material strength and a reduction of 25% in fracture energy were assumed for the extreme aging condition in the low temperature simulations.

In the viscoelastic cohesive-softening contact model, two independent fracture parameters are required out-of-the-three possible choices - tensile strength (maximum contact force), maximum displacement and fracture energy. Chapter 3 showed that the DC(T) test was sufficient to obtain all the required model fracture parameters because tensile strength can be derived from DC(T) peak load. In this study, tensile strength and fracture energy were extracted from test results, and the maximum displacement in the local fracture model was then calculated. The tensile strength and fracture energy values at different pavement depths used in the model are presented in Table 5.2.

**Table 5.2.** Fracture properties of Asphalt Mixtures in viscoelastic cohesive-softening contact model at different pavement depths for the Short-term Aging Condition (1:1) and Extreme Aging Condition (1:0.75)

<table>
<thead>
<tr>
<th>Distant from the bottom (m)</th>
<th>1:1 Fracture Energy (J/m²)</th>
<th>Max. Contact Force (N)</th>
<th>Max. Displacement (m)</th>
<th>1:0.75 Fracture Energy (J/m²)</th>
<th>Max. Contact Force (N)</th>
<th>Max. Displacement (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00E+00</td>
<td>287.5</td>
<td>247.6</td>
<td>9.1E-05</td>
<td>287.5</td>
<td>247.6</td>
<td>9.1E-05</td>
</tr>
<tr>
<td>8.16E-03</td>
<td></td>
<td></td>
<td></td>
<td>287.5</td>
<td>247.6</td>
<td>9.1E-05</td>
</tr>
<tr>
<td>1.63E-02</td>
<td></td>
<td></td>
<td></td>
<td>285.5</td>
<td>251.3</td>
<td>8.9E-05</td>
</tr>
<tr>
<td>2.45E-02</td>
<td></td>
<td></td>
<td></td>
<td>283.2</td>
<td>257.5</td>
<td>8.6E-05</td>
</tr>
<tr>
<td>3.27E-02</td>
<td></td>
<td></td>
<td></td>
<td>280.3</td>
<td>267.4</td>
<td>8.2E-05</td>
</tr>
<tr>
<td>4.08E-02</td>
<td></td>
<td></td>
<td></td>
<td>274.6</td>
<td>279.8</td>
<td>7.7E-05</td>
</tr>
<tr>
<td>4.90E-02</td>
<td></td>
<td></td>
<td></td>
<td>266.0</td>
<td>297.1</td>
<td>7.0E-05</td>
</tr>
<tr>
<td>5.72E-02</td>
<td></td>
<td></td>
<td></td>
<td>253.0</td>
<td>316.9</td>
<td>6.3E-05</td>
</tr>
<tr>
<td>6.53E-02</td>
<td></td>
<td></td>
<td></td>
<td>235.8</td>
<td>341.7</td>
<td>5.4E-05</td>
</tr>
<tr>
<td>7.35E-02</td>
<td></td>
<td></td>
<td></td>
<td>215.6</td>
<td>371.4</td>
<td>4.6E-05</td>
</tr>
</tbody>
</table>
5.2.3 *Thermal Properties*

Aside from the vertical material viscoelasticity gradient, the temperature gradient with respect to depth from the surface is a main factor contributing to the rate of block cracking in an asphalt pavement. According to typical temperature profiles, the temperature change per hour is approximately \(-0.5^\circ\text{C}/\text{h}\) in a mild (coastal) region and \(-2^\circ\text{C}/\text{h}\) in cold regions with high temperature gradients (mid-contintental regions). Accordingly, a temperature change of \(-0.5^\circ\text{C}/\text{h}\) was taken for the mild climate, and \(-2^\circ\text{C}/\text{h}\) was used for the more severe climate. The temperature change at the top of the pavement surface layer was assumed to be three times faster than that of the bottom and followed an exponential distribution, as shown in Figure 5.5. The coefficient of thermal contraction of asphalt mixture was selected as \(7.42\times10^{-5}/^\circ\text{C}\) (Dave, 2009) in the present parametric study based on previous literature.

A built-in thermal feature in PFC software was adopted to simulate temperature change in DEM. The thermal contact model was developed by Li *et al.* (2016) to simulate heat conduction. This model allowed thermally-induced stresses to develop among particles in the pavement model by changing the radius of each particle; additional details were provided in Chapter 2.

The simulation was initiated at \(-12^\circ\text{C}\) to save computational time as previous simulations had shown that material relaxation did not result in significant stress build up prior to this temperature. The temperature was modeled to decrease following the assigned cooling rate, and crack activity as predicted by the simulation was assessed every 0.01°C.
5.3 Comparison of the 3D DEM model with the Present 3D Analytical Model

To verify the DEM model, the 3D simulation results were compared to the analytical solution presented earlier. Two material contact models were used in the DEM model: linear bond contact model to simulate elastic behavior of material, and viscoelastic cohesive-softening contact model to simulate viscoelastic and softening behavior of materials. The sizes and loading condition of three models were the same. The boundary condition of the analytical model and the DEM model were different; the analytical model was assumed to have a frictional interface achieved by the use of shear springs, whereas the DEM model was assumed to have a fully bonded interface (bottom elements pinned along the underside of the asphalt layer in x, y, and z directions). Due to the elastic assumption of the analytical model and the design to conduct an elastic DEM simulation in the verification phase, the instantaneous relaxation modulus from the mixture master curve was taken as the elastic modulus input, which was obtained through interconversion of the creep compliance master curve at a reference temperature of -12°C. A summary of the information and assumptions of each model is listed in Table 5.4, a schematic diagram of the analytical model and the DEM model are presented in Figure 5.6.
Table 5.3 Summary of information and assumptions of the analytical model, elastic DEM model and viscoelastic DEM model

<table>
<thead>
<tr>
<th>Assumptions</th>
<th>Analytical</th>
<th>DEM_elastic</th>
<th>DEM_viscoelastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimension (mm)</td>
<td>400<em>84</em>26</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Material</td>
<td>PG 64-22 asphalt mixture</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assumptions</td>
<td>Material</td>
<td>Homogeneous, isotropic</td>
<td>Elastic</td>
</tr>
<tr>
<td>Boundary Condition</td>
<td>Frictional interface</td>
<td></td>
<td>Fully-bonded interface</td>
</tr>
<tr>
<td>Loading</td>
<td>Uniform thermal loading</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 5.6 (a) Schematic diagram of analytical model; (b) DEM model.

