Laboratory Investigation and Discrete Element Analysis in Open-graded Friction Course and Railway Crosstie-ballast Interaction

Weimin Song

*University of Tennessee, Knoxville, wsong8@vols.utk.edu*

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I am submitting herewith a dissertation written by Weimin Song entitled "Laboratory Investigation and Discrete Element Analysis in Open-graded Friction Course and Railway Crosstie-ballast Interaction." I have examined the final electronic copy of this dissertation for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, with a major in Civil Engineering.

Baoshan Huang, Major Professor

We have read this dissertation and recommend its acceptance:

Eric C. Drumm, Z. John Ma, David B. Clarke, Xiang Shu

Accepted for the Council:

Dixie L. Thompson

Vice Provost and Dean of the Graduate School

(Original signatures are on file with official student records.)
Laboratory Investigation and Discrete Element Analysis in Open-graded Friction Course and Railway Crosstie-ballast Interaction

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Abstract

Granular materials, such as ballast and aggregate, are widely used nowadays in civil and transportation engineering. Discrete element method has been extensively used to simulate the behavior of granular materials. In this study, properties of granular materials used in pavement and railway were investigated by laboratory tests and discrete element modeling.

Firstly, the bonding performance between pavement layers was evaluated. Open-graded friction course pavement and traditional dense asphalt mixture pavement were both explored. Two dense asphalt mixtures (D, BM) and one open-graded friction course (OGFC) mixture were selected for the comparison. The laboratory test results show that, for traditional dense samples, the interlock effect between layers played an important role in pavement layer bonding. For specimens composed of OGFC and a dense mixture (D or BM), OGFC-BM showed a better shear performance than OGFC-D, due to the double effects of a larger interface contact area and a larger interface roughness than OGFC-D. The DEM modeling was focused on the interlock effect between pavement layers by conducting the direct shear box test. Results from DEM modeling show that D-BM gave a higher shear strength, which agreed with the laboratory test.

Secondly, laboratory tests were conducted to investigate the shear fatigue performance between OGFC and underlying layer. Results indicate that contact area between OGFC and underlying layer play the critically important role. The larger the contact area, the better the shear fatigue performance.

Thirdly, a full scale laboratory test was conducted to investigate the pressure distributions under a single steel or timber crosstie. It is found that pressure distribution was different for steel and timber crossties. Cyclic loading could change the pressure distribution under both
steel and timber crossties, but the effect of cyclic loading was more obvious on steel crosstie
than on timber crosstie.

Last, one coupled framework between discrete element method (DEM) and finite element
method (FEM) was developed to investigate the ballast-tie interaction. The normal contact
condition and the stress distribution beneath the steel crosstie and timber crosstie were
obtained from the simulation. Stress distribution obtained by DEM-FEM simulation was
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Chapter 1

Research background and objectives
1.1 Research background

1.1.1 Discrete element method

A granular material is a conglomeration of discrete solid, macroscopic particles characterized by a loss of energy whenever the particles interact (Duran, 2012). Discrete and continuous models are two methods generally used in the granular materials simulation.

The continuum approach averages the physics across many particles and thereby treats the material as a continuum on a macro scale level (Chen, 2009). In the framework of continuum mechanics, the mechanical behavior of granular materials have been described by several constitutive models (Wu et al., 1996; De Borst and Sluys, 1991; Fleck and Hutchinson, 1997). In a two-dimensional bi-axial geometry, some parameters, such as the shear stress, anisotropy, modulus and deformation, were investigated in the granular media by Luding and Perdahcioğlu (2011). However, many particle collections have a very big porosity and are inherently inhomogeneous, such as ballast. It is difficult to treat such materials as continuum media, especially in the microscope analysis. In this case, the discrete element method appeared and is more and more popular used nowadays.

The Discrete Element Method (DEM) is a discontinuous approach and a powerful numerical tool for computing the motion of a large number of particles such as granular material and now it is becoming the main numerical tool in the analysis of granular materials. The discrete element method (DEM) has been introduced by Cundall & Strack for problems in geotechnical engineering (Cundall and Strack, 1979). Since then DEM is more and more widely adopted as an analysis tools to characterize the behaviors of granular materials in many fields, such as the geotechnical engineering, material engineering etc. DEM which is sometimes also called soft particle Molecular Dynamics is closely related to Molecular Dynamics (MD). Both methods are employed to solve the particles’ behaviors, such as movement, mechanical behaviors, etc. The classic Newton’s second motion law is used in the calculation of the particle movement. Different from traditional continuum computational method, in DEM each element is separated and can have independent movement. All particles are assumed rigid bodies and the interaction only happens at contacts or interfaces between these bodies. Behavior at the contacts uses a soft-contact approach and rigid
particles are allowed to overlap one another at contact points. According to the force-displacement law, the overlap in every contact will generate an interaction force between particles. The contact force will cause the motion of particles which is calculated by Newton’s second law. The motion of particles consequently change the contact situation and results in the changes of contact forces between particles, which continually bring about new motion of particles.

With the development of DEM theory and programming languages, lots of DEM codes are being gradually developed and becoming the very popular tools in solving the problems concerning granular materials. Several commercial DEM softwares are widely used, such as PFC 2d, PFC3d, EDEM, etc. Commercial software could give good user experience because of the easy operability and strong post processing capability. Among the commercial DEM softwares, PFC2d and PFC3d are the most commonly used in granular materials analysis (Buttlar and You, 2001; Wang et al., 2003). For commercial softwares, the functions and features are pre-packaged and it is difficult for researchers to modify the source codes according to the research needs, especially in the very complicated analysis. Besides the commercial softwares, open source code is modifiable and allows the user to improve/modify the existing codebase according to the need of research. YADE, Liggghts and Esys-particle are the three most widely used open source codes used for DEM analysis. YADE Open-DEM is a 3D Open Source GNU/GPL Software framework designed with dynamic libraries and implemented in C++ language. New algorithms and interfaces can be independently implemented. Python is used for rapid and concise scene construction, simulation control and post-processing. After more than ten years’s maintenance, YADE provides a stable environment and high computing efficiency in DEM simulation. Yade has been widely used in geotechnical engineering, materials processing, etc. (Camborde et al., 2000; Belheine et al., 2009; Harthong et al., 2009; Jerier et al., 2010). With the extendable and modifiable character of open source code, YADE DEM code could also be coupled with other codes/methods, such as FEM (Stránský and Jirásek, 2012), CFD (Chen, 2009), which expands the research scope. In this study, YADE open source code is used to do the discrete element modeling.
1.1.2 Bonding properties between open-graded friction course and underlying layer

Open-graded friction course (OGFC) is a special type of hot mix asphalt (HMA) placed on the traditional dense asphalt mixture. OGFC consists of a hot mix asphalt (HMA) mixture that is designed to have a high air voids percentage so that water can drain through and over the surface of this mixture (Brown et al., 2009). The high quality open-graded aggregate is used in OGFC to provide the high air voids and good stone-on-stone contact, which provides OGFC resistance to permanent deformation and disintegration (Brown et al., 2009; Association et al., 2002; Alvarez et al., 2006, 2011). Because of the high air voids, OGFC shows good water permeability and high noise-reduction effectiveness. Besides, OGFC improves ride quality and visibility of pavement markings at night and in wet weather; reduces the risk of hydroplaning, wet skidding and the urban heat island effect (Alvarez et al., 2006; Stempfihar et al., 2012). Because of its multiple benefits, OGFC is attracting extensive concerns nowadays. Surveys show that the percentage of the states using OGFC increased from 38% to 61% from 1998 to 2010 (Kandhal and Mallick, 1998; Kline, 2010).

However, OGFC does not neglect of shortcomings (Kline, 2010; Nielsen, 2006). Similar with traditional dense asphalt pavement, pavement layers dis-bonding is one common disease occurring in OGFC pavement (Kline, 2010; Nielsen, 2006; Song et al., 2015). In contrast with the dense asphalt pavement layers, the contact interface between OGFC and underlying layer is more complicated in consideration of the higher air voids in OGFC (Chen and Huang, 2010; Song et al., 2016). Figure 1.1 shows the schematic diagram of the interface between dense graded asphalt concretes (DGAC) and the interface between OGFC and DGAC (Chen and Huang, 2010). Bonding property between traditional dense graded asphalt concretes (DGAC) has been explored by many researchers (Chen and Huang, 2010; Canestrari et al., 2005; Collop et al., 2009; Mohammad et al., 2002; Raab et al., 2012b; Raposeiras et al., 2012; Al-Qadi et al., 2012). It is generally believed that within a certain temperature range the bonding behavior degrades as temperature goes up (Chen and Huang, 2010; Canestrari et al., 2005; Mohammad et al., 2002; West et al., 2005; Al-Qadi et al., 2012). Research on bonding properties between OGFC and different underlying layers showed similar phenomenon (Song, 2010).
et al., 2015). Tack coat materials are commonly applied in pavement construction to ensure good bonding between pavement layers (Kandhal and Mallick, 1998; King Jr et al., 2013). Some researchers (Mohammad et al., 2002; Raposeiras et al., 2012) concluded that there existed an optimal tack coat application rate at which the maximum shear strength can be obtained. However, contradictory conclusion was drawn that better shear strength values can be obtained without tack coat application (Song et al., 2015; Collop et al., 2009). Some other factors, including tack coat type (Mohammad et al., 2002; West et al., 2005) and tack coat breaking time (Chen and Huang, 2010; Tashman et al., 2006) were also evaluated.

![Figure 1.1. Interlayer surface](image)

As far as the aggregate gradation is concerned, several authors (Song et al., 2015; Canestrari et al., 2005; West et al., 2005; Sholar et al., 2004) agreed that good interlock at the interface can be obtained from good mix design. To explore the interlock between pavement layers, model materials were employed as the aggregate in the research of shear property of double layers (Raab et al., 2012a). Results showed that samples composed by small aggregate in the upper layer and big aggregate in the underlying layer present the better bonding property. Texture depth also plays important role in the pavement bonding. Research showed that the optimal tack coat application rate related to the texture depth of the underlying layer (West et al., 2005; Mrawira and Damude, 1999), which was because the fact that the larger the texture depth, the larger the underlying layer surface area. Larger texture depth usually results in better aggregate interlock effect between pavement layers, and thus leads to better shear performance (Song et al., 2015; Santagata et al., 2008;
By milling the surface of underlying layer, better shear resistance can be obtained in contrast with specimens with the non-milled surface (West et al., 2005; Tashman et al., 2006; Sholar et al., 2004). The effect of other factors on the bonding property between pavement layer, such as surface contamination (Sangiorgi et al., 2002), interface moisture (Raab et al., 2012b), were also evaluated in some research.

Pavement interlayer bonding behavior degrades during the service life, indicting the shear fatigue performance between pavement layers is important. Tack coat and temperature are found to be two important factors that affect the fatigue properties between pavement layers (Li et al., 2014). Some researchers used power law equations to describe the relationship between fatigue life and stress level for direct shear test and oblique shear test (Li et al., 2014; Diakhaté et al., 2011). As in shear strength test, tack coat type is a major factor affecting the fatigue performance. Li and Yu (2013) found that epoxy tack coat gives a remarkably superior shear fatigue performance than styrene-butadiene-styrene (SBS)-modified asphalt and emulsified asphalt tack coat. Energy approach has been utilized for many years in the analysis of asphalt concrete fatigue research. The dissipated energy method (Rowe, 1993; Ghuzlan and Carpenter, 2000; Carpenter et al., 2003) and the concept of ratio of dissipated energy change (RDEC) (Shu et al., 2008; Carpenter et al., 2003) are commonly employed to explore the fatigue performance of asphalt mixtures.

### 1.1.3 Interactions between crosstie and railway ballast

The rapid development of railroad transportation requires a longer service life of the crossties and more safe train travel. Research on the pressure distribution under crossties offers a better understanding of the interactions between ballast and crossties, and further provides assistance for rail transportation guidelines.

Two main types of railroad track are used in the world: track supported by ballast and ballastless track. The ballastless track is designed mainly for high speed railway transit. For the ballasted track system, crossties and ballast are two key components. Timber crossties account for about 90-95% of all the crossties in the US (Csenge et al., 2015). Steel crossties make up only a very small part and they are generally used for light density secondary track. Although timber ties are more widely used in America, light rail transit systems constructed
with timber ties need to replace a large percentage of the timber crossties only after a service life of 20 to 30 years (Brinckerhoff, 2012). In contrast, the steel crosstie usage is steadily rising because of such benefits as a long service life, easy installation, and cost effectiveness.

Pressure distribution under crossties is of great importance to a railway track system. Previous research shows that the tie-to-ballast pressure is not uniformly distributed under the ties. The American Railway Engineering and Maintenance-of-Way Association (AREMA) recommends that the calculation of the pressure should take into account of the distribution and impact factors (AREMA, 2016). In the railway track design, AREMA proposes four equations describing the relationships between tie-to-ballast pressure and other parameters, including ballast depth and wheel load (AREMA, 2016). McHenry et al. (2015) conducted one field test to determine the stress distribution under timber and concrete crossties and considered such factors as the contact area between the ballast and crossties. Discrete element method and finite element method were utilized in the research of ballast under static and cyclic loadings (Indraratna et al., 2009; Hossain et al., 2007; Recuero et al., 2011; Kuo and Huang, 2009). Ballast degradations are observed under the external load especially under the cyclic loading, and the crossties cannot be fully supported by the ballast, which may accelerate the degradation of the crossties (Indraratna et al., 2009; Hossain et al., 2007; Recuero et al., 2011; Sun et al., 2015; Anderson and Fair, 2008).

1.2 Objectives

To better understand the bonding properties between OGFC and underlying layer and the interactions between ballast and ties from the perspective of granular mechanics, the overall objectives of the proposed Ph.D. study are by using discrete element method (DEM) to (1) evaluate the bonding performance between OGFC and different underlying layers; (2) investigate the pressure distribution under steel crosstie and timber crosstie.

For the bonding performance evaluation between OGFC and different underlying layers, the direct shear test is to be simulated by DEM modeling to investigate the aggregate interlock effect between pavement layers. The shear test and fatigue test are also to be performed in laboratory tests.
For investigation of the pressure distribution under railway tie, full scale laboratory test and DEM modeling are to be conducted to solve two questions: (1) how the wood tie and steel tie affect the pressure distribution? (2) how the cyclic loading affects the pressure distribution? The differences in the shape and stiffness between the steel tie and wood tie affects the pressure distribution under the ties. Cyclic loading can accelerate the granular flow and further affects the pressure distribution under the sleepers. The flow chart of the proposed research is shown in Figure 1.2.

