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Future Design of Perpetual Pavements for New Mexico

Prepared by:
University of New Mexico
Albuquerque, NM87131

Prepared for:
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14. Abstract

This study provides guidance for future designs of perpetual pavements in New Mexico. The perpetual pavement in this study refers to only hot mix asphalt (HMA) pavement sections designed for a useful life of 50 years or more without having major distresses such as fatigue and rutting. This project conducts an in-depth literature search of state Departments of Transportation and foreign agencies pertaining to design and application of perpetual pavements. This report highlights the perpetual pavements designed by state highway agencies nationwide and summarizes their experience, as well as their conclusions on perpetual pavement performance. Mechanistic Empirical Pavement Design Guide (MEPDG) is used as analysis tool. This study evaluates the effects of moisture infiltration on perpetual pavements. A full literature review conducted on moisture damage testing indicates that dynamic modulus testing of wet and dry hot mix asphalt samples is found to be an appropriate approach to account for moisture damage in perpetual pavements. This study determines the combination of layer, stiffness, and thickness to produce optimal perpetual pavements. This is achieved by creating a test matrix of varying MEPDG input parameters. Input parameters such as HMA layer thickness, HMA mix design, and performance grade (PG) binders are varied and analyzed using the MEPDG. From the trial designs, perpetual pavements have been found for moderate to high truck traffic using HMA thicknesses varying from 10 to 15 inches. Perpetual pavements have been found both with and without rich binder layers (RBLs). Results shown in this study indicate that fatigue cracking is not a major concern for designing perpetual pavements in New Mexico's conditions (using MEPDG), rather rutting is more of a concern.

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Future Design of Perpetual Pavements for New Mexico

Final Report

BY
Rafiqul A. Tarefder, Ph.D., P.E.
Assistant Professor of Civil Engineering
University of New Mexico

Damien Bateman
Graduate Research Assistant
University of New Mexico

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NMDOT Research Bureau
7500B Pan American Freeway NE
PO Box 94690
Albuquerque, NM 87199-4690
(505) 841-9145

PREFACE

The research reported herein gathers data related to the use of perpetual pavements as a viable alternative for designing pavements in New Mexico. This project conducts an in-depth literature search of state Departments of Transportation and foreign agencies pertaining to design and application of perpetual pavements. This project evaluates New Mexico's US 70 Hondo Valley perpetual pavement using the mechanistic empirical pavement design guide. Perpetual pavements discovered and presented in this study are considered implementable on New Mexico State highways. They range from 10–15 inches thickness and cater for moderate to high truck traffic.

NOTICE

DISCLAIMER

ABSTRACT

This study provides guidance for future designs of perpetual pavements in New Mexico. Therefore, perpetual pavements presented in this study are to be considered for implementation on New Mexico State highways. These perpetual pavement structures include a strong and flexible hot mix asphalt (HMA) base layer to reduce potential fatigue cracking and a rut resistant intermediate HMA layer. An in-depth literature review was conducted, and state Departments of Transportation and foreign agencies were contacted pertaining to design and application of perpetual pavements. This report summarizes their experience, as well as their conclusions on perpetual pavement performance. The information gathered from the literature review indicates that 19 state DOTs have tested or implemented perpetual pavements on their highways and that 37 perpetual pavements are in operation in the United States today. Many of the perpetual pavements are performing as expected. However, some of them have shown to have moisture-related problems with de-bonding a major issue due to high permeability and moisture damage. The thickness of perpetual pavements can be as high as 17 inches (HMA). The traffic volume on perpetual pavements varies from 1,500 to 20,000 annual average daily truck traffic (AADTT). Five sections of New Mexico's US 70 Hondo Valley perpetual pavement are also reviewed using the Mechanistic Empirical Pavement Design Guide (MEPDG).

This report evaluates the effects of moisture infiltration on perpetual pavements. A full literature review conducted on moisture damage testing shows that the Environmental Conditioning System (ECS) is probably the most advanced testing available. However, the majority of state DOTs still use the American Association of State Highway and Transportation Officials (AASHTO) T-283 test. Some laboratory testing was undertaken by the research team on asphalt mixes used by NMDOT to determine their moisture characteristics. However, this area of research is very challenging and not within the scope of this project. From the literature, dynamic modulus testing of wet and dry HMA samples was also found to be appropriate method in accounting for moisture damage in perpetual pavements. Dynamic modulus data can be used in MEPDG which makes this approach very appealing. At present, there is no integrated system that accounts for the effect of moisture infiltration on HMA in the MEPDG.

One of the main goals of this study was to determine the combination of layer, stiffness, and thickness to produce an optimal perpetual pavement. Using a test matrix of varying MEPDG level 3 input parameters, an optimal perpetual pavement was discovered. Input parameters such as HMA layer thickness, HMA mix design, and performance grade (PG) binders were varied and the resulting trial designs were analyzed using MEPDG. From the trial designs, perpetual pavements have been found for moderate to high truck traffic using 10 – 15 inches HMA. Perpetual pavements have been found both with and without rich binder layers

(RBLs). RBLs in perpetual pavements have been known to cause moisture-related problems due to their high density. This study recommends using a perpetual pavement that does not have a RBL. One example is a pavement that has a 3 in. surface layer containing a fine HMA mix, and a 7 in. intermediate layer that uses a coarse HMA mix. This perpetual pavement carries up to 180 million equivalent single axle loads (ESALs) over its 50 year design life. Very low bottom-up fatigue cracking (< 12%) as well as little or no top-down cracking (< 0.2 ft/mi) was observed at the end of 50 years. Rutting in the intermediate layer was also low (< 0.05 in.) at the end of the 10-year rehabilitation cycle. At the beginning of the next 10 year cycle, rutting and IRI in the surface course are set to zero and the pavement is considered rehabilitated. Based on this study, it is shown that fatigue cracking is not a major concern for designing perpetual pavements in New Mexico's conditions (using MEPDG), rather rutting is more of a concern.

Another key factor that was investigated and presented in this report was de-bonding of HMA layers. Infiltration and accumulation of moisture in the interface of these layers is the main cause and this can reduce the design life of the perpetual pavements significantly. MEPDG level 3 analysis shows that 88% of the perpetual pavements discovered in this study will fail by top-down cracking if de-bonding occurs between two HMA layers. Bottom-up cracking also increases significantly in a de-bonded environment. Analysis of de-bonding of perpetual pavements was also verified using KENLAYER, which is a multi-layer elastic analysis software.

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Chapter 1

INTRODUCTION

1.1 Background and Significance

Perpetual pavement in this study is defined as an asphalt pavement designed and built to last 50 years or more without requiring major structural rehabilitation or reconstruction. With perpetual pavements, the potential for traditional fatigue cracking is reduced, and pavement distress is typically confined to the upper layer of the structure. Thus, when surface distress reaches a critical level, an economical solution is to remove and replace the top layer. The perpetual pavement concept can be used for any pavement structure where it is desirable to minimize rehabilitation and reconstruction costs as well as minimize closures to traffic. These considerations are especially important on high-traffic volume freeways where user delay costs may be prohibitive. In particular, in urban areas where new roads are being built, use of perpetual pavements may minimize future costs due to user delays and construction. Perpetual asphalt pavement is a very appealing alternative to concrete pavements, especially for large metropolitan areas.

Traditionally, asphalt pavements have been designed for a 20-year life, whereas perpetual pavements are expected to perform for 50 years or more. While there are some successes with perpetual pavements, there is a big gap in our understanding the design of this pavement. The main deficiency with the current perpetual pavement design method is that it does not ensure optimum structure and/or layers that have yet to satisfy 50 year design periods. However, through a sound pavement design methodology, it is possible to obtain optimal asphalt pavement structures that will last 50 years or more requiring only periodic top surface replacement. Such design methodology should include mechanistic pavement design, materials selection/innovation to improve durability and fatigue resistance, prediction of field performance, analysis of remaining service life and life-cycle cost. To this end, this study determines the combination of layer, stiffness, and thickness to produce optimal perpetual pavements. In particular, this study analyzes various alternatives of perpetual pavement structure through varying the thickness and stiffness of pavement layers. These analyses use the mechanistic-empirical design approach, and include an evaluation of the life cycle costs of the resulting alternatives.

The effects of moisture within the pavement section and interlayer de-bonding can be principal factors in reduction of pavement performance over time, yet both effects are only accounted for in a relatively crude manner. Thus, there was a need to examine the effects of moisture infiltration and bonding/de-bonding on the design life of perpetual pavements. To this end, this study considered the feasibility of removing one or more layers of a perpetual pavement design and evaluated the effects of de-bonding due to moisture infiltration.

1.2 Structure of Perpetual Pavements

Perpetual pavement has been around since 1960. For example, two sections of Interstate 40 in downtown Oklahoma City are now more than 33 years old (built in 1967) and are still in excellent condition. These sections, which support 3 to 3.5 million ESALs per year, have been overlaid but the base and intermediate courses have lasted since construction without any additional work. While there are some successes with perpetual pavements, there is a big gap in our understanding the design of this pavement.

In the literature, several definitions of perpetual pavements can be found (Newcomb et al. 2001; Haas et al. 2006). A perpetual pavement has been closely connected to thick asphalt pavements comprising a three-layered asphalt pavement including a wear-resistant and renewable top layer, a rut-resistant and durable intermediate layer, and a fatigue-resistant and durable base layer. Figure 1.1 is a typical structure of a perpetual pavement consisting of three HMA layers built on a stabilized subgrade or foundation. These layers are described below:

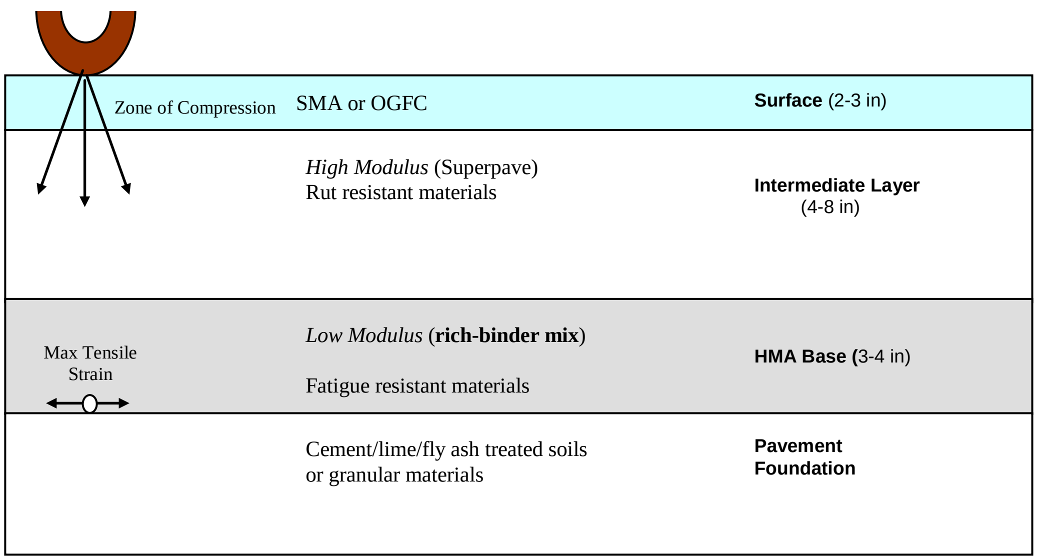


Figure 1.1 Schematic of a Perpetual Pavement

<u>Surface Layer</u>: The top surface layer is a renewable surface that can be designed for specific applications. The choice of the surface layer depends on the functional requirements. These

could be a combination of comfort, durability, stability, skid resistance and noise reduction. There may be additional requirements like surface water drainage or very low water impermeability. A wide range of bituminous surface layer products can be considered appropriate depending on specific requirements. In some instances, the use of a conventional dense-graded Superpave mixture is adequate. In very high-traffic areas, the use of Stone Matrix Asphalt (SMA) may be attractive, provided that the materials are available to construct it. In some places, engineers may want to use an Open Graded Friction Course (OGFC) on the surface, to reduce splash and spray and to provide better skid resistance during rainstorms. Both OGFC and SMA have the advantage of reducing road noise due to tire-pavement friction.

Intermediate Layer: The intermediate layer is designed specifically to carry most of the traffic load. Therefore, it must be rut-resistant and durable. Rut resistance can best be provided by using stone-on-stone contact in the coarse aggregate and using binders having appropriate high-temperature properties. The intermediate layer can consist of one or two layers of Superpave mixtures. Superpave mixture is a dense and/or gap graded bituminous mixture in which the aggregate and performance grade (PG) binders are major contributors to the rut resistance behavior of HMA.

HMA Base layer: This HMA base layer is designed specifically to resist fatigue cracking. Two approaches can be used to resist fatigue cracking in the base layer. First, the total pavement thickness can be made great enough such that the tensile strain at the bottom of the base layer is insignificant. Alternatively, the HMA base layer could be made using an extra-flexible HMA. This can be most easily accomplished by increasing the asphalt content, that is, rich–binder mix (RBM) layer. Recently, the need for the rich bottom fatigue layer has been questioned especially when the total HMA thickness is greater than 12 inches.

Pavement Foundation: The perpetual pavement structure is usually built on a solid foundation layer which may consist of an untreated base course and subgrade. Usually, the foundation soil is improved by treatment with lime, cement or fly ash. To date, no study has investigated whether the treated layer may not provide the permanent support needed for the 50-year design life of perpetual pavements. In this study, the influence of foundation stiffness on the performance of perpetual pavement was evaluated.

1.3 Analysis Tool

In this study, the Mechanistic-Empirical Guide for Design (MEPDG) Version 1.0 was used as a perpetual pavement evaluation tool. MEPDG is a uniform and comprehensive set of procedures for the design and analysis of new and/or rehabilitated pavements. The MEPDG is based on mechanistic-empirical principles, where it assumes that pavement can be modeled as a multi-layered elastic structure. The mechanistic characterization of paving materials allows for the application of the principles of engineering mechanics, namely stress and strain, to the pavement analysis. Being able to input different material characteristics in the design model allows the pavement engineer to predict the performance of the pavement, improved procedures to evaluate premature failures, and greatly aid in pavement forensic investigation. MEPDG also considers the effects of temperature and moisture on a project basis using site-specific environmental data from nearby weather stations.

While the mechanistic approach to pavement design and analysis is much more rational than the empirical approach, it also is much more technically demanding. However, there are some specific advantages of MEPDG design over traditional empirical procedures, including consideration of changing load types, better utilization and characterization of available materials, improved performance predictions, better definition of the role of construction by identifying parameters that have the most influence over pavement performance, relation of material properties to actual pavement performance, better definition of existing pavement layer properties, and accommodation of environmental and aging effects on materials. These advances in the analytical approach over the traditional approaches to pavement design make it very attractive to this study to utilize the MEPDG.

The interactions between geometrics, material properties, traffic, and environmental conditions in the MEPDG are illustrated in Figure 1.2. The layer thicknesses were obtained through an iterative process in which predicted performance is compared against the design criteria for the multiple predicted distresses until all design criteria are satisfied to the specified reliability level. There are three levels of inputs in the MEPDG analyses. In level 1, materials properties such as dynamic modulus of asphalt concrete and resilient modulus of soils and aggregate are obtained from laboratory tests. In level 2, these properties are determined using locally calibrated correlation equations. In level 3, the dynamic and resilient modulus are calculated from index properties such as soil classification, plasticity, aggregate gradation, binder content, etc using the existing national correlation equation. In this study, level 3 inputs were used to determine optimal structure.

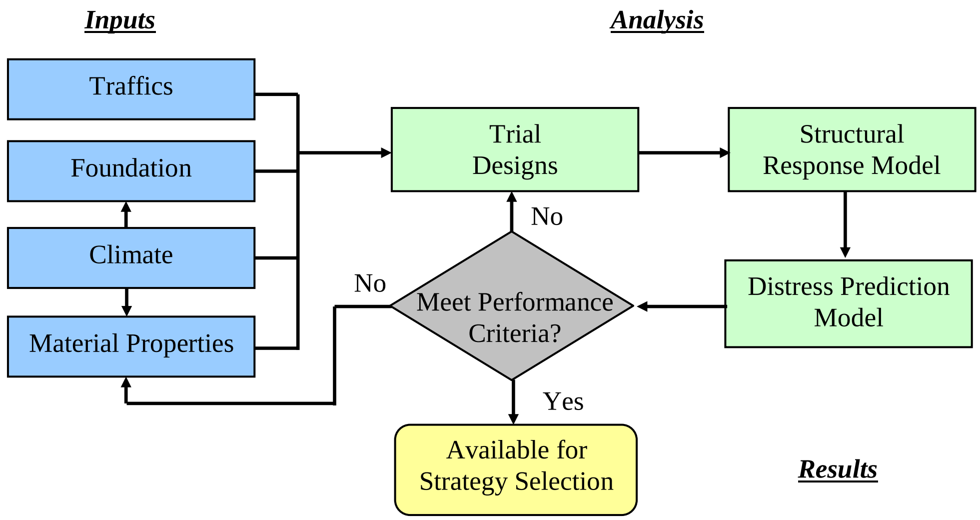
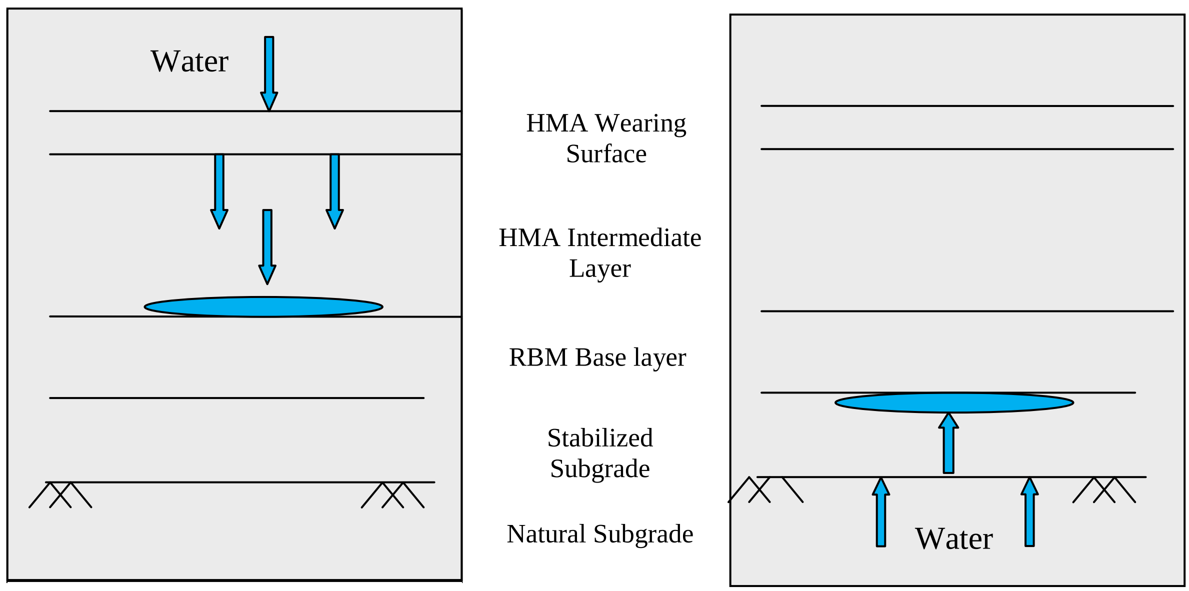


Figure 1.2 Mechanistic Empirical Pavement Design Guide

1.4 Moisture Infiltration in Perpetual Pavement

Water within pavement layers is one of the causes of pavement deterioration. Specific problems associated with water include: stripping of asphalt pavement; reduction in pavement strength due to asphalt-aggregate bond damage or weakening; shrinking and swelling of sub-grade materials due to water content changes; and frost heave and thaw weakening due to upward (capillary) flow beneath pavements. Water-related problems are thus responsible for decreased pavement life, and increased costs for maintenance, and occur throughout all regions and climates of the US. A National Cooperative Highway Research Program study estimated that excess water reduces the life expectancy of pavement systems by more than half (Christopher and McGuffey, 1997). New Mexico is certainly not immune to water related problems: for example, the pavement of US 70 is known to have moisture wicking problem.

Figure 1.3 illustrates possible cases of moisture infiltration in perpetual pavements. Figure 1.3(a) shows downward moisture infiltration through the top of the pavement section. As moisture seeps down through the porous surface and intermediate layers, it encounters the rich-binder mix (RBM) base layer. The RBM layer has low permeability as it contains higher percentages of asphalt binder compared to those in traditional Superpave mixes. As a result, moisture is prevented from passing through the RBM base layer due to restricted flow paths. This scenario may lead to the collection of moisture on top of the RBM base layer. The additional water in the pavement structure may lead to a reduction of the stiffness of intermediate and wearing surface layer. There may several other problems such as bond damage in the asphalt-aggregate, softening of asphalt binder, and erosion of fines. Figure 1.3(b) shows upward infiltration that begins at the bottom of the pavement section. This type of moisture infiltration may occur due to high groundwater table and capillary rise of water in the natural subgrade. This infiltration can also lead to problems in the structural performance due to loss of stabilized subgrade strength as well as interface de-bonding. This type of infiltration may lead to permanent damage of the perpetual pavement structure.



(a) Top-down Infiltration (b) Bottom-up Infiltration

Figure 1.3 Possible Moisture Infiltrations in Perpetual Pavement

Water movement and accumulation within pavement sections is difficult to predict for a number of reasons, including three-dimensional permeability, partially saturated flow, complex boundary conditions due to precipitation, evaporation, and non-uniform distribution of moisture due to layered system. Numerical simulation of moisture movement within a pavement section requires hydraulic properties of HMA materials, which is beyond the scope of this study.

1.5 LCAA Analysis

The Life Cycle Cost Analysis (LCCA) provides a sound basis for economically evaluating a number of feasible pavement design alternatives to identify one that may be the most cost-effective to build and maintain. The candidate costs that can be considered in a LCC analysis are:

- Initial construction
- Maintenance
- Rehabilitation
- Salvage value

- User delay (during future maintenance or rehabilitation)
- Vehicle operating cost.

The first four are agency costs that typically have the most impact on strategy selection. However, when considered, the last two user costs have been shown to have a major effect on the selection of a strategy that is most cost-effective overall. In this study, LCCA includes expenses to the State DOT, such as construction, operation, and maintenance costs considering the identical costs of user delay and vehicle operation among the perpetual pavement alternatives measured in.

1.6 Bonding and De-bonding

Most of the modern theories and methods for analysis and design of flexible pavements assume that there is a complete bonding between all the layers. In complete bonding conditions, the stresses at the bottom of a layer is entirely transferred to the top of the layer immediately below it, and the displacements at these points will be the same. If continuity conditions are satisfied at the layer interfaces, there will be identical vertical stress, shear stress, vertical displacement and radial displacement in the both layers. For frictionless interface, the continuity of shear stress and radial displacement is replaced by zero shear stress at each side of the interface. These assumptions are made to facilitate the modeling of a layered asphalt system and its solution. However, in reality, these assumptions are not completely satisfied. The materials used for the different layers are different from each other, so the response to loading cannot be the same. Paving materials consist of grains and particles with small voids between their particles; these voids are frictionless and complete bonding between the layers cannot be achieved.

Recent studies (i.e., TxDOT, Illinois, Kentucky, California, UK) indicate that some layers in perpetual pavements are not effectively bonded together. This is a major structural concern. All of the mechanistic design procedures consider the asphalt layers as a composite beam while experiencing tensile strains induced by traffic loads. Having de-bonded layers within the HMA structure defeats the purpose of the RBM as the fatigue cracking may initiate at the de-bonded interface. The more de-bonding occurs in the pavement structure, the more severe the consequence will be on the pavement’s fatigue life.

In addition, the insufficient bonding between a surface course and its underlying layer can causes slippage cracking. This occurs most often in areas where braking or turning wheels cause the pavement surface to slide or deform (e.g., intersection, sharp curves), but can also occur under a simple rolling wheel load. If an intermediate HMA layer is fully bonded to the base but loses the bonding with the wearing course, the location of critical strain, where the cracks are more likely to initiate, is at the bottom of the wearing course. Unless a new series of high friction materials is developed and used for pavements, the design of pavements based on completely bonding conditions could be unreliable in the future. With the ever increasing traffic volumes, traffic speed and tire pressures, a new model will be required to address the load carrying capacity of the pavements to carry new loading conditions. In this study, attempts were made to quantify pavement performance based on bonding and de-bonding conditions.

Chapter 2

RESEARCH OBJECTIVES AND TASKS

2.1 Research Objectives

The main goal of this research was to provide guidance for future designs of perpetual pavements in the state of New Mexico. The specific objectives are to:

1. Assess current design, testing, and evaluation methods of perpetual pavements, with particular emphasis on their applicability to the design of perpetual pavements with a design life 50 years or more in conditions typical of New Mexico pavements, materials, and environments.
2. Evaluate the effects of top-down and bottom-up moisture infiltration on the current perpetual pavement performance considering reduction in stiffness and strength properties of HMA and foundation layers.
3. Determine the optimal perpetual pavement structure using the mechanistic-empirical design approach to develop and evaluate design alternatives based on pavement stiffness and thickness.
4. Quantify the impact of removing layers, and considering various degrees of bonding within a perpetual pavement section.
5. Document the literature, analysis, findings, and recommendations for perpetual pavement method and guidelines for consideration to be incorporated into the current NMDOT perpetual pavement design.

2.2 Research Tasks

Task 1: Review of Current Practices

Subtask 1A: Literature Review

Conduct a full literature review to assemble the current pool of written knowledge about perpetual pavements all over the world.

Subtask 1B: State DOT and FHWA Review

Review state DOTs, FHWA and AASHTO studies involving design, construction, testing, recent specifications, design practices, and typical sections of perpetual pavements.

Subtask 1C: New Mexico US-70 Hondo Valley Project Review

Collect and analyze the available design, construction, and performance data of US-70 pavement (Hondo Valley project), which was designed and constructed using the perpetual pavement concept.

Subtask 1D: Preliminary Analysis

Analyze the data collected in the above three subtask so as to facilitate MEPDG input data such as pavement layer and materials, traffic load and class, and local weather information. In particular, this subtask is to determine the high, low, and mean values thickness of all perpetual pavement.

Task 2: Evaluate Effects of Moisture Infiltration on Perpetual Pavements

Subtask 2A: Evaluate Top-down Moisture Infiltration

The top-down moisture infiltration is attributed to causing moisture damage of HMA layers due to loss of the cohesive bond within the asphalt binder and/or the loss of the adhesive bond between the aggregate and binder. Thus, the top-down infiltration can be accounted for a reduction in the HMA layer moduli (stiffness) and strength. This task is to review the current state-of-the-art equipment, methods, and values of these reduced modulus and strength. Using these reduced moduli as inputs to the mechanistic pavement analysis, the performance of the moisture infiltrated perpetual pavement can be evaluated.

Subtask 2B: Evaluate Bottom-up Moisture Infiltration

The bottom-up moisture on perpetual pavement is viewed as to affect the subgrade soils only. The subgrade soil strength and modulus varies with the seasonal moisture content of soils due from the water table below. This subtask is to consider the seasonal variation of subgrade soil resilient modulus (M_R) in the MEPDG analysis through the use of Integrated Climatic Model (ICM) model and water table, which affects the bottom-up moisture infiltration based on its depth below the subgrade.

Task 3: Evaluate Design Alternatives

Subtask 3A: Develop Design Alternatives Based on Layer Stiffness and Thickness

Determine the optimum pavement structure that gives highest performance (i.e. low rut and low fatigue). Varying a number of input parameters in MEPDG to determine an optimal structure is a daunting task. Therefore, design trials are created based on only stiffness and thickness. For simplicity of the trial matrix, natural soil properties are not varied. The trial designs are considered non-perpetual if the performance values exceed the thresholds limits. If such, changes in the layer thickness are made until the pavement responses are below the threshold values at the end of the 50-year design life.

Subtask 3B: Determine Sensitivity of HMA Field-Mix Variations

Examine how selected volumetric properties of the field-mixture affect the performance of perpetual pavement.

Subtask 3C: Perform Traffic Modeling Appropriate for Perpetual Design

For perpetual pavements in New Mexico, evaluate appropriate AADTT and ESAL values of medium to high traffic volume roads (for example, I-25).

Subtask 3D: Evaluate Design Alternatives Based on Life Cycle Cost Analysis

Perform a Life Cycle Cost Analyses (LCCA) to assess the most economic design among the alternative designs which pass the design life criteria of 50 years or more in the Subtasks 3A.

Task 4: Determine Effects of Layer and Bonding

Subtask 4A: Evaluate Impacts of Removing a Layer of a Perpetual Pavement

Examine whether there is a need for a rich binder mix (RBM) layer in the design of perpetual pavement structure in New Mexico.

Subtask 4B: Determine Effects of Bonding and De-bonding

Examine the behavior of the layers' interface due to non-bonding and bonding environments.

Task 5: Preparation of Research Report

Subtask 5A: Preparation of Quarterly Reports and Presentations

Provide the findings of the study in formal quarterly reports.

Subtask 5B: Preparation of Final Report

Prepare the final comprehensive and summary project reports.

Subtask 5C: Implementation Plan

Compile the thickness and stiffness governing the optimal design of perpetual pavement. Provide pavement engineers with guidelines for the design and selection of perpetual pavements that last 50 years or more.

Chapter 3

REVIEW OF CURRENT PRACTICES

3.0 Introduction

In this chapter the current pool of written knowledge about perpetual pavements is assembled. Results from research conducted here within the US and by foreign agencies, particularly in the United Kingdom, Europe, Asia and Australia regarding design, testing, and evaluation of perpetual pavements is included. In particular, all available information on New Mexico's own perpetual pavement is presented.

3.1 Task 1A. Literature Review through Database Search

A comprehensive literature search was conducted through Transportation Research Information Services (TRIS), Transportation Research Board (TRB), Research in Progress (RIP), UNM library, and interlibrary loans. Through Interlibrary facility of UNM library, it is possible to access to the materials of 10,000 libraries in the United States. In addition, state DOTs and foreign agencies were contacted through email, fax, and phone to collect perpetual pavement data.

3.1.1 Transportation Research Information Services (TRIS)

Kansas - First Findings from the Kansas Perpetual Pavements Experiment, No. 08-1384 (Romanoschi 2008)

The Kansas Department of Transportation constructed four thick flexible pavement structures on a new alignment on highway US-75 near Sabetha, Kansas. These four pavements were instrumented with gages for measuring the strains at the bottom of the asphalt base layers. Seven sessions of pavement response measurements under known vehicle load were performed between July 2005 and October 2007, before and after the pavement sections were opened to traffic. The design value of limiting strain or fatigue endurance limit (FEL) for each of the test sections was 70 microstrain ($\mu\varepsilon$). Another study is currently under consideration by KDOT to measure strain in the field pavements and fatigue endurance limits of HMA mixes. The goal is to compare the measured strain with the fatigue endurance limit.

Illinois - Strain and Pulse Duration Considerations for Extended-Life Hot-Mix Asphalt Pavement Design, No. 08-0135 (Garcia and Thompson 2008)

Paper No. 08-0135 presents the results of a study performed at the University of Illinois test facility that focused on the evaluation of the most critical pavements responses for extended life hot mix asphalt (HMA) pavements design. A very strong relationship between the longitudinal and transverse pulse durations was found. In general, the transverse pulse durations were about three times those in the longitudinal direction. A relationship between longitudinal and transverse tensile strains was also found. The transverse tensile strains were about 1.5 times greater than those in the longitudinal direction.

Wisconsin - Perpetual Pavement Instrumentation for the Marquette Interchange Project, Phase 1 Final Report WHRP 07-11(Hornyak et al. 2007)
Report WHRP 07-11 provides the on-going design, installation, and monitoring of a pavement instrumentation system for the analysis of load-induced stresses and strains within a perpetual hot mix asphalt pavement system. The goal is to estimate the fatigue life of the perpetual HMA pavement and to modify, as necessary, pavement design procedures used within the State of Wisconsin.

UK - Long-Life Surfaces for Busy Roads, Summary Document 9789282101582 (Joint Transport Research Center 2007)
That study discusses new materials such as epoxy asphalt for use in wearing courses. Some laboratory testing was performed to determine the potential of epoxy-asphalt for long service life. The results proved that the epoxy-asphalt outperforms conventional binders in terms of long service life. In the UK, flexible pavement structures are designed to last longer than 40 years with a planned structural overlay at 20 years (Nunn 1997). The structural section for a perpetual pavement includes the use of granular base and subbase layers below a thick HMA layer. The HMA thickness ranged from 200 to 380 mm (8-15 inches) for 1–80 million ESALs, respectively. Studies of the performance of British roads (Nunn and Ferne 2001) show that: (a) pavements having a total asphalt of less than 180 mm are prone to structural rutting and (b) the rutting in thicker HMA is confined to the top of the structure. Therefore, a thickness of less than 200 mm (8 inches) for the HMA paving is not recommended even for lightly trafficked roads that are required to endure for 40 years. Furthermore, additional pavement thickness beyond that required for 80 million ESALs would not provide additional benefit, so the British researchers placed an upper limit on HMA thickness. Highway M-5 is an example of a perpetual pavement in the UK.

China – Perpetual Pavement in China (Yang et al. 2006)
In 2004 the first perpetual pavement experimental road in China was designed. This project in China documents the design and construction of the subgrade layers, the hot mix layers and properties of the aggregates and asphalt binder used. It also documents construction processes used for the different layers of HMA. The Shandong Perpetual Pavement Project was constructed in the summer of 2005 and was opened to traffic in December 2005. This project involved a team of Chinese and U.S. engineers and the Shandong Highway Bureau and the Shandong Transportation Research Institute. Three structural cross-sections were

designed using PerRoad (Yang et al. 2006). The fatigue criteria for the three designs are 70 and 125 με. The conservative design resulted in approximately 20 in. of HMA materials above the lime-stabilized soil while the less conservative design consisted of 15 inches of HMA. Each experimental section featured a bottom HMA layer as a fatigue-resistant dense-graded material with an asphalt content 0.6% above optimum (Yang et al. 2006).

3.1.2 Transportation Research Record (TRB)

Forensic Investigation of De-bonding in Rich Bottom Pavement, No. 2040 (Willis and Timm 2007)

That study describes the behavior of a rich bottom layer when slippage occurs between the asphalt layers. Testing done at the National Center for Asphalt Technology test track evaluated the effectiveness of RBL in controlling fatigue. It was concluded that slippage between layers (a construction issue) may lead to the early failure of the section.

Europe - A Review of Practical Experience throughout Europe on Deterioration in Fully-Flexible and Semi-Rigid Long-Life Pavements, Vol. 7 No. 2 (Merrill et al. 2006)

Article No. 01038332 documents the behavior of fully-flexible and perpetual pavements in a number of European countries. The European Long-Life Pavements Group (ELLPAG) aims to develop coordinated research to help promote the construction of perpetual pavements.

Europe - Long-life Pavements - A European Study by ELLPAG, Vol. 7 No. 2 (Ferne 2006)

ELLPAG is conducting studies to report on the current state of knowledge on long-life pavements in Europe. The goal of that study is to document how to design, build, and maintain European perpetual pavements to give long structural lives.

Ohio – Strongly Recommended: Ohio Decides To Go With Tougher Perpetual Pavement for I-77, Vol. 41 No. 1 (Wilson 2003)

This article in the Ohio Magazine “Roads and Bridges” discusses the design and construction of the perpetual pavement on I-77 in Ohio. The section length is 2.3 miles and is a bid-build project worth $16 million. Construction was completed in 2003. No data is available as yet.

Australia – Asphalt pavements incorporating some perpetual pavement principles are outperforming expectations in Sydney, Australia, according to the Asphalt Pavement Association of Australia (AAPA). One example is Southern Cross Drive, on Sydney’s Orbital Route, which provides the main access to Sydney Airport and its southern suburbs. This full-depth asphalt pavement was constructed in 1969. “It’s a pavement which was virtually maintenance-free for 25 years, before deterioration became apparent and rehabilitation was required,” the AAPA says. AC pavements designed in the state of

Victoria use either rich bottom layers varying from 8-11 inches or cement-treated crushed rock or untreated aggregate bases varying from 2-5 inches (Bushmeyer 2002).

Oregon - Oregon Answers Perpetual Pavement Analysis with a Field Test, Vol. 75 No. 11 (Estes 2005)
A ten-mile segment of Interstate 5 has been completed recently by the Department of Transportation in Oregon. A section of the roadway contains instrumentation to help engineers evaluate performance.

Ohio - Ohio Takes Perpetual Pavement Another Step Forward, Vol. 75 No. 11 (Ursich 2005)
This article describes the state of Ohio’s newest perpetual pavement project, an eight-mile stretch of U.S. 30 in Wayne County. The article discusses the objectives of various research projects set to examine the mechanical properties of the materials used, the collection of environmental and load response data, and the validation of the perpetual pavement design procedures. The findings of such research projects have yet to be published.

3.1.3 Research in Progress (RIP)

Ohio - Monitoring and Modeling of Pavement Response and Performance, Contract/Grant Number: 134287 (Sargand 2011)
“Monitoring and Modeling of Pavement Response and Performance” in Ohio is an ongoing research project in which new perpetual AC pavements in Ohio are being monitored. The project is due to be completed in 2011.

3.2 Task 1B: State DOT and FHWA Review

There have been several state DOTs, FHWA and AASHTO studies involving the design, construction, and testing data of existing perpetual pavements. The UNM research team contacted these agencies to get their most recent specifications, design practices, and typical sections of perpetual pavements. Project specific information collected from these agencies includes: design, testing, and field monitoring methods, the number of pavement layer, thickness, and stiffness, construction details, present and past performance of pavements as a function of time etc. An in-depth study on the NMDOT’s existing guidelines for designing perpetual pavements is also conducted. Finally, the information gathered from this literature review is presented in a spreadsheet so as to allow easy understanding of all relevant perpetual pavement data.

