MODELING ANALYSIS OF PAVEMENT LAYER INTERFACE BONDING CONDITION EFFECTS ON CRACKING PERFORMANCE

By

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To my wife and parents
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The structure of asphalt pavements are characterized by composition of pavement layers. Previous research has suggested that the bond conditions between pavement layers significantly affect the cracking performance. Open Graded Friction Course (OGFC), which is widely used in Florida, was generally considered as a function course and has lower cracking resistance. While for the top-down cracking, OGFC may be the first front in resisting the cracking. The primary objective of this study is to develop a model to evaluate the effects of interface bonding conditions on top-down and reflective cracking performance.

Continuum and multi-scale modeling of the pavement structure were conducted for different layer combination and bonding conditions. The FEM modeling is implemented with Florida HMA Fracture Mechanics to evaluate the effects of bonded interface on cracking performance. It was found that the OGFC with a poor bond such as the conventional tack coat may reduce the cracking resistance of the structure. A well bond between the OGFC and underlying layer may result in a better cracking performance. It was determined in the multi-scale analysis that the interface affects the cracking performance by affecting the stress transmission through the interface in the pavement.
structures. The continuum model of the pavement structure with different bond conditions indicates that the different bond conditions may lead to totally different stress distribution in the pavement layers. Pavement with Novabond® as bonding agent has higher cracking resistance than the pavement with conventional tack coat as bonding agent.

The modeling of composite specimen tests was performed to investigate the mechanism of the bonding effects on top-down and reflective cracking performance. Different bonding conditions were assumed due to the different application rate and type of the bonding agent. The modeling results show agreement with test results. The model and the assumption were considered reasonable. Novabond® was determined has a positive effect on top-down and reflective cracking performance. The model clearly proved that the interface bonding conditions have significant effect on cracking performance.
CHAPTER 1
INTRODUCTION

1.1 Background

Open graded friction course (OGFC) has been widely used as a functional layer since 1950 in different parts of the United States. The evaluation of OGFC performance has primarily focused on durability, permeability and surface friction. However, OGFC may play a key role in top-down cracking performance of pavement since it is directly exposed to surface loading and environmental effects. Based on the analysis of data from pavement field sections in Florida (e.g. State Road 16, I-10 Madison County, and I-95 St. John’s County), there is suggestive evidence that top-down cracking performance is affected by the quality of OGFC and by the effectiveness of the bond between the OGFC and underlying pavement layer. Thus it is necessary to evaluate the contribution of OGFC in fracture resistance.

Based on laboratory tests (Koh 2009), OGFC has lower values of fracture energy density and dissipated creep strain energy density to failure than dense graded asphalt mixture. Therefore OGFC applied on top of dense graded asphalt mixture may reduce the crack resistance of pavement with conventional tack coat compared to pavement without OGFC (Chen 2011). However, the composite mechanisms of OGFC and bonding agent effects on top-down cracking resistance are not well understood. There is a need to conduct a model analysis to understand and evaluate the effects of OGFC and bonding agent on cracking performance.

Asphalt pavement is a bonded layered system. The interface bond between pavements layers plays an important role in the pavement performance. Poor bond between HMA layers is the cause of many pavement problems, such as the slippage
failure, surface layer delamination and top-down cracking (Hachiya and Sato 1997, NAPA 2000). Composite specimen tests conducted by Chen (2011) suggest that polymer modified asphalt emulsion used at the interface results in better cracking performance. The modified asphalt emulsion seals the existing pavement and provides an excellent bond between OGFC and underlying layer. Also, the high percentage of air voids of OGFC and high application rate of the emulsion allows the agent to migrate into the OGFC to form a thick cohesive interface of saturated mixture with enhanced fracture properties (Varadhan 2004). In addition, the bond and saturated mixture may reduce the stress transfer through the interface. Recent work at the University of Florida (Birgisson et al. 2006) suggests that cracks developed in the OGFC or the HMA layer can be effectively arrested and/or deterred with appropriate interface conditions formed at the bottom of the OGFC. However, the tests cannot capture the detailed stress and fracture characteristics near the interface of the specimen, so the mechanism is not well understood.

Therefore, it is necessary to identify and develop an analysis method to evaluate the effects of the interface on top-down cracking and reflective cracking. This method may provide thorough understanding of the mechanism of interface effects on crack initiation and propagation either in OGFC layer or HMA structural layer.

1.2 Hypothesis

The introduction of better bond conditions between asphalt pavement layers by using high application rate of polymer modified asphalt emulsion as bonding agent can increase the resistance to initiation or propagation of top-down cracking or reflective cracking by reducing the stress transmission through the interface and increasing the mixture fracture properties.
1.3 Objectives

The overall objective of this research is to understand the effects of the bonded interface on cracking performance. More specific objectives of this research can be summarized as follows:

- Develop an analysis method to evaluate the effects of bonded interface on top-down and reflective cracking resistance;
- Evaluate the effects of the OGFC on top-down cracking;
- Evaluate the effects of bonded interface on top-down and reflective cracking;
- Understand the effects of the bonded interface on stress distribution near the interface;
- Determine the key change in behavior induced by bonded interface that has the greatest potential to enhance top-down and reflective cracking performance;
- Evaluate the effects of the bonding agent application rate on cracking performance.

1.4 Scope

This study primarily focuses on developing an approach to understand how the bonded interface affects the top-down and reflective cracking. The approach can also evaluate the effects of OGFC on top-down cracking performance. In order to understand the effects of the bonded interface characteristics on stress distribution in the layers and cracking performance, continuum and multi-scale analysis are performed. Different bonding conditions and bonding agent application rates were used in the model to evaluate their effects. In the continuum analysis, four different layer combination models were created and examined. In the multi-scale model, different local effects on stress transmission were evaluated. Three models with local effects were examined. The composite specimen tests were introduced into the models and the analysis was performed to evaluate the bonded interface effects.
1.5 Research Approach

This study focuses primarily on understanding the effects of bonded interface on top-down and reflective cracking. The approach used in this study involves the following steps:

- Review previous research on pavement interface evaluation, top-down and reflective cracking mechanism, HMA fracture mechanics, specimen tests of bonded interface and methods for bonded interface analysis.

- Conduct model analysis of composite specimen tests to assess the validity of the models and evaluate the effects of bonded interface on top-down and reflective cracking.

- Conduct continuum and multi-scale analysis: (1) Determine the analysis methods; (2) Conduct continuum analysis to evaluate bonded interface effects; (3) Conduct multi-scale analysis to evaluate local effects and interface effects.
CHAPTER 2
LITERATURE REVIEW

2.1 Background

It is now well accepted that top-down cracking is a major type of distress in asphalt pavements. The top-down cracking has been reported to occur in many parts of the United States, as well as in France, South Africa, Japan, Netherlands and UK. Different from the conventional bottom-up fatigue cracking, top-down cracking initiates at the surface of the pavement and propagates downward. The causes of top-down cracking are complicated and cannot be explained by traditional fatigue mechanisms used to explain fatigue cracking that initiates at the bottom of the pavements.

An Open Graded Friction Course (OGFC) is usually designed as the top pavement layer for quick draining and noise reduction purpose. There is a growing recognition that the OGFC may be the first layer in resisting top-down cracking. The analysis of finding from pavement field section in Florida (e.g. State Road 16, I-10 Madison County, and I-95 St. John’s County) suggests that good quality and well-bonded Open Graded Friction Course may result in better top-down cracking performance. In addition recent work at UF suggests that cracks that develop either in the OGFC or the HMA structural layer can be effectively arrested or deterred with appropriate interface conditions.

The existing OGFC design procedures are primarily empirical and do not take the fracture resistance into consideration. Improved guidelines and methods that can be used to evaluate and optimize fracture resistance for OGFC are needed. The contribution of the OGFC and the interface condition between the friction course and underlying layer to cracking resistance of the total pavement structure is also should be identified.
In the conventional macro-mechanical approach, the asphalt mixture is assumed to exhibit isotropic properties and the continuum model is used to describe the behavior of the composite. The constitutive model includes elastic, viscoelastic, etc. The parameters of the model are obtained mainly through laboratory investigations. While hot mix asphalt (HMA) is a complex composite that consists of asphalt binder, aggregate particles and air voids. The macroscopic behavior of the composite is determined by the properties of these constituents. Many studies have shown that considering the internal structure in constitutive models is essential to capture the stress and strain conditions of HMA. A fundamental crack growth law has been developed by the research team based on the principles of viscoelastic fracture mechanics. And this law has been incorporated into the HMA fracture mechanics model. The HMA fracture mechanics model allows for prediction of crack initiation and crack growth in asphalt mixture subjected to any specified loading history. The Florida top-down cracking model was built by introducing the energy ratio concept into the HMA fracture mechanics.

2.2 Pavement Layer Interface

2.2.1 Background

The structure of asphalt pavements are characterized by composition of pavement layers. The study of in-service pavements indicates that the condition of interface bonding between pavement layers plays an important role in the performance of whole structure. Current practice in the field is to construct layered asphalt pavements with a good bond that will ensure continuous displacements at the interface. Inadequate layer bonding may lead to slippage or separation of the layers. This may result in distresses such as slippage cracking, delamination and potholes etc. in the pavement system. While layer interface debonding or separation often occurs in pavement structure due to
the poor construction or poor bonding (Walubita and Scullion 2007). To obtain longer lifetime and appropriate loading capacity of the pavement, good bonding quality must be guaranteed. The state of the bond at the interface affects their performance by influencing the stress levels within the layers.

Tack coat is the most commonly used bond agent during the pavement construction. There are two types of tack coat which are used in the pavement systems. The most commonly used tack coat is emulsified asphalt. The second type of tack coat that can be used is asphalt binder, which is rarely used in comparison to emulsions. Emulsified asphalt consists of three basic ingredients: asphalt binder, water and emulsifying agent. Emulsions are typically classified as slow-setting grades, rapid-setting grades and quick-setting grades by how quickly they set. Slow-setting emulsions are most commonly used for tack coat (Chaignon and Roffe 2001; Santucci 2009). Rapid-setting emulsions should be considered for used at night or in cooler weather since their break time is quicker. Quick-setting emulsions are used at night or in cooler weather when rapid construction is needed (California Tack coat guidelines 2009).

2.2.2 Effect Factors on Interface Strength

In 1999, the International Bitumen Emulsion Federation conducted a world-wide survey of the use of tack coats (Randy, Jingna and Jason 2005). The survey indicates that the properties of interface were affected by several factors. These factors include the type of tack materials, application rates, curing time, temperature, test methods and construction methods.

Several different materials, including hot asphalt cements, emulsified asphalts, cutback asphalts and trackless tack, are generally used as bond between asphalt pavement layers. Mohammad and Raqib (2002) indicate that asphalt emulsion CRS-2P
provided significantly higher interface shear strength than asphalt cement PG 64-22, PG 76-22M and emulsion SS-1, CSS-1 and SS-1h. However, some studies reported that paving grade asphalt binders used as interface bond materials have higher shear strength compared to asphalt emulsions due to the higher viscosity (Buchanan and Woods 2004). The test conducted by Canestrari and Ferrotti (2005) shows that modified emulsions produce higher interlayer shear resistance than other interface treatments. The study performed by Leng et al. (2008) indicated that, in the use as interface between AC overlay and PCC pavement, the asphalt emulsion SS-1hP provided higher strength than cut back asphalt RC-70. Mohammad and Bae (2009) pointed out that among three types of emulsified tack coats, CRS-1, SS-1h and trackless, the trackless tack coat exhibited the highest shear strength and CRS-1 exhibited the lowest. In order to evaluate the rutting resistance of the asphalt pavement layer system, Leng et al. (2009) performed the Accelerated Pavement Testing. The test results shows that PG 64-22 and SS-1hP have better performance than RC-70 at the same residue application rate.

