LONG TERM MONITORING AND EVALUATION OF DRAINABLE BASES AT I-90 TEST ROAD

A Dissertation

Presented to

The Graduate Faculty of The University of Akron

In Partial Fulfillment

of the Requirements for the Degree

Doctor of Philosophy

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August, 2009
LONG TERM MONITORING AND EVALUATION OF DRAINABLE BASES AT

I-90 TEST ROAD

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Dissertation

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ABSTRACT

It is a well recognized fact that pavements with poor subsurface drainage properties prematurely exhibit distress and have higher life-cycle cost. Thus, providing adequate subsurface drainage in a pavement system to remove the infiltrated moisture in a timely manner is an important design consideration. Currently, Ohio Department of Transportation (ODOT) accepts several types of materials specifications for use as drainable base materials. They are ODOT 307 (NJ, IA, and CE), ODOT 308 (asphalt treated), and ODOT 306 (cement treated). However, the effectiveness of these base materials in actual service has not been conclusively established in previous ODOT studies. Consequently, a research project, so-called ATB-90, was initiated in 2002 to provide additional data to assess the merits of the drainable bases in the asphalt pavement.

At the present time, most of the current drainage criteria have been developed on the basis of describing water flow in saturated conditions. Yet this could happen in a very limited circumstance and time duration. In this study, a transient analysis using SEEP/W program was conducted to simulate water flow in a flexible pavement system. A new predictive equation along with design charts for estimating the time required to drain 50 percent saturation were developed by utilizing the results of SEEP/W analysis. The predictive equation could be used to design a flexible pavement system for effective subsurface drainage.
A new model was proposed to predict the resilient modulus of subgrade cohesive soils. In this model, octahedral normal and shear stresses as well as soil matric suction were used. To validate the accuracy of the new proposed model, resilient modulus conducted in the laboratory and gathered from the literature were used. Validation process revealed that the proposed model performs well and provides advantages over the other models in which it provides better prediction accuracy as well as reducing the number of soil specimens needed to determine the regression coefficient in the model. Also, empirical models to predict the k-coefficients in the proposed model from basic soil properties were developed.

In the new Mechanistic-Empirical Pavement Design Guide (MEPDG), the effect of environmental factors on the pavement materials characteristics have been full considered through the use of a climatic modeling computer program called the Enhanced Integrated Climatic Mode (EICM). However, designing pavements confidently using the MEPDG requires knowing the accuracy of the EICM compared to field data. In this study, the prediction capability of the new EICM (version 3.2) has been evaluated by comparing the predicted values of the temperature, moisture, and frost depth with those measured at I-90 site over a period of six years.

The impact of base material types on pavement performance has been evaluated using the latest version of the MEPDG software (version 1.0). Cement treated base was seemed to work the best, from providing effective drainage and reducing thickness requirement of asphalt layer.
DEDICATION

This Dissertation is dedicated to

My parents

My wife, and

To my son Adam

With appreciation for support, help and encouragement I received during the years that were devoted for writing this dissertation.
ACKNOWLEDGEMENTS

First and foremost, all thanks to Allah (S.W.T) for the blessing and opportunity for me to finish my dissertation.

During the writing of this thesis, I had the support of many people. Firstly, I would like to take this opportunity to express my sincere and deepest appreciation and gratitude to my advisor, Professor Robert Liang, for his intelligent guidance and constant encouragement he gave me throughout the course of this research project. I am also grateful to my committee members: Dr. Ala Abbas, Dr. Ping Yi, Dr. Xiaosheng Gao, and Dr. Kevin Kreider for their time, patience, and valued feedback.

I owe a deep dept for my parents, who cherished me in childhood, and supported me step by step until I reached this point of success. Even though I have been on the other side of the world for years, I have never felt very far away. They have shown me that when the door to one opportunity closes, another will open in its place. They have taught me to do my best and have supported me in everything I have ever done. Without their love and support I wouldn't be the person that I am today.

Many thanks go to my wife, Nour, for being by my side, and for her encouragement and her patience for being a wife, a mother, a worker, and a student in the same time. I could not have done this without her, and I am so grateful for her presence in my life. I also express my gratitude to Adam, my son, who was born during the last few months of
preparing this dissertation. When he was smiling for me I was given more hope and more energy to be done with the dissertation, and so thank you my little one.

I would like to thank my friends and colleagues for their friendship and support during the time of my studying. I feel so lucky to have met you. You have always been there when I needed you. I wish you all the best with your life.

Furthermore, I would like to thank my in-law family, my brothers, and my sisters, for their love and support. I appreciate what my mother-in-law did for us. The time that she spent caring for Adam made it possible for me to work on some of the most difficult portions of this dissertation. Her help and support were invaluable.
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1.1 Statement of Problem

Sustained long period of water staying within a pavement system is undesirable because the free water can not only adversely affect the load carrying capacity of the pavement but also cause premature pavement failures. According to Huang (1993), accumulated water in the pavement structure can cause one or more of the following forms of deterioration: reduction in soil shear strength, pumping action in rigid pavements, migration of fines into drainable base layers, frost heave and thaw, differential swelling in expansive soils, stripping of asphalt in flexible pavement, and cracking in rigid pavement. These water induced premature pavement failures can cause serious safety hazards to traffic as well as undermine the serviceability of the pavement (Moulton, 1980).

The use of drainable base in a flexible pavement has been a widely adopted practice to ensure that no excessive water remains within pavement layers to cause any early distresses. Currently, Ohio Department of Transportation (ODOT) accepts several types of materials specifications for use as drainable base materials. They are ODOT 307 (NJ, IA, and CE), ODOT 308 (asphalt treated), and ODOT 306 (cement treated). However, the
effectiveness of these base materials in actual service has not been conclusively established in previous ODOT studies.

To evaluate the effectiveness of ODOT drainable base materials in preventing excessive moisture migration and staying in different layers of the flexible pavement, a research project located at I-90 highway, Ashtabula County, Ohio, was initiated. A comprehensive instrumentation, including time-domain reflectometry (TDR) probes, MRC thermistors and resistivity probes, to measure soil water content, pavement temperature profiles and pavement frost depths, respectively, are installed at each section to measure and study the effectiveness of ODOT drainable base materials. In addition, a weather station with the capability of monitoring solar radiation, air temperature, wind speed, wind direction, and rainfall is installed at the site.

The resilient modulus of cohesive soils has widely been adopted in the Mechanistic-Empirical Design Approach (NCHRP 2004) as an important material parameter for design and analysis of both flexible and rigid pavements. Environmental factors, such as moisture, temperature, and freeze-thaw cycles have been found to exert significant impact on pavement material properties and their field performance (Mohammad et al. 1995, Romero and Pamukcu 1996, Fredlund et al. 1977, Yang and Nazarian 2003, Wolfe and Butalia 2004). According to George (2004), the resilient modulus of cohesive soils is a function of state of stresses, soil structure and water content. Due to complexity of conducting resilient modulus testing, numerous efforts have been conducted to develop predictive equations by incorporating state variables such as confining stress, bulk stress, deviator stress, and soil physical properties. However, very little efforts were attempted to predict the resilient modulus with changes in soil’s water content. Therefore, there is a
practical need for a predictive equation for the resilient modulus as a function of stress states and moisture content.

Most of the time the flow of water near the pavement surface occurs under unsaturated (or partially saturated) conditions where the hydraulic conductivity of the soils is not a constant value, but a function of the soil water content (Stormont and Zhou, 2005). Birgisson and Roberson (2000) showed that the performance of conventional drainage system can only be understood if unsaturated flow principles are considered. Nevertheless, most of the current drainage criteria have been developed on the basis of describing water flow in saturated conditions (Ariza and Birgisson, 2001). Unsaturated flow in pavement section is a complex problem since the moisture flow is driven in part by suction gradients, which can result in upward or lateral flow in some cases. Therefore, a finite element method like SEEP/W program can be used as a powerful analysis tool. SEEP/W is a 2-Dimensional, finite element modeling software package for simulating pore-water movement and pore pressure distribution within porous materials, such as soil and rock. SEEP/W simulations can incorporate both saturated and unsaturated seepage processes with material properties specific to the different layers comprising a pavement section.

To study the relative impact of using different base materials on pavement performance, there is a need to have the ability to predict future damages that occur in pavement systems. The development of the Mechanistic-Empirical Pavement Design Guide (MEPDG) under the National Cooperative Highway Research Program (NCHRP) project 1-37A and 1-40D has significantly improved the ability of pavement designers to model and simulate the effects of the combination of pavement materials, traffic and
climate on future pavement performance. Unlike empirical pavement design methods (1993 AASHTO Guide), MEPDG combines both empirical and mechanistic procedures. Mechanistic methods are used to predict pavement responses (stress, strain and displacement), while empirical methods are used to relate the damage over time to pavement distress and smoothness level. Comparing with 1993 AASHTO pavement design, the new MEPDG requires comprehensive material, traffic and climatic inputs to predict pavement performance. However, the MEPDG procedure has built-in adequate flexibility to enable agencies to adopt it in a manner that is commensurate with the resources available to them at any given time. The flexibility is afforded by the three hierarchal input levels, nationally calibrated performance models, and nationally established default input values for several parameters.

Environment is considered as one of the essential factors that influence pavement material properties and their field performance. Consequently, a sound pavement design requires a good ability to estimate or predict the seasonal variations in pavement material properties. In the new MEPDG, the effect of moisture and temperature variations on the pavement materials characteristics have been fully considered through the use of a climatic modeling computer program called the Enhanced Integrated Climatic Mode (EICM). The EICM is a one-dimensional coupled heat and moisture flow model used to simulate the changes in the behavior and characteristics of pavement and subgrade materials in conjunction with climatic conditions over several years of operation. Since climatic conditions vary from region to region, it is difficult to develop standards models to account for seasonal variation in material properties that apply to all regions. Therefore,
the need to develop regional models becomes an essential requirement for most transportation agencies.

1.2 Objectives

The objectives of this research are as follows.

- Evaluate the effectiveness of various types of drainable base materials adopted by ODOT by analyzing the moisture regimes under different permeable base materials over six years of data monitoring.

- Perform a comprehensive parametric study using two-dimensional SEEP/W program to evaluate the effect of different design parameters on sub-surface drainage performance, develop a new predictive equation and design charts to estimate the time required to drain 50 percent of water from the drainage layer, and provide general design guidelines to enhance future sub-surface drainage designs.

- Develop a new predictive model to characterize the resilient modulus of cohesive soils using both octahedral stresses and soil suction concepts.

- Develop empirical relationships between the basic soil physical properties and the regressed k-values of the proposed model.

- Evaluate the impact of using six different drainable base materials on the long-term performance of the as-built pavement sections using the latest version of MEPDG software.

- Perform a comparative study where the influence of different types of permeable base materials and subgrade soils on HMA layer thickness can be evaluated.
- Evaluate the prediction capability of the new EICM (version 3.2) by comparing the predicted values of the temperature, moisture, and frost depth with those measured at I-90 site over a period of six years.

1.3 Dissertation Outlines

Chapter II presents a literature review of the work done by other researchers on the interrelated subjects.

Chapter III presents and analyzes the field moisture monitoring data obtained from I-90 pavement sections.

Chapter IV presents a new predictive equation along with design charts to predict time-to-drain 50 percent of saturation based on unsaturated flow conditions. Also in this chapter, design principles and guidelines for effective subsurface pavement drainage are provided.

Chapter V presents the development of a new predictive equation for cohesive subgrade soils. Also, a comprehensive statistical analysis is conducted to develop empirical correlations between the proposed model K-coefficient and soil physical properties.