Current State of Practice in the United States

Table 3.1 presents a list of perpetual pavements in the US. The information collected for this literature review indicates that there are 19 state DOTs that have tested or implemented perpetual pavements on their highways. In total, there are 39 perpetual pavement sections listed in Table 3.1. A list of state DOT officials and their contact information is also presented in here.

Table 3.1 – Perpetual Pavements in US States

	State	Existing Perpetual Pavement		Contact	Email	Position
1	Alabama	No Reply		John Lorentson	lorentsonj@dot.state.al.us	Maintenance Engineer
2	Alaska	0		Stephan Saboundjian	steve.saboundjian@alaska.gov	Pavement Engineer
3	Arizona	0		Julie Nodes	jnodes@dot.state.az.us	Pavement Materials Testing Engineer
4	Arkansas	0		Jerry Westerman	Jerry.Westerman@arkansashighways	Materials Engineer
5	California	1	I-710	Carl Monismith	clm@maxwell.berkeley.edu	Director of the Pavement Research Center
6	Colorado	Waiting for Response		Tim Aschenbrener, Jay Goldbaum	Tim.Aschenbrener@dot.state.co.us	Materials Engineer
7	Connecticut	No Reply		James Norman	James.Norman@po.state.ct.us	Acting Engineering Administrator
8	Delaware	No Reply		Barry Benton	bbenton@mail.dot.state.de.us	Delaware DOT Bridge Design
9	Florida	0		Bouzid Choubane	Bouzid.Choubane@dot.state.fl.us	Materials Engineer
10	Georgia	No Reply		Brent Story	brent.story@dot.state.ga.us	Road Design Engineer
11	Hawaii	0		Casey Abe	Casey.Abe@hawaii.gov	Engineering Program Manager
12	Idaho	No Reply		Jeff Miles	jeff.miles@itd.idaho.gov	Materials Engineer
13	Illinois	1	I-70	LaDonna Rowden	LaDonna.Rowden@illinois.gov	Pavement Technology Engineer
14	Indiana	No Reply		Dan Drewski	dandrewski@indot.state.in.us	Materials Engineer
15	Iowa	1	US 60	James Berger	james.berger@dot.state.ia.us	Office of Materials Director
16	Kansas	1	US 75 (4sections)	Dick McReynolds	dick@ksdot.org	Engineer of Research

Table 3.1 – Perpetual Pavements from US State DOTs (cont.)

	State	Existing Perpetual Pavement		Contact	Email	Position
17	Kentucky	2	I-64, I-65	Brian Wood	brian@paiky.org	Executive Director The Plantmix Asphalt Industry of Kentucky, Inc.
18	Louisiana	1	I-49	Kim Garlington	KimGarlington@dotd.la.gov	Pavement and Geotechnical Services Engineer Administrator
19	Maine	0		Brian Burne	Brian.Burne@maine.gov	Pavement Engineer
20	Maryland	1	I-695	Larry Michael	lmichael@sha.state.md.us	Pavement Engineer
21	Massachusetts	0		Carol Hebb	carol.hebb@mhd.state.ma.us	Special Projects Engineer
22	Michigan	3	US-24, I-96, M-84	Michael Eacker	belcherd@michigan.gov	Supervisor of the Engineering Services Unit
23	Minnesota	5	I-35, TH71, TH10, TH18, TH61	Erland Lukanen	Erland.Lukanen@dot.state.mn.us	Pavement Preservation Engineer
24	Mississippi	No Reply		Keith Purvis	kpurvis@mdot.state.ms.us	Asst. Rdwy. Design Eng
25	Missouri	0		John Donahue	John.Donahue@modot.mo.gov	Pavement Engineer
26	Montana	0		Mark Wissinger	mwissinger@state.mt.us	Roadway and Structures Engineer
27	Nebraska	0		Moe Jamshidi	mjamshid@dor.state.ne.us	Materials and Research Engineer
28	Nevada	0		Dean C. Weitzel	dweitzel@dot.state.nv.us	Chief Materials Engineer
29	New Hampshire	1	I-93	Eric Thibodeau	Ethibodeau@dot.state.nh.us	Pavement Management
30	New Jersey	1	I-287	Robert Sauber, Ron Gruzlovic	Ron.Gruzlovic@dot.state.nj.us, Robert.Sauber@dot.state.nj.us	Engineering Technician

Table 3.1 – Perpetual Pavements from US State DOTs (cont.)

	State	Existing Perpetual Pavement		Contact	Email	Position
31	New Mexico	1	US 70	Larry Velasquez	larry.velasquez@state.nm.us	District 2 Engineer
32	New York	No Reply		Orlando Picozzi	opicozzi@dot.state.ny.us	Pavement Engineer
33	North Carolina	No Reply		Steve Dewitt	sdewitt@dot.state.nc.us	Pavement Engineer
34	North Dakota	0		Ron Horner, Ken E. Birst	rhorner@state.nd.us, kbirst@state.nd.us	Materials and Research Engineer
35	Ohio	2	US 30, I-77	Roger Green	Roger.Green@dot.state.oh.us	Pavement Research Engineer
36	Oklahoma	1	SH-152	Reynolds H. Toney	rtoney@odot.org	Materials Engineer
37	Oregon	2	I-5, I-90	Rene Renteria	Rene.A.RENTERIA@odot.state.or.us	Pavement Design Engineer
38	Pennsylvania	No Reply		Tim Ramirez	tramirez@state.pa.us	Division Chief for the Engineering Technology
39	Rhode Island	0		Deborah Munroe	dmunroe@dot.state.ri.us	Research and Technology Development
40	South Carolina	0		Merrill Zwanka	zwankaME@scdot.org	State Materials Engineer
41	South Dakota	No Reply		Joe Feller	joe.feller@state.sd.us	Materials & Surfacing
42	Tennessee	0		James Maxwell	James.Maxwell@state.tn.us	Manager-Research & Product Evaluation Section
43	Texas	8	SH 114, I-35 (7Sections)	Daor Hao Chen	DCHEN@dot.state.tx.us	Pavement Analysis Supervisor
44	Utah	0		Richard Sharp	rsharp@utah.gov	Research Specialist
45	Vermont	No Reply		Bill Ahearn	bill.ahearn@state.vt.us	Research and Testing Engineer

Table 3.1 – Perpetual Pavements from US State DOTs (cont.)

	State	Existing Perpetual Pavement		Contact	Email	Position
46	Virginia	1	I-95	Bill Bailey	bill.bailey@virginiadot.org	Assistant State Materials Engineer for Operations
47	Washington	1	I-90	Jeff S. Uhlmeyer, Linda Pierce	UhlmeyJ@wsdot.wa.gov, PierceL@wsdot.wa.gov	Pavement Design Engineer
48	West Virginia	0		Larry Baker	lbarker@dot.state.wv.us	Asphalt & Hot-Mix Unit Supervisor
49	Wisconsin	2	STH 50, I-94	Steven W. Krebs	steven.krebs@dot.state.wi.us	Chief Materials Management Engineer
50	Wyoming	0		Rick Harvey	Rick.Harvey@dot.state.wy.us	State Materials Engineer

<u>Texas</u>

Currently, the Texas Department of Transportation has eight perpetual sections in service (Scullion 2006). The current structural design and analysis method of Texas full depth asphalt pavements (FDAP) is mechanistic-empirically based using the Flexible Pavement System (FPS) 19W software (Scullion and Liu, 2001). However, the thickness design is also often checked with the PerRoad software (Timm 2004; Timm and Newcomb 2006).

Texas - Perpetual Pavements in Texas: The Fort Worth SH 114 Project in Wise County FHWA/TX-07/0-4822-2; Report 0-4822-2 (Walubita and Scullion 2007)
This TxDOT report provides a case study describing the design, construction, initial structural evaluation, and performance predictions of the full-depth perpetual pavement constructed on SH 114 in the Fort Worth District. Findings from that report indicate that using VESYS, PerRoad, and FPS 19W software analysis, the pavement section will require rehab at the end of 20, 30, 30 and 24 yrs respectively. MEPDG level 1 analysis indicates that the pavement meets expected performance criteria for a 30 year design life

Texas - Perpetual Pavements in Texas: State of the Practice FHWA/TX-06/0-4822-1; Report 0-4822-1(Scullion 2006)
The aim of this TxDOT research project is to monitor field performance of full-depth asphalt pavements to validate design procedures. Project 0-4822 was initiated to perform a structural assessment of perpetual pavements, to identify strengths and weaknesses in the existing structures, and to provide guidance for future designs. Findings from that report indicate that using stone-filled mixes rather than traditional dense-graded mixes provides a

significantly stiffer pavement. However, these stone-filled layers were found to be prone to vertical segregation. De-bonding between layers was also found to be a major problem in many of the perpetual pavements.

Texas - Laboratory Testing and MEPDG Performance Predictions of Perpetual Pavements, No. 07-3370 (Walubita 2007)
The objective of Project No. 07-3370 is to present as a case study of the laboratory and computational performance predictions of Texas perpetual pavement structures with a focus on the rut-resistant and fatigue-resistant layers. The MEPDG Version 0.910 was used for computational analyses and performance predictions. The pavement is predicted to have little or no potential for bottom-up fatigue cracking. No permanent deformation is expected in the intermediate layer. However, surface treatment is expected within the first 23 years of service.

<u>California</u>

The perpetual pavement concept has been used in rehabilitation and reconstruction of part of Interstate highway 710 in southern California. The projected design lane traffic was estimated at 100–200 million equivalent single axle loads (ESALs) of 80 kN for 40 years period. The total thickness of the HMA perpetual pavement was 12 inches (Martin et al. 2001, Monismith et al. 2004).

<u>Illinois</u>

The Illinois Department of Transportation (IDOT) uses its own mechanistic–empirical design method in the construction and design of perpetual pavements. The typical pavement design in Illinois is based on a maximum tensile strain of 60 $\mu\varepsilon$. Interstate highway 70 is an example of a perpetual pavement in Illinois (Asphalt Institute 2004).

<u>Michigan</u>

Michigan Asphalt Paving Association (MAPA) developed a catalogue of structural sections for use as perpetual pavement. The structural sections are listed according to the traffic levels expected in the first 20 years. The total HMA thickness ranges from 12 to 16 inches for the four traffic levels. The pavement foundation in MAPA catalogue consists of 8 – 13.5 inches of non-frost susceptible material under crushed aggregate subbase or a crushed stone base course for low (< 10 millions ESAL) and high (> 20 millions ESAL) traffic levels, respectively. There are three recently constructed perpetual pavements in Michigan; they are US-24, I-96 and M-84 (Von Quintus 2001, APA 2002).

Kansas

Kansas Department of Transportation (KDOT) developed a field trial to investigate the suitability of this concept for Kansas highway pavements. The experiment involved the construction of four thick pavement structures on a new alignment highway US-75 near Sabetha, Kansas, in Brown County. They were designed to have a perpetual life and have layer thicknesses close to those recommended by the current KDOT's structural design method for flexible pavements, based on the 1993 AASHTO Design Guide (Romanoschi et al. 2008).

Ohio

In Ohio, there are two perpetual pavements: US-30 and I-77. These perpetual pavements are located in Wooster County and North Canton respectively. The section on I-77 was completed in 2003 and is 2.3 miles ling. The section on US-30 was completed in 2005-2006, and is 8 miles long. The total thickness of both perpetual pavements is 20.5 and 22.25 inches, respectively. Both perpetual pavements used a rich binder layer (RBL) as a base layer. The section on US-30 had a limiting strain at the bottom of the pavement of 70 $\mu\varepsilon$ (Powers 2007).

New Mexico

There is one perpetual pavement in New Mexico, US-70 in the Hondo Valley area. A review of the perpetual pavement section on US 70 is presented in Task 1C in this report.

Other States

States such as Iowa, Kentucky, Michigan, New Jersey, Oklahoma, and Washington are known to have perpetual pavements. However, very little to no information on the perpetual pavements in these states is available in the literature.

3.3. Task 1C. New Mexico US 70 Hondo Valley Project Review

In this task, the UNM research team collected all available design, construction, and performance data of US-70 pavement, which was designed and constructed using the perpetual pavement concept. In particular, the UNM researchers consulted NMDOT's engineers who were associated with the design and construction phases of US-70 in order to document their experience in this project. In addition, QC/QA information for the subgrade, asphalt layers, and samples of US-70 was also gathered.

NM US-70 Hondo Valley Project Review

The available design, construction and performance data of US-70 Hondo Valley project is described and summarized below (AMEC 2007).

Design Criteria

According the AMEC's report on the US-70 Hondo Valley Project, this perpetual pavement was designed using a mechanistic-design (ME) procedure that included models from the Asphalt Institute (AMEC 2007). These models assume a distress level of:

- ≤ 10 percent fatigue cracking at the end of the pavement life (fatigue failure); and
- ≤ 0.5 inches of subgrade permanent deformation at the end of the pavement life (rutting failure).

The US-70 Hondo Valley project is a design/build project that was designed and built by Sierra Blanca Constructors (SBC). The SBC design used the concept of using a "rich bottom" mix as a base layer in the HMA portion of the pavement structure. SBC indicated to the NMDOT and the FHWA that the use of this concept would increase the life of the HMA pavement from 20 years to 30 years. On contacting the NMDOT District 2 office, no information was available regarding the limiting strain at the bottom of the asphalt layer.

Traffic, Thickness, and Design Life

In the US-70 Hondo Valley project, a two-lane highway was removed and replaced with a four-lane highway from Milepost 264.4 to 302.1 for a length of 37.7 miles. The pavement structure was built during the 2003 - 2004 time period. The OGFC was placed during the summer of 2005. The asphalt pavement structure used on the project consisted of nine inches of Plant Mix Bituminous Pavement (PMBP) on a six inch untreated base course over a compacted subgrade. The PMBP layer consisted of 2.5 inches of SP-III over a SP-III layer with 20 percent recycled asphalt pavement (this layer varied in thickness (4, 4.25 and 4.5 inches) depending on the strength of the subgrade in that area) over a 2.5 inch SP III layer which was designed with additional asphalt binder to provide a rich bottom layer (to improve the fatigue resistance of the pavement). The design life is 30 years. The design traffic level used by SBC as provided by the NMDOT was 8.8 million ESALs.

Material Properties

Tables 3.2(a) to (c) show the material properties of the five sections of the US-70 Hondo Valley project. Each table shows the three main layers of a perpetual pavement, the wearing surface, the intermediate layer and the base layer. The thickness of each layer is given along with its properties such as binder grade and content, air void content and modulus. This data can be directly inputted to mechanistic-empirical software such as MEPDG.

Table 3.2(a) – Properties of US-70 Perpetual Pavement Sections

Pavement Layer	Pavement Composition	New Mexico US 70(a)		New Mexico US 70(b)	
Wearing Surface	Material & Thickness	0.63 in OGFC	2.5 in HMA	0.63 in OGFC	2.5 in HMA
	Asphalt Binder	PG 70-22	PG 70-22	PG 70-22	PG 70-22
	(%) Binder Content	4.8	4.8	4.8	4.8
	(%) Air Voids	4	4	4	4
	Modulus (psi)	650000		650000	
Intermed. Course	Material & Thickness	4.25 in HMA		4.5 in HMA	
	Asphalt Binder	PG 64-22		PG 64-22	
	(%) Binder Content	4.9		4.9	
	(%) Air Voids	4.0		4	
	Modulus (psi)	868000		868000	
Base	Material & Thickness	2.5 in HMA	6 in GB	2.5 in HMA	6 in GB
	AASHTO Material	NA	A-1-a	NA	A-1-a
	Asphalt Binder	PG 70-22	NA	PG 70-22	NA
	(%) Binder Content	5.2	NA	5.2	NA
	(%) Air Voids	2.8	NA	2.8	NA
	Modulus (psi)	977000	23100	977000	27400
Subgrade	AASHTO Material	GM		GM	
	Resilient Modulus (psi)	10100		8700	

Note: NA = Not Applicable, AMEC (2007).

Table 3.2(b) – Properties of US 70 Perpetual Pavement Sections (cont.)

Pavement Layer	Pavement Composition	New Mexico US 70(c)		New Mexico US 70(d)	
Wearing Surface	Material & Thickness	0.63 in OGFC	2.5 in HMA	0.63 in OGFC	2.5 in HMA
	Asphalt Binder	PG 70-22	PG 70-22	PG 70-22	PG 70-22
	(%) Binder Content	4.8	4.8	4.8	4.8
	(%) Air Voids	4	4	4	4
	Modulus (psi)	650000		650000	
Intermediate Course	Material & Thickness	4.25 in HMA		4 in HMA	
	Asphalt Binder	PG 64-22		PG 64-22	
	(%) Binder Content	4.9		4.9	
	(%) Air Voids	4.0		4	
	Modulus (psi)	868000		868000	
Base	Material & Thickness	2.5 in HMA	6 in GB	2.5 in HMA	6 in GB
	AASHTO Material	NA	A-1-a	NA	A-1-a
	Asphalt Binder	PG 70-22	NA	PG 70-22	NA
	(%) Binder Content	5.2	NA	5.2	NA
	(%) Air Voids	2.8	NA	2.8	NA
	Modulus (psi)	977000	25500	977000	26700
Subgrade	AASHTO Material	GM		GM	
	Resilient Modulus (psi)	10300		11800	

Note: NA = Not Applicable, AMEC (2007).

Table 3.2(c) – Properties of US 70 Perpetual Pavement Sections (cont.)

Pavement Layer	Pavement Composition	New Mexico US 70(e)	
Wearing Surface	Material & Thickness	0.63 in OGFC	2.5 in HMA
	Asphalt Binder	PG 70-22	PG 70-22
	(%) Binder Content	4.8	4.8
	(%) Air Voids	4	4
	Modulus (psi)	650000	
Intermed. Course	Material & Thickness	4 in HMA	
	Asphalt Binder	PG 64-22	
	(%) Binder Content	4.9	
	(%) Air Voids	4.0	
	Modulus (psi)	868000	
Base	Material & Thickness	2.5 in HMA	6 in GB
	AASHTO Material	NA	A-1-a
	Asphalt Binder	PG 70-22	NA
	(%) Binder Content	5.2	NA
	(%) Air Voids	2.8	NA
	Modulus (psi)	977000	29100
Subgrade	AASHTO Material	GM	
	Resilient Modulus (psi)	13000	

Note: GB = Granular Base, NA = Not Applicable, AMEC (2007).

Performance Data

In May 2007, the NMDOT conducted a deflection survey using a Falling Weight Deflectometer (FWD) on the Hondo Valley portion of US-70. This data was analyzed using a back calculation program to determine the modulus of the HMA Layer, the UTBC layer and the subgrade layer. The computer program used is called “Deflection Analysis of Design Structures”. It is based on a Finite Element Code developed by the University of Illinois. The result of that analysis is shown in Table 3.3 (AMEC 2007). The reported strengths (surfacing, base and subgrade) were determined using the procedures detailed in the AASHTO 93 Pavement Design Guide. It was found that the surfacing strengths as determined by the FWD are considerably lower than that used for the design. Reasons for this are unknown. The base strengths for Sections 1 and 2 are higher than those used for design (AMEC 2007).

Table 3.3 – Pavement Sections Based on FWD Data

		Pavement Section 1	Pavement Section 2*	Pavement Section 3	Pavement Section 4	Pavement Section 5
		0.63 in OGFC	0.63 in OGFC	0.63 in OGFC	0.63 in OGFC	0.63 in OGFC
		2.5 in PMBP	2.5 in PMBP	2.5 in PMBP	2.5 in PMBP	2.5 in PMBP
		4.25 in RAP	4.5 in RAP	4.25 in RAP	4.0 in RAP	4.0 in RAP
		2.5 in Rich Bottom	2.5 in Rich Bottom	2.5 in Rich Bottom	2.5 in Rich Bottom	2.5 in Rich Bottom
Modulus of Surface Layer		538,530		366,415	533,200	538,530
-		6 in UTBC	6 in UTBC	6 in UTBC	6 in UTBC	6 in UTBC
UTBC/RAP Layer	Modulus	64,283 *	46062 *	28130	28517	32453
	SC	0.29	0.21	0.13	0.13	0.15
Modulus of Subgrade layer (psi)		5,413	4,655	7,588	9,038	12,518
Structural Number of Section (structural coefficient = 0.44)		5.81	5.44	4.85	4.74	4.86
Required Structural Number using the AASHTO Design Guide using FWD subgrade strength		5.65	5.92	5.08	4.79	4.28
Life expectancy of pavement (ESALs) Based on actual structural number using FWD subgrade strength		14800000	4670000	6400000	8200000	20700000

*The modulus values for the base course in pavement Sections 1 and 2 are considerably higher than what is normally encountered in an untreated base course. This may be due to a high percentage of RAP in these two sections or it may be a testing anomaly. This will need to be further evaluated during the Phase 2 portion of this study, AMEC (2007).

Note: OGFC = Open Graded Friction Course, PMBP = Plant Mix Bituminous Pavement, UTBC = Untreated Base Course, RAP = Recycled Asphalt Pavement, SC = Structural Coefficient.

QC/QA Information for Pavement Layers and Samples

The quality assurance test data was made available by the NMDOT to AMEC (AMEC 2007). Tables 3.4(a) – (c) present the average results of the quality assurance and quality control testing accomplished by the NMDOT and the contractor. The conclusion as to the acceptability of the project was based the NMDOT's quality assurance data. The review of the NMDOT quality assurance data by AMEC and the available contractor quality control data showed that the project was built in accordance with the specifications for the project.

Table 3.4(a) – Summary of Quality Assurance/Quality Control Test Data Surfacing Mix

Property	Spec Limits	DOT Data		SBC Data	
		Avg.	Std.	Avg.	Std.
Asphalt	4.80 ± 0.30	4.62	0.11	4.67	0.13
Density	95 ± 3.00	93.7	1.3	93.4	1.5
Air Voids	4 ± 1.30	3.62	0.81	3.16	0.95
VMA	14 ± 1	-	-	13.4	0.71
VFA	65 to 78	-	-	76.8	6.1
Nominal	95 ± 5.00	94.3	1.8	-	-
½ inch	-	-	-	79.2	3.6
3/8 inch	64 ± 6.00	59.8	3.9	-	-
No. 4	-	-	-	38.4	2.8
No. 8	28 ± 4.00	25.5	1.8	26.4	1.6
No. 16	21 ± 4.00	18.5	1.3	-	-
No. 30	-	-	-	14	1.1
No. 50	11 ± 4.00	10.1	1.1	-	-
No. 200	6.5 ± 2.00	5.68	0.82	5.15	0.65
FA Angularity		-	-	47.2	0.59
Sand Equivalent		-	-	54.1	4.4
Dust Proportion	0.6 to 1.6	-	-	1.18	0.16

Table 3.4(b) – Summary of Quality Assurance/Quality Control Test Data RAP Mix

Property	Spec Limits	DOT Data		SBC Data	
		Avg.	Std.	Avg.	Std.
Asphalt	3.30 ± 0.30	3.22	0.11		
Density	95 ± 3.00	93.7	1.4	93.4	1.6
Air Voids	4 ± 1.30	3.65	0.92	3.46	1.1
VMA	14 ± 1	-	-	12.6	0.96
VFA	65 to 78	-	-	72.9	6.5
Nominal	95 ± 5.00	95.2	1.9	-	-
½ inch	-	-	-	82.4	3.7
3/8 inch	69 ± 6.00	67.5	4.4	-	-
No. 4	-	-	-	42.8	4.0
No. 8	28 ± 4.00	26.1	2.0	26.8	2.5
No. 16	21 ± 4.00	18.6	1.6	-	-
No. 30	-	-	-	14.5	1.67
No. 50	12 ± 4.00	10.7	1.3	-	-
No. 200	6.4 ± 2.00	6.6	0.78	6.6	0.86
FA Angularity		-	-	45.7	0.51
Sand Equivalent		-	-	60.1	5.0
Dust Proportion	0.6 to 1.6	-	-	1.71	0.22

AMEC (2007)

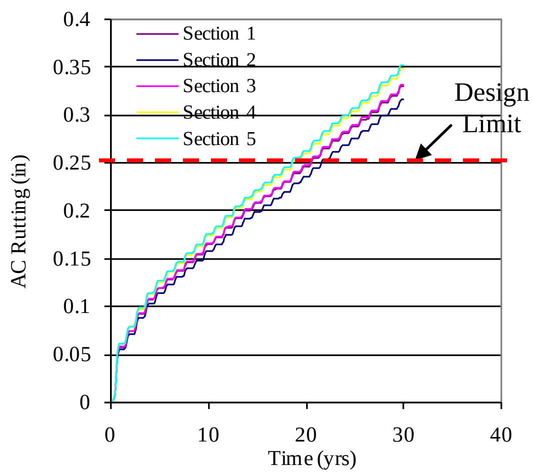
Table 3.4(c) – Summary of Quality Assurance/Quality Control Test Data Rich Bottom Mix

Property	Spec Limits	DOT Data		SBC Data	
		Avg.	Std.	Avg.	Std.
Asphalt	5.10 ± 0.30	5.2	0.14	5.02	0.08
Density	96 ± 3.00	95.2	1.6	94.6	1.7
Air Voids	2 ± 1.30	1.91	0.84	1.75	0.66
VMA	14 ± 1	.	-	13.0	0.49
VFA	-	-	-	86.2	4.6
Nominal	95 ± 5.00	95.1	2.0	-	-
½ inch	-	-	-	81.3	3.4
3/8 inch	64 ± 6.00	60.9	4.4	-	-
No. 4	-	-	-	39.6	3.5
No. 8	28 ± 4.00	25	1.9	27.2	2.2
No. 16	21 ± 4.00	19.3	1.5	-	-
No. 30	-	-	-	14.8	1.2
No. 50	11 ± 4.00	10.6	1.2	-	-
No. 200	6.5 ± 2.00	5.82	0.66	5.6	0.72
FA Angularity		-	-	47.2	0.41
Sand Equivalent		-	-	58.6	6.7
Dust Proportion	0.6 to 1.6	-	-	1.2	0.15

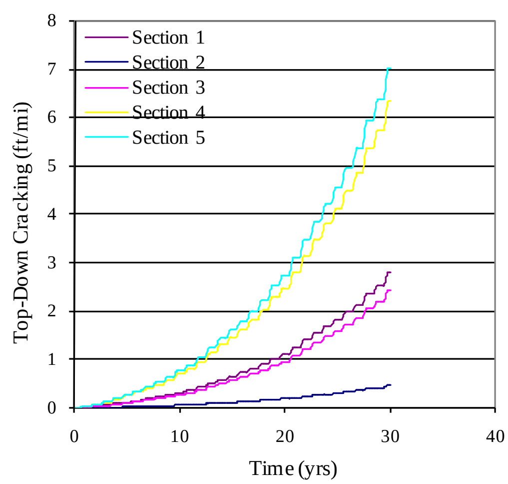
AMEC (2007)

MEPDG Level 3 Analysis of US-70

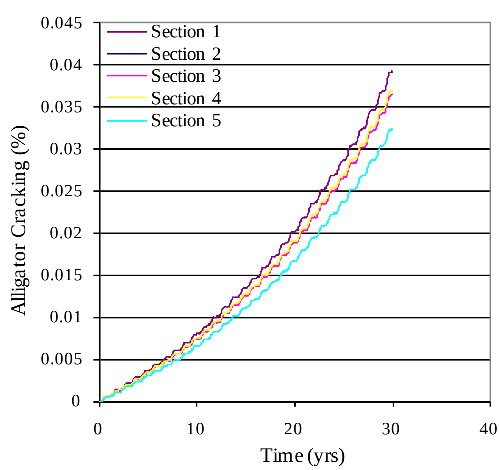
The MEPDG analyses were conducted on the five sections of US-70. Figures 3.1(a) – (d) compare the results obtained by the UNM research team, of AC rutting, top-down cracking, bottom-up (alligator) cracking and IRI for each of the five sections. The assumed design limit for total and AC rutting is 0.75 inches and 0.25 inches respectively. All analysis was done using 90% reliability. Each pavement section is predicted to fail due to excessive rutting (AC and total) and International Roughness Index (IRI). All five pavement sections passed the design limit for bottom-up (alligator), transverse and top-down cracking. Note that failure criteria is based on MEPDG recommended values.



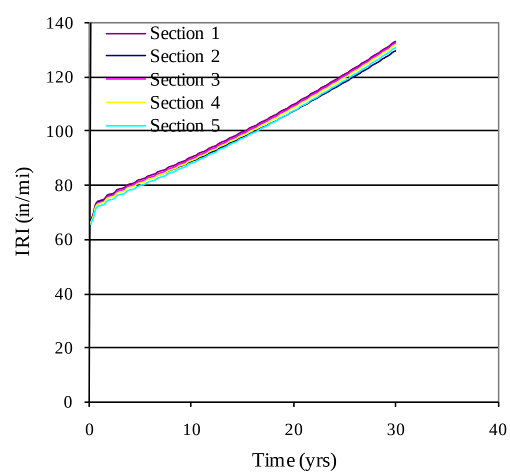
(a) AC Rutting



(b) Top-Down Cracking



(c) Alligator Cracking



(d) IRI

Figure 3.1 – MEPDG Level 3 Analysis of US-70 Pavement Sections

3.4. Task 1D. Preliminary Analysis

In this task, information regarding the use of rich binder layers and potential permeability and de-bonding issues in perpetual pavements is presented. Input from state DOTs from all over the country highlights these problems and solutions are provided to prevent such problems from occurring.

3.4.1 Rich Binder Layer in Perpetual Pavements

The perpetual pavement concept mentioned earlier describes the two approaches that are actively used both in the U.S. and internationally. Table 3.5 presents state DOTs that use a RBL in their perpetual pavements. Eight of 14 US perpetual pavements investigated use rich-bottom (binder) layers, and 9 out these 14 pavements also use a limiting strain or FEL in their base layer. A liming strain of 70 $\mu\varepsilon$ is the most common value used. Polymer modified binders are more commonly used in surface and intermediate layers, in order to prevent rutting. The use of polymer-modified binders in base layers is rare. Based on the literature reviewed, only one state DOT out of seventeen has used a polymer-modified binder in its base layer. Illinois DOT used polymer modified binders in the surface, intermediate and base layers on a 12 mile section of I-70. The IDOT decided against using a 'rich bottom layer' as the potentially higher permeability of SuperPave mixtures might allow moisture to infiltrate and become entrapped and this would lead to premature stripping of the HMA. Maryland and Oregon DOTs did not include a RBL in their respective perpetual pavements, I-695 and I-5. Texas, California and Ohio DOTs use both RBLs and polymer modified binders in their respective base and upper asphalt layers, to prevent fatigue cracking and rutting.

Also shown in Table 3.5 is the design method/design program used for each state perpetual pavement. Many US states still use AASHTO 1993 Pavement Guide for the design of new or reconstructed perpetual pavements, such as Kansas, Washington, Oregon and New Jersey. Several states have come up with their own mechanistic–empirical design method such as California, Illinois and Texas, while others are in the process of developing a mechanistic-empirical method for newly constructed pavements. DOTs that use their own method of design usually include software programs which allow them to estimate an appropriate thickness for the pavement. For instance Illinois and Texas use ILLIPAVE and FPS 19W software, respectively. The majority of state DOTs however are evaluating the implementation of the 2002 AASHTO Mechanistic Empirical Pavement Design Guide.

Table 3.5 – U.S. Perpetual Pavement Structural Design Methods

No.	State Pavement	Perpetual Pavement Concept: Limiting Strain in Base Layer	Perpetual Pavement Concept: Rich-Binder Layer	Design Method/ Software
1	California (I-710)	70 με	YES	CIRCLY, CA-4PRS
2	Illinois (I-70)	60 με	NO	ILLIPAVE, IDOT ME Design
3	Kansas (US-75)	70 με	YES	1993 AASHTO, EVERSTRESS
4	Kentucky (I-695)	70 με	NO	ME Design Method
5	Minnesota (I-35)	NO	NO	ELSYM 5, Von Quintus Catalog 2001
6	New Jersey (I-287)	NO	NO	-
7	New Mexico (US-70)	YES	YES	Asphalt Institute
8	Ohio (US-30)	70 με	YES	Kenlayer
9	Oklahoma (SH-152)	70 με	YES	PerRoad
10	Oregon (I-5)	70 με	YES	AASHTO 1993, WESLEA
11	Texas (SH-114)	70 με	YES	FPS 19W, PerRoad
12	Virginia (I-95)	NO	NO	-
13	Washington (I-90)	NO	NO	AASHTO 1993, EVERSERIES
14	Wisconsin (STH-50)	-	YES	AASHTO 1972, WisPave

Note: NO = no limiting strain or rich-binder used in base layer, YES = there is a limiting strain or rich binder used in base layer, “–” = No data available

3.4.2. Permeability in Perpetual Pavements

Many state DOTs deal with permeability issues by ensuring proper construction practices (compaction levels, lift densities) and the use of liquid additives in the asphalt mixes. Illinois DOT uses liquid additives when it determines a mixture is susceptible to moisture damage (stripping). Along with the mandatory use of lime, current IDOT design standards require longitudinal under-drains to be placed under the shoulder/pavement joint on both sides of the pavement. An open graded drainage layer is also used in a number of perpetual pavement test sections on the Bin-Bo expressway in Shandong Province, China. These drainage layers are to help remove moisture and mitigate stripping problems. State DOTs that currently have permeability preventative measures in their perpetual pavement sections are listed in Table 3.6.

Table 3.6 – Design for Permeability and De-bonding in U.S. Perpetual Pavements

Pavement Section	Permeability	Bonding/ Debonding
California I-710	Drainage Layers placed in Pavement	Prime coats and tack coats used between layers
Illinois I-70	AASHTO T-283 test with liquid additives (lime) for all mixes	Polymer Priming between all layers in pavement
Iowa US-60	-	-
Kansas US-75	Underdrains placed in pavement, at shoulders and median area	-
Kentucky I-64	-	-
Maryland I-695	-	-
Michigan I-96	No Design	No Design
Minnesota I-35	No Design	No Design
New Jersey I-287	-	-
New Mexico US-70	-	-
Ohio US-30	AASHTO T-283 test for all mixes	Tack coats applied between all layers
Oklahoma SH-152	Low Permeability specifications for all mixes (12.5x10^-5 cm/s)	-
Oregon I-5	Lime and latex anti-strip	-
Texas SH-114	Edged drains and chip seal application	Tack coats applied between all layers
Virginia I-95	-	-
Washington I-90	Drainage Layers placed in Pavement, anti-strip modifier (liquid additives), crack seal products	Tack coats applied between all layers
Wisconsin STH 50	-	-

Note: "–" = No data available

3.4.3. De-bonding in Perpetual Pavements

Recent research studies from this literature review indicate that poor construction practices during placement of asphalt layers and failure to use tack coat between layers were primarily responsible for de-bonding in flexible pavements. For instance, de-bonding in Texas's SH-114 was found to be a major concern as cores de-bonded at one or two locations. The interfaces did not show any indication of tack coat. Information on state DOT methods on preventing de-bonding is available in Table 3.6. The latest edition of MEPDG has a design for bonding/de-bonding in its program. This software does not provide partial bonding for asphalt layers in a pavement section. A pavement can either have a full bond between its layers or none at all. From this literature review it is believed that mechanistic design procedures work on the premise that the asphalt layers are bonded together and that the traffic loads will bend the composite beam of asphaltic materials and induce tensile strains at the bottom of the RBL layer, which was specifically designed to accommodate tensile strains without initiating fatigue cracking. Having de-bonded layers within the HMA structure will defeat the purpose of the RBL as the fatigue cracking will initiate at the de-bonded interface, and the higher the de-bonding in the pavement structure, the more severe the consequence will be on the pavement's fatigue life. Permeability and bond/de-bonding standards for highways in foreign countries are also presented in Table 3.7.

Table 3.7 – Design for Permeability and De-Bonding in International Perpetual Pavements

Pavement Section	Permeability Standards	De-Bonding Standards
Australia	Chip seal and Crack seal also used. Drainage layers also used	-
China	Drainage Layers placed in Pavement	-
Israel	-	-
New Zealand	Crack seal and chip seal used. Drainage layers also used	-
United Kingdom	-	Tack Coat placed in between Asphalt Layers

Note: "–" = No data available

3.5. Task 1E Preliminary Findings

This task presents a summary of the perpetual pavements built and designed here in the US. Information regarding the construction of these pavements and the specifications employed as well as the current performance of these pavements is presented below. Data collected in the above subtasks is also organized as MEPDG input data. As well as acquiring the input information for MEPDG, the predicted performance of each design is also determined and then compared to the expected performance.

3.5.1 What Was Designed and the Experience To Date

Table 3.8 presents the perpetual pavement sections built by their respective state highway agency. Also shown is the design life and surface life of each pavement, the perpetual design concept employed, traffic volume, pavement thickness (individual and total), and the material properties (layer stiffness and PG binder) of each pavement section. Also shown in Table 5 is some statistical analysis of the data. The mean layer thickness is determined as well as the range (standard deviation) of thicknesses for each asphalt layer. This information is used in the Chapter 5 to determine an optimal perpetual pavement structure based on layer thickness and stiffness.