Studies conducted on asphalt pavement interface strength indicate that no bond or insufficient bond decreases pavement bearing capacity and may cause slippage. Insufficient bond may also cause tensile stress concentration at the bottom of the wearing course which may accelerate fatigue cracking (Mohammad et al. 2002). While higher application rates may increase interfac e shear strength, excessive bonding agent may increase the slippage and reduce the air void content because of the migration of the agent into the HMA. The investigation performed by Mrawira and Damude (1999) suggested that the applied tack coat weakened the interface bond by introducing a slip
plane. Uzan et al. (1978) conducted direct shear tests to measure shear strength parameter at failure to evaluate the interface adhesion properties. The study indicates that there was an optimum tack coat application rate at which the shear resistance reached a maximum value. Therefore, it is important to determine the optimum bonding material application rate to ensure good bonding strength. The optimum residue application rate for modified asphalt emulsion CRS-2P and CRE-2L have been determined to be 0.02 gal/sy for HMA pavement layers (Mohammad et al. 2002). Leng et al. determined that 0.04 gal/sy is the optimum residue application rate for SS-1hP when it is used between HMA and PCC. Mohammad et al. (2009) use interface shear strength test to evaluate the effect of application rates on bonding strength. Three application rates, 0.14, 0.28 and 0.70 l/m² were evaluated. All tack coat materials, including CRS-1, SS-1h and Trackless tack, showed the highest strength at application rate of 0.70 l/m². For SS-1h and trackless, shear strength consistently increased as application rate increased. While shear strength for CRS-1 appeared to stabilize at an application rate around 0.30 l/m². It is difficult to determine the optimum residual application rate within the range. The optimum application rate may also affected by other characteristics including the surface, temperature and construction methods. The application rate of the bond agent shows different effects on bond strength for different types of asphalt mixture. The study conducted by West et al. (2005) shows that for CSS-1 and CRS-2 emulsions, lower application rate result in higher bond strength when the emulsions are used between fine-graded mixture layers. While the effects of application rate on bond strength are not significantly for coarse-graded mixture. Mohammad et al. (2002) reported that at lower testing temperature, increasing the tack
coat application rates generally resulted in a decrease in interface shear strength. However, the interface shear strength was not sensitive to the application rate at the high temperature.

The effects of curing time on the bond strength are not clear. Canestrari et al. (2005) reported that the interface strength does not depend on curing time. The curing time has little effect on the interface shear strength. While other studies suggest that interface shear strength increased with longer curing time (Chen et al. 2008).

The asphalt mixture is a temperature susceptible material. So the temperature has significant effect on the bond strength. The simple shear test of the interface strength (Mohammad et al. 2002) indicates that for all of the six bond material (emulsions CRS 2P, SS-1, CSS-1 and SS-1h, asphalt binder PG64-22 and PG 76-22M), the interface strength increase with the decrease of the temperature. These results agree with the findings by Uzan et al. (1978). The bond strength will increase 2.3 times with the temperature decrease from 77°F to 50°F (West et al. 2005). At high temperature, the effect of the bond on interface strength is relatively small because the contribution of bond adhesion to the strength is insignificant (Canestrari et al. 2005).

Different test methods and loading conditions may result in different conclusions. Romanoschi and Mecalf (2001) revealed that the horizontal loads acting on the pavement surface, compared to circular distributed vertical loading, led to a dramatic increase in the tensile strains at the top and bottom of the wearing course and the at the top of the binder course. They also indicated that for the interface with a tack coat, the shear strength was not affected by the normal stress level. While for an interface without a tack coat, the effect was quite significant. Canestrari et al. (2005) reported that
the interface shear strength increases with the increase of normal stress applied for a given temperature. Leng et al. (2008) also reported that the interface shear strength increase with the increase of shear loading rate. The application of confinement on specimens may result in the higher interface shear strength (Mahammad et al. 2009).

Generally, the tack coat should be applied on a dry and clean pavement surface to obtain better bonding strength. The pavement surface conditions, including surface roughness, cleanliness and wetness, have significant effect on the interface bonding strength. Collop et al. (2003) found that the shear strength is lower when the tack coat was applied on a dirty pavement surface. However, Mohammad et al. (2009) reported that dusty conditions exhibited higher interface strength than clean conditions based on the statistical analysis of their test results. The explanation for these results is that a mastic-like material is developed during the construction. This mastic-like material has higher viscosity or consistency and result in a greater shear resistance. For the PCC surfaces, Leng et al. (2008) found that the tined surfaces provide higher interface shear strength than smooth surface at low tack coat application rate. While at optimum tack coat application rate, smooth surface can provider better bonding at intermediate temperature and no normal forces are applied. West et al. (2005) reported that pavement milling provided a better bond at the interface. The effects of the application of tack coat on interface shear resistance for milled surface are not significant (Tashman et al. 2008).

For the effect of wetness on the interface shear strength, Sholar et al. (2002) pointed out that the application of water on the tack coat surface may reduce the shear
strength. While the study conducted by Mohammad at al. (2009) indicates that the wetness does not have significant effects on the interface shear strength.

2.2.3 Pavement Interface Test Methods

Several tests were developed to evaluate the shear resistance of the interface. These tests including ASTRA test, layer-parallel direct shear (LPDS) test, Louisiana Interface Shear Strength Tester (LISST), Superpave Shear Tester (SST), Shear fatigue test, FDOT shear tester, The UTEP Pull-Off Device (UPOD), Wedge Splitting Test and Toque bond test.

ASTRA test is conducted in a direct shear box, which was developed by Anocona Shear Testing Research and Analysis program (Canestrari et al. 2005), as shown in Figure 2-1. In the test, the sample is held by two half boxes with the interlayer shear zone in the center. The normal and horizontal load were applied while shear force, horizontal and vertical displacement are recorded during the test.

To evaluate the interlayer shear strength, the test can be performed at different loading rates and temperatures. The normal load also can also be changed. Samples with different types of bond materials, application rates and curing times can be tested in the device to evaluate the effects of those factors on the shear strength.

LPDS test is a layer-parallel direct shear (LPDS) test device, which was developed by Swiss Federal Laboratories for Materials Testing and Research (EMPA) (Canestrari et al. 2005). It is a modified version of the Leutner test developed in Germany in 1979. This test does not have a normal confining load. It allows pure direct shear testing of cylindrical samples with a diameter of 150mm or prismatic specimens (width=150mm, height=120mm).
As shown in Figure 2-2, one part of the test sample is laid on a circular U-bearing and held by semicircular clamp. The other part remains suspended. The shear yoke apply the shear load to the specimen head at a constant deformation rate producing fracture within the predefined plane. The shear force and the relative displacement of the two specimen parts were recorded by the device and to be used to determine the peak shear stress.

Louisiana Interface Shear Strength Tester (LISST) is a direct shear device developed for the characterization of interface shear strength of cylindrical specimens (Mohammad et al. 2009). This test device consists of two main parts, the shearing frame and the reaction frame. The reaction frame is kept stationary and the shearing frame is allowed to move. A cylindrical specimen is placed inside the shearing and reaction frames. Loading is then applied to the shearing frame. Shear failure occurs at the interface as the vertical load is gradually increased. The vertical and horizontal displacements were measured during the test.
Figure 2-2. LPDS test (Photo courtesy of Raab and Partl 2008)

Figure 2-3. LISST test (Mohammad et al. 2009)

Superpave Shear Tester (SST) was developed to measure mixture properties that can be used to evaluate the mixture’s resistance to permanent deformation. In the SST tester, vertical, horizontal and confining loads can be applied on the specimens. The temperature of the chamber can be changed in the test. The specimen was hold by a
shearing mold. The mold has two parts. Each part has 5.9 inch diameter and 2 inch deep groove. The test was performed by controlling the load at a constant rate. The applied vertical and horizontal load, deformation will be recorded during the test (Mohammad et al. 2005).

Shear fatigue test was developed to evaluate the pavement layer interface shear properties under repeated loading condition. As shown in the Figure 2-5, the load was applied on the top plate. The specimen was hold in the two cups that has an angle of $25.5^\circ$ from the vertical. This angle makes the shear stress at the interface half of the normal stress. The load was applied at a frequency of 5Hz on the specimen. And loading period and pulse was used to simulate the vehicle repeated load. The normal and tangential deformations at the interface are recorded to each loading cycle. The number of loading cycles that created a 1mm permanent shear displacement was used to evaluate the fatigue properties of the layer interface (Romanoschi et al. 2005).

The Florida DOT developed the FDOT shear tester to evaluate the bond strength of the tack coat after premature failures in pavements overlaid on wetted tack coat. The simple direct shear device was used in the materials testing system (MTS). The specimens with 6 inches in diameter were used in the test. The gap between the shearing rings was set at 3/16 inches to reduce the skewness effects, bending stress effects and irregular surface effects. The load is strain controlled and set at 2 in/min. The specimen was conditioned at $77^\circ$F for a minimum period of 2 hours before testing. Then the specimen was placed into the shear rings and kept the shear direction parallel to the traffic direction.
Figure 2-4. Superpave shear tester (Photo courtesy of Mohammad et al. 2005)

Figure 2-5. Shear fatigue test (Photo courtesy of Romanoschi and Metcalf 2001)
Toque bond test was originally developed in Sweden to evaluate the pavement layer interface (Walsh et al. 2001). This test was then adopted in UK to evaluate the interface of thin surface layer pavements. In the test, the pavement is cored deeper than the interface (Roffe and Chaignon 2002). Then a toque is applied manually on the top the core introducing a twisting shear failure at the interface. The maxima measured toque value is indicative of the bond shear strength. Recently, Choi et al. (2005) developed a laboratory based toque test allowing the test to be conducted easier and controllable. In this test, the core is taken in the field and then the bottom of the core below the interface is clamped using a gripping unit. On the top of the core, a steel plate was attached to the surface. A toque is applied using a toque wrench through the plate until the specimen fails. The force for failure is recorded to calculate the shear strength of the interface. The laboratory toque bond test is conducted at a temperature of $20\pm2^\circ \text{C}$.
The UTEP Pull-Off Device (UPOD) was developed at the University of Texas at El Paso (UTEP). Unlike the tests discussed above, the UPOD measure the tensile strength of the tack coat rather than shear strength (Deysarkar 2004). The device is around 23 lbs and can be leveled with the pivoting feet. A torque wrench is used to pull the plate up from the tack coat surface and the torque is recorded to calculate the tensile strength. The procedure of this test is simple. After tack coat is applied and set for specified time, the device is placed on the tack coat surface. The torque wrench is rotated clockwise to let the plate set on the tack coat. A 40 lbs load then is applied on the weight key which is on top of device for ten minutes in order to confirm the contact. The load is removed and the torque wrench is rotated counter clockwise to detach the plate from the tack coat surface. The toque required to detach the plate is recorded and converted to tensile strength.

Wedge Splitting Test was developed in 1995 to characterize the fracture mechanics properties of bonding between the pavement layers (Tschegg et al. 1995). As shown in Figure 2-9, the specimen was prepared with a groove at the interface. The vertical load is applied on the wedge of a specified angle and is transmitted to a horizontal force, producing a tensile stress on the interface. Vertical and horizontal
displacements and vertical loads were measured to obtain the load-displacement curve. The fracture energy and specific fracture energy can be derived from the load-displacement curve. The test was repeated at different temperatures. At high temperature, the plastic ductile fracture behavior with low maximum horizontal force was observed. At low temperature, the interface material shows brittle with higher maximum horizontal forces. It was reported that the tensile strength decreases with the temperature increasing while the fracture energy reaches the maximum in the medium temperature. The specific fracture energy was more appropriate to characterize the fracture power than the maximum load.

Figure 2-8. UTEP pull-off device (Photo courtesy of Deysarkar and Tandon 2004)
2.3 Theoretical Analysis and Modeling of Interface Effects

2.3.1 Introduction

The deformation behavior and performance of asphalt concrete are affected by various factors, including time, temperature, loading, aging and moisture. Models have been developed to evaluate the effects of these factors on pavement performance. The development of mechanistic models made a significant impact on material specifications, mixture and pavement design and construction. Continuing improvement of computational power and test techniques will result in more realistic, accurate models to predict the performance of asphalt mixtures and pavements.