The main objective of Chapter VI is to evaluate the accuracy and the capability of the new Enhanced Integrated Climatic Model, EICM (Version 3.2), to predict moisture content, temperature, and frost depth profile within the pavement structures. ATB-90 environmental data will be used for this purpose by comparing predictions with measured data.
Chapter VII introduces the Mechanistic-Empirical Pavement Design Guide (MEPDG) as a powerful tool to evaluate the pavement performance over time of different types of permeable base materials taking into account materials properties, traffic characteristics and environmental conditions. Also, a comparative study to come up with the most effective HMA layer thickness is introduced in this chapter.

Chapter VIII presents summary of the work done, conclusions, and recommendations for future study.
CHAPTER II
LITERATURE REVIEW

2.1 Introduction

Environment is one of the essential factors that influence both flexible and rigid pavement materials behaviour and consequently their performance. Environmental conditions can be classified into external and internal factors (MEPDG, 2004). External factors including precipitation, temperature, freeze-thaw cycles, and depth to water table play a key role in defining the bounds of the impact the environment can have on the pavement performance. Internal factors such as the susceptibility of the pavement materials to moisture and freeze-thaw damage, drainability of the paving layers, infiltration potential of the pavement, and so on define the extent to which the pavement will react to the applied external environmental conditions. Among the external factors, moisture in a pavement structure has widely been recognized by the pavement community as a major factor responsible for premature pavement failure. According to Stormont and Zhou (2001), excessive moisture in pavements can cause one or more of the following forms of deteriorations: stripping of asphalt pavements, joint displacement in concrete pavements, reduction in pavement strength due to positive pore water pressure in the base course layer, shrinkage and swelling of subgrade materials due to water content change, and frost heave and thaw weakening due to upward (capillary)
flow beneath pavements. The first step to preventing these forms of moisture-induced deterioration is to detect the moisture before damage occurs. Electromagnetic techniques, such as Time-Domain Reflectometry (TDR) probes, are very attractive for determining soil moisture content because they exploit the large contrast between the dielectric constant of free water and dry paving materials. The dielectric constant of paving materials ranges from 3 to 8 (depending on density and conductivity) whereas the dielectric constant of free water is given as 81 (depending on frequency and temperature) (Diefenderfer 2002). There are numerous experimental evidences on the impact of increased moisture contents on asphalt layer, base course and subgrade materials. Properties related to pavement performance, such as resilient modulus and deflection under load, have been shown to degrade in the presence of increased moisture. Mark and Baker (1997) stated that if a pavement base is saturated as little as 10 percent of the time, the useful life of the pavement could be reduced by 50 percent.

In order to remove the excess moisture in the pavement, many states adopted the use of permeable base or subbase layers. The permeable base allows the water to flow into discharge pipes. The effectiveness of a permeable base layer is dependent on the required time to drain; the more permeable the layer is, the quicker it drains the excess moisture. Aggregate to be used for drainage layers should consist of sound, clean and open-graded materials. They must have a high permeability to accommodate the free passage of water, and need to protect from clogging by means of filter. Also, they must be able to provide stability against imposed traffic loads.
2.2 Source of Moisture in Pavement Systems

Free water enters the pavement system from many sources. Cedergren et al. (1973) stated the following as important sources of the water in the pavement system: atmospheric precipitation, snow, hail, condensing mist, dew, and melting ice. This water reaches the structural section in several ways:

- Cracks in the pavement
- Infiltration through the shoulders
- Infiltration from the side ditches
- Melting of an ice layer from a frost area during the thawing cycle
- Free water from pavement base. If the base is not properly drained, it may act as a source of free water for the subbase and subgrade
- High groundwater table
- Condensation of water vapor (small amounts).

The first five sources mentioned above can be particularly important if the surface drainage is not properly designed or maintained (Cedergren et al., 1973). A study conducted by the Federal Highway Administration (FHWA) showed that 33 to 50 percent of the precipitation over asphalt concrete sections, and 50 to 67 percent of precipitation over Portland cement section could infiltrate through the pavement to the Base (Mark and Baker, 1997).

2.3 Field Measurement of Soil Moisture

The moisture content of soil is required for many important design considerations, such as resilient modulus, freeze-thaw capacity and settlement. Based upon previous
studies, Time-Domain Reflectometry (TDR) probes were chosen as the best instrument available to monitor volumetric water content. It has been widely used by several agencies, especially in the nationwide Long-Term Pavement Performance Program (LTPP). The TDR method of monitoring subgrade water was introduced to pavement engineering around 1989 (Neiber and Baker 1989). The large contrast between the dielectric constant of free water and dry soil materials potentially makes TDR an effective non-destructive technique to monitor unsaturated soils and material moisture. The dielectric constant of water is approximately 80, that of dry soil is between 3 and 8 (depending on soil type and density), and that of wet soil is between 3 and 60 (depending on the moisture content) (Jiang and Tayabji, 1999).

A TDR measurement system typically includes a transmission line (wave guide or probe), a coaxial connecting cable, and a TDR instrument that generates fast-rise-time voltage pulses and measures the time required for a pulse to travel between two points on the transmission line. The voltage pulse propagates along the transmission line through the dielectric medium surrounding it. The transmission line is embedded in the soil, and the impedance changes at the beginning of the transmission line and the end of the transmission line cause a reflected pulse. The velocity of the pulse through the medium is influenced by the dielectric constant ($K_a$) of the materials, and is given by (Topp, et al., 1980):

$$v = \frac{c}{\sqrt{K_a}}$$  \hspace{1cm} (2-1)
where \( v \) is the velocity of the electromagnetic wave travelling down the TDR (m/s); \( c \) is the velocity of electromagnetic wave in free space \( (3.998 \times 10^8 \text{ m/s}) \); and \( K_a \) is the apparent dielectric constant of the surrounding materials (unitless).

The travel time of the wave travelling down and back in a cable of length \( L \), is given by:

\[
t = \frac{2L}{V}
\]

(2-2)

where \( L \) is the waveguide length and is defined as the actual length of the TDR probe (m), and \( t \) is the measured travel time in seconds.

The dielectric constant is also affected by the moisture changes in the materials. When the pulse reaches the end of the probe, a portion of the signal is reflected back through the shielding of the coaxial cable and will be measured by the cable tester unit, such as (Tektronix or TDR100). In general, as the water content increases, the travel time of the applied pulse increases. Combining Equations 2-1 and 2-2 yields:

\[
K_a = \left(\frac{t_c}{2L}\right)^2
\]

(2-3)

The travel time of the signal is also dependent on the dialectic constant, which include signal propagation in the soil-moisture mixture; hence, the apparent probe length can be determined by the travel time of the signal if it were propagating at the speed of light as:

\[
L_a = \frac{tc}{2}
\]

(2-4)

where \( L_a \) is the apparent probe length (m); and \( t \) is the measured travel time (down and back) in seconds.
Therefore, the dielectric constant of the soil can be expressed as the ratio of the apparent length to actual length of TDR probes from Equations 2-3 and 2-4:

$$K_a = \left(\frac{L_a}{L}\right)^2$$  \hspace{1cm} (2-5)

The apparent length ($L_a$) of a TDR wave should be determined in order to measure the apparent dielectric ($K_a$) using Equation 2-5. The $L_a$ is defined as the distance between the first and second inflection points on the TDR wave form, as shown in Figure 2.1. This $L_a$ represents the reflected wave distance at the beginning and end of the TDR probe. Then $L_a$ was compared to the actual length of the probe ($L$) to get the apparent dielectric conductivity ($K_a$) of a material.

Figure 2.1  The location of the first and second inflection points on a typical TDR wave signal, captured by TDR100 device
Many studies had investigated the relationship between the dielectric constant and the water content. The methods currently in use to determine the volumetric moisture contents (VMC) form dielectric constants are mainly based on empirical approaches:

a) Topp’s equation

Topp’s equation employs empirical regression functions to relate the dielectric constant to the VMC (Topp et al., 1980). The third-order polynomial function developed by Topp is widely used for calculating moisture content of soil materials. Topp’s equation was found to best fit the crushed concrete materials (Henrik et al., 1995 and Van der Aa et al., 1997)

\[ \theta = -5.3 + 2.923K_a - 0.055K_a^2 + 0.00043K_a^3 \]  

(2-6)

where \( \theta \) is volumetric moisture content (%).

b) Ledieu equation

Many researchers adopted Ledieu equation as the best fit for granular sand base materials (Apul et al., 2002, Roberson et al., 2001, and Birgisson et al., 2000). Ledieu equation is given as follows:

\[ \theta = 0.1138\sqrt{K_a} - 0.1758 \]  

(2-7)

c) Siddiqui and Drnevich equation

Siddiqui-Drnevich (1995) calibration equation accounts for both soil density and type. This equation was adopted in the new method for water content and in-situ density determination known as Purdue Mehtod (Drnevich et al. 2002).
Siddiqui - Drnevich equation is given as follows:

\[ \theta_g = \frac{1}{b} \left[ \frac{\rho_w}{\rho_d} \sqrt{k_a} - a \right] \]  

(2-8)

where \( \theta_g \) is the gravimetric water content; \( \rho_d \) is the dry density (g/cm\(^3\)); \( \rho_w \) is the density of water which is approximately equal to 1.0 (g/cm\(^3\)); \( b \) and \( a \) are soil-dependent calibration constants.

2.4 Environmental Field Monitoring Studies

It has long been recognized that environmental changes are the major factor in pavement design. The effect of seasonal variation on pavement performance is generally considered to be very important. Therefore, several test sites across the United State have been instrumented to evaluate the effect of environmental factors, such as moisture, temperature and frost depth on pavement performances. A brief description of previous studies on environmental seasonal variations is presented in the following sections.

2.4.1 Ohio Test Sites

US Highway 23 in Delaware County, Ohio, was equipped in 18 locations to monitor climatic effect on pavement performance. The southbound lanes of the roadway are constructed of asphalt concrete (AC) pavement and the northbound lanes are constructed of Portland cement concrete (PCC) pavements. The subgrade materials at the site classified as A-4, A-6 and A-7-6 under the AASHTO soil classification system. Seasonal monitoring on the Ohio SHRP Test Road consisted of the periodic measurement of
temperature, moisture, and frost depth to a depth of six feet below the pavement surface. Pavement temperatures on the test road were monitored with thermistors, or temperature sensitive resistors which consist of individual, but interconnected probes for both pavement and soil temperature measurements. Pavement moisture contents were monitored by time-domain reflectometry probes (TDR). TDR probes consist of a coaxial cable leading to three pronged probe installed in the subgrade. TDR performance has been extremely satisfactory throughout the duration of the project (Sargand et al. 2007). Sargand et al. (2007) reported that the maximum moisture contents were occurred in the months of July and August, while the minimum moisture contents were occurred in the months of January and February. The depth of frost penetration in the subgrade and the number of Freeze/thaw cycles were monitored using probes developed by the U.S. Army Corps of Engineer’s CRREL. This probe consisted of a 73-inch long solid PVC pipe upon which 36 metal wire electrodes were spaced every two inches. No useful data could be obtained from the resistivity probes. Therefore, Sargand et al. (2007) made a recommendation to redesign of the probes and/or electronic interface with the data logger.

Piezometers were installed in some of the test sections with seasonal instrumentation to measure the depth to the water table. A total of nine piezometers were installed at the sites. The highest ground water tables occurred from April to June and the lowest ground water tables occurred from October to November (Sargand et al. 2007). To assist in monitoring climatic changes along the test road, a weather station was installed to monitor solar radiation, air temperature, wind speed, wind direction, relative humidity, and rainfall. The weather station data has been monitored and collected By Dr. J. Luwig Figueroa form case Western Reserve university three to four times each year since 1994.
Heydinger (2003) found that sinusoidal relationships could be used to express the seasonal variations of soil temperature and moisture as a function of time, where time is expressed as day of the years (based on five years of data). According to Heydinger (2003), seasonal changes in moisture content occur at a site with a fine-grained subgrade soil with no base drainage and a high water table.