**Figure 1.2.** Flow chart of the proposed research

### 1.3 Structure of dissertation

The dissertation is divided into seven chapters. Chapter 1 provides background and literature support for the studies presented herein. Chapter 2 presents the shear bonding performance of two types pavement - OGFC pavement and dense-grade asphalt mixture pavement. Chapter 3 investigates the interlock effect between pavement layers by discrete element modeling. Chapter 4 presents the shear fatigue test between OGFC and underlying layers. Chapter 5 presents a full scale laboratory test work to investigate the pressure distribution beneath steel crosstie and timber crosstie under static loading and dynamic loading. Pressure
distributions of steel crosstie and timber crosstie were comparatively evaluated. Chapter 6 develops a DEM-FEM framework to investigate the ballast-tie interaction. The interaction between ballast and steel/timber sleeper was explored. The final chapter, chapter 7, presents an outline of the conclusions as well as the recommendations for future research.
Chapter 2

Effects of asphalt mixture type on asphalt pavement interlayer shear properties
A version of this chapter has been submitted for publication by Weimin Song, Xiang Shu, Baoshan Huang and Mark Woods:


Weimin Song was the principle researcher and author of ”Effects of asphalt type on asphalt pavement interlayer shear properties”. Weimin Song’s contribution was conducting all literature review, testing, data analysis, and writing the text contained in the manuscript. Dr. Xiang Shu, Dr. Baoshan Huang and Mark Woods provided guidance throughout the research process as well as editorial assistance.

2.1 Abstract

This study compares the interlayer shear properties of pavement layers composed of different asphalt mixture types. Two dense asphalt mixtures (D, BM) and one open-graded friction course (OGFC) mixture were selected for the comparison. The direct shear test was conducted with and without normal stress to assess the shear strength and shear stiffness at different tack coat application rates. Results show that the friction angle between BM aggregate and OGFC aggregate was larger than the angle between D aggregate and OGFC aggregate. At 0.2 MPa normal stress, the specimens composed of two dense mixtures gave higher shear properties than those with OGFC as the upper layer. This is due to the fact that the non-contact area between OGFC and underlying layer compromised the bonding between the two layers. Among the samples composed of two dense layers, the D-BM specimens showed a higher interlayer shear resistance resulting from a larger interlayer roughness caused by a better aggregate interlock between D and BM mixtures. This indicates an upper layer with a small nominal maximum aggregate size (NMAS) and an underlying layer with a large nominal maximum aggregate size(NMAS) could provide a better bonding. For specimens composed of OGFC and a dense mixture (D or BM), at the optimal tack coat application rate, OGFC-BM showed a better shear performance than OGFC-D, due to the double effects of a larger interface contact area and a larger interface roughness than OGFC-D.
2.2 Introduction

Bond properties between pavement layers have drawn extensive concerns during recent years due to the steadily increasing requirement on the long-term pavement performance under ever increasing traffic. It is vital to ensure that different pavement layers work as an integral system during pavement service life. Poor bonding between pavement layers is one of the most influential factors contributing to pavement distress, such as slippage (Figure 2.1), cracks, etc. (Mohammad et al., 2002; Song et al., 2015; Caltabiano and Brunton, 1991).

![Figure 2.1. Pavement interlayer shear](image)

To guarantee good bonding between pavement layers, tack coats are generally applied on the interfaces. Raposeiras et al. (2012) and Mohammad et al. (2005) evaluated the effect of different tack coat dosages and concluded that there exists an optimal tack coat dosage at which the shear strength reaches the maximum value. Too much tack coat may degrade the interface by introducing a slip plane instead of bonding between the layers (Partl et al., 2012; Song et al., 2015). However, contradictory results also reported that better shear properties could be obtained without tack coat applied (Collop et al., 2009; Raab et al., 2012a), which possibly resulted from other factors, such as aggregate gradation etc.

In addition to the adhesion properties of the binder, the interface characteristics and the aggregate interlock between pavement layers also play an important role in the pavement layer bonding. As far as aggregate gradation is concerned, several authors (Canestrari et al., 2005; Sholar et al., 2004; Song et al., 2015; West et al., 2005) agreed that good interlock at the interface can be obtained from good mix design. To explore the interlock between pavement layers, model materials were employed as the aggregates in the research of shear properties of double layers (Raab et al., 2012b). Results showed that samples composed of small aggregate in the upper layer and large aggregate in the underlying layer present a
better bonding property caused by the more significant aggregate interlock effect. Texture depth also plays an important role in pavement bonding. Research showed that the optimal tack coat application rate is related to the texture depth of the underlying layer (Mrawira and Damude, 1999; West et al., 2005), which was due to the fact that the larger the texture depth, the larger the underlying layer surface area. A larger texture depth usually results in a better aggregate interlocking effect between pavement layers, and thus leads to a better shear performance (Santagata et al., 2008; Song et al., 2015). By milling the surface of underlying layer, a better shear resistance can be obtained in contrast with specimens with the non-milled surface (Sholar et al., 2004; Tashman et al., 2006; West et al., 2005). The effect of other factors on the bonding properties between pavement layers, such as surface contamination (Collop et al., 2003; Raab and Partl, 2016; Sangiorgi et al., 2002), interface moisture Raab et al. (2012a); Sholar et al. (2004), were also evaluated in some studies.

Due to the increasing awareness in safety, environmental friendliness and economical efficiency, permeable pavement is becoming more and more popular nowadays (Alvarez et al., 2011; Song et al., 2015). Open graded friction course (OGFC) is a thin permeable asphalt layer placed on the traditional dense asphalt layer. OGFC mixtures designed for the requirement of stone-on-stone contact and high connected air void content are a special type of asphalt mixture characterized by the use of high-quality open-graded aggregate (Huber, 2000; Kandhal and Mallick, 1998; Mallick et al., 2000; Kline, 2010). OGFC provides many benefits, including drainability, good noise reduction, improving the skid-resistance, improving the visibility in rainy days and easing the urban heat island effect et al (Huber, 2000; Kandhal and Mallick, 1998; Stempihar et al., 2012). Because of these benefits, surveys showed that the percentage of the states in US using OGFC increased from 38% to 61% from 1998 to 2010 (Kandhal and Mallick, 1998; Kline, 2010). However, the high air void content in OGFC may lead to poor bonding between OGFC and the underlying layer and make the bonding condition even more complicated than conventional pavements (Chen and Huang, 2010; Song et al., 2015).
2.3 Objective and scope

The objective of this study was to compare the interlayer shear properties of pavement layers composed of dense asphalt and OGFC mixtures. The shear strength and shear stiffness were obtained from a direct shear test to evaluate the shear properties. The factors of normal stress, tack coat application rate, and surface characteristics were evaluated through their effects on bonding properties of pavement layers. The aggregate interlock effect between pavement layers was evaluated and the contact area between OGFC and the underlying layer was obtained.

2.4 Experiment program

2.4.1 Materials

Three asphalt mixtures were tested in this study: one type of OGFC and two types of dense-graded asphalt mixtures (which are called BM and D in Tennessee, USA). Table 2.1 presents the material information of OGFC, D, and BM mixtures and Figure 2.2 shows the aggregate gradation of the three mixtures. It can be seen that BM is coarser than D. The nominal maximum aggregate size (NMAS) was 12.5 mm, 19 mm, and 9.5 mm, respectively, for OGFC, BM and D. An anionic slow-setting asphalt emulsion with grade SS-1 was selected as the tack coat material. The anionic slow-setting emulsion is commonly used based on Tennessee experience. In this study, four tack coat residual application rates were used: 0, 0.15 l/m², 0.30 l/m² and 0.45 l/m².

Table 2.1. Mixture property

<table>
<thead>
<tr>
<th>Mixture property</th>
<th>OGFC</th>
<th>TLD</th>
<th>BM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt PG grade</td>
<td>PG 76-22</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>AC content</td>
<td>6.4%</td>
<td>6.2%</td>
<td>4.2%</td>
</tr>
<tr>
<td>Aggregate type</td>
<td>Gravel</td>
<td>Limestone</td>
<td>Limestone</td>
</tr>
</tbody>
</table>
2.4.2 Sample preparation

The double layer specimens with a 150 mm diameter were compacted using a Superpave Gyratory Compactor (SGC). First, the underlying layer was compacted to obtain a layer thickness of 50 mm with approximately 4% air voids. The underlying layer was extruded from the mold and cooled down to ambient temperature (20 °C). Then the tack coat material was uniformly spread on the top surface of the underlying layer at a predetermined application rate. Before the compaction of the upper layer mixture, the specimen was left for 30 mins to allow the tack coat to break. OGFC was compacted to an air void content of about 18%. According to the Tennessee experience, the height of OGFC is 32 mm. To simplify the analysis, the height of all the upper layers was 32 mm in this study.

Table 2.2 presents the information of the double layer specimens. The following rule was used for the name of specimens: upper layer-underlying layer. As an example, specimen D-BM means that D and BM mixtures were used as the upper layer and the underlying layer, respectively. To evaluate the interlock effect, four types of interlayers (BM-BM, BM-D, D-BM and D-D) were included in the study.
Table 2.2. Specimen information

<table>
<thead>
<tr>
<th>Interlayer</th>
<th>No. of specimens</th>
<th>Normal stress (MPa)</th>
<th>Air voids</th>
</tr>
</thead>
<tbody>
<tr>
<td>BM-BM</td>
<td>3</td>
<td>0.2</td>
<td>4%-4%</td>
</tr>
<tr>
<td>BM-D</td>
<td>3</td>
<td>0.2</td>
<td>4%-4%</td>
</tr>
<tr>
<td>D-BM</td>
<td>3</td>
<td>0.2</td>
<td>4%-4%</td>
</tr>
<tr>
<td>D-D</td>
<td>3</td>
<td>0.2</td>
<td>4%-4%</td>
</tr>
<tr>
<td>OGFC-BM</td>
<td>3</td>
<td>0.2</td>
<td>18%-4%</td>
</tr>
<tr>
<td>OGFC-D</td>
<td>3</td>
<td>0.2</td>
<td>18%-4%</td>
</tr>
<tr>
<td>OGFC-BM</td>
<td>3</td>
<td>0.4</td>
<td>18%-4%</td>
</tr>
<tr>
<td>OGFC-D</td>
<td>3</td>
<td>0.4</td>
<td>18%-4%</td>
</tr>
<tr>
<td>OGFC-BM</td>
<td>3</td>
<td>0</td>
<td>18%-4%</td>
</tr>
<tr>
<td>OGFC-D</td>
<td>3</td>
<td>0</td>
<td>18%-4%</td>
</tr>
</tbody>
</table>

2.4.3 Direct shear test

In this study, a direct shear test device was employed to conduct the shear test (Figure 2.3). A spring is used to apply a normal stress on the shear plane. A dial indicator reads the displacement of the spring to determine the applied normal stress. In this study, the direct shear test was performed at a constant rate of 50 mm/min. The shear strength was calculated as the measured peak shear force divided by the area of the interface. The direct shear test was conducted in triplicate. All the tests were conducted at 20 °C.

Figure 2.3. Direct shear test setup

A typical shear stress-displacement curve was plotted in Figure 2.4. Besides the shear strength, interface shear stiffness was employed to characterize the bonding property between pavement layers. The interface stiffness is defined as follows by the Goodmans constitutive law (Canestrari et al., 2005):
\[ k = \frac{\tau}{\varepsilon} \]  \hspace{3cm} (2.1)

where, \( k \) is the interface stiffness (MPa/cm); \( \tau \) is the shear strength (MPa); \( \varepsilon \) is the displacement within the interface (cm).

Figure 2.4. Strength displacement curve

2.5 Results and discussion

2.5.1 Effect of normal stress

Figure 2.5 shows the shear strength of OGFC-D with and without normal stress applied. It shows that for the specimens with different tack coat application rates, the shear strength increased with the increase of the normal stress, which was accounted from the Mohr-Coulomb failure law (Canestrari et al., 2005), as shown in Equation 2.2. In Figure 2.5, the fitting equation was for specimens without tack coat applied.

\[ \tau_{\text{peak}} = c_0 + \sigma \cdot \tan \phi_p \]  \hspace{3cm} (2.2)
where $c_0$ is the pure shear resistance; $\phi_p$ is the friction angle; $\sigma$ is the normal stress and $\tau_{\text{peak}}$ is the failure shear strength.

The values of $\tan \phi$ at different tack coat application rate of OGFC-D and OGFC-BM were displayed in Figure 2.6. It can be apparently observed that the value of $\tan \phi$ of OGFC-D was lower than that of OGFC-BM at the same tack coat dosage, indicating the friction angle between OGFC and BM was larger than that of OGFC and TLD.

**Figure 2.5.** Shear strengths of OGFC-D at different normal stress levels

**Figure 2.6.** $\tan \phi$ at different tack coat application rates
2.5.2 Effect of tack coat application rate

Figure 2.7 and Figure 2.8 show the test results of shear strength and shear stiffness of specimens at different tack coat application rates at the stress level of 0.2 MPa.

In Figure 2.7, it can be seen that the shear strength development trend was significantly different between OGFC-D and OGFC-BM. The optimal tack coat application rate of the specimens with BM as underlying layer was 0.30 l/m². However, for the specimens with D as the underlying layer, the maximum shear strength was obtained when no tack coat was applied. When tack coat was applied, the shear strength decreased with the increase in tack coat rate due to the slip induced by tack coat (Partl et al., 2012; Song et al., 2015). Besides, for the specimens with the same underlying layer, regardless of D or BM, the specimens with D as upper layer showed the highest shear strength and the OGFC specimens the lowest shear strength. The development trend of the shear stiffness (Figure 2.8) was generally similar to that of the shear strength, except for a few points for specimens D-D and BM-D.

![Figure 2.7. Shear strength at 0.2 MPa normal stress](image-url)
2.5.3 Effect of surface characteristics

Figure 2.9 shows the ranking of the shear strength and the shear stiffness was the same at 0.2 MPa normal stress and the optimal tack coat application rate, which was D-BM > D-D > BM-BM > BM-D > OGFC-BM > OGFC-D.

Research (Raab et al., 2012a) has shown that the roughness capacity of the interlayer surface can be defined as:

$$ R = f\left(\frac{d_1}{d_2}\right) = C\left(\frac{d_1}{d_2}\right) $$  \hspace{1cm} (2.3)

where \( R \) is the roughness capacity, \( d_1 \) is the aggregate size of underlying layer, \( d_2 \) is the aggregate size of upper layer, \( C \) is a constant.

This equation was established for double layer specimens compacted by single size aggregates in the upper layer and the underlying layer. In this study, to simplify the analysis, an average aggregate size was used for a mixture, which was the mean value of all the aggregate sizes considering their volume fractions. Because \( C \) is a constant, \( R' \) is defined as follows and used to characterize the interface roughness:
\[ R' = \frac{R}{C} = \frac{d_1}{d_2} \]  

(2.4)

\[ R'_{D-BM} = \frac{d_{BM}}{d_D} = \frac{\sum_{i=1}^{n} \varphi_i \cdot d_{BM_i}}{\sum_{j=1}^{n} \varphi_j \cdot d_{D_j}} = 2.34 \]  

(2.5)

where \( R'_{D/BM} \) is the interface roughness of D-BM, \( d_{BM} \) is the average aggregate size of BM, \( d_D \) is the average aggregate size of D, \( \varphi_i \) is the volume fraction of BM aggregate at sieve size \( i \), \( \varphi_j \) is the volume fraction of D aggregate at sieve size \( j \), \( d_{BM_i} \) is the sieve size \( i \), \( d_{D_j} \) is the sieve size \( j \).