For traditional approach, the responses of the pavement are calculated based on the structure and the material properties. Laboratory test results and responses of the specimens are used to estimate the life of the pavement. The weakness of this method is that the properties and response for different materials and structures may not be
determined accurately in the tests. The loading mode in the test, which has a significant effect on the response, is relatively simple, either stress controlled or strain controlled. The boundary conditions of the pavement simulated in the test are limited and cannot represent the real condition in the field. The mechanistic approach can overcome these weaknesses. In the mechanistic model, the stress-strain behavior of the material can be used to calculate the pavement response in a wide range of pavement boundary conditions.

Micromechanical approach and continuum approach are two general approaches of mechanistic approaches. Micromechanical approach can describe the microscopic parameters, including microcrack, aggregate size and shape, and air voids. Continuum approach takes the pavement structure as a homogeneous continuum. Significant development has been made in the modeling of asphalt mixture in both micromechanics and continuum mechanics. Many works have been done in combining these two approaches to predict the response and performance of asphalt concrete, which is called multiscale-model analysis. The development of imaging technique, like digital imaging, x-ray tomography, provides more accurate microstructures of asphalt pavement. The combination of multiscale-model and the imaging technique will be an efficient tool to evaluate the effects of mixture parameters and pavement structures on the behavior of asphalt concrete.

2.3.2 Modeling of Interface Effects

Tschegg et al. (1995) run a simple FE calculation of reflective crack propagation to investigate the bonding between asphalt layers. A four layers pavement system with a load on top of the surface layer was used to create the FE model. The calculation is performed to obtain the load-displacement curves. The calculation results are compared
with the experimental results and an iteration procedure was conducted to obtain the appropriate “damage parameters”. The modeling indicates that the reflective crack propagates from the lower to the upper layer without damage of the interface if the bonding between asphalt layers is good. While, for a poor bonding, the separation of the layers within the interface takes place before the crack tip arrives the interface. Therefore the upper layer is not damaged and a higher deflection is necessary to produce a damage zone in the upper layer.

Romanochi and Metcalf (2003) used modeling of layer interface condition to evaluate the pavement layer moduli backcalculation. Due to the complexity of pavement structures and the mixture properties, there is no exact solution to model the interface accurately. The author used the ABAQUS finite element program to model the pavement interface. The contact interaction feature in the program is used to represent the bond of the interface. The layer at the interface was considered as two rigid bodies and always remains in contact. The model of the problem is axis-symmetric with the axis in the center of the circular load. The structure thickness of the test section was used for meshes. The molded pavement structures include one semi-rigid and one flexible. Appropriate boundary conditions were applied on the axis and bottom. Four node quadrangle elements were used to mesh the model. All materials were considered as being linear elastic and static vertical load was simulated. The layer moduli were back calculated from the deflection data and were compared to the initial moduli values to evaluate the effectiveness of the interface modeling.

The study indicates that the modeling of the interface leads to a low backcalculated surface and base layer moduli for both flexible and semirigid structures.
For the same pavement structure and interface between binder and base layer, different interface models between wearing and binder layer lead to similar surface deflections and backcalculated layer moduli. For semirigid pavements, improper modeling of interface between binder and base layer leads to higher errors than those generated by interface between wearing and binder layer. The surface layer moduli are overestimated, while the modulus of base layer is underestimated. The modeling for both interlayers does not affect the backcalculated subgrade modulus a lot.

Bozkurt and Buttlar (2002) used three-dimensional finite element modeling to evaluate benefits of interlayer stress absorbing composite for reflective crack mitigation. The interlayer stress absorbing composite (ISAC) was been installed at airports to control reflective crack development in asphalt overlays. Field data indicates that the ISAC can significantly reduce reflective crack development, while there are no modeling calculation results to quantitatively show how interlayers affect the reflective crack development. Therefore, the 3-D finite element modeling was undertaken to evaluate the contribution of the interlayer to reflective crack resistance.

The interlayer was modeled with 3-D solid elements with viscoelastic material properties. The geotextile was modeled with 2-D plane stress elements due to the small thickness and stiffness. Continuum solid elements, infinite elements and plane stress elements were used in the modeling. In the thermal analysis, different cooling cycles are included in the analysis. Regular and modified mixes with and without ISAC are also considered in the modeling. The analysis indicates that the use of interlayer can significantly reduce the temperature and load related stresses at the bottom of an AC overlay. This may due to joint opening and closing during temperature cycling. The
using of modified asphalt mixtures lead to lower tensile stresses at the top of the overlay. Joint load transfer may have significant influence when the reflective crack simulation included in the modeling.

Kruntcheva et al. (2005) performed a theoretical study to understand the effects of poor bond on flexible pavement performance. Two different modeling approaches have been included in the analysis. First, a layered linear elastic program, BISAR, was taken into account to analyze the pavement structure. The pavement is considered as an elastic layered system with simplified structure and loading condition. A five layers structure was considered in the study. The load included in the modeling is a standard vertical dual-wheel load. And in addition, a horizontal load simulating friction force combined with the vertical load was applied to the surface of the structure. The results indicate that the bond between the binder course and the base can reduce the life of the pavement structure by up to 80%. The pavement life was found to be sensitive to horizontal loading if the bond between surface layer and binder course is poor.

The second approach conducted to investigate the effect of bond conditions between pavement layers is the linear finite element analysis. The ANSYS program was used to perform the calculations. The materials were considered linear elastic and the distributed load was assumed static. The plain strain element PLANE82 was used to mesh the structure. To model the full slip at the interface, the model was set with free and unrelated horizontal displacement at the interface. The partial bond was modeled by inserting a thin, soft and fully bonded interlayer between the surface course and the binder course. Different interlayer thickness and stiffness were considered in the study. The modeling results, including maximum surface deflection, the maximum tensile strain
at the bottom of the bituminous layers and the maximum compressive strain on the top of the subgrade, are compared to the BISAR calculation results. The overall trend of the results shows agreement with each other. The bond between the surface layer and underlying layer play an important role if the horizontal traffic loads applied.

Ozer et al. (2008) developed a friction model to characterize pavement layer interfaces. Zero thickness interface elements were formulated and implemented in the FE framework. The tangential and normal tractions on the interface plane were used to describe the interface behavior. The strength degradation was formulated based softening rules. The coupled softening mechanism can take different failure modes into consideration. An elasto-plasticity equation was implemented to model interface tractions and displacement jumps. Different loading conditions, including pure shear, pure shear, shear with tension and shear with compression, can be considered in the model. The calculate results show that the model is suitable for interfaces under various loading conditions.

In the 3D problems, an interface element was inserted to the brick element to represent the interface to evaluate the sensitivity of the interface model. The failure in this case is tensile separation model. The post peak softening behavior was captured by using Riks method. The parameters used in the model were optimized by matching the test results with the simulation results. For the interface between the HMA and the concrete, the experimental results conducted by Donovan et al. (2000) were used to develop a 3D interface model. Yield surface definition was determined in this model by performing a parametric study. The parameters were optimized by matching the maximum shear stress from the test results with that form modeling. The calculation
results, including load displacement, dilation and maximum shear stress, agreed with the test results.

Baek and Al-Qadi (2009) used three dimensional finite element modeling to investigate the fracture behavior of HMA overlays. A pavement structure consists of a 57 mm thick overlay over a 200 mm thick jointed plain concrete pavement (JPCP) was used to develop the model. The JPCP used to create the model is 6.0m long and 3.6m wide. A cohesive zone model (CZM) was introduced to model the crack progress directly. The cohesive elements are inserted directly over the transverse joints, which are the potential reflective cracking locations. The linear brick elements with eight node and reduced integration point were used for pavement layers. Two interface conditions were considered in the model. One condition is that the wearing surface and leveling binder are full bonded. Another interface condition is that the other layers in the model are assumed partially bonded. Material properties from laboratory complex modulus and disk-shape compact tension tests were used in the model. The fracture energy and the tensile strength were determined using DCT test. Two interlayer systems, sand mix and steel netting interlayer, were selected in the simulation.

The modeling in this study shows that the reflective cracking occurs in the middle of a leveling binder and propagates upward for a pavement without interlayer system installed. While for the pavement system with sand mix interface, reflective cracking jumps to the wearing course at an early stage. For the pavement overlay with the steel netting, the fracture is significantly reduced since the steel netting will reduce tension at the bottom zone of the leveling binder.
Hu and Walubita (2011) used a three-dimensional finite-element program to characterize the effects of layer interfacial bonding conditions on the mechanistic responses in asphalt. Two interfacial bonding conditions, fully bonded and debonded, were considered in the model. The actual measured vertical tire pavement contact pressure (TPCP) and assumed horizontal pressure were incorporated in the model. Three typical pavement structures, flexible base layer pavement, semirigid base layer pavement and PCC base layer overlay pavement, were conducted in the modeling. The 3D finite element program ANSYS was used to do the calculation. Calculation results, including the maximum tensile strains and horizontal shear strains at the surface and the bottom of the wearing course, the maximum tensile strains and horizontal shear strains at the bottom of the asphalt binder layer, the compressive strains on top of the subgrade and the maximum shear stress in the asphalt concrete layer, were used to evaluate the effects of interface bonding condition.

The simulation results indicate that the layer interfacial bonding condition hardly influence the mechanistic responses at the surface of the pavement. While, the tensile strains at the bottom of the wearing course were significantly affected by the bonding condition, particularly for flexible base pavement structures. The influence of the layer bonding condition on the shear strains at the bottom of the AC layer is a function of the base layer modulus. The effects of bonding condition on the maximum tensile strains at the top of the binder layer are negligible. Bonding condition has a significant effect on the maximum shear stress in the asphalt concrete layers. The change of the maximum tensile and shear strains at the bottom of the asphalt binder layer is similar when the bonding condition changed from fully bonded to separate. Debonding of the interface
affect the pavement distresses such as slippage cracking, fatigue cracking, shear deformation and rutting.

**2.3.3 Summary**

The interface bonding conditions between pavement layers have a significant effects on the service life of the pavement structures. Poor bonding conditions may result in serious problems such as top-down cracking, reflective cracking, and premature failure. Significant efforts have been done to investigate the effects of the interface bonding conditions in pavement layers on the structure response and pavement performance in both experimental and numerical way.

Finite element (FE) analysis is one of the most commonly used numerical approaches in modeling interface. Different from the laboratory test methods, numerical method can describe the material’s constitutive behavior in the representative volume element (RVE). The stress-strain behavior of the material in the RVE can be defined continuously for each load increment. This approach allows the accurate evaluation of the effects of changes in the layer characteristics such as material stiffnesses, bonding conditions and structure deformation due to the damage growth in the pavement. With the improvements in computing power and numerical techniques, the numerical approach will be an efficient tool to demonstrate the effects of changing testing conditions and mixture design parameters on the behavior and performance of asphalt concrete.

**2.4 HMA Fracture Mechanics Model**

Recently, Roque and Zhang (2001, 2002) developed the HMA fracture mechanics model. In this model, a suitable crack growth law was developed to predict the initiation and growth of cracking in asphalt mixture. The HMA Fracture Mechanics Model is
based on the fact that asphalt mixture has been determined to have a dissipated creep strain energy threshold. Once the threshold is exceeded, a crack will initiate or propagate.

In this model, it was found that the threshold is independent of loading mode or loading history. The development and propagation of cracks in asphalt mixture can be determined by calculating the dissipated creep strain energy for any loading condition. The crack will develop or propagate in any region where the dissipated creep strain energy exceeds the threshold.

The mixture properties from the Superpave IDT, Resilient modulus, creep compliance power law parameters $D_1$ and $m$-value and the tensile strength, are required to do the calculation using HMA Fracture Mechanics Model. Tensile strength and the dissipated creep energy threshold can be obtained from the stress strain response due to the constant rate loading test. The tensile stresses within the zone of the asphalt mixture were used to calculate the number of cycles to initiate or propagate a crack along that zone.

