EFFECT OF SUBSURFACE CONDITIONS ON FLEXIBLE PAVEMENT BEHAVIOR: NON-DESTRUCTIVE TESTING AND MECHANISTIC ANALYSIS

by

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DEDICATION

Dedicated to all scholars pursuing knowledge advancement for the human race.

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ABSTRACT

The behavior of flexible pavements under traffic and environmental loading can be significantly affected by subsurface conditions. Inadequate support conditions under the surface can lead to excessive pavement deformations, often leading to structural and functional failure. This research effort focused on assessing the effects of base/subbase and subgrade layer conditions on flexible pavement behavior. The results of this study are presented in the form of two journal manuscripts.

The first manuscript focuses on utilizing pavement structural and functional evaluation data in making pavement rehabilitation decisions. Visual distress surveys and Falling Weight Deflectometer (FWD) testing are often carried out by agencies as a part of their pavement preservation programs. Although back-calculation of individual layer moduli from FWD data is a common approach to assess the pavement's structural condition, the accuracy of this approach is largely dependent on exact estimates of individual layer thicknesses. Considering the lack of pavement layer thickness information for all locations, this study used Deflection Basin Parameters (DBPs) calculated from FWD test data to make inferences regarding the structural condition of individual pavement layers in conventional flexible pavements. The adequacy of DBPs to assess the structural condition of individual pavement layers was assessed through Finite-Element (FE) Modeling. Subsequently, four selected pavement sections in the state of Idaho were analyzed based on this method to recommend suitable rehabilitation strategies.

The second manuscript focused on studying how improvements to subsurface layers can affect the flexible pavement behavior over expansive soil deposits. A recently completed research study at Boise State University investigated a particular section of US-95 near the Idaho-Oregon border that has experienced significant differential heave due to expansive soils. Laboratory characterization of soil samples indicated the presence of highly expansive soils up to depths of 7.6 m (26 ft.) from the pavement surface. Through subsequent numerical modeling efforts, a hybrid geosynthetic system comprising geocells and geogrids was recommended for implementation during pavement reconstruction. This research effort focused on evaluating the suitability of polyurethane grout injection as a potential remedial measure for this pavement section. Laboratory testing of unbound materials treated with a High-Density Polyurethane (HDP) demonstrated that resilient modulus and shear strength properties could be improved significantly. Finite Element modeling of the problematic US-95 pavement section indicated that depending on the treated layer thickness, the differential heave magnitude can be reduced significantly, presenting polyurethane injection as a potential nondestructive remedial measure.

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CHAPTER 1: INTRODUCTION AND BACKGROUND

Problem Statement

The United States has the world's largest transportation system, with a road network spanning more than 3.9 million miles. (www.fhwa.dot.gov/ohim/onh/onh.pdf). The pavements in this large network become deteriorated over time due to traffic and environmental loading. Generally, pavement sections deteriorate at an increasing rate. Initially, the rate of deterioration is comparatively slow when there are few distresses in the pavement. However, with time, distresses due to traffic loading and environmental exposure increase, accelerating subsequent damage to the pavement. Pavement maintenance and rehabilitation are two major strategies generally used to increase pavement service life (Johnson 2018). Typically, maintenance activities target improvement of the pavement surface at early stages of distresses. This slows down the rate of pavement deterioration by correcting small pavement defects before they worsen and contribute to further damage in the pavement layer. However, beyond reasonable pavement distress limits, maintenance activities are no longer an effective option to correct pavement distress. In such cases, pavement rehabilitation activities are required to repair the damaged pavement layers. In some cases, complete reconstruction is the only option. Thorough identification and documentation different distress types, along with structural and functional pavement evaluations are essential in prioritizing maintenance, rehabilitation and reconstruction activities.

According to guidelines provided by the American Association of State Highway and Transportation Officials (AASHTO), two major levels of pavement management decisions are included in a Pavement Management System (PMS): (1) network level and (2) project level. Network-level decisions are concerned with programmatic and policy issues for an entire network. These decisions include: establishing pavement preservation policies, identifying priorities, estimating funding needs, and allocating budgets for Maintenance, Rehabilitation, and Reconstruction (MR&R) (Alkire 2009; AASHTO-1993). Project-level decisions address engineering and technical aspects of pavement management, i.e., the selection of site-specific MR&R actions for individual projects and groups of projects. The entire success of a PMS depends on the availability of sufficient data to evaluate the pavement network, and establish an efficient project level pavement preservation strategy. Whether evaluating a huge pavement network or selecting a particular pavement treatment strategy, the most influential factors are traffic interruption and cost. For this reason, over the last few decades, nondestructive evaluation processes and treatment technologies are becoming increasingly popular due to significant time and cost reductions. One common nondestructive pavement structural evaluation technique involves Falling Weight Deflectometer (FWD) testing. In FWD testing, surface displacements induced due to the application of an impulse load are used to make inferences about the structural condition of the pavement. Once the need for rehabilitation has been established for a particular pavement sections, different alternatives can be considered before the most sustainable and resilient rehabilitation approach is selected.

The research effort documented in the current master's thesis focused on nondestructive pavement evaluation as well as the implementation of one particular nondestructive rehabilitation approach. First, the effects of subsurface conditions on pavement response under loading are studied by utilizing the FWD testing approach. Subsequently, polyurethane grout injection has been studied as a potential rehabilitation measure to reduce the problem of recurrent differential heaves on flexible pavements constructed over expansive soil deposits.

Background

Manuscript - 1

The Idaho Transportation Department (ITD) is currently in the process of rehabilitating several sections of highways across the state. Depending on their geographical location, these highway segments are often built over different subgrade conditions, and are exposed to different levels of truck traffic & environmental conditions. Rehabilitation design is therefore carried out at the district level after collection of relevant project information. Due to time and resource constraints, extensive evaluation of pavement structural condition across the network is often not feasible. Accordingly, functional evaluation results with limited structural assessment data are often used to make pavement maintenance and rehabilitation decisions. However, the success of pavement maintenance and rehabilitation decisions are largely dependent not only on the functional quality of the pavement, but also on its structural condition. Visual distress surveys and nondestructive pavement structural evaluation technique such as FWD testing are often carried out by agencies as part of their pavement preservation programs. Although back-calculation of individual layer moduli from FWD data is a common approach to assess a pavement's structural condition, the accuracy of this approach is largely dependent on exact estimates of individual layer thicknesses. Coring operations to determine pavement layer thicknesses require significant time and resource commitments, and hence cannot always be accommodated within an agency's operational constraints. Ground Penetrating Radar (GPR) is one way to assess pavement layer thickness. However, like coring data, GPR testing data is not usually available in the PMS. Therefore, an alternative analysis method to assess the pavement's structural condition from FWD data is desired for those cases where layer thickness data is not available. In manuscript-01, the research is primarily focused on the combined use of visual distress survey data and Deflection Basin Parameters (DBPs) calculated from FWD test data to make inferences regarding the structural condition of individual pavement layers in a network level database. The manuscript (Chapter 2 of this thesis) evaluates the accuracy of different DBPs through a detailed numerical modeling effort. Subsequently, the DBP approach is used to evaluate the structural condition of four different highway segments selected within the state of Idaho. The usability of the DBP approach as a network-level tool for pavement rehabilitation decisions is explored.

<u>Manuscript - 2</u>

Flexible pavement sections constructed over expansive soil deposits often undergo significant damage due to the volume changes in the underlying soil strata induced by moisture fluctuations. Repetitive changes in volume of the underlying soil mass leads to corresponding changes in support conditions underneath the pavement; this change in volume often manifests itself through pavement surface distresses such as cracking and surface undulations. Generally, in cases where the expansive soil deposits are confined to shallow depths underneath the pavement surface, conventional rehabilitation treatments such as pre-wetting, chemical stabilization, removal and replacement, etc., can be pursued.

However, such treatment strategies become impractical for cases where the expansive soil deposit lies more than 1 m (3 ft.) underneath the pavement surface. In such cases, implementation of alterative remedial measures that can reinforce the pavement section, and dissipate the soil-generated swelling stresses is desired. Several research initiatives have been undertaken regarding this issue and it was found that uniform dissipation of excessive swelling energy/stress within the pavement layers is very effective for heave mitigation.

Recurrent damage caused by the expansive soil strata underneath a particular stretch of US-95 north of the Oregon-Idaho border has led ITD to explore different stabilization alternatives to minimize the costs associated with recurrent maintenance and rehabilitation activities. A recently completed research study at Boise State University conducted extensive laboratory characterization of soil samples obtained from the corresponding pavement section, and it was observed that the expansive soil deposits were often deeper than 2 m (6 ft.) from the pavement surface thus rendering chemical stabilization-based approaches impractical. In the second manuscript, the effectiveness of a High-Density Polymer (HDP) grout injection as a remedial measure to address the problem of recurrent pavement damage due to expansive soils is explored; such an approach can be particularly useful as it will not require removal of the existing pavement layers. HDP expanding polymer grout injection has the potential to result in the formation of a “flexible layer” within the pavement system where the polymer-soil or polymer-aggregate mixture can serve to uniformly dissipate the swell pressures from the underlying soil layers. Laboratory testing and numerical modeling was utilized to assess the suitability of HDP injection as a potential remedial measure to reduce recurrent heaving in pavement sections constructed over expansive soil deposits; findings from this study have been reported in Chapter 3 of this thesis.

Research Objectives, Tasks, and Manuscripts Prepared

The overall objective of this master's thesis research was to quantify how changes in subsurface conditions can affect the response and performance of flexible pavement sections. The research work has been reported in the form of two different manuscripts. The first manuscript focused on the use of nondestructive testing using FWD to draw inferences regarding the substructure layer conditions. To do so, the research task was divided into two parts. First, the applicability of Deflection Basin Parameters (DBPs) and their thresholds were evaluated using a commercial finite element modeling software ABAQUS®. Once accuracy of DBPs were established for typical pavement configurations, the next task involved using the DBPs to evaluate four pavement sections across Idaho. The four highway segments represented different functional classifications, were built over varying subgrade conditions, and are subjected to varying levels of truck traffic. Detailed outcomes of this evaluation approach have reported in Chapter 2 of this master's thesis emphasizing primary advantages, and highlighting inherent assumptions and shortcomings.

The primary objective of the second manuscript was to evaluate the effectiveness of HDP grout injection into the base or subgrade layer in a flexible pavement system as an alternative remedial measure to mitigate the problem of differential heave. The research tasks carried out to fulfill this objective can be broadly categorized into two groups. First, laboratory tests were carried out to establish the resilient modulus and shear strength properties of different unbound materials (aggregates and soils) used in the study.

Subsequently, Finite Element modeling was carried out to assess how the surface heaves can be reduced by injecting the HDP into the base/subbase or subgrade layers. Findings from these tasks have been detailed in Chapter 3 of this thesis. Table 1-1, lists the individual tasks carried out under the scope of this master's thesis, and maps each of the tasks to the technical manuscripts prepared.

Table 1-1: Individual Research Tasks mapped with Respective Manuscripts

Tasks	Name	Manuscript
1	The accuracy and applicability check of DBPs using Finite Element modeling	Manuscript #1
2	Field Application of DBPs with Visual Distress Data for PMS	
1	Laboratory Characterization of HDP grout injection in Base and Subgrade soil	Manuscript #2
2	Numerically evaluate the effectiveness of HDP grout injection to reduce differential heaving of pavements due to underlying expansive soil layers.	

Organization of the Thesis

This Master's thesis document comprises four chapters. Chapter 2 contains results reported in the first manuscript. The title of the manuscript is, "Using FWD Deflection Basin Parameters for Network-Level Pavement Condition Assessments". Chapter 3 contains findings reported in manuscript # 2, titled "Use of Polymer Grouting to Reduce Differential Heave in Pavements over Expansive Soils". Chapter 4 summarizes results and findings from the two manuscripts, and presents recommendations for future research tasks.

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MANUSCRIPT ONE – USING FWD DEFLECTION BASIN PARAMETERS FOR NETWORK-LEVEL PAVEMENT CONDITION ASSESSMENTS[1]

Abstract

Decisions regarding the selection and implementation of appropriate pavement rehabilitation methods is usually based on pavement functional and structural condition data. Visual distress surveys and Falling Weight Deflectometer (FWD) testing are often carried out by agencies as parts of their pavement preservation programs. Although backcalculation of individual layer moduli from FWD data is a common approach to assess a pavement's structural condition, the accuracy of this approach is largely dependent on exact estimates of individual layer thicknesses. Coring operations to determine pavement layer thicknesses require significant time and resource commitments, and hence cannot always be accommodated within an agencies' operational constraints. Accordingly, alternative analysis methods to assess the pavement's structural condition from FWD data are often desired. An ongoing research study at Boise State University is focusing on combined usage of pavement structural and functional evaluation data for making pavement rehabilitation decisions. Considering the lack of pavement layer thickness information for all locations, this study is using Deflection Basin Parameters (DBPs) calculated from FWD test data to make inferences regarding the structural condition of

[1] This chapter includes results already reported in the following publication. Contribution of the coauthor is sincerely acknowledged: Rabbi, M. F., and Mishra, D. (2018). "Using FWD Deflection Basin Parameters for Network-Level Pavement Condition Assessments". Submitted to the International Journal of Pavement Engineering (Under Review)

individual pavement layers. This manuscript presents findings from this study, and establishes DBPs as reasonable alternatives to be used in network-level pavement condition evaluation practices. The adequacy of DBPs to assess the structural condition of individual pavement layers was first assessed through Finite-Element Modeling. A series of analyses were performed by assigning typical modulus values to individual pavement layers, and the corresponding DBPs were calculated. The calculated DBP values mostly fell within typical ranges specified in the literature for different layer conditions. Once the DBPs were established as adequate alternatives for making network-level pavement assessment decisions, four selected pavement sections in the state of Idaho were analyzed based on this method, and the results were compared against those obtained from visual distress assessment routines.

Introduction

The success of an effective pavement maintenance and preservation program relies heavily on adequate functional and structural assessment of the pavement network. State and local transportation agencies often adhere to manual pavement condition ratings, windshield surveys, and/or the use of automated distress survey vehicles to maintain a database of pavement functional conditions. Structural assessment of pavements on the other hand, is commonly accomplished through some form of deflection testing, often using Falling Weight Deflectometers (FWDs), or more recently using Rolling Weight Deflectometers (RWD) or Traffic Speed Deflectometers (TSD). A well-performing pavement network is characterized by satisfactory functional as well as structural condition. Pavement distress surveys usually rate the ride quality and condition of the pavement surface, whereas structural condition assessment using FWD can evaluate the condition of individual pavement layers through backcalculation of layer moduli.

The accuracy of any backcalculation approach is largely dependent on the exact estimates of individual layer thicknesses. Highway agencies often carry out coring operations, or Ground Penetrating Radar (GPR) scans to establish individual pavement layer thicknesses. Coring operations are significantly time consuming, and resource intensive. Similarly, not all agencies have yet adopted GPR into regular practice to establish pavement layer thicknesses at the network level. Accordingly, alternative (and relatively quick) analysis methods to assess the pavement's structural condition from FWD data are desired. One such method involves the use of Deflection Basin Parameters (DBPs), which are indicators of the pavement deflection basin shape. Several researchers in the past (Horak 1987; Kim et al. 2000; Gopalakrishnan and Thompson 2005; Horak 2008; Donovan 2009; Talvik and Aavik 2009; Carvalho et al. 2012; and Horak et al. 2015) have highlighted the effectiveness of deflection basin parameters in evaluating the structural condition of in-service pavements. One of the most significant studies involving in-depth analysis of the pavement deflection data was carried out under the scope of the National Cooperative Highway Research Program (NCHRP) (Kim et al. 2000). This study involved the analysis of field as well as synthetic pavement deflection data to evaluate the significance of different DBPs, and attempted to develop empirical equations to predict individual pavement layer moduli from the DBPs without going through the rigors of backcalculation.