From Table 3.8 and information gathered from various State DOTs, the following observations were made;

- California, Kansas, Iowa, Illinois, Maryland, New Jersey, New Mexico, Oklahoma, Oregon, Virginia, and Washington all have one perpetual pavement in operation.
- New Jersey's I-287 was not originally built as a perpetual pavement, but the rehabilitation project in 1996 considered the new design as a perpetual pavement (Rowe et al. 2001).
- New Mexico's first perpetual pavement, US-70, was built in 2005 near Hondo Valley (AMEC 2007).
- Oklahoma's SH-152 is the only perpetual pavement in the state. Similar test sections were built at the NCAT test track (Timm et al. 2008).
- Oregon has built perpetual pavement sections on I-5, near Albany between 2005 and 2008. Pavement sections contain rubblized concrete with an asphalt overlay and full depth sections (Scholz et al. 2007).
- Washington's I-90 was not originally built as a perpetual pavement, but is considered a perpetual pavement due to its performance and design life (Mahoney 2001). Ohio and Wisconsin all have two perpetual pavements sections in their respective states.
- Ohio's perpetual pavement sections, I-77 and US-30, are located in North Canton and Wooster County and were constructed in 2003 and 2004 (Powers 2007).
- I-94 and STH-50 contain Wisconsin's only perpetual pavement test sites. Five test sections were built on STH-50 in 2000 and two test sections were constructed at a truck weigh station near Kenosha in 2002. Two of the test sections built on STH-50 are control sections, reflecting normal Wisconsin construction procedures (Krebs 2008).Michigan has constructed three perpetual pavement sections pavements carrying different volumes of traffic. The perpetual pavement sections on M-84, US-

24, and I-96, were designed to carry low, medium, and high traffic volume. These pavement sections were constructed between 2002 and 2007, and the Michigan DOT are currently in the process of constructing a fourth perpetual pavement section (Eacker 2008).

- Texas DOT has the most experience in constructing and maintaining perpetual pavements, with eight pavement sections currently in service, constructed in 2003 (2 sections), 2004, 2005, 2006 (2 sections), and 2007 (2 sections). Seven pavement sections are located on I-35, with four in the Laredo District, two in the Waco District, and one in San Antonio District. Another perpetual pavement section is located on SH-114 in the Fort Worth District (Scullion 2006).

3.5.2 Field Performances to Date

Although there was no performance data to prove these evaluations, the perpetual pavement sections on California's I-710 appear to be in excellent condition, according to state DOT officials (Monismith 2008). Pavements sections on Kansas's US-75, Maryland's I-695, Michigan's I-96, New Jersey's I-287, Ohio's US 30, Oregon's I-5, and Washington's I-90 are also performing as expected. There was also no performance data available for Kansas's US-75, Oklahoma's SH 152, and Michigan's I-96, reports from all three state DOTs indicate that both pavements are performing satisfactorily.

Table 3.9 shows the field performance of the pavement sections in terms of rutting, IRI, bottom-up and top down cracking. Performance data accrued from state DOTs in Maryland, New Jersey, Ohio, Oregon, and Washington show that these pavements are performing satisfactorily. New Mexico's US-70 has encountered problems with permeability, with water said to be weeping out of the pavement onto the surface (AMEC 2007). Reasons for this are thought to be because of accumulation of moisture between the rich binder layer and the HMA layer above it. An investigation revealed that the surface layer was highly permeable and that the subgrade strengths were lower than expected (AMEC 2007).

Personal communication with Oregon DOT revealed that moisture-related problems, such as stripping, have been discovered on Oregon's I-5. ODOT uses lime or anti-strip additives when required in design mixes, but the state has been re-examining this policy and the use of lime or latex has been increased in areas where stripping is evident on I-5 (Renteria 2008).

Texas perpetual pavements are also experiencing similar problems. Ground penetrating radar (GPR) data indicated areas of tapped moisture in Texas's SH 114. Further field investigation revealed that trapped water was accumulating between the asphalt layers, in particular the stone-filled asphalt layers. Pavement section cores taken from perpetual pavements in the Fort Worth, San Antonio, and Waco Districts all exhibited varying degrees of vertical segregation. Other problems identified are de-bonding and honeycombing at the interfaces. Performance data indicates high rutting on SH 114 (Scullion 2006). In 2007, performance data indicated high top-down cracking on Wisconsin's I-94 and high transverse cracking on STH-50 (Battaglia 2009).

Table 3.8 – Summary of Perpetual Pavement Data

State PP	Highway/Interstate	Surface Life (yrs)	Design Life (yrs)	Traffic AADTT	RBL	FEL (με)	Layer Thickness (in) OGFC/SMA	HMA_1	HMA_2	HMA_3	HMA_4	Total	GB	TSG	Layer Stiffness GB M_R (psi)	TSG M_R (psi)	PG Binder HMA_1	HMA_2	HMA_3	HMA_4
California	I-710	19	40	10000	YES	70	1 OGFC	3	6	3	-	13	6	-	-	-	-	64-40	64-16	-
Illinois	I-70	19	30	11760	NO	60	2 SMA	5.5	10	-	-	17.5	8	-	-	28000	76-28	76-28	70-22	-
Iowa	US-60	15	40	8250	-	70	-	2	2	7.5	3	14.5	9	18	50000	20000	64-34	64-34	58-38	58-38
Kansas	US-75 (a)	-	10	450	NO	70	-	1.5	2.5	9	-	13	-	6	-	5000	70-28	70-28	70-22	-
	US-75 (b)	-	10	5000	NO	70	-	1.5	2.5	7	-	11	-	6	-	2500	70-28	70-28	64-22	-
	US-75 (c)	-	10	450	YES	70	-	1.5	2.5	9	-	13	-	6	-	2500	70-28	70-28	64-22	-
	US-75 (d)	-	20	900	NO	70	-	1.5	2.5	12	-	16	-	6	-	2500	70-28	70-28	64-22	-
Kentucky	I-64	20	40	10000	NO	70	-	2	9	-	-	11	-	-	200,000	-	76-22	76-22	-	-
	I-65	20	40	-	-	-	-	-	-	-	-	-	-	-	200,000	-	76-22	70-20	-	-
Maryland	I-695	12.5	30	15750	YES	-	2 SMA	1.5	12	-	-	15.5	6	12	30,000	8000	70-20	-	-	-
Michigan	US-24			-	YES	65	-	2.5	3	4.5	-	10	12	14	30,000	5,000	70-28	70-22	70-22	-
	I-96	20	40	8900	YES	65	-	1.5	2.5	10	-	14	16	12	30,000	12,000	76-22	76-22	70-22	-
	M-84			-	YES	65	-	1.5	2	3	-	6.5	12	-	-	5000	70-28	70-28	58-22	-
Minnesota	TH-71			-	NO	-	-	4.5	1.5	-	-	6	4.5	-	-	A-2-4	52-34	-	-	-
	TH-10			-	NO	-	-	3.5	3	3	-	9.5	6	18 SB	30,000	29,500	-	58-28	58-34	-
	TH-18			-	NO	-	-	4	3	1.5	-	8.5	4.5	10	30,000	29,500	52-34	52-34	-	-
	TH-61			-	NO	-	-	3	1	3	2	9	6	12	30,000	29,500	58-28	58-28	58-34	-
	I-35	16	30	864	NO	-	-	4	4	4	-	16	3	9	100,000	22,000	58-28	58-28	64-28	-
New Jersey	I-287	12	20	12000	NO	NO	-	2	2	7	-	11	8	10	30,000	32,000	76-22	76-22	64-22	-
New Mexico	US-70	8	30	1338	YES	YES														
Ohio	I-77			-	YES	60	-	1.5	1.75	10	4	17.25	6	-	30,000	-	76-22	76-22	58-28	58-28
	US-30	20	50	3747	YES	70	1.5 SMA	1.75	9	4	-	14.75	6	-	30,000	-	76-22	76-22	64-22	64-22
Oklahoma	SH-152	20	50	2000	YES	70	2 SMA	3	3	3	3	14	-	-	-	30,000	76-28	64-22	64-22	64-22
Oregon	I-5	15	30	12240	YES	70	2 OGFC	2	8	-	-	12	12	-	30,000	-	64-22	64-22	-	-
Texas	IH-35 (a)			-	YES	70	3 SMA	3	13	4	-	23	-	8	-	30,000	76-22	70-22	64-22	-
	IH-35 (b)			-	YES	70	3 SMA	3	8	2	-	16	-	8	-	30,000	76-22	70-22	64-22	-
	IH-35 (c)			-	YES	70	3 SMA	3	8	3	-	17	-	8	-	30,000	76-22	70-22	70-22	-
	IH-35 (d)			-	YES	70	1.5 OGFC	2	2	12	4	21.5	-	6	-	30,000	76-22	64-22	64-22	-
	IH-35 (e)			-	YES	70	1.5 OGFC	2	3	10	4	20.5	-	6	-	30,000	76-22	70-22	64-22	-
	IH-35 (f)			-	YES	70	1.5 OGFC	2	3	12	4	20.5	-	6	-	30,000	76-22	70-22	64-22	-
	SH-114	20	30	18000	YES	70	2 HDAMA	3	13	4	-	22	-	8	-	30,000	76-22	70-22	64-22	-
Virginia	I-95	-	-	2950	NO	-	1.5 SMA	2	17	-	-	20.5	3	6	30,000	30,000	70-22	64-22	-	-
Washington	I-90	18.5	50	5400	NO	NO	2 SMA	14	-	-	-	16	12	-	29,500	-	64-22	-	-	-
Wisconsin	STH-50 (a)	-	20	770	YES	-	-	2	3.5	3.5	-	9	4	8	29,500	30,000	58-28	64-22	64-22	-
	STH-50 (b)	-	20	770	YES	-	-	2	3.5	3.5	-	9	4	8	29,500	30,000	64-28	58-28	58-28	-
	STH-50 (c)	-	20	770	YES	-	-	2	3.5	3.5	-	9	4	8	29,500	30,000	58-28	70-22	70-22	-
	I-94 (a)	-	20	9476	YES	-	-	2	4.5	4.5	-	11	4	17	29,500	30,000	76-28	70-22	64-22	-
	I-94 (b)	-	20	9476	YES	-	-	2	4.5	4.5	-	11	4	17	29,500	30,000	70-28	70-22	64-22	-
Australia		-	40	-	YES		1.2 OGFC	1.6	4	Optional	3.5	10.3+	9-16	6	200,000	30,000	-	-	-	-
	Craigieburn Bypass	-	40		YES		1.6 OGFC	5	3	-	-	9.5	7	6	200,000	29,500				
	Vic Roads FDA	-	40		YES		1.6 OGFC	8	3	-	-	12.6	6	-	30,000	-				
United Kingdom	M-5	-	40	-	NO		SMA	-	-	-	-	12.5	-	-	-	-	-	-	-	-
China	Bin-Bo	-	-	-	YES		2 SMA	3	3.5-8	3.75	3	15-20	-	6	-	30,000	76-22	64-22	70-22	64-22
STATISTICAL ANALYSIS	Average (Mean) =	17	31	6303	-	68	2	3	5	6	3	14	7	9	61958	23203	-	-	-	-
	Standard Deviation =	3.77	12.15	5374	-	-	-	2.24	3.9	3.3	0.70	4.4	3.4	3.7	64737	10846	-	-	-	-
	Mode =	20	40	770	-	70	-	2	3	3	4	11	6	6	30000	30000	76-22	70-22	64-22	64-22

Table 3.9 – Current Field Performance

Pavement Section	% Alligator Cracking	Total Rut Depth (in)	IRI (in/mi)	Top-Down Cracking (ft/mi)	Age Since Rehab (yrs)
Maryland I-695	0	0.09	81	0	3
New Jersey I-287	0	0.11	84	0	7
New Mexico US-70	0	0.24	62	2.19	3
Ohio US-30	0	0.03	76	0	2.5
Oregon I-5	0	0.05	80	0	4
Texas SH-114	0	0.5	-	0	2
Washington I-90	0	0.2	63	0	8
Wisconsin I-94	0.1	0.00	-	890.6	5

Note: "–" = No data available.

3.5.3 Predicted Performance

The design life for California's I-710 is 40 years with surface rehabilitation expected every 19 years. MEPDG level 3 analysis indicates that the pavement sections will require surface treatment at the end 32 years. The AASHTO 1993 Design Guide predicted that the four sections on US-75 in Kansas have design lives of 6, 2.5, 6, and 10 years respectively (Romanoshi et al. 2008). Rehabilitation is scheduled on Maryland's I-695 every 12.5 years. The design life is 30 years. The expected design life for Michigan's I-96 is 40 years. No information on when the pavement is expected to receive rehabilitation was available.. No MEPDG level 3 analysis was conducted on pavement sections from Kansas, Maryland, and Michigan due to incomplete input data.

The perpetual pavement located on New Jersey's I-287 is expected to last 50 years since last rehabilitated in 1996, according to state DOT officials. MEPDG level 3 analysis shows that the pavement section will not perform satisfactorily for 50 years, with its first rehabilitation predicted at the end of 10 years, instead of 12 years. MEPDG level 3 analysis showed that none of the pavement sections on US-70 in New Mexico will perform satisfactorily for the respective design lives. The design life for Ohio's US-30 is 50 years with expected surface rehabilitation every 20 years. MEPDG level 3 analysis indicates that the pavement will perform satisfactorily for a 50 year design life.

Oklahoma's SH 152 was not designed with a specific design life. Oklahoma DOT expects the pavement to be resurfaced every 20 years and to last at least 50 years. MEPDG level 3 analysis indicates that the pavement section will perform satisfactorily for a 50 year design life. The design life for Oregon's I-5 is 30 years and expected resurfacing is every 15 years.

MEPDG level analysis indicates that the pavement will not perform satisfactorily for a 30 year design life, considering surface treatment will be required at the end of 10 years. The design life for Texas's SH 114 is 30 years and expected surface treatment is every 20 years. A study done by Walubita and Scullion (2007) using VESYS, PerRoad, and FPS 19W software analysis shows that the pavement section will require rehab at the end of 20, 30, 30 and 24 yrs respectively. MEPDG analysis indicates that the pavement meets expected performance criteria for a 30 year design life. However, the pavement section will not perform satisfactorily at the end of 50 years.

Washington's I-90 has a design life of 50 years and surface life of 18.5 years. The structural sections for I-90 are all intact and there has been no significant reconstruction to date. These pavement sections are approaching 30 years of service and are expected to continue to perform for at least another 20 years. MEPDG level 3 analysis indicates that the pavement sections will perform satisfactorily for a 50 year design life. Wisconsin's I-94 and STH 50 have a design life of 20 years. MEPDG level 3 analysis indicates that the pavement sections on I-94 will not perform satisfactorily for a 20 year design life as surface treatment is required within 5 years.

3.6 Summary

About 75% of the pavements investigated are performing as expected, as shown in Table 3.10. Of the pavement sections analyzed using MEPDG, 50% of them perform to their expected design life. In regards to moisture-related problems, the pavement sections highlighted from New Mexico, Oregon, and Texas suffer from high permeability and moisture damage. It must be noted that all three of these designs contain a RBL. However, mix design criteria of each pavement are not available. It would be interesting to see if there are similarities in their mix designs which might explain the high permeability and moisture damage found in the respective pavements.

Table 3.10 – Summary of Perpetual Pavement Findings

State	What was Designed	The Experience to Date	Concluion of the Performance to Date	Predicted Performance
California	Full-depth sections and overlays on I-710	I-710 is California's first PP to date.	The pavement appeared to be in excellent condition and "rode well"	Design life is 40 yrs. Surface life is 19 yrs. MEPDG analysis indicates that the pavement sections will require rehabilitation at the end of 32 years so as to perform satisfactorily for a 40 yr design life.
Kansas	4 Sections on US-75	The PP was developed as a field trial to investigate the perpetual concept	Pavement is performing as expected	The AASHTO 93 Guide showed that sections 1, 2, 3, and 4 on US 75 have design lives of 6, 2.5, 6, and 10 yrs respectively. No MEPDG analysis conducted due to incomplete data.
Maryland	I-695	I-695 is the only PP in Maryland, and this is their first PP project.	Pavement is performing as expected	Rehabilitation in 12.5 yrs. Design life is 30 yrs. Incomplete data for MEPDG analysis.
Michigan	3 Sections on US-24, I-96 and M-84	Michigan have built 3 PP sections beginning in 2002 and are currently building a fourth in 2008.	Performance is as expected	Design life is 40 yrs. Incomplete data for MEPDG analysis.
New Jersey	I-287	The original structure was a conventional AC pavement, but was rehabilitated as a perpetual pavement in 1993.	Currently, after 4 years of traffic, the project is performing to acceptable standards with little signs of distress.	The chief engineer for the Turnpike expects the pavement to last another 50 years. MEPDG analysis shows that the pavement section **will not** perform satisfactoily for 50 years. Rehabilitation will be required a the end of 10 yrs.
New Mexico	5 sections on US 70, and 3 sections on US 550.	First PP built in 1999, US 550, with another one designed in 2005, US 70.	The surfacing course was found to have high water permeability. FWD data indicates the subgrade strengths are lower than anticipated.	The AASHTO 93 Guide showed that US 70 sections 2 and 3 may not perform satisfactorily for the 30-year design life. MEPDG analysis showed that **none** of the pavement sections on either US 70 or US 550 **will perform** satisfactorily for the respective design lives.
Ohio	US 30 and I-77	First PP built was on I-77, in 2003, and US 30 was built in 2005.	Performance is as expected	Design life for US 30 is 50 yrs and surface life is 20 yrs. MEPDG analysis indicates that the pavement **will perform** satisfactorily for a 50 yr design life.
Oklahoma	SH 152	ODOT also built a test section at the NCAT test track	Performance is as expected. Visual surveys reveal minimal rutting with no signs of cracking.	The PP section was not designed with a specific design life. ODOT expect the pavement to be resurfaced at the end of 20 years and to last at least 50 years. MEPDG analysis indicates that the PP **will perform** satisfactorily for a 50 yr design life.
Oregon	I-5 near Albany, and I-205 near Oregon City	Oregon have built 2 PP between 2005 and 2008.	Pavement is performing as expected	Design life for I-5 is 30 yrs and surface life is 15 yrs. MEPDG analysis indicates that the pavement **will not** perform satisfactorily for a 30 yr design life.
Texas	SH 114, I-35 (7 sections)	Texas have 8 PP, the first one built in 2005	3 major problems have been identified with the PP sections. Vertical segregation, debonding, and moisture damage.	Design life for SH 114 is 30 yrs and surface life is 20 yrs.VESYS, PerRoad, and FPS 19 software analysis shows that the PP section will require rehab at the end of 20, 30, 30 and 24 yrs respectively. MEPDG analysis indicates that the pavement **meets expected performance criteria for a 30 yr** design life. However, the PP section **will not** perform satisfactorily for a **50** yr design life.
Washington	I-90	Washington's I-90 is the only PP built in the state.	Pavement is performing as expected	Design life is 50 yrs and surface life is 18.5 yrs. MEPDG analysis indicates that the PP section **will perform** satisfactorily for a 50 yr design life.
Wisconsin	STH 50 (3 sections), I-94 (2 sections)	First PP built in 2000, STH 50, and I-94 PP sections was built in 2002	In 2007, the pavement performance data is showing high top-down cracking on I-94 and high transverse cracking on STH 50.	Design life for I-94 and STH 50 is 20 yrs. MEPDG analysis indicates that the PP section will not perform satisfactorily for a 20 yr design life.

Chapter 4

EFFECTS OF MOISTURE ON PERPETUAL PAVEMENTS

4.0 Introduction

Water within pavement layers is one of the causes of pavement deterioration. Water-related problems are thus responsible for decreased pavement life, and increased costs for maintenance, and occur throughout all regions and climates of the US. New Mexico is certainly not immune to water related problems with sections of US-70 known to have moisture wicking problems. Moisture can infiltrate perpetual pavements in two ways; from the surface (top-down infiltration) and from beneath the pavement (bottom-up infiltration). This section focuses on how this issue is dealt with both here in the US and abroad, along with laboratory testing of asphalt mixes that are akin to New Mexico state highways. The effects of moisture are presented by strength and stiffness properties, which can be used as inputs of MEPDG.

4.1 Task 2A. Evaluate Top-down Moisture Infiltration

Top-down moisture infiltration is attributed to causing moisture damage of the HMA layers due to loss of the cohesive bond within the asphalt binder and/or the loss of the adhesive bond between the aggregate and binder. Top-down infiltration is accounted for by reducing the HMA layer stiffness. The reduced stiffness can be used as input to the mechanistic pavement analysis and the performance of the moisture infiltrated perpetual pavement can be evaluated.

4.1.1 Review of Top-Down Moisture Infiltration

A comprehensive literature search is conducted through Transportation Research Information Services (TRIS), Transportation Research Board (TRB), Research in Progress (RIP), UNM library, and interlibrary loans.

<u>Transportation Research Information Services (TRIS)</u>

Evaluation of the Extent of HMA Moisture Damage in Wisconsin as it Relates to Pavement Performance, Report No. WHRP 03-07(Kanitpong and Bahia 2003)
Wisconsin Highway Research Program conducted a study to evaluate the relationship between the performance of asphalt pavements in the field and the tensile strength ratio (TSR) values measured in the laboratory on the original asphalt mixtures used in constructing the pavements. The TSR is the ratio of wet tensile strength to dry tensile

strength. This ratio accounts for moisture damage in asphalt concrete. Analysis of TSR and pavement collection data indicated that there is a poor relationship between TSR and field pavement performance, especially pavement distresses that are known to be related to moisture damage such as surface raveling and rutting.

Transportation Research Record (TRB)

Moisture Susceptibility of Asphalt Mixtures with Known Field Performance – Evaluated with Dynamic Analysis and Crack Growth Model (Arambula et al. 2007)

This study evaluated the moisture susceptibility of asphalt mixtures with known field performance using dynamic analysis and a crack growth model to characterize the asphalt mixtures and corresponding asphalt mastics. The model parameters were obtained from surface energy measurements, uniaxial dynamic testing for the asphalt mixtures, and dynamic shear testing for the asphalt mastics. Results showed good differentiation between the moisture-conditioned (wet) and unconditioned (dry) specimen behavior and provided a good correlation with the reported field performance of the asphalt mixtures.

Estimating Directional Permeability of HMA by Numerical Simulation of Microscale Water Flow (Kutay et al. 2007)

All current permeability methods used by pavement engineers rely on measuring vertical permeability. However, the majority of HMA pavements have anisotropic and heterogeneous internal pore structure, which has a direct influence on the magnitude of permeabilities in different directions. Three-dimensional numerical modeling of fluid flow is a viable alternative to study the water transport characteristics of HMA pavements. Independent studies using numerical simulations concluded that the horizontal permeability was much higher than the vertical permeability because of the anisotropic and heterogeneous nature of air void distribution.

NCHRP-589 Improved Conditioning Procedure for Predicting the Moisture Susceptibility of HMA Pavements (Solaimanian 2007)

During the last several years, parallel to efforts for improving moisture damage tests, there has been significant research effort toward the development of a simple performance test (SPT) to complement the Superpave volumetric mix design method. The primary conclusion from the Phase I of the NCHRP 9-34 study was that the dynamic modulus test was the most suited of the three simple performance tests (static creep and repeated load permanent deformation tests were the other two tests).

Modeling Water Flow Patterns in Flexible Pavements (Hansson et al. 2004)
A study was undertaken to investigate the applicability of hydrological theories and methods to the road/environment system. The effect of rain intensity, precipitation amount, and/or fracture conductivity on the flow patterns inside the road was investigated using particle tracking. This study was concerned with mechanisms associated with water entering the roadbed. The recently modified numerical code, Hydrus 2D, is used to predict water flow paths in roads. The asphalt layer, except the fracture zone was assumed impermeable. The numerical simulations showed that the surface runoff and the infiltration capacity controlled the water flow.

Inclusion of Moisture Effect in Fatigue Test for Asphalt Pavements (Lu and Harvey 2007)
Conventional tests, such as the tensile strength ratio (TSR) test, do not fully simulate field conditions, in which traffic loading is an essential component. This paper developed a typical fatigue-based test procedure for comparative evaluation of moisture sensitivity of different mixes. It is a controlled-strain flexural beam fatigue test performed at 20°C, 10 Hz, and 200με on specimens pre-saturated under 635 mm-Hg vacuum for 30 minutes and preconditioned at 60°C for one day. Test results show that the fatigue based test procedure can distinguish mixes with different moisture sensitivities, and give a ranking of mixes consistent with prior experience.

<u>UNM Library (Compendex Plus)</u>

Evaluation of moisture damage in hot mix asphalt using simple performance and Superpave indirect tensile tests (Chen and Huang 2008)
An investigation was conducted to evaluate the moisture damage of dense-graded surface HMA mixture using simple performance test (SPT) and Superpave (TM) indirect tensile test (IDT). Specimens were conditioned using cycles of freeze thaw (ASTM D4867) and cycles of pore pressure pulses with a moisture induced stress tester (MIST). The dynamic modulus, Superpave IDT creep, resilient modulus and strength tests were performed on conditioned and unconditioned specimens. The results indicated that the dynamic modulus test and the Superpave IDT with the F-T or MIST conditionings were effective to characterize lab-measured moisture susceptibility of HMA mixtures. MIST was developed to simulate the repeated generation of pore pressure in saturated pavement under traffic load. The system supplies compressed air to load and apply vacuum to force water out and in through a HMA sample, which is saturated at a constant temperature.

4.1.2 Current State of Practice

Moisture Susceptibility of HMA Pavements

HMA moisture damage is a problem that is not unique to New Mexico. Moisture susceptibility is a primary cause of distress in hot mix asphalt (HMA) pavements. There is good evidence that moisture susceptibility is influenced by aggregate mineralogy, aggregate surface texture, asphalt binder chemistry, and the interaction between asphalt and aggregate (Solaimanian 2007). Two of the principal mechanisms that induce moisture damage in the asphalt mixture are advective flow and water diffusion (Arambula et al. 2007). Advective flow occurs when water flowing through the voids of the asphalt mixture causes desorption of the outer layers of the asphalt mastic, ultimately breaking the bond between the asphalt mastic and the aggregate. Diffusion occurs when water coming from an underground source or moisture from the environment permeates through the asphalt mastic, diminishing its cohesive bond strength. When the asphalt mastic coating the aggregate is completely displaced by water, stripping occurs.

Moisture Damage Prediction Tests

There are a great number of different aggregate mineralogies and numerous types of unmodified and modified asphalt binders used across the United States. If these factors are coupled with varied environmental conditions, traffic, and construction practices, this makes testing to accurately predict HMA moisture susceptibility a difficult task. The use of a test method to determine the potential of moisture damage in asphalt mixtures is used in specifications for highway construction nationwide. Numerous test procedures have been developed to evaluate HMA stripping potential in the laboratory. The most commonly used procedures include the boiling test, tensile strength test (TSR), static immersion, Lottman, modified Lottman, and Root-Tunnicliff tests. However, several disadvantages are associated with the current test methods, and the effectiveness of these procedures has been questioned.

A survey of 55 agencies (including 50 states) compiled by Colorado DOT (Solaimanian 2007) indicated that 39 agencies used a tensile strength ratio obtained from specimens tested with and without moisture conditioning to evaluate moisture sensitivity. According to the survey, AASHTO T283 was by far the most popular, with 30 agencies using this method. State highway agencies have reported mixed success with AASHTO T283, resulting in continued research to refine the procedures and to investigate other alternatives. Examples of such alternatives include the Hamburg Wheel Tracking Device (HWTD) and the Asphalt Pavement Analyzer (APA), which were introduced in the early 1990s. The HWTD has gained popularity as a moisture sensitivity test and has been the subject of several research projects.

The Strategic Highway Research Program (SHRP) extensively investigated mechanisms of moisture susceptibility and developed new methods for its prediction. The Environmental Conditioning System (ECS) was designed to determine the moisture susceptibility of compacted HMA specimens under conditions of temperature, moisture saturation, and dynamic loading similar to those found in pavements. The ECS test showed promise, but the visual stripping, permeability, and modulus procedures used in TP34 to evaluate moisture susceptibility gave results that were not any more precise or accurate than those of AASHTO T283. This led to a report by the National Cooperative Highway Research Program (NCHRP 9-34) who investigated whether combining a field-validated simple performance test (SPT) with an improved ECS procedure would offer an enhanced ability to predict moisture susceptibility.

The primary conclusion from NCHRP Project 9-34 was that the dynamic modulus test was the most suited of the three simple performance tests for possible use with the ECS in an improved moisture sensitivity test. The combined test procedure was able to differentiate between mixes made with stripping resistant aggregates and those with aggregates prone to stripping. The testing showed that the dynamic modulus decreases significantly when a moisture sensitive material is conditioned using the ECS conditioning procedure. However the ECS is an expensive and complex system, which limits its popularity. Table 4.1 presents dynamic modulus test results of HMA specimens from six different states. This testing was conducted at Pennsylvania State University in 2007. The dynamic modulus testing was conducted with a uniaxial sinusoidal load inducing approximately 100 $\mu\varepsilon$ in the specimen. All dynamic modulus tests were conducted at 25°C. Selection of the 25°C test temperature was based on the findings of research under NCHRP Project 9-29 (Solaimanian 2007). Table 4.1 presents the state DOT mix, along with the sample no. Also shown are the % air voids in the samples and degree of saturation of each sample. Testing frequencies vary from 1 – 25 Hz. Stress (kPa) and strain ($\mu\varepsilon$) measurements, and dynamic modulus values are recorded before and after saturation. A dynamic modus ratio (E^*_{wet}/E^*_{dry}) is then calculated based on these results. The retained dynamic modulus test results show a drop in modulus for all specimens after full conditioning. For a frequency of 25 Hz, it can be seen that HMA specimens from Oklahoma, Pennsylvania, and Wyoming could not be taken due to problems during testing. However, 3 out of the six HMA mixes showed retained modulus ratios of 85% or greater from testing at 10 Hz. The remaining three HMA mixes all have retained modulus ratios less than 80% at this frequency.

Table 4.1 Dynamic Modulus Test Results of State DOT Mixes conducted at the Pennsylvania State University

State DOT Mix	HMA Sample	Air Voids, %	Deg of Sat. %	Test Freq, Hz	Before Conditioning Stress (KPa)	Before Conditioning Strain (μe)	Before Conditioning Modulu (MPa)	After Conditioning Stress (KPa)	After Conditioning Strain (μe)	After Conditioning Modulus (MPa)	Moduli Ratio After/ Before
Georgia	307.26	7.6	75	25	599.9	102	5871	342.0	88	3878	0.66
				10	498.6	104	4815	258.7	87	2962	0.62
				5	422.5	104	4051	206.0	89	2324	0.57
				2	333.2	106	3148	160.5	98	1633	0.52
				1	260.0	103	2514	119.8	99	1208	0.48
Wisconsin	313.16	7.2	83	25	400.0	102	3921	277.3	92	3022	0.77
				10	290.2	99	2920	267.6	122	2188	0.75
				5	225.9	100	2256	173.6	110	1571	0.70
				2	152.0	99	1528	103.3	106	978	0.64
				1	111.9	100	1118	68.1	101	676	0.60
Kentucky	315.13	7.1	79	25	539.8	93	5805	451.8	86	5227	0.90
				10	429.4	94	4578	428.0	106	4053	0.89
				5	351.1	94	3734	342.4	107	3190	0.85
				2	283.7	104	2727	233.6	106	2205	0.81
				1	211.0	104	2030	173.0	107	1619	0.80
Oklahoma	316.21	7.1	67	25	NA	NA	NA	NA	NA	NA	NA
				10	558.4	97	5773	472.8	95	4953	0.86
				5	502.7	99	5078	412.5	97	4250	0.84
				2	404.7	98	4127	331.8	99	3351	0.81
				1	331.1	97	3397	275.2	101	2736	0.81
Pennsylvannia	317.11	7.0	80	25	NA	NA	NA	NA	NA	NA	NA
				10	484.2	99	4914	398.0	93	4278	0.87
				5	442.9	108	4092	335.1	98	3418	0.84
				2	335.9	109	3071	239.8	101	2381	0.78
				1	257.3	110	2335	174.0	98	1774	0.76
Wyoming	318.18	6.6	89	25	NA	NA	NA	NA	NA	NA	NA
				10	497.8	92	5400	317.2	95	3336	0.62
				5	430.7	94	4602	258.9	96	2699	0.59
				2	356.1	102	3504	205.6	103	1993	0.57
				1	282.7	100	2817	162.8	105	1546	0.55

Integrated System in Pavement Design

At present, there is no integrated system that accounts for dynamic modulus reduction due to moisture infiltration in the MEPDG. Integrating dynamic modulus test results on unconditioned and moisture-conditioned test specimens is the most promising option. Such an integrated system has the potential to allow moisture sensitivity to be considered in flexible pavement performance models.

4.1.3. Laboratory Investigation of Moisture Movement in Asphalt Concrete

Moisture infiltration is critical in understanding moisture damage in perpetual pavements and associated premature failure. An investigation was undertaken to study the behavior of moisture in asphalt concrete. From the literature reviewed, it is has been recognized that asphalt concrete has a wide range of hydraulic conductivities. Asphalt concretes with air voids above the range of 5 to 8% can possess a substantial saturated hydraulic conductivity due to interconnected void structure. Field measurements of the hydraulic conductivity of some asphalt concrete pavements with air voids greater than 8% indicate values well in excess of 10^{-4} cm/s, which is similar to sandy soils (Schmitt et al. 2007).

Below the surface, asphalt layers are not always saturated or dry. Infiltration, evaporation, and water retention within the surface layer all depend to some extent on the unsaturated state of the near-surface materials. The moisture characteristic curve and the unsaturated hydraulic conductivity function are commonly used to describe a material's unsaturated hydraulic characteristics, and are used in analytical and numerical solutions of near-surface water movement.

Measuring and applying unsaturated hydraulic characteristics of soils is well established, but there is virtually no information available as to the unsaturated hydraulic properties of asphalt concrete. Numerical analysis of water movement in pavement sections has assumed the asphalt concrete to be impermeable, with water entering via discrete cracks (Šimůnek et al. 2008).

The hydraulic properties of asphalt can play a major role in the design and performance of asphalt pavements if asphalt is considered a porous material rather than impermeable. For instance, surface and sub-surface drainage can be designed based on the amount of moisture entering (infiltration + storage) and leaving (run-off + drainage) a pavement system. In this way, a more accurate prediction of pavement design life can be provided that accounts for moisture damage due to water retained within the asphalt.

<u>Moisture Movement in Porous Media</u>

The mathematical model developed by van Genuchten (1980) describes water retention properties and unsaturated hydraulic conductivity, based on a relationship presented by Mualem (1976) which relates relative hydraulic conductivity to the moisture characteristic curve. van Genuchten (1980) related the volumetric moisture content to the pressure head with the following equation:

$$\theta = \left[\theta_r + \left(\theta_s - \theta_r\right)\cdot\left[\frac{1}{\left[1 + (\alpha\cdot h)^n\right]^m}\right]\right] \qquad \textit{Eq. 4.1}$$

where: θ_r = residual moisture content (dimensionless [L^3/L^3])

θ_s = saturated moisture content (dimensionless [L^3/L^3])

α= curve fitting parameter (1/L)

n = curve fitting parameter (dimensionless)

m = 1 - 1/n (dimensionless)

θ = volumetric moisture content (dimensionless [L^3/L^3])

h = pressure head (assumed positive for convenience [F/L^2])

and the hydraulic conductivity to the moisture content:

$$K = K_{sat}.\left[R_{wc}^{0.5}.\left[1 - \left[1 - \left(R_{wc}^{\frac{1}{8}}\right)\right]^m\right]^2\right] \qquad \textit{Eq. 4.2}$$

where: K = hydraulic conductivity (L/T)

K_{sat} = saturated hydraulic conductivity (L/T)

$R_{wc} = \Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r}$ = Reduced Water Content

These relationships predict the hydraulic conductivity for any given pressure head or moisture content, if the parameters θ_r, θ_s, α, n (referred to here as the van Genuchten parameters), and K_{sat} are known for that material.

In our study, laboratory testing was performed on various asphalt concrete samples which are prepared in accordance to NMDOT's mix design criteria, so as to measure the drying, moisture characteristic curve and the saturated hydraulic conductivity.

Moisture Retention Testing

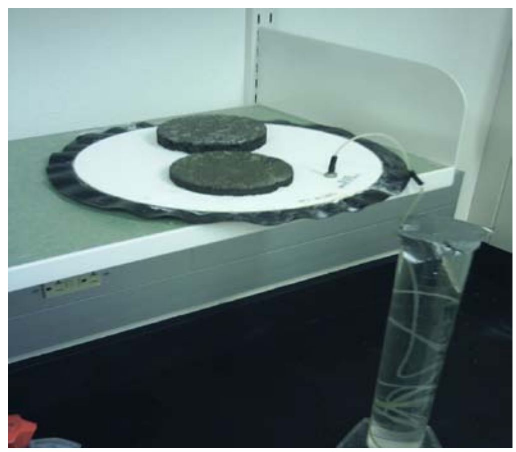
HMA samples were selected from mixes used in the city of Albuquerque (SP-II, SP-B and SP-C). Samples 1 and 2 were SP-C mixes and sample 3 was an SP-B mix. Sample 4 was an experimental mix of SP-II that contained small amount of fines (some material passing #4 sieves removed). Design air voids for these (Superpave) mixes are about 4%. Asphalt concrete samples were prepared in accordance with New Mexico Department of Transportation (NMDOT) mix compaction specifications. These samples (1, 2, 3, 4) were compacted to 6±1% air voids to represent field conditions. Moisture retention testing of asphalt samples was done using hanging-column and pressure plate methods in accordance with ASTM D6836. Moisture characteristic curves describe the relationship between suction and volumetric water content, gravimetric water content, or degree of water saturation. They are also referred to as water retention curves, water release curves, or capillary pressure curves.

Hanging Column Test

The hanging column test is suitable when applying suctions or negative pressure in the range of 0 to 15 kPa. This method is typically used to determine the beginning stages of the moisture characteristic curve for most soils. The pressure chamber test is suitable for applying suctions in the range of 0 to 1500 kPa. Both methods were combined, and used to provide a detailed description of the moisture characteristic curve of asphalt concrete.