**2.5 Cracking Mechanism**

**2.5.1 Top-Down Cracking Mechanism**

The top-down cracking, which initiates at the surface of the pavement and propagates downward, has been reported to occur in many parts of the United States (Roque and Ruth 1990, Mayers et al. 1998, Uhlmeyer et al. 2000, Kim 2005). Traditional fatigue mechanism, which considers failure initiates at the bottom of the asphalt surface layer, cannot explain the top-down cracking phenomenon. Conventional multilayer pavement system subject to a circular, uniformly distributed vertical loading will not predict maximum tensile stresses and strains near the pavement surface.
For years, many efforts have been made to identify the mechanisms that lead to top-down cracking. The first factor that causes the top-down cracking is the near surface tensile stress and strain induced by the non-uniform contact stress between the tire and the pavement surface. Study performed by Myers et al. (1998) indicates that tensile stresses are yield under the threads of the tire. Jacobs et al. (1995) reported that the most significant tension is found under the widest thread.

The second factor that induces surface tensile stress lead to top-down cracking may be the thermal effects. Research conducted by Dauzats and Rampal (1987) showed that thermal stress in the asphalt surface layer may initiate top-down cracking. This viewpoint also supported by Myers et al. (1998) and Kim (2005). Their studies indicate that thermal stresses developed at the surface layer of the pavement affect the initiation and propagation of the surface cracks.

Aging hardening is considered as another factor that may lead to the top-down cracking. Svasdisant et al. (2002) reported that pavement with higher surface modulus due to aging hardening has higher potential for top-down cracking. The surface layer is more affected by the environment than underlying layers. The aging due to the exposure and oxygen will increase the stiffness of the surface layer.

Another factor might be the stiffness gradients caused by the temperature change, age-hardening and cooling rates. Stiffness gradients may lead to tensile response in the surface layer of the pavement. And also the construction quality may affect the surface cracking.

It is obvious that the surface layer properties have high effects on the top-down cracking. The tensile stresses that developed in the surface layer and lead to the
surface cracking are significant affected by the material characteristics. In Florida, the top layer of pavement is consisted of a thin Open Graded Friction Course (OGFC). This layer may be the first one in resisting top-down cracking. So to accurately understand and capture the effects of OGFC on top-down cracking is an important task.

2.5.2 Reflective Cracking Mechanism

Reflective cracking is a cracking develops in pavement overlays above a discontinuity. The driving force of the reflective cracking is the stress in the HMA overlays induced by the movements at the cracks and joints due to the thermal and mechanical loading.

Temperature loading is a cause that leads to reflective cracking. Periodic temperature change result in repeated contraction and expansion. The contraction will create tensile stress in the overlay. Reflective cracking develops when the tensile stress exceeds the overlay strength. Daily pavement temperature changes vary by depth. Temperature gradient in the pavement results in higher thermal stresses. Reflective cracking may occurs either at the bottom or on top of the asphalt concrete overlay (Castell et al. 2002, Song et al. 2006).

Traffic loading can cause vertical and horizontal movement at the cracks and joints. Tensile and shear stresses were developed at the bottom of the overlay due to the bending and movements induced by the traffic load. The support conditions in the layer underlying the PCC slab may also lead to shear and bending stresses. Different from the thermal stresses, the stresses induced by traffic load occur quickly and may lead to more quick reflective cracking.
CHAPTER 3
CONTINUUM ANALYSIS OF INTERFACE EFFECTS

3.1 Background

Pavement structures are comprised of asphalt mixture layers with a bonding at the interface. The study of the in-service pavements indicates that the bonding conditions between the pavement layers have significant effects on the performance. Inadequate bonding may lead to distresses such as delamination, top-down cracking and premature deformations. Appropriate bonding conditions between pavement layers are essential to guarantee a longer pavement life and better performance.

Continuum analysis is always performed to evaluate the interface effects on the pavement performance. In the continuum analysis, the mixture is represented as a homogeneous continuum material on a larger scale than the flaw size. Constitutive model of macroscopic mechanical behavior of materials is provided without the microstructural details. The computing of continuum model is more efficient.

3.2 Near-Surface Stress States

The Top-down cracking has been found to be a predominant mode of distresses of asphalt pavements in Florida. The Top-down cracking is near-surface phenomenon which affects the top part of the asphalt concrete layer. Therefore, the near-surface distresses are greatly related to the near-surface stress states.

The study of tire-pavement model conducted by Wang (2009) shows that the tensile stress may be produced due to high shear stress at tire groves, as shown in Figure 3-1 and Figure 3-2. While if the load was considered as uniform load, there is no tensile stress occurs near surface.
The load response model was performed by Zou (2010) to predict bending-induced maximum surface tensile stresses. The bending-induced tensile stresses at the pavement surface were predicted. Figure 3-3 shows the bending-induced surface tensile stress in the AC layer. Zou also indicates that the transvers thermal stress contributes to top-down cracking. Figure 3-4 shows the schematic plot for thermal response in the AC layer.

Figure 3-1. Line contours of principal stress in the tire-pavement model (Photo courtesy of Wang 2009)

Figure 3-2. Principal tensile stresses for different tires (Photo courtesy of Wang 2009)
3.3 Continuum Analysis of Different Layers Combination

The objective of the continuum analysis is to evaluate the contribution of OGFC and the bond between the OGFC and underlying layer to the resistance of top-down cracking model. In the continuum analysis of the mixture structure, the FEM model was created to capture the stress distribution in the structure. Then the Florida HMA fracture mechanics was implemented to calculate the load cycles needed to create the different size cracks. The crack resistance of the structure was evaluated based on the calculated load cycles with the length of the cracks.

3.3.1 Models of Different Layers Combination

Based on the introduction of near-surface stresses distribution above, four cases of the pavement layer combination, as shown in Figure 6-1 to 6-4, were considered:

1. Dense graded asphalt mixture,
2. Dense graded asphalt mixture with 1 inch OGFC on top,
3. Dense graded asphalt mixture with 1 inch saturated OGFC on top and

4. Dense graded asphalt mixture with 0.4 inch saturated OGFC and 0.6 inch OGFC on top.

Case 1 is the top of the pavement without the friction course. Case 2 is the pavement with the OGFC on top. The OGFC and underlying dense graded asphalt concrete are bonded with conventional tack coat. Case 3 and Case 4 is to simulate the structure with OGFC on top. The layers are bonded with Novabond®.

Figure 3-5. Case 1: Dense graded asphalt mixture

Figure 3-6. Case 2: Dense graded asphalt mixture with 1 inch OGFC on top
In Case 3 and Case 4, the saturated OGFC is included to present the bond agent permeation. The application rate of Novabond® is generally much higher than the conventional tack coat. During the construction of the pavement, the bond agent will permeate into the open graded asphalt concrete due to the high void ratio of the friction course. The permeation of the agent will create saturated OGFC. Case 4 represent 0.4 inch of the OGFC is saturated. Case 3 represent all the OGFC is saturated.
In the model, the asphalt mixture layers were assumed homogenous and isotropic. The layers are thought as full bonded with each other. The pressure 100psi is applied. The thickness of dense graded asphalt concrete in the model is 6 inches. The dimension of the model in the tension direction is 18 inches. The plain stress 8-node elements were used in the analysis. On the top of each pavement structures, a half hole is created in the middle to create the stress concentration.

### 3.3.2 Calculation Procedure

In the HMA fracture mechanics model, a fundamental crack growth law was developed that can be used to predict the initiation and propagation of cracking. This law is based on the fact that the asphalt mixture has a dissipated creep strain energy threshold. Once the damage in the mixture over the threshold, the crack will initiate or propagate.

The stress distribution is calculated using finite element method (FEM) to calculate the dissipated creep strain energy for each load cycle. The load cycles to initiate or propagate the crack for each crack size are calculated based on the DCSE threshold. The stress distribution is then calculated for the structure with new crack size. The load cycles to crack the second zone are calculated. This process is repeated until the structure cracked. Each crack zone was set 0.005m. For each crack length of each structure, a FEM model was created and the stress distribution is calculated. The stress then is adjusted to satisfy that it cannot exceed the material strength.

### 3.3.3 Material Properties and Boundary Condition

The material properties are shown in Table 3-1. The OGFC and dense graded asphalt mixture properties were determined in the IDT strength test and creep test. And
the properties of saturated OGFC are taken from the IDT tests of previous work that has been done before.

3.3.4 Calculation Results

Figure 3-9 to Figure 3-12 are the calculation results of the four cases mixture structures using the Florida top-down cracking model. As shown in Figure 3-10, the load cycles to crack the first zone in the dense graded asphalt concrete are different. For the dense graded asphalt concrete, the load cycles to create the crack in dense graded asphalt concrete are 5450. For the OGFC on dense graded asphalt concrete, the load cycles are 7302. For the OGFC with bond on the dense graded asphalt concrete, the load cycles are 9221. And for saturated OGFC on dense graded asphalt concrete, the load cycles are 10289. The load cycles calculated show that when the OGFC is bond to the underlying layer, if the bond creates a 0.4 inch saturated OGFC, then the load cycles needed to create crack in the dense graded asphalt concrete will increase 26%, from 7320 to 9221. If the bond saturates all the 1 inch OGFC, the load cycles needed to create crack in the dense graded asphalt concrete will increase 41%, from 7320 to 10289.

Table 3-1. Material properties used in FEM calculation

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<td>4.4</td>
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</table>
Figure 3-9. Load cycles with crack length

Figure 3-10. Load cycles with crack length in the dense graded asphalt concrete
Figure 3-11. Load cycles to crack the first zone in dense graded asphalt concrete

Figure 3-12. The cycles needed to create 1 inch crack in the dense graded asphalt concrete
Figure 3-9 is the crack length change with change of the load cycles of the four different cases. The load cycles to initiate the crack in Case 2 and Case 4 are very close, which are 3473 and 3264 accordingly. It needs more load cycles to crack the Case 1 dense graded asphalt concrete and Case 3 saturated OGFC on dense graded asphalt concrete. For Case 1, it needs 5454 load cycles to crack the first zone. For Case 3, it needs 10289 load cycles to crack the first zone. As shown in Figure 3-9, it need more load cycles to get the same length crack for Case 1, Case 3 and Case 4, compared to the Case 2.

As shown in Figure 3-10, for the OGFC with bond on dense graded asphalt concrete and the saturated OGFC on dense graded asphalt concrete, the crack length increases quickly with the load cycles increase. It needs more cycles for these two cases to initiate and propagate to the same length compared to the structure of OGFC on dense graded asphalt concrete. For the OGFC on dense graded asphalt concrete, the crack grows quickly after the crack propagates into the dense graded asphalt concrete. While the growth rate for this structure is lower than other three cases. It needs more load cycles to crack each zone, and to crack 1.2 inch length in the dense graded asphalt concrete, the load cycles almost near that of the saturated OGFC and OGFC with bond on dense graded asphalt concrete.

The analysis results show that the add of layer OGFC, saturated OGFC and OGFC with bond will increase the load cycles that needed to crack the first zone in the dense graded asphalt concrete. To include an OGFC layer that without bond will not increase the resistance of top-down cracking. It will increase the crack propagation rate if the crack length is more than 0.4 inch in the dense graded asphalt concrete. The
adding of saturated OGFC layer and OGFC with bond will increase the resistance of top-down cracking. It needs more load cycles to create the same crack length in the dense graded asphalt concrete.

Figure 3-12 is the load cycles needed to create 1 inch crack in the dense graded asphalt concrete. The load cycles for Case 2 OGFC on dense graded asphalt concrete are 8688, which is the smallest one. And for Case 1 dense graded asphalt concrete, Case 3 saturated OGFC on dense graded asphalt concrete and Case 4 OGFC with bond on dense graded asphalt concrete, the load cycles are 10315, 10921 and 10890 correspondingly. This shows that the bond create 0.4 inch saturated OGFC will increase 25% the load cycles to create 1 inch crack in dense graded asphalt concrete.