With the design and development of modern FWD equipment and increased emphasis on pavement management systems, agencies are moving towards extensive FWD testing across entire roadway networks. Although recent trend has been to use RWDs or

TSDs for network-level pavement assessment, these equipment are still not widely available (a total of two-three devices are available throughout the United States), and therefore, their use by highway agencies is not very common. Most highway agencies still rely on FWD testing at a network level to develop a database of pavement condition data under their respective pavement management programs. This data, combined with automated distress survey results can be used to identify structural deficiencies in individual pavement layers, ultimately leading to the selection and implementation of appropriate maintenance and rehabilitation methods. However, the usefulness of FWD test data without detailed information on individual pavement layer thicknesses still remains uncertain as far as the state of practice among transportation agencies is concerned.

Objectives and Scope

The primary objective of this research effort was to assess the suitability of Deflection Basin Parameters (DBPs) established through FWD testing as indicators of pavement structural condition at a network level. First, an extensive review of published literature was carried out to identify typical DBPs and corresponding threshold values proposed by researchers as indicators of pavement structural condition. This was followed by finite-element analysis of typical flexible pavement section configurations to calculate representative DBP values under simulated FWD loading. An extensive parametric analysis was conducted to establish ranges for DBP values for different layer modulus values assigned to individual pavement layers. DBP values established for these simulated pavement sections were compared against typical threshold values proposed in the literature. Once the suitability of DBPs as pavement structural condition indicators was established, four different pavement sections were selected across the state of Idaho, and their structural conditions were established using FWD data. Inferences related to the structural condition of these pavement sections were combined with functional evaluation records to propose suitable rehabilitation measures for implementation by the Idaho Transportation Department (ITD). This integration of pavement structural and functional condition assessments has been proposed as a suitable approach for pavement maintenance and rehabilitation selection.

FWD Testing as a Part of Routine Pavement Condition Evaluation

The pavement management programs implemented by most state transportation agencies typically involve Falling Weight Deflectometer (FWD) testing. Usually, FWD testing across the entire pavement network managed by a transportation agency is scheduled at periodic intervals. Moreover, pavement sections that are already identified for rehabilitation/reconstruction are also tested on “as-needed” basis, and the corresponding data is used in the design of the rehabilitated sections. Although FWD testing of pavement sections is usually carried out as part of the routine pavement evaluation program, the data is not used unless a particular pavement section has been identified for rehabilitation/ reconstruction. Based on current practice, pavement sections are typically selected for rehabilitation/reconstruction based on their functional condition assessment (such as visual distress survey, roughness measurements, etc.) results only. This approach is based on the assumption that deterioration in the structural health of a pavement section ultimately leads to deterioration in the functional condition, and therefore selecting pavements for rehabilitation/reconstruction based on functional condition data is acceptable. However, the functional condition of a pavement section does not automatically identify the layer(s) contributing towards the condition deterioration. Accordingly, detailed understanding of the structural health of individual pavement layers can facilitate the selection of optimal maintenance/rehabilitation approaches. Implementing “relatively quick” methods to get a good understanding of pavement structural health from FWD data will encourage transportation agencies to implement this practice to a greater extent. One such “relatively quick” approach to make inferences regarding the pavement structural condition from FWD data involves the use of DBPs

Commonly Used DBPs

Researchers in the past have defined different DBPs to make inferences about the structural conditions of individual pavement layers. These definitions, although similar in most cases, occasionally differ from each other. Moreover, threshold values for different DBPs demarcating the boundaries between different structural condition ratings differ from one agency to another. The current research effort made use of two distinct sets of DBP definitions used by practitioners and researchers in the field of pavement engineering. The first set was developed and is used in South Africa (Horak 1987; Horak 2008, Horak et al, 2015), whereas the second set was developed for use in the United States (Kim et al. 2000). Mathematical expressions used to calculate these DBPs have been given below. Note that D_r in the following expressions represents the surface deflection in µm (or mils) measured by a sensor placed at a distance of ‘r’ mm (or in.) from the center of the load plate.

DBPs Used in the United States

Surface Curvature Index (SCI): $SCI = D_0 - D_{12}$

Base Curvature Index (BCI): $BCI = D_{24} - D_{36}$

Note:

Sensor positions are marked in inches (1 in. = 25.4 mm)

Deflections are measured in mils (1 mil = 0.001 in.)

DBPs Used in South Africa

Base Layer Index (BLI): $BLI = D_0 - D_{300}$
Middle Layer Index (MLI): $MLI = D_{300} - D_{600}$
Lower Layer Index (LLI): $LLI = D_{600} - D_{900}$
Note:
Sensor positions are marked in mm
Deflections are measured in μm (1 μm = 0.001 mm)

Table 2-1, lists different DBPs and corresponding threshold values as found in the literature. Table 2-1-a lists the DBPs and threshold levels commonly used in the US, whereas Table 2-1-b lists DBPs and corresponding threshold values used in South Africa.

Table 0-1: Deflection Basin Parameters and Corresponding Threshold Values Obtained from Literature: (a) (Chang et al., 2014); (b) (Horak et al, 2015)

(a) (Chang et al., 2014);			
	Inference Related To	**Threshold Ranges (mils)**	**Inference**
Surface Curvature Index (SCI)	Asphalt Layer	< 4	Very Good Asphalt Layer
		4 - 6	Good Asphalt Layer
		6 – 8	Fair Asphalt Layer
		8 – 10	Poor Asphalt Layer
		> 10	Very Poor Asphalt Layer
Base Curvature Index (BCI)	Base Layer	< 2	Very Good Base Layer
		2-3	Good Base Layer
		3-4	Fair Base Layer
		4-5	Poor Base Layer
		> 5	Very Poor Base Layer
Deflection of the Sensor at 60-in. offset (W_{60})	Subgrade Layer	< 1	Very Good Subgrade Layer
		1 – 1.4	Good Subgrade Layer
		1.4 – 1.8	Fair Subgrade Layer
		1.8 – 2.2	Poor Subgrade Layer
		> 2.2	Very Poor Subgrade Layer

(b) (Horak et al, 2015)				
Performance Indicator	**Inference Related to**	**Categorization Based on Structural Condition**		
		Sound	**Warning**	**Severe**
D_0 **(μm)**	Entire Pavement Structure	< 625	625 – 925	> 925
Base Layer Index (BLI, **μm)**	Base Layer	< 250	250 – 475	> 475
Middle Layer Index (MLI,**μm)**	Subbase/Subgrade Layer	< 115	115 – 225	> 225
Lower Layer Index (LLI, **μm)**	Subbase/Subgrade Layer	< 65	65 – 120	> 120

From the definition of the DBPs, it can be clearly seen that same numeric value of DBP is sometimes denoted by different names in the two conventions. For example, the Surface Curvature Index (SCI) has the same numeric value as the Base Layer Index (BLI). However, the threshold value for SCI (see Table 2-1-a) are specified to make inferences regarding the asphalt layer, whereas threshold values for BLI (see Table 2-1-b) are used to makes inferences about the structural condition of the base layer. Considering these differences, the current study adopted an approach where the inferences drawn from the DBPs have incorporated both the US and South African practices.

Finite Element Modeling of FWD Testing on Flexible Pavements

Once commonly used DBPs and the corresponding threshold values were identified, the next task involved mechanistic evaluation of the suitability of these parameters as indicators of the structural quality for individual pavement layers. This involved calculating the DBPs for pavement sections with typical layer configurations and properties, and comparing them against threshold values specified in the literature. Establishing the approximate relationships between DBP values and corresponding moduli of various pavement layers could provide a means to evaluating the suitability of the literature-proposed threshold values for implementation by state and local transportation agencies. This was accomplished through Finite Element Modeling (using a general purpose finite element analysis software package, ABAQUS®) of representative pavement sections comprising layers with different modulus values. Threshold values suggested by Horak et al. (2015) were used to categorize the pavement structural response based on the predicted surface deflection values under simulated FWD loading.

A typical 3-layer pavement section comprising a 115-mm thick Hot-Mix Asphalt (HMA) layer overlying a 152.4-mm thick granular base layer constructed over a subgrade layer of infinite depth was modelled during this research effort. All layers were modelled as linear-elastic; the viscoelastic nature of HMA and stress-dependent behavior of unbound (base and subgrade) layers were ignored for this analysis. Although these simplifying assumptions can be treated as limitations of the modeling approach, they should not significantly limit the applicability of the findings from this research study, as has been established in the literature. Researchers in the past (Xie et al. 2015; Tarefder & Ahmed 2013) have successfully used linear-elastic models to simulate FWD testing of flexible pavement systems. Tarefder and Ahmed (2013) argue that under the short-duration impact loading (pressure ~700 kPa) as during typical FWD testing, the HMA layer can be safely assumed to exhibit linear-elastic behavior. Furthermore, application of 700kPa stress on the surface of the pavement typically does not generate failure/yield stress in the HMA or base layers; stress states sufficiently below the failure stress levels means the assumption of linear-elastic behavior is reasonable.

The authors do recognize that temperature can have a significant effect on the viscoelastic behavior of HMA. However, as FWD testing is typically carried out when pavement temperatures are between 70° and 90°F, the effect of temperature variation on central deflection is often insignificant. As per the 1993 AASHTO design guide (AASHTO 1993) the variation in central deflection during FWD testing introduced by temperature changes can be approximately 20%. This variation was considered to be insignificant during this research effort, and therefore, a linear-elastic, constant modulus modeling approach was pursued. The authors are well-aware of this being a limitation of the current analysis approach; future research efforts will focus on considering the non-linear behavior of individual pavement layers. Results from such analyses will be presented in future publications.

Model Generation and Optimization

Modeling a pavement section under FWD loading can be accomplished using several different approaches, such as: (1) 2-Dimensional, (2) 3-Dimensional, (3) Quarter-Cube, and (4) Axi-symmetric models. Although different simplifications can often be used with reasonable accuracy depending on the model and loading configurations, three-dimensional models have been shown to be the best alternative as far as capturing all three directional response components is concerned (Kim 2007). Moreover, significant increase in computational power over the past decade has eliminated the major limitations associated with 3-D finite element modeling. Accordingly, the current study utilized a 3-D FE model to simulate FWD testing of flexible pavement systems.

Geometry

Reviewing the literature and common practices for pavement construction by various agencies, a three layer (HMA, base and subgrade) pavement configuration was selected as the primary model. As already mentioned, the layer thickness selected for the initial model were 114 mm (4.5 in.), 152 mm (6.0 in.) and 12192 mm (480 in.) for HMA, base and subgrade, respectively. Here the thickness of HMA and base are the minimum typical thicknesses used for interstate and state highway road construction. The thickness of the subgrade layer was selected so that presence of the rigid boundary at the bottom does not affect the simulation results.

Mesh

The accuracy of the simulation results is highly dependent on mesh refinement, construction and the aspect ratio of elements. Smooth transitioning of stress and strain between elements is very important for convergence of the model (Kim 2007). Analysis time is also a very important consideration. In general, decreasing the precision of a model will decrease the analysis time. The computational time associated with a fine mesh is generally higher than that for a coarser mesh. In this model, the generation of mesh directly underneath the FWD loading area was done using a wedge-shaped mesh element; the element type used was C3D6, a 6-node linear triangular prism-type element. The surrounding influence areas were meshed using hex-shaped elements: C3D8R, an 8-node linear brick element. As C3D8R elements are susceptible to hour-glassing, active hourglass controls were used to minimize this effect (ABAQUS 2015). Reduced integration elements were used to increase the overall computational efficiency. Except for the central FWD loading zone (where higher deflections are expected), all other areas of the model were meshed using a structural mesh technique to significantly increase the model efficiency. To reduce the overall model convergence time, only the central zone of interest (DBP calculation zone) used a finer mesh (see Figure 2-1).

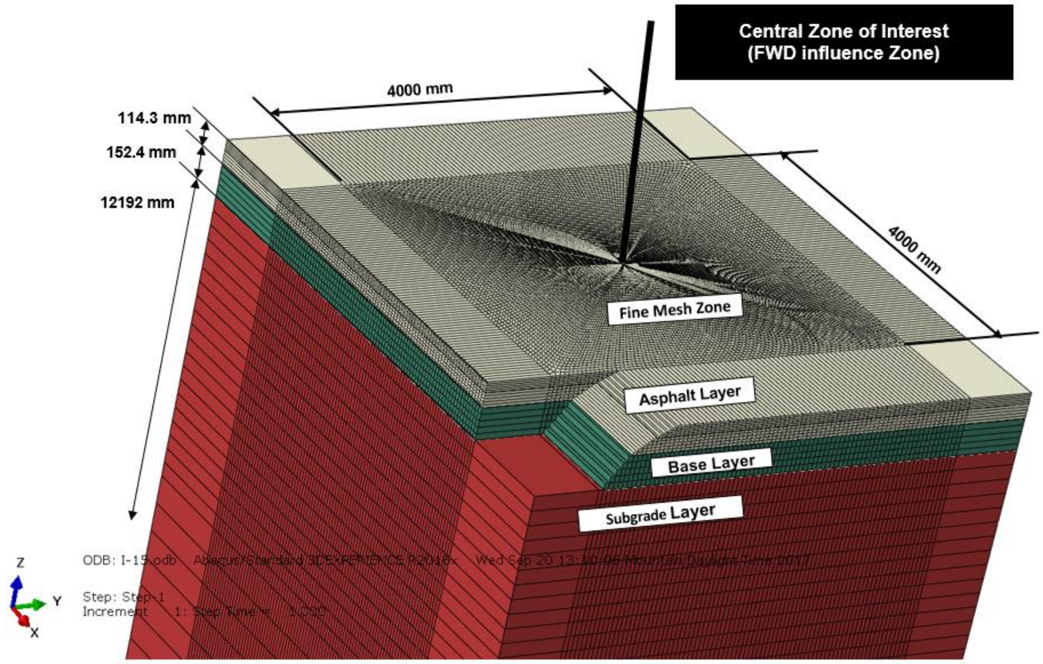


Figure 0-1: Snapshot of the ABAQUS model of the Pavement Section Analyzed, showing Relevant Dimensions

As boundary conditions have a significant effect on the stress-strain behavior exhibited by the simulated pavement section, model size was another important consideration during this verification process. Initially, a model size of 4000 mm X 4000 mm (in the horizontal direction) was selected. Later, the model dimensions were gradually increased until no change in the simulation results were observed due to change in model size. Note that increasing the model size also resulted in an increase in the computational time requirements. Several researchers in the past have studied the effects of model size and boundary conditions on simulation results. Kim (2007) mentioned that axisymmetric modeling and inappropriate treatment of boundary conditions can significantly affect the model accuracy. He also performed an axisymmetric finite element analysis to study the truncation effects of boundary conditions, and proposed that the effect of boundary conditions is negligible if the domain size is larger than 20 times radius of the loading area in the horizontal direction, and larger than 140 times the radius in the vertical direction (Kim 2007). Previously, Duncan et al. (1968) had observed that to eliminate boundary effects, the model geometry had to be extended to a depth of 50 times the radius of the loading area in vertical direction, and 12 times in horizontal direction. Uddin et al. (1994) also performed a study to determine the optimum domain size for a three-layer pavement configuration. The layer thicknesses used were similar to the primary model used in this study. They concluded that the optimum domain size required was: 18.3 m (length) x 26.6 m (width) x 12.2 m (depth). The dimensions suggested by Uddin et al. (1994) were used in the current study during preparation of the base model. However, it was observed that for larger deflections (very low modulus values assigned to individual layers), these dimensions needed to be changed to eliminate boundary effects. After fixing the model domain size, the model mesh size was optimized for both low and high modulus case scenarios. Once the mesh size was stabilized, mesh optimization was performed by making the mesh coarser outside the central area of interest. Later, the accuracies of the model-predicted deflection values were checked by comparing with the commonly used axi-symmetric pavement analysis software, KENLAYER (Huang 2004). The comparisons were carried out with extreme (within reasonable limits) modulus values assigned to the individual pavement layers.

Material Elastic Modulus Range Selection

Initially, a range of possible elastic properties of HMA, base and subgrade were selected upon discussions with agency pavement engineers. The material properties upper and lower limits are shown in Figure 2-2 (table inset). Here, the upper limit of modulus is taken to be representative of a “well-performing” pavement layer, whereas the lower limit indicates a “poor” pavement layer. Six different modulus values were assigned to each of the three pavement layers, resulting in a total of 6 x 6 x 6 = 216 pavement sections that were simulated under FWD loading conditions.