Therefore, the interface roughness capacity of D-BM, D-D, BM-BM and BM-D is shown in Equation 2.6. Studies have shown that roughness makes a positive effect on the bonding between pavement layers (Song et al., 2015; West et al., 2005; Sholar et al., 2004). Therefore, it can be inferred that the bonding properties of D-BM are the highest, followed by D-D, BM-BM and BM-D. There may be a small difference between D-D and BM-BM due to the same roughness capacity but different average aggregate size. Figure 2.9 shows that bonding properties of D-D were higher than those of BM-BM at their optimal tack coat application rates. A possible explanation is that the finer mixture (D-D) resulted in a larger contact area and thus made the tack coat application more effective (West et al., 2005).

\[
\begin{pmatrix}
R'_{D-BM} \\
R'_{D-D} \\
R'_{BM-BM} \\
R'_{BM-D}
\end{pmatrix} = \begin{pmatrix}
2.34 \\
1 \\
1 \\
0.43
\end{pmatrix}
\]  

(2.6)

The obvious difference between the specimens of OGFC-dense underlying layer and those of two dense layers is that the contact area between OGFC and its underlying layer is lower because of high air voids of OGFC (Song et al., 2015). In this study, to simplify the analysis, the contact between dense asphalt mixture layers was deemed a perfect contact. To quantify
the contact area between OGFC and the underlying layer, the image binarization technique was employed. Image binarization is a process that converts an original image into a binary image featured by just two colors-black and white.

Figure 2.10 and Figure 2.11 show the original pictures of the failed interface of OGFC and the pictures after the binarization processing for the specimens OGFC-BM and OGFC-D. The pictures in Figure 2.10 and Figure 2.11 were obtained at 0.15 l/m² tack coat rate. Because of the non-contact part between OGFC and the underlying layer, there was an obvious color difference between the non-contact part and the contact part. The darkest part in the original picture is the OGFC binder without contact with the underlying layer. Through image binarization, the darkest area in the original picture can be identified and quantified, which is shown at right in Figure 2.10 and Figure 2.11.
Figure 2.10. Interface images of OGFC-BM: (a) original picture and (b) picture after binarization

Figure 2.11. Interface images of OGFC-D: (a) original picture and (b) picture after binarization
Figure 2.12 shows the contact area between OGFC and the underlying layer. As can be seen, the interface contact area of OGFC-D was larger than that of OGFC-BM at the same tack coat rate. At the optimal tack coat rate (0 and 0.30 l/m$^2$ for OGFC-D and OGFC-BM, respectively), the contact area percentage was 75% and 85% for OGFC-D and OGFC-BM, respectively and the non-contact area percentage was 25% and 15% for OGFC-D and OGFC-BM, respectively. Compared to the specimens of two dense layers, the non-contact area of OGFC-dense layer also contributed to the difference in bonding properties.

![Figure 2.12. Percentage of contact area at different tack coat application rates](image)

At the optimal tack coat rate, the contact area between OGFC and D was lower than that of OGFC-BM (75% vs. 85%). Also from Equation (5), the ratio of the interface roughness of OGFC-D to OGFC-BM can be obtained and written as Equation (7) (the ratio is $\frac{1}{2.1}$). OGFC-BM had a higher roughness capacity, resulting from the better interlock between OGFC and BM. The higher shear performance of OGFC-BM was a combined effect of a larger interface contact area and a higher larger interface roughness.

$$\frac{R'_{OGFC-D}}{R'_{OGFC-BM}} = \frac{d_D}{d_{BM}} = \frac{\sum_{j=1}^{n} \varphi_j \cdot d_{Dj}}{\sum_{i=1}^{n} \varphi_i \cdot D_{BMi}} = \frac{1}{2.1} \quad (2.7)$$
2.6 Summary and conclusion

A laboratory study was conducted to investigate the effects of mixture type on the interlayer shear properties. Two dense mixtures and one OGFC mixture were selected for this purpose. Interlayer shear strength and shear stiffness were obtained through direct shear testing. Interface characteristics were analyzed and interlock effect was evaluated between upper and underlying layers. The contact area between OGFC and the underlying layer was also characterized and correlated to the shear performance. Based on the results of the study, the following conclusions can be drawn:

- When D mixture was used as the underlying layer, the specimens without tack coat showed the highest shear performance. When BM was used as the underlying layer, the specimens showed the maximum shear strength and stiffness at the optimal tack coat application rate of 0.30 l/m².

- With the increase in normal stress, the interlayer shear strength and stiffness increased as well. According to the Coulomb failure law, the friction angle between OGFC and BM was larger than that of OGFC and D.

- For dense pavement layers, the specimens with a small aggregate size in the upper layer and a large aggregate size in the underlying layer provided the highest shear resistance. The shear strength and stiffness ranking were D-BM > D-D > BM-BM > BM-D. Interlayer roughness played a dominant role in the interlayer shear resistance.

- At the optimal tack coat application rate, the contact area between OGFC and an underlying layer was smaller than that of two dense layers, leading to the lowered shear performance of OGFC pavements in comparison to that of dense pavement layers.

- At the optimal tack coat application rate, the interlayer shear performance of OGFC-D was lower than that of OGFC-BM, which was caused by the dual effects of a smaller contact area and a lower interface roughness.
2.7 Acknowledgements

The project was financially sponsored by the Tennessee Department of Transportation (TDOT). The authors would like to thank TDOT engineers and contractors who helped provide the materials for the study. The contents of this study reflect the views of the authors only. The first author would also like to thank the China Scholarship Council (CSC) for their support.
Chapter 3

Interlock effect between pavement layers on asphalt pavement interlayer shear properties
3.1 Abstract

This study evaluated the effect of pavement layer interlock on asphalt pavement interlayer shear properties. Aggregate gradations in two dense asphalt mixtures (D, BM) and one open-graded friction course (OGFC) were selected. Direct shear test was modeled with different normal stresses to assess the shear strengths. Results show that the friction angle between BM aggregate and OGFC aggregate was larger than the angle between D aggregate and OGFC aggregate. For samples composed by two dense asphalt layers, D-BM mixtures gave the larger interlock effect. The results from DEM modeling agreed with the results from numerical analysis.

3.2 Introduction and objective

Under the combined effects of traffic loading and environmental effects, asphalt pavements suffer lots of pavement distresses, such as cracking, slipping, rutting, etc. Bonding between pavement layers is critically important to ensure the pavement durability. Pavement layer bonding properties have attracted enormous attention during recent years due to the stricter requirements of the pavement design methodologies and increasing requirements on long term performance under ever increasing traffic. This led to the development of different test
devices and the evaluation of a variety of test procedures, where the influence of different parameters, such as temperature, tack coat application or normal stress has been discussed in numerous papers and scientific contributions (Diakhaté et al., 2006; Raab et al., 2012a; Song et al., 2015, 2016; Mohammad et al., 2002).

As for the bonding condition between pavement layers, the adhesion properties of asphalt binder and tack coat play important role (Song et al., 2015; Mohammad et al., 2002). In addition to the adhesion properties of the binder, the interface characteristics and the aggregate interlock between pavement layers also play an important role in the pavement layer bonding (Raab et al., 2012b; Song et al., 2017a). As far as aggregate gradation is concerned, several authors (Canestrari et al., 2005; Sholar et al., 2004; Song et al., 2015; West et al., 2005) agreed that good interlock at the interface can be obtained from good mix design. To explore the interlock between pavement layers, model materials were employed as the aggregates in the research of shear properties of double layers (Raab et al., 2012b). Results showed that samples composed of small aggregate in the upper layer and large aggregate in the underlying layer present a better bonding property caused by the more significant aggregate interlock effect. Texture depth also plays an important role in pavement bonding. Research showed that the optimal tack coat application rate is related to the texture depth of the underlying layer (Mrawira and Damude, 1999; West et al., 2005), which was due to the fact that the larger the texture depth, the larger the underlying layer surface area. A larger texture depth usually results in a better aggregate interlocking effect between pavement layers, and thus leads to a better shear performance (Santagata et al., 2008; Song et al., 2015). By milling the surface of underlying layer, a better shear resistance can be obtained in contrast with specimens with the non-milled surface (Sholar et al., 2004; Tashman et al., 2006; West et al., 2005). The effect of other factors on the bonding properties between pavement layers, such as surface contamination (Collop et al., 2003; Raab and Partl, 2016; Sangiorgi et al., 2002), interface moisture Raab et al. (2012a); Sholar et al. (2004), were also evaluated in some studies.

Due to the increasing awareness in safety, environmental friendliness and economical efficiency, permeable pavement is becoming more and more popular nowadays (Alvarez et al., 2011; Song et al., 2015). Open graded friction course (OGFC) is a thin permeable
asphalt layer placed on the traditional dense asphalt layer. OGFC mixtures designed for the requirement of stone-on-stone contact and high connected air void content are a special type of asphalt mixture characterized by the use of high-quality open-graded aggregate (Huber, 2000; Kandhal and Mallick, 1998; Mallick et al., 2000; Kline, 2010). OGFC provides many benefits, including drainability, good noise reduction, improving the skid-resistance, improving the visibility in rainy days and easing the urban heat island effect et al (Huber, 2000; Kandhal and Mallick, 1998; Stempihar et al., 2012). Because of these benefits, surveys showed that the percentage of the states in US using OGFC increased from 38% to 61% from 1998 to 2010 (Kandhal and Mallick, 1998; Kline, 2010). However, the high air void content in OGFC may lead to poor bonding between OGFC and the underlying layer and make the bonding condition even more complicated than conventional pavements (Chen and Huang, 2010; Song et al., 2015).

In order to study the influence of the geometrical interlock and the combination of aggregate sizes on the shear strength, discrete element method (DEM) was utilized in this study to analyze the interlock effect. The Discrete Element Method (DEM) is a discontinuous approach and a powerful numerical tool for computing the motion of a large number of granular particles and offers one micro-mechanical insight into its behavior (Cundall and Strack, 1979). Single rigid particle motion is calculated from resultant contact and body forces acting on the particle according to Newtons second law of motion. With its inherent advantage, the DEM has been used widely for studying the behavior of soils and granular materials (Ting et al., 1989; Jing, 2000; Yao and Anandarajah, 2003; Zhao et al., 2015). You and Buttlar (2006) applied the DEM model to predict the dynamic modulus of asphalt mixtures. Wang et al. (2007) and Chen et al. (2012) used DEM to study the compaction of asphalt mixture. Collop et al. (2004, 2006) used DEM to simulate the behavior of a highly idealized bituminous mixture (single-sized spherical particles mixed with bitumen) under uniaxial and triaxial compressive creep loading. The DEM predicted results are in reasonable agreement with experimental data. In the research of the interlock effect between granular layers, the direct shear box test is commonly conducted Zhao et al. (2015).

In this study, the interlock effect between aggregate layers was evaluated using DEM. The numerical computation was also conducted. The open source source code (YADE) (Šmilauer
et al., 2010) was used to make the DEM simulation. The cores of YADE program is written in C++ using flexible object model, allowing independent implementation of new algorithms and interfaces. Python is used for model construction, simulation control and postprocessing.

3.3 DEM modeling

3.3.1 Contact model

The contact-stiffness model, which is a common contact model in DEM, defines an elastic relation between the contact force $F$ and relative displacement $\delta$ (Equation 3.1). The contact force can be decomposed into two orthogonal components: normal force $F_n$ along the contact normal, and tangential force $F_s$ along the contact tangential direction.

\[ F = K\delta \quad (3.1) \]

where $K$ is the contact stiffness, determined by the material property of the two touching particles.

In the calculation of the shear force $F_s$, a standard incremental algorithm (Smilauer et al., 2010) is applied in YADE. In each time step, because of the motion and rotations of the particles, an additional shear displacement increment $\Delta u$ is generated, and the shear force is adjusted by the increment $\Delta F_s$.

\[ \Delta F_s = K_s \Delta u \quad (3.2) \]

Where $K_s$ is the contact shear stiffness of the material.

In the calculation of shear force, the Coulomb sliding criterion (Equation 3.3) is employed when a slide occurs at the contact. When the shear force calculated by Equation 3.2 is larger than $F_n \cdot \tan\phi$, a slide occurs and the shear force is reduced to $F_n \cdot \tan\phi$. 

\[ F_s \leq F_n \cdot \tan \phi \] (3.3)

where \( \phi \) is the friction angle.

Some main parameters used in DEM are shown in Table 3.1.

**Table 3.1. Some parameters in DEM**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle density</td>
<td>2600 kg/m(^3)</td>
</tr>
<tr>
<td>Friction angle</td>
<td>30°</td>
</tr>
<tr>
<td>Damping coefficient</td>
<td>0.3 s</td>
</tr>
<tr>
<td>Time step increment</td>
<td>(2 \times 10^{-6}) s</td>
</tr>
<tr>
<td>Gravity</td>
<td>9.8 m/s(^2)</td>
</tr>
</tbody>
</table>

### 3.3.2 Numerical direct shear test

Three types of asphalt mixtures were tested in this study: one type of OGFC and two types of dense-graded asphalt mixtures (which are called BM and D in Tennessee, USA). Figure 3.1 shows the aggregate gradation of the three mixtures. It can be seen that BM is coarser than D. The nominal maximum aggregate size (NMAS) was 12.5 mm, 19 mm, and 9.5 mm, respectively, for OGFC, BM and D.

**Figure 3.1. Aggregate gradation**
In the DEM simulation, six types of samples were modeled. The following rule was used for the name of specimens: upper layer-underlying layer. As an example, specimen D-BM means that D and BM mixtures were used as the upper layer and the underlying layer, respectively. The samples were labeled as BM-BM, BM-D, D-BM, D-D, OGFC-BM and OGFC-D. Two normal stress levels (0.2 MPa and 0.4 MPa) were selected to evaluate the interlock effect under different normal stress levels.