The stress fields, along the x-axis from the center to the edge at the top of the asphalt layer, of three models are plotted in Figure 5.7. The horizontal stress in each model was maximum at the center of the modeled asphalt pavement segment, and was expected to diminish to approximately zero at the free edge (edge of pavement segment under investigation). Differing from the other two cases, the stress at the edge of the viscoelastic DEM model did not reach zero when the model reached the assigned equilibrium condition. This might be because the stress saved in the isolated dashpot in Burger’s model was not fully relaxed by the time at which the model reached the equilibrium condition. The maximum stress of both elastic solutions was very close, the overall DEM elastic results were larger than that of analytical solution due to the stricter interface condition utilized in the DEM model. The overall viscoelastic results yielded smaller stresses as compared to the other cases, due to the stress relaxation capacity of the viscoelastic material. Because of the differences in the boundary conditions and materials property inputs, the verification study was concluded, as the overall trends in stress versus position in the pavement
were observed. In addition, the goal of the study was to make qualitative predictions of block cracking behavior, rather than quantitative estimates.

![Figure 5.7 Results comparison between 3D analytical model, 3D elastic DEM model, 3D viscoelastic DEM model.](image)

### 5.4 Simulation Results and Discussion

#### 5.4.1 Crack Extent

Four combinations of material aging extent and temperature gradient were simulated: newly paved pavement in a mild climatic region, aged pavement in a mild climatic region, newly paved pavement in a severe climatic region and aged pavement in a severe climate region. These cases were denoted as Uni. PROP & Uni. TEMP, Grad. PROP & Uni. TEMP, Uni. PROP & Grad. TEMP and Grad. PROP & Grad. TEMP, respectively.

The number of predicted microcracks versus temperature in each case is presented in Figure 5.8. If a pavement composed of the investigated asphalt mixture was placed in a mild region, no cracks were initiated at the beginning of the simulation (case Uni. PROP & Uni. TEMP). However, in the extreme aging condition (later in pavement life), microcracks were predicted to
form near the onset of the simulation and reached a plateau rapidly (case Grad. PROP& Uni. TEMP). It was found that predicted cracks were primarily contained in the pavement surface due to the higher material brittleness at the surface. Thus, a relatively smaller total number of microcracks throughout the volume were computed. If the same pavement was placed in a cold region, crack activity was minimal at first, followed by gradual growth, and finally, a plateau was reached (case Uni. PROP& Grad. TEMP). This simulation, as compared to the previous case predicted that cracks were located both in the pavement surface and deeper below the surface because of the smaller temperature gradient. In the trial where the material property was simulated to have severe oxidative aging and placed in a severe climate, cracks developed very rapidly, channeled throughout the surface, and then propagated downwards (case Grad. PROP& Grad. TEMP). The simulation was terminated before the plateau was reached, due to a memory overflow problem. However, the results suggested a highly saturated crack pattern (extensive microdamage) would exist in the surface. In all simulation cases, after the number of cracks reached a plateau, crack widths continued to grow. Clearly, material and temperature gradients play an important role in block cracking behavior. This suggests that avoiding severe aging gradients is important, especially in climates with rapid cooling cycles.
The location and extent of predicted microcracks was obtained from PFC and then plotted in AutoCAD. An example of the crack propagation process and final crack pattern is presented in Figure 5.9. When temperatures dropped, isolated microcracks started to initiate within the pavement surface when the minimum required surface energy to create a new crack surface exceeded the material fracture energy. The first microcrack was initiated at -12.15°C. Next, isolated cracks began to channel across the pavement surface and a denser block crack pattern became visible at -12.28°C. As more channeling occurred, cracks became interconnected and formed smaller, block-shaped crack patterns. Finally, the surface of the pavement was predicted to contain
a highly saturated crack pattern starting at -12.43°C. Crack saturation near the surface was reached in a very narrow temperature range in the simulation, indicating the severe microdamage formation potential after a given temperature threshold was reached. A similar phenomenon was observed by Behnia (Behnia, 2013), indicating that the primary, large microcracks, or higher energy events, occurred within temperature range of approximately 2–5°C depending on the material tested. This microcracking process was termed as a “transition zone”, and it can be demonstrated that the temperature range of the transition zone would be smaller if the material experienced higher degrees of aging or if it was cooled at a faster rate. In particular, the embrittlement temperature of the utilized virgin PG64-22 asphalt mixture was determined to be -17.77°C with a transition zone of approximately 1–3°C. Considering that the material used in the simulation was aged, the embrittlement temperature and temperature range of the transition zone of the aged material was expected to be higher and smaller, correspondingly, as predicted by the simulation. Details about the acoustic emission results of the utilized virgin PG 64-22 asphalt mixture was reported by Hill et al. (2013). In addition, the narrower temperature range of the simulation result was also likely due to the fact that homogeneous mastic properties existed at a given depth in the pavement (random assignment was not used in the 3D modeling). However, it was unlikely that these microcracks would be visible or lead to major pavement distress. Consequently, they may not permit moisture ingress and may have the potential to be healed during heating cycles. The majority of the initial block cracking pattern was predicted to reside in the middle of the pavement (from a plan view perspective). Being away from free edges, material restraint was highest in this location. Conversely, along the longer boundary edge, more transversely-aligned cracks were predicted by the simulation. These two distinct cracking patterns have been observed in the field. This was evidenced by comparing the top region of Figure 2.1b to the center of the paving lane in
Figure 2.1b, or the left region of Figure 2.1c to the center of the lane in Figure 2.1c.