Comparing the FE model-generated results against those from KENLAYER, it was observed that the model performed significantly well when the modulus values assigned to the individual pavement layers were in the intermediate-to-high range; the results from the FE model differed slightly (still less than 10% difference in the predicted deflection values) from those predicted by KENLAYER when significantly low modulus values were assigned to the pavement layers. In Figure 2-2, the red dotted lines show the deflected shape plotted using KENLAYER. The group of solid lines (consisting of 216 combinations of various layer modulus values) in between the KENLAYER lines are the deflection basins obtained from the ABAQUS model for the different combination of modulus values. Later, this model was used for the DBP verification effort. Although layer thicknesses are also important governing factors that influence the deflection basin, only one set of thicknesses were considered in this study to verify the suitability of DBPs as structural condition indicators for pavement layers. Table 2-2, lists the range of modulus values assigned to different layers in this modeling effort.

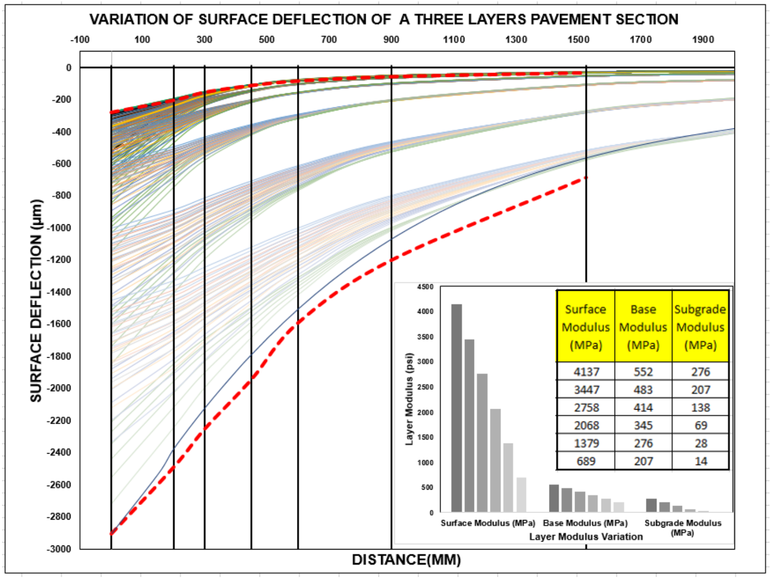


Figure 0-2: Variation of Pavement Surface Deflection with Variation of Pavement Layer Modulus

Table 0-2: Pavement Layer Properties used during the Simulation Efforts

Pavement Layer	Thickness (mm)	Poisson's Ratio	Elastic Modulus (MPa)		
			Max.	Min.	Control Case
HMA	114.3	0.30	4137	689	2758
Base	152.4	0.35	414	34	276
Subgrade	12192	0.35	138	1	69

Verification of DBP Range Threshold Values

Upon completion of the model verification efforts, the next task involved using the model to check the applicability of DBPs as indicators of the structural conditions for individual pavement layers. DBP values calculated for the different pavement configurations were compared against threshold values specified in the literature, and inferences were drawn regarding the validity of the results. The modulus values for the surface, base, and subgrade layers were individually varied to isolate the effects of each layer on the calculated DBP values. Results from this parametric analysis effort have been presented in Figure 2-3. DBP values were calculated for each modulus value (represented by a line on the plot). It is observed that for very low modulus values, the deflections are considerably high.

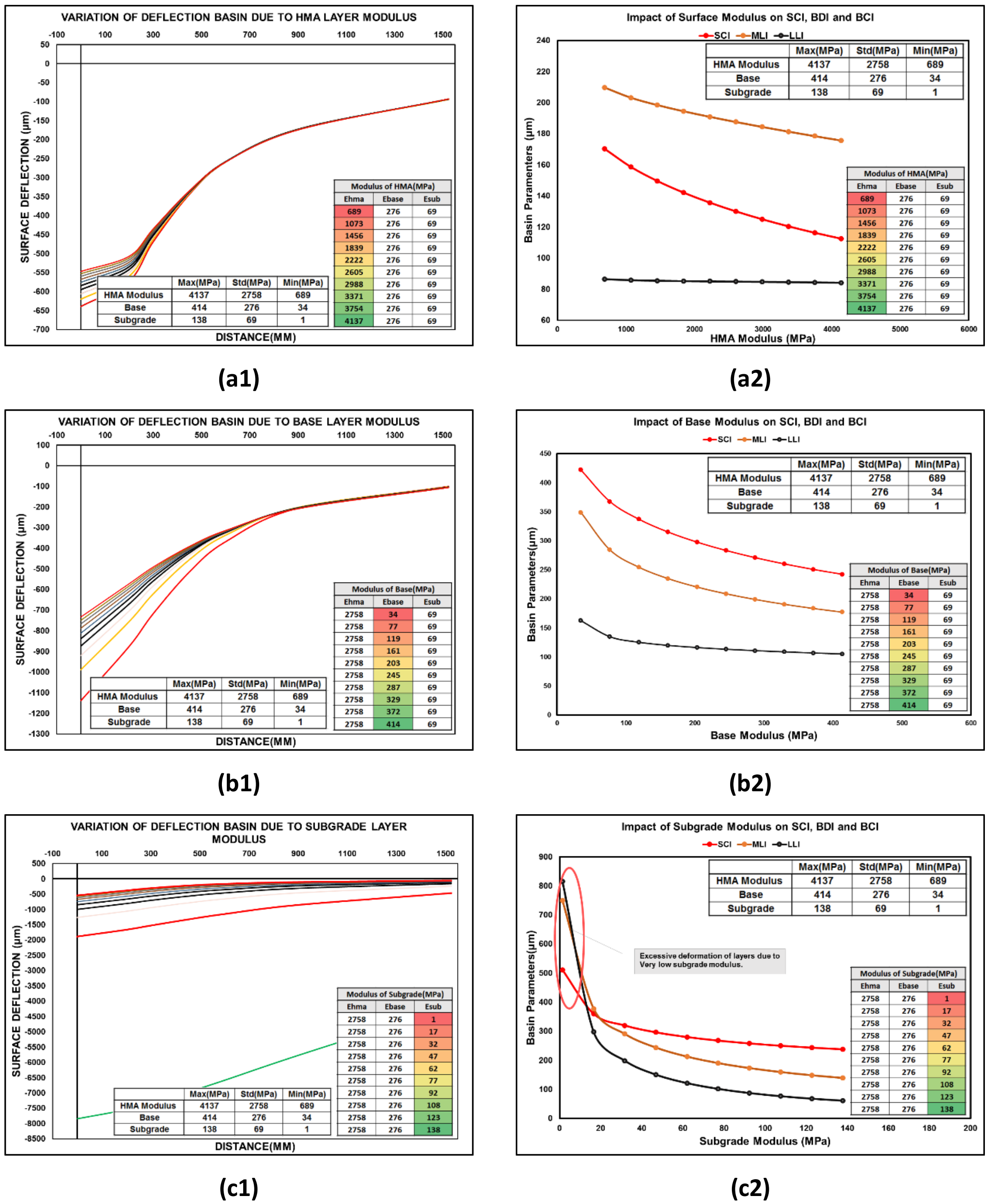


Figure 0-3: Variation of Surface Deflection Basin Shape and Basin Parameters with Varying (a) HMA, (b) Base and (c) Subgrade Modulus

From Figure 2-3-c1, it is clearly noticeable that the variation of subgrade layer modulus has a significant impact on the deflection of the farthest sensor. On the other hand, the variation of surface modulus has considerable influence on the shape of the deflection basin in the region closest to the point of load application (See Figure 2-3-a1). Variations in the base layer modulus affects the shape of the deflection basin both near the point of load application, and up to a certain distance from the load. It is therefore evident that commonly used deflection basin parameters accurately capture modulus variations in different pavement layers, which in turn can be related to layer quality. Figure 2-3-a2 shows that variation in the surface layer modulus has very little influence on the LLI parameter. Similar results were found for the base layer case (See Figure 2-3-b2). Only the subgrade layer variation causes significant changes in the LLI value. For 712% increase (increase from 17 MPa to 138 MPa) in subgrade modulus, a 79% reduction in the LLI value was observed (reduction from 297μm to 61 μm). Neglecting other influential factors, the LLI value can be used as a reasonably accurate indicator of subgrade quality. On the other hand, a 500% increase in surface modulus (increase from 689 MPa to 4137 MPa) causes a 34% reduction in the SCI value (reduction from 172 μm to 112 μm). This has a corresponding influence on the MLI value (approximately 16% reduction) (see Table 2-3 & Table 2-4). In the case of base layer modulus variation, it was observed that 1100% increase in base modulus (from 34 MPa to 414 MPa) resulted in a 49% reduction in the MLI value. All the variation of modulus values and the corresponding variation in DBP are listed in Table 2-3. Table 2-4, presents the variations in the DBP values in terms of percentages.

Table 0-3: Range of Modulus Values Assigned to Different Pavement Layers, and the Corresponding Variations in Deflection Basin Parameters

Layer	Elastic Modulus(MPa)			PI (%)	SCI/BLI(μm)		MLI/BDI(μm)		LLI/BCI(μm)	
	Mini.	Max.	Control Case		Mini.	Max.	Mini.	Max.	Mini.	Max.
HMA	689	4137	2758	500	112	170	176	210	85	86
Base	34	414	276	1118	242	422	177	349	105	162
Sub.G	17	138	69	712	238	360	139	377	61	297

**PI=Percentage Increment; Sub.G= Subgrade

Table 0-4: Variation of DBPs (Expressed as Percentages) with Variations in Individual Pavement Layer Modulus

Layer	SCI (μm)		BDI (μm)		BCI (μm)		SCI(μm)	MLI(μm)	LLI(μm)
	Min.	Max.	Min.	Max.	Min.	Max.	PD (%)	PD (%)	PD (%)
HMA	112	170	176	210	85	86	-34	-16	-1
Base	242	422	177	349	105	162	-43	-49	-35
Sub.G	238	360	139	377	61	297	-34	-63	-79

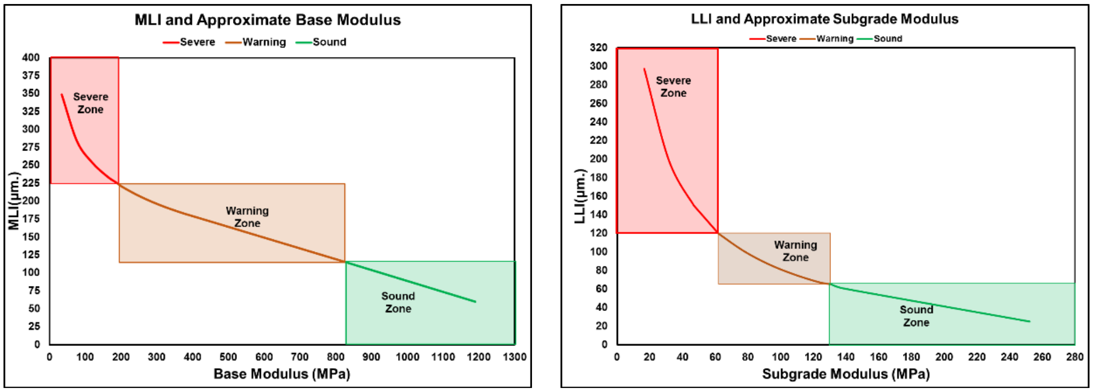
**PD (%)=Percentage Decrease;

From the above tables, it can be observed that the SCI values calculated for the lowest and highest modulus values do not match with the typical SCI value ranges used in South Africa. This is primarily because the SCI value is significantly influenced by the Base and Subgrade conditions (besides being governed by the HMA layer modulus). From Table 2-3 and Table 2-4 it can also be seen that large variations in the base and subgrade modulus values can affect the SCI value significantly. For example, 1118% increase in the base modulus and 712% increase in the subgrade modulus caused 43 and 34 percent reduction in the SCI value, respectively. Therefore, the SCI value is not solely dependent on the surface layer modulus, and can be affected by structural condition of the underlying layers. On the other hand, the definition of good and bad surface layer (HMA) cannot be defined based on its modulus value. Because depending on the environmental temperatures variation on a particular region, a high modulus HMA layer can cause significant surface cracking and a low modules can cause considerable rutting. According to Mehta and Roque

(2003), ninety-five percent of the deflection measured on the surface of the pavement due to the load is case of subgrade condition and remaining five are the attribution of pavement system above subgrade. Hence, SCI thresholds as a performance indicator of HMA layer always may not be indicative of the true condition of HMA layer.

Generally, subgrade layer modulus values less than 69 MPa (~10 ksi) are considered as bad subgrade and above 137 MPa (~20ksi) are considered as good. Figure 2-4-b shows that for subgrade modulus values lower than 62 MPa (~9 ksi), the value of LLI increases beyond the South Africa-suggested upper limit of 120µm (upper limit of Warning Zone). Similarly, when the value of Subgrade modulus increases beyond 130 MPa (~19ksi), the LLI value falls below the 65 µm value; LLI values below this value are considered to be indicative of very good subgrade conditions. Similar trends can be observed from Figure 2-4-a for the MLI parameter. MLI values lower than 115 µm correspond to base modulus values higher than 860 MPa (~125ksi), whereas MLI values higher than 220 µm represents base modulus values lower than 203 MPa (~29 ksi).

The above discussions establish that typical threshold values for the MLI and LLI parameters implemented in South Africa match with typical layer modulus values (for the base and subgrade layers, respectively) indicative of different base and subgrade structural conditions. However, similar conclusions cannot be drawn for the HMA layer based the numerical modeling results. Therefore, it appears that implementing the DBP thresholds to make inferences about base and/or subgrade conditions may be acceptable, whereas solely depending on the SCI parameter to make inferences about the HMA layer may not provide a complete picture of the surface layer conditions.



(a) (b)

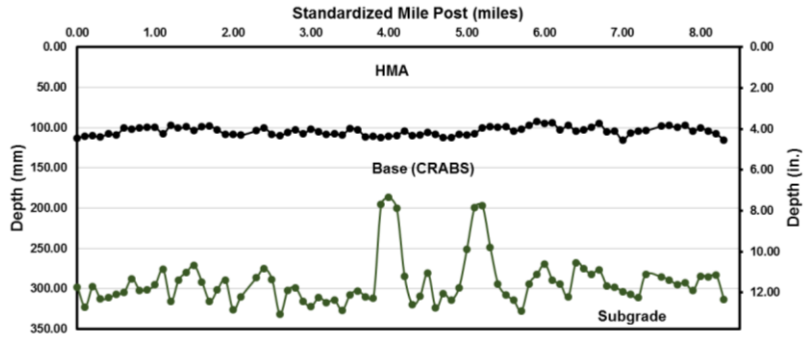
Figure 0-4: Relationship between Layer Modulus and Deflection Basin Parameter Threshold Values: (a) Middle Layer Index or MLI; (b) Lower Layer Index or LLI

Using DBPs for Network-Level Pavement Assessment: Case Study

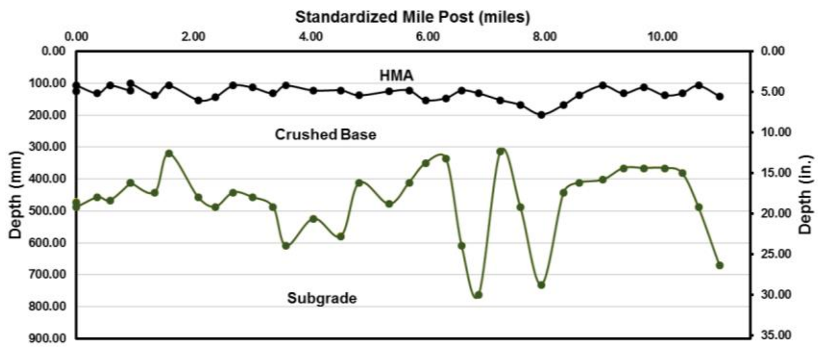
Once the adequacy of DBPs as structural quality indicators of individual pavement layers was established, the next task involved implementing this approach for network-level pavement condition assessment in the state of Idaho. To properly assess the adequacy of this approach, it was important to analyze different pavement sections corresponding to different functional classifications, as well as traffic loading levels. The current study focused on four different pavement sections selected from different locations across the state of Idaho. The four pavement sections were: (a) Interstate Highway 15 (I-15) near Pocatello; (b) Interstate Highway 84 (I-84) near Caldwell; (c) US-95 near Payette; and (d) SH-55 near Middleton. The four selected pavement sections corresponded to different traffic levels, and also different pavement configurations. Even though both the I-15 and I-84 locations corresponded to interstate highways, the truck traffic volume on the I-84 section was significantly higher than that for the I-15 section. Selecting roadway segments exposed to different levels of truck traffic ensured that the suitability of the proposed assessment method could be evaluated for network-level applications.