Figueroa (2004) found that the degree of saturation vs. depth among sections with and without drains did not show any significant difference in reducing the degree of saturation in sections with drains. A related comparison between increased precipitation and degree of saturation indicated a lag of 80 to 85 days in increased degree of saturation after substantial precipitation occurs. Generally, the subgrade degree of saturation ranges from 85 to 100 percent. The highest saturation was found in the summer and the lowest in the winter. Both spring and fall periods showed intermediate saturation values.

Figueroa and Sargand (2006) studied the effects of pavement design features on the volumetric moisture content (VMC) of subgrade soils through the analysis of Time Domain Reflectometry (TDR) data obtained at the Ohio SHRP Test Road. It was found that pavement type (HMA or PCC) did not have any significant effect on the VMC of the subgrade. Also, it was also found that the VMC did not always increase with depth, instead the average VMC for each sensor were found to oscillate within a very narrow range with depth. All subgrade TDRs showed a characteristic annual sinusoidal variation in VMC, reaching a peak during the month of August and a valley during the month of February. This is indicative of a lag that occurs between the normally high precipitation in the spring and the high subgrade moisture that occurs in August (Figueroa and Sargand 2006). VMC time series was fitted with a sinusoidal function by regression of data for
each sensor, including VMC as the dependent variable and the day of the year (in the Julian calendar) as the independent variable.

Sargand et al. (2008) developed a comprehensive instrumentation plans to monitor both the effect of environmental conditions in pavement structures and their response when subjected to dynamic loading at WAY-30 test pavements. The environmental instrumentations include: Time-domain reflectometry (TDR) for base and subgrade moisture measurements, MRC thermistor probe for pavement, base and subgrade temperature measurements, CRREL (Cold Region Research and Engineering Laboratory) resistivity probe for base and subgrade frost depth measurements, and Piezometers for groundwater table monitoring. An automated weather station also monitors air temperature, precipitation (rain and snow), wind speed and direction, relative humidity, and incoming solar radiation, all on an hourly basis. It was found that the volumetric moisture content (VMC) beneath the pavement, as measured at all depths within the subgrade, tends to increase and stabilize to values above the initial reading, in some instances to values approaching saturation. It was found that the subgrade moisture content is stabilized after about 6 months. Only one year of data had been analyzed, so annual or seasonal recurring patterns have not been determined.

2.4.2 Tennessee Test Sites

A comprehensive instrumentation system was installed at four sites across the State of Tennessee to monitor seasonal variations for factors affecting flexible pavement response. The four instrumented sites referred to as the Blount County Site, Overton County Site, Summer County Site and McNairy County Site. The instrumentation
included a weather station at each of the test sites, time-domain reflectometry probes (TDR), temperature sensors, free-drainage lysimeters, and resistivity probes (Rainwater et al. 1999). Weather stations were equipped with the capability of monitoring air temperature, precipitation, relative humidity, wind speed, and solar radiations. TDR probes and temperature sensors were installed horizontally beneath the outer wheel path of the roadway from a trench constructed in the shoulder. Automated data collection and remote data access were designed into the monitoring systems to provide comprehensive data without requiring frequent travel and extended hours at each site. In addition to the automated instrumentation, nondestructive falling weight deflectometer (FWD) tests were regularly conducted at the sites to correlate the pavement response with the measured environmental condition (Rainwater et al. 1999).

Bulk soil samples and undisturbed tube samples were collected to determine various soil properties, including bulk density, in-situ water content, resilient modulus, and water retention curves (Soil Water Characteristic Curves, SWCC). The subgrade materials at the sites classified as A-4, A-7-5 and A-7-6 under the AASHTO Soil Classification system. Very small changes in subgrade water content were detected at Tennessee sites. According to Rainwater et al. (1999), this was related to the newly condition of the all instrumented pavement sections. However, Rainwater et al. (1999) anticipated that pavement weathering and additional loading will increase the seasonal moisture changes in the subgrade materials.
2.4.3 Iowa Test Sites

A newly constructed section along US Highway 20 near Fort Dodge, Iowa, was equipped to monitor climatic effect on pavement performance constructed of Portland cement concrete during the period of May 2005 to April 2008. The in-ground instrumentations measured variations in temperature, frost depth, groundwater levels, and moisture regime below the pavement surface (Thang et al. 2008). A limited weather station was installed at the data logger location to measure air temperature and precipitation. The subgrade was generally characterized as being very stiff, well compacted, and relatively uniform. The average dry density and moisture content of the subgrade layer obtained from in-situ testing were 116.5pcf and 10.2%, respectively. The average resilient modulus of the subgrade layer was 119 MPa. Vibrating wire (VW) piezometers were installed at two offset locations a distance of approximately 10 feet on both sides (east and west) of the instrumentation trench. The ground water table under pavement surface fluctuated from 9 to 13 feet.

Thang et al. (2008) found that moisture content in the subbase layer remained relatively constant throughout the year except during freezing periods. The subgrade moisture content was lower in the winter season compared to summer. Moisture contents of the subgrade layer increased deeper into the layer and were affected by seasonal variations. In addition, they found that in the subgrade layer, freezing penetrated downward, but thawing occurred in both downward and upward directions.
2.4.4 New Jersey Test Sites

New Jersey Department of Transportation (NJDOT) initiated a research study to calibrate the AASHTO temperature and seasonal adjustment models or to develop new models (Zaghloul et al. 2006). Accordingly, twenty-four flexible, rigid, and composite pavement sections throughout the state of New Jersey were instrumented to continuously monitoring temperature, frost-thaw, moisture, ground water table and environmental changes (air temperature and rainfall). In addition, Deflection and seismic testing were conducted on a monthly basis, except during the spring-thaw period that is on a bi-monthly basis, for a period of two years. In addition, samples recovered in the field were taken for a detailed laboratory evaluation. A comprehensive instrumentation, including time-domain reflectometry (TDR) probes, MRC thermistors and CRREL (Cold Region Research and Engineering Laboratory) resistivity probes, to measure soil water content, pavement temperature profiles and pavement frost depths, respectively, are installed at each section. The depth of the ground water table was measured through observation piezometers. The piezometers were placed in the shoulder, a few feet outside the pavement. Air temperature and rainfall were measured using an air temperature probe and a tipping-bucket rain gauge on a pole next to the equipment cabinet.

The environmental data collected from the instrumented sections were analysed to produce general trends for New Jersey (Zaghloul et al. 2006). Accordingly, the following conclusions were made:

- The effect of rainfall on the base, subbase, and subgrade moisture contents was observed over the course of an 8-hour period after the initial start of a rain event. It was observed that the moisture content would initially increase as the rainwater
entered the pavement, and then decrease with time, as rainwater moved through the pavement layers.

- Models relating air, pavement surface, and mid-depth pavement temperatures were developed for thin and thick pavements for different periods of the day. These models can be used for predicting the mid-depth temperatures, and for temperature correction of FWD data.

- The measured frost penetration depths were compared to the empirical frost penetration depths developed by the Corps of Engineers. This relationship between the empirical and measured data showed a weak relationship. This was expected since the empirical curve does not accurately represent New Jersey’s environment or pavement conditions.

2.5 Drainage Impact on Pavement System Performance

Pavement surface/subsurface drainage has long been recognized as an important factor in pavement design. Effective surface water drainage is essential to maintaining a desirable level of service and traffic safety. Hydroplaning is the main problem associated with the poor surface drainage. Hydroplaning results when vehicle tires move fast relative to the wet pavement surface, such that there is insufficient time to channel the moisture away from the center of the tire. The result is that the tire is lifted by the water away from the road and all traction is lost. In addition to surface drainage, pavement must be designed to allow adequate subsurface drainage. The presence of free water in any pavement structure, whether flexible or rigid, has detrimental effects on the pavement performance. Cedergren (1988) reported that water is responsible for a large number of
non-load related distresses such as D-cracking in concrete pavement, and accelerated aging and oxidation in asphalt pavements. Cedergren (1988) also reported that pavement life can be extended up to three times if adequate subsurface drainage systems are installed and maintained. Forsyth et al. (1987) presented a number of case studies related to pavement drainage. They reported that the use of edge drains usually improve the durability of pavements. Forsyth et al. concluded that the percentage of cracked slabs in the undrained sections exceeds that in the drained sections by a ratio of 2.4 to 1.0. Forsyth et al. (1987) also reported at least 33 percent increase in service life for asphalt pavements and a 50 percent increase for PCC pavements. Ray and Chriistory (1989) observed premature pavement distresses in an undrained pavement section in France, inferring a reduction in service life of nearly 70 percent as compared with a drained section. On the other hand, somewhat controversial and unexpected results were concluded from NCHRP Project 1-34. The overall conclusion was that incorporating subsurface pavement drainage systems into pavements did not necessarily result in improved pavement performance. The results suggested that neither permeable bases nor edge drains were found to significantly reduce faulting of jointed concrete pavement. Further, daylighted permeable bases were found to improve the performance of flexible pavements compared to those with edge drains. These unexpected tentative results are being further evaluated by follow-up studies (NCHRP Project 1-34C and NCHRP Project 1-34D).

Under NCHRP Project 1-34C, “Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements,” which was completed in 2003, data from the LTPP SPS-1 and SPS-2 experiments were analysed. For Hot Mix Asphalt (HMA), measures of
cracking and International Roughness Index (IRI) were best when undrained dense-graded asphalt-treated base (DGA/ATB) were present compared to either undrained dense-graded aggregate bases or drained permeable asphalt-treated bases (Hall and Correa, 2003). For Portland cement concrete (PCC) pavements, measures of cracking and IRI were best when drained permeable asphalt-treated bases were present compared to either undrained dense-graded aggregate bases or undrained lean concrete bases.

Under NCHRP Project 1-34D, “Effect of Subsurface Drainage on Pavement Performance,” which was completed in 2006, data form LTPP SPS-1 and SPS-2 experiments, data form field inspection and flow testing of the pavement drainage systems, and deflection data from Release 19 of the LTPP data base were all analysed. The over all conclusion was that the best performing HMA pavements is SPS-1 experiment were those with the stiffest bases (incorporating a dense-graded asphalt-treated base layer), whether drained or undrained (Hall and Crovetti, 2007). For PCC pavements in SPS-2 experiments, the best performing were those with bases that were neither too weak nor too stiff.

2.6 Pavement Drainage Systems

Drainage systems, such as permeable aggregate bases and edge drains, are becoming an essential part of construction and reconstruction plans. The permeable aggregate base must consist of sound, clean and open graded materials. In addition, it must have a high permeability to allow the free passage of water. Agencies have different practices regarding the drainage layer. For the most part, Iowa, Kentucky, Michigan, Minnesota, New Jersey, Pennsylvania, and Wisconsin use untreated base. Indiana, California, North
Carolina, and West Virginia use treated base (Hossam and Thomas, 1996). An untreated base uses smaller aggregates than a treated base to provide stability. The treated base uses a stabilizer (asphalt or cement) to provide stability, which results in materials that are more open. Thus, untreated bases generally have a lower coefficient of permeability than treated bases. Although there is no consensus regarding the most desirable degree of permeability, agencies agree that rapid base drainage is extremely important. Therefore, a minimum coefficient of permeability of 300 m/day is suggested (Cedergren, 1974).

2.6.1 Open Graded Permeable Bases

Open-graded materials usually contain less sand and fines than non-open graded bases and subbases, and more fine than stabilized permeable bases. These open-graded materials provide better resistance to the effects of moisture than dense-graded materials with high fines contents. However, stability of these untreated permeable base layers is a major concern because settlement can lead to serious problems and needs to be addressed adequately. Therefore, open graded materials should have a gradation that is a trade-off of constructability/stability and permeability. AASHTO (1993) showed that the base density is maximized at fines content between 6 and 20 percent and load-carrying capacity and permeability decreases when the fines content exceeds about 9 percent. Baumgardner (1993) recommended that open graded permeable base be composed of 100 percent crushed stone and supposed that open graded permeable base has a coefficient of permeability on the order of 0.35 to 1.05 cm/s (1000 to 3000 ft/day). To meet the stability requirement, AASHTO (1993) specified the coefficient of uniformity as an indicator. Salter (1997) surveyed six states that use open-graded base course material for their
permeable bases. He observed that the six states surveyed experienced a significant increase in life expectancy and a decrease in maintenance costs for roadway sections with open-graded permeable bases.