Figure 3-13 to Figure 3-16 is to show the crack initiates and propagates of the 4 cases. For the Case 1 dense graded asphalt concrete and Case 2 OGFC on dense graded asphalt concrete, the crack initiates at the front of half hole and crack each zone one by one in front of the crack tip. For Case 3 saturated OGFC on dense graded asphalt concrete, the crack initiates on the top of the dense graded asphalt concrete, as shown in Figure 3-15. After the crack propagates to the fifth zone in the dense graded asphalt concrete, the zone in front of the half hole in the saturated OGFC cracked. And then the crack in the dense graded asphalt concrete propagates again. For the Case 4 OGFC with bond on dense graded asphalt concrete, the crack initiates at the front of the half hole in the OGFC, as shown in Figure 3-16. And the crack initiate on the top of the dense graded asphalt concrete after the crack in the OGFC propagate to the saturated OGFC. After the crack propagate to the 4th zone of the dense graded asphalt concrete,
the first zone of the saturated OGFC cracks. Then the crack continues propagating in the dense graded asphalt concrete.

### 3.4 Summary of Continuum Analysis

The continuum analysis of the four case mixture structures was conducted using FEM and Florida HMA fracture mechanics. Different structures and different bond agent were included in the modeling to evaluate the effects on cracking performance. Based on the discussion of the calculate results, the follows key points of the analysis can be summarized.

The OGFC does not help to increase the resistance of the structure to top-down cracking. Instead, it will accelerate the initiation and propagation of the crack.

The bonded OGFC will increase the load cycles needed to crack the same size crack in the structure about 25% to 41%, that means the bonded OGFC structure have 25% to 41% higher top-down cracking resistance.

Figure 3-13. The propagation of the crack of case 1: Dense graded asphalt concrete
Figure 3-14. The propagation of the crack of Case 2: OGFC on dense graded asphalt concrete

Figure 3-15. The propagation of the crack of case 3: Saturated OGFC on dense graded asphalt concrete
Figure 3-16. The propagation of the crack of Case 4: OGFC with bond on dense graded asphalt concrete
CHAPTER 4
MULTI-SCALE ANALYSIS OF INTERFACE EFFECTS

4.1 Background

In the continuum analysis of different layer combinations, the materials are considered as homogeneous and isotropic. The bond between the layers was thought as full bonded without sliding and separating. However, the asphalt mixture structures are complex composites that consist of asphalt binder, aggregate particles and air voids. Effective material properties are not sufficient representation of the composite response, as it averages the fluctuations in stress and strain distributions caused by the wide range of particle sizes and the stiffness differences of materials in the constituents. The local distribution and fluctuations of these stresses and strains are important factors that influence the overall mix response, especially at the microscopic level. The macroscopic behavior of the composite is determined by the properties of these constituents. Studies have reported that considering the internal structure in constitutive models is essential to describe the behavior of the composites. As stated by Graham and Baxter (2001), assuming homogeneous response of a composite material represented only by effective properties ignores the local microscopic response often associated with failure phenomena.

As presented above, the properties of OGFC and saturated OGFC from Superpave IDT tests are totally different. These results have been discussed also
by B. Birgisson and R. Roque (2006) in the report of Evaluation of Thick Open Graded and Bonded Friction Courses for Florida. The high resistance to top-down cracking of saturated OGFC and OGFC with bond on dense graded asphalt concrete, as discussed above, is due to the structure difference that a thick bond was placed between the OGFC and the dense graded asphalt concrete and the saturated OGFC has high DCSE limit. To study the effect of the interface layer and other micro mixture characteristics such as the gap of the aggregate particles, the contact size of the particles and the underlying layer and the mastic modulus, the multi-scale models were created to capture the stress distribution for the different conditions and the effects of each factor. And then the Florida top-down cracking model is used to calculate the relationship between the load cycles and the crack length.

4.2 Objective of Multi-Scale Analysis

The objective of multi-scale analysis is to evaluate the effects of microstructure characteristics to the stress distribution. Then the stress in the microstructure was captured and the Florida HMA fracture mechanics was implemented to predict the load cycles with crack length. The resistance to the top-down cracking were evaluated based on the load cycles analysis.

4.3 Multi-Scale FEM Model

The multi-scale FEM analysis is conducted to evaluate the effects of the OGFC to the fracture resistance of the pavement system.
A finite element model was used to predict the global pavement response, which defined the boundary conditions for the micro-mechanical model used to predict the local response of the OGFC and the interface. For the local response of the micro-mechanical model, different contact conditions between the aggregate particles and the underlying layer is assumed and calculated. These conditions include the gap between the aggregate particles, the contact between the particles and the AC layer. The aggregate particles are not perfect sphere and have different size faces, so the contact between the particles and the underlying asphalt concrete layer may be different. And the conditions also include the thickness of the layer that bonds the aggregate particles and the underlying layer together. For the interface without bond, the contact between particles and underlying layer is more like penetration or inserting into the AC layer, which will cause much higher friction between these two. While, for the highly polymer-modified Nova-bond or tack coat, it may create a relatively thin, but highly effective layer that allows the aggregate to move independently of the HMA surface, thereby change the stress state on the top of the HMA layer.

The model is created based on the structure of the pavement with OGFC on top, as shown in Figure 4-1, which is part of the OGFC pavement system. Figure 4-2 is the two dimension multi-scale model. The size of the model is based on the direction tension test of the OGFC composites. In Figure 4-2, (a) is the global model that the OGFC and the underlying layer were taken as homogenous
material with 6 inch length and 2 inch thickness, 1 inch OGFC and 1 inch underlying asphalt concrete.

Figure 4-2 (b) is part of the global model, in which the OGFC is taken as non-homogenous material that composed of aggregate particles and mastic, which includes the fine aggregates, asphalt and air voids. The aggregate particles are simplified as sphere with different diameters.

Figure 4-2 (c) is the local model that composed of the two half same size aggregate particles, mastic and the underlying AC layer. The local model is part of the global model with 0.4 inch height and 0.2 to 0.25 inch width, depends on the gap between the two particles. The particle diameter is assumed 0.2 inch, so the half is 0.1 inch.

![Figure 4-1. Structure of OGFC](image)

The global model and the local model are meshed using different elements. For the global model, the quadrangle elements are used. And for the local model,
the aggregate particles and the underlying AC layer are meshed use quadrangle elements; while the mastic is meshed use triangle element since it has very sharp angle. Figure 4-3 is the mesh of the local model.

![Mesh of the local model](image)

Figure 4-2. The multi-scale model

The materials of the global and local model are considered as elastic. The parameters of the model are shown in the Table 4-1. The parameters used here are assumed based on the lab tests. The purpose of the model is to evaluate the contribution of the OGFC to the fracture resistance of the pavement system, so the exactly parameters value are not required.

The displacement of 0.0019 inch is applied on the global model. The stress, strain and displacement in the model are calculated. The boundary conditions,
displacement, of the local model are taken out from the global model calculation results, and applied on the local model. The stress distribution of the local model is calculated. As shown in Figure 4-5, the horizontal tensile stress distribution along the vertical center line in the local model and corresponding line in the global model are totally different. The horizontal stress distribution in global model is very even for the same material, while in local model, due to the micro structure, there is stress concentration in the middle where the gap between the two half particles are smallest. This difference shown that the micro mechanics is more accurate to capture the real stress happened in pavement system.

### 4.4 Microstructure Characteristics Effects

The model is run for different conditions to get the stress distribution in the OGFC and underlying asphalt concrete. For the gap between the two half particles, the 0.002, 0.01, 0.02, 0.05, 0.1 inch are used to check the gap effect. The different bond thickness, no bond, 0.01, 0.002 inches, is used to check the bond thickness effects. For the aggregate particles may have different size faces, the contact between the particle and the underlying asphalt concrete are different. The contact size 0, 0.01, 0.02, 0.05, 0.1 inches is used in the model. And also, the different mastic modulus is used to check if it has effects on the stress distribution.
Table 4-1. Material properties used in the multi-scale analysis

<table>
<thead>
<tr>
<th>Material</th>
<th>E (psi)</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Underlying AC</td>
<td>500000</td>
<td>0.4</td>
</tr>
<tr>
<td>Aggregate Particles</td>
<td>7000000</td>
<td>0.2</td>
</tr>
<tr>
<td>Bond</td>
<td>10000</td>
<td>0.45</td>
</tr>
<tr>
<td>Mastic</td>
<td>50000</td>
<td>0.4</td>
</tr>
<tr>
<td>OGFC</td>
<td>250000</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Figure 4-3. Mesh of the local model

4.4.1 Effect of the Particle Gap

The gap size 0.002, 0.01, 0.02, 0.05 and 0.1 inch are modeled to check the gap effect. The local model is created from the global model with the different gap size. The accordingly boundary conditions calculated from global model are applied to the local model. The stress in the local model is calculated.
Figure 4-5 is the stress distribution on top of the underlying AC layer. The highest stress on the top of the underlying AC layer is around 190 psi. And in the global model, the stress is about 163 psi. So in the micro-mechanic model, considering the different components separately results in about 15.6% stress increase on the top of the underlying asphalt concrete.

As shown in the Figure 4-6, with the change of the gap size between two aggregate particles, the highest stress on top of underlying asphalt concrete layer does not change so much, that means the gap between two aggregate particles doesn’t have significant effect on the stress concentration on top of underlying layer.

4.4.2 Effect of the Bond Thickness

The different bond conditions between OGFC and the underlying AC layer may be described as shown in the Figure 4-7. For (a), there is no bond layer between the OGFC and the underlying layer, the contact is more like a penetration or inserting into the underlying layer, which may result in a high contact friction. And for (b), the two layers are bonded by a thin layer that might offer a high reduction of the stress concentration on top of the underlying AC layer by allowing the smooth moving of the bottom of OGFC layer.

Different bond thickness, 0, 0.002, 0.01 inches is inserted into the global and local model to check the effect of different bond conditions. The gap between aggregate particles used in the model is 0.02 inch, as shown in Figure 4-8.
Figure 4-4. The difference of global model and local model

Figure 4-5. The stress distribution on the top of the underlying AC Layer
The calculation results of stress distribution on top of the underlying AC layer for different bond conditions are shown in Figure 4-9 and Figure 4-10. For no bond, the highest stress of local model is 190.337 psi, 16.6% higher than the global model, which is 162.812 psi. And if a 0.002 inch bond is added to the
model, the highest stress on top of the underlying AC layer 180.104 psi. And if 0.01 inch bond is added to the model, the highest stress on the top of the underlying AC layer is 168.355 psi. This results means the bond condition can affect the stress well, and a certain thickness bond can decrease the stress.

Figure 4-11 shows the horizontal stress distribution along the center vertical line in global and local model for different bond conditions. As shown in the Figure 4-11, with the bond thickness increase, the highest stress on top of the underlying asphalt concrete layer decreases dramatically. The highest tensile stress on top of the underlying asphalt concrete layer is close to the stress of the global model if the bond thickness is equal to 0.01 inch.

Figure 4-8. The model used to calculate the stress for different bond thickness
Figure 4-9. Highest horizontal tensile stress change of different bond thickness

Figure 4-10. The stress distribution on top of the underlying AC layer
4.4.3 Effect of Contact Size

Since the aggregate particles are not perfect sphere, they have different size faces. So the contact between aggregate particles and the underlying asphalt concrete may different. The different contact conditions are modeled using different contact size in the local model as shown in Figure 4-12. The contact sizes 0, 0.01, 0.02, 0.05 and 0.1 inches are modeled in the local model. Figure 4-13 shows the mesh of different contact condition local model. The stress of each model is calculated.
Figure 4-12. The model used to calculate the stress for different contact conditions

Figure 4-13. Mesh of the different contact conditions

Figure 4-14 shows the calculation results of the horizontal stress distribution on top of the underlying asphalt concrete. As shown in the Figure 4-14, with the increase of contact size, the highest horizontal stress on top of the underlying asphalt concrete layer increases. When the contact size is 0, that means the particle just has one point connected with the underlying asphalt concrete layer,
the highest horizontal stress is about 200 psi. When the contact size is 0.05 inch, the highest horizontal stress increased to 240 psi. While the contact size is 0.1 inch, and the aggregate particle become a square, the stress increases to 730 psi. The calculation results show that the contact size has high effect on the highest stress on the top of underlying asphalt concrete layer.