**Figure 5.9** Microcrack initiation and propagation process for an aged pavement model assumed to be placed in a mild climatic region.
5.4.2 Crack Depth

Cracks propagated downward to the bottom of the surface layer as temperature in the simulation continued to decrease. Figure 5.10 plotted the crack penetration depth versus temperature for three simulation cases. The case of Uni. PROP_Uni. TEMP was not included because very little cracking was predicted in this case. This suggests that, in the case of the mixture investigated, preservation of the initial asphalt properties or use of the mixture in a mild climate would likely lead to minimal block crack formation. As expected, when the pavement experiences a high aging gradient, cracks formed predominantly in the pavement surface. Thus, although crack channeling around the surface occurred rapidly, downward propagation was deterred by material toughness and lower thermal straining at lower pavement depths (Grad. PROP_Uni. TEMP). When placing the same mixture in a mild region, cracks suddenly initiated when a critical temperature condition was reached. Next, cracks propagated downward gradually as the temperature continued to decrease (Uni. PROP_Grad. TEMP). Depending on the local stress state around the crack tip, cracks either preferred to channel around in the surface or downward toward to the bottom. Finally, if materials were highly aged and used in a severe climate, cracks were found to propagate deeper and faster than the other two cases (Grad. PROP_Grad. TEMP).
5.5 Application of Results to the Field Practice

This research study demonstrated that once microcracks initiated, they channeled through the pavement surface rapidly, and interconnected with each other to form block cracks. The majority of microcracks predicted in the simulation were located near the pavement surface. This was due to lower pavement temperatures, larger temperature variations, and higher material stiffness and brittleness at the surface. This finding agreed with the field observation that block cracking has mainly been found to be a partial-depth cracking form (Buttlar, et al., 2010). In the case of this simulation, the predicted block cracks were categorized as hexagonally-shaped. This agreed with the aforementioned field observation of hexagonally-shaped block cracks that are typically observed in asphalt pavements where other forms of cracking are less dominant, i.e. in the absence of other transverse or longitudinal cracking patterns. This phenomenon was shown in
Figure 2.1b with the initial cracking pattern exhibited on the surface of a full-depth asphalt pavement. This pavement did not demonstrate reflective cracking potential, was constructed without noticeable longitudinal paver segregation, and did not develop thermal cracking prior to block crack development. The field observations, in conjunction with the insight provided in this simulation study, appeared to show that the pavement was comprised of thermal and block cracking resistant asphalt at the time of construction. However, the surface mixture lost block cracking resistance after approximately 8 years of field service. These conditions, combined with the lack of transverse longitudinal cracks in this low traffic volume frontage road to I-74 in Champaign, Illinois, led to the onset of randomly-oriented, hexagonal block cracks.

5.6 Simulation-informed Strategies for the Prevention of Block Cracking

Numerous forms of asphalt pavement cracking exist, creating significant damage to pavement structures and requiring millions of dollars in rehabilitation and repair every year. For example, flexible pavement maintenance and repair costs in New York State alone topped $425 million in 2008 (Gee and Paterson, 2008). Block cracks, when they form, generally propagate over a large portion of the asphalt pavement surface area, which accelerates pavement deterioration by allowing moisture to penetrate into a wide extent of the pavement system. When combined with moisture damage and/or raveling, block cracking can progress into even more serious deterioration patterns such as potholes. Pavements experiencing block cracking have been shown to incur increased maintenance and rehabilitation costs (Brown et al., 2009).

Inspired by the simulations results, strategies need to be taken for block cracking prevention. These would include: accounting for aging and block cracking during design, evaluating existing structures, and developing preventive and rehabilitative methods to deter block crack development.
5.6.1 Pavement design

According to the simulation results, the initiation temperature of block cracking was predicted to be higher than that of thermal cracking. To confirm this finding, ILLI-TC model, a powerful tool for thermal cracking prediction, was used to simulate thermal cracking tendencies of the presented simulated cases. In this model, cracking severity is expressed as the number of critical events and length of transverse/thermal cracking per 500m of pavement (if cracked). The bulk viscoelastic and fracture properties of the four simulated cases were inputted to the ILLI-TC model, and the results are provided in Table 5.4. ILLI-TC conducts a full simulation of temperature cycles for the fall, winter, and spring months, including heating and cooling cycles. It was found that only the case gradPROP_gradTEMP was predicted to have thermal cracking issues, and the cases uniPROP_gradTEMP and gradPROP_uniTEMP were not predicted to have thermal cracking potential. Although ILLI-TC uses simulated temperature gradients predicted by the Integrated Climate Model (Larson and Dempsey, 1997) while the block cracking simulations used an assumed cooling rate. The results nevertheless provided additional evidence that block cracking likely occurs at a warmer temperature relative to the onset of thermal cracking. Taking this into account the current simple performance test criteria for thermal cracking mitigation might not be sufficient to allow for the design of block crack resistant pavements, especially at long service life age and in climates with severe aging potential. Further research is needed to identify the best approach to delineate mixtures that are most prone to aging and subsequent block cracking.
Table 5.4 Thermal cracking potential of the simulated cases as predicted by ILLI-TC

<table>
<thead>
<tr>
<th>Input</th>
<th>Output</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fracture Energy (J/m²)</td>
</tr>
<tr>
<td>UniPROP_uniTEMP</td>
<td>389</td>
</tr>
<tr>
<td>UniPROP_gradTEMP</td>
<td>291</td>
</tr>
<tr>
<td>gradPROP_uniTEMP</td>
<td></td>
</tr>
<tr>
<td>gradPROP_gradTEMP</td>
<td></td>
</tr>
</tbody>
</table>