Background on Selected Pavement Sections

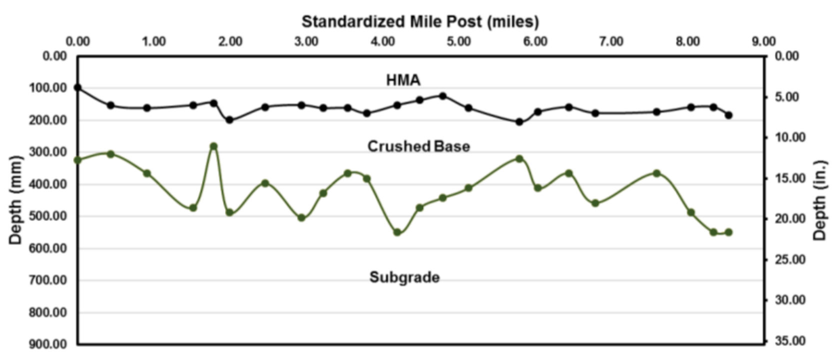
Figure 2-5, shows the variation in pavement layer thicknesses within the selected segments of I-84, US-95, and SH-55 as extracted from boring logs and GPR data; no such data was readily available for the I-15 section. It is important to note that gathering complete construction and maintenance histories of in-service pavements is often not possible for state and local transportation agencies. Maintenance on small sections of pavements are often carried out in small increments as seasonal funds become available. Unless the maintenance activities are completed in the form of a formal construction project with plans and specifications, detailed records are not maintained, and hence extracting information regarding the exact layer thicknesses, last resurfacing activity, etc. often become a challenging task. All desired data concerning the four roadway segments selected in this study could not be obtained. Nevertheless, all available data have been compiled, and have been used to make inferences during analysis of the FWD and visual distress survey data. Note Figure 2-5-a shows a sudden change in the base layer thickness near standardized mile posts 4.0 and 5.0. These two locations correspond to two overhead structures, and a Cement Treated Base (CTB) was used at these locations to ensure sufficient vertical clearance. The authors hypothesize that the sudden change in layer configuration and/or the presence of the overhead structures somehow resulted in the drastically different layer thicknesses obtained from GPR surveys (due to some form of interference). However, it should be noted that this is just a hypothesis, and the authors have not been able to gather any evidence to support or contradict this hypothesis.



(a)



(b)



(c)

Figure 0-5: Pavement Layer Profiles for the (a) I-84, (b) US-95, and (c) SH-55 Pavement Sections (1 mile = 1.6 km)

Table 2-5, presents the subgrade layer information for the US-95 and SH-55 segments as established from laboratory testing of borehole samples. As seen from the Table 2-5, the laboratory-determined R-values for the SH-55 section was higher (average R value = 53.8) than that for the US-95 section (average R-value = 46.4) indicating better subgrade conditions. Similarly, from Figure 2-5, the thickness of the crushed base layer for the SH-55 segment is relatively more consistent compared to that for the US-95 segment. This, combined with the R-values reported in Table 2-5 indicate better base and subgrade layer conditions for the SH-55 segment compared to the US-95 segment, which is most likely due to reduced subgrade intrusion into the base layer. These inferences will be evaluated later in this manuscript using the DBPs.

Table 0-5: Subsurface Investigation Data for (a) US-95, and (b) SH-55 Sections

(a) US - 95								
Bore Hole Number	1	2	3	4	5	6	7	8
Liquid Limit	31	18	21	19	20	16	23	43
Plastic Limit	24	NP	NP	NP	NP	NP	NP	24
R Value	36	69	42	32	46	60	47	39
Unified Classification	ML	GP-GM	ML	SM	SM	SM	ML	CL

(b) SH - 55						
Bore Hole Number	1	2	3	4	5	6
Liquid Limit	23	21	31	25	24	22
Plastic Limit	NP	NP	NP	NP	NP	NP
R Value	47	60	62	N/A	49	51
Unified Classification	ML	ML	SM	CL-ML	ML	ML

Pavement Condition from Visual Distress Survey

The first step in assessing the pavement conditions involved synthesis of pavement condition data from ITD’s visual distress survey database (ITD Pathway Website)**.** Individual distress levels were then compared with threshold values used by ITD to assess the pavement condition (ITD, 2014). Surface cracking is usually reported by ITD in the form of a Cracking Index (CI), where CI = 5 indicates a brand new pavement with no cracks, and CI = 0 represents a completely failed pavement (ITD 2011). Pavement roughness on the other hand, is represented using two different indices. The first one, International Roughness Index (IRI) (Paterson 1986) is an international standard, and is usually measured in mm/m or inch/mile (1 mm/m = 63.5 inch/mile). The IRI values are then scaled by ITD to calculate a Roughness Index (RI), where RI = 0.0 indicates a "very rough" pavement surface, with RI = 5.0 indicating a "very smooth" pavement surface. Table 2-6, lists different distress types and corresponding indices/magnitudes for the four selected roadway segments along with the corresponding condition ratings. Note that all four roadway segments can be categorized as "Interstates" or "Arterials", to compare with the corresponding threshold values.

Table 0-6: Summary of Distress Types, Extent, and Corresponding Condition Ratings for the Four Selected Roadway Segments (1 in. = 25.4 mm; 1 mile = 1.6 km)

Distress Type	Distress Severity / Magnitude							
Pavement Section	I-15		I-84		US-95		SH-55	
	Value	Rating	Value	Rating	Value	Rating	Value	Rating
Cracking Index	2.6	Fair	3.8	Good	2.2	Poor	1.6	Poor
International Roughness Index (IRI, in./mi)	< 95	Good	56 (avg.)	Excellent	90.5 (avg.)	Good	156 (avg.)	Poor
Roughness Index (RI)	3.40	Good	3.95	Good	3.33	Good	2.51	Fair
Average Rut Depth (in.)	0.43"	Fair	0.24"	Good	0.46"	Fair	0.24"	Good

*The data was taken from ITD's visual distress survey database. IRI values for the I-84, US-95, and SH-55 segments were extracted from reports prepared by ITD. IRI values for the I-15 segment are extracted from the visual distress survey database

From Table 2-6, it is evident that the I-84 segment is in "Good" condition as far as cracking is concerned. The I-15 segment is in "Fair" condition, whereas both US-95 and SH-55 sections are in "Poor" condition. Based on the RI values, the two interstate sections appear to be smoother than the two low-volume segments. The SH-55 segment has the lowest RI value, indicating a relatively rough surface. However, note that based on ITD's threshold values, this segment is rated as “Fair” for roughness. As far as rutting is concerned, the I-15 and US-95 segments are in “Fair” condition, whereas the I-84 and SH-55 segments are in “Good” condition.

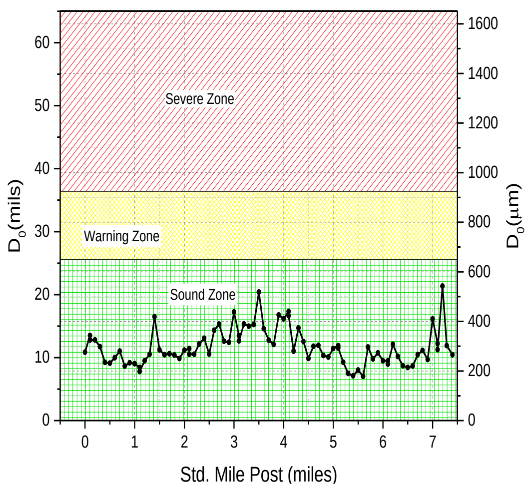
Pavement Structural Condition Assessment using DBPs

Once the functional conditions of the four pavement sections were established, the next step involved using the DBPs to make inferences about their structural conditions, and evaluate whether or not there was a link between the functional and structural condition assessments. Results from these evaluation efforts are presented in the following subsections.

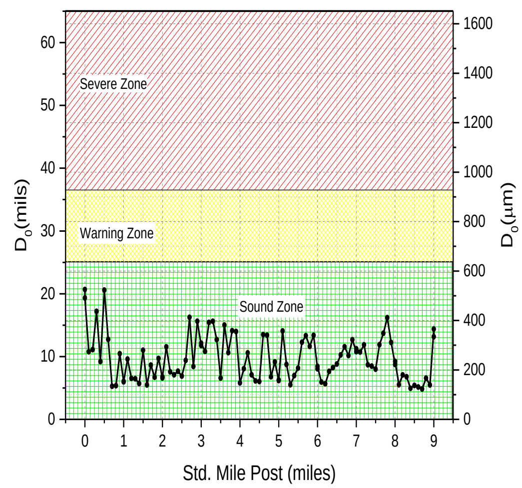
Inferences Concerning the Entire Pavement Structure using DBPs

Deflection under the Load Plate (W_0 or D_0)

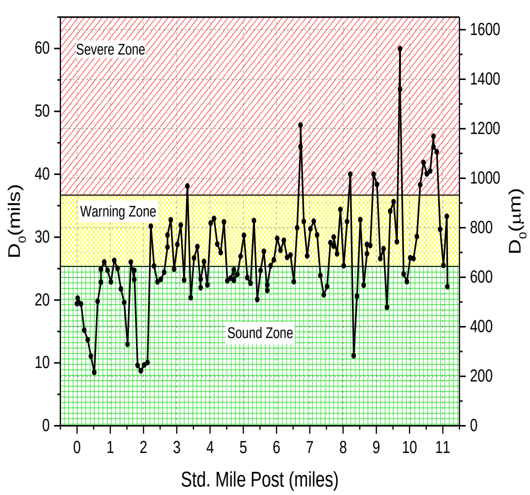
Deflection under the load plate (often expressed as W_0 or D_0) can be used as an indicator for the overall structural condition of the entire pavement structure. Note that all deflection data presented in this paper have been normalized to a load value of 53.37 kN (12000 lb) as is the practice in the state of Idaho. Figure 2-6 shows the variation in the D0 magnitude with mile post for the four roadway segments. Note that all DBP graphs in this manuscript have been plotted using both English and SI units on two vertical axes. However, threshold values corresponding to particular DBPs have been marked on the graphs using the unit originally used by the developers. For example, threshold values for D0 have been given by Horak (2008) using SI units, and have been marked on Figure 2-6 accordingly. Based on the trends presented in Figure 2-6, both the interstate highway segments (I-15 and I-84) appear to be in sound structural condition. This is expected as interstate highway pavements are usually designed targeting high structural capacity. The other two highway segments on the other hand, exhibited significantly higher D_0 values (average D_0 value ≈ 700 μm). Based on the D_0 values, portions of the roadway segments extended into "severe" structural condition, thus highlighting the need for immediate structural rehabilitation. At this point, a comparison can be made between results from the visual distress survey, and inferences based on the D_0 values. As listed in Table 2-6, the Cracking Index values for the US-95 and SH-55 section indicated "poor" surface condition. This is directly translated to the D_0 values for these two segments that indicate "warning" to "severe" structural conditions.

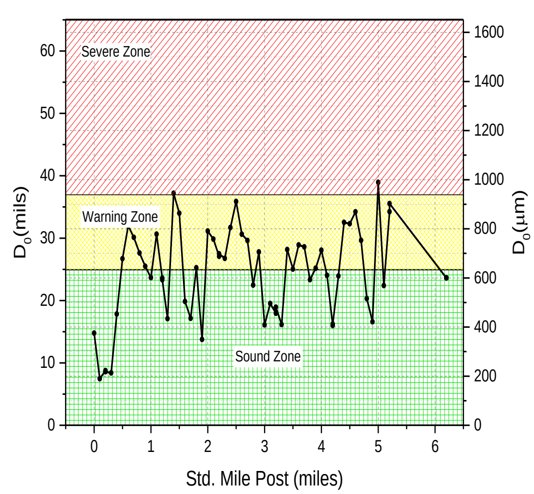


(a) I-15



(b) I-84





(c) US-95 **(d) SH-55**

Figure 0-6: Deflection at the Center of the Loading Plate (D0) for the Selected Pavement Sections (a) I-15, (b) I-84, (C) US-95, (d) SH-55

Inferences Concerning the Upper Pavement Layers

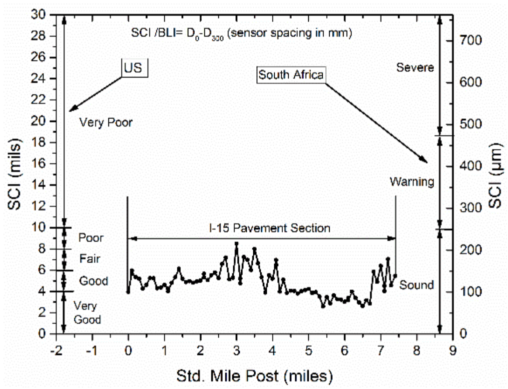
Surface Curvature Index (SCI)

The Surface Curvature Index (SCI) calculated as the difference between the deflections measured at the center of the load plate to that at a distance of 305 mm (12 in.) can be used as an indicator of the structural quality of the upper layers (asphalt layer in particular) of the pavement system (Kim et al. 2000). Note that small SCI values indicate structurally sound upper layers in the pavement structure Figure 2-7 shows the SCI values for the four selected pavement sections. From the Figure 2-7, the upper layers in both I-84 and I-15 segments appear to be in good condition, with the I-84 section being in relatively better condition (no data point above 127 µm or 5.0 mils). This is in direct agreement with trends observed from the Cracking Index (CI) values; based on the CI values, the I-15 section was rated as “fair” whereas the I-84 section was rated as “good”. Note that the other pavement performance indicators such as the IRI value, Roughness Index (RI), and Rut Depths exhibit the same trend, indicating that the upper layers of the I-84 segment are in comparatively better condition than those for the I-15 segment. Therefore, the SCI value when used as a structural quality indicator for the asphalt layer, leads to similar inferences as extracted from the visual distress survey data.

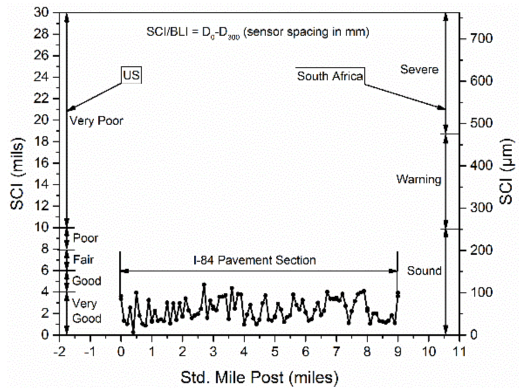
SCI values for the US-95 and SH-55 segments indicate “poor” to “very poor” condition of the upper layers. As shown in Figure 2-7-c, SCI values for the US-95 segment increase significantly after standardized milepost 6.5. Close inspection of the visual distress survey database indicated that this section of the roadway segment exhibited excessive surface cracking compared to the other sections. Therefore, trends observed from the SCI could be directly corroborated from field data, and clearly indicated an asphalt layer in “poor” to “very poor” structural condition. At this point it is important to note that from the numerical modeling verification effort, the SCI values did not directly correspond to typical modulus values observed for HMA layers in practice.