4.1(a) Hanging Column Test Setup

A hanging water column consists of a water-saturated highly permeable porous ceramic plated connected to a water column or a reservoir. The test set up includes one ½ bar porous plate and two 3 inch diameter Buchner funnels. Two 4 inch diameter asphalt samples, with heights of 0.2 and 0.6 inch respectively, are placed on the ½ bar porous plate. The Buchner funnels contain two rectangular asphalt samples, approximately 3 inches wide and 0.75 – 1 inches high. Figures 4.1(a) and 4.1(b) illustrate the test set-up. The asphalt concrete (AC) samples were weighed initially and recorded. The samples were previously air dried using moisture absorbing desiccants. Each sample was then placed on a porous stone (porous plate and Buchner funnels) and the attached reservoirs were raised to a height equal to the top of the AC samples. In regards to the porous plate, the reservoir was raised to a height well above the height of the samples in order to maintain a positive pressure on the samples, as water will escape/flow off the edge of the porous plate. The samples were under a positive pressure for 3 days. The asphalt samples on the porous plate exhibit moisture on the top surface which is a good indication that they were at or near saturation. The porosity of the samples was also calculated so as to predict the weight of the saturated samples.



4.1(b) Hanging Column Test Set-Up Using ½ Bar Pressure Plate

Once saturated, the reservoirs were lowered to a new height, distance H, below the top of the plate, as shown in Figure 4.1(c). Matric suction was induced by reducing the pore water pressure while maintaining the pore gas pressure at atmospheric condition. By the equilibrium principle, water will flow from the soil samples through the ceramic plate to the reservoir until the total water potential of the system is constant. The potential of the free reservoir may be set equal to zero and at the soil sample height H we may write (assuming that z = 0; $P = P_{atm}$)

$$\psi_m + \psi_z = 0 = \psi_m + \rho_w gH \rightarrow \psi_m = -\rho_w Gh \qquad \textit{Eq. 4.3}$$

where ψ_m = matic potential, ψ_z = gravitational potential, ρ_w = density of water, g = acceleration of gravity, and H = height of reservoir. Diatomaceous earth slurry was used to ensure good contact between the sample and the porous plate as it is necessary for water to move between the sample and the porous stone. The samples equilibrated for 6 to 7 days and their weight was recorded thereafter. Water content was determined based on the change in mass or weight of the sample. The reservoir was then lowered further and another data point was obtained once equilibrium was reached. Suctions of 10, 30, 80 and 150 cm were applied to the samples repeating the above procedure.

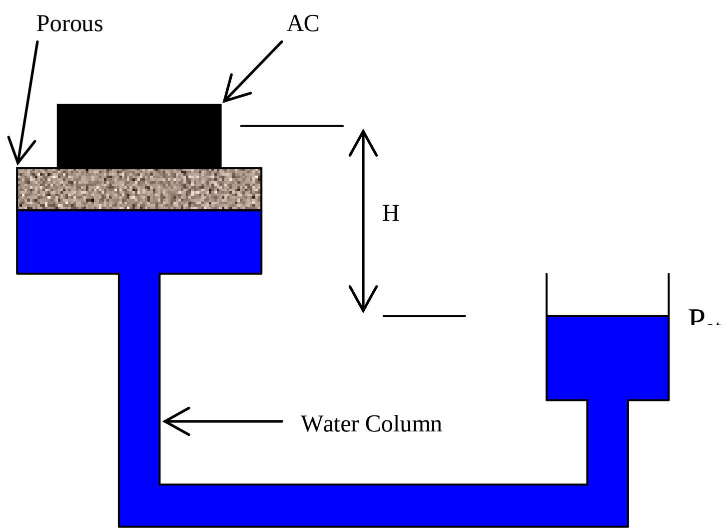


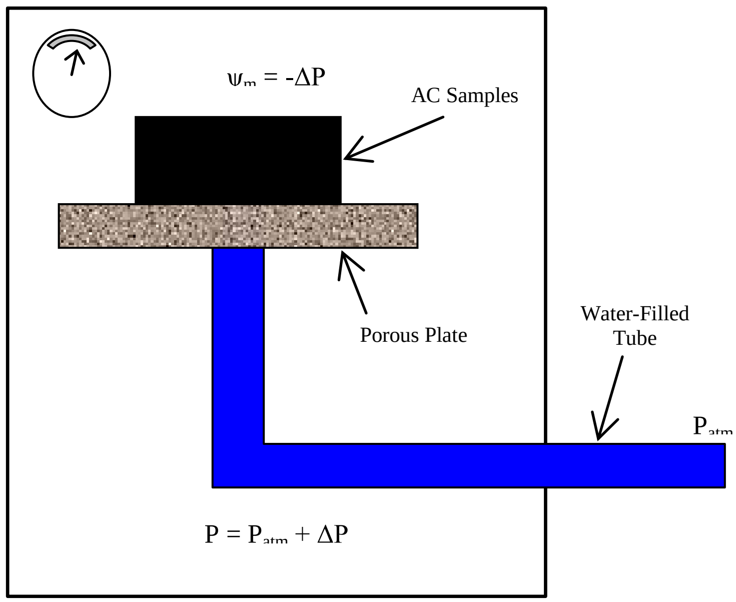
Figure 4.1(c) – Schematic of Hanging Column Test Set-Up

Pressure Plate Test

Figure 4.2(a) presents a schematic of the pressure plate test set-up. The pressure plate consists of an airtight chamber enclosing a porous ceramic plate connected on its underside to a tube that passes through the chamber to the open air, as shown in Figure 4.2(b). Saturated AC samples are placed in contact with the ceramic on the top side. The chamber is then pressurized, which causes water to flow from the asphalt concrete pores through the ceramic and out the tube. The pressure in the chamber is monitored using a pressure gauge illustrated in Figure 4.2(c). Once the system reaches equilibrium, flow through the tube will cease. The total potential may be set equal to zero at the point where the water exits the tube. Inside the plate we may write (assuming z = 0)

$$\psi_m + \psi_a = 0 = \psi_m + \Delta P \rightarrow \psi_m = -\Delta P \qquad \textit{Eq. 4.4}$$

where ψ_a = air pressure potential and ΔP = difference in pressure. When equilibrium is reached, the chamber may be depressurized and the sampled weighed. In this method an assumption is made that the matric potential of the sample does not change as the air pressure is lowered to atmospheric. Since these plates have a very high flow resistance, a substantial time may be required to remove water from the samples.



(a) Pressure plate diagram

(b) Samples in Pressure Chamber

(c) Pressure at 300 cm of Suction

Figure 4.2 – Pressure Plate Test Set-Up

Test Setup

24 hours prior to testing, the Buchner funnels and ½ bar pressure plates were placed under water so as to saturate them, as illustrated in Figure 4.3. When filling the reservoir with water, care was taken to ensure that the tubing was free from air bubbles that could rise into bottom portion of Buchner funnel or pressure plate. The water content and dry density of the samples should be known prior to placement on porous plate or in Buchner funnels.

Figure 4.3 – Buchner Funnels Soaking in Water

Results of Moisture Retention Testing

Saturated hydraulic conductivity testing on each of the four samples was done at D.B. Stephen's Laboratory under the guidance of Dan O'Dowd. The testing followed ASTM D-5084 Method C (falling-head, rising tailwater) standards for measuring hydraulic conductivity of saturated porous materials using a flexible wall permeameter.

Although testing followed the ASTM D-5084 standards, some modifications were made accordingly depending on unusual circumstances. Saturation of the asphalt concrete cores was assumed by calculating a B-value (see ASTM D5084, the ratio of a change in cell pressure to a change in sample pore pressure) greater than or equal to 0.95. Sample porosities varied from 7 to 18.5% and the saturated hydraulic conductivity varied from $4.74e^{-06}$ to $3.28e^{-04}$ cm/s. Table 4.2(a) presents the properties of HMA cores after saturation. Whether this is expected or not is unknown as there is no previous data for the porosity of these HMA mixes. The coarse mix has a porosity of 18.5%. Higher porosity is expected here due to the presence of larger aggregates, no fines, and thus more voids in the coarse mix.

Table 4.2(a) – Properties of HMA Cores after Saturation

Sample ID	Dry Weight (g)	Wet Weight (g)	Volumetric Moisture Content (θ)	Calculated Porosity %	Sat. Hydraulic Conductivity (Ksat) (cm/sec)
1	467.04	477.40	0.052	7.07	1.82 E-04
2	449.25	461.90	0.064	6.97	4.74 E-06
3	502.01	519.50	0.076	10.14	1.71 E-04
4	900.43	930.43	0.066	18.51	3.28 E-04

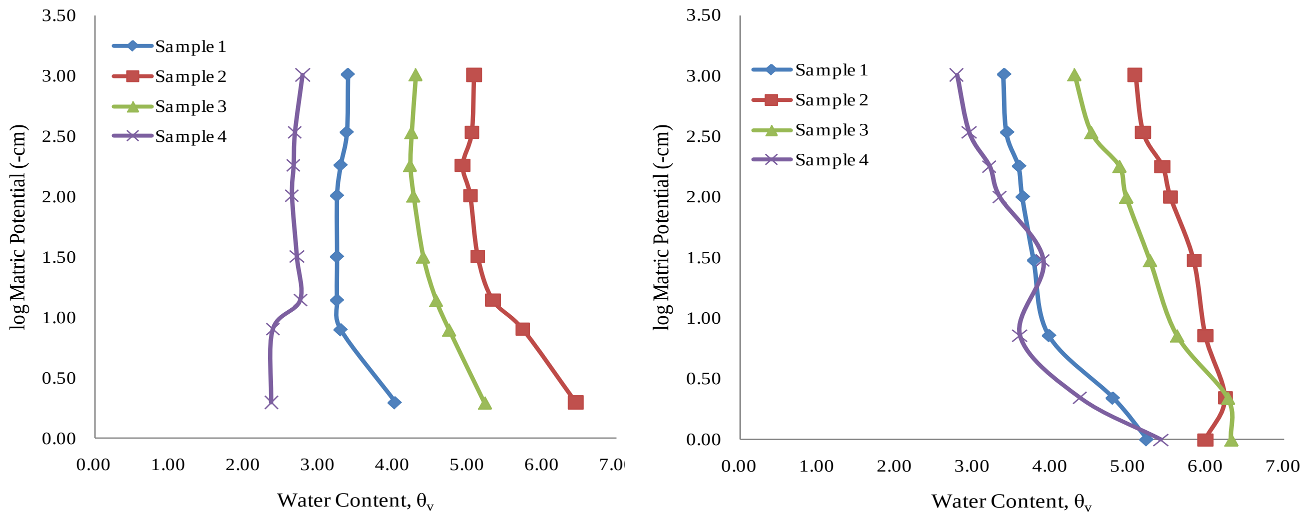
Table 4.2(b) presents sixteen data points have been collected from the moisture retention testing. The first eight data points describe the dry moisture characteristic curve and Figure 4.4(a) presents curves developed from this data. The dry moisture characteristic curve

represents the amount moisture in asphalt as it dries. The water content decreases as the matric potential (log) increases. HMA samples 1 and 4 lost a lot of moisture initially, but very little moisture was removed as the pressure (suction) was increased.

The last eight data points describe the 'wet' moisture characteristic curve of the asphalt samples and this curve is presented in Figure 4.4(b). The relative steepness of these curves indicate that the asphalt samples remained dry until positive pressure (suction) was applied as seen in samples 2, 3, and 4. This suggests that asphalt has a hydrophobic nature and will resist moisture infiltration unless positive pressure (head) is applied. More raw data is available in Appendix A.

Table 4.2(b) – Moisture Retention Data for Dry and Wet Curves

Date Checked	Time	ΔH Hanging Col. (cm)	ΔH Pressure Plate (bars)	Sample 1 + Membrane (g)	Sample 2 + Membrane (g)	Sample 3 + Membrane (g)	Sample 4 + Membrane (g)
5-Feb	10:55 AM	0	-	483.1	465.9	522.1	936.8
11-Feb	10:00 AM	-2.2	-	482.2	466.4	522	932
18-Feb	10:30 AM	-7.2	-	480.5	465.9	520.5	928.4
25-Feb	9:15 AM	-30	-	480.1	465.6	519.7	929.8
4-Mar	10:38 AM	-100	-	479.8	465	519	927.2
10-Mar	10:39 AM	-179	-	479.7	464.8	518.8	926.6
19-Mar	11:00 AM	-	0.333	479.38	464.3	517.97	925.38
24-Mar	10:55 AM	-	1	479.3	464.1	517.47	924.65
31-Mar	11:35 AM	-	0.333	479.27	464.04	517.34	924.18
7-Apr	12:35 AM	-182	-	479.1	463.8	517.3	924.1
15-Apr	11:00 AM	-102	-	479	464	517.4	924
22-Apr	11:47 AM	-32	-	479	464.2	517.7	924.3
28-Apr	10:18 AM	-14	-	479	464.6	518.1	924.5
7-May	10:10 AM	-8	-	479.1	465.4	518.5	922.8
15-May	11:24 AM	-2	-	480.6	466.8	519.6	922.7
22-May	3:45 PM	4.3	-	485	469.2	526.6	955.6
28-May	3:45 PM	10	-	484.6	470	527.4	952.7
			Relative Humidity Box Data (minus membrane wt.)				
28-May	10:45 AM	-	-	477.4	463.4	517.4	925.4
3-Jun	11:45 AM	-	-	472.4	457.6	509	905.3



(a) Dry Moisture Characteristic Curve

(b) Wet Moisture Characteristic Curve

Figure 4.4 Moisture Characteristic Curves of NMDOT HMA Samples

4.2. Task 2B. Evaluate Bottom-up Moisture Infiltration

Bottom-up moisture infiltration in perpetual pavements is viewed in this report to affect the subgrade soils only. A full literature review is presented on how highway agencies tackle this problem and the testing they use to determine the effect of moisture in this region of the pavement. The effect of moisture infiltration is also studied using the MEPDG through the seasonal changes in moisture content of subgrade soil. Integrated within the MEPDG, the Integrated Climatic Model (ICM) is used to simulate the changes in the subgrade soil properties due to moisture infiltration. In particular, the ICM simulations utilize climatic and groundwater conditions specifically for New Mexico.

4.2.1 Review of Bottom-Up Moisture Infiltration

<u>Transportation Research Record (TRB)</u>

Application of Soil-Water Characteristic Relationship in Estimating Load Bearing Capacity for Pavements (Fernando et al. 2008)
The Texas Department of Transportation (TxDOT) uses the modified triaxial design procedure to check pavement designs from the Department's flexible pavement system program. TxDOT pavement engineers have noted the design method's conservatism particularly in dry climatic areas of the state or where the soils are not as moisture susceptible. An investigation was conducted to verify the triaxial design method and accompanying modifications were made to take into account moisture effects based on the soil-water characteristic curve of the subgrade. Subsequent load bearing capacity results determined from the current modified triaxial design procedure were compared to full scale

plate bearing test results. Researchers observed that the current modified triaxial design method relatively underestimates pavement load bearing capacity.

Resilient Modulus as a Function of Soil Moisture – Summary of Predictive Models, (Witczak et al. 2000)

The objective of this TRB report was to select and then summarize existing models from the literature that incorporate the variation of resilient modulus with moisture. Using these published literature models it was then desirable to select a model or models that would analytically predict changes in modulus due to changes in moisture. This model (models) was considered for implementation in the 2002 Guide for the Design of New and Rehabilitated Pavement Structures.

Resilient Modulus as a Function of Soil Moisture – A Study of the Expected Changes of the Resilient Modulus of the Unbound Layers with Changes in Moisture for 10 LTPP Sites, (Witczak et al. 2000)

As part of the overall effort to develop a working, practical subsystem to predict resilient moduli (M_R) for unbound material throughout the life of a pavement system, the effect of moisture changes on M_R has been studied and evaluated. A simple model relating changes in modular ratio to changes in degree of saturation has been adopted to assess changes in the M_R values. This TRB report presents the results of the application of this simple model to 10 LTPP sites where moisture content variation data predicted by the Integrated Climate Model (ICM) are available. The results show that the seasonal variations in M_R (for non frost affected zones) are typically fairly small, of the order of +/- 10 to 15%. Oscillations as high as +/- 25 to 50% occasionally occur, but are not frequent. Overall, these seasonal oscillations appear to be much smaller than the oscillations expected from freezing and thawing.

Selection of Resilient Moduli for Frozen/Thawed Unbound Materials, (Witczak et al. 2000)

The purpose of this study was to derive reasonable values of M_R for both frozen and thawed unbound materials through evaluation of published results. The values of M_R for frozen materials were extracted as absolute values of M_{Rfrz}. For thawed materials the focus was on a reduction factor, RF, which could be multiplied times the unfrozen (normal) modulus, M_{Runfrz}, to get the modulus after thawing, M_{Rmin}. The following values were found to be reasonable for the material types indicated: coarse-grained materials – $M_{Rfrz} \sim 3x10^6$ psi; Fine-grained silts and silty sands – $M_{Rfrz} \sim 2x10^6$ psi; clays – $M_{Rfrz} \sim 1*10^6$ psi. Average values of RF and ranges in RF were found for gravel, sand, silt and clay. In consideration of all data collected it was possible to develop recommendations for RF as function of percent passing no. 200 sieve, P200, and plasticity index, PI. An algorithm is proposed for using these M_{Rfrz} and RF values in conjunction with ICM to produce time-varying values of M_R at a point, throughout the year.

Improvement for the Integrated Climatic Model for Moisture Content Predictions, (Witczak 2000)
The Enhanced Integrated Climatic Model (EICM) was designed to simulate the behavior of pavement materials and subgrade materials over several years of operation. An evaluation of the model's moisture prediction capabilities showed that its performance with regard to moisture predictions for the unbound materials was initially poor and exceeded the error typically found in field moisture measurements. Those findings pointed to the need for significant modifications and additions to the EICM moisture content prediction algorithms. The required modifications were subsequently made, creating version 2.6 of the EICM.

Modifications to the EICM included the addition of a better functional fit for the soil-water characteristic curve (SWCC); the incorporation of an algorithm capable of predicting the SWCC based on soil index properties; the addition of an algorithm for the prediction of the unsaturated hydraulic conductivity based on the SWCC; and, the development of sets of default soil parameters based on the ASSHTO Soil Classification System. Verification of the Version 2.6 showed great improvement on the prediction of the moisture content for the unbound materials.

UNM Library (Compendex-Plus Database)

Incorporation of Environmental Effects in Pavement Design (Zapata et al. 2007)
Currently a new independent review project is reviewing the climatic modeling tool called the Enhanced Integrated Climatic Model (EICM), which was implemented to incorporate the changes in temperature and moisture of unbound materials in the 2002 AASHTO design guide. The aim of this review is to correct errors and to develop further enhancements to produce a final methodology ready for acceptance by AASHTO in 2006.

Use of Ground Penetrating Radar to Diagnose Highway Structural Composition and Moisture Problems (Berthelot 2004)
A structural investigation was undertaken in Canada using ground penetrating radar (GPR) to determine if localized failures observed over recent years in areas of known ground water problems, was a result of structural composition and/or faulty substructure drain operation. The GPR survey results indicate in the vicinity of the drains, there is less moisture content which indicates that the drains are operating as expected. Thus confirming that the localized failures observed in the area was not a result of faulty operation of the subsurface drains.

ASCE Journal of Transportation Engineering

Comprehensive Monitoring Systems for Measuring Subgrade Moisture Conditions (Rainwater et al. 1999)

Four sites across the state of Tennessee were instrumented with comprehensive monitoring systems that collect subgrade water content, infiltration, and temperature data. These data will be used to develop a rational method to account for environmental effects (e.g., seasonal changes in subgrade water content, in flexible pavement design). Tension-free pan lysimeters were installed at three of the test sites to measure infiltration through the pavement layers. The comprehensive monitoring systems detect small changes in subgrade water content. Asphalt layers below the surface layer and binder layer are permeable and will allow significant amounts of infiltration into the subgrade if left uncovered.

Ground-Penetrating Radar for Cold In-Place Recycled Road Systems (Berthelot et al. 2001)

Due to the increase of commercial truck traffic on many Saskatchewan roads, the Saskatchewan Department of Highways and Transportation is investigating cold in-place recycling as a rehabilitation alternative. However, different construction practices and years of maintenance and rehabilitation have led to many of these thin-paved roads having varied structural composition. The materials and structural design of cold in-place recycled thin-paved road systems can be highly uncertain. However, ground-penetrating radar (GPR) can be used as an engineering diagnostic tool that can determine in-situ structural composition and help reduce the proposed uncertainty of cold in-place recycled pavements. A summary of the principles of ground-penetrating radar is presented and well as discussions about the use of GPR as an engineering diagnostic tool for cold in-place recycled pavements.

4.2.2 Current State of Practice

Moisture Susceptibility of Subgrade

Subgrade resilient modulus is highly dependent on water content, which can vary significantly with a number of environmental factors. High subgrade water content, with the resulting decrease in subgrade strength and stiffness, is detrimental to roadway pavement response. Although the variation in subgrade water content and the effects on pavement response have been investigated for some time, the magnitude of these variations and the relationships involved are not yet well understood (Rada et al. 1994). Subgrade moisture content changes due to infiltration and capillary rise of moisture from high groundwater tables. Because of difficulties in modeling infiltration through a pavement system, changes in subgrade modulus are often assumed to occur only as a result of changes in the water table elevation, ignoring infiltration.

Subgrade Moisture Damage Prediction Tests

In current Texas DOT practice, the modified triaxial method requires the triaxial class of the subgrade as derived from laboratory test results. The Texas triaxial class (TTC) is determined based on triaxial test results on samples that undergo moisture conditioning by capillary saturation. Depending on the Mohr-Coulomb failure envelope obtained from the tests, materials were categorized into different classifications. Higher TTCs correspond to lower quality subgrade materials requiring thicker pavements due to lower subgrade shear strength (Fernando 2008).

The time domain reflectometry (TDR) method of monitoring subgrade water was introduced to pavement engineering around 1989. A TDR measurement system is a nondestructive test that measures soil water content. The principle of TDR is to relate the dielectric response of a soil to its water content (Ekblad 2006). Ground penetrating radar (GPR) has also been used extensively as in-situ method to identify regions of high moisture content in pavement systems. GPR is also a non-destructive test that sends discrete pulses of radar energy into the pavement system and captures the reflections from each layer interface within the structure. The amplitudes of reflection and the time delays between reflections are used to calculate both layer dielectrics and thickness. The dielectric constant of a material is an electrical property that is most influenced by moisture content and density. An increase in moisture will cause an increase in layer dielectric (Scullion 2006).

Integrated Climatic Model

MEPDG uses the Integrated Climatic Model (ICM) to simulate changes in subgrade soil properties due to moisture infiltration. The ICM determines the water distribution in a pavement system by taking into account the factors that cause changes in the moisture content, which includes weather information, groundwater depth, and drainage properties. The weather information includes the air temperature which is required by the heat balance equation in the ICM to define the frozen/thawing periods within the analysis time-frame, and to determine the number of freeze-thaw cycles. Precipitation, which is required to compute infiltration, is also included in the weather information. The groundwater depth is an estimate of the annual depth or the seasonal average depth. The drainage properties include the following parameters:

- Infiltration potential – none, minor (<10%), moderate (<50%), extreme (100%) of precipitation enters pavement. Based on this input, the EICM determines amount of water available on top of the first unbound layer.

- Drainage path – Distance of pavement slopes (cross and longitudinal). This input is used in the EICM's infiltration and drainage model to compute time required to drain an unbound layer from an initially wet condition.

- Pavement Material Input Parameters – Saturated hydraulic conductivity which is required to determine the transient moisture profiles in compacted unbound materials and to compute their drainage characteristics.

ICM simulations also uses an adjustment factor, F_{env} that accounts for the effects of environment conditions such as moisture content changes, freezing, thawing, and recovery from thawing. The adjustment factor, F_{env}, can vary with position (within the pavement) and time (throughout analysis period). It is multiplied by the M_{Ropt}, which is the resilient modulus at optimum conditions and at any state of stress, to obtain the resilient modulus, M_R as a function of position and time. The M_R is expressed as:

$$M_R = F_{env} * M_{Ropt} \qquad \textit{Eq. 4.5}$$

where F_{env} is considered a function of the environmental factors and can be computed by EICM without actually knowing M_{Ropt}. Once the EICM generates the required information, the following outputs (in regards to moisture distribution) are generated for use by other components of the MEPDG software:

- Composite environmental adjustment factors, F_{env}, are computed for every sublayer at each node. These factors are sent forward to the structural analysis modules where they are multiplied by M_{Ropt} to obtain M_R as function of position and time.
- An average value of moisture content for each sublayer is reported for use in the permanent deformation model for the unbound materials.

Chapter 5

PEPRPETUAL PAVEMENT DESIGN ALTERNATIVES

5.0 Introduction

In this chapter, design alternatives are developed based on a test matrix of varying design parameters used as inputs to MEPDG. Traffic volume, climate, and pavement distress criteria are selected to represent traffic and climatic conditions of New Mexico. Pavements that pass the performance criterion are considered perpetual pavements. Flow charts of how these design alternatives achieve perpetual status are presented herein. Detailed information of the successful perpetual pavements that pass the fatigue and rut criteria is also provided. In addition, HMA mixes used in successful perpetual pavements are analyzed with particular emphasis on performance in the intermediate and rich binder layers. Traffic data for the state of New Mexico is presented along with Life Cycle Cost Analysis (LCCA) of selected perpetual pavements.

5.1 Task 3A. Develop Design Alternatives Based on Layer Stiffness and Thickness

In this subtask, the optimum pavement structure that gives highest performance (i.e. low rut and fatigue) is studied using MEPDG. Design trials are created based on layer stiffness and thickness. The optimum perpetual pavement structure is found by varying the following parameters: (i) thickness, (ii) typical NMDOT mix design and (iii) PG-Grade binders. A test matrix is devised to combine these parameters as input to MEPDG.

5.1.1 Selection of Test Matrix

A test matrix was devised to combine these parameters as level 3 input to MEPDG. Table 5.1(a) presents the test matrix. This test matrix was created to determine how a perpetual pavement performs based on the individual layer thicknesses, HMA mix design, and PG-binder grades. Traditionally, NMDOT allows only one type of materials in the untreated base course, which are granular materials or granular base (GB). The granular base thickness considered for the first 1872 trial runs was 10 in. The thickness of the granular base in New Mexico traditionally varies from 8 to 12 in. Later, based on recommendations from the project panel (Project Panel 2008), the NMDOT granular base thickness was set to 6 in. The thickness of the treated subgrade layer is 12 in.

Table 5.1(a) – Parameters Used for MEPDG Test Matrix

Layer Type	Layer Thickness (in)	Mix Design	PG Binder
Surface, T_1	1.5 2 - 2.5 3	SP-III SP-IV	76-22 70-22
Intermediate	15-T_1-T_3 15-T_1-T_3-2 15-T1-T3-4	SP-II SP-III SP-IV	76-22 70-22
Rich Binder Layer (RBL), T_3	3 5 7	SP-II SP-III SP-IV	64-22
Granular Base	6 10	A-5	NA
Treated Subgrade	12	A-5	NA

Note: NA = Not Applicable, SP = SuperPave

Review of NMDOT projects reveal that most of the NMDOT flexible pavement thicknesses fall below 15 inches. Therefore, the maximum thickness of HMA layer was set to 15 inches. The surface layer thickness varies from 1.5 to 3 inches. As discussed in the literature, the RBL thickness varies from 3 to 7 inches all over the United States. A layer thickness of 5 inches is the mean thickness taken from 39 perpetual pavement data collected. From this data the standard deviation is also calculated and used to determine the range of thicknesses of the rich-binder layers. The thicknesses of the intermediate layer are calculated based on the total HMA layer thickness minus the surface and RBL layers. An additional 2 and 4 inches. are subtracted from the resulting intermediate layer thickness to account for total thickness of 11 and 13inches. Adjustments are made here to reduce the total thickness so as to produce more feasible alternatives for NMDOT according to the Project Panel. Pavements with high intermediate layer thicknesses ($\geq$ 10 inches) are reduced to 6 inches and pavements with low intermediate layer thickness (< 10 inches) are reduced to 4 inches.

Other parameters that are varied in the matrix are the mix design and the PG-binder grade. Two modified PG-binder are used in the surface and intermediate layers. These are PG 76-22 and PG 70-22 binders. PG 64-22 is used in the rich binder layer for flexibility. The test matrix contains 3213 runs. The number of runs is calculated using the formula below:

$$\begin{aligned}\text{Test Matrix} &= \text{Surface} \times \text{Intermediate} \times \text{RBL} \times \text{Mix Design} \times \text{PG Binder} \\ &= (T_1 \times 3) \times (T_3 \times 3) \times (15 - T_1 - T_3) \times (13 - T_1 - T_3) \\ &\quad \times (11 - T_1 - T_3) \times (Mix_1^2) \times (Mix_2^3) \times (Mix_3^3) \times (PG_1^2) \times (PG_2^2)\end{aligned}$$

where T_1 = thickness of surface layer, T_3 = thickness of rich binder layer, 15-T_1-T_3 = thickness of intermediate layer, Mix_1 = SP-II, Mix_2 = SP-III, Mix_3 = SP-IV, PG_1 = PG 70-22, and PG_2 = PG 76-22, and SP = SuPerPave (Superior Performing Pavement).

Table 5.1(b) shows Superpave mix gradations used by NMDOT. Mix SP-II is a coarse mix. Mixes SP-III and SP-IV are fine mixes. As it can be seen from Table 5.1(b), mix SP-IV contains higher percentage of fine aggregates (% passing < #200 sieve) than mix SP-III. Therefore, only SP-III and SP-IV mixes are considered for the surface course. The intermediate and RBL layers use mix type SP-II, SP-III, and SP-IV. The percentage air voids and effective binder content are also shown in Table 5.1(b). They are set to the criterion specified by the NMDOT for these mixes. The surface and intermediate HMA mix designs contain 6% air voids. In New Mexico, HMA mixes are compacted at 6±1% air voids in the field but designed at 4±1% air voids. The RBL contains 3% air voids. In general, rich binder layers use a higher percentage of binder than traditional surface mix. The extra binder fills the voids in the mineral aggregate and thus creates a low air-void mix. Effective binder content, instead of total binder content, is an input to MEPDG Version1.0. The effective binder content used in each of the HMA mix designs is calculated using the equations shown below (Roberts et al. 1996);

$$G_{se} = \frac{100 - P_b}{\frac{100}{G_{mm}} - \frac{P_b}{G_b}} \qquad \textit{Eq. 5.1}$$

$$V_{be} = G_{mb} \left[\frac{P_b}{G_b} - (100 - P_b)\left(\frac{G_{se} - G_{sb}}{G_{se} . G_{sb}}\right)\right] \qquad \textit{Eq. 5.2}$$

where G_{se} = effective specific gravity of the aggregate , P_b = binder content by weight, G_b = specific gravity of the binder, G_{mb} = bulk specific gravity of the mix, G_{mm} = maximum theoretical specific gravity of the mix, G_{sb} = combined bulk specific gravity of the aggregate, and V_{be} = the effective binder content by volume. These data were collected from the mix design specifications provided by the NMDOT and used in the equations above to calculate volumetric binder content. In order to reduce potential rutting, adjustments are made to increase the amount of coarse materials in the asphalt mixes. Also the percentage air voids and asphalt content in the asphalt mixes are reduced. Table 5.1(b) shows the changes made to the mix gradations are highlighted in red and in parenthesis. However, these changes were made to reduce AC rutting using MEPDG only and are not considered recommendations for NMDOT mixes.

Table 5.1(b) – NMDOT HMA Mix Gradations

Mix Design	Percent Passing Sieve Size						% Asphalt Content (Volumetric)	% Air Voids	Lift Thickness Range (in)
	1″	3/4″	1/2″	3/8″	#4	#200			
SP-II	95	85 (90)	-	55 (60)	33 (23)	4	9 (8.5)	6 (5,4)	3 - 3.5
SP-III	100	97 (100)	90	65 (70)	41 (33)	5	10.5 (10)	6 (5,4)	2.5 - 3.5
SP-IV	-	100	95	75 (80)	45 (40)	5.5	11.5 (11)	6 (5,4)	1.5 - 3
Granular Base	Stiffness, E = 20,000 psi						NA	7	6. - 12
Treated Subgrade	Stiffness, E = 8,000 psi (E = 16,000 psi)						NA	5	10. - 12

Note: SP = SuperPave, NA = Not Applicable

5.1.2 Simulations Through MEPDG

The MEPDG Version 1.0 is used as a perpetual pavement evaluation tool. The MEPDG is based on mechanistic-empirical principles, where it assumes that pavement can be modeled as a multi-layered elastic structure. There are three levels of inputs in the MEPDG analyses. In Level 1, materials properties such as dynamic modulus of asphalt concrete and resilient modulus of soils and aggregate are obtained from laboratory tests. In Level 2, these properties are determined using existing or local correlation equations. In Level 3, the dynamic and resilient moduli are calculated from index properties such as soil classification, plasticity, aggregate gradation, binder content, etc. using uncalibrated or nationally calibrated correlations or equations. In this study, Level 3 inputs are used to determine optimal perpetual pavement structure.

Traffic and Climatic Data

The annual average daily truck traffic (AADTT) used is 1750, 5000, and 10000 with a truck traffic classification (TTC) factor of 1, which considers predominantly single trailer trucks (Class 9 traffic). Traffic growth is similar to typical New Mexico interstate traffic growth which is 4%. Two lanes and 50 percent trucks in the design direction, operational speed of 70 mph, and vehicle tire pressure of 120 psi were used. Climate conditions play a major role in the performance of perpetual pavements. Pavements located in arid/semi-arid regions require specific design criteria to withstand the extreme temperature changes. The climatic data used in this study is taken from the weather station at Albuquerque, New Mexico. The water-table depth is set to 10 ft.

Base and Soil Input Data

The granular base consisted of compacted crushed gravel with a resilient modulus (M_R) of 20000 psi. The treated subgrade consists of compacted A-5 material with a M_R of 8000 psi. The research team decided to increase the resilient modulus (M_R) of the treated subgrade (TSG) which is conservative considering the natural subgrade has a resilient modulus of 5000 psi. The MEPDG does not include a special type of material for treated subgrade, so this layer is treated as sandy soil (A-4) with an increased modulus of 16,000 psi. The natural subgrade also contains A-5 material with MR of 5000 psi.

Distress Criterion

MEPDG level 3 analysis is calibrated to nationwide standards. However, it would be more beneficial if level 1 analysis is used where the MEPDG is calibrated to meet local highway standards. Unfortunately, this has yet to be done in New Mexico so level 3 analysis was performed. The pavement performance criteria are shown in Table 5.2. Pavements are considered failed when the predicted distress is equal to these target distress values. The MEPDG predicted results are analyzed based on surface-down cracking, fatigue cracking, AC rutting, total rutting, and IRI over a period of 50 years. In this study however, priority is given to fatigue (bottom-up) cracking and asphalt concrete (AC) rutting. Indeed, the definition of a perpetual pavement is that a pavement having no bottom-up fatigue failure. Fatigue cracking is a major contributor to pavement failure and requires costly reconstruction. The subgrade rutting-model in MEPDG over-predicts rutting so less weight is given to the total rutting predicted by MEPDG (AASHTO 2008). Also, subgrade rutting can easily be addressed with stabilization of the soils with additives such as lime, fly-ash, or cement. International Roughness Index (IRI) is not considered in the results as the use of OGFC or SMA, which is not optional in MEPDG, on top of an optimum perpetual pavement, reduces potential IRI. In this study, the predicted distress at the end of 50 years is directly compared to the target distress values shown in Table 5.2. These target distress values are recommended by the MEPDG for primary highway routes. Ideally, one should consider the reliability of distress model which is not considered in this study.

Table 5.2 – Performance Criteria for HMA Pavements on Primary Routes

Performance Criteria For Primary Roads	Max. Value
Alligator Cracking (% of Lane Area)	20
Total Rutting (in)	0.50
10 Year AC Rutting (in)	0.25
Thermal Cracking (ft/mi)	700
Surface-Down Cracking (ft/mi)	700
IRI (in/mi)	200

MEPDG Options Used

There are options available in MEPDG when considering fatigue cracking. One option is including a fatigue endurance limit for the pavement. A fatigue endurance limit considers the tensile strain experienced at the bottom of the HMA layer under traffic loading. If the tensile strain remains below the endurance limit, the pavement will have an infinite fatigue life. The fatigue endurance limit of NMDOT mixes are not known yet. Therefore, this study does not use fatigue endurance limit criteria. Rather, it uses the nationally calibrated fatigue model which predicts the percentage fatigue cracking. However this version of MEPDG does not give the "value of strain" at the bottom of the HMA layer. Therefore, the strain value is not reported in this report.

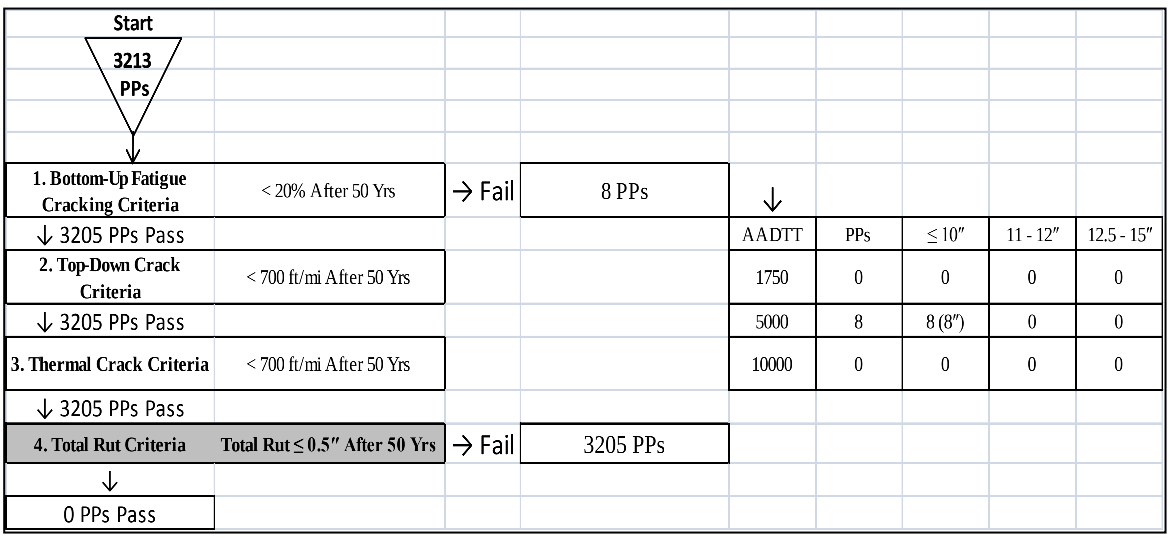
5.1.3 Analysis of Perpetual Pavements Using Flow Charts

Using the inputs shown in Tables 5.1(a) – (b) and performance criteria in Table 5.2, MEPDG simulations are run for 50 years. From the 3213 simulations run, none of the pavements experienced any thermal cracking over a period of 50 years. The use of modified binders and the location of the climate data (Albuquerque) might reduce the impact of thermal cracking on the pavements analyzed. None of the 3213 pavements failed by surface-down cracking (> 700ft/mi). Thus, further investigation falls into the category of rut and bottom-up (fatigue) cracking, which is done through flow charts in the next two sections.