5.6.2 Existing structure evaluation

The widths of block cracks simulated in this study were small and could potentially be healed during warm temperature cycles. However, with additional temperature cycling, material aging and overall degradation, crack widths in the field are observed increase and therefore healing ability diminishes. As such, block cracks likely form undetected and continue to grow in size and depth until being observed by maintenance crews and engineers. Experience has shown that early block cracking is most easily perceived immediately after a rainfall event followed by partial drying of the surface, and then disappears when the pavement surface dries completely. A challenge with this behavior from a maintenance standpoint is that pavement damage due to block cracking generally accelerates once it becomes easily observable in dry conditions. It was also reported that the amount and severity of block cracking are generally underestimated by automated distress techniques (Buttlar et al., 2018), i.e. PASER, because they might either be undetected or improperly categorized as transverse and/or longitudinal cracks. Therefore, better techniques for early block crack detection are needed in order to better diagnose and treat pavements as they become susceptible to block cracking.

For example, the non-linear ultrasonic technique can be effectively used to detect subsurface micro cracks in field cores, and acoustic emission source location techniques are capable of detecting location and depth of thermal and block cracks (Behnia et al., 2018,
McGovern et al., 2017, Sun et al., 2015). Furthermore, a fracture energy threshold for fracture testing of a field core that accounts for aging effect is also preferable, in order to identify the fracture resistance of the existing pavement. However, more work in the application of non-destructive testing techniques to detect the early onset of block cracking is needed.

5.6.3 Preventive and rehabilitative methods to deter block crack development

Building on a previous concept, irreversible viscoelastic effects in the cracked structure, along with the effects of oxidation, volatilization, low-temperature physical hardening and volume loss, raveling and stripping at crack faces – can all be contributors to increased block crack density and crack width with time. All of these effects could be mitigated with improved initial asphalt mixture properties, increased density at the time of construction and through surface maintenance. It should be noted that some of the mechanisms only occur after macro cracks form, and thus, proactive maintenance prior to the occurrence of macro cracking may yield significant life extension. This strategy was experimentally validated by Paulino et al. (2006).

Appropriate treatment for early-stage block cracking is very important to deter block cracking formation. For instance, fog seal and other rejuvenation techniques might be suitable treatments for asphalt pavements to restore flexibility for an existing asphalt pavement and improve its fracture resistant to block cracking. Fog seals are only feasible in situations where skid resistance is not an issue – such as parking lots and low speed roadways. In other cases, slurry seals, micosurfacings, and even thin-bonded overlays. The latter include spray-paver applied systems, which have excellent bond, which is needed to resist high interface shear forces present in the thin overlay (Ahmed, S., 2011). If the existing block cracking propagates downward and imparts partial-depth cracking, thin mill and fill might be a cost-effective maintenance method to address block cracking issues. According to the simulation results, as much as the upper 1 in of the
top layer might need to be milled depending on the severity of block cracking experienced.

Figure 5.11. Preventive and rehabilitative methods to deter block cracking development: (a) Fog seal; (b) Thin mill-and-fill (Kent County Road Commission, 2016).

In summary, by combining these tools with the simulation techniques presented herein, block cracking can be controlled through: (1) better initial design of modern, recycled asphalt surfaces, preferably using effective cracking performance tests, plus; (2) monitoring and treatment of pavement surfaces to avoid embrittlement threshold states that can lead to rapid crack channeling in the pavement surface. Properly reported pavement management measurements, which clearly and accurately delineate between block, thermal, longitudinal (wheel path fatigue), and reflective cracking are also needed to advance this field of practice.

5.7 Summary

In this chapter, a 3D viscoelastic and inhomogeneous microstructure-based pavement model subjected to stepwise thermal stresses was presented for the study of block cracking. A typical PG 64-22 Illinois asphalt mixture was adopted as the baseline material which typically exhibits block cracking towards the end of its service life. Effects of aging and cooling rate were studied to investigate block cracking extent and depth. Based on the findings of the study, the following conclusions were drawn:
Due to the vertically-graded material system caused by severe near-surface aging, block cracking formed near the pavement surface at a temperature significantly warmer than the thermal cracking threshold, and saturated rapidly;

Block cracking formation was separated into two steps: (1) initiation of microcracks as the result of insufficient stress relaxation capacity and/or fracture resistance of the material, and; (2) channeling of cracks in the pavement surface, the path of which was highly dominated by local material properties and boundary conditions, and;

Based on simulation results, block cracking appeared to be a partial-depth phenomenon for most overlays, due to the extreme gradient (from brittle to ductile) caused by the combined existence of material and temperature gradients.
CHAPTER 6: SUMMARY, CONCLUDING REMARKS AND FUTURE EXTENSIONS

6.1 Summary

This dissertation aimed to study the formation mechanism of the block cracking phenomena in asphalt pavements through analytical and discrete element modeling methods. The main goals of the study were: to develop a three-dimensional pavement model to obtain the stress and displacement field equations which can roughly estimate block crack size of a given pavement structure; to develop a two-dimensional heterogeneous and anisotropic micromechanical models to study crack initiation and propagation behavior underlying block cracking development, and; to develop a three-dimensional homogeneous and anisotropic micromechanical model to further study cracking mechanisms associated with the block cracking phenomenon.