2.6.2 Stabilized Permeable Bases

A stabilized permeable base course is intended to quickly remove water that infiltrates the pavement to an edge drain system, thus reducing the chance of the infiltrated water to reach the lower pavement layers and reducing their strength. Generally, stabilized permeable bases are more open-graded and more permeable than unstabilized materials. Stability is provided by the cementing action of the stabilizer material. The AASHTO No. 57 and No. 67 gradations shown in Table 2.1 are recommended by many highway pavement agencies for use with stabilized permeable bases.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>AASHTO Stabilized Permeable Bases Gradations</th>
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<tbody>
<tr>
<td></td>
<td>AASHTO Stabilized Permeable Bases Gradations</td>
</tr>
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<td>inch</td>
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<td>1.5</td>
</tr>
<tr>
<td>25</td>
<td>1</td>
</tr>
<tr>
<td>19</td>
<td>¾</td>
</tr>
<tr>
<td>12.5</td>
<td>½</td>
</tr>
<tr>
<td>9.5</td>
<td>3/8</td>
</tr>
<tr>
<td>4.75</td>
<td>No. 4</td>
</tr>
<tr>
<td>2.38</td>
<td>No. 8</td>
</tr>
</tbody>
</table>

2.6.2.1 Portland Cement Stabilized Permeable Bases

Generally, 1.5 to 2.5 bags of cement per cubic meter or 2 to 5 percent by weight are used in Portland cement stabilized permeable base. Surface adhesion forces develop
between the paste and the soil particles causing the soil particles to bind together. Portland cement base materials have considerable initial strength (FHWA, 1992). However, factors such as freeze-thaw cycles could affect the long-term strength of these materials by breaking down the cementation bonds between the paste and the soil particles (Baumgardner, 1993). According to Forsyth et al. (1987), an increase of 4 years in pavement service life may occur due to cement-treaded permeable bases incorporation. The estimated savings in pavement cost due to this extension in service life is approximately 21 percent.

2.6.2.2 Asphalt Cement Stabilized Permeable Bases

Asphalt cement stabilized permeable base is generally mixed in a batch plant according to the design requirements. In general, 2 to 3 percent by weight are used in the asphalt cement permeable base. The FHWA (1992) recommends the use of harder grade of asphalt cement (AC 40 or AR 8000) to improve the stability of the base during construction. Adequate asphalt cement film thickness around the aggregates is required to ensure the long-term durability by increasing the stripping resistance. Forsyth et al. (1987) reported that the extension of the service life of asphalt cement stabilized permeable base would reduce pavement cost by approximately 21 percent not including user and maintenance cost.

2.6.3 Edge Drains

Edge drains consist of longitudinal pipes that run alongside the pavement. They are placed 2-in from the bottom of a trench dug on the side of the pavement adjacent to the
lane-shoulder joint. Two types of edge drains are commonly available: pipe edge drain and prefabricated geocomposite edge drains (PGEDs). Edge drain pipes should have a minimum diameter of 4 inches for maintenance purposes and should have the necessary hydraulic capacity to handle water being discharged from the permeable base.

2.7 Pavement Drainage Criteria

Based on saturated flow conditions, there are two different approaches that are typically considered for the hydraulic design of pavement systems: the steady-state flow conditions and the time-to-drain conditions. However, difficulties in estimating the design rainfall rate and the portion of rainfall that enters the pavements make the application of steady-state analyses tedious under most circumstances. Therefore, many engineers nowadays prefer the time-to-drain approach. This approach is based on flow entering the pavement until the aggregate base course is saturated. Excess runoff will not enter the pavement section after it is saturated; this flow will simply run off on the pavement surface. After the precipitation event, the base will drain to a drainage system. Table 2.2 presents the different drainage levels for a pavement structure, according to AASHTO (1998), for 50% of drainage. This approach drains 50 percent of the water that can be drained. It does not consider the water retained by the effective porosity quality of the material (AASHTO, 1998).
Table 2.2 AASHTO drainage recommendations for Time-to-Drain 50 percent of saturation

<table>
<thead>
<tr>
<th>Quality of Drainage</th>
<th>Time to Drain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>2 Hours</td>
</tr>
<tr>
<td>Good</td>
<td>1 Day</td>
</tr>
<tr>
<td>Fair</td>
<td>7 Days</td>
</tr>
<tr>
<td>Poor</td>
<td>1 Month</td>
</tr>
<tr>
<td>Very Poor</td>
<td>Does Not Drain</td>
</tr>
</tbody>
</table>

For heavily trafficked roads, a time to drain 50 percent of the drainable water in 1 hour is recommended as a criterion; whereas for most other Interstate highways and freeways, a time to drain 50 percent of the drainable water in 2 hours is recommended (AASHTO, 1998).

Two methods to calculate the time-to-drain in the drainage layer of a pavement system are in use: Barber and Sawyer method (1952) and Casagrande and Shannon method (1952). The Barber and Sawyer (1952) equation is given below.

\[ t = T \times \left[ \frac{n_e L_R^2}{kH} \right] \]  

where \( t \) is time for drainage to reach a percentage of drain, \( U \); \( n_e \) is the effective porosity of the drainage layer; \( L_R \) is the resultant length of the drainage path; \( k \) is the permeability of the drainage layer; \( H \) is the thickness of the drainage layer; \( T \) is the time factor, calculated as:

For \( 0.5 \leq U \leq 1.0 \)

\[ T = 0.5S_R - 0.48S_R^2 \log(1 + 2.4/S_R) + 1.15S_R \log \left[ \frac{S_R - US_R + 1.2}{(1-U)(S_R + 2.4)} \right] \]
For $0 \leq U \leq 0.5$

$$T = US_R - 0.48S_R^2 \log(1 + 4.8U/S_R)$$  \hspace{1cm} (2-11)

Where $U$ is the percent drainage (expressed as a fraction); $S_R$ is the resultant slope of the drainage path.

Casagrande and Shannon method (1952) uses the same equation in the above, but the time factor ($T$) is determined based on the following equations:

For $U > 0.5$

$$T = \left(1.2 - 0.4/S_R^{1/3}\right) \left( S_R^2 - S_R \ln\left(S_R + 1\right) + S_R \ln\left(2S_R - 2US_R + 1\right) \right)$$  \hspace{1cm} (2-12)

For $U > 0.5$

$$T = \left(1.2 - 0.4/S_R^{1/3}\right) \left( 2US_R - S_R^2 \ln\left(S_R + 2U/S_R\right) \right)$$  \hspace{1cm} (2-13)

2.8 Saturated/Unsaturated Flow through Pavements

A good understanding of the flow of water in the subsurface of the pavement is an essential factor in designing a good pavement drainage system. There are two different types of fluid flow, saturated and unsaturated. In the saturated flow all the soil voids are filled with water, therefore the volumetric water content is equal to the porosity. Under saturated flow conditions, the hydraulic conductivity ($K$) is considered as a constant value. The driving forces that are required for saturated flow are gravitational and pressure-potential gradients (Tindall and Kunkel, 1999). In reality, saturated flow conditions could happen in very limited circumstances and time duration. Most of the
time the flow of water near the pavement surface occurs under unsaturated (or partially saturated) conditions where the hydraulic conductivity of the soils is not a constant value, but a function of the soil water content (Stormont and Zhou, 2005). As one would expect, the hydraulic conductivity of soils increases with increasing the volumetric water content (Freeze and Cherry, 1979). In general, coarser-grained soils are less conductive than finer-grained soils under most unsaturated conditions because the larger pores that are responsible for the large saturated conductivities will drain readily under small values of suction and are not available for flow under unsaturated conditions. Current studies suggest that the performance of conventional drainage systems can only be understood if unsaturated flow principles are considered (Birgisson and Roberson, 2000). Two material properties are needed to describe the drainage behaviour of soils under any given saturation level, namely the soil water characteristic curves (SWCC) and the permeability function.

2.8.1 Soil Water Characteristic Curve (SWCC)

The relationship between water content and the associated matric suction can be empirically described by the SWCC, which is defined as the water storage capacity of a soil at a given soil suction. Figure 2.1 shows a typical soil water characteristic curve, displaying the relationship between volumetric water content and soil suction. The air-entry value of the soil is the matric suction at which air starts entering the largest pores in the soil. It can be determined by drawing a tangent line from the first inflection point. The curve in the low-suction range can be approximated by another line.
The air-entry value can be approximated as the ordinate of the point at which the two lines intersect, as illustrated in Figure 2.2.

Residual water content is the water content of the soil when a large amount of suction pressure is required to remove the additional water from the initially saturated soil. A consistent way to define the residual water content is also shown in Figure 2.2. The shape of the soil-water characteristic curve is highly dependent on the material type. Fine-grained soils (such as clays) generally have higher matric suction than coarse-grained soils, while loose clays can undergo large volumetric changes as a result of changes in suction (Heath et al. 2004).

![Figure 2.2 Typical Soil Water Characteristic Curve (Fredlund and Xing, 1994)](image)

Since getting the volumetric water content function directly is a tedious work, it may be of benefit to be able to get an estimation of the volumetric water content function
using either a closed-form solution that requires user-specified curve-fitting parameters, or to use a predictive method that uses a measured grain-size distribution curves. Mathematical equations to approximate the soil-water characteristic curve have been proposed by several researchers (Gardener 1958; Van Genuchten, 1980; Arya and Paris, 1981; Fredlund and Xing, 1994; Aubertin et al, 2003). The most popular equations are those of Gardner (1958), as shown in Equation 2-14 and Fredlund and Xing (1994) given by Equation 2-15, and 2-16.

\[
\theta_w = \theta_r + \frac{\theta_s + \theta_r}{1 + \left(\frac{h}{a}\right)^b} \tag{2-14}
\]

\[
\theta_w = C(h) \times \left[ \frac{\theta_s}{\ln\left[\exp(1) + \left(\frac{h}{a}\right)^b\right]} - 1 \right] \tag{2-15}
\]

\[
C(h) = 1 - \left[ \frac{\ln\left(1 + \frac{h}{h_r}\right)}{\ln\left(1 + \left(\frac{10^b}{h_r}\right)\right)} \right] \tag{2-16}
\]

where \(\theta_w\) = volumetric water content; \(\theta_s\) = saturated volumetric water content; \(\theta_r\) = residual volumetric water content; \(h\) = soil matric suction; \(h_r\) = soil parameter, function of the suction at which residual water content occurs; \(a\) = soil parameter, function of the air entry value of the soil; \(b\) = soil parameter, function of the rate of water extraction from the soil; \(c\) = soil parameter, function of the residual water content; \(C(h)\) = adjustment factor which forces all curves through a suction of 1,000,000 kPa at zero water content.
A more recent version of the EICM allows the users to select the Gardner or Fredlund and Xing equations. The most recent version of the EICM, Version 3.0, uses the Fredlund and Xing equation and empirical equations determined by Zapata, et al. (2002) to approximate the parameters required for the equation.