Base Layer Index (BLI)

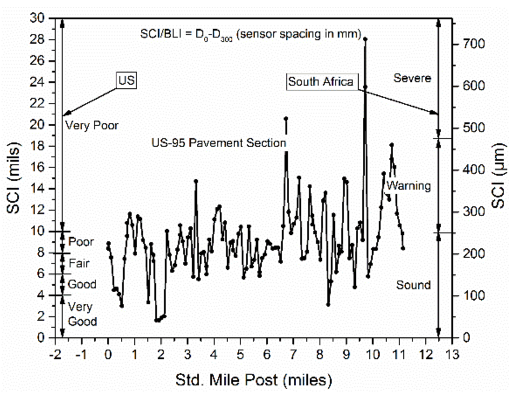
The Base Layer Index (BLI) is numerically identical to the SCI. However, per the South African standard (Horak 2008; Horak et al. 2015), the BLI value is used as an indicator of the structural condition of the base layer. Combining the two conventions, it can be said that the BLI (or SCI) value indicates the structural condition of the upper layers of the pavement structure, which in turn is related to the nature of stress dissipation by the upper layers. Different threshold values are used for the SCI and the BLI as the inferences concern different layers within the pavement structure. Figure 2-7, presents both SCI and BLI values (numerically identical) for the four pavement sections under consideration. Threshold values used in the US (Chang et al. 2014) have been marked on primary ordinate axis, whereas threshold BLI values used in South Africa (to make inferences about the base layer) have been presented along the secondary ordinate axis. As seen from the Figure 2-7, the base layers for I-15 and I-84 segments appear to be structurally sound whereas those for the US-95 and SH-55 segments appear to be in need of rehabilitation. As already mentioned, the I-84 section comprises a cement-stabilized base layer whose effect gets directly reflected through the low BLI values.



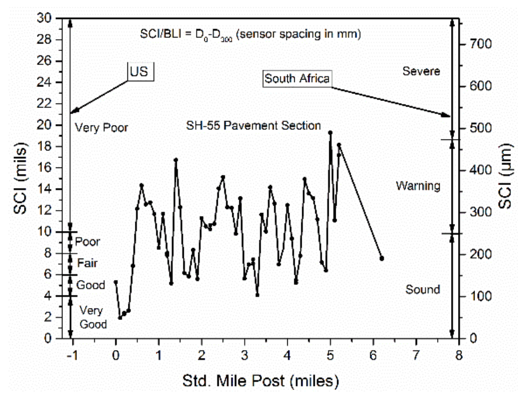
(a) I-15



(b) I-84



(c) US-95



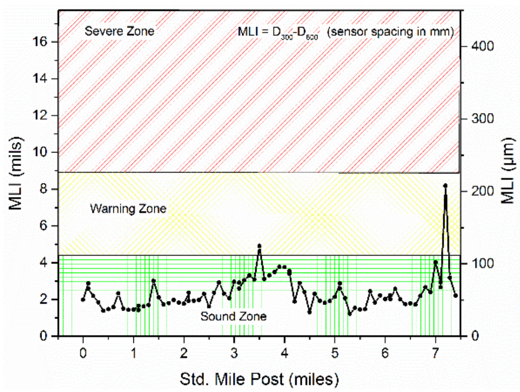
(d) SH-55

Figure 0-7: Surface Curvature Index (SCI) / Base Layer Index (BLI) Values for the Selected Pavement Sections Showing the Threshold Ranges Recommended by Researchers in the US as well as in South Africa: (a) I-15, (b) I-84, (C) US-95, (d) SH-55

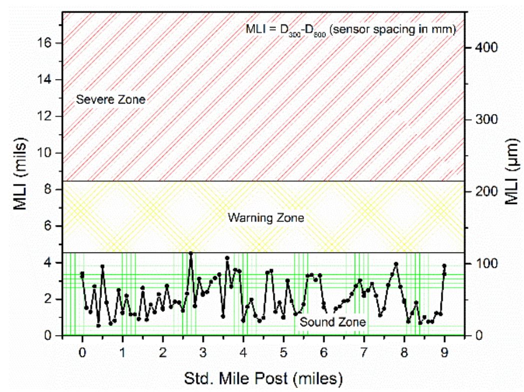
Inferences Concerning Intermediate Pavement Layers

Middle Layer Index (MLI)

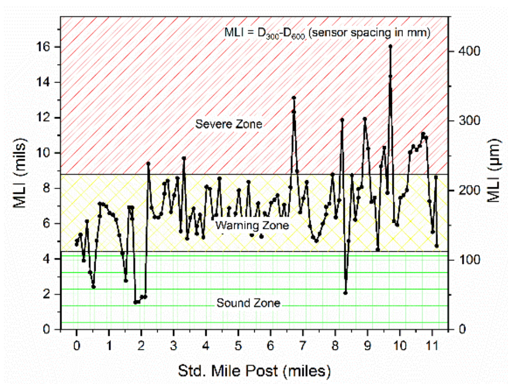
The Middle Layer Index is used as an indicator of the structural quality of the subbase/subgrade layer. In absence of detailed information about the pavement layer configuration, the MLI value can be used to make inferences about the intermediate and lower pavement layers. As before, MLI values for the I-15 and I-84 segments indicate structurally sound subbase/subgrade layers, whereas the data for US-95 and SH-55 segments indicate underlying layers in need of repair (see Figure 2-8).



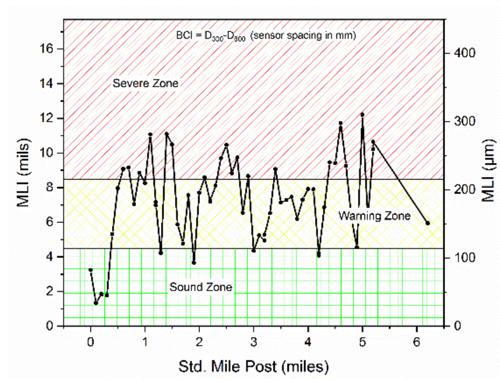
(a) I-15



(b) I-84



(c) US-95



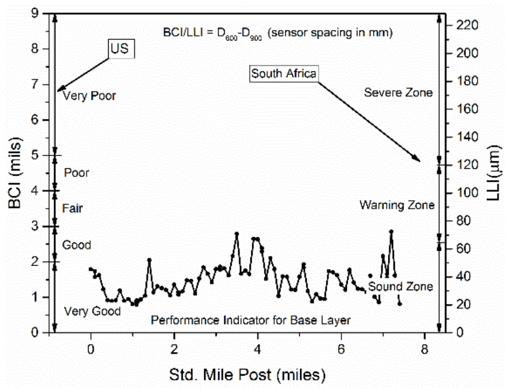
(d) SH-55

Figure 0-8: Middle Layer Index (MLI) Values for the Selected Pavement Sections (a) I-15, (b) I-84, (C) US-95, (d) SH-55

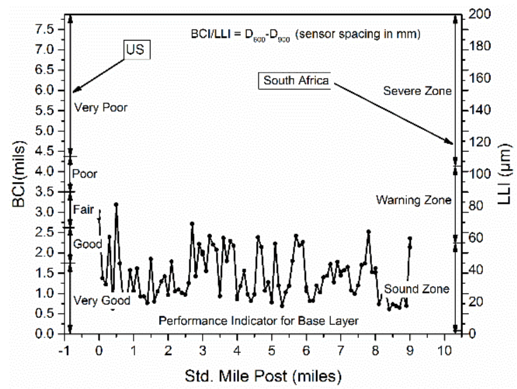
Inferences Concerning Lower Pavement Layers

<u>Base Curvature Index (BCI)</u>

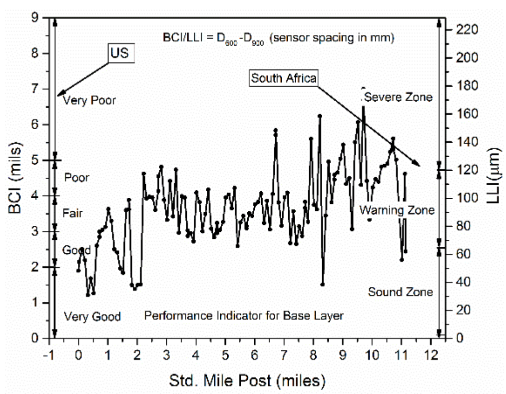
The Base Curvature Index (BCI) is used as an indicator of base quality per the conventions used in the US. Kim et al. (2000) observed that BCI was a good indicator of subgrade quality. BCI values calculated for the four selected roadway segments have been plotted in Figure 2-9. As shown in the figure, the base layers for the I-15 and I-84 segments appear to be in "good" or "very good" condition. Parts of the base along the SH-55 segment appear to be in "poor" condition. However, a larger portion of the base along US-95 appears to be in structurally worse condition compared to the SH-55 section. Once again, this matches with the observation that the base layer along SH-55 is more consistent in thickness compared to that for US-95. As already mentioned, this may be a result of increased subgrade intrusion into the base layer along the US-95 segment.



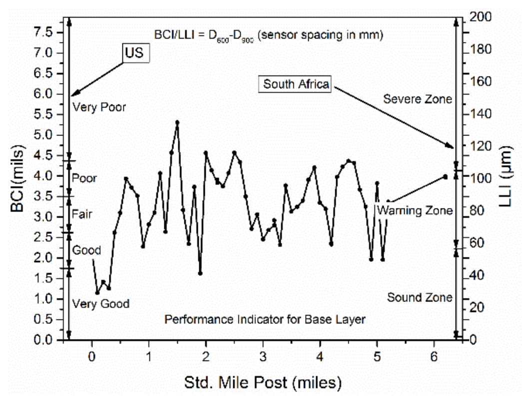
(a). I-15



(b). I-84



(c). US-95



(d). SH-55

Figure 0-9: Base Curvature Index (BCI)/ Lower Layer Index(LLI) Values for the Selected Pavement Sections (a) I-15, (b) I-84, (c) US-95, (d) SH-55

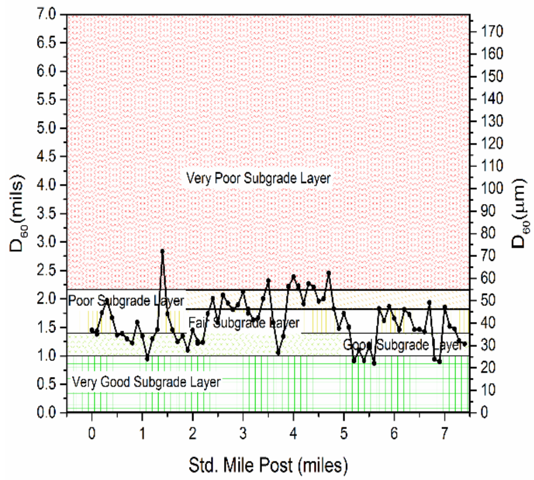
<u>Lower Layer Index</u>

The Lower Layer Index (LLI) is numerically identical to the BCI, and is used to make inferences about structural condition of the subgrade layer. Figure 2-9, shows the LLI values for the four roadway segments along with the threshold levels separating the “sound”, “warning”, and “severe” zones. As observed from the other DBPs, the subgrade layers for the interstate highway segments appear to be in significantly better condition compared to the US-95 and SH-55 roadway segments. The subgrade for SH-55 appears to be in relatively better condition compared to that for US-95.

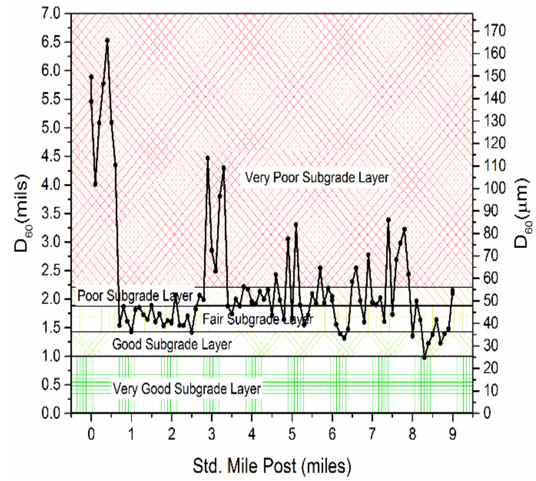
Deflection under the 7th Sensor (W60 or D60)

Deflection under the 7th sensor, often denoted as W60 or D60 can be used as an indicator of subgrade condition. This stems directly from the nature of stress distribution in flexible pavements, where upper layers in the pavement structure affect the surface deflection at locations relatively close to the point of load application. Moving radially away from the load, the surface deflection is governed to a large extent by properties of the subgrade layer. It is therefore common practice to use the surface deflection recorded by the 7th sensor (at a distance of 1524 mm from the center of the loading plate) as an indicator of the structural condition of the subgrade layer.

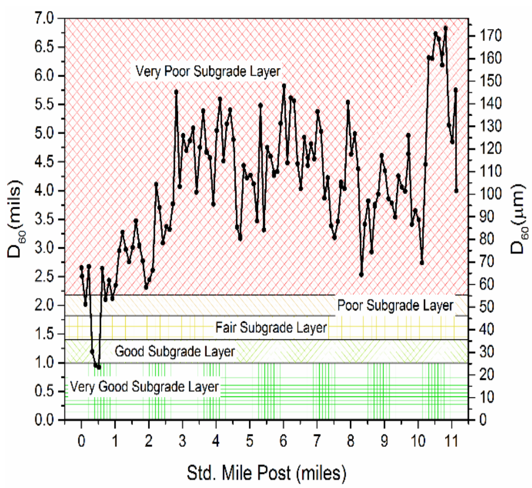
Based on the D60 values (see Figure 2-10), the US-95 and SH-55 segments are in significantly worse condition compared to the I-15 and I-84 sections. Furthermore, the subgrade along the SH-55 segment appears to be in relatively better condition compared to that along US-95. This is in direct agreement with the R-value trends as well as the inference regarding lower subgrade intrusion along the SH-55 segment. Interestingly, the D60 trace for I-84 shows two distinct “spikes” near standardized milepost values 0.25 and 3.0. Close inspection of site conditions indicated that these two locations corresponded to two underpasses. The structural discontinuity caused by the underpasses somehow resulted in very high D60 values for these two locations. This may even be due to excessive vibrations of the geophone caused by stress wave reflections from the near-by structure. Nevertheless, the primary observations from the D60 plots concern the distinctively worse subgrade conditions for US-95 and SH-55 compared to the two interstate highway segments.



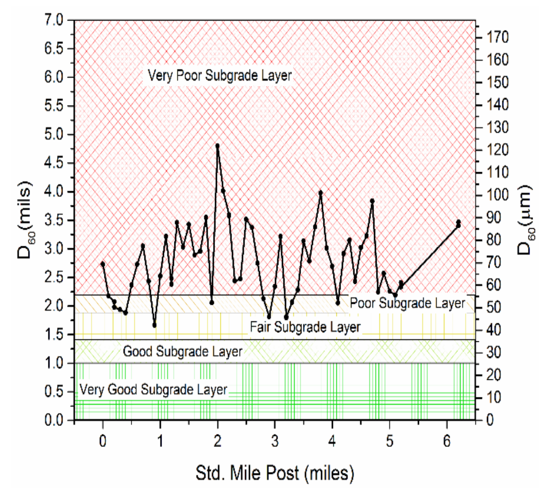
(a). I-15



(b). I-84



(c). US-95



(d). SH-55

Figure 0-10: Deflection Measured by the 7th Sensor (D60) for the Selected Pavement Sections (a) I-15, (b) I-84, (C) US-95, (d) SH-55

Implementation as a Network-Level Pavement Rehabilitation Selection Approach

As already mentioned, surface distresses observed from the visual distress surveys could be directly linked to the structural condition of individual pavement layers through the use of DBPs. The DBPs essentially capture the shape of the deflection basin, which is a function of the load distribution characteristics of the pavement structure. Use of the

DBPs can help engineers identify problematic layers within a pavement structure to facilitate the selection of appropriate rehabilitation methods. This phenomenon can be clearly illustrated by taking examples of the I-15 and US-95 segments analyzed in the current research effort. Both the I-15 and US-95 segments were classified as “Good” per the Roughness Index (RI) criterion, and “Fair” per the Rutting criterion. The I-15 segment was classified as “Fair” based on the Cracking Index value, whereas the US-95 segment was classified as “Poor” based on the CI value (see Table 2-6). This information may lead an engineer to infer that the surface layer in US-95 is problematic, and hence pavement resurfacing may appear to be a reasonable approach to improve the pavement condition. However, detailed analysis of the DBPs clearly indicated that the base and subgrade layers along the US-95 segment were in “poor” structural condition, and hence rehabilitation activities along this roadway segment should target improvement of the underlying layers. More importantly, this information could be extracted without the need for backcalculation of layer moduli from the FWD data. This is particularly advantageous for in-service pavements for which detailed layer thickness data may not be readily available. With the advent of modern FWD equipment capable of testing several miles of road segments per day without significantly affecting the traffic flow conditions, collection of network-level FWD data is now common practice among state and local transportation agencies. This data can be used for “quick calculation” of the DBPs, which can then be matched against standard threshold values to assess the structural conditions of individual pavement layers. Such a rapid, reliable, and cost-effective analysis approach will help engineers with educated decisions on pavement rehabilitation method selection.