Generally, spherical particles are directly generated in DEM modeling and the simple spherical particles make the DEM analysis in a high computing efficiency. In this study, to improve the computing efficiency, spherical particles were selected to represent the aggregates. The computation efficiency is affected by particle numbers, generally speaking, the larger the number, the lower the computation efficiency. Simpson and Tatsuoka (2008) predicted that the simulation of up to \(10^5\) circular (2D) or spherical (3D) particles can be conducted in YADE. With the development in super computing, computation efficiency improved very much in last two decades. However, it is still insufficient to model most real boundary value problems on a particle-by-particle basis (Chen et al., 2012). In order to improve the computing speed, one common assumption in DEM simulation is to model practical situations using much larger particles. In asphalt mixture DEM simulation, it is common to simulate only the aggregate particles bigger than a particular size and to assume that fine aggregate particles and asphalt are mixed together as mastic, which is taken into account by the contact law between particles. Collop et al. (2004, 2006) used 1.18-mm spherical ball models to simulate aggregate to study bulk material properties and viscoelastic behavior of asphalt mixture. Chen et al. (2012) used aggregates with size larger than 2.36 mm to model the compaction of asphalt mixture. In the present study, the maximum aggregate size of D, BM and OGFC was 16 mm, 25.4 mm and 19 mm, which are much greater than 0.6 mm, and the mixtures had a relatively low proportion of fine aggregate smaller than 0.6 mm. Thus for D mix and BM, only aggregates greater than 0.6 mm were simulated in the DEM. For OGFC, aggregates greater than 2.36 mm were selected due to the very low volume of the aggregates with the size lower than 2.36 mm.

In DEM modeling, under the action of gravity, aggregates gradually fall downwards. After the aggregates come to the static state, an external wall is applied and loaded with
sinusoidal loading from 0 to 10000 N at a frequency of 10 Hz on the top for 3 seconds to make the ballast particles achieve the initially compacted state. The compaction process can be observed as Figure 3.2. After the compaction of the dual-layer aggregates, the direct shear test was conducted (Figure 3.3).

![Figure 3.2. Compaction process](image)

In the DEM modeling, the lower box, including its walls and boundary particles moves laterally at a constant shear displacement increment of $8 \times 10^{-4}$ m/s, while the upper box keeps fixed. The normal stress was applied on the top plate. To keep a constant vertical stress, the vertical position of the top plate changes at each time step using a numerical servo-control mechanism (Wang et al., 2003).
3.4 Results and discussion

3.4.1 Macro shear behavior

In Figure 3.4 and Figure 3.5, the results of the direct shear test at 0.2 MPa and 0.4 MPa are depicted, showing the shear strengths for all six combinations.

It can be seen that the individual curves are quite unsteady because of the move of the particles in the shear process, so two tests were conducted for each sample type. The results demonstrate that the larger the normal stress, the larger the shear stress. Considering the aggregate combinations which belong to dense asphalt mixture aggregate (BM-BM, BM-D, D-BM and D-D), although the curves were unsteady and bewildering, shear stress seems to have the highest value when the sample combination is D-BM, followed by BM-BM, and the combination BM-D and D-D do not have a distinct tendency. For the aggregate combinations which belong to OGFC pavement samples (OGFC-BM and OGFC-D), it can be seen very clearly that OGFC-BM presents larger shear strength.

Table 3.2 presents the maximum shear strengths for the six types of samples. Quantitatively, the peak shear stress and friction angle are present in Figure 3.6. Both the friction angle $\phi$ and the cohesion $c$ are calculated based on Mohr-Coulomb criteria, the ranking of the friction angle for dense asphalt samples is: $\phi_{D-BM} > \phi_{BM-BM} > \phi_{D-D} > \phi_{BM-D}$. This results indicate that the combination of upper layer and lower layer present the better interlock effect when the aggregate size in the upper layer is small and aggregate size in the
lower layer is big. The friction angle of OGFC-BM is larger than that of OGFC-D, indicating the interlock effect between OGFC and BM is more significant that of OGFC-D.

From chapter 2, the interface roughness ranking of the dense asphalt samples is $R_{D-BM} > R_{BM-BM} = R_{D-D} > R_{BM-D}$, and for OGFC samples, the interface roughness ranking is $R_{OGFC-BM} > R_{OGFC-D}$. The results generated from DEM modeling is similar with the laboratory tests. However, some considerations should be taken in this study. The particle shape in this study is spherical, which is different with the actual geometry. However, this study can also give some conclusions about how the aggregate size influences the interlock between pavement layers.
Table 3.2. Aggregate type and asphalt cement

<table>
<thead>
<tr>
<th>Normal stress (MPa)</th>
<th>Samples</th>
<th>Shear stress (MPa)</th>
<th>Standard error</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>BM-BM</td>
<td>0.162</td>
<td>0.004</td>
</tr>
<tr>
<td></td>
<td>BM-D</td>
<td>0.121</td>
<td>0.0035</td>
</tr>
<tr>
<td></td>
<td>D-BM</td>
<td>0.171</td>
<td>0.0035</td>
</tr>
<tr>
<td></td>
<td>D-D</td>
<td>0.112</td>
<td>0.0025</td>
</tr>
<tr>
<td></td>
<td>OGFC-BM</td>
<td>0.132</td>
<td>0.0045</td>
</tr>
<tr>
<td></td>
<td>OGFC-D</td>
<td>0.116</td>
<td>0.0035</td>
</tr>
<tr>
<td>0.4</td>
<td>BM-BM</td>
<td>0.261</td>
<td>0.0065</td>
</tr>
<tr>
<td></td>
<td>BM-D</td>
<td>0.205</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>D-BM</td>
<td>0.278</td>
<td>0.0035</td>
</tr>
<tr>
<td></td>
<td>D-D</td>
<td>0.201</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>OGFC-BM</td>
<td>0.251</td>
<td>0.0075</td>
</tr>
<tr>
<td></td>
<td>OGFC-D</td>
<td>0.189</td>
<td>0.005</td>
</tr>
</tbody>
</table>

Figure 3.6. Relationship between peak shear stress and normal stress

Considering the lower layer, the bigger size of the particles produce deeper cavities and therefore more shear roughness. For upper layer, the smaller size of the particles, the larger shear roughness since the smaller the diameter, the better the cavities can be filled. So that the interface roughness equation Equation 2.4 \( R = \frac{d_{\text{low-layer}}}{d_{\text{upper-layer}}} \) was proposed. However, when the diameter of the particle in the upper layer is lower than the critical size \( d^* \), the center of the particles is below the shear plane, the particles will loss the interlock effects. As shown in Figure 3.7, the interlock effect becomes more obvious from left to right. Although
there are some particles whose diameters may be lower than the critical size \( d^* \) in this DEM study, they do not affect the final results of the interlock ranking.

Figure 3.7. Schematic drawing of different combinations

### 3.4.2 Force chain

Contact forces among particles have been regarded as a significant factor affecting the mechanical behavior of granular assemblies. The contact force chains are used to show how an applied force is transmitted through the particles. Previous investigations show that the normal contact force has more pronounced contribution to the deviator stress tensor than the tangential contact force (Thornton, 2000; Zhao et al., 2015; Yang et al., 2008). Such a feature has also been observed in the present simulations. Therefore, it is of interest to give an observational investigation into the evolution of the normal contact force distribution (i.e., the normal contact force chain) during shearing.

Figure 3.8 to Figure 3.13 presents the evolutions of normal contact force chains in three categories of assemblies under a vertical stress 0.4 MPa during shearing. Before the direct shear test starts \((\varepsilon = 0)\), the normal contact forces in D-D is more uniform and the number of force chains in D-D is larger than in other assemblies, which is because the aggregate number in D-D is larger and there are more contacts between the aggregates in D-D. During the shearing process, the contact force near the shear plane gradually increases with its direction redistributing. It can be seen that a growing shear band with obviously directional shear force chains occurs near the shear plane at some level of shear strain.
Figure 3.8. Normal contact force distributions of BM-BM at different shear strain

Figure 3.9. Normal contact force distributions of BM-D at different shear strain

Figure 3.10. Normal contact force distributions of D-BM at different shear strain

Figure 3.11. Normal contact force distributions of D-D at different shear strain

Figure 3.12. Normal contact force distributions of OGFC-BM at different shear strain
Figure 3.13. Normal contact force distributions of OGFC-D at different shear strain

The specimen in the direct shear test is sheared through laterally moving the lower shear box from left to right, thereby pushing the lower part of the specimen to move rightward. Therefore, both the top-right and the bottom-left parts of the specimen are more severely squeezed than any other parts, which can be observed from the normal contact force distributions. Figure 3.14 presents the displacement vectors of the particles of OGFC-BM at normal stress 0.4 MPa and shear strain 0.3. Three zones can be observed: upper zone (zone 1), shear zone (zone 2), lower zone (zone 3). In the upper box, although great of normal contact forces applied on the particles in the top-right, the top plate also applied vertical load on the particles in the upper box, the particle movement in the upper box was not obvious. In the shear zone (zone 2), particles move right forward, but also downward. In lower zone (zone 3), under the combined effects of the horizontal and vertical loads, the particles move right forward and downward. For particles in the right part of zone 3, there is no vertical load and boundary control in the vertical direction, particles can also move upward. This phenomenon is similar with the published results (Indraratna et al., 2012; Wang and Gutierrez, 2010).
3.5 Conclusions

The interlock effect between pavement layers was investigated by DEM modeling in this study. Traditional dense asphalt pavement and OGFC pavement were both selected. Direct shear tests were modeled on four types of dense asphalt samples and two types of OGFC samples at normal stress 0.2 MPa and 0.4 MPa. All the DEM tests were conducted using open source code YADE. Conclusions can be drawn from this study:

• Shear stresses of D-BM are the largest both at the normal stress 0.2 MPa and 0.4 MPa, followed by BM-BM, BM-D and D-D. The shear stresses of OGFC-BM are larger than that of OGFC-D at stress 0.2 MPa and 0.4 MPa.

• For dense asphalt mixtures, the ranking of the friction angle is \( \phi_{D-BM} > \phi_{BM-BM} > \phi_{D-D} > \phi_{BM-D} \). For OGFC samples, the ranking is \( \phi_{OGFC-BM} > \phi_{OGFC-D} \).

• Three zones can be clearly observed in the direct shear test. In the upper box, the particle movement is not obvious. Particles in the shear zone move long with the
moving direction of the underlying box, besides, particles also move downward. Under the combined effects of the vertical and horizontal loads, particle movement in lower box is more obvious.
Chapter 4

Laboratory investigation of interlayer shear fatigue performance between open-graded friction course and underlying layer
A version of this chapter was originally published by Weimin Song, Xiang Shu, Baoshan Huang and Mark E. Woods:


Weimin Song was the principle researcher and author of "Laboratory investigation of interlayer shear fatigue performance between open-graded friction course and underlying layer". Weimin Song’s contribution was conducting all literature review, testing, data analysis, and writing the text contained in the manuscript. Dr. Xiang Shu, Dr. Baoshan Huang and Mr. Woods E. Mark provided guidance and ideas throughout the research process as well as editorial assistance.

4.1 Abstract

The objective of the study was to evaluate the bonding fatigue performance between open-graded friction course (OGFC) and underlying layer through laboratory testing at different tack coat application rates. Direct shear fatigue test was performed to obtain the shear fatigue properties of the composite specimens composed of OGFC and its underlying layer. Two types of dense graded asphalt mixture (named BM and TLD in Tennessee) were selected as the underlying layer. In addition to the fatigue life determined according to the conventional 50% stiffness reduction method, energy approach was also employed to analyze the fatigue behavior of the composite specimens. Results showed that the OGFC-TLD structure gave a better shear fatigue performance than OGFC-BM. With the increase in tack coat application rate, the plateau value (PV) increased and the total cumulative dissipated energy decreased for both combined structures. The contact area between OGFC and the underlying layer was measured and correlated to the fatigue life. The contact area between OGFC and TLD was larger than that between OGFC and BM, leading to a better fatigue performance of OGFC-TLD.
4.2 Introduction

Open graded friction course (OGFC) is a special type of hot mix asphalt mixture with good drainability and noise reduction effectiveness. Because of its safety improvement and environmental benefits, OGFC has become more and more popular in these days (Alvarez et al., 2011; Watson et al., 2004; Mallick et al., 2000; Alvarez et al., 2006; Tappeiner, 1993; Raab and Partl, 2009). To ensure that OGFC layer and its underlying layer are well bonded to behave as a monolith system, it is essential to evaluate the bonding properties between OGFC and the underlying layer.

In recent years, interface shear performance between different pavement layers has been a hot research topic (Raab and Partl, 2009; Raposeiras et al., 2012; Mohammad et al., 2002; Collop et al., 2009; West et al., 2005; Chen and Huang, 2010; Canestrari et al., 2005; Song et al., 2015). Researchers have explored different factors that affect the bonding properties, including tack coat, mixture type, temperature, and surface characteristics (Mohammad et al., 2002; West et al., 2005; Chen and Huang, 2010; Song et al., 2015). It is commonly concluded that the bonding properties generally become worse as temperature goes up (Mohammad et al., 2002; West et al., 2005; Chen and Huang, 2010; Canestrari et al., 2005). Chen and Huang (2010) compared the bonding strength of specimens combined by dense-graded asphalt concrete (DGAC), SMA, and porous asphalt concrete (PAC) and their ranking of the shear strength was DGAC-DGAC (upper layer-lower layer) > PAC-DGAC > PAC-SMA. Mohammad et al. (2002) and Raposeiras et al. (2012) analyzed the effects of tack coat dosage and concluded that there exists one optimal tack coat dosage at which the shear strength reaches the maximum value. However, Collop et al. (2009) found that better shear strength results can be obtained sometimes even without tack coat application. Other researchers also investigated the effects of tack coat type (Mohammad et al., 2002; West et al., 2005) and tack coat breaking time (Chen and Huang, 2010; Tashman et al., 2008).

The shear fatigue behavior between pavement layers has also been studied by many investigators (Boudabbous et al., 2013; Li et al., 2014; Diakhaté et al., 2011; Li and Yu, 2013). Tack coat and temperature are found to be two important factors that affect the fatigue properties between pavement layers (Li et al., 2014). Some researchers used power
law equations to describe the relationship between fatigue life and stress level for direct shear test and oblique shear test (Li et al., 2014; Diakhaté et al., 2011). As in shear strength test, tack coat type is a major factor affecting the fatigue performance. Li and Yu (2013) found that epoxy tack coat gives a remarkably superior shear fatigue performance than styrene-butadiene-styrene (SBS)-modified asphalt and emulsified asphalt tack coat.

Since its first introduction in 1972 (Van Dijk et al., 1972), energy approach has been extensively used in asphalt concrete fatigue research and the dissipated energy method has been used to evaluate the fatigue properties of asphalt mixtures.

Through a series of experiments, many researchers developed the relationships between the total cumulative dissipated energy and the fatigue life of asphalt mixtures (Van Dijk, 1975; Chomton and Valyer, 1972; Carpenter and Jansen, 1997) and they found that the relationship is not significantly affected by loading mode, frequency, temperature, and rest period, but highly dependent on the material itself. Research also shows that the initial dissipated energy serves as a good indicator of fatigue performance (Rowe, 1993; Ghuzlan, 2001). Baburamani and Potter (1996) found that there exists a good correlation between the initial dissipated energy and fatigue life. But other researchers showed that the correlation was not consistent at low strain/damage levels (Carpenter and Shen, 2006). Tayebali et al. (1992) introduced stiffness ratio and dissipated energy ratio and found a unique relationship between the two, but not necessarily between cumulative dissipated energy and fatigue life. Boudabbous et al. (2013) used cumulative energy and dissipated energy ratio to characterize the fatigue properties of pavement surface layers in both strain control and stress control loading modes.