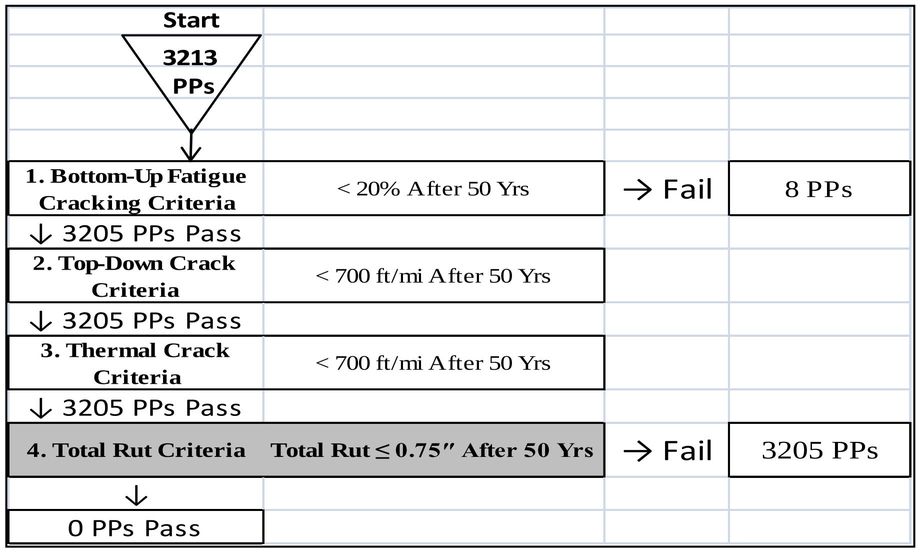
A. Perpetual Pavements with No Rehabilitation

In this section, perpetual pavements are identified that last 50 years without requiring rehabilitation. A flow chart of 3213 perpetual pavements is created based on the performance criteria mentioned earlier and is illustrated in Figure 5.1(a). It can be seen that when fatigue cracking criteria of≤ 20% at the end of 50 years is applied, only 8 pavements fail. Interestingly, thin pavements failed at relatively high annual average daily truck traffic (5000 AADTT). It can be noted that AADTT of 10000 was not applied to pavements with asphalt concrete (AC) thickness < 10″. Top-down and thermal cracking criterion (≤ 700 ft/mi) is then applied to the remaining 3205 pavements and here it can be seen that none of the pavements failed by this performance criterion. However, when total rut criterion of ≤ 0.5″ is applied, none of the pavements pass. Figure 5.1(a) illustrates that using total rut criterion of ≤ 0.5″ and fatigue cracking ≤ 20%, no perpetual pavements can be found. The 8 eight pavements which failed by bottom-up cracking all have 8″ asphalt thickness and carry 5000 AADTT. None of these pavements contain a rich binder layer.

As described above, no pavements passed for total rutting. As a next step, the performance criterion is lowered from 0.5 to 0.75″. Once again, none of the pavements passed this criterion as Figure 5.1(b) illustrates.



(a) Total Rut ≤ 0.5″ after 50 Years



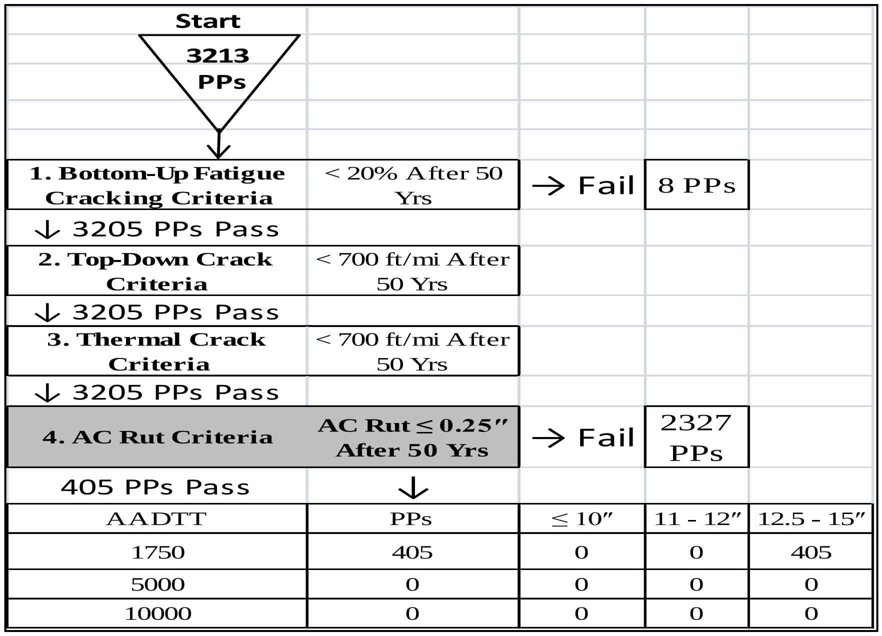
(b) Total Rut ≤ 0.75″ after 50 Years

Figure 5.1 – Pavement Performance Flow Charts Based on Total Rut Criteria (No Rehab)

Since none of the pavements passed the performance criterion for total rutting (AC + base + subgrade), priority is given to those pavements that pass by AC rutting. Pavements that pass this criterion can have their subgrade material modified to prevent rutting in this layer. Hence, Figure 5.2(a) presents a flow chart which screens 3213 perpetual pavements based on AC rut criterion≤ 0.25″ at the end of 50 years. It can be seen that about 405 perpetual pavements pass for AC rut≤ 0.25″ and fatigue cracking ≤ 20%. However, all of these pavements have 15″ AC thickness for AADTT of 1750. No perpetual pavements can be found using 5000 and 10000 AADTT.

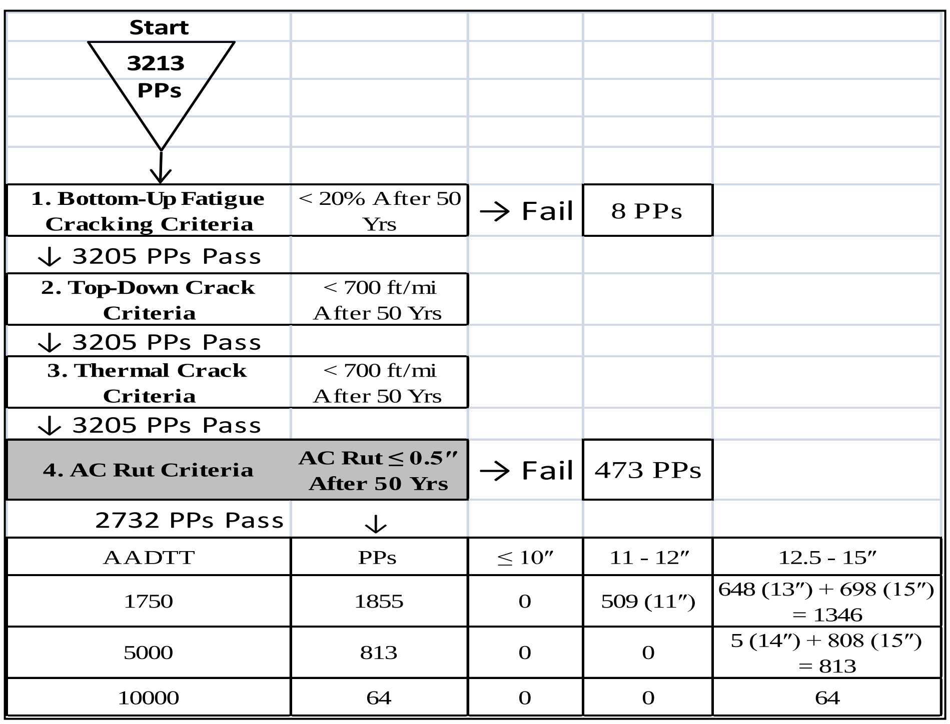
AC rut failure criterion is increased from 0.25 to 0.5″ at the end of 50 years. Based on a flow chart plotted in Figure 5.2(b), additional pavements can now be considered for further analysis. For AC rut≤ 0.5″ and fatigue cracking ≤ 20%, perpetual pavements can be found for the following;

- For 1750 AADTT – 509 have 11″ and 648 have 13″ HMA thickness
- For 5000 AADTT – 5 have 14″ and 808 have 15″ HMA thickness
- For 10000 AADTT – 64 have 15″ HMA thickness



(a) AC Rut ≤ 0.25″ after 50 Years

Figure 5.2 – Pavement Performance Flow Charts Based on AC Rut Criteria (No Rehab)



(b) AC Rut ≤ 0.5″ after 50 Years

Figure 5.2 – Pavement Performance Flow Charts Based on AC Rut Criteria (No Rehab)

Summary

Table 5.3 presents perpetual pavements with 14″ HMA thickness and AADTT of 5000. All 5 pavements have a 3″ surface layer, 4″ intermediate layer, and 7″ RBL. All of these pavements use PG 76-22 in their surface layers. All of these pavements have 8000 psi resilient modulus (M_R) in the treated subgrade layer.

Table 5.3 – Design Criteria of 14″ Perpetual Pavements Carrying 5000 AADTT

		Layer Thickness (in)					Mix Design			PG Binder			50 Yr. MEPDG Predicted Distress			10 Yr. Predicted Distress		
Run	Traffic (AADTT)	Surf	Itmd.	RBL	Total	GB	Surf. Mix	Itmd. Mix	RBL Mix	Surf. PG	Itmd. PG	Treated SG M_R (psi)	Top-Down Cracking (ft/mi)	Bottom-Up Cracking (%)	Thermal Cracking (ft/mi)	IRI (in/mi)	AC_1 Rut (in)	AC_2 + AC_3 Rut (in)
2730	5000	3	4	7	14	6	SP-IV	SP-II	SP-IV	76-22	76-22	8000	0	0.208	0	96.7	0.09	0.02
2731	5000	3	4	7	14	6	SP-IV	SP-III	SP-III	76-22	76-22	8000	0	0.362	0	97.9	0.09	0.02
2732	5000	3	4	7	14	6	SP-IV	SP-III	SP-IV	76-22	76-22	8000	0	0.308	0	98.0	0.09	0.02
2716	5000	3	4	7	14	6	SP-III	SP-IV	SP-II	76-22	70-22	8000	0	0.458	0	97.5	0.08	0.03
2717	5000	3	4	7	14	6	SP-III	SP-IV	SP-III	76-22	70-22	8000	0	0.358	0	97.7	0.08	0.03

B. Perpetual Pavements with Rehabilitation Included

Figure 5.3(a) presents the same performance criteria (AC rut ≤ 0.25″) as shown in Figure 5.2(a) but this time rehabilitation is considered every 10 years, if needed. Rehabilitation is required if pavements show AC rutting more than 0.25″ at the end of 50 years. Therefore, every 10 years pavements are allowed to have 0.05″ AC rutting. Thus at the end of 20 years, pavements are allowed to have 0.1″ AC rutting. Pavements failing this performance criterion need resurfacing (AC_1 rut → 0) and combined rutting in the intermediate and base layers will be monitored for having $AC_2 + AC_3 \leq 0.05''$ rutting every 10 years. At the end of 30 years, pavements are allowed to have 0.15″ rutting and so on. This criterion is checked against AC rut of intermediate and base AC layers but not for surface layers. This is logical as the surface layer is expected to have some treatment or resurfacing (open-graded friction course) at 10 year intervals to maintain surface IRI and smoothness criteria. Therefore, in the analysis, the rut of the AC layer is set to zero after each rehabilitation cycle.

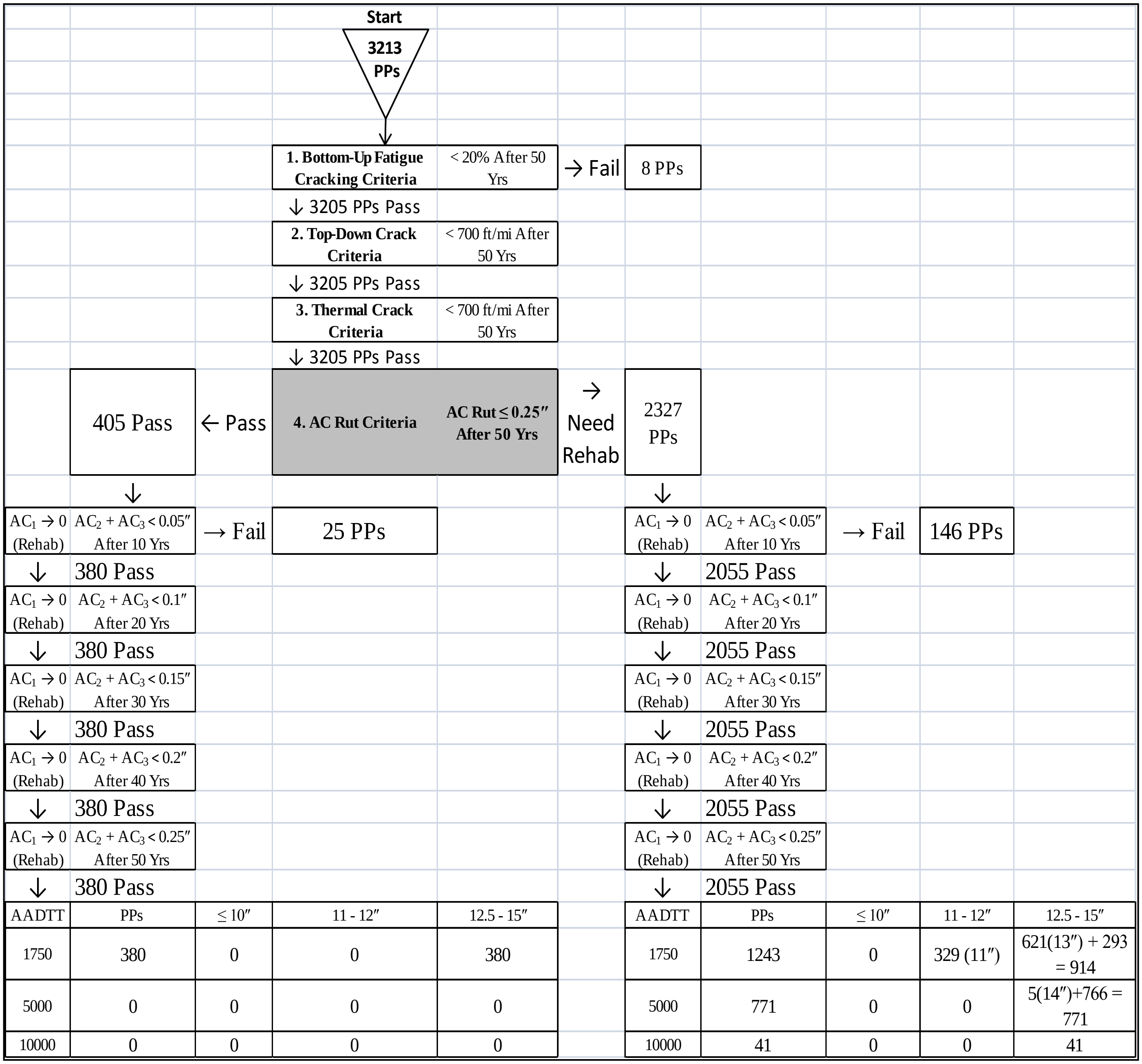
For AC rut ≤ 0.25″ and fatigue cracking ≤ 20%, perpetual pavements can be as follows;

- For 1750 AADTT – 329 pavements have 11″, 621 pavements have 13″, and 293 pavements have 15″ HMA thickness
- For 5000 AADTT – 5 pavements have 14″ and 766 have 15″ HMA thickness
- For 10000 AADTT – 41 pavements have 15″ HMA thickness

Pavements with 14″ HMA thickness and 5000 AADTT are the same pavements mentioned earlier. Figure 5.3(b) presents a flow chart of pavements that are checked against AC rut criterion of 0.5″ at the end of 50 years. Rehabilitation is also considered here if pavements show ≥ 0.5″ AC rutting after 50 years. Pavements failing this performance criterion are subjected to resurfacing (AC_1 rut → 0) and combined rutting in the intermediate and base layers are monitored (AC_2 + AC_3 rut ≤ 0.1). For AC rut ≤ 0.5″ and fatigue cracking ≤ 20%, perpetual pavements can be found as follows;

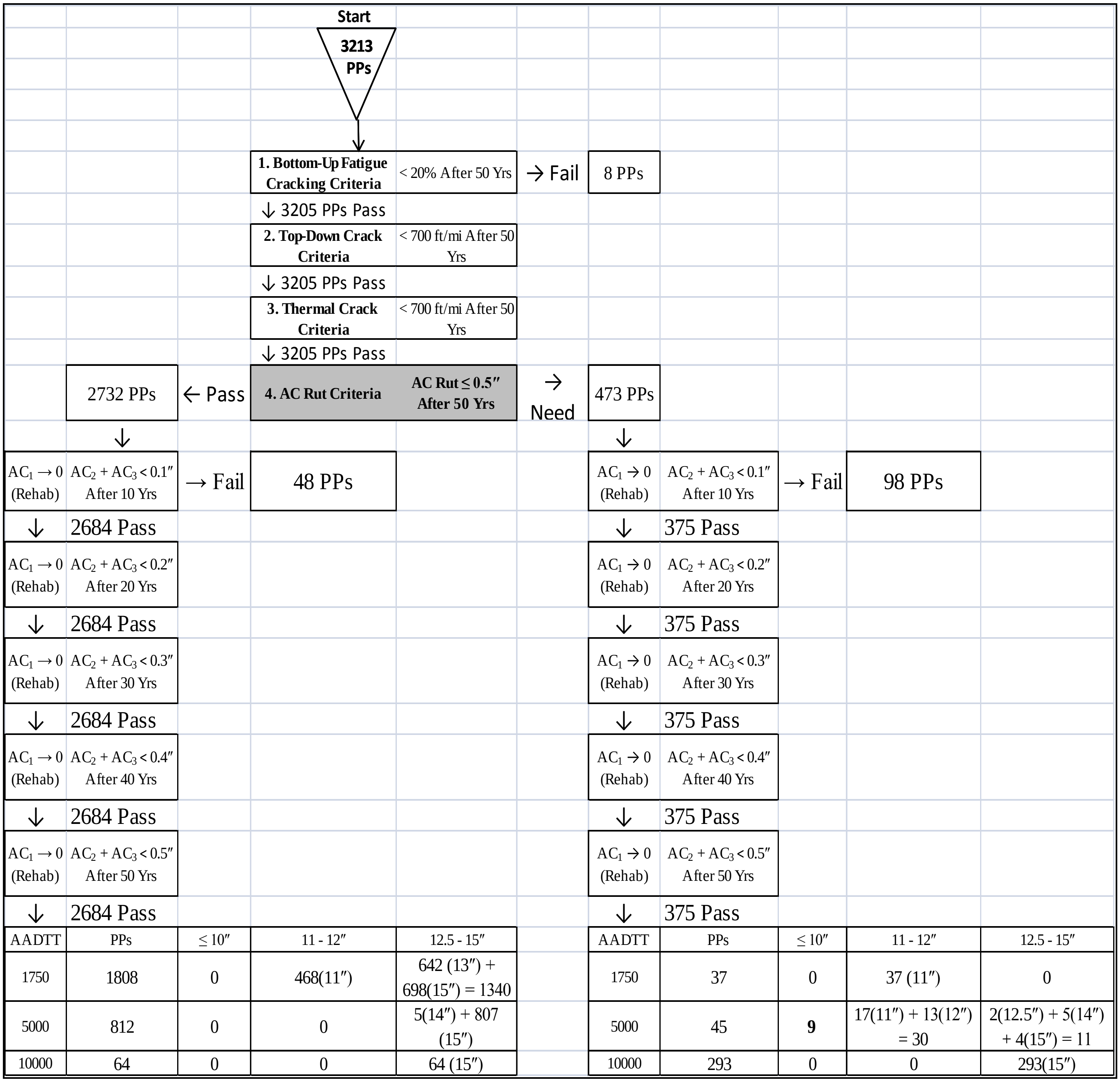
- For 1750 AADTT – 37 pavements have 11″ HMA thickness
- For 5000 AADTT – 9 pavements have 10″, 17 pavements have 11″, 13 pavements have 12″, and 4 pavements have 15″ HMA thickness
- For 10000 AADTT – 357 pavements have 15″ HMA thickness

It can be seen that 9 perpetual pavements have 10″ HMA thickness and 17 perpetual pavements have 11″ HMA thickness, each carrying 5000 AADTT, that pass the performance criteria for fatigue cracking and AC rutting. These pavements are described in the next section.



(a) AC Rut ≤ 0.25″ after 50 Years (Rehab = AC_1 rut → 0, AC_1 + AC_1 rut ≤ 0.05)

Figure 5.3 – Pavement Performance Flow Charts Based on AC Rut Criteria (Rehab Included)



(b) AC Rut ≤0.5″ after 50 Years (Rehab = AC_1 rut → 0, AC_1 + AC_1 rut ≤ 0.1)

Figure 5.3 – Pavement Performance Flow Charts Based on AC Rut Criteria (Rehab Included)

Pavements Carrying 5000 AADTT

Table 5.4(a) presents perpetual pavements with 10″ AC thickness and 5000 AADTT

- Run 2739 is not feasible due to lift thickness of the surface layer and the nominal size aggregate in this mix (SP-III).
- The remaining eight pavements have 3″ surface layer and use fine mixes (SP-III, SP-IV) and PG 76-22 in this layer.
- Six out the eight pavements have a 4″ intermediate layer.
- Two out of the eight pavements do not have a RBL, but they have thicker intermediate layers (7″).
- All of these pavements have 8000 psi resilient modulus (M_R) in their treated subgrade.
- None of the pavements shown in Figure 5.3(b) pass the total rut criterion of 0.75″. Rehabilitation must be considered as total rutting will not be as high if the pavement is rehabilitated every 10 years

Table 5.4(a) – Design Criteria of 10″ Perpetual Pavements Carrying 5000 AADTT

		Layer Thickness (in)					Mix Design			PG Binder			50 Yr. MEPDG Predicted Distress			10 Yr. Predicted Distress		
Run	Traffic (AADTT)	Surf	Itmd.	RBL	Total	GB	Surf. Mix	Itmd. Mix	RBL Mix	Surf. PG	Itmd. PG	Treated SG M_R (psi)	Top-Down Cracking (ft/mi)	Bottom-Up Cracking (%)	Thermal Cracking (ft/mi)	IRI (in/mi)	AC_1 Rut (in)	AC_2 + AC_3 Rut (in)
2739	5000	2	8	-	10	6	SP-III	SP-III	-	76-22	76-22	8000	0.25	8.62	0	107	0.09	0.10
2712	5000	3	4	3	10	6	SP-III	SP-IV	SP-IV	76-22	70-22	8000	0.48	1.8	0	107.6	0.14	0.05
2711	5000	3	4	3	10	6	SP-III	SP-IV	SP-III	76-22	70-22	8000	0.41	2.1	0	107.5	0.14	0.05
2710	5000	3	4	3	10	6	SP-III	SP-IV	SP-II	76-22	70-22	8000	0.31	2.8	0	107.3	0.14	0.05
2744	5000	3	7	-	10	6	SP-III	SP-IV	-	76-22	70-22	8000	0.45	8.1	0	107.7	0.14	0.05
2726	5000	3	4	3	10	6	SP-IV	SP-III	SP-IV	76-22	76-22	8000	0.42	1.7	0	107.7	0.16	0.04
2725	5000	3	4	3	10	6	SP-IV	SP-II	SP-IV	76-22	76-22	8000	0.42	1.6	0	107.4	0.16	0.04
2724	5000	3	4	3	10	6	SP-IV	SP-II	SP-II	76-22	76-22	8000	0.27	2.5	0	107.2	0.16	0.04
2751	5000	3	7	-	10	6	SP-IV	SP-II	-	76-22	76-22	8000	0.14	11.8	0	107.1	0.16	0.03

It can be noted that 10″ pavements carrying 5000 AADTT show very high rutting in the subgrade. Subgrade rutting is a major contributor to the failure of all pavements to pass the total rut criterion of 0.75″. However, by improving the material stiffness (M_R) of the subgrade from 5000 psi to 15,500 psi (Default for A-5 material in MEPDG), subsequent rutting in this layer reduces, as Figure 5.4 illustrates. Figure 5.4 also shows the effect of this change to total rutting. Points to note from this analysis: Subgrade rutting reduces by 60% and total rutting reduces by 30% due to improved MR in the subgrade. However, AC rutting slightly increases in all of the pavements shown in Figure 5.4.

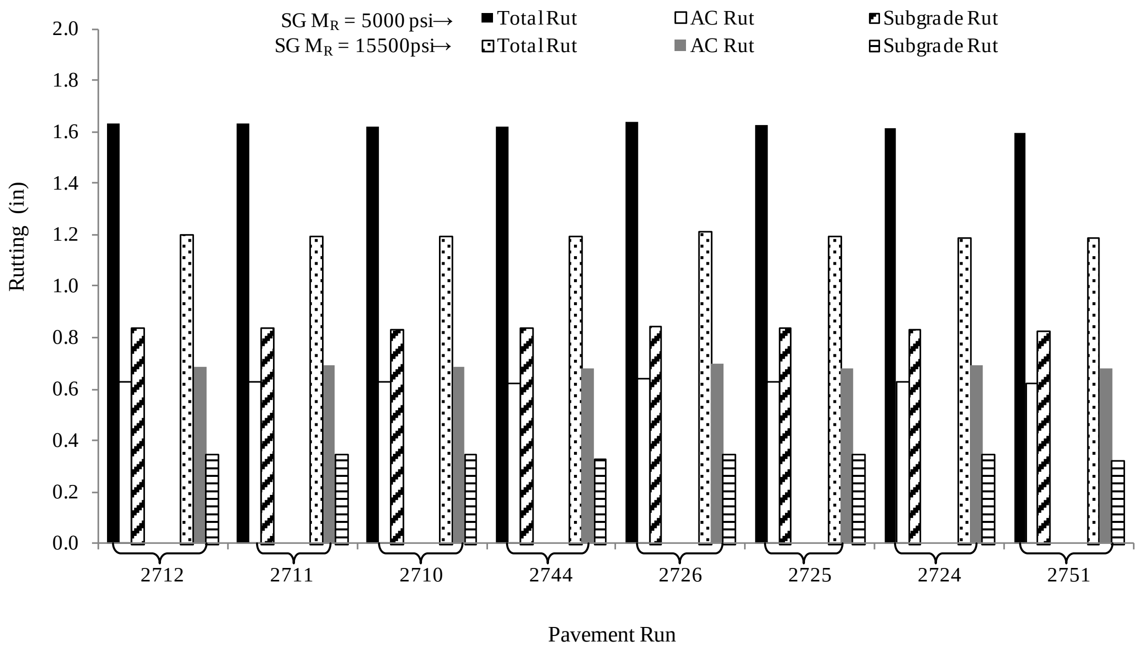


Figure 5.4 – Effects of Improving SG M_R on Total, AC, and Subgrade Rutting for 10″ Perpetual Pavements

Table 5.4(b) ranks the perpetual pavements having 11″ HMA thickness (5000 AADTT) based on AC rut criterion (AC_2+AC_3 ≤ 0.1″ after 10 years.

- Some of these pavements have AC_1 = 0.09″ as illustrated in Table 5.4(a). However, these pavements are not feasible due to lift thickness of the surface layer and the nominal size aggregate in this mix (SP-III). SP-III mix requires a minimum lift thickness of 2.5″.
- All of these pavements contain a RBL.
- All of these pavements have 8000 psi resilient modulus (M_R) in their treated subgrade.
- Top-down cracking increases by 100% due to increased stiffness in the subgrade. However, all of the pavements pass the top-down performance criterion (≤700ft/mi).
- Rutting in the combined AC_2 and AC_3 layers does not change due to increased stiffness in the subgrade.

Table 5.4(b) – Ranking of 11″ Perpetual Pavements Based on Rutting in AC_2 + AC_3

			Layer Thickness (in)					Mix Design			PG Binder			50 Yr. MEPDG Predicted Distress			10 Yr. Predicted Distress		
Rank	Run	Traffic (AADTT)	Surf	Itmd.	RBL	Total	GB	Surf. Mix	Itmd. Mix	RBL Mix	Surf. PG	Itmd. PG	Treated SG M_R (psi)	Top-Down Cracking (ft/mi)	Bottom-Up Cracking (%)	Thermal Cracking (ft/mi)	IRI (in/mi)	AC_1 Rut (in)	AC_2 + AC_3 Rut (in)
1	2686	5000	2	6	3	11	6	SP-III	SP-II	SP-II	70-22	76-22	8000	0.03	1.42	0	104.8	0.11	0.07
2	2688	5000	2	6	3	11	6	SP-III	SP-II	SP-III	70-22	76-22	8000	0.04	1.06	0	104.9	0.11	0.07
3	2690	5000	2	6	3	11	6	SP-III	SP-II	SP-IV	70-22	76-22	8000	0.05	0.90	0	105.0	0.11	0.07
4	2695	5000	2	4	5	11	6	SP-III	SP-II	SP-II	70-22	76-22	8000	0.03	1.50	0	104.9	0.11	0.08
5	2697	5000	2	4	5	11	6	SP-III	SP-II	SP-III	70-22	76-22	8000	0.04	1.16	0	105.1	0.11	0.08
6	2699	5000	2	4	5	11	6	SP-III	SP-II	SP-IV	70-22	76-22	8000	0.05	1.01	0	105.2	0.11	0.08
7	2692	5000	2	6	3	11	6	SP-III	SP-III	SP-II	76-22	76-22	8000	0.02	1.51	0	104.5	0.09	0.09
8	2693	5000	2	6	3	11	6	SP-III	SP-III	SP-III	76-22	76-22	8000	0.03	1.13	0	104.6	0.09	0.09
9	2694	5000	2	6	3	11	6	SP-III	SP-III	SP-IV	76-22	76-22	8000	0.04	0.96	0	104.7	0.09	0.09
10	2687	5000	2	6	3	11	6	SP-III	SP-II	SP-II	76-22	70-22	8000	0.03	1.50	0	104.6	0.09	0.09
11	2701	5000	2	4	5	11	6	SP-III	SP-III	SP-II	76-22	76-22	8000	0.02	1.52	0	104.6	0.09	0.09
12	2689	5000	2	6	3	11	6	SP-III	SP-II	SP-III	76-22	70-22	8000	0.03	1.12	0	104.8	0.09	0.09
13	2691	5000	2	6	3	11	6	SP-III	SP-II	SP-IV	76-22	70-22	8000	0.04	0.95	0	104.9	0.09	0.09
14	2702	5000	2	4	5	11	6	SP-III	SP-III	SP-III	76-22	76-22	8000	0.03	1.18	0	104.8	0.09	0.09
15	2696	5000	2	4	5	11	6	SP-III	SP-II	SP-II	76-22	70-22	8000	0.02	1.52	0	104.7	0.09	0.09
16	2698	5000	2	4	5	11	6	SP-III	SP-II	SP-III	76-22	70-22	8000	0.03	1.18	0	104.9	0.09	0.09
17	2700	5000	2	4	5	11	6	SP-III	SP-II	SP-IV	76-22	70-22	8000	0.04	1.02	0	105.1	0.09	0.10

Pavements Carrying 1750 AADTT

Eight pavements carrying 1750 AADTT have 10″ HMA thickness. These pavements are listed in Table 5.5(a). In total, 546 perpetual pavements are found carrying 1750 AADTT and that have 11″ thickness. Table 5.5(b) presents the design information of 24 of the 546 perpetual pavements.

- All of the pavements contain a RBL and have 10″ granular base thickness.
- All of the pavements use SP-IV mix in the surface layer.
- 20 out of 24 pavements use SP-II in the intermediate layer.
- All of these pavements have 8000 psi resilient modulus (M_R) in their treated subgrade.

Table 5.5(a) – Design Criteria of 10″ Perpetual Pavements Carrying 1750 AADTT

		Layer Thickness (in)					Mix Design			PG Binder			50 Yr. MEPDG Predicted Distress			10 Yr. Predicted Distress		
Run	Traffic (AADTT)	Surf	Itmd.	RBL	Total	GB	Surf. Mix	Itmd. Mix	RBL Mix	Surf. PG	Itmd. PG	Treated SG M_R (psi)	Top-Down Cracking (ft/mi)	Bottom-Up Cracking (%)	Thermal Cracking (ft/mi)	IRI (in/mi)	AC_1 Rut (in)	AC_2 + AC_3 Rut (in)
3214	1750	3	4	3	10	6	SP-III	SP-IV	SP-IV	76-22	70-22	8000	0.06	0.9	0	100.6	0.09	0.03
3215	1750	3	4	3	10	6	SP-III	SP-IV	SP-III	76-22	70-22	8000	0.06	0.9	0	100.6	0.09	0.03
3216	1750	3	4	3	10	6	SP-III	SP-IV	SP-II	76-22	70-22	8000	0.06	0.9	0	100.6	0.09	0.03
3217	1750	3	7	-	10	6	SP-III	SP-IV	-	76-22	70-22	8000	0.06	0.8	0	100.5	0.10	0.02
3218	1750	3	4	3	10	6	SP-IV	SP-III	SP-IV	76-22	76-22	8000	0.06	0.8	0	100.5	0.10	0.02
3219	1750	3	4	3	10	6	SP-IV	SP-II	SP-IV	76-22	76-22	8000	0.06	0.8	0	100.5	0.10	0.02
3220	1750	3	4	3	10	6	SP-IV	SP-II	SP-II	76-22	76-22	8000	0.1	2.8	0	100.9	0.09	0.03
3221	1750	3	7	-	10	6	SP-IV	SP-II	-	76-22	76-22	8000	0.03	4.2	0	100.3	0.10	0.02

Table 5.5(b) – 11″ Perpetual Pavements (1750 AADTT)

		Layer Thickness (in)					Mix Design			PG Binder		
Run	Traffic (AADTT)	Surf	Itmd.	RBL	Total	GB	Surf. Mix	Itmd. Mix	RBL Mix	Surf. PG	Itmd. PG	Treated SG M_R (psi)
1621	1750	2	4	5	11	10	SP-IV	SP-II	SP-II	70-22	70-22	8000
1622	1750	2	4	5	11	10	SP-IV	SP-II	SP-II	70-22	76-22	8000
1623	1750	2	4	5	11	10	SP-IV	SP-II	SP-II	76-22	70-22	8000
1624	1750	2	4	5	11	10	SP-IV	SP-II	SP-II	76-22	76-22	8000
1654	1750	2	4	5	11	10	SP-IV	SP-IV	SP-IV	70-22	76-22	8000
1656	1750	2	4	5	11	10	SP-IV	SP-IV	SP-IV	76-22	76-22	8000
1693	1750	2	2	7	11	10	SP-IV	SP-II	SP-II	70-22	70-22	8000
1694	1750	2	2	7	11	10	SP-IV	SP-II	SP-II	70-22	76-22	8000
1695	1750	2	2	7	11	10	SP-IV	SP-II	SP-II	76-22	70-22	8000
1696	1750	2	2	7	11	10	SP-IV	SP-II	SP-II	76-22	76-22	8000
1697	1750	2	2	7	11	10	SP-IV	SP-II	SP-III	70-22	70-22	8000
1698	1750	2	2	7	11	10	SP-IV	SP-II	SP-III	70-22	76-22	8000
1699	1750	2	2	7	11	10	SP-IV	SP-II	SP-III	76-22	70-22	8000
1700	1750	2	2	7	11	10	SP-IV	SP-II	SP-III	76-22	76-22	8000
1701	1750	2	2	7	11	10	SP-IV	SP-II	SP-IV	70-22	70-22	8000
1702	1750	2	2	7	11	10	SP-IV	SP-II	SP-IV	70-22	76-22	8000
1703	1750	2	2	7	11	10	SP-IV	SP-II	SP-IV	76-22	70-22	8000
1704	1750	2	2	7	11	10	SP-IV	SP-II	SP-IV	76-22	76-22	8000
1783	1750	3	5	3	11	10	SP-IV	SP-III	SP-III	76-22	70-22	8000
1784	1750	3	5	3	11	10	SP-IV	SP-III	SP-III	76-22	76-22	8000
1841	1750	3	3	5	11	10	SP-IV	SP-II	SP-III	70-22	70-22	8000
1842	1750	3	3	5	11	10	SP-IV	SP-II	SP-III	70-22	76-22	8000
1843	1750	3	3	5	11	10	SP-IV	SP-II	SP-III	76-22	70-22	8000
1844	1750	3	3	5	11	10	SP-IV	SP-II	SP-III	76-22	76-22	8000

5.2 Task 3B. Determine Sensitivity of HMA Field-Mix Variations

The volumetric properties of the field-mixture are arguably the most critical factor influencing performance of perpetual pavements. The mix volumetric properties can directly used as MEPDG inputs. As MEPDG can account for slight changes in design mixes used in a perpetual pavement project, a more accurate prediction in terms of design life can be produced. Pavement performance can be exemplified by changing the mix properties. In this task, selected cases of perpetual pavements which pass the design life criteria of 50 years or more are examined.