2.8.2 Permeability Functions

The permeability function defines the relationship between unsaturated soil permeability and matric suction. Fluid flow in soil occurs when there are differences in total head. The rate of flow is depended on the total head gradient and the magnitude of the resistance to flow. The resistance to flow increases as the soil matric suction increases. The difficult task of measuring the unsaturated hydraulic conductivity function directly is often overcome by predicting the unsaturated hydraulic conductivity from either a measured or predicted volumetric water content function. Several mathematical equations have been proposed to approximate the unsaturated hydraulic conductivity function (Gardener 1958; Fredlund et al, 1994; Green and Corey, 1971; Van Genuchten, 1980). The most recent version of the EICM uses the Fredlund, et al (1994) equation. The equation used for the permeability function approximates a single drying curve so that hysteresis effects are not accounted for. Soil permeability is very sensitive to sample disturbance, type of test and conditions that occur during testing. Therefore, attempts to accurately define a permeability function from a permeability test would not necessarily improve the accuracy of the modeling predictions.
2.9 Finite Element Analysis for Pavement Drainage

The current drainage criteria used by the FHWA and AASHTO (1998) are all produced based on the assumption of saturated conditions. However, most pavements stay unsaturated most of the time and it is rare to have fully saturated conditions in pavements. Therefore, it is not defensible to consider fully saturated conditions for study pavement infiltration. Recent studies suggest that the performance of conventional drainage systems can only be fully understood if unsaturated flow principles are considered (Birgisson and Roberson, 2000). Because of the complexity of the two-dimensional unsaturated flow, numerical simulation can be a useful and powerful analysis tool. Finite element modeling (FEM) can incorporate both unsaturated and saturated processes, and can include properties specific to the layers that comprise a pavement section. Material properties and pavement layer configurations can be varied in FEM simulations to allow for parametric studies without the need for expensive and time-consuming experimentation.

A program for simulating pavement drainage should be capable of handling transient, two-dimensional, saturated/unsaturated flow. In addition, the FE program should provide relatively sophisticated models for near-surface processes. Climatic data should be easily input into the FE program. FE program should have the ability to utilize a number of different functions for describing the moisture characteristic curve and unsaturated hydraulic conductivity functions. More recently FEM have been used to compare the effectiveness of pavement drainage design, taking into account the unsaturated conditions present in the base and subbase layers (Stormont and Zhou, 2001; Hassan and White, 1998; Lebeau, et al. 1998). SEEP/W program (GEO-SLOPE, 2004) is two-dimensional
finite element software currently in use to model the moisture movement and pore-water
pressure distribution within porous materials such as soil and rock. It includes three
executable programs: DEFINE (for defining the model), SOLVE (for computing the
solution), and CONTOUR (for graphic viewing the results). Ariza and Birgisson (2001)
provide justification for the use of SEEP/W to model moisture movement in pavement
systems. Input parameters such as the soil water characteristic curve, soil hydraulic
conductivity function, precipitation, and depth to water table, are used to estimate flow
patterns below the pavement surface. Mahboub, et al. (2003) used SEEP/W software to
evaluate the movement of water in a Broken and Seated Joint Reinforced Concrete
Pavement (B&S JRCP). A steady-state saturated flow analysis was used to obtain the
flow path of the infiltrated water and flux quantity through the cross-sectional area in the
pavement. Various pavement drainage scenarios were modeled successfully using
SEEP/W finite element modeling technique.

SEEP/W assumes that flow in unsaturated soil above the water table follows Darcy 's
law in a similar manner to flow in saturated soil. The governing differential equation,
Richards’ equation, for 2-dimensional saturated/unsaturated flow is used by SEEP/W.

\[
\frac{\partial}{\partial x} \left( k_x \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial H}{\partial y} \right) + Q = \frac{\partial \Theta}{\partial t} \tag{2-17}
\]

where \( h \) is the total head; \( k_x \) is the hydraulic conductivity in x-direction; \( k_y \) is the
hydraulic conductivity in y-direction; \( Q \) is the applied boundary flux; \( \Theta \) is the volumetric
water content; and \( t \) is the time.

36
Rechiard’s equation states that the difference between the flow entering and leaving an element volume at a point in time is equal to the change in the volumetric water content. Under a steady state condition, the right part of Richards’ equation would become zero.

2.10 Cohesive Soils Resilient Modulus

It is generally recognized that the performance of a pavement is strongly affected by the characteristics of the subgrade soils. Resilient modulus (M_r) of subgrade soils in a pavement structure is one of the most important input parameters for the design of new and the rehabilitated flexible pavements. Seed, et al. (1962) suggested the definition of M_r using the relationship between applied stress, referred to as deviator stress (σ_d) and recoverable strain, referred to as resilient strain (ε_r) measured in repeated triaxial dynamic tests on compacted cohesive soils. Numerically, M_r is the ratio of the deviator stress to the resilient strain as shown in the following equation:

\[ M_r = \frac{\sigma_1 - \sigma_3}{\varepsilon_{\text{r}}} = \frac{\sigma_d}{\varepsilon_{\text{r}}} \] (2-18)

where M_r is the resilient modulus; σ_1 is the major principal stress; σ_3 is the minor principal stress; σ_d is the repeated axial deviator stress; and ε_r is the recoverable (resilient) axial strain.
2.10.1 Factor Affecting Resilient Modulus Response of Cohesive Soils

Over the past decades several researchers have studied the factors affecting the resilient response of cohesive soils and developed procedures for estimating $M_r$. Mohammad et al. (1995, 1999) observed an increase in resilient modulus with the increase in confining pressure. Several studies (Mohammad et al., 1995, Mohammad et al., 1999, and Andrei, 1999) showed that the resilient modulus of cohesive soils is affected by deviator stress. These observations confirm the stress-dependent nature of the resilient modulus of subgrade soil. Several other investigators (Allen, 1996, Monismith, 1989, Drumm et al., 1997) reported that dry unit weight, size of the specimen and stress pulse shape used in repeated load triaxial testing are other factors influencing the resilient modulus of the cohesive soils.

The presence and variation of moisture affect the durability and strength characteristics of soils and, consequently, their ability to support the traffic loads. Changes in subgrade moisture content have a significant impact on the resilient modulus, which is the primary soil property used in many pavement design procedures. Modulus reduction of over 50 percent can be expected for saturated fine-grained subgrade soils (AASHTO, 1998). Many studies (Mohammad et al., 1995, Temple and Shah, 1987, Fredlund et al., 1977) showed that the resilient modulus of the soil decreases as the moisture content increases. Therefore, it is essential that the modulus chosen for design accurately reflect the in situ moisture conditions of the project site. The AASHTO Design Guide (1993) describes a procedure for accounting for seasonal variations in water content by dividing the year into 12 or 24 periods. Each period is assigned a resilient modulus value based on the estimated water content, which is used to determine a relative damage or reduction factor.
The mean of the relative damage factor is then used to obtain a single value of \( M_r \), known as the “effective roadbed soil resilient modulus” for design. Drumm et al. (1997) provided a method for estimating the decrease in subgrade resilient modulus with a known increase in the degree of saturation. The gradient of the resilient modulus with respect to degree of saturation was used to obtain a correction or reduction to the resilient modulus determined at optimum water content and maximum dry density. The propped correction was shown to yield estimates of the reduced resilient modulus that were suitable for routine design. Salem et al. (2003) addressed the seasonal variation of the subgrade resilient modulus with the change in the moisture content at various seasons. They developed regression models that enable design engineers to assess the seasonal changes in the resilient modulus. It was concluded that the modulus decrease with increasing moisture content. In the new MEPDG (2004) changing in moisture profile in the pavement structures and subgrade over the design life of a pavement are fully considered through a sophisticated climatic modelling tool called the Enhanced Integrated Climatic Model (EICM). The EICM accounts for changes in stiffness of the pavement asphalt layer with temperature and the stiffness of the unfrozen base with the moisture content in each layer through a set of adjustment factors.

2.10.2 Resilient Modulus Models

Over the past decades, many researchers have studied the characteristics of \( M_r \) for various soils and attempted to relate it to the engineering properties and stress states, so as to reduce costs and time associated with laboratory testing. AASHTO design procedure recommends the use of bulk and deviatoric stress models to describe sands and
clays, respectively. The bulk stress and deviatoric stress models, developed by Seed et al. (1967) and Moossazadeh and Witzak (1981), respectively, are as follows:

\[ M_r = k_1 \theta^{k_2} \]  
\[ M_r = k_1 \sigma_d^{k_2} \]

where \( M_r \) is the resilient modulus; \( \theta \) is the bulk stress; \( \sigma_d \) is the deviatoric stress; and \( k_1 \) and \( k_2 \) are model constants.

For slightly cohesive fine grained soils, Thompson and Robnett (1976) proposed a bilinear function of the applied deviator stress to model the resilient moduli obtained from the repeated load triaxial test:

\[ M_r = K_1 + K_2 (K_3 - \sigma_d) \quad \sigma_d \leq K_3 \]  
\[ M_r = K_4 - K_5 (\sigma_d - K_3) \quad \sigma_d \geq K_3 \]

where \( K_1, K_2, K_3, K_4 \) and \( K_5 \) are material constants obtained from laboratory repeated tests.

Since the bulk stress model does not account for shear stresses and strains developed during loading and the deviatoric stress model does not capture the influence of confining pressure on resilient modulus accurately, several models were developed to combine the effect of both bulk and deviatoric stresses in the resilient modulus predictive equations. Among these models, a model presented by Witzak and Uzan (1988) called a universal model. This model incorporates the effect of both the bulk stress as well as the octahedral stress as follows:
\[ M_r = K_1 P_a \left( \frac{\theta}{P_a} \right)^{K_2} \left( \frac{\tau_{oct}}{P_a} \right)^{K_3} \]  

(2-23)

where \( \theta \) is the bulk stress = \((\sigma_1+\sigma_2+\sigma_3)\), \( \sigma_1, \sigma_2, \sigma_3 \) are three principal stresses; \( \tau_{oct} \) is the octahedral shear stress, for triaxial condition \( \tau_{oct} = \sqrt{2/3} (\sigma_1 - \sigma_3) \); and \( P_a \) is the atmospheric pressure.

Mohammad et al. (1999) proposed a relationship known as the octahedral stress state model. This model provides results in octahedral stress environments, which are assumed to represent the realistic stress states as they occur in the field (Houston et al., 1992). This model incorporates the effect of octahedral normal and shear stress as follows:

\[ \frac{M_r}{\sigma_{atm}} = k_1 \left( \frac{\sigma_{oct}}{\sigma_{atm}} \right)^{k_2} \left( \frac{\tau_{oct}}{\sigma_{atm}} \right)^{k_3} \]  

(2-24)

where \( \sigma_{oct} \) is the octahedral normal stress = \(1/3(\sigma_1+\sigma_2+\sigma_3)\); \( \tau_{oct} \) is the octahedral shear stress; and \( \sigma_{atm} \) is the atmospheric pressure.

The MEPDG (2004) adopted the generalized resilient constitutive model shown in the next equation:

\[ \frac{M_r}{P_a} = k_1 \left( \frac{\theta}{P_a} \right)^{k_2} \left( \frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \]  

(2-25)

The coefficients \( k_1, k_2, \) and \( k_3 \) in above equations are regression constants. The stress and resilient modulus were normalized with respect to atmospheric pressure that helps in non-dimensionalizing the constants.
Wolfe and Butalia (2004) developed a model that predicts the resilient modulus of the subgrade soil as a function of the stress state and the basic engineering properties of the soils. The predictive model is given as:

$$\frac{M_r}{P_a} = k_1 \left[ \frac{\sigma_{oct}}{P_a} \left( \frac{\tau_{oct}}{P_a} \right)^2 \right]^{k_2} = k_1 \left[ \frac{9P_a}{2} \left( \frac{1}{3\sigma_d} + \frac{\sigma_3}{\sigma_d^2} \right) \right]^{k_2} \quad (2-26)$$

where $M_r$ is resilient modulus (kPa); $P_a$ is the atmospheric pressure (101 kPa); $\sigma_{oct}$ is the octahedral normal stress (kPa); $\tau_{oct}$ is the octahedral shear stress (kPa); $\sigma_1$ is the major principal stress (kPa); $\sigma_d$ is the deviator stress (kPa); $\sigma_3$ is the confining stress (kPa); and $k_1, k_2 = \text{model constants}$.