More recently, researchers introduced the concept of the ratio of dissipated energy change (RDEC) (Carpenter and Shen, 2006; Shen and Carpenter, 2005; Ghuzlan and Carpenter, 2000; Shu et al., 2008). RDEC represents the total effect of fatigue damage without the consideration of mixture type and loading modes. It provides a true indication of the damage being done to the mixture from one cycle to another by comparing the previous cycles energy level and determining how much damage it caused (Carpenter et al., 2003).
4.3 Objective and scope

The objective of the paper was to investigate the interlayer shear fatigue performance of the composite specimens consisting of OGFC and the underlying layer through laboratory testing. Two types of dense graded asphalt mixture were used as the underlying layer, combined with one gravel OGFC to make the two-layered composite specimens. Direct shear fatigue test was performed to obtain their fatigue properties. In addition to the conventional 50% stiffness reduction method, the cumulative dissipated energy and ratio of dissipated energy change of the energy approach were also employed to analyze the fatigue behavior. The interface contact area was measured and further related to the fatigue properties of the composite specimens.

4.4 Laboratory fatigue test

4.4.1 Direct shear fatigue test device

The direct shear fatigue device includes four semi-circular steel rings (Figure 4.1). It was mounted to an MTS machine to apply shear force in the vertical direction. During each loading cycle, both the shear force and displacement were recorded at a predetermined frequency.

4.4.2 Experimental program and loading condition

One type of gravel OGFC mixture was used in the study. Two types of dense graded asphalt mixture were selected as the underlying layer material and they are called BM and TLD in the state of Tennessee. Table 4.1 presents the material information of OGFC, TLD, and BM mixtures. Figure 4.2 shows the aggregate gradation of the three mixtures. The nominal maximum aggregate size (NMAS) was 12.5mm, 19.5mm, and 9.5mm, respectively, for OGFC, BM and TLD. The tack coat material was an anionic slow setting asphalt emulsion of grade SS-1, which meets the specification of the Tennessee Department of Transportation (TDOT).
Table 4.1. Aggregate type and asphalt cement

<table>
<thead>
<tr>
<th>Mixture property</th>
<th>OGFC</th>
<th>Underlying layer</th>
<th>TLD</th>
<th>BM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate type</td>
<td>Gravel</td>
<td>Limestone</td>
<td>Limestone</td>
<td></td>
</tr>
<tr>
<td>Asphalt PG grade</td>
<td>PG 76-22</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
<td></td>
</tr>
<tr>
<td>AC Content</td>
<td>6.4%</td>
<td>6.2%</td>
<td>4.2%</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.2. Aggregate gradation
The test specimens consisted of two layers with a 150-mm diameter. The bottom layer was underlying layer (BM or TLD) and the upper layer was OGFC. The height was 31.8 mm and 50.8 mm, respectively, for OGFC and underlying layer, which was determined following the specifications of the Tennessee Department of Transportation (TDOT). To fabricate the composite specimens, the underlying layer was first compacted using a Superpave gyratory compactor (SGC). Then the specimen was extruded. After it cooled to ambient temperature (20 °C), tack coat material was evenly applied to the surface of the underlying layer specimen. Four tack coat application rates were used: 0, 0.15 l/m², 0.3 l/m², and 0.5 l/m². The tack coat application rate was calculated based on the residual solid binder from tack coat. After about 30 minutes, the specimen was put back into the gyratory mold with the tack coat surface facing upwards, and then the loose mix of OGFC was put into the mold and compacted. The compressive pressure was 600 kPa and the compaction temperature of OGFC was 145 °C. OGFC was compacted to approximately an air void content of 18%, while BM and TLD were compacted to 4% air voids. Tiral compaction of a single layer OGFC was first conducted to obtain the air voids of 18%, from which the amount of OGFC material needed and the number of gyrations required were determined. The required number of gyrations was then used for the compaction of the OGFC layer of the two-layer specimens so that the air voids of the OGFC layer could be controlled at approximately 18%.

The direct shear fatigue test was performed using a sinusoidally cyclic stress loading mode at 10 Hz and 20 °C. Two stress levels of 0.4 MPa and 0.8 MPa were used based on the shear strength of the composite specimens and the corresponding stress ratios were approximately 0.3 and 0.6. The reason for selecting 10 Hz loading frequency is that it represents a common traffic speed of approximate 80 km/h (Boudabbous et al., 2013). The fatigue test was conducted in triplicate.

4.4.3 Fatigue analysis methods

Two methods were employed to analyze the results from the shear fatigue test: the traditional 50% stiffness reduction method and the energy approach. For the 50% stiffness reduction method, the failure criterion is conventionally defined as a 50% decrease in the
maximum value of the shear stiffness modulus (Diakhaté et al., 2011; Kim et al., 1995, 1997) (Figure 4.3). The fatigue life is determined to the number of loading cycles to failure.

\[ RDEC = \frac{|w_j - w_i|}{(j - i)w_i} \quad (4.1) \]

where, \( w_j \) is the dissipated energy in \( j^{th} \) cycle, \( w_i \) is the dissipated energy in \( i^{th} \) cycle, \( j > i \).

Figure 4.4 shows a typical plot of RDEC vs. loading cycle. Three regions can be clearly observed. In the first stage, RDEC decreases significantly as the loading cycle increases. In the second or plateau stage, the RDEC value is almost constant, which is called plateau value (PV) and can be used as an indicator of fatigue property. PV represents a constant percentage of input energy that is turned into fatigue damage, thus providing an indication of fatigue performance. Generally, the lower the PV, the lower the percentage of input energy being turned into damage, the better the fatigue performance (Shen and Carpenter, 2006; Wu et al., 2013). During the third stage, RDEC increases with the increase in loading cycles. Carpenter and Shen (2006) suggested that PV be used as a fatigue failure criterion.
4.5 Results and discussion

4.5.1 Fatigue life performance

The fatigue life of OGFC with two different underlying layers was determined according to the 50% stiffness reduction method. Figure 4.5 shows the fatigue life of OGFC-BM and OGFC-TLD at different stress levels and different tack coat dosages. At the same tack coat application rate, OGFC-TLD appeared to have a longer fatigue life than OGFC-BM. As tack coat application rate increased, the number of loading cycle to failure generally decreased, indicating that the maximum fatigue life was obtained when no tack coat was applied. This phenomenon may be caused by the cyclic loading mode used in the laboratory fatigue test. The continuous cyclic loading may convert energy into heat, which warmed up tack coat and made it serve as a lubricating agent instead of a bonding agent. In the field, there is almost always a rest period between two traffic loadings during which heat could dissipate into the atmosphere. Although the maximum fatigue life was obtained for the specimens without tack coat in the study, it is recommended that proper tack coat application rate be carefully selected considering the differences between laboratory and field loadings as well as many other factors including bonding, shear strength, and durability properties of pavements.
Figure 4.5. Fatigue life results based on 50% stiffness reduction method

4.5.2 Ratio of dissipated energy (RDEC)

Figure 4.6 shows the load vs. displacement hysteresis loops of the first, 100\textsuperscript{th}, 9000\textsuperscript{th}, and 10000\textsuperscript{th} loading cycles of OGFC-TLD at the shear stress of 0.4 MPa for the specimens without tack coat applied. Because of viscoelastic nature, the maximum load of the first loop did not reach the predetermined load level. Figure 4.7 reveals that during the first several cycles, the dissipated energy increased significantly, and then dissipated energy became stable and showed a slight increase, which indicates that fatigue damage developed gradually (appearance of micro cracking). During the last several cycles, the dissipated energy showed a great increase again, implying that significant damage was occurring, leading to the failure of the specimen. The significant damage in the last phase was caused by both the coalescence and rapid propagation of macroscopic cracks at the interface.
Figure 4.6. Load-displacement hysteresis loops

Figure 4.7. Dissipated energy vs. Load cycle
RDEC was obtained according to Equation 4.1. The plateau value (PV) was used to analyze the fatigue behavior. Figure 4.8 presents the evolution of RDEC of OGFC-BM with the loading cycles at 0.4 MPa without tack coat applied.

![Figure 4.8. RDEC vs. number of loading cycle](image)

Figure 4.9 shows the change of PV with tack coat dosage and Figure 4.10 shows the relationship between PV and fatigue life. Generally, as the tack coat dosage increased, OGFC-BM and OGFC-TLD both experienced an increase in PV. As is known, the lower the PV, the lower the percentage of input energy being turned into damage, the better the fatigue performance. The relationship between PV and fatigue life shown in Figure 4.10 further validated this phenomenon. This also indicates that OGFC-BM and OGFC-TLD with a lower PV would show the best fatigue performance when tack coat was not applied. At the same tack coat dosage, OGFC-BM showed a higher PV than OGFC-TLD, indicating that OGFC-TLD would perform better than OGFC-BM in terms of fatigue performance. The relationship between $N_{f_{50}}$ and PV is plotted in Figure 4.11. It can be seen that a power law equation could be used to describe the relationship between PV and $N_{f_{50}}$. This relationship is consistent with the findings of other researchers (Shen and Carpenter, 2006; Wu et al., 2013). The power function is shown as Equation 4.2. The power law equation was independent of the underlying layer material and tack coat application rate.

$$PV = 0.0336 N_{f_{50}}^{-0.585} \quad (R^2 = 0.954)$$  \hspace{1cm} (4.2)
Figure 4.9. PV results

(a) 0.4 MPa

(b) 0.8 MPa

Figure 4.10. PV vs. fatigue life

(a) 0.4 MPa

(b) 0.8 MPa

Figure 4.11. Fitting between PV and fatigue life
4.5.3 Cumulative dissipated energy

The cumulative dissipated energy of OGFC-BM and OGFC-TLD was obtained at each loading cycle. The total dissipated energy was plotted against the tack coat dosage and shown in Figure 4.12 for both stress levels. Figure 4.12 indicates that OGFC-TLD released a higher cumulative dissipated energy than OGFC-BM at all tack coat dosages when fatigue failure happened. For both OGFC-TLD and OGFC-BM, when tack coat dosage increased, the total dissipated energy decreased at both 0.4 MPa and 0.8 MPa stress levels.

![Graph showing Total dissipated energy vs Tack coat application rate](image)

**Figure 4.12.** Total dissipated energy

To further examine the relationship between the cumulative dissipated energy ($W_{N_f}$) and the fatigue life determined according to the 50% stiffness reduction method ($N_{f_{50}}$), $W_{N_f}$ was plotted against $N_{f_{50}}$ in Figure 4.13 for OGFC-TLD and OGFC-BM. The relationship between $W_{N_f}$ and $N_{f_{50}}$ was plotted in Figure 4.13. Power law equations were fitted to explore the relationship. It is found that there exists a good correlation between $W_{N_f}$ and $N_{f_{50}}$ for OGFC-TLD, but the $R^2$ value of OGFC-BM was a litter lower. With all the data used in the regression, the $R^2$ value of Equation 4.5 was 0.816, indicating that power law relationship between total cumulative dissipated energy and fatigue life was independent of tack coat application rate, but dependent on the underlying layer material. This is generally in agreement with the findings from other researchers (Van Dijk et al., 1972; Carpenter and Jansen, 1997). But further research is still needed in the future for the relationship between $W_{N_f}$ and $N_{f_{50}}$ considering the variables of underlying layer and tack coat application rate.
\[ W_{N_{f,TLD}} = 151.77 N_{f_{50}}^{0.489} \quad (R^2 = 0.907) \quad (4.3) \]
\[ W_{N_{f,BM}} = 16.12 N_{f_{50}}^{0.707} \quad (R^2 = 0.797) \quad (4.4) \]
\[ W_{N_{f,All}} = 30.82 N_{f_{50}}^{0.661} \quad (R^2 = 0.816) \quad (4.5) \]

Figure 4.13. Fitting between total energy and fatigue life

### 4.6 Interface characterization

Interface characteristics between OGFC and its underlying layer play an important role in interlayer bonding (Chen and Huang, 2010). The contact area between OGFC and the underlying layer is vital to ensure a good bonding between different layers. Figure 4.14 and Figure 4.15 show the bottom surface photographs of OGFC compacted on BM and TLD after shear fatigue failure. The left pictures are original pictures after failure and the right ones are those after image processing of binarization. A binary image is a digital image that has only two possible values for each pixel: 0 or 1.
Figure 4.14. Failure OGFC surface (OGFC-BM)

In the original picture, some of the darkest spots can be clearly differentiated from others. The darkest spots represent the asphalt binder of OGFC, where there were voids and thus no contact area between OGFC and the underlying layer. Using image binarization, the dark areas could be identified and removed from the total area. Thus, the contact area between OGFC and its underlying layer could be calculated.

Figure 4.15. Failure OGFC surface (OGFC-TLD)

Figure 4.16 shows the results of the contact area between OGFC and the underlying at tack coat application rates 0.15 l/m$^2$ and 0.30 l/m$^2$. It clearly shows that the contact area between OGFC and TLD was larger than that of OGFC-BM. The contact area at the tack coat rate of 0.30 l/m$^2$ was also found to be larger than that at 0.15 l/m$^2$, which may be attributed to the fact that as more tack coat was applied, more tack coat filled the voids at the interface after the compaction of OGFC.
Figure 4.17 present the relationship between fatigue life and contact area at the tack coat rates of 0.15 l/m² and 0.30 l/m² respectively. It can be clearly seen that at the same tack coat rate, the fatigue life of the specimens increased with the increase in the contact area between the two pavement layers. Although the tack coat rate of 0.30 l/m² resulted in a higher contact area than 0.15 l/m², the higher rate also increased the lubricating effect, leading to a compromised fatigue life than 0.15 l/m².

![Graph of Figure 4.17: Contact area vs. fatigue life at tack coat rate 0.15 l/m² and 0.30 l/m²](image)

**Figure 4.16.** Results of contact area

![Graph of Figure 4.16: Contact area vs. fatigue life at tack coat rate 0.15 l/m² and 0.30 l/m²](image)

**Figure 4.17.** Contact area vs. fatigue life at tack coat rate 0.15 l/m² and 0.30 l/m²
4.7 Summary and conclusions

This study evaluated the laboratory shear fatigue performance of OGFC combined with different underlying layers (BM and TLD). Fatigue life, cumulative dissipated energy, and RDEC were used to analyze the fatigue behavior of the composite specimens. The contact area between OGFC and its underlying layer were tested and correlated to the fatigue performance. Based on the results and analyses, the following conclusions can be drawn:

- The fatigue life of OGFC-TLD was longer than that of OGFC-BM. With the increase in tack coat dosage, the number of loading cycle to failure decreased, which may be attributed to the lubricating effect of tack coat.