Figure 4-15 shows the horizontal tensile stress distribution on top of the underlying asphalt concrete for different contact conditions. As shown in the Figure 4-15, for the global model, the horizontal tensile stress distributes evenly. When comes to the local model, even the contact is just one point, the horizontal tensile stress distribution is totally different. The highest horizontal tensile stress concentrates quicker with the increase of the contact size.

![Graph showing highest horizontal tensile stress distribution](image)

Figure 4-14. Highest horizontal tensile stress distribution
Figure 4-15. Horizontal tensile stress distribution on top of the underlying AC

Figure 4-16. Horizontal tensile stress distribution on the center vertical line for different contact conditions
4.4.4 Effect of the Mastic Properties

The different mastic modulus are used in the model to check if the mastic properties have effect on the stress concentration on top of the underlying asphalt concrete layer. Different mastic modulus 5000, 10000, 20000 and 50000 psi are modeled. The results are shown in the Figure 4-17 and 4-18. The local model is calculated for two contact sizes, 0 and 0.02 inches. As shown in the Figure 4-17 and Figure 4-18, the highest horizontal tensile stress on top of the underlying asphalt concrete does not change much with the increase of the mastic modulus for two contact conditions.

![Graph showing stress concentration for different mastic modulus](image)

Figure 4-17. Highest horizontal tensile stress change for different mastic modulus (Contact=0)

4.5 Crack Growth for Different Bonding Conditions

Three cases, as shown in Figure 4-19 and Figure 4-20, were considered:

1. OGFC on the dense graded asphalt concrete.
2. OGFC with bond on the dense graded asphalt concrete.
3. OGFC with bond and film on dense graded asphalt concrete. The film in Case 3 is 0.01 inch thickness bond that bond the OGFC and underlying dense graded asphalt concrete layer. Since the thick bond will permeate into the OGFC and form the saturated OGFC, the 0.2 inch saturated OGFC layer is used in Case 2 and Case 3. A 0.8 inch crack was pre-created in the OGFC. The displacement 0.0019 inch (create around 136psi average stress) was applied to the global model. The stress distribution in the structure is calculated using FEM. The boundary conditions from the global model are applied to the local model. The stress distribution of the local model was calculated using FEM. The material properties are the same as Table 3-3.

![Diagram](image)

Figure 4-18. Highest horizontal tensile stress change for different mastic modulus (Contact=0.02 inch)
The load cycles needed to create the structure in the local model are calculated using the Florida top-down cracking model. Since the size of the model is small compared to the structure of continuum analysis, the cracking zone size is set as 0.04 inch in the local model. For each crack length, the boundary conditions of the local model are calculated from the global model with the same size crack length. The stress
distribution before the crack tip is then calculated using FEM. The stress distribution is used to calculate the load cycles needed to crack the next zone.

Figure 4-22, Figure 4-23 and Figure 4-24 are the calculation results of these two structures. As shown in Figure 4-22, the Case 2 OGFC with bond on dense graded asphalt concrete need more load cycles to create the same length crack. For Case 1 OGFC on dense graded asphalt concrete, it needs 3321 load cycles to crack the first zone. While for Case 2 OGFC with bond on dense graded asphalt concrete and Case 3 OGFC with bond and film on dense graded asphalt concrete, it needs 5714 load cycles to crack the first zone. The bond and film increase the load cycles 72% to crack the first zone.

To create a 0.2 inch crack in the local model, it needs 8340 load cycles for Case 1 OGFC on dense graded asphalt concrete. For Case 2 OGFC with bond on dense graded asphalt concrete, it needs 12735 load cycles to create a 0.2 inch crack in the local model. For Case 3 OGFC with bond and film on dense graded asphalt concrete, it needs 13011 load cycles to create a 0.2 inch crack in the local model. The bond increases the load cycles 53% to create 0.2 inch crack. The bond and film increase 56% to create 0.2 inch crack.

To create a 0.4 inch crack in the local model, Case 1 OGFC on dense graded asphalt concrete needs 11495 load cycles; Case 2 OGFC with bond on dense graded asphalt concrete needs 13623 load cycles; Case 3 OGFC with bond and film on dense graded asphalt concrete needs 14679 load cycles. Case 2 increase the load cycles by 19%. Case 3 increase the load cycles by 28%.
4.6 Summary of Multi-Scale Analysis

The effects of microstructure to the stress distribution were evaluated using the multi-scale model. The load cycles needed to create crack in the local model were calculated using the Florida HMA fracture mechanic. The follows are the summarized key points of the multi-scale analysis.

The microstructure characteristics of the OGFC, such as the particle gap, film thickness, mastic modulus, have large effects to the stress distribution in the structure.

The multi-scale analysis shows that the bond between the OGFC and underlying layer will increase the load cycles 19% to 72% to create the same size crack.

The bond and film will increase the load cycles 28% to 72% to create the same size crack.

![Graph showing loading cycles with crack length](image_url)

Figure 4-21. Loading cycles with crack length
Figure 4-22. Loading cycles to create 0.2 inch crack in the local model

Figure 4-23. Loading cycles to create 0.4 inch crack in the local model
CHAPTER 5
MODELING OF TEST OF INTERFACE EFFECTS ON TOP-DOWN CRACKING

5.1 Background

A symmetrical composite specimen test system was developed by Chen (2011) to evaluate the effects of the bond interface between the pavement layers on the top-down cracking performance. The test results show that Polymer modified asphalt emulsion, like Novabond®, increases the reflective cracking performance of composite specimen when compared with conventional tack coat. Novabond® interface application rate plays a significant role on the resistance of cracking. While the specimens with OGFC, which is bonded with dense-graded mixture using conventional tack coat, has lower cracking resistance compared with specimens without OGFC. The polymer modified asphalt emulsion is needed to bond the pavement layers to maintain a good cracking performance of pavements with OGFC overlay. The test results show that the system is effective to evaluate the relative effects of pavement layer interface characteristics on cracking performance.

However, the analysis of test results is dependent on the measured data from the tests. In this composite specimen test, the data measured during the test include loads from the loading heads and the strain gage displacement. It is hard for the test to capture the detailed stress-strain data across the specimen. To better understand the acting mechanism of the interface bond on the cracking performance, it is important to obtain the progress of crack initiation and propagation, as well as the stress-stain distribution, during the test.

The modeling of the test may provide a comprehensive understanding of the effects of the interface on the top-down cracking performance.
5.2 Symmetrical Composite Specimen Test

5.2.1 Development of the Test

Since pavement layer interface plays a significant role on the cracking performance in the overlay, it is important to have a test to evaluate the effects of the interface on the performance of the pavement layers. One possible test is the direct tension test on composite specimens. The newly developed Dog Bone Direct Tension test (DBDT) by Koh (2009) at the University of Florida has the potential to perform this test. While, the non-uniform stress distribution and complex stress state make the DBDT improper for composite specimen testing. A new composite specimen test is needed for the interface evaluation.

After a review of exiting composite tests, a two-layer specimen with 1 inch dense graded asphalt concrete and 1 inch OGFC was conceived. The specimen has a stress concentrator at the center of the OGFC layer. It was expected that the crack will initiate at the concentrator from OGFC layer. Monotonic strength tests performed on the specimen shows that it is not a good choice for interface evaluation since the testing time is too short to identify the crack initiation time. And also not as expected, the specimen broke from dense graded asphalt concrete under slow loading.

Repeated loading test then was performed on the two-layer specimen. The specimen broke near the loading head instead of the groove. The reason for this may be the end effect of the specimen. And also, the high void of the OGFC near the end may create a natural stress concentrator. To erase this effect, partial of the OGFC layer were sliced off. The test of the specimen with rough face of OGFC sliced off shows that the damage accumulate in OGFC and no damage was induced in the dense graded
asphalt mixture until the crack propagate to it. This feature can be used to evaluate the effect of the interface condition on cracking performance.

The thickness of the dense graded asphalt concrete in the specimen was increased to 3 inch to make sure the crack is still stable to evaluate the effect of the interface. It was found that the damage was accumulating in dense graded asphalt concrete due to the larger thickness carry more load. The increase of the thickness of the dense graded asphalt concrete was considered not appropriate to obtain stable crack propagation.

The thickness of the dense graded asphalt concrete was set back to 1 inch and a rectangular groove was introduced to the OGFC layer to create a high enough stress concentration so that the dense graded damage is caused by stable crack propagation. Test results show huge variations. The reason was determined to be the rectangular groove and the bending effects of the specimen.

To eliminate the bending effects, two single two-layer composite specimens were bonded at the OGFC surface with a spacer throughout the symmetric plane to create a symmetrical composite specimen. The spacer induced in the specimen functioned as a stress concentrator. Repeated loading was applied with the special made loading system. The test results showed that the crack initiate from the stress concentrator in the OGFC, and propagated through the interface. While, the effect of the interface conditions was not identified due to the loading methods.

To reduce the load carried by the dense graded mixture, it was determined that the load was applied on the OGFC mixture part only. The geometry was determined to be 1.5 inch wide, 2 inch thick and 3 inch in diameter due to a finite element analysis. A
new set of loading head with 3 inch diameter and 2 inch width was used to apply the load on the OGFC. The test results show that the shear stress induced in the OGFC reached its failure limit. The specimens were failed by shear stress, not the tensile stress as expected.

It was determined to apply the load internally on the concentration groove to obtain the tensile stress distribution that can initiate and propagate the crack across the interface. Monotonic loading was applied on the new loading system. The test results show that the specimen with tack coat failed at the symmetric plane due to the bending effect after the crack initiate from the bottom of the groove and propagate to the interface. The specimen with Novabond® failed due to the higher shear stress after the crack propagates to the interface.

In order to eliminate the effects of bending and shear, carbon fiber is used at the specimen side faces. The test results show that the crack propagated through the interface and into the dense-graded asphalt concrete. However, no difference was observed between specimen with tack coat interface and Novabond® interface due to the fracture energy. The reason seems to be that the Novabond® interface cannot release the stress accumulated under monotonic loading.

The repeated load was used to allow the stress release at the interface. The 6 inch diameter specimen was used to conduct the test for a longer shear path. The test results show that the testing configuration and loading mode is appropriate to evaluate the effects of the interface conditions on top-down cracking. And also, it can to be used to evaluate the effects of interface on reflective cracking if the cracked pavement layer is used in the specimen.
5.2.2 Specimen Preparation and Testing

The composite specimen preparation is a complex process. First, the dense-graded mixture was compacted using SGC to desired air voids. Slice the compacted dense-graded mixture specimen into the desired dense-graded mixture thickness to be used in the composite specimen. Then apply the conventional tack coat or Novabond® to the cut side of the dense-graded mixture. Place desired weight of open-graded mixture into the compaction mold on top of the base material and compaction mold was loaded to produce the composite specimen, as shown in Figure 5-1.

![Figure 5-1](image_url)

Figure 5-1. Compacted composite specimen (Photo courtesy of Chen 2011)

Next step is slicing the top and bottom of the compacted composite specimen to obtain the desired thickness for both open-graded and dense-graded mixtures. Then the sliced specimen was cut to the desired width using the diamond-tip saw, as shown in Figure 5-2.

The sliced specimens were bonded together at the OGFC surface to create a symmetrical composite specimen. The specimen was drilled at the center to create the groove for stress concentration, as shown in Figure 5-3. In order to make sure the
loading head and specimen curved end surface have solid contact, the specimen were sanded to get rid of the loose material until aggregates were fully exposed.

Four gage points were affixed with epoxy to each side of the specimen. Carbon fiber sheet was glued to the curved end surface of the specimen using epoxy, as shown in Figure 5-4. The completed composite specimen is shown in Figure 5-5.