Summary and Conclusions

This paper presented findings from on ongoing research study at Boise State University focusing on the development of a network-level pavement rehabilitation selection approach based on the analysis of visual distress survey data and calculation of Deflection Basin Parameters (DBPs) from Falling Weight Deflectometer (FWD) test data. First, a numerical modeling effort was carried out to mechanistically verify the validity of different DBPs, and their typical threshold values recommended by researchers. A total of 216 pavement sections were analyzed by assigning a range of modulus values to the HMA, base, and subgrade layers. Results from the numerical modeling effort indicated that typically used DBP threshold values for the base and subgrade layers were in general agreement with typical ranges of layer moduli observed in practice. However, the DBP corresponding to the surface layer (Surface Curvature Index or SCI) was significantly affected by moduli of the underlying layers, and therefore, cannot be used as the primary indicator of surface layer conditions.

This was followed by detailed structural and functional evaluation of four different roadway segments across the state of Idaho. The objective was to assess whether or not combined use of DBPs along with the functional condition data will facilitate better understanding of different pavement layer conditions. Integrated analysis of the visual distress data and the DBPs could accurately identify problematic layers within a pavement section. The primary advantage of this method based on the analysis of DBPs is that it does not rely on pavement layer thickness data. Adopting this unified assessment approach, the research team successfully recommended suitable rehabilitation methods to the Idaho Transportation Department (ITD). Continued work along this line can facilitate integration of this assessment method into the network-level pavement maintenance program in Idaho to facilitate effective and economical pavement preservation practices.

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MANUSCRIPT TWO – USE OF POLYMER GROUTING TO REDUCE DIFFERENTIAL HEAVE IN PAVEMENTS OVER EXPANSIVE SOILS[2]

Abstract

Flexible pavement sections constructed over expansive soil deposits often undergo significant damage due to the volume changes in the underlying soil strata. In cases where expansive soil deposits are confined to shallow depths, conventional rehabilitation methods such as pre-wetting, chemical stabilization, removal and replacement, etc. can be pursued. However, such treatment strategies become impractical in cases where the expansive soil deposit lies more than 1 m (3 ft.) underneath the pavement surface. In such cases, implementation of alternative remedial measures are desired to dissipate the soil-generated swelling stresses. A recently completed research study at Boise State University investigated the differential heaving problem along a particular section of US-95 near the Idaho-Oregon border. Laboratory characterization of soil samples indicated the presence of highly expansive soils up to depths of 7.6 m (26 ft.) from the pavement surface. Through subsequent numerical modeling efforts, a hybrid geosynthetic system comprising geocells and geogrids was recommended for implementation during pavement reconstruction. A follow-up research study has focused on evaluating the suitability of polyurethane grout injection as a potential remedial measure for this pavement section. Laboratory testing of

[2] This chapter includes results already reported in the following publication. Contribution of the coauthors is sincerely acknowledged: Rabbi, M. F., Boudreau, R. L., Chittoori, B., Sotirin, M., and Mishra, D. (2018). “Use of Polymer Grouting to Reduce Differential Heave in Pavements over Expansive Soils”. Submitted to the Ground Improvement Journal (Under Review)

unbound materials treated with a High-Density Polyurethane (HDP) indicated significant improvements in resilient modulus and shear strength properties. Finite Element (FE) modeling of the problematic pavement section indicated that depending on the treated layer thickness, the differential heave magnitude can be reduced by up to 75% compared to untreated sections. For the particular section of US-95 studied, 25-38% reduction in differential heaving can potentially be achieved through polyurethane grout injection shapes.

Key Words: Expansive Soils, Polyurethane Grout Injection, High-Density, Polyurethane (HDP), Differential Heave, Finite Element Modeling

Introduction

A study sponsored by the National Science Foundation (NSF) reported that every year maintenance/rehabilitation costs associated with infrastructure damage due to expansive soils are significantly higher than other natural disasters such as floods and earthquakes (Jones Jr and Holtz 1973). The yearly cost of damage was reported to be approximately $2.3 billion (Gromko 1994). Besides repairs to structures damaged by expansive soils, maintenance and rehabilitation efforts also focus on soil stabilization/treatment to address the root cause(s) of the problem. Common approaches involve chemical stabilization, pre-wetting, removal and replacement, etc. However, such treatment alternatives become impractical in cases where the expansive soil deposit extends beyond 1 m (3 ft.) from the surface. Exposing soil layers that are deeper than 1 m (3 ft.) from the surface for treatment/replacement can be extremely uneconomical. In such scenarios, in-situ stabilization alternatives need to be explored. For flexible pavement sections constructed over expansive soil deposits, polyurethane grout injection can be a viable treatment alternative to reduce recurrent differential heaving and cracking of the surface. A recently completed research study at Boise State University evaluated the effectiveness of polyurethane grout injection as an alternative remedial measure to address the problem of recurrent pavement damage in flexible pavements constructed over expansive soil deposits. If found effective, the injection of High-Density Polyurethanes (HDPs) can be particularly useful as it does not require removal of the existing pavement layers. Injection of HDPs into existing pavement layers has the potential to result in the formation of a “flexible layer” (through polyurethane-soil or polyurethane-aggregate mixing) that can uniformly dissipate the swell pressures from the underlying soil layers.

Background and Problem Statement

A recently completed research study at Boise State University (sponsored by the Idaho Transportation Department, ITD) investigated the problem of recurrent differential heaving along a particular section of US-95 near the Idaho-Oregon border. Constructed over an expansive soil deposit, this roadway section has been experiencing recurrent pavement damage over the past several decades. Several rehabilitation and reconstruction efforts have been carried out over the years with limited or partial success. Extensive laboratory characterization of soil samples collected from underneath the problematic roadway section indicated very high Montmorillonite contents. Moreover, the expansive soil deposit often extended up to depths of beyond 7.6 m (26 ft.) from the pavement surface. Presence of the expansive soil deposit at such depths renders the application of conventional stabilization methods impractical. Through numerical modeling efforts, the research team recommended a hybrid geo-synthetic system (comprising geogrids and geocells) for placement within the base layer during pavement reconstruction efforts. The hybrid geosynthetic layer was found to be able to uniformly dissipate the soil-generated swelling pressures, thereby reducing damage caused to the pavement surface (Chittoori et al. 2016a; Chittoori et al. 2018). Large-scale box testing in the laboratory exhibited considerably reduced differential heave magnitudes when the unbound base layer was reinforced using the hybrid geosynthetic system (Tamim 2017). A subsequent research effort has focused on evaluating the suitability of polyurethane grout injection as a potential nondestructive remedial measure to address the recurrent surface damage along this particular roadway section. Laboratory testing and numerical modeling were carried out to quantify the effect of polyurethane grout injection on the magnitude of differential heave observed at the pavement surface. Findings from this research effort are documented in this manuscript.

Research Objectives and Tasks

The primary objective of this research effort was to evaluate the effectiveness of polyurethane grout injection as an alternative “nondestructive” remedial measure for differential heave mitigation. The research task was divided in two phases. In phase-I, laboratory testing was carried out to evaluate the effect of High-Density Polyurethane (HDP) injection on the mechanical properties of unbound aggregates and expansive soils. The laboratory testing involved: (1) resilient modulus testing, and (2) rapid shear strength testing. Moreover, visual inspection of the specimens was carried out to assess the extent of HDP permeation for different injection procedures. In total, three types of base materials and one type of expansive soil were tested in the laboratory. The laboratory testing was designed to address two primary research questions: (1) Can uniform permeation of the HDP into the soil and aggregate specimens be accomplished in a laboratory setting? and

(2) Does the HDP have a significant effect on the mechanical properties (resilient modulus and shear strength) of the unbound materials? The primary challenge during the laboratory testing effort involved preparing HDP-treated aggregate and soils samples in a manner that was representative of actual field conditions. As no specifications or guidelines were available to prepare polyurethane-treated aggregate and soil specimens, three different methods of specimen preparation were investigated. Phase-II of the study involved numerical modeling of flexible pavement sections constructed over expansive subgrade layers. The first step in the numerical modeling process was to simulate the differential heaving induced in flexible pavements due to moisture infiltration into the expansive soil deposits. The next step involved simulating flexible pavement sections comprising polyurethane-stabilized base and subgrade layers. Model-predicted results were compared to quantify the effects of HDP injection into the base and expansive subgrade layers.

Review of Published Literature

Although several researchers have studied the effect of polyurethane injection on unbound layer performance under traffic (vehicular and railway) loading, very limited research initiatives have focused on studying the behavior of polyurethane-treated layers under upheaval pressures originating from expansive soils. The success of polyurethane grout injection into soil/aggregate layers is strongly dependent on the extent of permeation of the HDP into the material being treated. Generally, the HDP spreads easily within aggregate base layers, and can create a stabilized layer that is relatively uniform and has an increased modulus. Keene et al.(2012) reported that in coarse grained aggregates like ballast, the polymer can permeate easily through the void space during injection and expansion, and can form a uniform geo-composite layer. However, the effect of polyurethane grout injection on low-permeability clayey soil behavior is not very well understood. Sasaki (Sasaki 2008) reported that generation of a homogeneous polymer-treated samples is almost impossible. He also found that polymer grout can propagate for distances of more than a meter through voids in dried expansive clay soil. Due to this propagation of polymer grout in the soil the permeability of treated soil decreases (Sasaki 2008). As the swelling behavior is directly related to moisture permeation into the soil, reduced permeability can contribute towards a reduction in swelling potential. Buzzi et al. (Buzzi, Fityus and Sloan 2010) found, both through laboratory and field experiments, that the swelling potential of an expansive soil is reduced upon treatment with polymer grout. Furthermore, it was also reported that the yield stress of expansive soils increased significantly upon polymer treatment. However, the above-mentioned benefits are contingent upon ‘proper’ dispersion of the HDP within the layer being treated. Several small- and large-scale testing initiatives have focused on studying the permeation of HDP within aggregate/soil layers. Some laboratory studies mentioned that injected polyurethane grout can permeate easily in coarse materials, whereas others reported that injection into fine materials/cracked soil contributes towards filling up of the cracks without significantly affecting the rest of the soil (Sasaki 2008; Yu 2013; Getzlaf 2006; Mark et al. 2010) .Figure 3-1-a (Stephens and Honeycutt, Online Documentation) shows that the polyurethane treatment results in the formation of a relatively continuous composite layer, whereas Figure 3-1-b shows an instance of non-uniform permeation of the grout. The following sections present details about the laboratory testing effort carried out under the scope of the current study to quantify the effect of polyurethane grout injection into aggregate and soil specimens.

(a) (b)

Figure 0-1: Photographs Showing: (a) Comparatively Uniform Dispersion of Polymer; and (b) Non-Uniform Disperse of Polymer (Stephens and Honeycutt, Online Documentation)

Laboratory Testing of Geomaterials

Three types of base/subbase materials were selected for laboratory characterization: (1) Natural Sand (Sand); (2) Graded Aggregate Base (GAB); and (3) #57 Stone. The expansive soil material tested was collected from underneath the problematic section of US-95 near the Idaho-Oregon border. Extensive laboratory characterization of this soil was conducted under the scope of another research study, and detailed results have been published elsewhere (Chittoori et al. 2016; Islam 2017; Chittoori, Mishra and Islam 2018; Tamin 2017). Liquid Limit (LL) values for this soil were found to range between 44% and 185%, with Plasticity Index (PI) values ranging between 25%-136%. Photographs of the materials tested in the laboratory have been included in Figure 3-2.

(a) Sand

(b) GAB

(c) #57 Stone

(d) Expansive Soil (US-95)

Figure 0-2: Photographs Showing the Four Material Types Tested in the Laboratory: (a) Sand, (b) GAB, (c) #57 Stone, & (d) Expansive Soil (US-95)

Development of Polymer Injection System in the Laboratory

As already mentioned, the primary challenge during the laboratory testing effort was to ensure that the degree of permeation of polyurethane grout into aggregate/soil specimens achieved in the laboratory closely simulated actual field conditions. Three different types of injection methods were developed to simulate the polyurethane grout injection procedure in the laboratory. The first method involved the use of a 1.2 m x 1.2 m (4 ft. x 4 ft.) steel box filled with the geomaterial. The second method involved injection into a steel drum of 0.2 m3 (55 gallon) volume. The third method involved compaction of the soil/aggregate in a 152-mm (6-in.) diameter by 356-mm (14-in.) long PVC pipe. After a limited number of trials, it was observed that method-1 was significantly expensive and time-consuming; method-2 failed to generate specimens with uniform degree of grout penetration into the geomaterial. Figure 3-3, shows the sample preparation mold and extracted samples of method 1 and 2.

Figure 0-3: Method-1 and 2 Samples Preparation mold and Extracted Samples

Method-3 was found to be the most effective, and is the only one discussed in the current manuscript. The aggregate/soil in the mold was compacted to pre-determined moisture-density conditions established using the standard compaction method (AASHTO T99). Figure 3-4, presents a flow chart depicting different steps in the laboratory testing protocol and extracted samples.