- The plateau value (PV) of OGFC-TLD was lower than that of OGFC-BM under the same loading condition, leading to a better fatigue performance of OGFC-TLD. In addition, with the increase in tack coat dosage, PV increased for both OGFC-TLD and OGFC-BM, implying a degraded fatigue performance because more dissipated energy was turned to fatigue damage. Power law relationship was found to exist between PV and fatigue life, which was independent of the underlying layer type and tack coat dosage rate.

- The total dissipated energy of OGFC-TLD was larger than that of OGFC-BM at same stress and same tack coat dosage. As tack coat dosage increased, total dissipated energy decreased. There existed a power law relationship between cumulative dissipated energy and fatigue life, which was independent of tack coat application rate, but dependent on underlying layer material.

- The OGFC-TLD interface contact area was larger than that of OGFC-BM, leading to a better fatigue performance of OGFC-TLD.

- The plateau value failure criterion appeared effective for evaluating the shear fatigue performance of multilayer structures.
4.8 Acknowledgements

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Chapter 5

Laboratory evaluation of pressure distribution under steel and timber crossties in railroad track
A version of this chapter was originally published by Weimin Song, Xiang Shu, Baoshan Huang, Yiren Sun, Hongren Gong and David Clark:


Weimin Song was the principle researcher and author of Laboratory evaluation of pressure distribution under steel and timber crossties in railroad track. Weimin Song’s contribution was conducting all literature review, testing, data analysis, and writing the text contained in the manuscript. Dr. Xiang Shu, Dr. Baoshan Huang, Yiren Sun, Hongren Gong and Dr. David Clark provided guidance and test help throughout the research process as well as editorial assistance.

5.1 Abstract

The pressure distribution under the crossties plays a key role in railroad performance. In this study, laboratory testing was conducted to investigate the pressure distributions under two different types of crossties, steel and timber crossties. For each type, only one single tie was employed for the purpose of this study. Five pressure cells were equidistantly placed under the rail crosstie between the two rails to measure the pressure distribution. Both static and cyclic loadings were applied during the test. The pressure distributions were compared between the steel and timber crossties. The effect of the cyclic loading on the pressure distribution was also explored. It is found that pressure distribution was different for steel and timber crossties. Cyclic loading could change the pressure distribution under both steel and timber crossties, but the effect of cyclic loading was more obvious on steel crosstie than on timber crosstie. There existed differences in pressure distribution between loading and unloading processes.
5.2 Introduction

The rapid development of railroad transportation requires a longer service life of the crossties and more safe train travel. Research on the pressure distribution under crossties offers a better understanding of the interactions between ballast and crossties, and further provides assistance for rail transportation guidelines.

Two main types of railroad track are used in the world: track supported by ballast and ballastless track. The ballastless track is designed mainly for high speed railway transit. For the ballasted track system, crossties and ballast are two key components. Timber crossties account for about 90-95% of all the crossties in the US (Csenge et al., 2015). Steel crossties make up only a very small part and they are generally used for light density secondary track. Although timber ties are more widely used in America, light rail transit systems constructed with timber ties need to replace a large percentage of the timber crossties only after a service life of 20 to 30 years (Brinckerhoff, 2012). In contrast, the steel crosstie usage is steadily rising because of such benefits as a long service life, easy installation, and cost effectiveness. Besides the timber and steel crosstie, there is also a small part of concrete crossties used in US.

Pressure distribution under crossties is of great importance to a railway track system. Previous research shows that the tie-to-ballast pressure is not uniformly distributed under the ties. The American Railway Engineering and Maintenance-of-Way Association (AREMA) recommends that the calculation of the pressure should take into account of the distribution and impact factors (AREMA, 2016). In the railway track design, AREMA proposes four equations describing the relationships between tie-to-ballast pressure and other parameters, including ballast depth and wheel load (AREMA, 2016). McHenry et al. (2015) conducted one field test to determine the stress distribution under timber and concrete crossties and considered such factors as the contact area between the ballast and crossties. Research from Laryea et al. (2014) made the comparison of the stress under the concrete crosstie and steel crosstie. The research showed that for the concrete sleeper, the load is mainly transferred vertically. However, the steel sleeper section presented a large force concentration under its edge while a significant part of the load is still vertically transmitted through the underside
of the sleeper. After one million cycles loading, for the steel sleeper, the stress at the center of the sleeper crosstie was higher than the stresses beneath the rail seats. However, for the concrete crosstie, the stress beneath the rail seat was larger after the cyclic loading, and the pressure distribution belongs to the parabolic pattern (Laryea et al., 2014; Sadeghi, 2008, 2010).

The testing set up method of the pressure distribution measurement plays an important role in the accuracy of the results. In some studies, pressure cells were placed at the interface between ballast and crossties (McHenry et al., 2015; Sadeghi, 2008, 2010). In other studies, pressure cells were just placed on ballast, and then crossties were placed on the pressure cells. This setup method may cause inadequacy contact between crosstie and ballast, especially when the pressure cells were thick. Some researchers embedded pressure cells within the bottom of the ties so that the surfaces of the pressure plates would be flush with the bottoms of the ties (Sadeghi, 2008, 2010). This method could give more reliable test results.

Besides the laboratory tests and filed tests, discrete element method and finite element method were utilized in the research of ballast under static and cyclic loadings (Indraratna et al., 2009; Hossain et al., 2007; Recuero et al., 2011; Kuo and Huang, 2009). Ballast degradations are observed under the external load especially under the cyclic loading, and the crossties cannot be fully supported by the ballast, which may accelerate the degradation of the crossties (Indraratna et al., 2009; Hossain et al., 2007; Recuero et al., 2011; Sun et al., 2015; Anderson and Fair, 2008).

In the research of the crosstie behavior or the pressure distribution under the crossties, many technologies have been employed, such as the matrix-based tactile surface sensor (McHenry et al., 2015; Rapp et al., 2013; Rose et al., 2004), geokon pressure cell (Rose et al., 2004; Anderson and Rose, 2008; Jia et al., 2009), strain gauge (Wolf et al., 2015) et al. In this study, the geokon pressure cell was used to measure the pressure distribution.

The objective of this study was to investigate the pressure distribution under steel and timber crossties through laboratory testing. The pressure distributions under steel and timber crossties was compared and analyzed. The effect of cyclic loading on the pressure distribution under the ties was also examined.
5.3 System for pressure distribution measurement

This system is composed of four parts: (1) a power source which provides excitation voltage, (2) a data acquisition device, (3) pressure cells, and (4) a computer running the data acquisition and analysis software. They are shown in Figure 5.1.

A pressure cell is fabricated by welding two steel plates together, leaving a narrow space between them filling with hydraulic oil. The hydraulic oil is connected hydraulically to a pressure transducer where the oil pressure is converted into the electrical signal which can be read by the computer. The area of the pressure cell is 0.03 m². The data acquisition device (NI 9203) is a device with 10 terminals.

![Figure 5.1. System for pressure distribution measurement components](image)

5.4 Laboratory tests

A single steel and timber crossties were used in the test. The dimension is 2.54 × 0.16 × 0.14 m (length×width × height) for the steel crosstie and 2.59 × 0.23 × 0.18 m for the timber crosstie. The section information of the steel crosstie was shown in Figure 5.2. The thickness of the steel crosstie is 0.008 m. The bending rigidities of the steel and the timber crosstie are 0.73 MPa·m³ and 1 MPa·m³. The ballast gradation information is shown in Figure 5.3, which is classified as AREMA #4A.
5.4.1 Calibration of the pressure distribution measurement system

The calibration was conducted by utilizing a Material Testing System (MTS) machine. Ballast was filled in one strong wood box with the upper side open. The size of the wooden box is $0.4 \times 0.4 \times 0.14$ m (length × width × height). One wood block cut from a timber crosstie was used as a crosstie. A pressure cell was placed between the wood block and the ballast, as shown in Figure 5.4.

Pressures were calculated by dividing the recorded force from MTS by the area of the pressure cell. The tie loads were transferred through the crosstie over the total area of the cell. The calculated pressures were compared to the simultaneously measured values indicated by the pressure cells from 0 to 26700 N.
5.4.2 Laboratory test of the timber and steel crosstie

Test set up

In this study, five pressure cells (numbered from 24 to 28) were placed under the ties symmetrically. Figure 5.5 to Figure 5.7 show the locations of the pressure cells and the test set up. In Figure 5.5, the distance between the two adjacent pressure cells was 0.38 m. One steel beam was placed on the rails to apply the external load at the center of the crosstie.

Research showed that the hollow-shape steel crosstie presents a high stress concentration under its edges (Laryea et al., 2014). To reduce the effect of the edge of the crosstie on the pressure distribution, pressure cells were located at a depth of 0.13 m beneath the crosstie rather than at the interface between the ballast and crosstie, as Figure 5.5 shows.

Ballast was first compacted with a vibratory compactor before the placement of the pressure cell. The porosity after the ballast compaction was about 50%. The depth of the ballast beneath the sleeper is about 1 m, which meets the requirement of the AREMA specification (AREMA, 2016). After the placement of the pressure cell, the ballast was leveled and the crosstie was installed above the pressure cells. Before actual test, preloading was applied to ensure good contact between ballast and the pressure cells.
Figure 5.5. Schematic diagram of the test

Figure 5.6. Pressure cell placement

Testing procedure

The loading and testing procedure consisted of three parts. The first part was to apply a static loading. In this part, an external force was applied from 0 to 220000 N in five stages and then removed in the same manner (Figure 5.8(a)). The pressure values were measured and recorded at six load levels: 0, 44000 N, 88000 N, 132000 N, 176000 N and 220000 N. In the loading and unloading process, the pressure values read by the measurement system may be unstable sometimes, so the duration at each load level was set as 1 minute, which is long enough to give stable reading of the pressure values, as Figure 5.8(a) shows. Sinusoidal loading is commonly used to simulate the cyclic loading in laboratory tests (Anderson and Fair, 2008; Indraratna et al., 2009; Laryea et al., 2014). In this study, after the static
loading test, a cyclic loading ranging from 4400 N to 44000 N was performed at 2 Hz for 10000 cycles (Figure 5.8(b)). The loading amplitude was selected to simulate the 6-axle light weight loading. During this loading, the pressure development was recorded. After the cyclic loading, the static loading procedure was performed again in the same manner described above to evaluate the influence of the cyclic loading on the pressure distribution.
5.5 Results and discussion

5.5.1 Calibration of the pressure distribution system

The calibration was made at five loading levels, 0, 6700 N, 13400 N, 20100 N and 26800 N. Three repetitive tests were performed to evaluate the repeatability of the pressure cells. The relationship between the calculated pressures and measured pressures is shown in Figure 5.9. It can be seen that there is a good consistency between the calculated pressures and the measurements. The calibration results confirmed the reliability of the pressure distribution measurement system.

![Figure 5.9. Calibration of the pressure distribution system](image)

5.5.2 Result of the static and cyclic loading tests

Figure 5.10 displays the pressure distributions along the steel crosstie before and after the cyclic loading in the loading process from 0 to 220000 N.

For the steel crosstie during the first static loading, as the external force increased, the pressure increased as well. When a low external force (44000 N and 88000 N) was applied, the pressure at the center was higher than that at other locations. As the external force increased, the pressure at the halfway locations between the rail seat and the center (cell 25 and cell 26) experienced a marked increase and became higher than the pressures at
other locations. This trend continued until the highest loading level. After the 10000-cycles dynamic loading, the pressure distribution was different from that before the cyclic test. The pressure at the center was always the highest among all the locations at all external loading levels and the pressures at the two ends were the lowest, which agrees with the research from Laryea et al. (2014).

Figure 5.11 presents the pressure increase at every loading level for the steel crosstie before and after cyclic loading. It can be observed that at the rail seat, the pressure increment at every loading level increased before and after the cyclic loading. However, the pressure increment at the halfway location between rail seat and the center remained unchanged at every loading level. For the pressure at the center, the pressure increment decreased significantly at low level loading, and at high loading level (132000 N, 176000 N and 220000 N), the pressure increment remained constant before and after the cyclic loading.

Figure 5.12 shows the pressure distributions under timber crosstie before and after the cyclic loading. As in the steel crosstie testing, the pressures under the timber crosstie all experienced an increase with the increase in the applied load. However, the increase trend was different between steel and timber crossties. At the lowest loading level (44000 N), the pressures at the middle three locations were approximately equal and higher than those at both ends. As the external load increased, the increase in pressure under the timber crosstie
worked from outside to inside, indicating that the four outside locations experienced a higher increase than the center location. This caused the two locations midway between rail seat and center experienced the highest pressure at two intermediate loading levels (88000 N and 132000 N) and finally the two rail seat locations experienced the highest pressure at the high loading levels (176000 N and 220000 N). The pressure distribution under the high external loads was generally in agreement with those published results (Talbot, 1929). As the external force increased from 44000 to 220000 N, the pressures at two rail seats increased most quickly from 0.04 to 0.48 MPa, followed by those at the two locations midway between rail seat and center from about 0.1 MPa to about 0.42 MPa. In comparison, the center location only experienced an increase in pressure from 0.1 to 0.21 MPa when the load was increased from 44000 to 220000 N. After the cyclic loading of 10000 cycles, the pressure distribution was similar to that before the cyclic loading and the only difference was that at the lowest loading level (44000N), the pressure at the center was highest, decreasing farther away from the center.

Comparison of the pressure distribution under steel crosstie and timber crosstie (Figure 5.10 and Figure 5.12) shows that the pressure distribution under steel crosstie was more uniform than that beneath timber crosstie, especially at high loading levels. It may be because compared with the timber sleeper, the edges of the steel crossties restricted the
ballast movement in horizontal direction and made the ballast beneath the steel crosstie more compacted.

Figure 5.13 presents the pressure increment at every loading level for the timber crosstie before and after the cyclic loading. The pressure increment at the center and at the rail seat was similar with the steel crosstie. However, as the external load increased, the pressure increment decreased before and after the cyclic loading.
To further the understanding of the effect of the cyclic loading on the pressure distribution, the changes in maximum and minimum pressures with the loading cycles were examined. Figure 5.14 and Figure 5.15 show the changes in maximum and minimum pressures with the increase in loading cycles for steel and timber crossties, respectively. The maximum pressure was obtained at the external load of 44000 N, while the minimum pressure value was obtained at the 4400 N load. It can be observed that with the increase in loading cycles, the pressure at the center of the crosstie (cell 26) significantly increased for both steel and timber crossties, but the pressure at the two midway locations significantly decreased. The pressure beneath the rail seats did not show much change.