Seasonal variations are critical for determining the design $M_r$ for a particular project. The concept being used in development of the new MEPDG under NCHRP Project 1-37A is to apply the Enhanced Integrated Climatic Model (EICM) to predict changes in the physical properties of unbound pavement materials and soils and to estimate the effect those changes have on the resilient modulus.

The MEPDG (2004) adopts the following model to predict the change of modulus due to a change in degree of saturation of the soils:

$$\log \frac{M_r}{M_{ropt}} = a + \frac{b-a}{1 + \exp \left( \ln \frac{b}{a} + k_m \left( S - S_{opt} \right) \right)} \quad (2-27)$$

where $M_r/M_{ropt}$ is the Resilient modulus ratio; $a$ is the minimum of $\log(M_r/M_{ropt})$; $b$ is the maximum of $\log(M_r/M_{ropt})$; $k_m$ is the regression parameter; and $(S-S_{opt})$ is the
variation in degree of saturation expressed in decimal.

The values of a, b, and \( k_m \) for coarse-grained and fine-grained materials are summarized in Table 2.3.

Table 2.3 Values of a, b, and \( k_m \) for coarse-grained and fine-grained materials (MEPDG, 2004)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Coarse-Grained Materials</th>
<th>Fine-Grained Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>-0.3123</td>
<td>-0.5934</td>
</tr>
<tr>
<td>b</td>
<td>0.3</td>
<td>0.4</td>
</tr>
<tr>
<td>( k_m )</td>
<td>6.8157</td>
<td>6.1324</td>
</tr>
</tbody>
</table>

Because soil suction dictates the state of stress in unsaturated soils (Fredlund and Rahardjo, 1993), it is important to understand the influence of soil suction on resilient modulus. Sauer and Monismith (1968) observed that higher soil suction produced higher resilient modulus. Khoury et al. (2003) observed that the resilient modulus increased when matric suction (\( \psi_m \)) increased. Little to no changes in resilient modulus was caused by osmotic suction (\( \psi_\pi \)). The effects of the stresses and soil suctions are explicitly presented in only two models developed by Yang et al. (2005) and Liang et al. (2008). The model developed by Yang et al. (2005) is as follows:

\[
M_r = k_1 (\sigma_d + \chi_w \psi_m)^{k_2}
\]  

(2.28)

where \( \sigma_d \) is the deviator stress; \( \chi_w \) is the effective stress parameter; \( \psi_m \) is the matric suction; and \( k_1, k_2 \) are the regression coefficients.
The model developed by Liang et al. (2008) is as follows:

\[ M_r = K_1 p_a \left( \frac{\theta + \chi_w \psi_m}{p_a} \right)^{K_2} \left( \frac{\tau_{oct}}{p_a} + 1 \right)^{K_3} \]  

(2.29)

where \( \theta \) is the bulk stress; \( \tau_{oct} \) is the octahedral shear stress; \( \psi_m \) is the Matric suction; \( \chi_w \) is the Bishop’s parameter; \( p_a \) is the atmospheric pressure, and \( K_1, K_2, K_3 \) are the regression constants.

2.11 Mechanistic-Empirical Pavement Design Guide (MEPDG)

The American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures, based on the AASHO Road Test (1960s), is the primary document used by state highway agencies to design new and rehabilitated highway pavements. A 1996 survey completed as part of NCHRP Project 1-32 found that 80 percent of states use one of either the 1972, 1986, or 1993 versions of AASHTO Pavement Design Guide (Darter, et al. 1997). While the various versions of the AASHTO design guide have served well for several decades, many serious limitations exist for their continued use as the nation’s primary pavement design procedures. Various versions of the AASHTO design guide could not possibly evaluate all the different combinations of roadway materials and sections, the data collected is only applicable under the specific conditions of the test with regard to the time, place, environment, and material properties present during the test. Extrapolating the original data to different situations and current pavement design conditions has been extremely difficult and often extremely arbitrary.
In recognition of the limitations of the earlier Guides, AASHTO initiated an effort to develop an improved Guide. This work was complete over the past decade (beginning in 1997) under two major research projects: National Cooperative Highway Research Program (NCHRP) Project 1-37A and 1-40. The end product of these research efforts are the development of a mechanistic-empirical pavement design guide (MEPDG) and the accompanying software (ARA 2004, Darter et al., 2006). Unlike empirical pavement design methods (1993 AASHTO Guide), the new proposed MEPDG combines both empirical and mechanistic procedures in determining required pavement thickness for a given set of design inputs. The mechanical model is based on elementary physics and determines pavement response to the wheel loads or environmental condition in terms of stress, strain, and displacement. The empirical part of the design uses the pavement response to predict the life of the pavement on the basis of actual field performance (Timm, Birgisson, and Newcomb, 1998). MEPDG offers pavement design engineers greater flexibility over the current guide in term of:

- Evaluating the effects of various pavement materials, traffic loading conditions, design features, and construction practices;
- Considering both long-term (age) and short-term (temperature and moisture) changes in material properties (the MEPDG uses the Enhanced Integrated Climatic Model, EICM, as a moisture prediction tools for base, subbase, and subgrade);
- Providing more accurate performance predictions so that the frequency of premature failure is reduced;
• Improving the ability to evaluate premature failures and the factors contributing to exceptionally good performance;

• Compiling databases for the updating of pavement design input values as information becomes available.

The MEPDG procedure has adequate flexibility built into it to enable agencies to adopt it in a manner that is commensurate with the resources available to them at any given time. The flexibility is afforded by the three hierarchal input levels, nationally calibrated performance models, and nationally established default input values for several parameters. However, before successfully adopting this procedure, several implementation issues will likely need to be addressed by the state highway agencies (SHA’s). Two of the more implementation issues are (1) verification of the reasonableness of national defaults for program inputs using locally generated/used site and design information and establishment of local defaults and (2) validating and calibrating the nationally calibrated prediction models using agency-specific data (ARA, 2008).

2.11.1 Design Approach

Unlike the 1993 AASHTO guide, in which the pavement structure’s thicknesses are obtained directly from the design equation, the NCHRP 1-37A design process is an iterative process in which predicted performance of selected pavement structure is compared against the design criteria as shown in Figure 2.3. The structure and/or material selection are adjusted until a satisfactory design is achieved. A step-by-step description is as follows:
Definition of a trial design for specific site conditions: subgrade support, material properties, traffic loading, and environmental conditions.

Definition of design criteria for acceptable pavement performance at the end of the design period (i.e., acceptable levels of rutting, fatigue cracking, thermal cracking, and roughness).

Selection of reliability level for each one of the distresses considered in the design;

Calculation of monthly traffic loading and seasonal climate conditions (temperature gradients in asphalt concrete layers, moisture content in unbound granular layers and subgrade).

Modification of material properties in response to environmental conditions.

Computation of structural responses (stresses, strains and deflections) for each axle type and load and for each time step throughout the design period.

Calculation of predicted distresses (e.g., rutting, fatigue cracking) at the end of each time step throughout the design period using the calibrated empirical performance models.

Evaluation of the predicted performance of the trial design against the specified reliability level. If the trial design does not meet the performance criteria, the design (thickneses or material selection) must be modified and the calculations repeated until the design is acceptable.
2.11.2 Input Level Hierarchy

The reason behind the hierarchical approach is to customize between the level of engineering effort exerted in the design process and the relative importance, size and cost of the project. The hierarchical approach is mainly employed for traffic, materials, and environmental inputs (MEPDG, 2004). In general, three levels of inputs are provided.

- Level 1 (Highest) - Level 1 inputs provide for the highest level of accuracy and, thus, would have the lowest level of uncertainty or error. The input data is obtained from direct testing on the actual project material in question. Example is the dynamic modulus testing of an asphalt concrete mix.
- Level 2 (intermediate) - Level 2 inputs provide an intermediate level of accuracy. It is used when direct test results for a given parameter cannot be obtained but results from other related tests are available which can then be correlated to the required input. For example, asphalt concrete modulus can be estimated from the binder, aggregate and mix properties.

- Level 3 (lowest) - Level 3 inputs provide the lowest level of accuracy and intended for use for lower volume roadways. At level 3, not only the direct test results unavailable, but secondary test results (e.g., California Bearing Ratio, CBR) are also not available. Level 3 permits the user to enter values for specific parameters base on either historical agency specifications or MEPDG supplied national default values.

According to the NCHRP 1-37A report, level 1 is recommended for heavily trafficked pavements or wherever there is dire safety or economic consequences of early failure. Level 2 can be used for intermediate projects, while level 3 is recommended for minor projects, usually low traffic roads. In addition, level 3 may be appropriate for pavement management programs widely implemented in highway state agencies.

For a given design project, it is possible to mix and match the levels of input, such as concrete modulus of rupture from Level 1, traffic load spectra from Level 2, and subgrade resilient modulus from Level 3. In addition, it is important to realize that no matter what input design levels are used, the computational algorithm for damage is exactly the same. The same models and procedures are used to predict distress and smoothness no matter what levels are used to obtain the design inputs (MEPDG, 2004).
2.11.3 Pavement Material Characterization

Unlike AASHTO flexible pavement design procedure, MEPDG requires a large set of material properties. The material inputs needed for the design process may be classified in one of the three major groups: pavement response model material inputs, material-related pavement distress criteria, and climatic model material inputs. Climatic-related properties are used to determine temperature and moisture variation inside the pavement structure. The pavement response models use material properties to compute the state of stress/strain at critical locations in the structure due to traffic loading and temperature variations. These structural responses are used by the distress models along with complementary material properties to predict pavement performance.

2.11.3.1 Hot Mix Asphalt Concrete (HMA) Materials Characterization

The primary input parameter is the HMA dynamic modulus (E*). At level 1, the MEPDG recommends HMA dynamic modulus testing in the lab following guidelines presented in AASHTO TP 62 or ASTM D3497. Also, required at level 1 is the asphalt binder complex shear modulus and phase angle testing (AASHTO T315). These are used to develop an HMA E* master curve. For Level 2 and 3, Witczak’s dynamic modulus prediction model, which requires HMA gradation, air voids, volumetric binder content, and asphalt binder type as inputs, is used to estimate E* and develop the Master Curve. Additional testing is necessary to characterize HMA for predicting thermal cracking. The additional testing includes: Tensile strength (AASHTO T322), Creep compliance (AASHTO T322), and Thermal conductivity and heat capacity (ASTM E 1952 and ASTM D2766).
2.11.3.2 Portland Cement Concrete (PCC) Materials Characterization

The material characterization input parameters required for the new MEPDG to design/analysis of the rigid pavement include: the elastic constants (elastic modulus, Poisson’s ratio) of the PCC to compute the developed stresses and strains in the concrete slab, the modulus of rapture of flexural strength (MR), to estimate the fatigue life of the concrete, the thermal coefficient of expansion (CTE) to calculate the joint opining and curling-induced stresses in the slab, and composite modulus of subgrade reaction (k-value) to calculate the surface deflections and joint faulting.

2.11.3.3 Chemically Stabilized Materials Characterization

The chemically stabilized materials covered in the MEPDG include lean concrete, cement stabilized, cement treated open graded drainage layer, soil cement, lime, cement and fly ash treated materials. The elastic modulus of the layer is the primary input parameter for chemically stabilized materials. For lean concrete and cement treated materials in new pavements, the elastic modulus is determined using ASTEM C 469. For lime stabilized materials, AASHTO T 307 protocols apply.