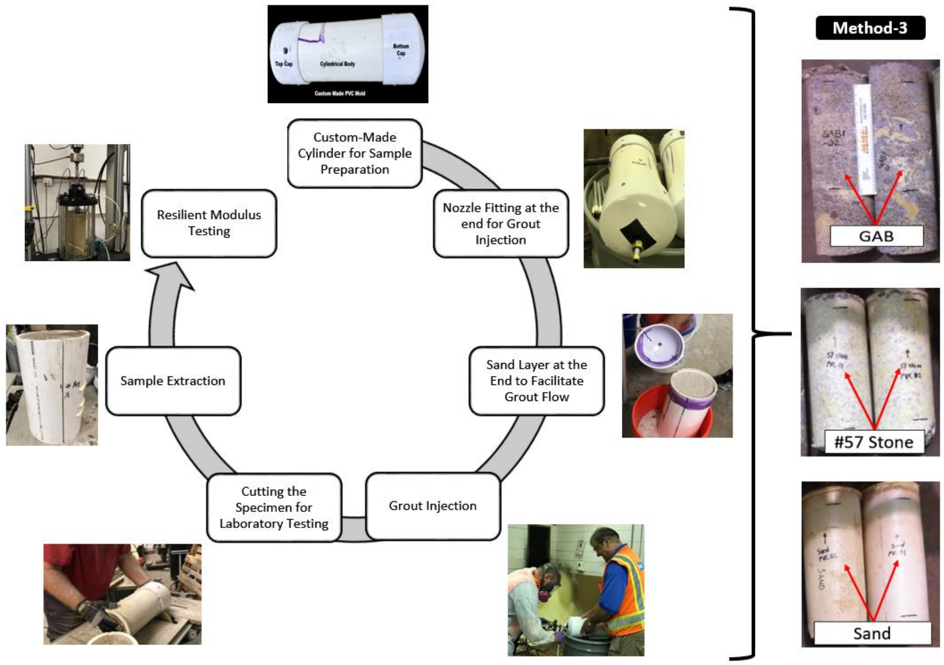


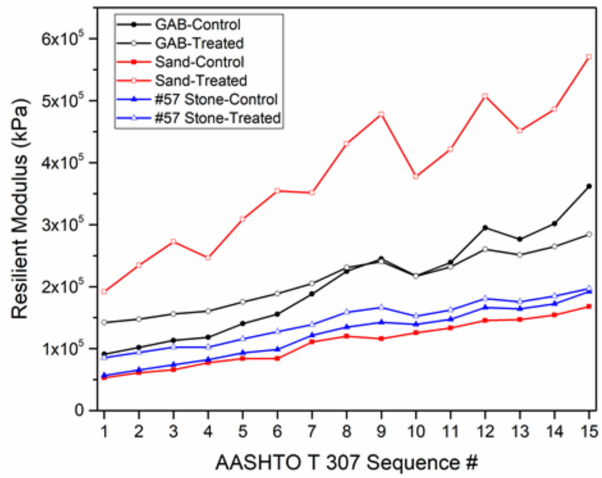
Figure 0-4: Flow Chart Depicting Different Steps in the Laboratory Testing Protocol

Effect of Polyurethane Grout Injection on the Mechanical Properties of Aggregates and Soils

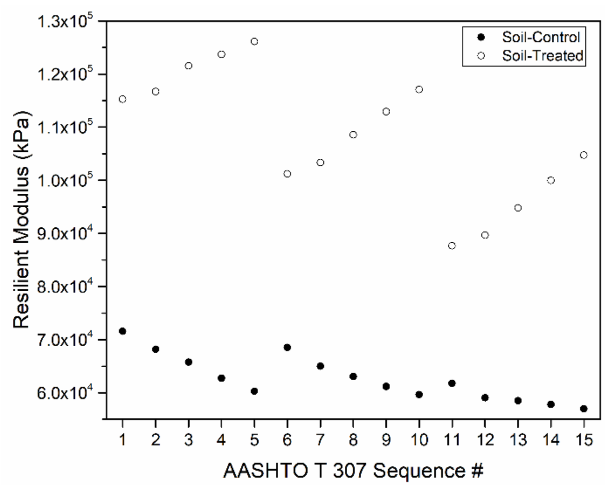
Triaxial testing was conducted in the laboratory to quantify the effect of polyurethane grout injection on aggregate/soil resilient modulus and shear strength. Resilient modulus testing was carried out per the AASHTO T-307 protocol; upon completion of the resilient modulus testing, quick shear testing was carried out by subjecting the same specimen to a controlled rate of axial deformation (1% axial strain per minute up to 5% strain). Results from the laboratory tests are discussed in the following sections.

Resilient Modulus Test Results

Figure 3-5 shows the resilient modulus test results for both untreated and HDP-treated base (Figure 3-5-a) and subgrade (Figure 3-5-b) materials. As seen from Figure 3-5-a, HDP injection into the GAB and #57 Stone did not have a significant effect on the resilient modulus values. However, pronounced increase in resilient modulus was observed for the HDP-injected natural sand (233% improvement in MR value corresponding to the seventh stress sequence of the AASHTO T-307 test protocol). Note that resilient modulus testing was also carried out on pure HDP specimens, and a constant modulus value of 31026 kPa (4.5 ksi) was observed; this data has not been included in the graph.



(a)



(b)

Figure 0-5: Resilient Modulus Test (AASHTO T-307) Results for Control and HDP-Treated (a) Base Materials; and (b) Expansive Soil Subgrade

Figure 3-5-b, shows resilient modulus test results for untreated and HDP-treated expansive soil subgrade specimens. Considering the extremely low permeability of clayey soils, the degree of grout penetration into clayey soil specimens is not likely to be the same as that in aggregate specimens. Accordingly, a natural hypothesis would be that the grout would contribute to densification of the expansive soil mass owing to the increased level of confinement within the PVC tube. It was therefore of interest to quantify the level of densification (and subsequent potential increase in stiffness) of the subgrade due to the pressure exerted by the expanding polyurethane grout. Subgrade soil specimens for resilient modulus testing were prepared following a procedure similar to that for the base materials. Resilient modulus testing was carried out following AASHTO T-307 protocols.

Even though expansive soil specimens were significantly less permeable compared to the base materials specimens, the HDP-treated soil sample showed considerably higher resilient modulus values compared to the untreated specimens (see Figure 5-2-a). Three replicate samples were tested in laboratory, and the results were considerably consistent with a coefficient of variation of 10.7%. This indicated that the specimen preparation approach developed in the laboratory led to repeatable specimen behavior under repeated loading. Table 3-1 lists the summary modulus values (corresponding to a Bulk Modulus, θ = 275.8 kPa during AASHTO T-307 testing) for all four materials under untreated and HDP-treated conditions.

Table 0-1: Laboratory Test Results: Elastic Modulus Improvement

Geo Materials Types	Resilient Modulus E, kPa (psi)		PI* (%)	Comments
	Untreated	Treated		
GAB	221000 (32000)	221000 (32000)	0%	No Change
Sand	124000 (18000)	413000 (60000)	233%	Significant Change
#57 Stone	138000 (20000)	159000 (23000)	15%	Minor Change
Subgrade	65000 (9427)	104000 (15084)	60%	Considerable Change

*Note: PI is Percent Increase

Quick Shear Test Results:

As already mentioned, quick shear tests were carried out on the resilient modulus specimens after completion of the AASHTO T-307 test sequence. Results from quick shear testing on all four material types (both treated as well as untreated) are presented in Figure 3-6. As seen from the figure, HDP injection resulted in significantly higher strengths for all four materials. The ultimate strength of tested materials increased by more than 500% and the stiffness (as measured by secant modulus) is improved by 700% to 1,000% for the #57 stone and GAB materials, and nearly 8,000% for the natural sand. Once the effect of polymer injection on the mechanical properties of the four material types materials was established, the next task involved using these properties in the numerical models to quantify the corresponding effect on predicted heave on the pavement surface. Details of the numerical modeling effort are presented in the following sections.

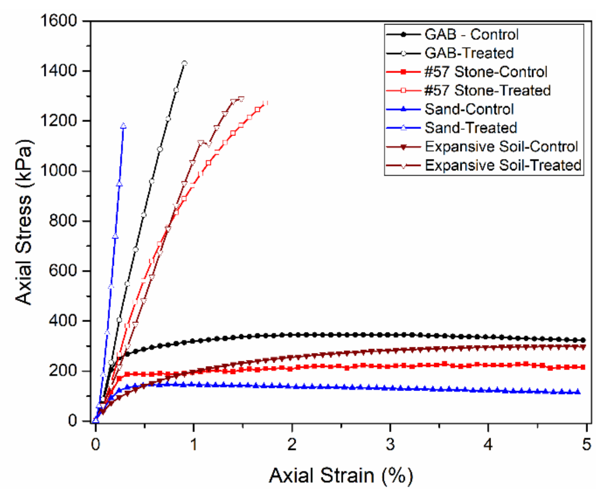


Figure 0-6: Quick Shear Test (AASHTO T-307) Results of Control and URETEK Treated Base Materials (Sand, Gap, #57 Stone & Expansive Subgrade Soil)

Numerical Modeling of Flexible Pavement Sections Constructed over Expansive Soil Subgrades

Numerical modeling is widely used to study complex natural phenomena. Swelling and shrinkage behavior of expansive soil is one such complex geotechnical phenomenon that can be studied using numerical modeling. To capture this complex swelling behavior and the impact of elastic modulus improvement in the base/subgrade layer on differential pavement heave, the research team used ABAQUS®, a commercially available Finite Element (FE) -based numerical modeling software package. Note that the FE method is primarily based on a continuum approach; to model a non-homogeneous layer (with frequently changing material properties) such as HDP-treated aggregate/ soil, detailed information about the spatial variation of properties within the layer is required. Although a HDP-treated aggregate/ soil is not perfectly homogeneous, this numerical modeling effort utilized several simplified assumptions to model the HDP-treated layers in a pavement system. The authors would therefore like to emphasize that results from this numerical modeling effort should not be used as "exact predictions" of field behavior; rather, they should be taken as representative trends of the expected field behavior. Details of the FE modeling approach and corresponding results are presented in the following sections.

Pavement Layer Configuration and Material Property Assignment

A representative section of the particular section of US-95 at the Idaho-Oregon border was modeled using ABAQUS. Representative thicknesses of individual pavement layers were obtained from the drilling effort carried out by Chittoori et al.(Chittoori et al., 2016 a & b). Based on data extracted from field boring logs, the expansive subgrade soil strata lies 259.5 cm (~102 in.) below the pavement surface. Figure 3-7, shows a schematic of the pavement section modeled in this study.

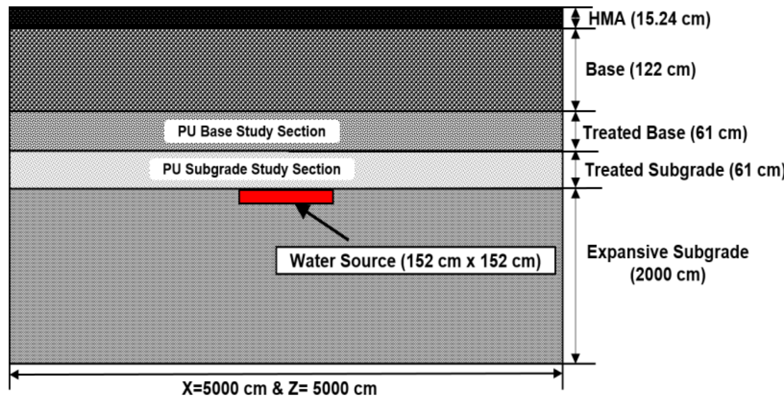


Figure 0-7: Simplified Representative Pavement Section of US-95 Roadway

Material properties for the aggregates and soils (both for untreated as well as treated conditions) used in this modeling effort were obtained from the laboratory test results. No testing was conducted on the Hot-Mix Asphalt (HMA) layer, and therefore, representative properties obtained from literature were used in the model. The HMA layer was modeled as linear-elastic, whereas the untreated base and subgrade layers were modeled as elasto-plastic (using the Mohr-Coulomb plasticity model). Expansive behavior of the subgrade layer was modeled using sorption and swelling data established in the laboratory (Chittoori et al, 2016 a & b; Islam 2017). For the moisture swelling model, the volume change behavior of expansive subgrade soil was determined by volumetric swelling testing (Chittoori et al. 2016 a & b; Islam 2017; Tamim 2017). The HDP-treated base and subgrade layers were modeled as linear-elastic, which closely simulates the stress-strain behavior of polyurethane grout-injected specimens tested in the laboratory. Material properties used in the modeling are listed in Table 3-2.

Table 0-2: Materials Properties used in the Modeling: (a) Control Section; (b) HDP-Treated Geomaterials

(a) Control Section					
Properties	HMA	Sand	GAB	#57 Stone	Expansive Subgrade
Mass Density, kg/m3(ρ lb/ft3)	2387(149)	1587(99)	2291(143)	1538(96)	1025(64)
Elastic Modulus, E, MPa(ksi)	5171(750)	124(18)	221(32)	138(20)	65(9.41)
Poisson's Ratio, ν	0.3	0.35	0.35	0.35	0.4
Internal Angle of Friction, φ	***	30	40	40	10
Angle of Dilation, ψ	***	13	13	13	3
Cohesion, c', kPa(psi)	***	2(0.29)	2(0.29)	2(0.29)	75(10.8)

(b) HDP-Treated Geomaterials				
Properties	Treated Sand	Treated GAB	Treated #57 Stone	Treated Subgrade
Mass Density, kg/m3(ρ lb/ft3)	1674(104.5)	2291(143)	1538(96)	1265(79)
Elastic Modulus, E, MPa(ksi)	414(60)	221(32)	159(23)	103(15)
Poisson's Ratio, ν	0.35	0.35	0.35	0.35
Internal Angle of Friction, φ	***	***	***	***
Angle of Dilation, ψ	***	***	***	***
Cohesion, c', kPa (psi)	***	***	***	***

The expansive behavior of a particular soil strata in the field is primarily affected by: (1) mineralogical characteristics and swell potential of the soil; and (2) moisture access conditions (governed by drainage and other geographic characteristics). Exact location of the moisture access conditions in the field is prohibitively difficult to identify. Therefore, a few simplified assumptions were made regarding the moisture boundary conditions for the expansive soil layer. In this model, the entire subgrade layer was modeled as expansive, but moisture access was limited to a certain pre-defined location. This leads to a localized increase in moisture content which ultimately distributes within the subgrade layer based on the laboratory-established permeability and suction properties (Chittoori et al. 2016 a & b; Islam 2017). The resulting volume change in the subgrade layer causes differential heaving on the pavement surface. The moisture access was limited to a 152 cm x 152 cm (5 ft. x 5 ft.) region at the interface between the subgrade and the base (see Figure 3-8). Note that details on how this dimension for the moisture source was established has been presented elsewhere(Chittoori et al. 2016 a & b; Islam 2017); the primary objective was to generate a model with surface heaving patterns similar to what was observed in the field.

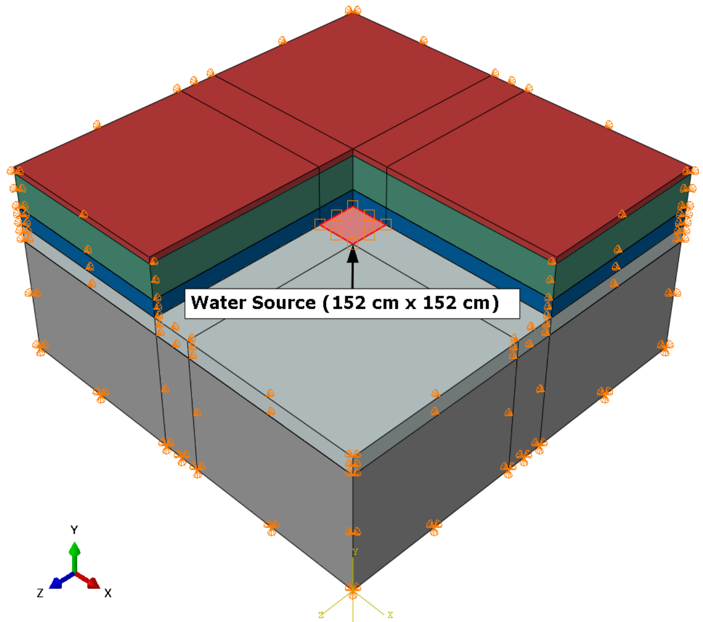


Figure 0-8: Snapshot of the ABAQUS Model showing the Location and Dimension of the Water Source

Initial saturation conditions, as well as soil-water characteristic curves for the expansive soil were input into the model based on laboratory test results. Some of the soil parameters required to model moisture flow through the expansive soil deposits are: (1) initial void ratio (e_0), (2) initial pore water pressure (U_0), and (3) initial saturation level (S_0). More details on laboratory testing carried out to establish these properties can be found elsewhere (Chittoori et al. 2016 a & b; Islam 2017).

Model Geometry Optimization

The thickness and properties of other layers present in the pavement structure can significantly affect the differential heave. The mechanism of volume change experienced by the expansive soil layer can be affected by the geologic formation surrounding it. A soil layer that has sufficient room to expand will not cause significant damage to the surrounding layers. However, moisture content change in a tightly confined expansive soil layer can exert very high pressures on the adjacent layers. This is an important aspect to consider while deciding on the model geometry to simulate pavement surface heaving due to volume changes in the underlying expansive soil layer. During the modeling effort it was observed that relative location of the fixed boundaries with respect to the water source had a significant effect on the heave observed on the pavement surface. A sensitivity study was first carried out to establish the dimensions of the model to closely simulate heaving patterns observed in the field. Two types of dimensional optimization studies were carried out: (1) to establish the optimal vertical (Y) dimension; and (2) to establish the optimal horizontal (X and Z) directions. Figure 3-9, presents results from this geometry optimization effort. The scatter plot in Figure 3-9, presents results from the vertical (Y) dimension optimization, whereas the bar charts present results from the horizontal (X and Z) dimension optimization. As seen from the figure, gradually increasing the subgrade layer thickness from 3 m to 20 m had a significant effect on the predicted surface displacement magnitudes. However, increasing the subgrade layer thickness beyond 20 m did not have as significant an impact on the predicted surface displacements. Similarly, increasing horizontal dimensions from 20 m to 30 m resulted in a reduction in the predicted surface displacement at the center of the model from 10.5 cm to 9.0 cm. Once the model dimension approached 50 m, the predicted surface displacement magnitudes stabilized. Based on these results, the horizontal dimensions of the model were fixed at 50 m x 50 m. The depth of the subgrade was fixed at 20 m.