Figure 5.14. Pressure development under steel crosstie during the cyclic loading

Figure 5.15. Pressure development under timber crosstie during the cyclic loading
Figure 5.16 and Figure 5.17 compare the pressures at different locations of crosstie before and after the cyclic loading. Figure 5.18 displays the percentage of the pressure change after the cyclic loading. For the steel crosstie, after the cyclic loading process, the pressure at rail set showed a slight decrease, and from Figure 5.18 the decrease was about 6% at 220000 N. The pressure at the halfway location also showed a decreasing trend with a decrease of about 17% at the maximum loading level. The decrease was more significant at the halfway location than at the rail seat. Unlike pressures at these two locations, the pressure at the center of the steel crosstie showed a significant increase after the cyclic loading, which was 13% at 220000 N. This indicates that cyclic loading could improve the ballast support to the steel crosstie at the center location, but compromise the support at other locations, especially at the halfway location between the rail seat and the center. For the timber crosstie, it can be observed from Figure 5.17 that as the external load increased, all the measured pressures increased, but the increase rate at the rail seat was the largest, followed those at the halfway and center locations. The pressure at the rail seat showed almost no difference before and after the cyclic loading. Similar to the steel crosstie, the pressure at the center of timber crosstie also increased after the cyclic loading, whereas the pressure decreased at the halfway location. The change in pressure was 14% for the center and -10% for the halfway location, indicating that cyclic loading could enhance ballast support at the center of the timber and reduce the support at the halfway location. However, comparison of the pressures under steel and timber crossties shows the effect of the cyclic loading on steel crosstie was more significant than on the timber for the center and halfway locations at the loading level from 44000 N to 220000 N (Figure 5.18). The bending rigidity values of the timber sleeper and steel sleeper are 1 MPa·m³ and 0.73 MPa·m³. Under the external loading, the steel crosstie is more easily to deform than the timber sleeper. The difference in bending rigidity between steel and timber crossties caused the pressure difference under the steel and timber crossties. The shapes of the steel and timber crossties are also different, which may contribute to the difference in pressure under steel and timber crossties.
Figure 5.16. Pressure comparison of steel crosstie between initial condition and after the cyclic loading

Figure 5.17. Pressure comparison of timber crosstie between initial condition and after the cyclic loading

Figure 5.18. Percentage of the pressure change after cyclic loading
The pressures of the five cells were also recorded during the unloading process from 220000 N to 0. Figure 5.19 and Figure 5.20 compares pressures obtained from the loading and unloading processes. The pressure results were obtained during the first static loading test prior to the cyclic loading. For both the steel and timber crossties, the pressures obtained during the unloading process at rail seat were lower than the corresponding values obtained from the loading process, but the opposite was observed for pressures at the halfway and center locations. Figure 5.21 shows the percentage of the pressure change after the unloading process. At the halfway and center locations, the general trend was that the percentage increase in pressure under the steel crosstie was larger than that for the timber crosstie. The decrease percentage of the pressure beneath the rail seat was similar for the steel and the timber crossties at the different loading levels in this study.

![Figure 5.19. Pressure comparison of steel crosstie between loading and unloading process](image)
5.6 Conclusions

A laboratory study was carried out to investigate the pressure distribution characteristics under a single steel or timber crosstie. Five pressure cells were equidistantly located under the crossties between the two rail seats. The pressure measurement system was first calibrated. The static and cyclic loadings were applied in the laboratory test. The difference in pressure distribution was analyzed between the steel and timber crossties. The effect of cyclic loading on the pressure distribution under the crossties was also examined. The following conclusions can be drawn from this study:
• The pressure measurement system was reliable for measuring the pressure distribution under the steel and timber crossties.

• In the static loading test, at a low loading level (44000 N and 88000 N), the pressure at the center of the steel crosstie was the highest among all the pressures measured at different locations. As the applied load increased, the pressures at the halfway location between the rail seat and the center gradually became the highest ones. But for the timber crosstie, with the increase in the loading level, the maximum pressure was obtained directly beneath the rail seats.

• Cyclic loading could increase the pressure at the center of the crosstie, and significantly decrease the pressure at the halfway locations between the rail seat and the center. The pressures beneath the rail seats showed a slight decrease.

• The effect of the cyclic loading on the pressure distribution of the steel crosstie was more significant than for timber crosstie.

• There existed a difference in pressure distribution between the loading and unloading processes. For both the steel and timber crossties, the pressures obtained from the unloading process beneath the rail seats were lower those from the unloading process, but the pressures at other locations were higher from the unloading process than from the loading process at the same loading level.

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Chapter 6

A discrete-finite element framework for 3D modeling of the interaction between crosstie and ballast under static and cyclic loading
A version of this chapter is in the process of being submitted for publication by Weimin Song, Baoshan Huang and Xiang Shu:


Weimin Song was the principle researcher and author of "A discrete-finite element framework for 3D modeling of the interaction between crosstie and ballast under static and cyclic loading". Weimin Song’s contribution was conducting all literature review, modeling test, data analysis, and writing the text contained in the manuscript. Dr. Baoshan Huang and Dr. Shu provided guidance and test help throughout the research process as well as editorial assistance.

### 6.1 Abstract

One discrete-finite element framework was set up to investigate the interaction between ballast and railway crosstie under the dynamic loading. Two types of crosstie, timber and steel crosstie, were selected in this study. The discrete element method (DEM) was used to model the behavior of ballast and the finite element method (FEM) was used to analyze the continuous railway crosstie. Two open source codes (YADE and OOFEM) were employed to conduct the DEM-FEM modeling work. The pressure distribution and the force chains in the ballast were obtained. Results show that cyclic loading significantly changed the stress distribution under both the steel and timber crosstie. The pressure under the rail seat decreased after the cyclic loading, while the pressure at the center of the crosstie increased. The effect of the cyclic loading was more significant on the steel sleeper. The stress distribution pattern of steel sleeper is different from the timber crosstie. There was significant stress concentration under the edges of the steel crosstie.
6.2 Introduction

Ballast is one extensively used material in transportation engineering. In railway engineering, the external loadings are applied on the rails and then transferred to the sleepers and ballast. Behaviors of ballast under the external loading are critical concerns in the engineering application of ballast and railway design. The interaction between ballasts and sleepers has been attracting enormous attention in the last few decades. Under the long term effect of train loadings, the flow of ballast occurs, so the interaction behavior of ballasts and railway tracks becomes complicated, accompanying the change of settlement and stiffness of ballast (Augustin et al., 2003; Jeffs et al., 1987; Ji et al., 2017; Baessler and Ruecker, 2003), the degradation of ballast (Lackenby et al., 2007; Indraratna et al., 1998), et al. It is of great importance to investigate the ballast behavior under external loading.

Some laboratory tests and field tests were conducted to explore the various behaviors of ballast (Aursudkij, 2007; Huang et al., 2009; Zhai et al., 2004). Although the laboratory and filed tests gave many valuable conclusions, the test work conducted in laboratories or field is generally laborious and it is difficult to control some variables which may lead to poor test results. In this case, numerical methods are becoming extensively used in the analysis of ballast behavior. The continuum approach (e.g., FEM) averages the physics across many particles and thereby treats the material as a continuum on a macro scale level (Chen, 2009). The FEM has proven to be a powerful tool in modeling structural elements, such as beam, pile, et al. Using the continuum approach, the stress-strain properties and the interaction between ballasts and tracks were studied (Zhai et al., 2004; Gao et al., 2016; Ricci et al., 2005). However, ballast is the collection of lots of distinct particles and inherently inhomogeneous. It is difficult to treat such a material as a continuum media, especially in the analysis of the interaction between ballast particles. The Discrete Element Method (DEM) is a discontinuous approach and a powerful numerical tool for computing the motion of a large number of granular particles and offers one micro-mechanical insight into its behavior (Cundall and Strack, 1979). Discrete element methods (DEM) are now widely used in the research of ballast behaviors and many valuable conclusions were drawn from microscopic perspectives, such as the research of ballast settlement (Jiang et al., 2017; Tutumluer, 2007),
the behavior of fouled ballast (Huang and Tutumluer, 2011; Indraratna et al., 2012), the ballast-geogrid interaction (Chen et al., 2012; Ngo et al., 2014) et al.

In the research of ballast-sleeper interaction, to take advantage of both the FEM and DEM, the sleeper can be modeled using finite element methods (FEM), while the ballast can be modeled using DEM. The coupling of FEM and DEM allows for both the sleeper and ballast to be simultaneously modeled. Stránský and Jirásek (2012) present a DEM-FEM coupling strategy using open source code softwares and applied this method in the research of ballast-sleeper interaction (Stránský, 2014). Tran et al. (2014) used similar method to investigate the soil-geogrid interaction in which the geogrid was modeled by FEM while the soil was modeled using DEM. Elmekati and El Shamy (2010) used this approach to model pile installation in granular soil in which the pile and the far-field soil were modeled by FEM while the near-field soil which surrounded the pile was modeled by DEM.

In this study, a coupled FEM-DEM framework that is capable of modeling ballast-sleeper interaction at the microscopic scale level is described. Both timber sleeper and steel sleeper were investigated in this study. The ballast-sleeper interaction was analyzed in both static loading and cyclic loading. The ballast contact, ballast movement, the change of porosity in ballast and the pressure distribution under sleeper were also investigated.

### 6.3 Coupled FEM-DEM framework

In this study, open source code OOFEM (Patzák, 2012) was used to conduct FEM modeling of sleeper while open source code YADE (Šmilauer et al., 2010) was used to conduct DEM modeling of ballast. The cores of YADE program is written in C++ using flexible object model, allowing independent implementation of new algorithms and interfaces. OOFEM is also written in C++ with object oriented architecture. Python is used for model construction, simulation control and postprocessing.

#### 6.3.1 Finite element method (FEM)

In this study, the explicit dynamic formulation of the finite element method is employed. Central difference time integration is used to calculate field variables at respective nodal
points. Since only a numerical solution is possible for a non-linear ordinary differential equation, this method is particularly suited for non-linear problems. Explicit methods calculate the state of a system at a later time from the state of the system at the current time, shown as in Equation 6.1.

\[ Y(t + \Delta t) = F(Y(t)) \]  \hspace{1cm} (6.1)

Where \( Y(t) \) is the current system state and \( Y(t + \Delta) \) is the state of the system at a later time.

In the FEM analysis, discretization is firstly conducted, as shown in Figure 6.1.

![Figure 6.1. FEM analysis](image)

The basic idea in FEM is the solve of the stiffness equations. Based on the element geometry and material parameters, the element stiffness matrix is constructed first. After the generation of the element stiffness matrix, global stiffness equation will be set up to get the relationship between deformation and force, as shown in Equation 6.2.

\[
[K]^e \{d\}^e = \{f\}^e \rightarrow [K]^g \{d\}^g = \{f\}^g
\]  \hspace{1cm} (6.2)

Where \([K]^e\) is the stiffness matrix of the element, \(\{d\}^e\) is the nodal displacement in the element, \(\{f\}^e\) is the force matrix of the element, \([K]^g\), \(\{d\}^g\) and \(\{f\}^g\) are the stiffness matrix, nodal displacement matrix and force of the system. \([K]^g\), \(\{d\}^g\) and \(\{f\}^g\) are assembled from the respective element matrix.
\[ K^e = \int_{\Gamma^e} B^e^T E A B^e \, dx \]  

(6.3)

Where \( B^e \) is the matrix of interpolation (shape) functions, \( EA \) represents the mass density.

### 6.3.2 Discrete element method (DEM)

In DEM simulation, contact detection is first conducted. When the distance between two particle centers is smaller than the sum of the two radius, contact happens. The contact direction is very important to the particle movement. The direction of both normal force \( (F_{c,n}) \) and tangential force \( (F_{c,s}) \) are determined by the contact direction. The contact direction for sphere particles is easy to get. The normal direction is along with the lines connected by two centers, while the shear direction is perpendicular to the normal direction (Figure 6.2). The origin of the normal stress and tangential stress locates at the center of the plan formed by the line connecting the two sphere particles (Figure 6.2). The normal force in the following equations:

\[ F_{c,n} = K_n \cdot \Delta l \]  

(6.4)

\[ K_n = \frac{K_1 \cdot K_2}{K_1 + K_2} \]  

(6.5)

where \( K_n \) is the normal stiffness, which is determined by the normal stiffnesses \( (K_1 \) and \( K_2) \) of the two contact spheres; \( \Delta l \) is the invasion distance.

In the calculation of the shear force \( F_{c,s} \), a standard incremental algorithm (Smilauer et al., 2010) is applied. It involves the correction of the shear force from the previous time step for changes in the normal direction and for rigid-body motion. Then, an additional shear displacement increment \( (\Delta u) \) caused by the mutual movements and rotations of ballast is calculated and the shear force is adjusted by the increment \( (\Delta F_{c,s}) \).
\[
\Delta F_{c,s} = K_s \cdot \Delta u \tag{6.6}
\]

where \(K_s (N/m)\) is the contact shear stiffness of the material.

In the calculation of shear force, the Coulomb condition (Equation 6.7) is employed when a slide occurs at the contact. When the shear force calculated by Equation 6.6 is larger than \(F_{c,n} \cdot \tan \varphi\), a slide occurs and the shear force is reduced to \(F_{c,n} \cdot \tan \varphi\).

\[
F_{c,s} \leq F_{c,n} \cdot \tan \varphi \tag{6.7}
\]

### 6.3.3 FEM-DEM coupling

In this FEM-DEM coupling framework, the ballast behavior is modeled using DEM, while the behavior of crosstie is analyzed using FEM. This coupling method, also called surface-coupling method, is probably the most common and straightforward method (Fakhimi, 2009; Nakashima and Oida, 2004; Onate and Rojek, 2004; Stránský, 2014). Using the surface-coupling framework, as long as there is no contact between ballast and crosstie, the analysis of DEM and FEM is conducted separately.
If collision is detected between ballast and crosstie, the contact force will act on the DEM particle and crosstie simultaneously. The position and velocity of the particle are computed using Newton’s second law, while the behavior of crosstie is analyzed using continuum mechanics. The contact force is also processed as a loading boundary condition (Figure 6.3).

The computing procedure in DEM-FEM coupling is shown as follow. Once the contact between ballast and crosstie occurs, the contact surface elements in FEM domain are mirrored in DEM domain. In this study, the contact surface elements are linear triangle elements. The surface elements are regarded as fixed geometrical boundary conditions. Then, DEM analysis is conducted. Positions of particles and the contact forces are generated. The contact forces between ballast and the surface elements are also derived and regarded as the external load in FEM domain. FEM analysis is conducted and values of nodal positions are passed back to DEM domain. Figure 6.4 shows the flow chart how DEM-FEM computing conducted.
6.4 FEM-DEM coupling framework setup

The dimensions of timber crosstie and steel crosstie are $2.5 \times 0.2 \times 0.18\text{m}$ and $2.6 \times 0.22 \times 0.12\text{m}$, respectively. The thickness of the hollow steel sleeper is $0.008 \text{m}$. One open-source meshing tool Gmsh (Geuzaine and Remacle, 2009) was used to generate meshes for both sleepers. Figure 6.5 and Figure 6.6 gave the mesh condition of timber crosstie and steel crosstie.