Figure 5-2. Diamond-tip saw used for specimen slicing and cutting (Photo courtesy of Chen 2011)

Figure 5-3. Composite specimen stress concentrator drilling setup (Photo courtesy of Chen 2011)
Figure 5-4. Composite specimen end surface with glued carbon fiber sheet (Photo courtesy of Chen 2011)

Figure 5-5. Symmetrical composite specimen (Photo courtesy of Chen 2011)
The loading heads designed especially for the composite specimen are shown in Figure 5-6. Internal loading was applied on the specimen by the loading pins through the loading system.

![Composite specimen loading system](Photo courtesy of Chen 2011)

5.2.3 Test Results

The loading magnitude from the loading system and strain gage displacement can be obtained from the testing. Those data can be used to calculate the stress and strain in the specimen. More material property parameters can be computed based on the test data. The loading cycles to fail the specimen and the recoverable deformation can be used to evaluate the crack performance.

5.2.4 Summary

The composite specimen test developed here is a direct tension test with internal loading applied system. The loading system is designed to eliminate the shear failure and bending effects of the specimen. The test system can capture the effects of the interface bonding conditions on cracking performance by inducing the crack initiation at the OGFC layer and propagate through the interface and dense graded asphalt mixture.
until failure. It is effective test system to evaluate the interface and OGFC effects on cracking resistance.

5.3 Modeling of the Composite Specimen Test

5.3.1 Model Geometry of Composite Specimen with Tack Coat

The geometry of the composite specimen is shown as Figure 5-7. The specimen has four layers with 1 inch in thickness. A circle hole was created in the center of the specimen between two OGFC layers to apply the internal load. The crack was expected to initiate at bottom to the hole and propagates through the interface and into the dense graded asphalt concrete.

Half of the specimen geometry was taken to create the model, as shown in Figure 5-8. Since the application rate of the conventional tack is only 0.045 gal/yd², the layers were considered as fully bonded together without any permeation of the tack coat into the OGFC.

![Figure 5-7. Geometry of the composite specimen](image-url)
Figure 5-8. Model geometry of composite specimen with tack coat

5.3.2 Model Geometry of Composite Specimen with Novabond®

Similar to the geometry of composite specimen with tack coat, the specimen with Novabond® at the interface is also considered half of the test specimen due to the symmetric characteristics.

Since the application rate of Novabond® is 0.3gal/yd² at the interface bonding, it was assumed that the bonding material will permeate into the open graded friction course due to the high void ratio of the OGFC. It was determined that a bond layer will form at the interface and partial of the OGFC will be permeated with the bonding material to form a saturated OGFC.

Determination of the bond interlayer thickness. According to the aggregate gradation of the open graded friction course, the main aggregate particles are No. 4, which is 0.19 inch. During the compaction of the composite samples, the friction course will saturated with the Novabond® tack. The top part of the friction course will be considered as bond layer, as shown in the Figure 5-9. The representative thickness b was calculated based on the half size of the particles and the void ratio:

$$b = \frac{0.19}{2} \times \text{void ratio}$$

- b=0.19/2*0.15=0.01425 if void ratio is 15%
- b=0.19/2*0.2=0.019 if void ratio is 20%
The thickness of the bond interface was set as 0.015 inch based on the calculation.

In the modeling of the composite specimen with Novabond®, an interface layer will be included in the model.

![Diagram](image.png)

Figure 5-9. Thickness calculation of Novabond® tack as a bond layer

**Determination of the saturated OGFC thickness.** In the test, the application rate of Novabond tack is 0.3 gal/\(yd^2\), which is 69.3 cube inch over 1296 square inch. So the thickness of the Novabond® before compaction is 0.0535 inch. During the compaction of the test sample, the Novabond tack will permeate into the friction course to form a saturated OGFC due to the compress load, as shown in the Figure 5-10. The thickness of the saturated OGFC is calculated based on the void ratio as:

\[
0.0535/0.15=0.357 \text{ inch if void ratio is 15%} \\
0.0535/0.2=0.2675 \text{ inch if void ratio is 20%}
\]
The thickness of the saturated OGFC in the model was then set as 0.315 inch.

The final geometry of the model of composite specimen test with Novabond® is shown in Figure 5-11.

Figure 5-10. Thickness of the saturated OGFC

Figure 5-11. Model geometry of the composite specimen with Novabond®

5.3.3 Materials and Boundary Condition

Materials included in the model are shown in Table 5-1. Material properties at 10°C were used in the model since the test was run at 10°C. Two materials are included in the model of composite specimen with tack coat, including dense graded asphalt.
concrete and OGFC. Four materials are included in the model of composite specimen with Novabond®, including dense graded asphalt concrete, OGFC, saturated OGFC and the Novabond® at the interface. The modulus of the Novabond at 10°C was set at 10000 psi due to the asphalt test results.

Table 5-1. Material properties used in the model

<table>
<thead>
<tr>
<th>Material</th>
<th>Resilient Modulus GPA</th>
<th>Tensile Strength MPa</th>
<th>m-value</th>
<th>D1</th>
<th>DCSE kJ/m^3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dense (unmond) ST</td>
<td>10.85</td>
<td>2.14</td>
<td>0.668</td>
<td>4.77E-07</td>
<td>3.99</td>
</tr>
<tr>
<td>FC-5 (NGran) ST</td>
<td>6.93</td>
<td>1.17</td>
<td>0.557</td>
<td>8.38E-07</td>
<td>1.2</td>
</tr>
<tr>
<td>Saturated Novachip</td>
<td>6.95</td>
<td>2.04</td>
<td>0.72</td>
<td>3.23E-07</td>
<td>8.9</td>
</tr>
</tbody>
</table>

The peak load in the repeated loading test is 570 lbs. The load was applied on the right steel bar. The left steel bar was fixed in x direction as in the test. The interface and two pins were set fixed in y direction at the symmetric line due to symmetric characteristics of the model. Contact between the pin and the OGFC was set tangential frictionless to allow the pin move free with the deformation of the structure and reduce the damage around the pin.

Eight node quadrangle plane stress element were used to mesh the model, as shown in the Figure 5-13. Fine mesh was used in the area where there is a contact interaction. Convergence analysis was performed to determine the appropriate mesh size. The layers were considered as full bonded.

The tensile stress distribution in the pavement layer was obtained after running the model. The area where the tensile stress is highest was considered to crack first. Based on the HMA fracture mechanics, the load cycles needed to crack first zone were
calculated. The size of crack zone was assumed to be 0.2 inch. After the first zone cracked, a new model with a crack in the system was created to capture the stress distribution in the new system. And the tensile stress distribution was used to calculate the load cycles to crack the next zone. The above process is repeated for each crack zone until all the layers cracked.

Figure 5-12. Load and boundary condition of the model

For each crack size in above calculation, the load cycles can be obtained. And the model can also supply the deformation data corresponding to the gage points in the tests. Those data from the model can be used to compare with the test results to make sure the model is proper.
5.3.4 Modeling Results

Figure 5-14 shows the results of composite specimen test with conventional tack coat. The tests results show that the three specimens have the similar deformation under the repeated loading. The curves show a high deformation after the load applied. Then the recoverable deformation enters a stable development progress. High recoverable deformation was found at the end of the test until the specimen broken.

Figure 5-15 shows the model calculate result of the composite specimen with conventional tack coat test. The calculation result shows a big difference from the test results. The reason for this may due to the unrecoverable deformation that occurs at the beginning of loading. The test results of the composite specimen at the beginning show the evidence of this assumption. And also, from the test specimen, large deformation was found around the load area. It was considered that the damage was accumulated at the beginning of the loading due to the material characteristics. The damage become less after the loading was applied for a while, and the strength of the material around the loading area is increased. The recoverable deformation enters to a stable progress.

While in the model, the material was considered as elastic. The model does not take the damage into consideration. So the deformation in the model shows a stable development at the beginning. The recoverable deformation shows steady increase with crack initiates and propagates.

To eliminate the effects of the damage and make the results comparable, the non-elastic deformation needs to be removed. The way to do this is to take the test data at the stable state and fit the date with linear line to get the initial deformation value.

As shown in Figure 5-16, the average value from the fitting lines of initial recoverable deformation is 0.000367. The difference between this value and the initial
value of recoverable deformation in the modeling is assumed as the damage effects. It should be pointed out that the material properties used in the modeling of the composite specimen test are from the laboratory IDT tests. The resilient modulus for OGFC was used as the elastic modulus. While, it was generally thought that this value for OGFC is higher than the actual one. That means the real value may be smaller. And therefore, a higher recoverable deformation in the model at the beginning of the loading was expected. The results of the recoverable deformation in the model at time 0 is smaller than the test results, as shown in Figure 5-15. And at this time, there should have no damage in the specimen.

![Figure 5-14. Composite specimen test results with conventional tack coat](image-url)
Figure 5-15. Model result of composite specimen test with tack coat

Figure 5-16. Data fitting of the composite specimen test result
Figure 5-17. Adjusted calculate results of the composite specimen test

Figure 5-18 shows the typical total recoverable deformation curve and the definition of the damage rate. The curve is determined by the total recoverable deformation versus time. Part of the curve that the crack propagates steady was considered as steady state. The damage rate is determined by the slope of the steady state of the total recoverable deformation curves.

Figure 5-19 shows the damage rate for the composite specimen test with conventional tack coat and the model analysis result. The damage rates for composite specimen test are 1.35E-8, 9.87E-9 and 1.49E-8. The damage rate for the model analysis is 1.29E-8. The value of damage rate for the test and model analysis is very close except that the damage rate for test Tack 2 is a little bit lower Tack 1, Tack 3 and model analysis. The damage rate for the test shows agreement with that for model analysis.
Figure 5-18. Typical total recoverable deformation and damage rate

Figure 5-19. Damage rate of composite specimen test with conventional tack coat
Figure 5-20 shows the test results of composite specimen with Novabond®. The curves indicate a good agreement in the development of recoverable deformation under the repeated loading. The damage was accumulated at the beginning of the load due to the curves showed in the Figure 5-20. After the load was applied for a while, the curves show a stable development. The specimens were failed at almost the same time for all the three tests. The test results also show that the composite specimen tension test developed is an effective test to evaluate the bond interface effects on the cracking performance.

Figure 5-21 show the calculation result of the model for the specimen with Novabond®. Similar to the calculation result of the model for specimen with conventional tack coat, the calculate result is different from the test results. The same procedure performed for modeling result adjustment above was conducted to eliminate the effects of the damage. The average initial value is 0.000402 for the three tests.
Figure 5-21. Model result of composite specimen test with Novabond®

Figure 5-22. Data fitting of the composite specimen test result
Figure 5-23. Adjusted calculate results of the composite specimen test

Figure 5-24 shows the damage rate of composite specimen test with Novabond®. The damage rates for Bond 1, Bond 2 and Bond 3 are 7.74E-9, 8.05E-9 and 6.34E-9 respectively. The damage rate for model calculation results is 6.88E-9. The damage rates for both test and model analysis show agreement with each other.

5.3.5 Summary

The modeling of the tests for both composite specimens with tack coat and with Novabond® provides a method to evaluate the effects of interface bonding on the cracking performance. Although the modeling does not consider the damage accumulation during the test, it provides a method to compare the results from the modeling with the tests after eliminating the damage effects. The adjusted molding results proved that the assumption of the interface between the OGFC layer and the dense graded asphalt concrete is appropriate. The bond consisted of tack coat can be
considered as full bonded and no material properties change for both layers due to the low application rate. While for the Novabond®, the permeation was considered in the model due to the high application rate. Partial of the OGFC layer was filled with Novabond® and form a layer of saturated OGFC. The fracture properties for saturated OGFC are totally different from OGFC. That may be one explanation for good cracking performance for pavements with Novabond® as the bonding material.