The key findings of the study included:

- According to the normal stress distribution from the analytical solution, the normal (tensile) stresses in the asphalt layer are maximum at the center of the uncracked block segments, and decrease gradually to approximately zero at the edge;

- The analytical solutions correlated well with the FEM simulation results;

- The analytical model provided a reasonable estimate for the major rectangular block cracking pattern observed on a field validation section in La Plata, Missouri;

- When there was a pre-existing crack in the model, no additional cracks were developed normal to and in the vicinity of the crack;

- When microcracks propagated in a symmetric model subjected to symmetric thermal loading, multiple Y-junction cracks were observed;

- The dimension of the pavement segment, relaxation behavior and aging susceptibility of
asphalt materials and cooling rate were all factors found to influence the block cracking pattern;

- Because the path of cracks is highly dominated by the local material properties, no unique crack pattern was found in any simulation;

- Due to the vertically-graded material system caused by severe near-surface aging, block cracking forms near the pavement surface at a temperature significantly warmer than the thermal cracking threshold, and saturates rapidly;

- In the 3D simulation, very little cracking was predicted in the case of Uni. PROP_Uni. TEMP, suggesting that the key to preventing block cracking in mild climates is through proper initial material selection, and;

- In the 3D simulation cases of Grad. PROP_Uni. TEMP and Uni. PROP_Grad. TEMP, the downward propagation of cracks was deterred by material toughness and lower thermal straining at lower pavement depths.

### 6.2 Conclusions

The following conclusion are made based on the findings of the study:

- The analytical solutions developed herein can be used for rough estimation of block cracking trends as a function of temperature change, layer thickness, existing block crack size, and layer properties.

- Block cracking patterns occur in pavement segments that are symmetric (nearly square) or in large, uncracked pavement segments, and when other type of cracks (i.e. longitudinal or
transverse cracks) are not present or infrequent;

• Thermal cracking patterns tend to occur in pavements where the long side is significantly longer than the short side (high aspect ratio of currently uncracked segments);

• The dimension of the pavement segment, relaxation behavior and aging susceptibility of asphalt materials and cooling rate are all factors contributing to the block crack pattern;

• Block cracking formation can be separated into two steps: (1) initiation of microcracks as a result of insufficient stress relaxation capacity and/or fracture resistance of the material, and; (2) channeling of cracks in the pavement surface, the path of which is highly dominated by local material properties and boundary conditions;

• When microcracks propagate in a symmetric or large pavement segment, they tend to develop in arbitrary directions to form Y-junction patterns;

• When growing cracks approach another crack, they tend to join the crack at right angles;

• Block cracking appears to be a partial-depth phenomenon for most overlays, due to the extreme gradient (from brittle to ductile) caused by the combined existence of material and temperature gradients.

• Due to the vertically-graded material system caused by severe near-surface aging, block cracking forms near the pavement surface at a temperature significantly warmer than the thermal cracking threshold, and saturates rapidly;

• Preservation of the initial asphalt properties or use of the mixture in a mild climate would likely lead to minimal block crack formation, and finally;

• The results presented herein suggest the problem with waiting to apply preventive
maintenance treatments until after a pavement begins to demonstrate visible block cracking.

6.3 Future Extensions

The following topics are recommended for future research:

- Improvements in the current damage and fracture model, for instance, developing a contact model that is capable of reloading and unloading. This will be helpful in studying cyclic fracture behavior in asphalt materials. Although a coupled reloading and unloading feature in PFC software will be difficult to develop, perhaps an energy-based approach will be feasible.

- A 3D discrete element model with more complex heterogeneous asphalt material representations will be useful for the block cracking study. As computing power increases and 3D material scanning becomes more commonplace, future studies can be directed towards exploring the effects of accounting for material morphology (individual aggregates modeled) in block cracking simulations. This will likely only improve upon minor details of block crack shape and cracking rates, but might allow for quantitative assessments of block cracking for given materials, climates, and field aging.

- There are many factors of block cracking patterns, include: thermal and material gradients, vehicular effects, freeze-thaw cycles, volume loss due to volatilization and physical hardening, raveling, stripping, irreversible viscoelastic effects in cracked systems subjected to temperature cycling, etc. Consideration of all the factors is beyond the capability of current computer models. Future modeling studies should continue to add more physics as
computing power increases. It will be particularly interesting to see if moisture damage is a key factor, and if wetting and drying cycles along are capable of driving block cracking damage (as is the case in soils).

- In terms of prevention, in-situ non-destructive techniques might be quite helpful in allowing pavement engineers to detect the onset of block cracking based on the results of this study. These techniques will help identify block cracking when it is difficult to visually detect, i.e., when it is likely still in the cracking/healing stage. Early detection allows time for preventive maintenance before permanent block cracking patterns form. In addition, findings presented herein suggest that current simple performance test criteria for thermal cracking mitigation might not be sufficient to design mixtures to resist block cracking formation. Therefore, future work is needed to establish cracking test criteria that can be used to control thermal and block cracking, either separately, or in a combined test and/or criteria scheme.
REFERENCES


Rock Mechanics.


APPENDIX A: ALGORISM OF VISCOELASTIC COHESIVE-SOFTENING CONTACT MODEL

This algorism is developed by Hill (2016), and rewritten by the author.

• Step 1: Calculate parameters A, B, C, D:

\[
A = 1 + \frac{K_k \Delta t}{2C_k} \\
B = 1 - \frac{K_k \Delta t}{2C_k} \\
C = \frac{\Delta t}{2C_k A} + \frac{1}{K_m} + \frac{\Delta t}{2C_m} \\
D = \frac{\Delta t}{2C_k A} - \frac{1}{K_m} + \frac{\Delta t}{2C_m}
\]

• Step 2: If the normal force does not reach the peak load, calculate normal force and shear force:

\[
f_n^{(t+1)} = \pm \frac{1}{C} [u_n^{t+1} - u_n^t + \left(1 - \frac{B}{A}\right) u_k^t - \pm D f_n^t]
\]

\[
f_s^{(t+1)} = \mp \frac{1}{C} [u_s^{t+1} - u_s^t + \left(1 - \frac{B}{A}\right) u_k^t - \pm D f_s^t]
\]

• Step 3: Check if the calculated normal force is tension or compression

  o Step 3.1: If tension, check if the calculated normal force reaches the peak load. If reached, record the maximum contact force and the corresponding displacement, calculate the maximum displacement:

    ▪ Step 3.1.1: Calculate normal force and shear force based on the post-peak softening model:

    ▪ Step 3.1.2: check if the plastic displacement reaches the maximum displacement. If reached, then the contact breaks. If not, continue to the next calculation step.

  o Step 3.2: If compression, check if the calculated normal force reaches the peak load. If
reached, record the maximum contact force and the corresponding displacement, calculate
the maximum displacement:

- **Step 3.2.1:** Calculate normal force and shear force based on the post-peak softening
  model:

- **Step 3.2.2:** check if the plastic displacement reaches the maximum displacement. If
  reached, then the contact breaks, If not, continue to the next calculation step.

$$p^k \left( \frac{U_p}{U_{p,lim}} \right) = F_{\text{max}} \left( 1 - \frac{U_p}{U_{p,lim}} \right)$$

(0-1)

Noting that the normal stress and shear stress only calculate once every calculation step.

The code of Burger’s model is shared by the current version of software, so only that of
the softening model is provided herein. It is important to note that the definition of each parameter
is required to build this contact model in the C++ environment, so only copying this piece of code
will not work if the definition part is not complete. Some parameters are not necessary for the
stress calculation, because they are defined only for debugging. If coupling this contact model
with the thermal contact model, additional code is required.

The code of softening model is as follows:

```cpp
if (peak_met_==0.0) {
  //Please copy the code of burger’s model here.

double Fn = force_.x();
double Fs = sforce_old.mag();
double fmag = sqrt(Fn * Fn + Fs * Fs);
double alpha = 2.0 * acos(fabs(Fn / (fmag ? fmag : 1.0))) / dPi;
double Fmax = (1.0 - alpha) * bb_tenF_ + alpha * bb_shearF_;
double max_force = Fmax * (1.0 - softened_);
if (fmag >= max_force) {
  peak_met_ = 1.0;
}
if (force_.x()<=0) {   //tension
  if (peak_met_==1.0) {
    if (u_nindt_==0.0) {
      u_nindt_ = -overlap;
    }
  }
}
```

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knt_ = fabs(force_.x()/overlap) ? fabs(force_.x()/overlap) : knk_; 
knt_ /= 1000; 
ks_ = fabs(force_.x()/overlap) ? fabs(force_.x()/overlap) : knk_; 
ks_ /= 1000; 
double uplimT = bb_tenF_ / (knt_ ? knt_ : 1.0); 
double uplimS = bb_shearF_ / (ks_ ? ks_ : 1.0); 
double uplimG = uplimT < uplimS ? uplimT : uplimS; 
uplim_ = uplim_ > uplimG ? uplim_ : uplimG; 
}
} 
else { //compression 
  if (peak_met_ == 1.0) {
    if (u_nindt_ == 0.0) {
      u_nindt_ -= overlap; 
      knc_ = uplim_ ? fabs(force_.x()/overlap) : knk_; 
      ks_ = knc_/1000; 
      double uplimT = bb_tenF_ / (knc_ ? knc_ : 1.0); 
      double uplimS = bb_shearF_ / (ks_ ? ks_ : 1.0); 
      double uplimG = uplimT < uplimS ? uplimT : uplimS; 
      uplim_ = uplim_ > uplimG ? uplim_ : uplimG; 
    }
  }
} 
else { 
  #ifdef THREED 
  DVect norm(trans.x(),0.0,0.0); 
  #else 
  DVect norm(trans.x(),0.0); 
  #endif 

  if (force_.x() > 0.0) { //COMPRESSIVE 
    double dun = trans.x() * knc_; 
    force_.rx() = force_.x() - dun; 
    sforce = sforce_old - trans*ks_; 

    double Dcoef; 
    double max_s_force = force_.x()*(fric_ * (1.0 - softened_) + rfric_ * softened_) + bb_shearF_ * (1.0 - softened_); //slip check 
    double sfmag = sforce_old.mag() ? sforce_old.mag() : 1.0; 
    DVect sunit = sforce_old/sfmag; 
    DVect max_rs_force = sunit*force_.x()*rfric_; //Residual force 
    if (sfmag >= max_s_force) { 
      peak_met_ = 1.0; 
    } 
  } 
  if (peak_met_ == 1.0) {
    if (u_nindt_ == 0.0) {
      u_nindt_ -= overlap; 
      knc_ = uplim_ ? fabs(force_.x()/overlap) : knk_; 
      ks_ = knc_/1000; 
      double uplimT = bb_tenF_ / (knc_ ? knc_ : 1.0); 
      double uplimS = bb_shearF_ / (ks_ ? ks_ : 1.0); 
      double uplimG = uplimT < uplimS ? uplimT : uplimS; 
      uplim_ = uplim_ > uplimG ? uplim_ : uplimG; 
    }
  }
}
Dcoef = broken_ == 1 ? 0. : bb_shearF_/ (uplim_ ? uplim_ : 1.0);
double duplas = (sfmag - max_s_force)/((ks_ - Dcoef) ? (ks_ - Dcoef) : 1.0);
sforce =sforce- sunit*ks_*fabs(duplas);
double sfmag = sforce.mag();
double rfmag = max_rs_force.mag();  //Max residual force
if ((sforce| max_rs_force) < 0. || (sfmag < rfmag)){  //state->fail
  sforce = max_rs_force;
  bb_state_=2;
}
uplas_ += fabs(duplas);
}
else {  //EXTENSILE
  if (fabs(softened_-1.0) < 1e-5) {
    broken_ = 1.0;
    bb_state_=1;
    force_.fill(0.0);
    sforce.fill (0.0);
  }
  else {
    double dun=trans.x() * knt_;
    force_.rx()=-dun;
    sforce =sforce_old - trans*ks_;
    double Fn=force_.x();
    double Fs = sforce.mag();
    DVect sunit = sforce_old/(Fs ? Fs : 1.0);
    double fmag = sqrt(Fn * Fn + Fs * Fs);
    double alpha = 2.0 * acos(fabs(Fn / (fmag ? fmag : 1.0))) / dPi;
    double Fmax = (1.0 - alpha) * bb_tenF_ + alpha * bb_shearF_;
    double max_force = Fmax * (1.0 - softened_);
    double CoefA = knt_ * knt_ * Fn * Fn / (fmag ? fmag : 1.0) + ks_ * Fs * Fs / (fmag ? fmag : 1.0);
    double CoefB = 2.0 * Fmax * Max * (1.0 - softened_) / (uplim_ ? uplim_ : 1.0) - 2. * knt_ * Fn * Fn / (fmag ? fmag : 1.0) - 2. * ks_ * Fs * Fs / (fmag ? fmag : 1.0);
    double CoefC = fmag * fmag - max_force * max_force;
    if (CoefA != 0. && (CoefB*CoefB-4.*CoefA*CoefC)>0.) {
      double duplas = 0.5*(-CoefB - sqrt(CoefB*CoefB-4.*CoefA*CoefC))/CoefA *timestep;
      sforce= sforce - sunit*(ks_*fabs(duplas*Fs))/(fmag ? fmag : 1.0));
      if (Fs*(sforce|sunit) < 0.) {
        sforce.fill (0.0);
      }
      double delta_F= (force_.x() < 0.0) ? (-fabs(knt_*fabs(duplas*Fn))/(fmag ? fmag : 1.0)) : (fabs(knt_*fabs(duplas*Fn))/(fmag ? fmag : 1.0));
      force_.rx() = delta_F;
    }
  }
}