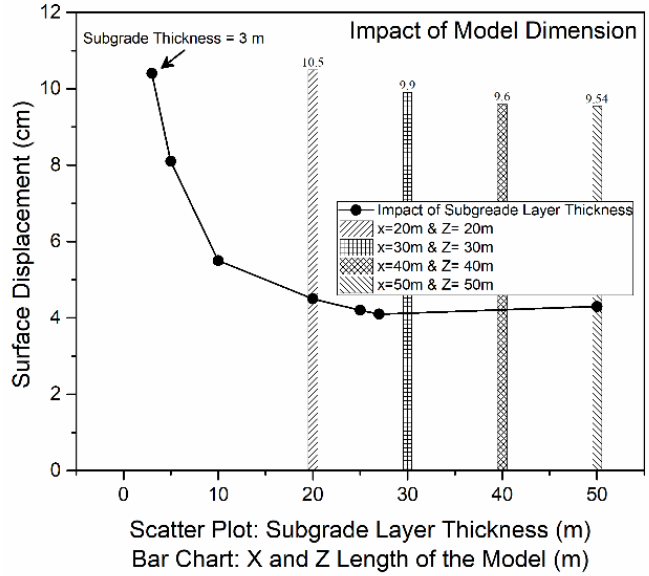


Figure 0-9: Effect of Model Dimension on Predicted Maximum Surface Displacement

Element Selection and Mesh Optimization

The use of appropriate element type and mesh size is integral to ensure accurate predictions from FE analyses. Element types used in this study to model different pavement layers were selected based on the material properties being modeled. The HMA and base layers were modeled using 8-noded, linear, hexahedral, reduced integration elements with hourglass control (C3D8R in ABAQUS) (ABAQUS 2016). The expansive soil, on the other hand, was modeled using 8-noded brick elements with trilinear displacement and trilinear pore pressure (C3D8RP in ABAQUS). Note that the C3D8RP element has the ability to simulate fluid flow through partially- or fully-saturated porous media (ABAQUS 2016).

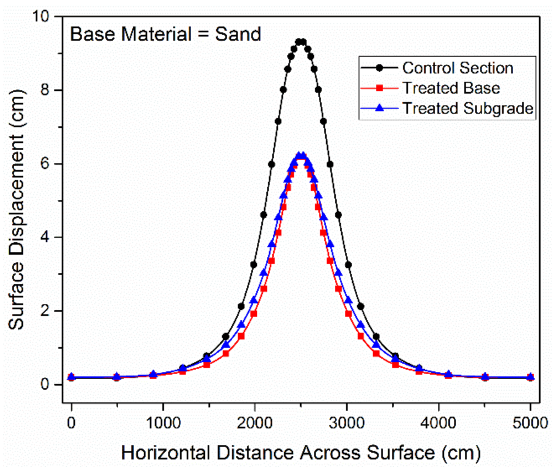
In this modeling effort, the size of the FE mesh was optimized after several trial analyses. Mesh size optimization was carried out based on predicted surface heave magnitudes. Note that a biased-meshing approach was used to reduce the computational time requirements. Each simulation with the fine mesh required approximately 48-65 hours to run on a standard desktop computer with 20 GB RAM and a 3.6 GHz Intel® Xeon® processor. Using the biased-mesh reduced this computational time requirement to approximately 2 hours. A comparative study was undertaken to compare the model-predicted results between a fine and a coarse mesh. A maximum difference of 5% was obtained when the surface heaving magnitudes were compared. It was therefore concluded that using a biased-mesh will not significantly affect the overall findings from this research study. All results reported in this manuscript correspond to models with biased-meshes.

Effect of Polyurethane Grout Injection on Pavement Surface Heave

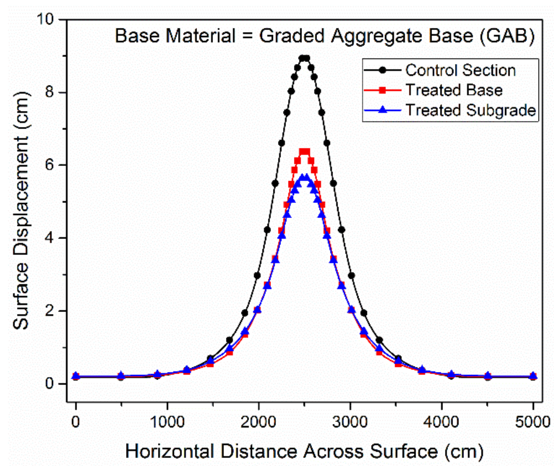
Results from FE modeling of pavement sections comprising HDP-treated base/subgrade layers are presented in this section. Note that different analyses were carried out to study the effect of HDP injection into the base layer or the subgrade layer. In either case, it was assumed that strategic placement of the injection ports will result in a 61-cm (2-ft.) thick composite layer generated by mixing of grout and soil/aggregate. Note that this assumption is reasonable for cases where the HDP is injected into the base layer (high permeability of the base layer ensures uniform permeation of the grout). However, in cases where the HDP is injected into the subgrade layer, the assumption of uniform permeation to generate a 61-cm (2-ft.) thick homogeneous layer is not very realistic. Nevertheless, this simplifying assumption was made to simulate the effect of a localized increase in subgrade modulus on the differential heave observed on this surface.

Results from the modeling effort are presented in Figure 3-10. Note that the three subfigures in Figure 3-10 correspond to pavement sections where sand, GAB, or #57 Stone was used in the base layer. The ‘control section’ represents the pavement section where neither the base nor the subgrade were treated using the Polymer. The ‘Treated Base’ model corresponds to the case where a 61-cm thick layer of HDP-treated base was placed on top of the untreated subgrade. The ‘Treated Subgrade’ model corresponds to the case where the top 61-cm of the subgrade was treated using HDP (the base layer was assigned untreated material properties).

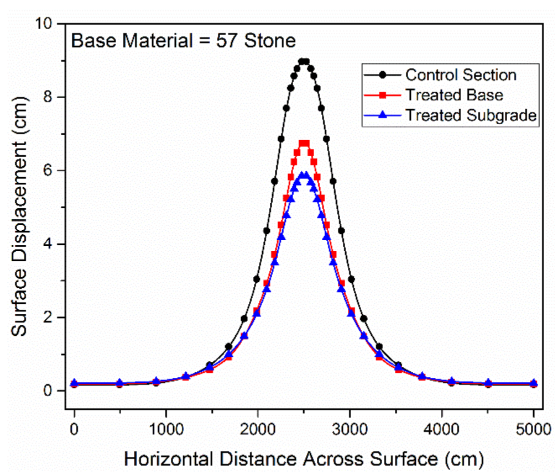
Each graph in Figure 3-10 shows the surface profile across the model geometry. From the figure, it can be seen that the polyurethane grout injection (either into the base or the subgrade layer) results in significant reduction in the surface heave in all cases. For pavement sections comprising natural sand in the base layer, HDP injection into the base or subgrade layer resulted in similar reductions in the surface heave (see Figure 3-10-a); a 34% reduction in the surface heave compared to the control section was observed. On the other hand, for models comprising GAB or #57 Stone in the base layer, the greater benefit of the HDP injection was observed for models comprising treated subgrade layers compared to treated base layers. Here it is necessary to mention that although no significant improvement of elastic modulus is observed in laboratory testing efforts for the case of GAB or #57 Stone, considerable reduction of heave is observed in numerical analysis because injection of the HDP transforms the layer from an elasto-plastic behavior to a linear elastic behavior. Referring back to Figure 3-6, a significant increase in the slope of the stress-strain curve was observed for all materials upon HDP injection. Injection of the HDP results in a composite layer with significantly higher bending stiffness, which in turn reduces the magnitude of surface heave. Comparing the results presented in Figure 3-10, it can be concluded that HDP injection into the base/subgrade layer has a potential to significantly reduce surface heaves in flexible pavement sections constructed over expansive soil deposits. Table 3-3 lists the predicted heave magnitudes for each of the models, and the percent reduction in heave achieved through HDP injection.



(a). Sand



(b). GAB



(c) #57 Stone

Figure 0-10: Deformed Surface Profiles Predicted by Numerical Modeling for Pavement Sections Comprising (a) Sand; (b) GAB; and (c) #57 Stone in the Base Layer

Table 0-3: Comparing the Model-Predicted Nodal Displacements for Pavement Sections with Treated and Untreated Base and Subgrade Layers

	Base Material: Sand			Base Material: GAB			Base Material: #57 Stone		
	Control	Treated-Base	Treated-Subgrade	Control	Treated-Base	Treated-Subgrade	Control	Treated-Base	Treated-Subgrade
	Disp. (cm)	Disp. (cm)	Disp. (cm)	Disp. (cm)	Disp. (cm)	Disp. (cm)	Disp. (cm)	Disp. (cm)	Disp. (cm)
Max	9.31	6.19	6.21	8.94	6.37	5.65	8.98	6.75	5.86
Min.	0.18	0.19	0.20	0.17	0.20	0.22	0.17	0.19	0.21
Heave	9.14	5.99	6.01	8.76	6.17	5.43	8.81	6.56	5.65
DHR	***	34	34	***	30	38	***	25	36

*DHR: Differential Heave Reduction (%)

Limitations of Current Study

Simplifications and assumptions made during the laboratory testing and modeling stages of the current study can be related to certain associated limitations: (1) HDP injection in the laboratory was carried out in a PVC tube, which can lead to significant confining pressures during expansion of the polymer. Such confining pressure levels may not be attained during field injection; (2) the water source in the model was defined at one particular location, and was assigned a fixed dimension. This is most likely different from actual field conditions where moisture flow into the pavement substructure can occur at multiple locations; (3) the HDP-treated layers were assumed to be homogeneous in nature and 61-cm thick. Although these numbers may not be very realistic for field conditions (especially when HDP is injected into the subgrade layer), the purpose is to highlight how the increased modulus and change in stress-strain behavior of the HDP-injected geomaterial can lead to significantly reduced heaves on the pavement surface. More accurate modeling of the homogeneous nature of HDP-injected layers can be possible only if large-scale box tests are conducted, and the spatial variation of aggregate/soil and HDP mixing is quantified. Nevertheless, exact quantification of this spatial variation is impossible in actual field applications.

Summary and Conclusions

This manuscript presented findings from a recent research effort at Boise State University that evaluated the effectiveness of polyurethane grout injection as a potential remedial measure to reduce the differential heaving in flexible pavement sections constructed over expansive soil deposits. Three different base material types and one expansive soil were characterized in the laboratory under both untreated as well as treated conditions to establish the resilient modulus and shear strength properties. Significant increase in the resilient modulus properties were observed for the natural sand and expansive soil materials. However, all four materials exhibited significantly higher shear strength properties upon treatment with the High-Density Polyurethane (HDP). Due to higher permeability of the base materials, greater degree of grout permeation was achieved during base treatment compared to subgrade treatment. HDP injection resulted in significant densification of the expansive soil specimen. Results from Finite Element (FE) modeling of flexible pavement sections constructed over expansive subgrades indicated significantly reduced surface heaves for models comprising HDP-treated base/subgrade layers. Findings from this study indicate that polyurethane grout injection can be an effective approach to reduce surface heaving in flexible pavement sections constructed over expansive soil deposits.

Acknowledgments

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SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

Summary

This thesis summarized two specific problems and their corresponding nondestructive, time-efficient solutions. The first problem addressed in manuscript # 1 (Chapter 2 of the thesis) was associated with pavement structural evaluation on a network level in the absence of pavement layer thickness data. In this manuscript, the feasibility of using Deflection Basin Parameters (DBPs) as a quick analysis approach (independent of layer thickness information) was first investigated using the finite element method. This approach was subsequently applied to evaluate the current conditions of four different roadway segments selected across the state of Idaho. This analysis effort concluded that for typical pavement configurations, some of the inferences, regarding the structural conditions of individual pavement layers, drawn using DBPs, are as reliable as results from rigorous back-calculation efforts.

The second problem addressed in manuscript # 2 (Chapter 3 of the thesis) was associated with the need to mitigate recurrent differential heaving problem in flexible pavement sections constructed over expansive soil deposits. In this manuscript, the feasibility of High-Density Polymer (HDP) grout injection as an alternative nondestructive solution to mitigate the differential heaving problem was investigated. For this purpose, both laboratory testing and numerical simulations were carried out. Findings from this research indicated that that adequate permeation of HDP grout into particular soil layer can significantly change its behavior. Results from the numerical modeling efforts confirmed that the level of confinement around the expansive soil layer can significantly change its behavior. Findings from this study indicated that HDP grout injection in flexible pavement sections constructed over expansive soils can potentially be used as an alternative nondestructive approach to mitigate the problem of recurrent differential heaving. However, the success of this approach is largely dependent on the extent of permeation of the grout into the layer being treated.

Conclusions & Limitations

Manuscript # 1

Following conclusions were drawn based on the research reported in Chapter 2 of this thesis.

1. DBPs are can be used as reliable alternatives to extensive back-calculation of layer moduli form FWD data, and can be particularly useful for network-level analysis of pavement structural conditions.
2. DBPs can be used to make relatively accurate assessments of the structural condition of pavement layers; the results are significantly more reliable for base or subgrade layers, compared to surface layers.
3. As DBPs are highly depended on pavement temperature and applied load levels, the threshold values may need to be adjusted when the temperature and loading conditions are different from usual operating conditions.
4. DBPs can be used along with functional evaluation data to make more “informed” decisions during the selection of pavement rehabilitation methods.

Manuscript # 2

Following conclusions were drawn based on the research reported in Chapter 3 of this thesis.

1. HDP grout injection could be used as an alternative nondestructive approach to mitigate differential heaving in pavements constructed over expansive soils.
2. HDP grout treatment can significantly increase in the resilient modulus values for natural sand and expansive clayey soils.
3. HDP grout injection can significantly improve the shear strength of soils and aggregates.
4. Permeability of the material being treated is the single most important factor governing the effectiveness of HDP grout as a treatment option.

Recommendations for Future Research

Manuscript # 1

1. The numerical model used in the current study to validate the DBP approach was static in nature. FWD testing on the other hand, is a dynamic testing. Accordingly, consideration of dynamic properties of individual pavement layers can improve the reliability and accuracy of the model;
2. The current study did not consider visco-elastic nature of the HMA layer, or the stress-dependent modulus of soils and aggregates. Consideration of these aspects will ensure more "realistic" simulation of actual pavement response under loading.

Manuscript # 2

1. HDP injection in the laboratory was carried out in a PVC tube, which can lead to significant confining pressures during expansion of the polymer. Such confining pressure levels may not be attained during field injection;
2. The water source in the model was defined at one particular location, and was assigned a fixed dimension. This is most likely different from actual field conditions where moisture flow into the pavement substructure can occur at multiple locations;
3. The HDP-treated layers were assumed to be homogeneous in nature, and 61-cm thick. Although these numbers may not be very realistic for field conditions (especially when HDP is injected into the subgrade layer), the purpose was to highlight how the increased modulus and change in stress-strain behavior of the HDP-injected geometrical can lead to significantly reduced heaves on the pavement surface.
4. More accurate modeling of the HDP-injected layers can be possible only if large-scale box tests are conducted, and the spatial variation of aggregate/soil and HDP mixing is quantified. Nevertheless, exact quantification of this spatial variation is impossible in actual field applications.