![Figure 5-24](image_url)

**Figure 5-24.** Damage rate of composite specimen test with Novabond®

### 5.4 Summary of Test Modeling on Top-Down Cracking

In this chapter, the composite specimen direction tension test system developed by Chen (2011) was discussed. The test results indicate that the test system is effective in evaluation of the effects of interface bonding on cracking performance.

Modeling analysis was performed based on the test specimen geometry to evaluate the interface effects. The modeling is considered as elastic and does not consider the viscous and/or plastic deformation. The interface bonding conditions were
assumed. The specimen with conventional tack coat as bonding material was considered as full bonded at the interface. The specimen with Novabond® as bonding material was considered that a saturated OGFC layer and a bond layer forms at the interface due to the high application rate and large air voids. The modeling results were compared to the test results by adjusting the modeling results based the initial recoverable deformation value. The damage effects were eliminated in this process. It was found that the assumption of the interface bonding conditions for both specimen with tack coat and specimen with Novabond® is appropriate according to the comparison of the results from tests and modeling.
CHAPTER 6
MODELING OF TEST OF INTERFACE EFFECTS ON REFLECTIVE CRACKING

6.1 Background

A good bonding between the pavement layers can help reduce the stress transfer through the interface and increase the resistance to fracture distress. The effects of interface materials on reflective cracking performance have been evaluated in the composite tests. Modeling analysis of composite tests will be conducted in this chapter to understand the mechanism of the interface effects on reflective cracking.

6.2 Composite Test of Interface Effects on Reflective Cracking

To evaluate the effects of interface on reflective cracking, six composite specimens were prepared by Road Science in their laboratory. The composite specimen is symmetric structure that composed of four layers of dense graded asphalt mixture with different bond agent, as shown in Figure 6-1. Two materials, conventional tack coat and Novabond® were used as bond agent in the preparation of test specimens.

Figure 6-1. Composite specimen geometry
The tests were performed under repeated haversine loading model with the peak load 570lbs. Tests results shows that both of two bond materials shows almost the same fracture resistance. These results might be explained by the fact that low void ratio of dense graded asphalt concrete does not allow the bond agent migrate into the mixture. Another fact is the dense graded asphalt mixture has higher fracture energy. More DCSE was accumulated in the outer dense graded asphalt mixture when the crack initiated and propagated before it reached the interface. This leads to short time for Novabond® to have an effect.

The specimen was modified and a teflon spacer was introduced to create more effective concentrate stress, as shown in Figure 6-2 and Figure 6-3. Peak loads were set at 430 and 520lbs due to the teflon spacer. Test results show that the specimen with Novabond® as bonding material has higher fracture resistance than with conventional tack coat as bonding material.

Figure 6-2. Composite specimen with teflon spacer (Photo courtesy of Chen 2011)
6.3 Modeling of the Interface Effects on Reflective Cracking

6.3.1 Model Geometry of Composite Specimen

The geometry of the composite specimen is shown in Figure 6-2. Half of the specimen was taken to create the model since it is a symmetric structure. The model is shown in Figure 6-4.
6.3.2 Effects of Bond Interface

The application rate of diluted conventional tack coat and Novabond® are 0.1 gal/sy and 0.2 gal/sy respectively. The bond interface was considered as an interlayer in the model. The thickness of the bond interlayer is 0.018 inch for tack coat and is 0.036 inch for Novabond® due to the application rate. The model with bond layer is shown in Figure 6-5.

The properties of the material are the same as in Table 5-1. The peak load was set 520 lbs in the model. The load was applied on the right steel bar and the left bar are fixed. The boundary condition is the same as in the top-down cracking model analysis.

Eight node quadrangle plane stress element were used to mesh the model. Convergence analysis was performed to determine the appropriate element size. The layers were considered as full bonded.

Figure 6-5. Geometry of the model with bond layer

Tensile stress distribution in the numerical modeling was captured. The stress distribution was used to calculate the load cycles needed to crack the first zone based on the HMA fracture mechanics. The size of the crack zone was set 0.2 inch. After the first zone cracked, a new FEM model with the crack in the model was created. The
stress distribution was calculated and used to calculate the load cycles to crack the next zone. The process above is repeated for each crack zone until the model failed.

The data of load cycles and crack size can be obtained from the calculation. The deformation data can also be obtained from the FEM model.

Figure 6-6 shows the comparison of the number of load cycles to failure for diluted conventional tack coat and Novabond® between the test results and model calculation results. For diluted conventional tack coat, the load cycles to failure in the model analysis are near 10000 less than the test. For the Novabond®, the load cycles to failure in the model analysis are almost 37000 less than that in the test. The disagreement between the test and model analysis indicates that the assumption of the bond layer based on the application rate of the bonding agent is not suitable. The bonding agent cannot be considered just as a bond layer. The migration of the agent should be taken into account in the analysis.

Figure 6-6. Number of cycles to failure
6.3.3 Effects of Bond Agent Migration

The analysis above shows that the migration of the bonding agent should be taken into consideration in the model analysis. The reason might be that the high application rate of the bonding agent may lead to migration since the void ratio of the dense graded asphalt mixture before compaction is around 17%. This high void ratio may allow the agent migrates into the mixture during the compaction to form a saturated mixture near the interface. This saturated mixture should have better fracture performance.

Therefore, a new model with saturated mixture near the interface was created, as shown in Figure 6-7. The thickness of the saturated mixture was calculated based on the void ratio of dense graded asphalt mixture and the application rate of the bonding agent. For diluted conventional tack coat, the application rate is 0.1 gal/sy and the void ratio of the dense graded asphalt mixture is 4%. The thickness was determined to be 0.2 inch. For Novabond®, the application rate is 0.2 gal/sy and the void ratio of the dense graded asphalt mixture is the same. The thickness of the saturated mixture was determined to be 0.4 inch. The saturated mixture was considered as a layer in the model with different fracture properties.

Figure 6-7. Geometry of the model with saturated mixture
The boundary conditions and load are the same as in 6.3.2. All the layers in the model were considered fully bonded. The model was meshed with eight node quadrangle plane stress elements.

Since there is no test data for saturated dense graded asphalt mixture, the IDT test data of dense graded asphalt mixture were used in the analysis except for the DCSE limit. DCSE limit of the saturated dense graded asphalt mixture was assumed based on the test data of OGFC and saturated OGFC. Three DCSE limit values, 6 kJ/m$^3$, 7 kJ/m$^3$ and 8 kJ/m$^3$, were assumed and included in the model analysis.

The calculation process is the same as in 6.3.2. The load cycles for each crack zone and crack size can be obtained from the calculation. The displacement can also be determined in the model. Figure 6-8 to Figure 6-10 shows the number of load cycles to failure for different DCSE limits of saturated dense graded asphalt mixture.

![Figure 6-8. Number of cycles to failure for DCSE limit equals to 6 kJ/m$^3$.](image-url)
Figure 6-9. Number of cycles to failure for DCSE limit equals to 7 kJ/m$^3$.

Figure 6-10. Number of cycles to failure for DCSE limit equals to 8 kJ/m$^3$. 

The results show better agreement between the test and model analysis. For the DCSE limit of saturated dense graded asphalt mixture equals to 6 kJ/m$^3$, calculated load cycles to failure for conventional tack coat are around 3000 less than that for test. The load cycles to failure for Novabond® are around 43000 for model analysis and 57000 for test. For the DCSE limit of saturated dense graded asphalt mixture equals to 7 kJ/m$^3$, load cycles to failure for conventional tack coat are around 29000 for test and 27000 for model analysis. For Novabond®, the load cycles to failure are around 57000 for test and 44000 for model analysis. For the DCSE limit of saturated dense graded asphalt mixture equals to 8 kJ/m$^3$, load cycles to failure for conventional tack coat are around 29000 for test and 28000 for model analysis. For Novabond®, the load cycles to failure are around 57000 for test and 45000 for model analysis.

Based on the analysis above, the calculation results indicate a good agreement between the test and model analysis for conventional tack coat. For Novabond®, the results of load cycles to failure show relatively higher disagreement between test and model analysis. This may due to that the model didn’t take the properties of polymer modified asphalt emulsion into consideration.

Figure 6-11 shows the damage rate of test and model analysis. The damage rate for conventional tack coat and Novabond® show high agreement between test and model calculation. In the model analysis, the damage rate for both conventional tack coat and Novabond® decreases with the increase of DCSE limit of the saturated dense graded asphalt mixture.

6.3.4 Summary

The modeling analysis of the interface effects on reflective cracking was conducted in this section. The interface was considered as a bond layer first. The
thickness of the bond layer was determined based on the application rate of bonding agent. The results show difference between test and model analysis. This means the assumption of the interface are not appropriate. The migration of the bonding agent into the dense graded asphalt mixture during compaction should be considered in the model analysis.

The model with the migration of bonding agent was created. The migration of the bonding agent was considered to form a saturated mixture near the interface. The saturated mixture was considered as a layer with different fracture properties in the model. The thickness of the saturated mixture is determined based on the application rate of the bonding agent and the void ratio of the mixture.

Figure 6-11. Damage rate of test and model analysis
The calculation results show better agreement between the test and the model analysis. This indicates that the introduction of the new saturated mixture layer near the interface is reasonable.
CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Continuum and multi-scale analyses were performed to understand and evaluate the effects of interfacial characteristics between pavement layers on top-down and reflective cracking resistance. Model analyses of composite specimen tests were conducted to assess the validity of the models. The primary findings of the study are summarized as follows:

Continuum model analysis shows that the introduction of an OGFC layer with conventional tack coat on dense graded asphalt concrete will reduce the resistance to top-down cracking compared to a system without OGFC.

The bonded OGFC with polymer modified asphalt emulsion will increase the load cycles needed to create a same size crack in the dense graded asphalt concrete, which means the bonded OGFC has higher top-down cracking resistance.

High application rate of the modified asphalt emulsion will increase the load cycles needed to initiate and propagate crack because of the migration of the emulsion into the OGFC to form a thicker cohesive interface.

Multi-scale analysis indicates that the application of polymer modified asphalt emulsion, compared to conventional tack coat, will increase cracking resistance of the OGFC.

The model analysis of composite specimen test for top-down and reflective cracking indicates that high quality bonded interface between OGFC and underlying dense graded asphalt mixture, may increase the cracking resistance due to enhanced fracture properties near the interface.
After comprehensive evaluation of the analysis of bonded interface effects on top-down and reflective cracking, the following conclusions can be drawn:

The introduction of polymer modified asphalt emulsion as bonding agent between the pavement layers enhances the top-down and reflective cracking performance;

The bonded interface improves the top-down and reflective cracking performance by dissipating the stress near the interface or reducing the stress transmission through the interface;

The application of OGFC layer on top of dense graded asphalt mixture with poor bond may reduce the cracking resistance. Good bond should be guaranteed to obtain good cracking performance.

### 7.2 Recommendations

Based on the studies completed, future works needed to be conducted are summarized as follows:

Micro structure near the interface should be determined to obtain more accurate stress and strain characteristics in that area. Theory analysis such as packing theory and new technique such as the tomography can be used to determine the microstructure.

More accurate material model such as the viscoelasticity should be included in the future analysis to capture the stress and strain changes during the loading process.

Age hardening and healing effects, which has significant effects on cracking performance, should be considered in the future analysis.

Modeling analysis of effects of the road structure on both top-down and reflective cracking should be conducted and evaluated.
LIST OF REFERENCES


BIOGRAPHICAL SKETCH

Xingsong Sun was born in Yangzhou, Jiangsu Province, People’s Republic of China. He received a Bachelor of Science degree in civil engineering from Hohai University in 2002, and then worked at Beijing Water Authority until 2004.

In August 2004, Xingsong started a Master of Engineering program in civil engineering at the Tianjin University. After finishing his master’s degree, Xingsong came to the United States in 2006. He joined the Ph.D. program of the materials group at the University of Florida and worked as a graduate research assistant with his doctoral advisor, Dr. Reynaldo Roque. He received his Ph.D. from the University of Florida in the fall of 2011. After completing his Ph.D., he plans to work in academia, government agencies, or private companies in civil engineering.