In this study, to simplify the analysis and save the computation time, all the ballasts are spheres. Number of ballast particles is 38295. The ballast gradation information is shown in Figure 6.7, which is classified as AREMA #4A. Before the external loading applied on the sleeper, ballast was first compacted to the porosity about 44%. The dimension of the ballast is $3.4 \times 0.7 \times 0.46\text{m}$ (Figure 6.8). Figure 6.9 shows the whole model of timber crosstie-ballast and steel crosstie-ballast.
Figure 6.5. Timber crosstie

Figure 6.6. Steel crosstie
Figure 6.7. Ballast gradation

Figure 6.8. Ballast

Figure 6.9. Initial condition of crosstie-ballast
Some parameters which are used in the DEM-FEM modeling are shown in Table 6.1.

### Table 6.1. Some parameters used in the simulation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ballast density</td>
<td>2600 kg/m³</td>
</tr>
<tr>
<td>Friction angle</td>
<td>35°</td>
</tr>
<tr>
<td>Time step increment</td>
<td>5 × 10⁻⁸ s</td>
</tr>
<tr>
<td>Gravity</td>
<td>9.8 m/s²</td>
</tr>
<tr>
<td>Density of timber crosstie</td>
<td>800 kg/m³</td>
</tr>
<tr>
<td>Density of steel crosstie</td>
<td>7800 kg/m³</td>
</tr>
<tr>
<td>Elastic modulus of timber crosstie</td>
<td>9 GPa</td>
</tr>
<tr>
<td>Elastic modulus of steel crosstie</td>
<td>200 GPa</td>
</tr>
</tbody>
</table>

The loading procedure consisted of three parts. In the first part, an external force linearly increased from 0 to 132000 N. The pressure values were measured and recorded at two load levels: 44000 N and 132000 N. In the second part, a cyclic loading was performed at a frequency of 100 Hz for 5 times from 4400 N to 44000 N. During this loading, the pressure development was recorded. After the cyclic loading, the linear increased load was performed again to evaluate the influence of the cyclic loading on the behaviors of ballast particles.

![Figure 6.10. Loading procedure](image-url)
6.5 Test results

6.5.1 Pressure distribution

Considering the computation of the pressure under the crosstie, the stress tensors \( (\sigma_{ij}) \) of per-particle can be directly generated in YADE. A weighted average of per-body stresses will give the average stress inside the solid phase (ballast). To simplify the analysis, the stress at one given location can be generated by the following equations:

\[
\sigma_{equ}^{ij} = (1 - \eta)\sigma_{solid}^{ij} \quad (6.8)
\]

\[
\sigma_{solid}^{ij} = \frac{\sum_{m=1}^{N} \sigma_{ij}^m}{N} \quad (6.9)
\]

where \( \sigma_{ij} \) is the stress tensor in \( ij \) direction, \( N \) is the particle number in given volume, \( \sigma_{solid}^{ij} \) is the average stress of the \( N \) particles, \( \eta \) is the porosity of the ballast in the given volume, \( \sigma_{equ}^{ij} \) is the stress finally calculated.

In this study, five points were selected to investigate the pressure distribution under the steel and timber crossties. As shown in Figure 6.11, the pressure points locate beneath the crosstie end, rail seat and the middle of the crosstie. Research (Laryea et al., 2014) shows that hollow-shape steel crosstie presents a high stress concentration under its edge. To reduce the effect of stress concentration and to ensure that a relatively normal pressure can be obtained, pressure cells were placed at a depth of 0.13 m beneath the crosstie rather than at the interface between ballast and crosstie.

![Figure 6.11. Schematic of pressure detection](image)

No. 1 to 5 represent five cuboid volume from left to right in which the stress will be calculated. The dimension of the cuboid is \( 0.22 \times 0.22 \times 0.1 \) m (length \( \times \) width \( \times \) height).
To calculate the pressure distribution using Equation 6.6, the porosity ($\eta$) should be obtained first. The pore scale volume method (Catalano et al., 2014) was used to compute the porosity in the cuboid volume.

$$p_i = \frac{v}{v'}$$  \hspace{1cm} (6.10)

$$\eta = 1 - \frac{\sum_{m=1}^{N} v}{\sum_{m=1}^{N} v'} = 1 - \frac{\sum_{m=1}^{N} v'}{V}$$  \hspace{1cm} (6.11)

where $p_i$ is the porosity of one single particle, $v$ is the volume of the single particle, $v'$ is the scale volume calculated by Voronoi tesselation method (as shown in Figure 6.12), $V$ is the total volume of the $N$ particles (the pore volume is included in $V$).

![Figure 6.12. Pore scale volume](image)

Figure 6.13 and Figure 6.14 give the stress distribution results under the timber and steel crossties. Laboratory tests were plotted in Figure 6.13 and Figure 6.14 to make comparison with the simulation results. Results show that there was some difference in the pressure obtained from the laboratory test and DEM simulation especially before the cyclic loading. The difference may be caused by two reasons: first, although the compaction was made before the DEM simulation, the compaction condition between the simulation and the laboratory...
test was different; second, the spherical particles were ideal model and can not represent the actual complicated shapes. However, after the cyclic loading, laboratory test and DEM-FEM simulation present similar pressure distribution for both steel and timber crosstie when the external load was 44000 N and 132000 N, respectively. Results show that after the cyclic loading, for timber crosstie, both the laboratory test and the DEM-FEM simulation show that the largest pressure value present under the rail seat at 132000 N. For steel crosstie, both the laboratory test and the DEM-FEM simulation show that the largest pressure value present under the center of the crosstie at both 44000 N and 132000 N. The DEM-FEM results and laboratory results agreed with the published results (Laryea et al., 2014; Song et al., 2017b).

(a) Pressure distribution before cyclic loading  
(b) Pressure distribution after cyclic loading

Figure 6.13. Pressure distribution under timber crosstie
Figure 6.14. Pressure distribution under steel crosstie

Figure 6.15 and Figure 6.16 compare the pressures at different locations of crosstie before and after the cyclic loading. It can be clearly observed that the pressure development trend is similar for laboratory test and DEM-FEM simulation. For both the steel and timber crossties, after the cyclic loading, the pressure at the center of the crosstie showed significant increase, whereas, cyclic loading decreased the pressure beneath the rail seat and crosstie end.

Figure 6.15. Pressure comparison of timber crosstie between initial condition and after the cyclic loading
6.5.2 Force chain

Figure 6.17 and Figure 6.18 show the force chain development of the timber and steel crosstie. The color and the thickness of the chains represented the amplitude of normal force. It can be observed clearly that at the initial stage, the external load was very small, the force chain in the ballast particles were not obvious. As the external loading increased, the force chains beneath the timber and steel crosstie became significantly obvious. For timber crosstie, the external load mainly transferred vertically. On the other hand, for steel sleeper, Figure 6.18(d) presents stress concentration under its edges, and meanwhile, load still transferred vertically.
Figure 6.19 to Figure 6.21 present the polar histograms of the normal contact orientation in XZ section for timber and steel crossties. At the initial condition, there was no significant difference in the normal contact distribution between the timber and steel crosstie. The contact angles between ballast were very uniform. At 132000 N, the contact orientation was different for the two types of crossties. For timber crosstie-ballast interaction, there was a larger number of contact which showed larger contact angles between the ballast particles. The edges of the steel crosstie separate the force transmission while there was no edge effect for the timber crosstie. After the cyclic loading, although there was ballast movement, there was still difference in the normal contact orientation.
Figure 6.18. Force chain of steel crosstie-ballast

(a) Initial condition

(b) Force chain at 132000 N (before cyclic loading)

(c) Force chain on xy section

(d) Ballast-crosstie interaction

Figure 6.19. Histograms of normal contact orientation at initial condition

(a) Timber crosstie

(b) Steel crosstie
Figure 6.20. Histograms of normal contact orientation at 132000 N (before cyclic loading)

Figure 6.21. Histograms of normal contact orientation at 132000 N (after cyclic loading)

6.5.3 Porosity

According to Equation 6.11, porosities at point 1 to point 5 could be obtained for both timber crosstie and steel crosstie. It can be observed that at the initial condition, for both timber and steel sleeper, porosities under the sleeper were relatively uniform. After the external load linearly increased to 132000 N, porosities beneath the rail seats were the lowest, while porosities beneath the sleeper ends were the largest. Comparing the porosities between the timber sleeper and steel sleeper, at the same location, porosity under the steel sleeper was lower than that of timber sleeper. Figure 6.18(d) and Figure 6.17(d) show the force chain in XY section. For timber sleeper, the inter-particle forces show that most of the external load transferred vertically, besides, inter-particle forces also spread horizontally. For steel sleeper,
the vertical edges provide the moving resistance of the ballasts in horizontal direction, so more vertical load was transferred. After five times cyclic loading, porosities under the sleeper decreased.

6.6 Conclusions

One DEM-FEM framework was developed in this study to investigate the ballast-sleeper interaction under static and cyclic loadings. Two open source codes OOFEM and YADE were utilized in this modeling to realize the DEM-FEM simulation. The stress distributions under the steel sleeper and timber sleeper were generated and comparison was made between the laboratory test and the modeling test for both types of crossties. The normal contact forces and normal contact distribution in ballast particles were also explored.

Results show that cyclic loading significantly changed the stress distribution under both the steel and timber crosstie. The pressure under the rail seat decreased after the cyclic loading, while the pressure at the center of the crosstie increased. The effect of the cyclic loading was more significant on the steel sleeper. The stress distribution pattern of steel sleeper is different from the timber crosstie. There was significant stress concentration under the edges of the steel crosstie.

6.7 Acknowledgements

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Chapter 7

Conclusions and recommendations
7.1 Conclusions

Two research topics were investigated in this study: bonding performance between pavement layers and the ballast-crosstie interaction. Laboratory tests were conducted to investigate the pavement layer bonding performance in traditional dense asphalt pavement and OGFC pavement. Discrete element modeling were performed to explore the interlock effect between pavement layers. The fatigue performance between OGFC and underlying layer was also studied. In the ballast-crosstie interaction, a full scale laboratory test was conducted to investigate the pressure distribution under a timber crosstie or a steel crosstie under static and cyclic loadings. One DEM-FEM framework was developed to simulate the ballast-crosstie interaction. Based on the results obtained from the aforementioned studies, the following conclusions can be made:

- In the study of the bonding performance tests, results show that the friction angle between BM aggregate and OGFC aggregate was larger than the angle between D aggregate and OGFC aggregate. At 0.2 MPa normal stress, the specimens composed of two dense mixtures gave higher shear properties than those with OGFC as the upper layer. This is due to the fact that the non-contact area between OGFC and underlying layer compromised the bonding between the two layers. Among the samples composed of two dense layers, the D-BM specimens showed a higher interlayer shear resistance resulting from a larger interlayer roughness caused by a better aggregate interlock between D and BM mixtures. This indicates an upper layer with a small nominal maximum aggregate size (NMAS) and an underlying layer with a large NMAS could provide a better bonding. For specimens composed of OGFC and a dense mixture (D or BM), at the optimal tack coat application rate, OGFC-BM showed a better shear performance than OGFC-D, due to the double effects of a larger interface contact area and a larger interface roughness than OGFC-D.

- Direct shear test was modeled using discrete element method to investigate the interlock effect between pavement layers. The friction angle between BM aggregate and OGFC aggregate was larger than the angle between D aggregate and OGFC.
aggregate. For samples composed by two dense asphalt layers, D-BM mixtures gave the larger interlock effect.

- Shear fatigue test was conducted to evaluate the fatigue performance between OGFC and different underlying layers. 50% stiffness reduction method, cumulative dissipated energy and ratio of dissipated energy change (RSEC) were employed to analyze the fatigue performance. OGFC-TLD showed better shear fatigue performance than OGFC-BM. The plateau value (PV) of OGFC-TLD was lower than that of OGFC-BM and the total dissipated energy of OGFC-TLD was larger than that of OGFC-BM. The interface contact area between OGFC and TLD was larger than that of OGFC and BM, leading to a better fatigue performance of OGFC-TLD.

- The laboratory pressure measurement system was reliable for measuring the pressure distribution under the steel and timber crossties. In the static loading test, at a low loading level (44000 N and 88000 N), the pressure at the center of the steel crosstie was the highest among all the pressures measured at different locations. As the applied load increased, the pressures at the halfway location between the rail seat and the center gradually became the highest ones. But for the timber crosstie, with the increase in the loading level, the maximum pressure was obtained directly beneath the rail seats. Cyclic loading could increase the pressure at the center of the crosstie, and significantly decrease the pressure at the halfway locations between the rail seat and the center. The pressures beneath the rail seats showed a slight decrease. The effect of the cyclic loading on the pressure distribution of the steel crosstie was more significant than for timber crosstie.

- One DEM-FEM framework was developed in this study to investigate the ballast-sleeper interaction under static and cyclic loadings. Two open source codes OOFEM and YADE were utilized in this modeling to realize the DEM-FEM simulation. The stress distributions under the steel sleeper and timber sleeper were generated and comparison was made between the laboratory test and the modeling test for both types crossties under static and cyclic loadings. The normal contact forces and normal contact distribution in ballast particles were also explored. Results show that cyclic
loading significantly changed the stress distribution under both the steel and timber crosstie. The pressure under the rail seat decreased after the cyclic loading, while the pressure at the center of the crosstie increased. The effect of the cyclic loading was more significant on the steel sleeper. The stress distribution pattern of steel sleeper is different from the timber sleeper.

7.2 Recommendations

On the basis of the conclusions obtained in this study, the following recommendations are provided here for further research work:

- In this study, the interlock effect was investigated between pavement layers using discrete element method. Asphalt binder bonding and the tack coat bonding were also critical factors contributing to pavement layer bonding. Asphalt binder and tack coat are viscoelastic materials. Contact models of viscoelastic materials, such as Burger’s model, are not included in YADE. Some code work needs to be done to include viscoelastic models in YADE to investigate the shear performance between pavement layers.

- In the study of the ballast-sleeper interaction, ballast particles with realistic shapes are recommended for future simulations. Clumping methods can be used in the generation complicated shape particles. In addition, complicated particles, such as polyhedrons, can also be directly generated in YADE. However, considering the large number of particles in the simulation, the computation will be time-consuming if complicated shapes are used in DEM simulation. Therefore, YADE source code needs to be modified and compiled to improve the computation efficiency. Super computers, such as Newton clusters, can help improve the computation speed in future studies.
Bibliography


Vita

Weimin Song was born in China in Puyang, Henan, China P.R. in 1987. He entered Central South University, Changsha, China in 2006 and received his Bachelors Degree in Civil Engineering in 2010. He studied as a master student in the Department of civil engineering materials in Central South University from 2010 to 2013, where his research interest concentrated on the fiber reinforced concrete. In July 2013, he joined the Department of Civil and Environmental Engineering in the University of Tennessee at Knoxville (UTK) to pursue the Ph. D. degree. During his stay in UTK, he extended his research area to pavement materials and discrete element method (DEM) analysis.