2.11.3.4 Unbounded Base, Subbase, and Subgrade Materials

For unbound materials included subgrade soils, the primary inputs required by the MEPDG include the resilient modulus at optimum moisture content, optimum moisture content (OMC) and maximum dry density (MDD), specific gravity, saturated hydraulic conductivity, and soil water characteristic curves (SWCC) parameters.
The resilient modulus input can be obtained as a function of stress state at level 1 for HMA pavements. However, this approach is not recommended at this time in the MEPDG. At level 2 and 3, they can be estimated with other, more easily obtained soil properties. For example, at level 2, the unbound layer $M_r$ can be estimated through correlations with several other commonly tested soil properties such as California Bearing Ratio (CBR), R-value, and AASHTO layer coefficients ($a_i$). At level 3, the resilient modulus of unbound materials is selected based on the unbound material classification (AASHTO or USC) either from agency-specific testing or by adopting the MEPDG defaults. The MEPDG provided a general range of typical modulus values (based on LTPP averages) for each unbound material classification at their optimum moisture content and maximum dry density.

2.11.4 Traffic Data

Traffic data are key inputs for the analysis and design of pavement structures. In the past, the AASHTO Design Guides quantify traffic in terms of equivalent single axle loads (ESALs). However, the MEPDG requires a lot more detailed traffic data. Essentially, the MEPDG requires the raw traffic data used to estimate ESALs, namely, the load distribution on each axle for each month of the design period. In addition, the MEPDG requires other traffic inputs not often considered in pavement design such as wheel wander, 24-hours truck counts, wheelbase distribution (i.e., distance between the drive axle and the first axle on the trailer), etc.
2.11.5 Climatic Factors

Climate conditions have a significant effect on the performance of both flexible and rigid pavements. Climatic factors, such as precipitation, temperature, freeze-thaw cycles, and frost penetration depth, play a key role in affecting the material properties and the performance of the pavement. Consequently, the susceptibility of the pavement materials to moisture and freeze-thaw induced deterioration, the drainability of the paving layers, the infiltration potential of the pavement, also define the extent to which the pavement will react to the climatic conditions.

As part of the MEPDG, the pavement engineers can carry out numerical simulations using the Enhanced Integrated Climatic Model (EICM) software to compute the moisture, temperature, frost depth of a pavement under the prescribed climatic conditions. Two options to specify the climate file are available:

- Import a previously generated climate .icm-file, or
- Generate the .icm-file using the weather data available in EICM for several weather stations across the United States.

2.11.6 MEPDG Performance Prediction Models

A brief description of the MEPDG models used to predict the pavement distresses (alligator cracking, transverse cracking, rutting, and smoothness, IRI) is presented in the following sections.

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2.11.6.1 Fatigue (Alligator) Cracking

Fatigue cracking is the result of repeated tensile stresses induced at the bottom of the surfacing layer bending under heavy loads. This type of cracking first show up as short longitudinal cracks in the wheel path that quickly spread and become interconnected to from a chicken wire/alligator cracking pattern. It is believed that the fatigue cracking initiates at the bottom of the HMA layer and propagates to the surface with continued application of heavy truck traffic. MEPDG utilizes cumulative damage concept approach (NCHRP, 2004.a) based on Miner’s law to predict the fatigue cracking in the flexible pavement sections. An incremental damage index is calculated by dividing the actual number of axle loads by the allowable number of axle loads within a specific time increment and axle load interval for each axle type (Miner, 1945).

2.11.6.2 Thermal (Transverse) Cracking

Cracking in flexible pavements due to cold temperatures or temperature cycling is commonly referred to as thermal cracks. According to Zubeck and Vinson (1996), transverse cracks typically occur as the result of the thermal contraction of the surfacing layer. Thermal cracks typically appear as transverse cracks on the pavement surface roughly perpendicular to the longitudinal axis of the road. These cracks can be caused by shrinkage of the HMA surface due to low temperatures, hardening of the asphalt, and/or daily temperature cycles (Mallela, et al., 2002). In MEPDG, the Enhanced Integrated Climatic Model (EICM) is used to generate a temperature file that defines the temperature–depth relationship for use in flexible pavement thermal cracking prediction. The file includes temperature values on an hourly basis for the entire analysis period. The
thermal analysis procedure is based on linear viscoelastic principles and requires data from the creep compliance test conducted at three temperatures (0, -10, -20 °C) or one temperature, depending upon the level of analysis and indirect tensile test conducted at -10 °C. Data from these tests are used to relate creep compliance, \( D(t) \), to relaxation modulus, \( E_r \), of the asphalt mix. This information is then used to estimate thermal stress at any given depth and time in the AC layer. Thermal cracking permits water infiltration into the underlying pavement layers that can cause structural failure of the pavement. Thermal cracking also contributes directly to a loss of smoothness (MEPDG, 2004).

2.11.6.3 Permanent Deformation (Rutting)

Rutting is a surface depression in the wheel paths caused by inelastic or plastic deformations in any or all of the pavement layers and subgrade (Mallela, et al., 2002). Regardless of the material type considered, there are generally three distinct stages for the permanent deformation behavior of pavement materials under a given set of material, load and environmental conditions which can be described as follows (MEPDG, 2004):

- **Primary Stage**: high initial level of rutting, with a decreasing rate of plastic deformations, predominantly associated with volumetric change.

- **Secondary Stage**: small rate of rutting exhibiting a constant rate of change of rutting that is also associated with volumetric changes; however, shear deformations increase at increasing rate.

- **Tertiary Stage**: high level of rutting predominantly associated with plastic (shear) deformations under no volume change conditions.
MEPDG employs the incremental damage approach to calculate the damage or rutting in each sub-layer (NCHRP, 2004a). Once the material type of each sub-layer is identified, appropriate model is used by the system to calculate the accumulated plastic strains at the mid-depth of each sub-layer in each sub-season. The total permanent deformation is then calculated as the sum of permanent deformation in each sub-layer.

2.11.6.4 Smoothness (IRI)

Pavement roughness is the result of vertical differences between the ideal surface profile and the actual surface profile. A rough pavement directly affects the ride quality. An incremental increase in surface distress causes surface roughness to increase (NCHRP, 2004). The NCHRP 1-37A project found that rutting, thermal cracking, and fatigue cracking were the most dominant distresses affecting roughness of a flexible pavement structure at a given point in time. Local environmental conditions and the base type supporting the surface layer were also considered as important factors in estimating the pavement roughness.

2.12 Enhanced Integrated Climatic Models (EICM)

Environment is considered as one of the essential factors that influence pavement material properties and their field performance. Consequently, a sound pavement design requires a good ability to estimate or predict the seasonal variations in pavement material properties. The major environmental factors that influence pavement design and performance are moisture, temperature, and freeze-thaw cycles. In the new mechanistic-empirical pavement design guide (Design Guide, 2002), the effect of moisture and
temperature variations on the pavement materials characteristics have been full
considered through the use of a climatic modeling computer program called the Enhanced
Integrated Climatic Mode (EICM). The EICM is a one-dimensional coupled heat and
moisture flow model initially developed for Federal Highway Administration (FHWA)
by Lytton et al (1989) and then upgraded by Larson and Dempsey (1997) and adapted for
use in the new mechanistic-empirical pavement design guide (MEPDG) developed under
NCHRP Project 1-37A. The EICM is used to simulate the changes in the behavior and
characteristics of pavement and subgrade materials in conjunction with climatic
conditions over several years of operation. The EICM program has undergone several
revisions since its initial development at the University of Illinois in the 1960’s. The
latest improvement on the EICM was done by incorporating the concept of Thornthwaite
Moisture Index (TMI) along with the improved models developed through a research
project conducted under NCHRP Project 9-23. The results of NCHRP Project 9-23 were
report in NCHRP Report 602 by Zapata and Houston (2008). The EICM consists of four
major components:

- The Climatic-Materials-Structures Model (CMS), developed by Dempsey et al.
  (1985), is a one-dimensional, forward finite difference heat transfer model. The
  CMS model is used to determine frost penetration and temperature distribution in
  the pavement system. CMS model uses weather data (sunshine percentage, wind
  speed, air temperature and solar radiation) along with materials thermal properties
  (heat capacity, thermal conductivity, and pavement surface absorptivity) to
  compute the heat flux boundary condition on the roadway surface and the
  resulting temperature profile throughout the pavement. The model reads the
changes in moisture content of the subgrade from the CRREL Model to compute changes in pavement, base and subgrade properties over time.

- The Precipitation Model, developed by Liang and Lytton (1989), uses average climatic data and mathematical concepts to simulate rainfall patterns that are considered acceptable for design purposes. This module of the ICM provides the amount of rain and the day on which rainfall occurs, which is in turn a required input to the Infiltration and Drainage Model. These data were used along with the drainage analysis to compute the probabilities of wet and dry days, wet and dry base courses and the probability of developing base course moduli associated with different degrees of saturation.

- The United State Army Cold Regions Research and Engineering Laboratory (CRREL) Frost Heave and Thaw Settlement Model, developed by Guyman et al. (1986), is a one-dimensional mathematical model of coupled heat and moisture transport in soils. The model computes phase changes of water to predict frost heave and thaw settlement. The model uses the temperature profiles through the asphalt layers as established by the CMS model to compute changes in the soil temperature profile, and thus frost penetration and thaw settlement.

- The Infiltration and Drainage Model (ID), developed by Liu and Lytton (1985), performs drainage analysis of granular bases to evaluate the design of the pavement base. The model requires precipitation data from the precipitation model or from actual data input by the user.
2.12.1 EICM Inputs

Application of the EICM requires assembly of a data set characterizing the pavement cross-section, materials, and external environment. This section provides a brief description of the input data required to model pavement sections using the EICM. The input data required by the EICM are as follows (Larson and Dempsey, 2003):

- Initialization data, which defines the analysis period, the geographic location of the section under consideration (latitude, longitude, and elevation) and the time increments to be used in the simulation and reporting of the results.

- Climatic boundary conditions, including temperature, rainfall, wind speed, percent sunshine, and water table depth data. User can use climatic data provided with the program where section-specific weather data are not available.

- Thermal properties, including surface shortwave absorptivity and limits freezing range. Thermal properties are necessary for computing heat transfer at the pavement surface and the top of pavement layers.

- Infiltration and drainage inputs, which are necessary to determine the amount of rainfall that infiltrates through the pavement, and the amount of drainage occurring in the base course and the amount of moisture infiltrating into the subgrade soil. Two methods are used for determining the base course moisture in the EICM, TTI infiltration and drainage model (ID) and Thornthwaite moisture index base course mode.

- Asphalt material properties, including layer thickness, mix design information, data defining the modulus-temperature relationship, and thermal characteristics.
Unbound material properties, including porosity of the layer, layer thickness, bulk specific gravity, saturated permeability, dry unit weight, and other data characterizing the base, subbase, and subgrade layers.

- The initial temperature and water content profile which enable the user to control the output depths and to input the initial temperature, but not the initial moisture content profiles at this time.