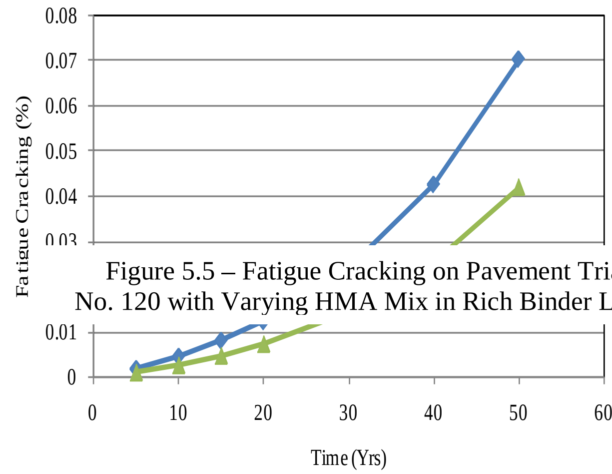


Figure 5.5 – Fatigue Cracking on Pavement Trial No. 120 with Varying HMA Mix in Rich Binder Layer

5.2.1 HMA Type in Rich Binder Layer

Additional simulations were run to determine the effect of HMA mixes in the RBL. The fine mix in the RBL (SP-IV) is replaced with the coarse mix (SP-II). The results presented in Figure 5.5 shows fatigue cracking of Pavement trial No. 120 using two different HMA mixes in the RBL. As expected, the fine mix performs better than the coarse mix in terms of fatigue cracking due to higher percentage binder content which provides added flexibility under bending stresses/strains.

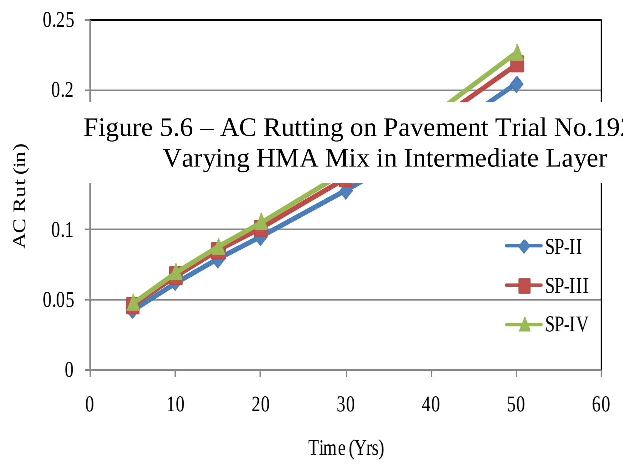


Figure 5.6 – AC Rutting on Pavement Trial No.192 with Varying HMA Mix in Intermediate Layer

5.2.2 HMA Type in Intermediate Layer

Once again, additional simulations are performed to determine the effect of the HMA mixes in the intermediate layer. The coarse HMA mix in the intermediate layer, SP-II, is replaced by a fine HMA mix, SP-III. Figure 11 shows AC rutting of Pavement trial No. 192 with varying HMA types in the intermediate layer. The results presented show that the pavement has slightly increased AC rutting. None of the pavements failed for AC rutting.

5.3 Task 3C. Perform Traffic Modeling Appropriate for Perpetual Design

Traffic modeling is done based on the literature study conducted in Task 1. Based on the functional classification of traffic, appropriate traffic class and growth factors are selected for designing perpetual pavements to last 50 years or more for major highways in New Mexico. Determining future traffic is one of the major challenges of pavement design. The basic required MEPDG input data is Annual Average Daily Traffic (AADT), percentage of trucks in the design direction and on the design lane, operational speed, and traffic growth rate. The traffic growth in MEPDG is compound. For this study, all other required traffic inputs, such as monthly and hourly truck distribution, truck class distribution, axle load distributions, and some other general traffic inputs, are derived from the design guide level 3 or default values. Table 5.6 presents the traffic volumes used in this study thus far. Table 5.6 also shows the equivalent number of ESALs to enhance the understanding of design traffic for 50+year perpetual pavements.

Table 5.6 – Conversion of AADTT to Cumulative ESALs

AADTT	20 Yr. Cum. ESALs	50 Yr. Cum. ESALs
1750	11,348,873	57,829,092
2700	15,329,341	78,111,924
5000	32,425,338	165,226,044
10000	64,850,667	330,451,948

5.4 Task 3D. Evaluate Design Alternatives Based on Life Cycle Cost Analysis

In this task, a Life Cycle Cost Analyses (LCCA) is performed to assess the most economic design among the 8 pavement alternatives sown in Table 5.4(a). FHWAs' computerized LCCA program, "LCCA 2002", is used along with a model developed by the National Lime Association to determine the most economic design. Both models are used to compare total cost of competing design alternatives, each of which passes the 50-year design life criteria of a perpetual pavement.

5.4.1 Using National Lime Association LCCA Model

The LCCA (life cycle cost analysis) software package is a Windows-based application that was developed to perform economic analyses of two pavement alternatives that are subject to future maintenance and rehabilitation (M&R) activities (Walls and Smith, 1998). The software allows analyses for both new construction and rehabilitation projects and allows inclusion of lane rental fees as a surrogate for user delay costs. Either a deterministic approach or a probabilistic approach can be utilized in a given analysis. The deterministic approach utilizes mean (average) values of the input variables. In the probabilistic approach, real world variability of certain input variables is mimicked through a Monte Carlo simulation process. This approach utilizes the means and standard deviations of the input variables that are varied in the Monte Carlo simulation process. The graphical user interface (GUI) interacts with a 32-bit Microsoft Access 97 database. Table 5.7 presents two perpetual pavements; (*i*) one without a rich binder layer (RBL), and (*ii*) the other with a RBL.

Analysis

Input data of these two pavements are identical except the data of a RBL. This LCCA model only allows the use of one HMA layer. It does not accommodate the use of separate HMA layers in one pavement. In order to account for separate layers in one pavement, they must be combined to form one layer. The selected perpetual pavements have 2 – 3 HMA layers. Hence, the analysis is done for one HMA layer that combines all of the individual HMA layers. Details of the LCCA analysis are provided in the next section:

<u>Pavement Alternative 1 = Perpetual Pavement (No RBL)</u>

3″ Surface HMA = $100/cy
7″ Intermediate HMA = $100/cy
6″ Granular Base = $42/cy
12″ Treated Subgrade = $36/cy
Perpetual Pavement costs about $278/cy

<u>Pavement Alternative 2 = Perpetual Pavement (RBL Incl.)</u>

3″ Surface HMA = $100/cy
4″ Intermediate HMA = $100/cy
3″ Rich Binder HMA = $120/cy
6″ Granular Base = $42/cy
12″ Treated Subgrade = $36/cy
Perpetual Pavement costs about $398/cy

No maintenance (OG layer, crack seal etc.) is required. Surface rehabilitation is done every 10 years if needed. Rehabilitation involves 2″ mill and fill and 1.5″ overlay. User cost (lane rental fee) for a high-volume facility (ADT > 15000) is $10,000 lane-mi/day. Probabilistic analysis is performed over 50 years with a discount rate of 4%.

<u>Results</u>

Mean Life Cycle Cost ($): Perpetual Pavement (No RBL) = $9.3 million
Perpetual Pavement (RBL Incl.) = $13.3 million

More detailed results are shown in Appendix B.

Table 5.7 – Input Values for LCCA of Two Perpetual Pavement Types

Alternative 1	Perpetual Pavement (Run 2744) - No RBL			*Alternative 2*	Perpetual Pavement (Run 2710) - RBL		
Initial Pavement Design	Initial Construction			**Initial Pavement Design**	Initial Construction		
	Mean	Std. Dev	COV		Mean	Std. Dev	COV
AADTT = Average Daily Traffic	5000	100	10%	AADTT = Average Daily Traffic	5000	100	10%
% T = Percent Trucks	25%	0.025	10%	% T = Percent Trucks	25%	0.025	10%
TF = Truck Factor	0.38	0.038	10%	TF = Truck Factor	0.38	0.038	10%
G = Growth Rate	4	0.004	10%	G = Growth Rate	4	0.004	10%
N = Analysis Period	50			N = Analysis Period	50		
PSIi = Initial PSI	4.2	0.2814	6.7	PSIi = Initial PSI	4.2	0.2814	6.7
PSIt = Terminal PSI	2			PSIt = Terminal PSI	2		
Mr = Effective Mr	5000	500	10%	Mr = Effective Mr	5000	500	10%
a1 = Surf Layer Coeff.	0.44	0.044	10%	a1 = Surf Layer Coeff.	0.44	0.044	10%
a2 = Base Layer Coeff.	0.14	0.014	10%	a2 = Base Layer Coeff.	0.14	0.014	10%
m2 = Base Drainage Coeff.	1	0.1	10%	m2 = Base Drainage Coeff.	1	0.1	10%
a3 = Subbase Layer Coeff.	0.11	0.011	10%	a3 = Subbase Layer Coeff.	0.11	0.011	10%
m3 = Subbase Drainage Coeff.	1	0.1	10%	m3 = Subbase Drainage Coeff.	1	0.1	10%
Surface Rehabilitation				**Surface Rehabilitation**			
Mill and Fill (in)	3			Mill and Fill (in)	3		
Route Classification				**Route Classification**			
1=US/State, 2=County	1			1=US/State, 2=County	1		
Discount Rate				**Discount Rate**			
Discount Rate, %	4			Discount Rate, %	4		
Unit Costs				**Unit Costs**			
Surf HMA ($/cy)	100	10	10%	Surf HMA ($/cy)	100	10	10%
Intermediate HMA ($/cy)	110	11	10%	Intermediate HMA ($/cy)	110	11	10%
Granular Base ($/cy)	42	4.2	10%	Rich Binder HMA ($/cy)	120	12	10%
Treated Subgrade ($/cy)	36	3.6	10%	Granular Base ($/cy)	42	4.2	10%
Surface Treatment ($/lane-mi)	10000	1000	10%	Treated Subgrade ($/cy)	36	3.6	10%
				Surface Treatment ($/lane-mi)	10000	1000	10%

5.4.2 Federal Highway Administration LCCA Model

FHWA's LCCA is an engineering economic analysis tool useful for comparing the relative economic merits of competing construction or rehabilitation design alternatives for a single project. By considering all of the relevant costs—agency and user—incurred during the projected service life of an asset, this analytical process helps transportation officials to identify the lowest cost option. Additionally, LCCA introduces a structured methodology that quantifies the effects of agency activities on transportation users and provides a means to balance those effects with the construction, rehabilitation, and preservation needs of the system itself. Once again, two types of perpetual pavements are considered in this analysis: (*i*) a perpetual pavement without a rich binder layer (RBL), and (*ii*) a perpetual pavement with a RBL.

Inputs for both pavements are identical except for initial costs of construction. Initial construction of the perpetual pavement with the rich binder layer is higher due to presence of three HMA different layers that require different construction practices (lift compaction, density, etc.).

- Perpetual Pavement (No RBL) – $1,835,000 (NMDOT Treatment Costs)
- Perpetual Pavement (RBL) – $2,100,000 (Assumed)

Traffic data is 30,000 AADT (total for both directions) with 35% heavy trucks which is equivalent to about 5000 AADTT in one direction. All other traffic inputs are taken from MEPDG input data. Traffic data that is not available (free flow capacity, queue dissipation capacity, etc) is calculated using the software default values.

Analysis Results

$$3'' \text{ HMA Surface Rehab} = \$174{,}000 \begin{cases} \text{NMDOT HMA Mill} = \$8000/\text{in} \\ \text{Thin Hot Mix Overlay} = \$50{,}000/\text{in} \end{cases}$$

Total Cost ($): Perpetual Pavement (No RBL) = $2.5 million
Perpetual Pavement (RBL Incl.) = $2.8 million

More detailed results are shown in Appendix B.

Chapter 6

EFFECTS OF LAYERS AND DE-BONDING

6.0 Introduction

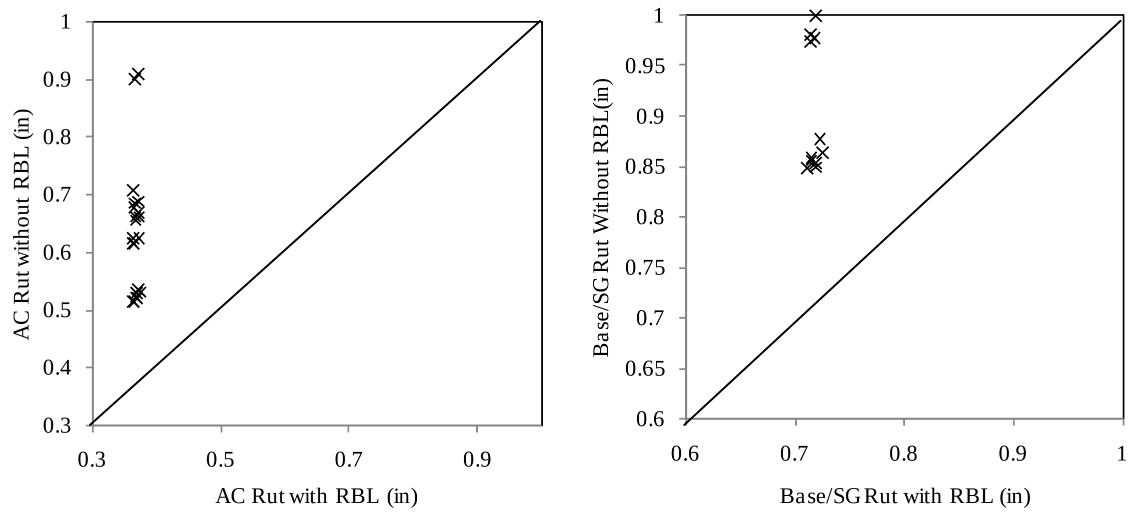
In this chapter, the research team investigates the use of rich binder layers in perpetual pavements. It is also debated whether or not this particular HMA layer are applicable for perpetual pavements discovered from this study. In addition, the potential of HMA layer de-bonding in selected perpetual pavements is addressed, and the subsequent performance of the pavements is determined if such circumstances arise. Analysis of HMA layer de-bonding is done through MEPDG and KENLAYER, both of which are multi-layered elastic programs.

6.1 Task 4A. Evaluate Impacts of Removing a Layer of a Perpetual Pavement

In this task, selected perpetual pavement structures are made less complex by removing the rich binder layers. The research team investigates whether the need for a rich binder mix (RBM) layer to minimize fatigue damage is justified.

Removing Rich Binder Layer

50 perpetual pavements were selected from Chapter 5 based on low AC rutting using 5000 AADTT. Pavements analyzed in this section are further reduced from 50 to 21 due to removal of the RBL. By removing this layer, many of the 50 reference pavements become identical in design and material properties. Figures 6.1, 6.2, and 6.3 highlight the effects of removing the rich binder layer from the pavement structure. The X and Y axes on Figures 6.1(a) – (b) have the same scale and the 45° line indicates where rutting values for both cases (with and without a RBL) are the same. The information presented in Figure 6.1(a) and (b) clearly shows an increase in rutting in both the AC and subgrade layers due to removal of the RBL. RBLs are generally used to minimize bottom-up fatigue cracking. An increase in rutting might be due to the reduced thickness of the pavement where stress intensity might have increased. As expected,



(a) AC-Rutting without Rich Binder Layer

(b) Base/Subgrade Rut without Rich Binder Layer

Figure 6.1 – MEPDG Predicted Rutting Without Using a Rich Binder Layer

Figure 6.2 shows a significant increase in fatigue cracking even though none of the pavements failed. Pavements that include RBLs show fatigue cracking ranging from 0 – 2%, but those without RBLs show fatigue cracking ranging from 3 – 35%. However minimizing fatigue cracking is important and the presence of a RBL ensures that the pavements shown in this study do not fail by fatigue cracking for 50 years or more. Figure 6.3 presents MEPDG predicted surface-down cracking for pavements without a RBL where a significant increase in cracking is observed. However, none of these pavements fail by surface-down cracking.

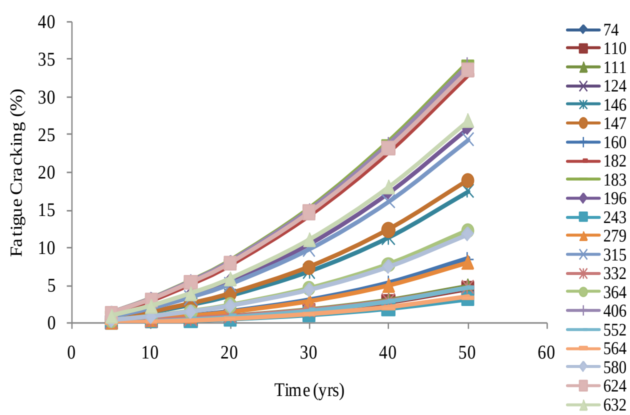


Figure 6.2 – Fatigue Cracking for Pavements without Rich Binder Layer

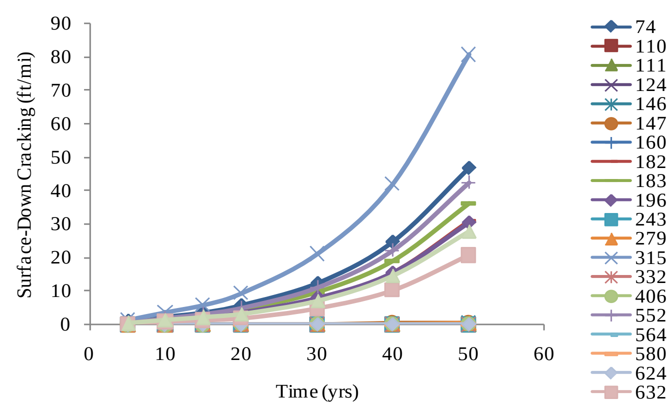


Figure 6.3 – Surface-Down Cracking for Pavements without Rich Binder Layer

6.2 Task 4B. Determine Effects of Bonding and De-bonding

The MEPDG program for flexible pavements accounts for bonding but only in terms of complete bonding or no bonding. In this task, complete and non-bonding environments are produced in another pavement analysis computer program, KENAYER. The results are analyzed to examine the performance of the perpetual pavements under non-bonding and bonding environments. Eight perpetual pavements passing the design life criteria of 50 years highlighted in Table 5.4(a) were analyzed for bonding and de-bonding.

6.2.1 De-Bonding Study Using MEPDG

As stated previously, de-bonding of asphalt layers can be addressed by MEPDG software. However, only full bonding (MEPDG input value = 1) or complete de-bonding (MEPDG input value = 0) is permitted, with no option for partial de-bonding of HMA layers, which is much more likely to occur in HMA pavements. To analyze the effect of de-bonding in MEPDG, eight optimal perpetual pavements with 10" HMA thickness and 5000 AADTT have complete de-bonding between HMA layers and are analyzed for 50 years. The results are then compared to the same pavements with full bonding between HMA layers.

Figure 6.4 presents MEPDG predicted top-down cracking in eight HMA perpetual pavements carrying 5000 AADTT at the end of 50 years for bonded and de-bonded cases. It can be seen here that all but one of the pavements fail (> 700 ft/mi) when there is complete de-bonding between the HMA layers with predicted top-down cracking in excess of 10,000 ft/mi for six of the eight pavements. These six pavements are predicted to fail within 2 years. The pavement that did not fail (Run 2751) showed an increase from 0.2 to 75 ft/mi.

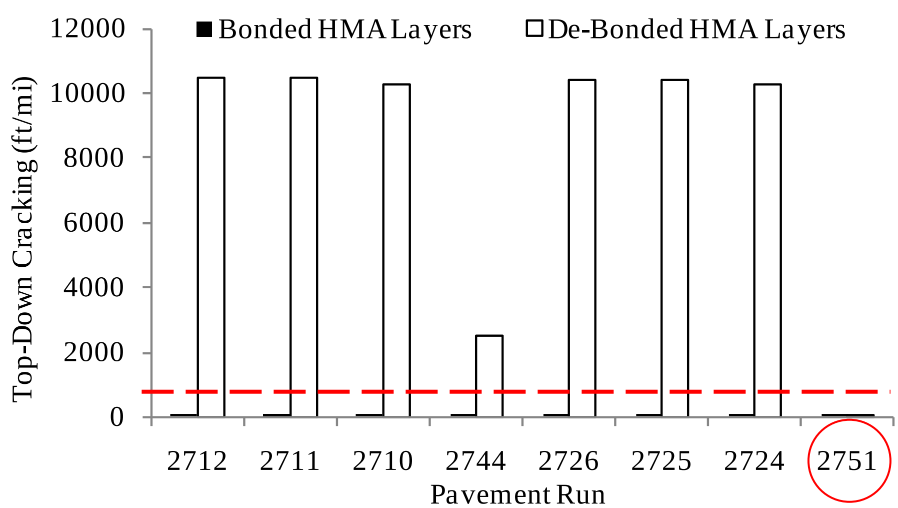


Figure 6.4 – MEPDG Predicted Top-Down Cracking in Bonded/De-Bonded HMA Pavements

Predicted bottom-up cracking is presented in Figure 6.5 for the same 8 perpetual pavements shown earlier. Bottom-up cracking has increased significantly (150 – 700%) in all eight pavements with two of them failing (> 20%). The failed pavements do not contain a rich binder layer and are predicted to fail at the end of 20 years.

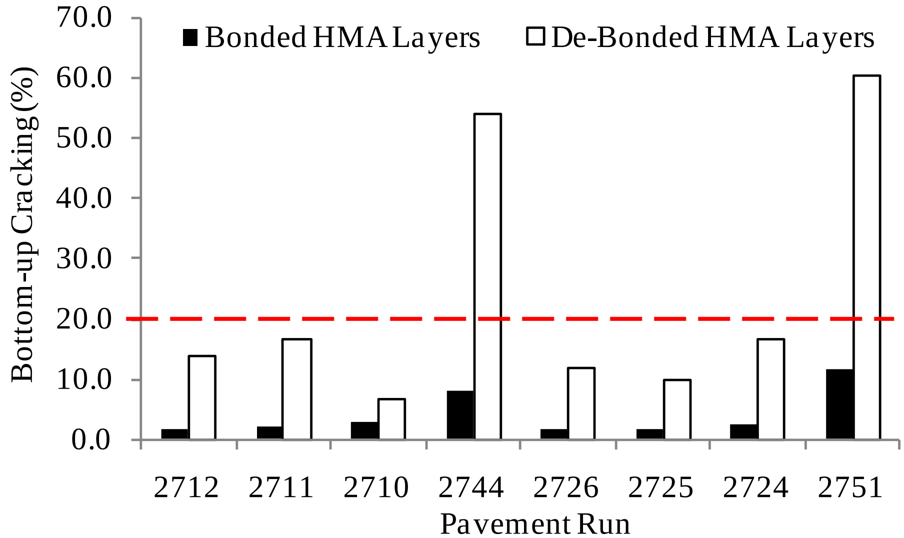


Figure 6.5 – MEPDG Predicted Bottom-Up Cracking in Bonded/De-Bonded HMA Pavements

In terms of AC rutting, all of the pavements are predicted to have more than twice the amount of AC rutting than those with bonded HMA layers. This is illustrated in Figure 6.6 where AC rutting increases from 0.2 to 0.5″ at the end of 50 years. However, all eight pavements still pass the AC rut criterion (≤ 0.5″ after 50 years) even with de-bonded HMA layers.

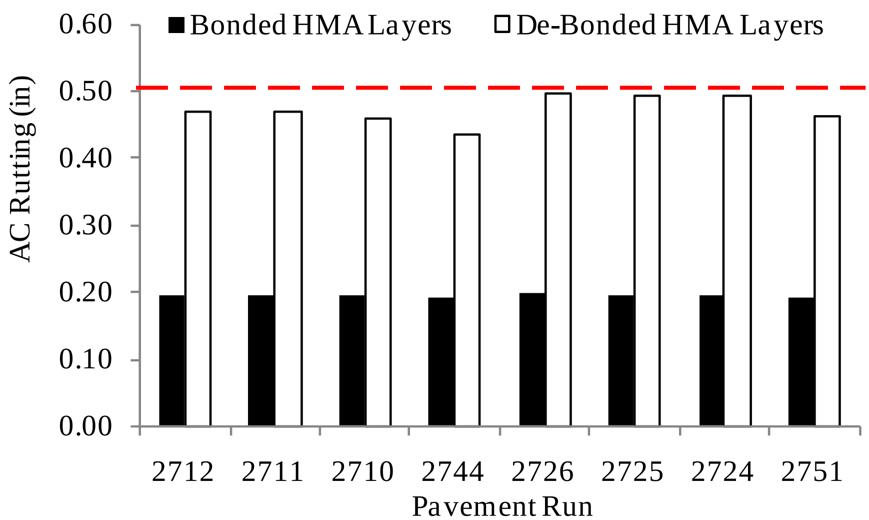


Figure 6.6 – MEPDG Predicted AC Rutting in Bonded/De-Bonded HMA Pavements

De-Bonding in Upper Asphalt Layers

The above analysis is based on de-bonding of all HMA layers in each perpetual pavement. However, not all layers are expected to experience de-bonding. Usually de-bonding occurs in the upper HMA layers due to the presence of moisture. Hence, analysis was done to determine the effect of de-bonding in upper HMA layers. Once again, the same eight pavements shown above are used in this analysis. Figure 6.7 presents predicted top-down cracking for these eight pavements with de-bonding occurring in the top two asphalt layers. None of the pavements fail but a significant increase in top-down cracking is predicted.

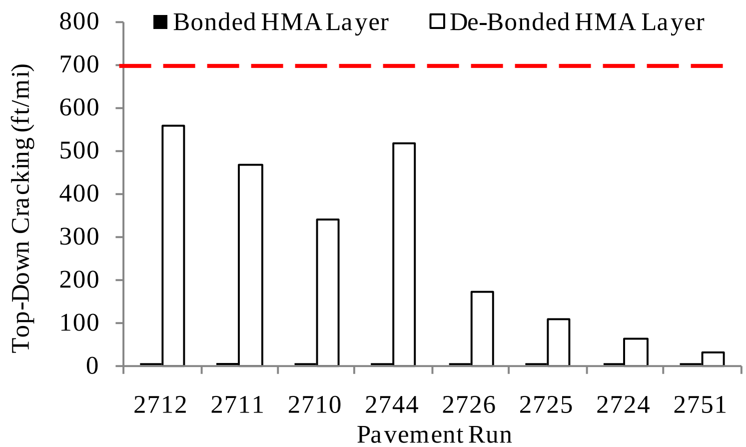


Figure 6.7 – MEPDG Predicted Top-Down Cracking in Bonded/De-Bonded HMA Pavements

Figure 6.8 presents predicted bottom-up cracking for eight perpetual pavements that have de-bonding in their upper asphalt layers. Bottom-up cracking increases in all of the pavements with two of them failing (> 20%). These two pavements do not contain RBLs and they are predicted to fail after 30 years.

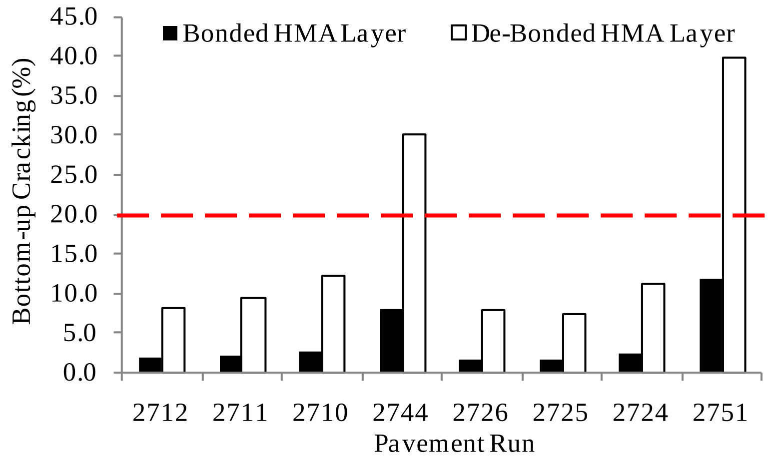


Figure 6.8 – MEPDG Predicted Bottom-Up Cracking in Bonded/De-Bonded HMA Pavements

AC rutting is once again shown to be very high when there is de-bonding in the upper asphalt layers. Figure 6.9 shows that AC rutting increases from 0.2 to 0.46″ which is almost the same as the predicted AC rutting with de-bonding in all HMA layers. Thus, from this analysis, we can say that preventing de-bonding of upper asphalt layers will reduce potential AC rutting.

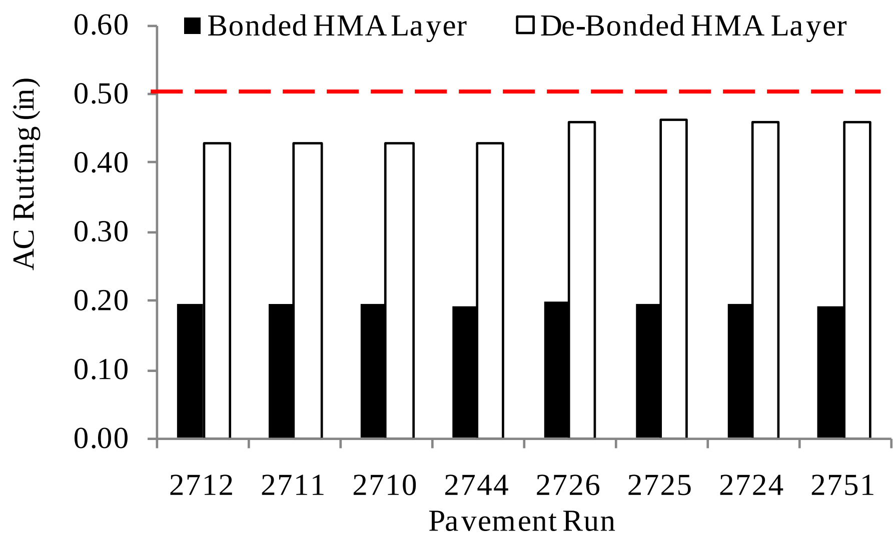


Figure 6.9 – MEPDG Predicted AC Rutting in Bonded/De-Bonded HMA Pavements

Summary

- A total of 7 out of 8 perpetual pavements with de-bonded HMA layers fail by top-down cracking (< 700 ft/mi) at the end of 50 years.

- For bottom-up cracking ≤ 20%, 2 of the 8 pavements failed due to de-bonding of all asphalt layers. Bottom-up cracking also significantly increased in the remaining 6 pavements (150 – 700%).
- AC Rutting more than doubled (0.2 to 0.5″) in all of the pavements due to the de-bonding of all HMA layers. However, none of the pavements failed by AC rutting.
- De-bonding of upper asphalt layers still causes a significant increase in top-down cracking. However none of the pavements fail. A significant increase in bottom-up cracking is also noted, as well as AC rutting. AC rutting values are similar to those values predicted when all HMA layers are de-bonded. Hence, preventing de-bonding of upper asphalt layers will significantly reduce potential AC rutting.

6.2.2 De-Bonding in KENLAYER

Complete and non-bonding environments ars produced in another pavement analysis computer programs called KENAYER. The results are analyzed to examine the behavior of the layers' interface due to non-bonding and bonding environments. KENLAYER is the solution for an elastic multilayer pavement system. KENLAYER can be applied to layered systems under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, linear elastic, nonlinear elastic or viscoelastic. KENLAYER, together with input program LAYERINP and graphic program LGRAPH, is part of a computer package called KENPAVE. Appendix C presents the results from this analysis.

Chapter 7

CONCLUSIONS

7.1 Conclusions

In this study, an optimal perpetual pavement is determined based on assumed material properties. This is done through a full literature review of current design, testing, and evaluation of perpetual pavements, evaluation of the effects of moisture infiltration on perpetual pavement performance, analysis of perpetual pavement alternatives based on layer stiffness and thickness, as well as quantification of the impact of removing layers and consideration of various degrees of de-bonding between layers of a perpetual pavement section. In the following paragraphs, the findings of this study are summarized:

- Eight of the fourteen U.S. perpetual pavements reviewed in this study use rich binder layers (RBLs). The use of RBL is the perpetual pavement structure can be seen in countries such as China, Australia, and Israel.

- Literature study revealed that polymer modified PG binders are more commonly used in surface and intermediate layers, in order to prevent rutting. The use of polymer-modified binders in base layers is rare. Based on the literature reviewed, only one state DOT has used a polymer-modified binder in its base layer.

- Perpetual pavements reviewed herein from New Mexico, Oregon, and Texas all suffer from high permeability and moisture damage. A number of perpetual pavements in Texas showed de-bonding between HMA layers.

- Preliminary analysis of the data gathered from literature and survey shows that about 75% of the perpetual pavements perform as expected. Of the pavement sections analyzed using the MEPDG, more than 50% of them should perform to their expected design life.

- MEPDG analysis of US-70 Hondo Valley pavement shows that it has high rutting and IRI but may not fail due to bottom-up and top-down cracking. However, if analysis considers de-bonding between HMA layers, US-70 shows very high top-down cracking and rutting.

- Moisture retention testing of some typical HMA mixes used in New Mexico show porosities varying from 7 to 18.5% and the saturated hydraulic conductivity varied from 4.74 x 10^{-06} to 3.28 x 10^{-04} cm/s. There is no previous moisture retention test data for these HMA mixes so results cannot be compared although the saturated hydraulic conductivity values look very high.

- Moisture characteristic curves produced from laboratory column testing suggest that asphalt has a hydrophobic nature and may resist moisture infiltration unless positive pressure (head) is developed.

- In the MEPDG, moisture damage can be addressed through using reduced dynamic modulus due to moisture infiltration. Dynamic modulus values of wet and dry sample (E^*_{wet}/E^*_{dry}) ratio is known to be 0.80 – 0.90 for other state DOT mixes. However, this ratio for New Mexico is not known. So, MEPDG analyses using reduced E* of HMA mixes were not pursued in our study.

- From 3213 MEPDG simulations, it is shown that none of the pavements experienced any thermal cracking. The use of modified binders and the location of the climate data (Albuquerque) might have reduced the impact of thermal cracking on the pavements analyzed. Pavements in the northern New Mexico may show thermal cracking. It will be interesting to examine whether NMDOT pavements show low temperature cracking in the extreme weather locations in New Mexico. In addition, the MEPDG thermal cracking module requires calibration.

- None of the 3213 pavements failed by surface-down cracking. For criteria: bottom-up cracking ≤ 20% at the end of 50 years, only 8 pavements failed.

- For AC rut≤ 0.25″ and fatigue cracking ≤ 20% at the end of 50 years (no rehab), 405 perpetual pavements are found to pass. However, all of these pavements have 15″ AC thickness for AADTT of 1750. No perpetual pavements can be found using 5000 and 10000 AADTT.

- For AC rut≤ 0.5″ and fatigue cracking ≤ 20% at the end of 50 years (thin resurfacing every 10 years), 37 perpetual pavements are found to have 11″ thickness for carrying 1750 AADTT. Perpetual pavements carrying 1750 and 5000 AADTT are also found using 10″ (9 pavements), 11″ (17 pavements), and 12″ AC thickness (13 pavements). Perpetual pavements carrying 10000 AADTT all have 16″ AC thickness (293 pavements).

- By increasing the subgrade resilient modulus *(M_R)* from 5000 psi to 15,500 psi (Default for A-5 material in MEPDG), subsequent rutting in this layer reduces by 60% and total rutting reduces by 30%. Therefore, improved subgrade is an important factor for perpetual pavement design.

- When RBL layer is removed in certain perpetual pavements, predicted AC and base/subgrade rutting increases significantly, as well as bottom-up cracking increases. However, this is not the case in all simulations. Some perpetual pavements without an RBL did not fail by rutting or bottom-up cracking. A combination of appropriate mix design and sufficient layer thickness may be the reason for this.

- Perpetual pavements are found both with and without rich binder layer (RBL). Life cycle cost analysis of these perpetual pavements shows that perpetual pavements that do not contain a RBL are the most economic design. Perpetual pavements of 10″ thickness (for 5000 AADTT) can be designed without RBL layers. Indeed, 2 perpetual pavements without RBL have shown to have performance similar to 7 perpetual pavements with RBL layer (10″ thickness, 5000 AADTT).

- A total of 8 perpetual pavements are studied for de-bonding. MEPDG analysis shows that 7 out of 8 perpetual pavements with de-bonded HMA layers fail due to top-down cracking criterion (< 700 ft/mi) at the end of 50 years. For bottom-up cracking ≤ 20%, 2 of the 8 pavements failed. Bottom-up cracking also significantly increased in the de-bonded pavements (150 – 700%). AC Rutting more than doubled (0.2 to 0.5) in all of the pavements due to t he de-bonding of all HMA layers. However, none of the pavements failed by AC rutting. When only the upper asphalt layers are de-bonded, a significant increase in top-down cracking is observed. A significant increase in bottom-up cracking is noted, as well as AC rutting. Bonding between surface and intermediate layers significantly reduces AC rutting.

- For New Mexico’s pavement conditions (using MEPDG), it is shown that fatigue cracking is not a major concern for designing perpetual pavements, rather rutting is more of a concern.

Chapter 8

IMPLEMENTATION PLAN

8.1 Perpetual Pavements for AADTT=5000

Based on the findings of this study, eight perpetual pavements can be implemented in the State of New Mexico. Table 8.1 presents these pavements which have shown to have the highest performance and the lowest thickness (10″). Material properties and pavement response data of these eight pavements are also shown. These perpetual pavements can carry up to 5000 annual average daily truck traffic (AADTT). This traffic is equivalent to 32 million ESALs at the end of 20 years, and 165 million ESALs at the end of 50 years. It can be seen that perpetual pavements can be designed with and without rich binder layers. The use of rich binder layers is an option for New Mexico DOT. However, the research team suggests excluding RBL from the perpetual designs as RBL (for example, US 70 Hondo Valley) can cause some moisture problems. RBLs are considered impermeable due to their high density (low air voids and high binder content) and moisture can accumulate above this layer and remain in the pavement, thus causing significant moisture damage. This can be avoided if a perpetual pavement does not contain a RBL. Therefore this study suggests two perpetual pavements (with and without RBL) as shown in Table 8.1 (Runs 2751 and 2744 are without RBL).