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uplas_ += fabs(duplas);

} else if ((CoefB*CoefB-4.*CoefA*CoefC)<0.) {
    sforce.fill (0.0);
    force_.rx() = 0.;
    uplas_ = uplim_;
}

if (fabs(broken_ -1)>1e-2) {
    softened_ = uplas_/((uplim_ ? uplim_ : 1.0) < 1.0 ? uplas_/((uplim_ ?
uplim_ : 1.0) : 1.0);
    if (fabs(softened_ -1.0) < 1e-5) {
        broken_ = 1;
        bb_state_ =1;
        force_.fill(0.0);
        sforce.fill (0.0);
    }
}

if (fabs(broken_ -1)>1e-2) {
    softened_ = uplas_/((uplim_ ? uplim_ : 1.0) < 1.0 ? uplas_/((uplim_ ?
uplim_ : 1.0) : 1.0);
    if (fabs(softened_ -1.0) < 1e-5) {
        broken_ = 1;
        bb_state_ =1;
        force_.fill(0.0);
        sforce.fill (0.0);
    }
}

if (state->canFail_) {
    if (bb_state_ == 2) { // Broke in shear (compression)
        if (cmEvents_[fBondBreak] >= 0) {
            QVariant p1;
            IContact * c = const_cast<IContact*>(state->getContact());
            TPtr<IThing> t(c->getIThing());
            p1.setValue(t);
            arg.push_back(p1);
            p1.setValue(bb_state_);
            arg.push_back(p1);
            p1.setValue(bb_state_);
            arg.push_back(p1);
            IFishCallList *fi = const_cast<IFishCallList*>(state->getProgram()->
->findInterface<IFishCallList>();
            fi->setCMFishCallArguments(c,arg,cmEvents_[fBondBreak]);
        }
    }

    if (bb_state_ == 1) { // Broke in tension
        bmode_ =1;
        if (cmEvents_[fBondBreak] >= 0) {

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FArray<QVariant,3> arg;
QVariant p1;
IContact * c = const_cast<IContact*>(state->getContact());
TPtr<IThing> t(c->getIThing());
p1.setValue(t);
arg.push_back(p1);
p1.setValue(bb_state_);
arg.push_back(p1);
p1.setValue(0);
arg.push_back(p1);
IFishCallList *fi = const_cast<IFishCallList*>(state->getProgram()->findInterface<IFishCallList>();
fi->setCMFishCallArguments(c,arg,cmEvents_[fBondBreak]);
}

// 2) Resolve sliding if no contact bond exists
if (bb_state_ < 3) {
    // No contact bond - normal force is positive only
    force_.rx() = std::max(0.0, force_.x());
    // No contact bond - sliding can occur. This is the normal force
    // multiplied by the coefficient of friction
    double crit = force_.x() * fric_;
    crit = max(0.0, crit);
    double sfmag = sforce.mag();
    if (sfmag > crit||bb_state_==2) {
        double rat = crit / sfmag;
        sforce *= rat;
        if (!s_ && cmEvents_[fSlipChange] >= 0) {
            FArray<QVariant,3> arg;
            QVariant p1;
            IContact * c = const_cast<IContact*>(state->getContact());
            TPtr<IThing> t(c->getIThing());
p1.setValue(t);
arg.push_back(p1);
p1.setValue(0);
arg.push_back(p1);
IFishCallList *fi = const_cast<IFishCallList*>(state->getProgram()->findInterface<IFishCallList>();
fi->setCMFishCallArguments(c,arg,cmEvents_[fSlipChange]);
        }
        s_ = true;
    }
    else {
        if (s_) {
            if (cmEvents_[fSlipChange] >= 0) {
                FArray<QVariant,3> arg;
                QVariant p1;
                IContact * c = const_cast<IContact*>(state->getContact());
                TPtr<IThing> t(c->getIThing());
p1.setValue(t);
arg.push_back(p1);
p1.setValue(1);
arg.push_back(p1);
IFishCallList *fi = const_cast<IFishCallList*>(state->getProgram()->findInterface<IFishCallList>();
fi->setCMFishCallArguments(c,arg,cmEvents_[fSlipChange]);
            }
        }
    }
}
s_ = false;
APPENDIX B: FULL SIMULATION RESULTS OF THE 2D 0.44m*0.38m MODEL WITH A COOLING RATE OF -0.03°C/STEP

The full simulated displacement field results of the model with the control PG 64-22 mixture with a cooling rate of -0.03°C/step are showed as follows.