Table 8.1: Design Criteria of 10″ Perpetual Pavements Carrying 5000 AADTT

		Layer Thickness (in)					Mix Design			PG Binder			50 Yr. MEPDG Predicted Distress			10 Yr. Predicted Distress		
Run	Traffic (AADTT)	Surf	Itmd.	RBL	Total	GB	Surf. Mix	Itmd. Mix	RBL Mix	Surf. PG	Itmd. PG	Treated SG M_R (psi)	Top-Down Cracking (ft/mi)	Bottom-Up Cracking (%)	Thermal Cracking (ft/mi)	IRI (in/mi)	AC_1 Rut (in)	AC_2 + AC_3 Rut (in)
2712	5000	3	4	3	10	6	SP-III	SP-IV	SP-IV	76-22	70-22	8000	0.48	1.8	0	107.6	0.14	0.05
2711	5000	3	4	3	10	6	SP-III	SP-IV	SP-III	76-22	70-22	8000	0.41	2.1	0	107.5	0.14	0.05
2710	5000	3	4	3	10	6	SP-III	SP-IV	SP-II	76-22	70-22	8000	0.31	2.8	0	107.3	0.14	0.05
2744	5000	3	7	-	10	6	SP-III	SP-IV	-	76-22	70-22	8000	0.45	8.1	0	107.7	0.14	0.05
2726	5000	3	4	3	10	6	SP-IV	SP-III	SP-IV	76-22	76-22	8000	0.42	1.7	0	107.7	0.16	0.04
2725	5000	3	4	3	10	6	SP-IV	SP-II	SP-IV	76-22	76-22	8000	0.42	1.6	0	107.4	0.16	0.04
2724	5000	3	4	3	10	6	SP-IV	SP-II	SP-II	76-22	76-22	8000	0.27	2.5	0	107.2	0.16	0.04
2751	5000	3	7	-	10	6	SP-IV	SP-II	-	76-22	76-22	8000	0.14	11.8	0	107.1	0.16	0.03

8.2 Implementable Perpetual Pavements for New Mexico

Figure 8.1 presents an implementable perpetual pavement from Pavement trial No. 2751 that does not contain a rich binder layer. This pavement has 3″ surface layer and 7″ intermediate layer. A fine mix (SP-IV) is used in the surface layer and a coarse mix (SP-II) is used in the intermediate layer. Both or these HMA layers contain modified binders and 6% air voids. Fatigue cracking, at the end of 50 years, is about 12% which is well below the failure value of 20%. It shows virtually no top-down or low-temperature (transverse) cracking. Very low rutting is predicted in the surface layer (< 0.2″) and intermediate layer (< 0.05″) at the end of 50 years.

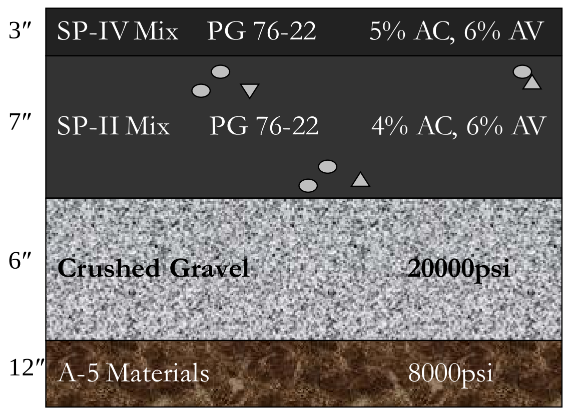


Figure 8.1 – Pavement Trial No. 2751

8.3 Optional Perpetual Pavement for New Mexico

Pavement trial no. 2725 is another choice of perpetual pavement and is presented in Figure 8.2. It has 3″ surface layer, 4″ intermediate layer, and 3″ rich binder layer. A fine mix (SP-IV) is used in both the surface and rich binder layers. A coarse mix (SP-II) is used in the intermediate layer. Superpave Performing Grade (PG) binders are used in the surface and intermediate layers, and a softer binder, PG 64-22, is used in the rich binder layer. Both the surface and intermediate layers are compacted to traditional 6% air voids, while the rich binder layer has to contain a non-traditional 3% air voids. This pavement is shown to have very low fatigue cracking (< 2%) and top-down cracking (< 0.5 ft/mi), as well as little or no rutting in the intermediate and base layers at the end of 50 years.

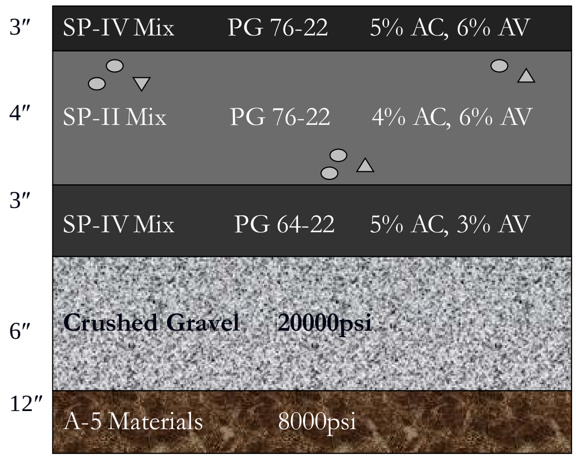


Figure 8.2 – Pavement Trial No. 2725

A key factor of any implementation plan is to have a successful resurfacing plan that fixes the rut in the top surface (if > 0.1″) every 10 years. This way, perpetual pavements can perform as expected and maintain a very high level of performance over the 50 year design life.

Another major concern of perpetual pavement, as well as any pavement, is potential de-bonding of HMA layers. De-bonding of HMA layers due to moisture can cause significant top-down and bottom-up cracking, as discussed in this report, which can lead to failure of these pavements. The selected perpetual pavements are implementable but procedures must be enforced to prevent moisture from initiating de-bonded environments. Currently, NMDOT does not have any specification for mix permeability to ensure that layer de-bonding does not occur. It is therefore required for NMDOT to determine the typical HMA mix permeability for designing moisture-resistant pavements.

This report recommends using MEPDG level 1 analysis as this will provide a clearer indication as to how the perpetual pavements will perform in this particular location (New Mexico). Level 1 analysis may include reduced dynamic modulus data which accounts for moisture damage in HMA mixes. The dynamic modulus data presented herein is taken from statewide projects. It would be more beneficial and economic for New Mexico if the NMDOT can implement level 1 dynamic modulus data into the MEPDG.

The perpetual pavements discovered in this study should be further validated by laboratory tested endurance limits, and field monitored stress/strain values at the bottom asphalt pavements. Using those fatigue endurance limits in MEPDG or any other multi-layer elastic program, it is possible to further verify that the suggested mixes are feasible for fabricating perpetual pavements in New Mexico. Usually, the fatigue endurance limit is compared to the tensile strain experienced at the bottom of the asphalt layer due to traffic loading. Therefore, field instrumentation is necessary to capture such strain values under real traffic. In conclusion, perpetual pavements discovered in this study should be further supported or validated by laboratory tested endurance limits, field monitoring of stress/strain values experienced in the asphalt pavement, and further investigation in the permeability of NMDOT mixes.

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APPENDIX A

Moisture Infiltration Test Results

A – 1. Moisture Retention Testing Results

HMA Sample 1

Saturated Hydraulic Conductivity
Flexible Wall Falling Head-Rising Tail Method

Job name: UNM Asphalt
Job number: 0
Sample number: 1 (fine)
Project: NA
Depth: NA

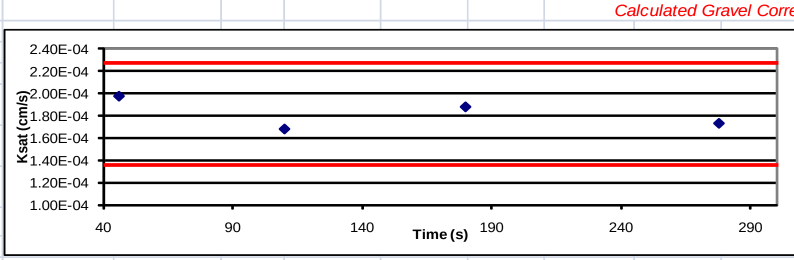
Remolded or Initial Sample Properties		Post Permeation Sample Properties		Test and Sample Conditions	
Initial Mass (g):	467.04	*Saturated Mass (g):*	477.7	*Permeant liquid used:*	Water
Diameter (cm):	10.224	*Dry Mass (g):*	467.04	*Sample Preparation:*	☐ In situ sample, extruded
Length (cm):	2.521	*Diameter (cm):*	10.224		☑ Remolded Sample
Area (cm^2):	82.10	*Length (cm):*	2.509	*Number of Lifts:*	NA
Volume (cm^3):	206.97	*Deformation (%)**:*	0.48	*Split:*	NA
Dry Density (g/cm^3):	2.26	*Area (cm^2):*	82.10	*Percent Coarse Material (%):*	NA
Dry Density (pcf):	140.87	*Volume (cm^3):*	205.98	*Particle Density(g/cm^3):*	2.44 ☑ Assumed ☐ Measured
Water Content (%, g/g):	0.0	*Dry Density (g/cm^3):*	2.27	*Cell pressure (PSI):*	70.0
Water Content (%, vol):	0.0	*Dry Density (pcf):*	141.55	*Influent pressure (PSI):*	68.0
Void Ratio (e):	0.08	*Water Content (%, g/g):*	2.3	*Effluent pressure (PSI):*	68.0
Porosity (%, vol):	7.5	*Water Content (%, vol):*	5.2	*Panel Used:*	☑ A ☐ B ☐ C
Saturation (%):	0.0	*Void Ratio(e):*	0.08	*Reading:*	☑ Annulus ☐ Pipette
		Porosity (%, vol):	7.1	B-Value (% saturation) prior to test*:	0.95
		Saturation (%):*	73.1	Date/Time:	2/4/09 945

* Per ASTM D5084 percent saturation is ensured (B-Value ≥ 95%) prior to testing, as post test saturation values may be exaggerated or skewed during depressurizing and sample removal.
**Percent Deformation: based on initial sample length and post permeation sample length.

Laboratory analysis by: D. O'Dowd
Data entered by: D. O'Dowd
Checked by: J. Hines

Date	Time	Temp (°C)	Influent Pipette Reading	Effluent Pipette Reading	Gradient (ΔH/ΔL)	Average Flow (cm^3)	Elapsed Time (s)	Ratio (outflow to inflow)	Change in Head (Not to exceed 25%)	k_{sat} T°C (cm/s)	k_{sat} Corrected (cm/s)
Test # 1:											
04-Feb-09	11:18:00	21.4	19.20	22.00	1.29	0.79	46	1.00	14%	2.04E-04	1.98E-04
04-Feb-09	11:18:46	21.4	19.40	21.80	1.10						
Test # 2:											
04-Feb-09	11:18:46	21.4	19.40	21.80	1.10	0.79	64	1.00	17%	1.74E-04	1.68E-04
04-Feb-09	11:19:50	21.4	19.60	21.60	0.92						
Test # 3:											
04-Feb-09	11:19:50	21.4	19.60	21.60	0.92	0.79	70	1.00	20%	1.94E-04	1.88E-04
04-Feb-09	11:21:00	21.4	19.80	21.40	0.74						
Test # 4:											
04-Feb-09	11:21:00	21.4	19.80	21.40	0.74	0.79	98	1.00	25%	1.79E-04	1.73E-04
04-Feb-09	11:22:38	21.4	20.00	21.20	0.55						

***Average Ksat* (cm/sec):** **1.82E-04**
Calculated Gravel Corrected Average Ksat (cm/sec): ---



ASTM Required Range (+/- 25%)
Ksat (-25%) (cm/s): 1.36E-04
Ksat (+25%) (cm/s): 2.27E-04

Saturated Hydraulic Conductivity
Flexible Wall Falling Head-Rising Tail Method

Job name:	UNM Asphalt
Job number:	0
Sample number:	2 (fine)
Project:	NA
Depth:	NA

Remolded or Initial Sample Properties		**Post Permeation Sample Properties**		**Test and Sample Conditions**	
Initial Mass (g):	449.25	*Saturated Mass (g):*	461.9	*Permeant liquid used:*	Water
Diameter (cm):	10.264	*Dry Mass (g):*	449.25	*Sample Preparation:*	☐ In situ sample, extruded
Length (cm):	2.41	*Diameter (cm):*	10.264		☑ Remolded Sample
Area (cm^2):	82.74	*Length (cm):*	2.392	*Number of Lifts:*	NA
Volume (cm^3):	199.41	*Deformation (%)**:*	0.75	*Split:*	NA
Dry Density (g/cm^3):	2.25	*Area (cm^2):*	82.74	*Percent Coarse Material (%):*	NA
Dry Density (pcf):	140.65	*Volume (cm^3):*	197.92	*Particle Density(g/cm^3):*	2.44 ☑ Assumed ☐ Measured
Water Content (%, g/g):	0.0	*Dry Density (g/cm^3):*	2.27	*Cell pressure (PSI):*	70.0
Water Content (%, vol):	0.0	*Dry Density (pcf):*	141.70	*Influent pressure (PSI):*	68.0
Void Ratio (e):	0.08	*Water Content (%, g/g):*	2.8	*Effluent pressure (PSI):*	68.0
Porosity (%, vol):	7.7	*Water Content (%, vol):*	6.4	*Panel Used:*	☐ D ☑ E ☐ F
Saturation (%):	0.0	*Void Ratio(e):*	0.07	*Reading:*	☐ Annulus ☑ Pipette
		Porosity (%, vol):	7.0	B-Value (% saturation) prior to test*:	0.83
		Saturation (%):*	91.7	Date/Time:	2/4/09 952

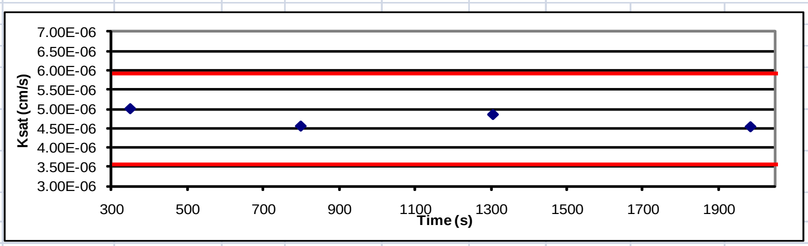
* Per ASTM D5084 percent saturation is ensured (B-Value ≥ 95%) prior to testing, as post test saturation values may be exaggerated during depressurizing and sample removal.
**Percent Deformation: based on initial sample length and post permeation sample length.

Laboratory analysis by:	D. O'Dowd
Data entered by:	D. O'Dowd
Checked by:	NA

Date	Time	Temp (°C)	Influent Pipette Reading	Effluent Pipette Reading	Gradient ($\Delta H/\Delta L$)	Average Flow (cm^3)	Elapsed Time (s)	Ratio (outflow to inflow)	Change in Head (Not to exceed 25%)	k_{sat} T°C (cm/s)	k_{sat} Corrected (cm/s)
Test # 1:											
04-Feb-09	11:10:02	21.4	14.80	17.80	1.45	0.17	350	1.00	13%	5.17E-06	5.00E-06
04-Feb-09	11:15:52	21.4	15.00	17.60	1.26						
Test # 2:											
04-Feb-09	11:15:52	21.4	15.00	17.60	1.26	0.17	449	1.00	15%	4.71E-06	4.55E-06
04-Feb-09	11:23:21	21.4	15.20	17.40	1.06						
Test # 3:											
04-Feb-09	11:23:21	21.4	15.20	17.40	1.06	0.17	506	1.00	18%	5.02E-06	4.85E-06
04-Feb-09	11:31:47	21.4	15.40	17.20	0.87						
Test # 4:											
04-Feb-09	11:31:47	21.4	15.40	17.20	0.87	0.17	678	1.00	22%	4.69E-06	4.54E-06
04-Feb-09	11:43:05	21.4	15.60	17.00	0.68						

***Average Ksat* (cm/sec):** **4.74E-06**

Calculated Gravel Corrected Average Ksat (cm/sec): ----



ASTM Required Range (+/- 25%)

Ksat (-25%) (cm/s):	3.55E-06
Ksat (+25%) (cm/s):	5.92E-06

HMA Sample 3

Saturated Hydraulic Conductivity
Flexible Wall Falling Head-Rising Tail Method

Job name:	UNM Asphalt
Job number:	0
Sample number:	3 (fine)
Project:	NA
Depth:	NA

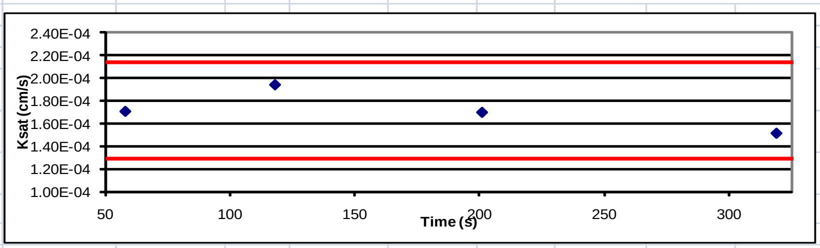
Remolded or Initial Sample Properties		Post Permeation Sample Properties		Test and Sample Conditions	
Initial Mass (g):	502.01	*Saturated Mass (g):*	519.5	*Permeant liquid used:*	Water
Diameter (cm):	10.2	*Dry Mass (g):*	502.01	*Sample Preparation:*	☐ In situ sample, extruded
Length (cm):	2.83	*Diameter (cm):*	10.2		☑ Remolded Sample
Area (cm^2):	81.71	*Length (cm):*	2.802	*Number of Lifts:*	NA
Volume (cm^3):	231.25	*Deformation (%)**:*	1.00	*Split:*	NA
Dry Density (g/cm^3):	2.17	*Area (cm^2):*	81.71	*Percent Coarse Material (%):*	NA
Dry Density (pcf):	135.52	*Volume (cm^3):*	228.96	*Particle Density(g/cm^3):*	2.44 ☑ Assumed ☐ Measured
Water Content (%, g/g):	0.0	*Dry Density (g/cm^3):*	2.19	*Cell pressure (PSI):*	70.0
Water Content (%, vol):	0.0	*Dry Density (pcf):*	136.88	*Influent pressure (PSI):*	68.0
Void Ratio (e):	0.12	*Water Content (%, g/g):*	3.5	*Effluent pressure (PSI):*	68.0
Porosity (%, vol):	11.0	*Water Content (%, vol):*	7.6	*Panel Used:*	☑ A ☐ B ☐ C
Saturation (%):	0.0	*Void Ratio(e):*	0.11	*Reading:*	☑ Annulus ☐ Pipette
		Porosity (%, vol):	10.1	B-Value (% saturation) prior to test*:	0.80
		Saturation (%):*	75.3	Date/Time:	2/4/09 947

* Per ASTM D5084 percent saturation is ensured (B-Value ≥ 95%) prior to testing, as post test saturation values may be exaggerated or skewed during depressurizing and sample removal.
**Percent Deformation: based on initial sample length and post permeation sample length.

Laboratory analysis by:	D. O'Dowd
Data entered by:	D. O'Dowd
Checked by:	J. Hines

Date	Time	Temp (°C)	Influent Pipette Reading	Effluent Pipette Reading	Gradient ($\Delta H/\Delta L$)	Average Flow (cm^3)	Elapsed Time (s)	Ratio (outflow to inflow)	Change in Head (Not to exceed 25%)	k_{sat} T°C (cm/s)	k_{sat} Corrected (cm/s)
Test # 1:											
04-Feb-09	10:02:27	21.3	17.60	20.50	1.20	0.79	58	1.00	14%	1.76E-04	1.71E-04
04-Feb-09	10:03:25	21.3	17.80	20.30	1.03						
Test # 2:											
04-Feb-09	10:03:25	21.3	17.80	20.30	1.03	0.79	60	1.00	16%	2.00E-04	1.94E-04
04-Feb-09	10:04:25	21.3	18.00	20.10	0.87						
Test # 3:											
04-Feb-09	10:04:25	21.3	18.00	20.10	0.87	0.79	83	1.00	19%	1.75E-04	1.70E-04
04-Feb-09	10:05:48	21.3	18.20	19.90	0.70						
Test # 4:											
04-Feb-09	10:05:48	21.3	18.20	19.90	0.70	0.79	118	1.00	24%	1.56E-04	1.52E-04
04-Feb-09	10:07:46	21.3	18.40	19.70	0.54						

***Average Ksat* (cm/sec):** **1.71E-04**
Calculated Gravel Corrected Average Ksat (cm/sec): ---



ASTM Required Range (+/- 25%)

Ksat (-25%) (cm/s):	1.29E-04
Ksat (+25%) (cm/s):	2.14E-04

HMA Sample 4

Saturated Hydraulic Conductivity
Flexible Wall Falling Head-Rising Tail Method

Job name: UNM Asphalt
Job number: 0
Sample number: 4 Coarse
Project: NA
Depth: NA

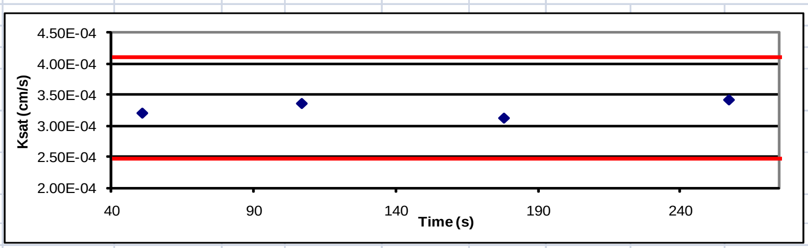
Remolded or Initial Sample Properties		Post Permeation Sample Properties		Test and Sample Conditions	
Initial Mass (g):	900.43	*Saturated Mass (g):*	930.8	*Permeant liquid used:*	Water
Diameter (cm):	10.26	*Dry Mass (g):*	900.43	*Sample Preparation:*	☐ In situ sample, extruded
Length (cm):	5.623	*Diameter (cm):*	10.26		☑ Remolded Sample
Area (cm^2):	82.68	*Length (cm):*	5.592	*Number of Lifts:*	NA
Volume (cm^3):	464.89	*Deformation (%)**:*	0.55	*Split:*	NA
Dry Density (g/cm^3):	1.94	*Area (cm^2):*	82.68	*Percent Coarse Material (%):*	NA
Dry Density (pcf):	120.91	*Volume (cm^3):*	462.33	*Particle Density(g/cm^3):*	2.39 ☑ Assumed ☐ Measured
Water Content (%, g/g):	0.0	*Dry Density (g/cm^3):*	1.95	*Cell pressure (PSI):*	70.0
Water Content (%, vol):	0.0	*Dry Density (pcf):*	121.58	*Influent pressure (PSI):*	68.0
Void Ratio (e):	0.23	*Water Content (%, g/g):*	3.4	*Effluent pressure (PSI):*	68.0
Porosity (%, vol):	19.0	*Water Content (%, vol):*	6.6	*Panel Used:*	☑ D ☐ E ☐ F
Saturation (%):	0.0	*Void Ratio(e):*	0.23	*Reading:*	☑ Annulus ☐ Pipette
		Porosity (%, vol):	18.5	B-Value (% saturation) prior to test*:	1.00
		Saturation (%):*	35.5	Date/Time:	2/4/09 950

* Per ASTM D5084 percent saturation is ensured (B-Value ≥ 95%) prior to testing, as post test saturation values may be exaggerated during depressurizing and sample removal.
**Percent Deformation: based on initial sample length and post permeation sample length.

Laboratory analysis by: D. O'Dowd
Data entered by: D. O'Dowd
Checked by: NA

Date	Time	Temp (°C)	Influent Pipette Reading	Effluent Pipette Reading	Gradient (ΔH/ΔL)	Average Flow (cm^3)	Elapsed Time (s)	Ratio (outflow to inflow)	Change in Head (Not to exceed 25%)	k_{sat} T°C (cm/s)	k_{sat} Corrected (cm/s)
Test # 1:											
04-Feb-09	10:50:04	21.3	18.70	21.95	0.67	0.76	51	1.00	12%	3.31E-04	3.21E-04
04-Feb-09	10:50:55	21.3	18.90	21.75	0.59						
Test # 2:											
04-Feb-09	10:50:55	21.3	18.90	21.75	0.59	0.76	56	1.00	14%	3.48E-04	3.37E-04
04-Feb-09	10:51:51	21.3	19.10	21.55	0.51						
Test # 3:											
04-Feb-09	10:51:51	21.3	19.10	21.55	0.51	0.76	71	1.00	16%	3.23E-04	3.13E-04
04-Feb-09	10:53:02	21.3	19.30	21.35	0.42						
Test # 4:											
04-Feb-09	10:53:02	21.3	19.30	21.35	0.42	0.76	79	1.00	20%	3.54E-04	3.43E-04
04-Feb-09	10:54:21	21.3	19.50	21.15	0.34						

***Average Ksat* (cm/sec):** **3.28E-04**
Calculated Gravel Corrected Average Ksat (cm/sec): ----



ASTM Required Range (+/- 25%)
Ksat (-25%) (cm/s): 2.46E-04
Ksat (+25%) (cm/s): 4.11E-04

A – 2. Retained Dynamic Modulus Test Data conducted by Pennsylvania State University

Table A-1 presents dynamic modulus test results of HMA specimens from six different states. This testing was conducted at Pennsylvania State University in 2007. The dynamic modulus testing was conducted with a uniaxial sinusoidal load inducing approximately 100 $\mu\varepsilon$ in the specimen. All dynamic modulus tests were conducted at 25°C. Selection of the 25°C test temperature was based on the findings of research under NCHRP Project 9-29 (Solaimanian 2007). Table A-1 presents the state DOT mix, along with the sample no. Also shown are the % air voids in the samples and degree of saturation of each sample. Testing frequencies vary from 1 – 25 Hz. Stress (kPa) and strain ($\mu\varepsilon$) measurements, and dynamic modulus values are recorded before and after saturation. A dynamic modus ratio (E*wet/E*dry) is then calculated based on these results. The retained dynamic modulus test results show a drop in modulus for all specimens after full conditioning. For a frequency of 25 Hz, it can be seen that HMA specimens from Oklahoma, Pennsylvannia, and Wyoming could not be taken due to problems during testing. However, 3 out of the six HMA mixes showed retained modulus ratios of 85% or greater from testing at 10 Hz. The remaining three HMA mixes all have retained modulus ratios less than 80% at this frequency.

Table A-1. Dynamic Modulus Test Results of State DOT Mixes conducted at the Pennsylvania State University

State DOT Mix	HMA Sample	Air Voids, %	Deg of Sat. %	Test Freq, Hz	Before Conditioning Stress (KPa)	Before Conditioning Strain (με)	Before Conditioning Modulu (MPa)	After Conditioning Stress (KPa)	After Conditioning Strain (με)	After Conditioning Modulus (MPa)	Moduli Ratio After/ Before
Georgia	307.26	7.6	75	25	599.9	102	5871	342.0	88	3878	0.66
				10	498.6	104	4815	258.7	87	2962	0.62
				5	422.5	104	4051	206.0	89	2324	0.57
				2	333.2	106	3148	160.5	98	1633	0.52
				1	260.0	103	2514	119.8	99	1208	0.48
Wisconsin	313.16	7.2	83	25	400.0	102	3921	277.3	92	3022	0.77
				10	290.2	99	2920	267.6	122	2188	0.75
				5	225.9	100	2256	173.6	110	1571	0.70
				2	152.0	99	1528	103.3	106	978	0.64
				1	111.9	100	1118	68.1	101	676	0.60
Kentucky	315.13	7.1	79	25	539.8	93	5805	451.8	86	5227	0.90
				10	429.4	94	4578	428.0	106	4053	0.89
				5	351.1	94	3734	342.4	107	3190	0.85
				2	283.7	104	2727	233.6	106	2205	0.81
				1	211.0	104	2030	173.0	107	1619	0.80
Oklahoma	316.21	7.1	67	25	NA	NA	NA	NA	NA	NA	NA
				10	558.4	97	5773	472.8	95	4953	0.86
				5	502.7	99	5078	412.5	97	4250	0.84
				2	404.7	98	4127	331.8	99	3351	0.81
				1	331.1	97	3397	275.2	101	2736	0.81
Pennsylvannia	317.11	7.0	80	25	NA	NA	NA	NA	NA	NA	NA
				10	484.2	99	4914	398.0	93	4278	0.87
				5	442.9	108	4092	335.1	98	3418	0.84
				2	335.9	109	3071	239.8	101	2381	0.78
				1	257.3	110	2335	174.0	98	1774	0.76
Wyoming	318.18	6.6	89	25	NA	NA	NA	NA	NA	NA	NA
				10	497.8	92	5400	317.2	95	3336	0.62
				5	430.7	94	4602	258.9	96	2699	0.59
				2	356.1	102	3504	205.6	103	1993	0.57
				1	282.7	100	2817	162.8	105	1546	0.55

APPENDIX B

Life Cycle Cost Analysis (LCCA)

National Lime Association LCCA Model

Perpetual Pavement **With** Rich Binder Layer Inputs

Lime Treated Alternative (Pavement Alternative 1): PERPETUAL PAVEMENT (RBL INCL.)

****

Project Information

****

Analysis Approach: Probabilistic
State: NM
County: County
Route: Route
Project ID: ID
Lane Configuration: Four; Divided
Starting Station: 1+50.75
Ending Station: 26+50.75
Project Length: 2500.00 feet
Lane 1 Included; Lane Width = 12.00 feet
Lane 2 Not Included
Lane 3 Not Included
Lane 4 Not Included
Lane 5 Not Included

****

Initial Construction Information

****

Mixture Type: Perpetual Pavements (Incl. RBL)
Production rate: 3.3 lane-mi./day
Expected Life (Mean): 50.0 years
Expected Life (Low): 40.0 years
Expected Life (High): 60.0 years
Initial Cost Based On: Unit Costs
Alternative Unit Cost (Mean): $398.00/sq yd - in
Alternative Unit Cost (Low): $358.00/sq yd – in
Alternative Unit Cost (High): $438.00/sq yd - in
HMA Layer Thickness: 10.00 in.

****

Maintenance Information

****

Time to Perform Maintenance Activities INCLUDED in Calculation of Delay Costs

Treatments Applied to Initial Construction Alternative

Maintenance Treatments NOT APPLIED to Initial Construction Alternative

Treatments Applied to Rehabilitation Alternative

Maintenance Treatments NOT APPLIED to Rehabilitation Alternative

~**
Rehabilitation Information
****
First Treatment

Treatment Type: 2 in. M/F + 1.5 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 7.0 years
Expected Life (High): 15.0 years
Unit Cost (Mean): $8.68/sq yd
Unit Cost (Low): $8.42/sq yd
Unit Cost (High): $8.94/sq yd
Production Rate: 5.0 lane-mi./day

Second Treatment

Treatment Type: 2 in. M/F + 1.5 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 7.0 years
Expected Life (High): 15.0 years
Unit Cost (Mean): $8.68/sq yd
Unit Cost (Low): $8.42/sq yd
Unit Cost (High): $8.94/sq yd
Production Rate: 5.0 lane-mi./day

Third Treatment

Treatment Type: 2 in. M/F + 1.5 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 7.0 years
Expected Life (High): 15.0 years
Unit Cost (Mean): $8.68/sq yd
Unit Cost (Low): $8.42/sq yd
Unit Cost (High): $8.94/sq yd

Production Rate: 5.0 lane-mi./day

Fourth Treatment

Treatment Type: 2 in. M/F + 1.5 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 7.0 years
Expected Life (High): 15.0 years
Unit Cost (Mean): $8.68/sq yd
Unit Cost (Low): $8.42/sq yd
Unit Cost (High): $8.94/sq yd
Production Rate: 5.0 lane-mi./day

****
User Costs Information
****
Facility: High Volume
Lane Rental Fee: $10000.00/lane-mi./day

****

Perpetual Pavement **With** Rich Binder Layer - Results

Analysis Period: 50 years
Real Discount Rate (Mean): 4.00%
Real Discount Rate (Min): 2.50%
Real Discount Rate (Max): 5.50%
Number of Iterations: 500

Description	Mean	Min.	Max.	Std. Dev.
Life-Cycle Cost ($)	13303049	11937588	14591304	691622
Life-Cycle Cost ($/sq yd)	3991	3581	4377	207
Life-Cycle Cost ($/lane-mi)	28096040	25212186	30816833	1460706
Discount Rate (%)	4.0	2.5	5.5	0.8
Life of Initial Construction Alternative (years)	49.7	40.3	59.9	4.9
Unit Cost of Initial Construction Alternative	398.8	358.1	437.7	20.7
Life of 1st Rehabilitation Alternative (years)	10.7	7.0	15.0	2.2
Unit Cost of 1st Rehabilitation Alternative	8.7	8.4	8.9	0.1
Life of 2nd Rehabilitation Alternative (years)	10.5	7.0	15.0	2.1
Unit Cost of 2nd Rehabilitation Alternative	8.7	8.4	8.9	0.1
Life of 3rd Rehabilitation Alternative (years)	10.7	7.0	15.0	2.2
Unit Cost of 3rd Rehabilitation Alternative	8.7	8.4	8.9	0.1
Life of 4th Rehabilitation Alternative (years)	10.6	7.0	15.0	2.0
Unit Cost of 4th Rehabilitation Alternative	8.7	8.4	8.9	0.1

National Lime Association LCCA

Perpetual Pavement **Without** Rich Binder Layer Inputs

Lime Treated Alternative (Pavement Alternative 1): PERPETUAL PAVEMENT (NO RBL)
****
Project Information
****
Analysis Approach: Probabilistic
State: NM
County: County
Route: Route
Project ID: ID
Lane Configuration: Four; Divided
Starting Station: 1+50.75
Ending Station: 26+50.75
Project Length: 2500.00 feet
Lane 1 Included; Lane Width = 12.00 feet
Lane 2 Not Included
Lane 3 Not Included
Lane 4 Not Included
Lane 5 Not Included

****
Initial Construction Information
****
Mixture Type: Perpetual Pavement (No RBL)
Production rate: 3.3 lane-mi./day
Expected Life (Mean): 50.0 years
Expected Life (Low): 40.0 years
Expected Life (High): 60.0 years
Initial Cost Based On: Unit Costs
Alternative Unit Cost (Mean): $278.00/sq yd - in
Alternative Unit Cost (Low): $250.00/sq yd - in
Alternative Unit Cost (High): $306.00/sq yd - in
HMA Layer Thickness: 10.00 in.

****
Maintenance Information
****

Time to Perform Maintenance Activities INCLUDED in Calculation of Delay Costs

Treatments Applied to Initial Construction Alternative

Maintenance Treatments NOT APPLIED to Initial Construction Alternative

Treatments Applied to Rehabilitation Alternative

Maintenance Treatments NOT APPLIED to Rehabilitation Alternative

~***
Rehabilitation Information
****
First Treatment

Treatment Type: 2 in. M/F + 2 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 8.0 years
Expected Life (High): 12.0 years
Unit Cost (Mean): $8.57/sq yd
Unit Cost (Low): $8.31/sq yd
Unit Cost (High): $8.83/sq yd
Production Rate: 4.4 lane-mi./day

Second Treatment

Treatment Type: 2 in. M/F + 1.5 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 7.0 years
Expected Life (High): 15.0 years
Unit Cost (Mean): $8.68/sq yd
Unit Cost (Low): $8.42/sq yd
Unit Cost (High): $8.94/sq yd
Production Rate: 5.0 lane-mi./day

Third Treatment

Treatment Type: 2 in. M/F + 1.5 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 7.0 years
Expected Life (High): 15.0 years
Unit Cost (Mean): $8.68/sq yd

Unit Cost (Low): $8.42/sq yd
Unit Cost (High): $8.94/sq yd
Production Rate: 5.0 lane-mi./day

Fourth Treatment

Treatment Type: 2 in. M/F + 1.5 in. Overlay; Lime-Treated
Expected Life (Mean): 10.0 years
Expected Life (Low): 7.0 years
Expected Life (High): 15.0 years
Unit Cost (Mean): $8.68/sq yd
Unit Cost (Low): $8.42/sq yd
Unit Cost (High): $8.94/sq yd
Production Rate: 5.0 lane-mi./day

****
User Costs Information
****
Facility: High Volume
Lane Rental Fee: $10000.00/lane-mi./day

****

Perpetual Pavement **Without** Rich Binder Layer - Results

Analysis Period: 50 years
Real Discount Rate (Mean): 4.00%
Real Discount Rate (Min): 2.50%
Real Discount Rate (Max): 5.50%
Number of Iterations: 500

Description	Mean	Min	Max	Std. Dev
Life-Cycle Cost ($)	9290041	8377858	10185132	459128
Life-Cycle Cost ($/sq yd)	2787	2513	3056	138
Life-Cycle Cost ($/lane-mi)	19620567	17694037	21510998	969679
Discount Rate (%)	4.1	2.5	5.5	0.8
Life of Initial Construction Alternative (years)	50.2	40.1	59.8	5.1
Unit Cost of Initial Construction Alternative	278.4	250.9	305.6	13.7
Life of 1st Rehabilitation Alternative (years)	9.9	8.0	12.0	1.1
Unit Cost of 1st Rehabilitation Alternative	8.6	8.3	8.8	0.1
Life of 2nd Rehabilitation Alternative (years)	10.8	7.0	15.0	2.1
Unit Cost of 2nd Rehabilitation Alternative	8.7	8.4	8.9	0.1
Life of 3rd Rehabilitation Alternative (years)	10.5	7.0	15.0	2.2
Unit Cost of 3rd Rehabilitation Alternative	8.7	8.4	8.9	0.1
Life of 4th Rehabilitation Alternative (years)	10.7	7.0	15.0	2.1
Unit Cost of 4th Rehabilitation Alternative	8.7	8.4	8.9	0.1

FHWA Life Cycle Cost Analysis Model Inputs

INPUT WORKSHEET	
1. Economic Variables	
Value of Time for Passenger Cars ($/hour)	$0.70
Value of Time for Single Unit Trucks ($/hour)	$0.83
Value of Time for Combination Trucks ($/hour)	$0.83
2. Analysis Options	
Include User Costs in Analysis	Yes
Include User Cost Remaining Life Value	Yes
Use Differential User Costs	Yes
User Cost Computation Method	Specified
Include Agency Cost Remaining Life Value	Yes
Traffic Direction	Both
Analysis Period (Years)	50
Beginning of Analysis Period	2009
Discount Rate (%)	4.0
Number of Alternatives	2
3. Project Details	
State Route	NM I-40
Project Name	Optimal Perpetual Pavement
Region	-
County	-
Analyzed By	Damien Bateman
Mileposts	
Begin	100.00
End	103.00
Length of Project (miles)	3.00
Comments	
4. Traffic Data	
AADT Construction Year (total for both directions)	14,535
Cars as Percentage of AADT (%)	55.0
Single Unit Trucks as Percentage of AADT (%)	35.0
Combination Trucks as Percentage of AADT (%)	10.0
Annual Growth Rate of Traffic (%)	4.0
Speed Limit Under Normal Operating Conditions (mph)	70
No of Lanes in Each Direction During Normal Conditions	2
Free Flow Capacity (vphpl)	
Rural or Urban Hourly Traffic Distribution	Urban
Queue Dissipation Capacity (vphpl)	
Maximum AADT (total for both directions)	14,535
Maximum Queue Length (miles)	

Perpetual Pavement **Without** Rich Binder Layer

5. Construction

Alternative 1	Perpetual Pavement (No RBL)	
Number of Activities	5	
Activity 1	Intial Construction	
Agency Construction Cost ($1000)	$1.835 mil	
User Work Zone Costs ($1000)	$60,000.00	
Work Zone Duration (days)	90	
No of Lanes Open in Each Direction During Work Zone	1	
Activity Service Life (years)	50.0	
Activity Structural Life (years)	50.0	
Maintenance Frequency (years)	10	
Agency Maintenance Cost ($1000)	174000	
Work Zone Length (miles)	3.00	
Work Zone Speed Limit (mph)	45	
Work Zone Capacity (vphpl)		
Traffic Hourly Distribution	Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7
Outbound	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7
Activity 2	Surface Rehab	
Agency Construction Cost ($1000)	$360,000.00	
User Work Zone Costs ($1000)	$50,000.00	
Work Zone Duration (days)	7	
No of Lanes Open in Each Direction During Work Zone	1	
Activity Service Life (years)	10.0	
Activity Structural Life (years)	10.0	
Maintenance Frequency (years)	10	
Agency Maintenance Cost ($1000)	174000	
Work Zone Length (miles)	3.00	
Work Zone Speed Limit (mph)	45	
Work Zone Capacity (vphpl)		
Traffic Hourly Distribution		

Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)

Inbound

	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Outbound

	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Activity 3	Surface Rehab
Agency Construction Cost ($1000)	$360,000.00
User Work Zone Costs ($1000)	$50,000.00
Work Zone Duration (days)	7
No of Lanes Open in Each Direction During Work Zone	1
Activity Service Life (years)	10.0
Activity Structural Life (years)	10.0
Maintenance Frequency (years)	10
Agency Maintenance Cost ($1000)	174000
Work Zone Length (miles)	3.00
Work Zone Speed Limit (mph)	45
Work Zone Capacity (vphpl)	
Traffic Hourly Distribution	

Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)

Inbound

	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Outbound

	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Activity 4	Surface Rehab
Agency Construction Cost ($1000)	$360,000.00
User Work Zone Costs ($1000)	$50,000.00
Work Zone Duration (days)	7
No of Lanes Open in Each Direction During Work Zone	1
Activity Service Life (years)	10.0
Activity Structural Life (years)	10.0
Maintenance Frequency (years)	10
Agency Maintenance Cost ($1000)	174000
Work Zone Length (miles)	3.00
Work Zone Speed Limit (mph)	45
Work Zone Capacity (vphpl)	
Traffic Hourly Distribution	

Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)

Inbound	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Outbound	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Activity 5	Surface Rehab
Agency Construction Cost ($1000)	$360,000.00
User Work Zone Costs ($1000)	$50,000.00
Work Zone Duration (days)	7
No of Lanes Open in Each Direction During Work Zone	1
Activity Service Life (years)	10.0
Activity Structural Life (years)	10.0
Maintenance Frequency (years)	10
Agency Maintenance Cost ($1000)	174000
Work Zone Length (miles)	3.00
Work Zone Speed Limit (mph)	45
Work Zone Capacity (vphpl)	
Traffic Hourly Distribution	

Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)

Inbound	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Outbound	Start	End
First period of lane closure	0	8
Second period of lane closure	23	7
Third period of lane closure	23	7

Perpetual Pavement **With** a Rich Binder Layer

Alternative 2	Perpetual Pavement (RBL Incl.)	
Number of Activities	5	
Activity 1	Initial Construction	
Agency Construction Cost ($1000)	$2,100,000.00	
User Work Zone Costs ($1000)	$60,000.00	
Work Zone Duration (days)	90	
No of Lanes Open in Each Direction During Work Zone	1	
Activity Service Life (years)	50.0	
Activity Structural Life (years)	50.0	
Maintenance Frequency (years)	10	
Agency Maintenance Cost ($1000)	174000	
Work Zone Length (miles)	3.00	
Work Zone Speed Limit (mph)	45	
Work Zone Capacity (vphpl)	1000	
Traffic Hourly Distribution	Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24
Outbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24
Activity 2	Surface Rehab	
Agency Construction Cost ($1000)	$350,000.00	
User Work Zone Costs ($1000)	$50,000.00	
Work Zone Duration (days)	7	
No of Lanes Open in Each Direction During Work Zone	1	
Activity Service Life (years)	10.0	
Activity Structural Life (years)	10.0	
Maintenance Frequency (years)	10	
Agency Maintenance Cost ($1000)	174000	
Work Zone Length (miles)	3.00	
Work Zone Speed Limit (mph)	45	

Work Zone Capacity (vphpl)	1000	
Traffic Hourly Distribution	Week Day 1	

Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)

Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24

Outbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24

Activity 3	Surface Rehab
Agency Construction Cost ($1000)	$350,000.00
User Work Zone Costs ($1000)	$50,000.00
Work Zone Duration (days)	7
No of Lanes Open in Each Direction During Work Zone	1
Activity Service Life (years)	10.0
Activity Structural Life (years)	10.0
Maintenance Frequency (years)	10
Agency Maintenance Cost ($1000)	174000
Work Zone Length (miles)	3.00
Work Zone Speed Limit (mph)	45
Work Zone Capacity (vphpl)	1000
Traffic Hourly Distribution	Week Day 1

Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)

Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24

Outbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24

Activity 4	Surface Rehab
Agency Construction Cost ($1000)	$350,000.00
User Work Zone Costs ($1000)	$50,000.00
Work Zone Duration (days)	7

No of Lanes Open in Each Direction During Work	1	
Activity Service Life (years)	10.0	
Activity Structural Life (years)	10.0	
Maintenance Frequency (years)	10	
Agency Maintenance Cost ($1000)	174000	
Work Zone Length (miles)	3.00	
Work Zone Speed Limit (mph)	45	
Work Zone Capacity (vphpl)	1000	
Traffic Hourly Distribution	Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	8
Second period of lane closure	21	8
Third period of lane closure	21	8
Outbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24
Activity 5	Surface Rehab	
Agency Construction Cost ($1000)	$350,000.00	
User Work Zone Costs ($1000)	$50,000.00	
Work Zone Duration (days)	7	
No of Lanes Open in Each Direction During Work Zone	1	
Activity Service Life (years)	10.0	
Activity Structural Life (years)	10.0	
Maintenance Frequency (years)	10	
Agency Maintenance Cost ($1000)	174000	
Work Zone Length (miles)	3.00	
Work Zone Speed Limit (mph)	45	
Work Zone Capacity (vphpl)	1000	
Traffic Hourly Distribution	Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24
Outbound	Start	End
First period of lane closure	0	5
Second period of lane closure	9	15
Third period of lane closure	20	24

FHWA Life Cycle Cost Analysis - Results

Total Cost				
	Alternative 1: Perpetual Pavement (No RBL)		**Alternative 2: Perpetual Pavement (RBL Incl.)**	
Total Cost	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Undiscounted Sum	*$2,531,000.00*	*$766.01*	*$2,796,000.00*	*$766.01*
Present Value	**$2,121,849.25**	**$766.01**	**$2,386,849.25**	**$766.01**
EUAC	$98,772.51	$35.66	$111,108.31	$35.66

Lowest Present Value Agency Cost	**Alternative 1: Perpetual Pavement (No RBL)**
Lowest Present Value User Cost	**Alternative 1: Perpetual Pavement (No RBL)**

NMDOT Treatment Costs

Treatment	Mean Cost/Lane Mile ($)	Min Cost/Lane Mile ($)	Max Cost/Lane Mile ($)	Source
Fog Sealing	1,156			NM DOT (1997)
	2,125	1,750	2,500	Nebraska DOT
Crack Sealing	14,600			NM DOT (1997)
	0.575/lin.ft^2	0.55/lin.ft^2	0.60/lin.ft^2	Nebraska DOT
Chip Sealing	7,893			NM DOT (1997)
	8,500	8,000	9,000	Nebraska DOT
Scrub Sealing	7,500	7,000	8,000	Nebraska DOT
Slurry Sealing	42,500	40,000	45,000	Nebraska DOT
Open Graded Friction Course (OGFC) Overlay	32,160			NM DOT (1997)
Plant Mix Wearing Course Overlay (Nova Chip)	43,400			NM DOT (1997)
2" Hot Mix Overlay	80,960			NM DOT (1997)
In Plant Recycle (Brazer)	49,000			NM DOT (1997)
Heater Scarification & Overlay (Cutler)	64,125			NM DOT (1997)
Microsurfacing	58,560			NM DOT (1997)
	42,000	41,000	43,000	Nebraska DOT
Mill (1")	8,000	7,500	8,500	Nebraska DOT
Cold Mill/Inlay	359,000			NM DOT (1997)
Cold In-Situ Recycle Overlay	350,000			NM DOT (1997)
Cold-in-Place Recycle	107,500	100,000	115,000	Nebraska DOT
Hot-in-Place Recycle	23,500	22,000	25,000	Nebraska DOT
Thin Cold Mix Overlay	21,500	18,000	25,000	Nebraska DOT
Thin Hot Mix Overlay (1")	50,000	45,000	55,000	Nebraska DOT
Pavement Extension Program (2" PEP)	100,000	80,000	120,000	Nebraska DOT
Thick Overlat (5")	205,000	195,000	215,000	Nebraska DOT
Rehabilitation	750,000			NM DOT (1997)
Reconstruction	1,835,000			NM DOT (1997)
	537,500	525,000	550,000	Nebraska DOT

APPENDIX C

KENLAYER Outputs (De-Bonding)

Effect of Bonding on Flexible Pavement Performance, Strain and Damage

Introduction

In the present, most of the modern theories and methods for analysis and design of flexible pavements assume that there is a complete bonding between all the layers. In complete bonding conditions, the stresses at the bottom of a layer will be entirely transferred to the top of the layer below the earlier, and the displacements at these points will be the same. These assumptions were made to facilitate the modeling of a layered asphalt system and its solution. However, in reality, these assumptions are not completely satisfied. The materials used for the different layers are different from each other, so the response to loading will not be the same. Also, the materials used consist of soil and grains, which have small voids between their particles, these voids are frictionless and complete bonding between the layers cannot be achieved. Unless a new series of high friction materials is developed and used for pavements, or the loads on the pavement decrease, the design of pavements based on completely bonding conditions could be unreliable in the future, with traffic volumes, traffic speed and tire pressures higher than in the present. Thus, a new model may be required to address the incapacity of the pavements to carry new loading conditions.

1. Background

The models reviewed by this research are: (i) the pavement as a homogeneous mass and (i) the layered system.

1.1 Homogeneous mass

This is one of the simplest ways to characterize the behavior of a flexible pavement under wheel loads. It consists of a homogeneous half-space with an infinitely large area, an infinite depth and a top plane on which the loads are applied. This theory was originally developed by Boussinesq (1885), and was used to determine the stresses, strains and deflections in the subgrade if the modulus ratio between the pavement and the subgrade is close to unity. This theory models the pavement as a single mass with a linear or non-linear behavior, so a complete bonding is assumed.

1.2 Layered system

Flexible pavements are layered systems with different materials and cannot be represented by a homogeneous mass. Burmister (1943) developed solutions for two-layer and three-layer systems. Huang (1967, 1968) applied the theory to a multilayer system with any number of layers. The basic assumptions to be satisfied are:

- Each layer is homogeneous, isotropic and linearly elastic with an elastic modulus *E* and a Poisson ratio *v*.
- The material is weightless and infinite in a real extent.
- Each layer has a finite thickness *h*, except for the lowest layer, which has an infinite thickness.
- A uniform pressure *q* is applied on the surface over a circular area of radius *a*.
- Continuity conditions are satisfied at the layer interfaces, as indicated by the same vertical stress, shear stress, vertical displacement and radial displacement. For frictionless interface, the continuity of shear stress and radial displacement is replaced by zero shear stress at each side of the interface.

This last assumption is of great importance in this topic. The theory assumes either complete or none bonding between the layers. Both conditions are far from what really happens in a pavement system.

2. Methods and Models Used

2.1 The Method

The method used consisted in recording the different pavement performance and response of the model used, by giving several values to key model characteristics, under bonding and non-bonding conditions. Regarding the pavement response to the loads applied to the model, the strain and damage of the model were analyzed and, as for the pavement performance, rutting and IRI were studied.

2.2 Tools Used

Specialized software for pavement analysis and design were used in the study. For the analysis of the pavement response, the KENLAYER program, developed by Huang at the University of Kentucky, was used. In the case of the pavement performance, the program used was the Mechanistic-Empirical Pavement Design Guide, developed by the National Cooperative Highway Research Program.

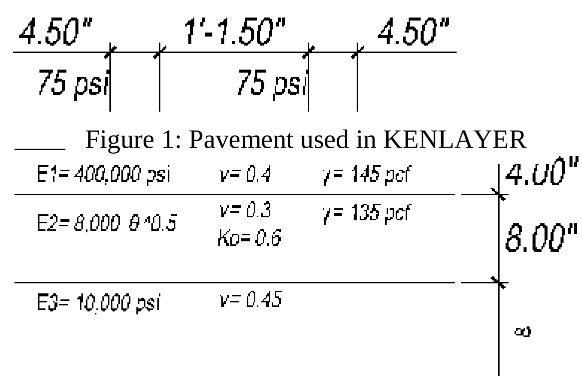


Figure 1: Pavement used in KENLAYER

2.2.1 KENLAYER

The used KENLAYER computer program applies only to flexible pavements with no joints or rigid layers. The backbone of KENLAYER is the solution for an elastic multilayer system under a circular loaded area. KENLAYER can be applied to layer systems under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, linear elastic, nonlinear elastic, or viscoelastic. KENLAYER, together with input program LAYERINP and graphic program LGRAPH, is part of a computer package called KENPAVE. LAYERINP facilitates entering and editing data, the program uses menus and data entry forms to create and edit the data file. Although the large number of input parameters appears overwhelming, default values are provided to many of them, so only a limited number of inputs will be required [3], [4].

2.3 The Models Used

For the analysis of the pavement response to the load, a model consisting of a three-layered system loaded by a dual-tire single-axle, as shown in Figure 1, was considered. The upper layer is hot mix asphalt (HMA) and lies over an intermediate layer of granular base. Finally, the subgrade is semi-infinite. All the parameters and thicknesses of each layer are shown in Figure.1. By using KENLAYER, the tensile strain at the bottom of the HMA layer and the compressive strain at the top of the subgrade were computed for both bonding and non-bonding conditions. Changes were made in the following parameters:

- Tire pressure
- Modulus of elasticity
- Layer thickness

In order to study the pavement performance, a similar model was used, consisting of a three-layered system, in which the upper layer is asphalt concrete, the intermediate a granular base and finally the subgrade which is the last layer. The Asphalt Concrete layer is 5 in-thick and has the following typical aggregate gradation: 4% retained at ¾ in. sieve, 41% retained at 3/8 in. sieve, 61% retained at No.4 sieve and 3% passing the No. 200 sieve. To provide non-bonding conditions, a crushed gravel layer was considered as the granular base; for the bonding case, a cement stabilized layer was used. The granular base is 10 in-thick and was assumed an A-3 subgrade, both of them with the program default values. Also, a design life of 10 years, a two-way annual average daily traffic -AADT- of 2,000, and 90% of heavy traffic were considered. The MEPDG program was used applied to analyze the previous model, and cracking, rutting and the International Roughness Index -IRI- for bonding and non-bonding conditions were obtained. Changes were done in the following parameters:

- Tire pressure
- Layer thickness
- AC penetration grade

3. Results and discussion

3.1 Response Analysis

From the analysis made with the KENLAYER program, the following results were obtained:

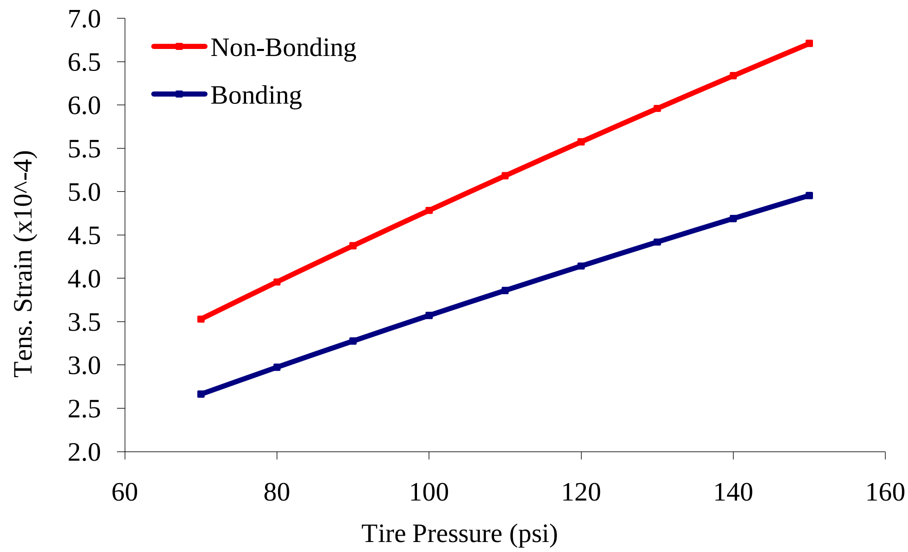


Figure 2. Effects of Bonding on Tensile Strain due to changes in Tire Pressure

In Figure 2 it is noticeable that tensile strains are higher in a frictionless interface, and that this last condition is even more critical as the load increases. This is because the layers' materials do not have a high tensile strength, and more importantly, the interface bonding restrains the tensile stresses.

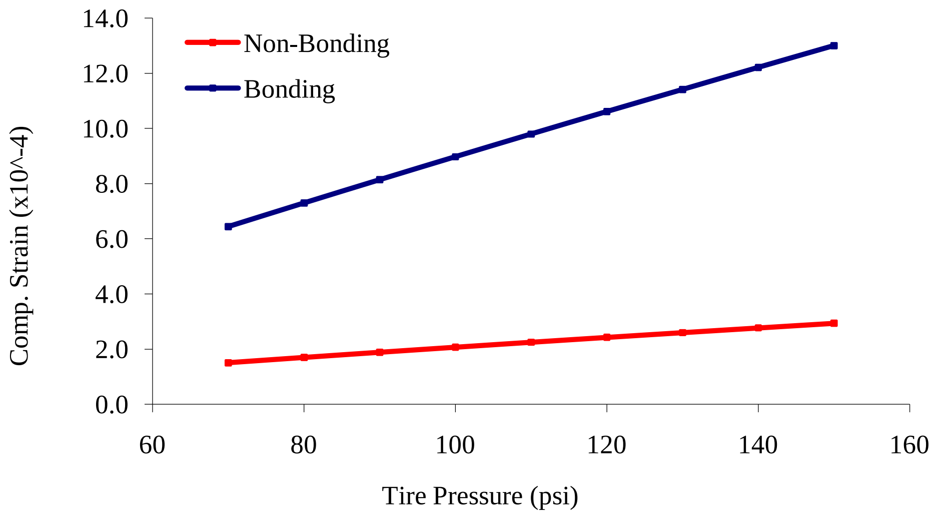


Figure 3. Effects of Bonding on Compressive Strain due to changes in Tire Pressure

In Figure 3 it is shown that the bonded interface has higher compressive strains than the frictionless interface. This is because the tensile strains dissipate the load pressure, so, a restrain over the tensile strains will produce higher compressive strains.

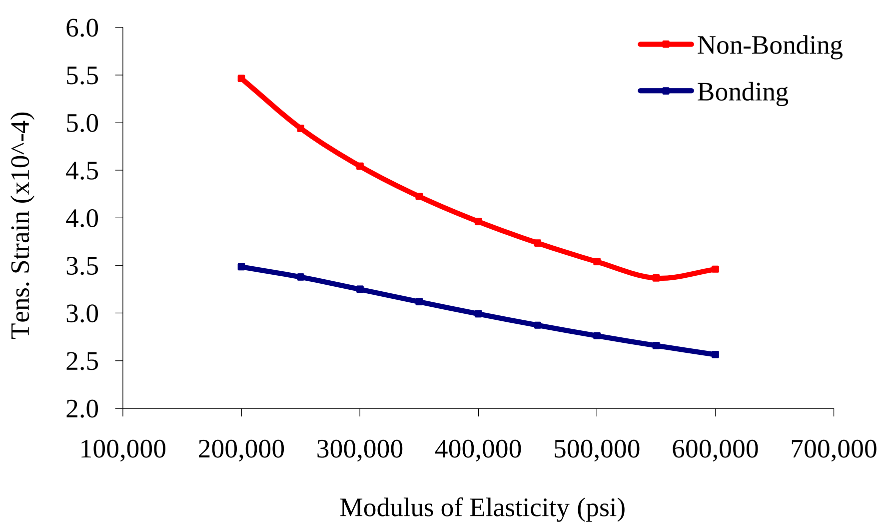


Figure 4. Effects of Bomding on Tensile Strain due to changes in Modulus of Elasticity

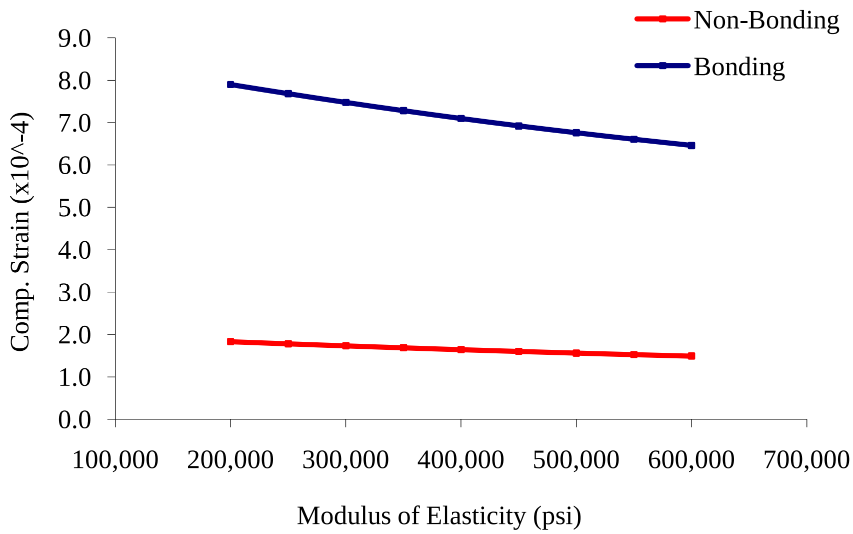


Figure 5. Effects of Bonding on Compressive Strain due to changes in Modulus of Elasticty

In Figures 4 and 5 can be observed that the influence of the load and the strength of the pavement are similar. Increasing the load produced similar results than the ones obtained decreasing the pavement strength. It should be noted, that the tensile strain in the last simulation of the non-bonding series in Fig.4 is higher than the previous, further analysis should be done to determine if this behavior is maintained with increase of the pavement strength.

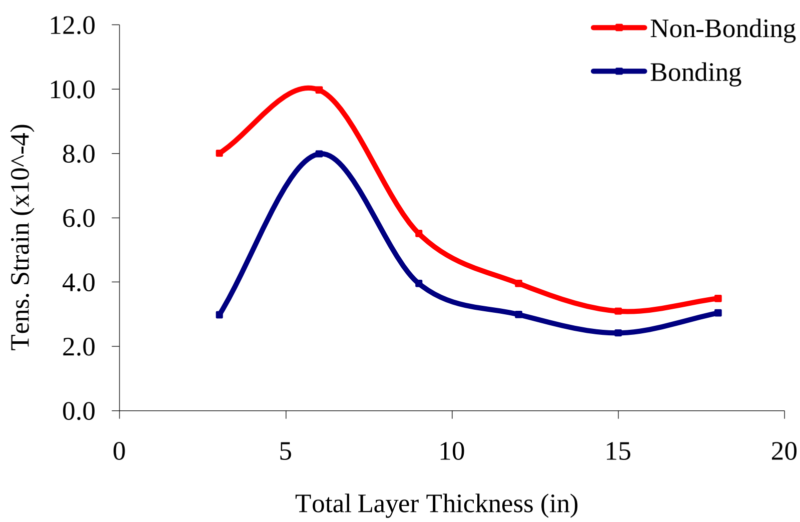


Figure 6. Effects of Bonding on Tensile Strain due to changes in Layer Thickness

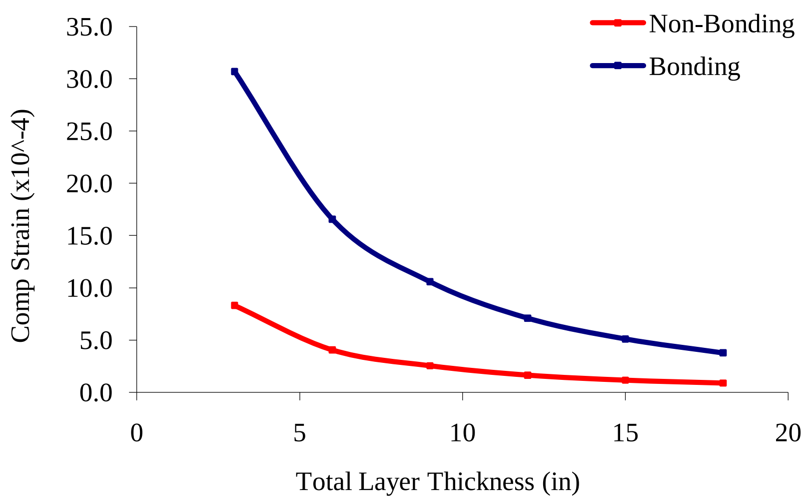


Figure 7. Effects of Bonding on Compressive Strain due to changes in Layer Thickness

In Figures 6 and 7 it is observed that as the thickness of the pavement system increases the strains decrease. There is an interesting zone, between the firsts and the thirds simulation of both series, in which the strains increase from the first to the second simulation, and then the strains decrease in the third simulation. Further analysis should be done inside that zone to determine the causes of that behavior.

3.1 Performance Analysis

From the analysis made through the MEPDG program, the following results were obtained:

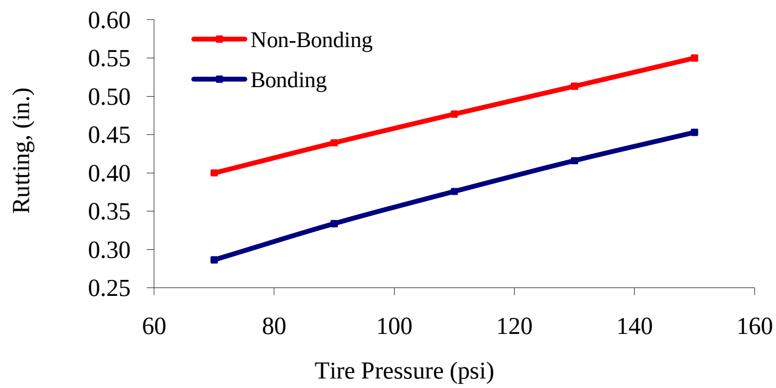


Figure 8. Effects of Bonding on Rutting due to changes in Tire Pressure

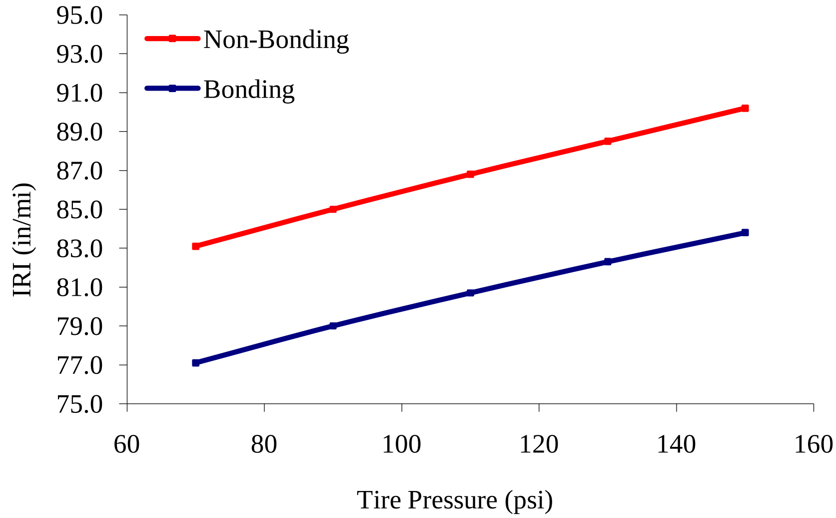


Figure 9. Effects of Bonding on IRI due to changes in Tire Pressure

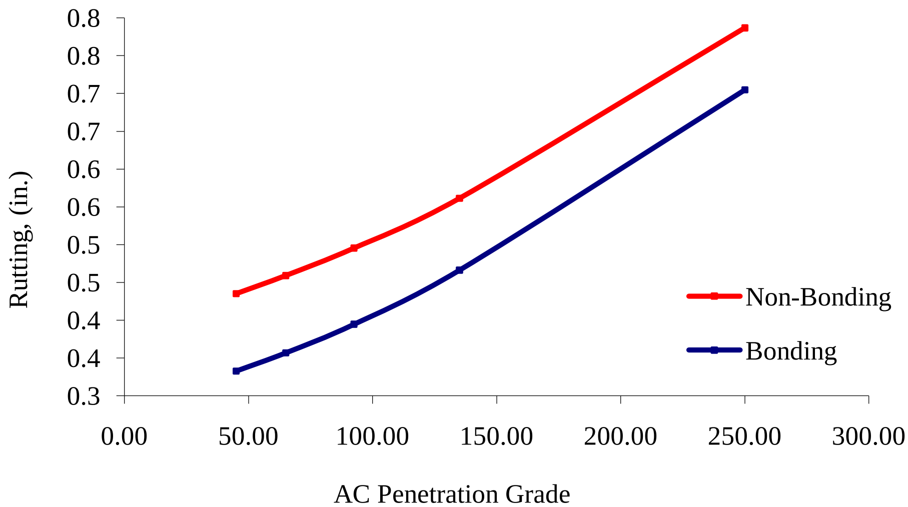


Figure 10. Effects of Bonding on Rutting due to changes in AC Penetration Grade

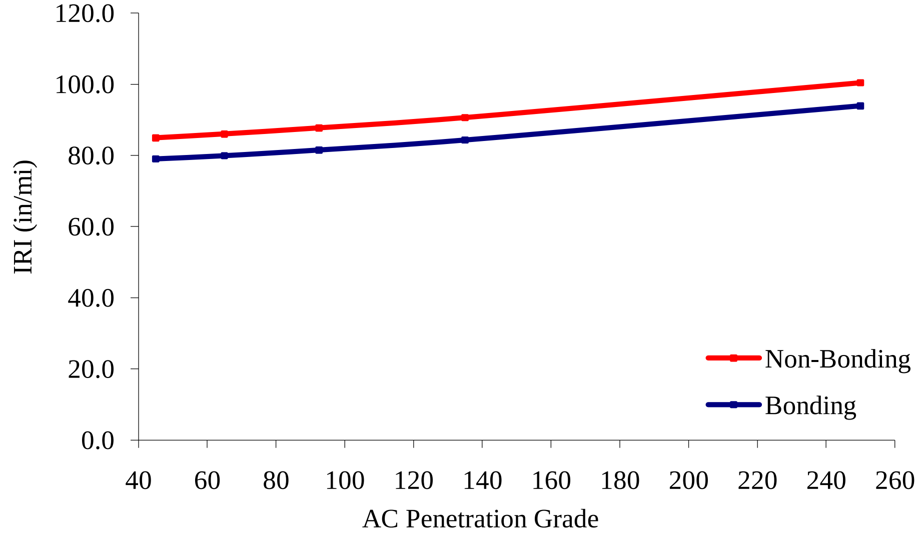


Figure 11. Effects of Bonding on IRI due to changes in AC Penetration Grade

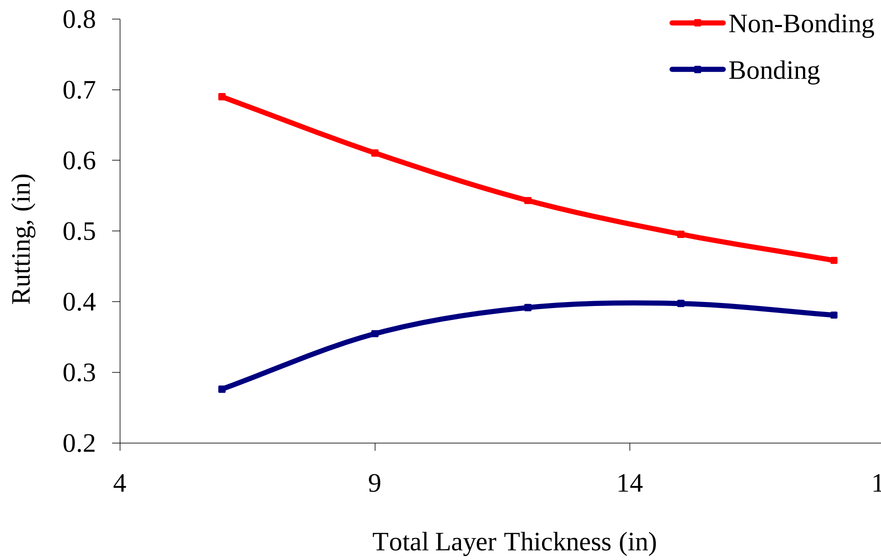


Figure 12. Effects of Bonding on Rutting due to changes in Layer Thickness

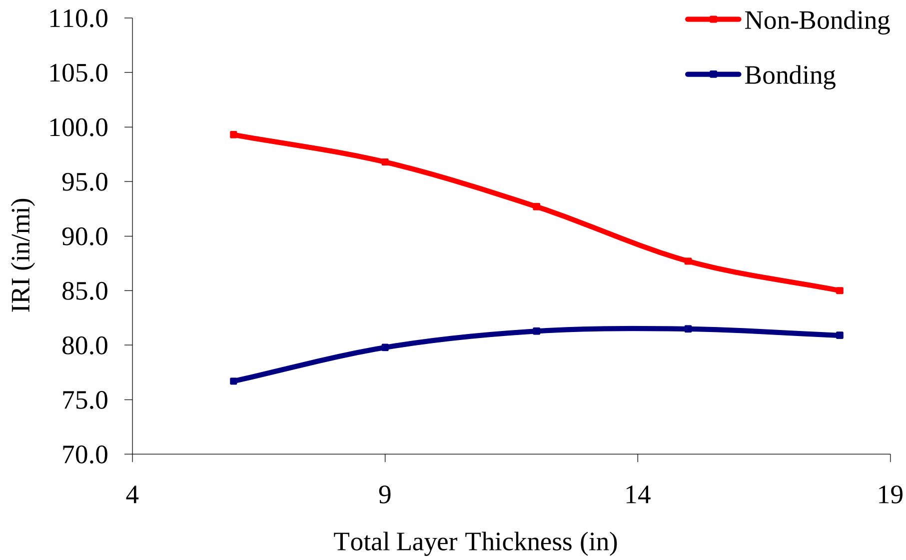


Figure 11. Effects of Bonding on IRI due to changes in Layer Thickness

From Figure 8 to Figure 13 can be observed that the non-bonding environment produced a more critical pavement performance, with higher rutting and IRI values. It should be noted, though, that the diagrams are the same in Figures 8-11, except that the one of the non-bonding conditions is higher than the bonding graph. In Figures 12 and 13 the strains tend to decrease with a increase in the pavement thickness, although, the difference is significantly at the firsts trials.

4. Conclusions

In an overall view, there is a more critical response and performance of the pavement when it is under non-bonding conditions. Although, the significant difference between both complete bonding and non-bonding conditions suggest that if the design is based only in non-bonding interfaces, the design could be over conservative, since the nature of a layer interface is not frictionless either. The development of a model that addresses the incomplete bonding nature of the pavement layers' interfaces will lead to more reliable and safer pavement designs.

Paper References

[1] Huang, Y. H., Pavement Analysis and Design, Pearson Prentice Hall, 2004.

[2] Kim, Sunghwan; Ceylan, Halil; Heitzman, Michael, Sensitivity Study of Design Input Parameters for Two Flexible Pavement Systems Using the Mechanistic- Empirical Pavement Design Guide, Proceedings of the 2005 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2005.