**Figure B.1.** Displacement field results of the model with the control PG 64-22 mixture at -12.3 °C, with a cooling rate of -0.03°C/step.

**Figure B.2.** Displacement field results of the model with the control PG 64-22 mixture at -12.6 °C, with a cooling rate of -0.03°C/step.
Figure B.3. Displacement field results of the model with the control PG 64-22 mixture at -12.9 °C, with a cooling rate of -0.03°C/step.

Figure B.4. Displacement field results of the model with the control PG 64-22 mixture at -13.2 °C, with a cooling rate of -0.03°C/step.
Figure B.5. Displacement field results of the model with the control PG 64-22 mixture at -13.5 °C, with a cooling rate of -0.03°C/step.

Figure B.6. Displacement field results of the model with the control PG 64-22 mixture at -13.8 °C, with a cooling rate of -0.03°C/step.
Figure B.7. Displacement field results of the model with the control PG 64-22 mixture at -14.1 °C, with a cooling rate of -0.03°C/step.

Figure B.8. Displacement field results of the model with the control PG 64-22 mixture at -14.4 °C, with a cooling rate of -0.03°C/step.
Figure B.9. Displacement field results of the model with the control PG 64-22 mixture at -14.7 °C, with a cooling rate of -0.03°C/step.

Figure B.10. Displacement field results of the model with the control PG 64-22 mixture at -15.0 °C, with a cooling rate of -0.03°C/step.
Figure B.11. Displacement field results of the model with the control PG 64-22 mixture at -15.3 °C, with a cooling rate of -0.03°C/step.

Figure B.12. Displacement field results of the model with the control PG 64-22 mixture at -15.6 °C, with a cooling rate of -0.03°C/step.

The full simulated force field results of the model with the control PG 64-22 mixture with a cooling rate of -0.03°C/step are showed as follows.
Figure B.13. Force field results of the model with the control PG 64-22 mixture at -12.3 °C, with a cooling rate of -0.03°C/step.

Figure B.14. Force field results of the model with the control PG 64-22 mixture at -12.6 °C, with a cooling rate of -0.03°C/step.
Figure B.15. Force field results of the model with the control PG 64-22 mixture at -12.9 °C, with a cooling rate of -0.03°C/step.

Figure B.16. Force field results of the model with the control PG 64-22 mixture at -13.2 °C, with a cooling rate of -0.03°C/step.
Figure B.17. Force field results of the model with the control PG 64-22 mixture at -13.5 °C, with a cooling rate of -0.03°C/step.

Figure B.18. Force field results of the model with the control PG 64-22 mixture at -13.8 °C, with a cooling rate of -0.03°C/step.
Figure B.19. Force field results of the model with the control PG 64-22 mixture at -14.1 °C, with a cooling rate of -0.03°C/step.

Figure B.20. Force field results of the model with the control PG 64-22 mixture at -14.4 °C, with a cooling rate of -0.03°C/step.
Figure B.21. Force field results of the model with the control PG 64-22 mixture at -14.7 °C, with a cooling rate of -0.03°C/step.

Figure B.22. Force field results of the model with the control PG 64-22 mixture at -15.0 °C, with a cooling rate of -0.03°C/step.
Figure B.23. Force field results of the model with the control PG 64-22 mixture at -15.3 °C, with a cooling rate of -0.03°C/step.

Figure B.24. Force field results of the model with the control PG 64-22 mixture at -15.6°C, with a cooling rate of -0.03°C/step.

The development of crack patterns of the model with the control PG 64-22 mixture with a cooling rate of -0.03°C/step are showed as follows.
Figure B.25. Crack patterns of the model with the control PG 64-22 mixture at -12.9°C and -13.2°C, with a cooling rate of -0.03°C/step.

Figure B.26. Crack patterns of the model with the control PG 64-22 mixture at -13.5°C and -13.8°C, with a cooling rate of -0.03°C/step.
Figure B.27. Crack patterns of the model with the control PG 64-22 mixture at -14.1°C and -14.4°C, with a cooling rate of -0.03°C/step.

Figure B.28. Crack patterns of the model with the control PG 64-22 mixture at -14.7°C and -15.0°C, with a cooling rate of -0.03°C/step.
Figure B.29. Crack patterns of the model with the control PG 64-22 mixture at -15.3°C and -15.6°C, with a cooling rate of -0.03°C/step.
APPENDIX C: FULL SIMULATION RESULTS OF THE 3D MODEL FOR SIMULATION OF MICROCRACK INITIATION AND PROPAGATION PROCESSES IN AN AGED PAVEMENT PLACED IN A MILD CLIMATIC REGION

Microcracks of each simulation state were plotted in AutoCAD according to the coordinates information outputted from PFC software, the corresponding crack patterns of each simulation state are showed as follows.

Figure C.1. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.15°C.

Figure C.2. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.18°C.
Figure C.3. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.20°C.

Figure C.4. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.23°C.
Figure C.5. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.25°C.

Figure C.6. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.30°C.
Figure C.7. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.33°C.

Figure C.8. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.35°C.
Figure C.9. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.38°C.

Figure C.10. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.40°C.
Figure C.11. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.43°C.

Figure C.12. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.45°C.
Figure C.13. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.48°C.

Figure C.14. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.50°C.
Figure C.15. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.55°C.

Figure C.16. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.58°C.
Figure C.17. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.61°C.

Figure C.18. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.63°C.
Figure C.19. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.66ºC.

Figure C.20. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.68ºC.
Figure C.21. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.71°C.

Figure C.22. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.73°C.
Figure C.23. Crack patterns of an aged pavement model assumed to be placed in a mild climatic region at -12.76°C.