2.12.2 EICM Outputs

One of the major outputs of the EICM is a set of adjustment factors for unbound materials layers that account for the effects of environmental parameters and conditions such as moisture content changes, freezing, thawing, and recovery from thawing. These adjustment factors are used to compute the composite environmental adjustment factor \( F_{\text{env}} \), which then used by the MEPDG to modify the unbound materials resilient modulus as a function of position and time. Other important outputs of the EICM include the predictions of in situ temperature and moisture profiles. In the MEPDG, the predicted temperature profile through the asphalt layer is used in both fatigue and permanent deformation prediction models; whereas the predicted moisture profile is used in the permanent deformation model for the unbound materials. Also, the EICM computes and predicts the following data throughout the pavement profile: pore water pressure, frost and thaw depth, and frost heave and drainage performance. The time period for which this data is generated can range from one day to one year. The time interval for the data output can ranges from 6 minutes to 12 hours.
3.1 Abstract

In 2002 the Ohio Department of Transportation initiated a research project to assess the effectiveness of the drainable base materials under in-service conditions. To that end, TDR probes, two weather stations, and piezometers were installed at I-90 project site to monitor moisture content, climatic data, and ground water elevations, respectively. The main focus of this chapter is to investigate the moisture regimes under different permeable base materials over six years of data monitoring. Pavement sections built with cement treated and asphalt treated permeable base materials showed lower variations in their subbase and subgrade moisture contents than those built with unbounded permeable base materials. Based on the field long-term moisture monitoring, it is not recommended to use either ODOT 304 or ODOT 307-IA as a drainable base materials in the subsurface drainage design. There was no evidence of full saturation in base and subbase materials. At shallow subgrade, ODOT 304, ODOT 307-IA, and ODOT 307-NJ pavement sections were experienced fully saturation conditions for almost 64, 10, and 5 percent of the time, respectively. Attempts to correlate the precipitation amounts with pavement moisture contents were unsuccessful.
3.2 Introduction

Sustained long period of water staying within a pavement system is undesirable because the free water can not only adversely affect the load carrying capacity of the pavement but also cause premature pavement failures. According to Huang (1993), accumulated water in the pavement structure can cause one or more of the following forms of deterioration: reduction in soil shear strength, pumping action in rigid pavements, immigration of fines into drainable base layers, frost heave and thaw, differential swelling in expansive soils, stripping of asphalt in flexible pavement, and durability cracking in rigid pavement. These water induced premature pavement failures can cause serious safety hazards to traffic as well as undermine the serviceability of the pavement (Moulton, 1980). Therefore, providing adequate drainage in pavement design to effectively drain off water from pavement structure has been considered as an essential ingredient to ensure satisfactory long-term performance of the pavement system. Water can get into pavement structure from different sources, such as pavement cracks, infiltration through shoulders or surface, infiltration from side ditches, and ground water capillary action (Huang, 1993).

The current trend to remove the water from different layers of a pavement system is to use a subsurface drainage system. This is accomplished by different approaches, such as using open graded drainable base, asphalt or cement stabilized bases, edge drains, and geosynthetics. Naturally, one of the questions concerning the available subdrainage design approach is the effectiveness of various types of drainable base courses. Field monitored moisture data over a long term service life of pavement sections built with different drainable base materials in the same climatic (precipitation) and subgrade soil
conditions may provide realistic evaluation of the drainage performance of these drainable base materials.

There have been some recent field monitoring research undertaken by several researchers, including Rainwater et al. (1999), Janoo and Shepherd (2000), Sargand (2002), Heydinger (2003), Wolfe and Butalia (2004), Figueroa (2004), Sargand et al (2007, 2008) and Thang et al. (2008). Generally, subgrade moisture content beneath the pavement was found to oscillate from season to season, with maximum values occurred in summer season and minimum values occurred in winter season. In some instances, the subgrade moisture content was found to approach the full saturation level. Also, if ground water table was near the pavement layers, the capillary action was hypothesized to have drawn groundwater to keep subgrade soils fully saturated. Time lag could be observed between the ground water elevation variation with time and the affected subgrade volumetric water content change. The effect of precipitation on measured or observed soil moisture content is not well defined. While some studies have suggested that precipitation has a considerable influence on subgrade moisture content (Figueroa 2004, Thang et al. 2008), others could not establish a firm relationship (Hall and Shreenath, TRR 1999, Sargand 2002, Diefenderfer 2002, Heydinger 2003,). In general, most of newly HMA surface layers resist the infiltration of the surface water which in turn leaves the subgrade moisture content unaffected by the amount of precipitation (Sargand 2002).

In the above mentioned research, however, none of them was dedicated to specifically comparing the effectiveness of various types of drainable base materials on the moisture regime in the pavement layers, including base, subbase, and subgrade.
The main purpose of using the drainable base in a flexible pavement is to facilitate drainage of the infiltrated water in a timely manner so as to reduce the probability of the water infiltrating into subbase or subgrade. According to Sargand (2002), pavement sections built with free-draining base materials had shown low variations in their subgrade moisture contents with time. However, in a more recent study, Wolfe and Butalia (2004) and Sargand et al. (2006) concluded that the several drainable base materials studied exerted no effects on the monitored subgrade moisture contents. The degree to which the subgrade moisture content is affected by the base type is greatly influenced by pavement surface, shoulders and side ditches conditions. Intact pavement surface, shoulders and side ditches ensure less amount of surface water that can be infiltrated through the pavement structures. In fact, the relative performance of different base course materials could be clearly distinguished over time as pavement conditions getting worse (Sargand 2002).

The main purpose of this chapter is to evaluate the effectiveness of various types of drainable base courses by analyzing the moisture regimes under different permeable base materials over six years of data monitoring. Base materials considered in this study consisted of both bound and unbound base courses. Unbound base materials included ODOT 304, ODOT 307NJ, 307IA, and 307CE. Bound base materials included asphalt treated, and cement treated base materials.

3.3 Project Description

For the purpose of evaluating effectiveness of different drainable base materials in flexible pavement, a section of totally re-built I-90 in the Ashtabula County, Ohio, was
selected for extensive instrumentation and monitoring of moisture regimes under six
different drainable base materials. The length of pavement with each type of drainable
base material is 500 ft long. The cross section of the as-built pavement is shown in Figure
3.1. The subsurface drainage system included two perforated underdrain pipes. The
shallow underdrain was a 4-inch diameter pipe within the shoulder edge, which was used
primarily to drain the water in the base course and subbase course. The underdrain pipe
was a 6-inch diameter perforated pipe placed underneath the outer edge of highway lane
for draining water infiltrated to that depth. The 4-inch diameter underdrain pipes were
wrapped with fabric cover to prevent potential clogging due to migration of fine materials.

Figure 3.1 Typical highway cross section in ATB-90 projects
3.4 Site Materials

Site materials consist of subgrade soils, subbase course, base course, and hot mix asphalt. A brief description of these materials is described in the following sections.

3.4.1 Subgrade Soils

The soil boring conducted during design and subsequent instrumentation installation stage showed that native subgrade silty clay soil (A-6a) extended at least 6 ft from the original ground surface. The liquid limit and plastic limit of native subgrade soils were 31.2 and 18.2, respectively. The initial water content was 15.7%.

3.4.2 Subbase Course Materials

The six-inch thick subbase layer was constructed with ODOT 304 course material, compacted over a six-inch ODOT 304 and stone rockfill materials. It should be noted that due to weak silty clay subgrade, geogrid mesh was placed on the native subgrade surface prior to constructing the 304 layer. Moisture content for the constructed ODOT 304 subbase course was 6.7%. The average dry density was 126 lb/ft³.

3.4.3 Base Course Materials

Five different drainable base materials and one controlled base material (ODOT 304) in the Ohio Department of Transportation Materials Specifications were built in six monitoring pavement sections (each section was 500 ft long). The designation of the base materials are as follows: unbound base materials include ODOT 304, ODOT 307-IA, ODOT 307-NJ, and ODOT 307-CE, and bound base materials include ODOT 306
(cement treated base) and ODOT 308 (asphalt treated base). The thickness of base layer in all pavement sections was 4 inches except for ODOT 307-CE where 6 inches was used. The cement treated base (ODOT 306) was made by mixing AASHTO #57 stones with cement at 250 lb per cubic yard and a water cement ratio (w/c) of 0.36. The asphalt treated base (ODOT 308) was made by mixing AASHTO #57 stone with 1.5 to 3.5 percent by weight of PG 64-22 asphalt binder. The gradation of AASHTO #57 stone, ODOT 304 (typical base material), and other unbound base materials (ODOT 307 series) are shown in Figure 3.2. The average measured moisture content was 5.7% and 4.8% for the ODOT 304 base and ODOT 307 series, respectively. The average dry density was 134 lb/ft$^3$ and 126 lb/ft$^3$ for ODOT 304 and ODOT 307 series, respectively.

Figure 3.2  Gradation curves for ODOT 307, ODOT 304 and No. 57 base course materials
3.4.4 Hot Mix Asphalt (HMA) Surface Layer

The HMA layer was constructed in two lifts of asphalt concrete base (ODOT 302), an intermediate course layer (Item 858, Type A. 19.0 mm) and a surface course layer (Item 858, Type A. 12.5 mm). The total thickness of the HMA layer was 15.25 inches.

3.5 Hydraulic Conductivity

The base, subbase, and subgrade soils used for constructing the pavement sections for field monitoring were brought into the testing laboratory for determining the hydraulic conductivity according to ASTM D2434-68 standard (2000) and AASHTO T215-70 (1993) specifications. The constant head test was used for granular base materials, while the falling head test was conducted for subgrade materials. The test results are shown in the Table 3.1. If Cedergren (1974) recommended values (i.e., 3000 ft/day.) for drainable base materials was considered, then ODOT 304 and ODOT 307 IA coarse gradation, ODOT 307 NJ, CE median, and the bound materials using No. 57 stones will meet the requirements.
Table 3.1 Laboratory hydraulic conductivity test results (after Liang, 2007)

<table>
<thead>
<tr>
<th>Material</th>
<th>Hydraulic Conductivity, k (ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>304 - Fine</td>
<td>206</td>
</tr>
<tr>
<td>304 - Median</td>
<td>1417</td>
</tr>
<tr>
<td>304 - Coarse</td>
<td>5443</td>
</tr>
<tr>
<td>No. 57</td>
<td>26563</td>
</tr>
<tr>
<td>307 – NJ Fine</td>
<td>2234</td>
</tr>
<tr>
<td>307 – NJ Median</td>
<td>3824</td>
</tr>
<tr>
<td>307 – NJ Coarse</td>
<td>7850</td>
</tr>
<tr>
<td>307 – IA Fine</td>
<td>873</td>
</tr>
<tr>
<td>307 – IA Median</td>
<td>2277</td>
</tr>
<tr>
<td>307 – IA Coarse</td>
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<td>Asphalt Stabilized @ 107º F</td>
<td>25061</td>
</tr>
<tr>
<td>A-6 Subgrade Soils</td>
<td>15.8</td>
</tr>
</tbody>
</table>

* Number of freezing/thawing cycles.

3.6 Field Instrumentations

Extensive soil investigations were performed on the subgrade soils before the instrumentation sites were selected. There was no high ground water table observed in the preliminary soil investigations at the instrumentation sections. This would ensure avoiding the influences of the capillary action of high ground water table near the pavement surface on the outcome of the performance assessment of the drainable bases. The water content monitoring started on the 4th of September 2003 in the Asphalt treated and Cement Treated sections. Monitoring of the 307-IA, 307-NJ and 307-CE sections
started on the second week of October 2003. Finally, monitoring of ODOT 304 section started on the 24th of December 2003.

Filed instrumentation at the ATB-90 site consists of using embedded TDR probes to measure the soil water contents. A weather station with the capability of monitoring solar radiation, air temperature, wind speed, wind direction, and rainfall was also installed at the site. The ground water table at the site was monitored using piezometers.

It should be noted that TDR has been successfully used in the past for monitoring moistures in subgrade, as reported by Neiber and Baker (1989). For more detailed information on TDR, the original work by Fellner-Feldegg (1969), Davis and Chudobiak (1975), and Topp (1985) can be consulted. In this study, TDR probes manufactured by Campbell Scientific Inc. were used. A total of 20 TDR probes and 18 TDR probes are used in unbounded and bounded base material sections, respectively. For each monitored section, two complete duplicate sets of sensors were installed: one beneath the centerline of the driving lane and the other one beneath the centerline of the passing lane. The CR10X data loggers were used to monitor and store the data at each test section. A 12 volt DC power was used to provide power source to each data logger. The depth and spacing of each TDR sensor are depicted in Figure 3.3 for pavement sections with both bound and unbound base courses.
3.7 Data Monitoring

Precipitation data from the on-site weather station was processed and plotted in Figure 3.4 to show the daily precipitation over the monitoring period. The gravimetric moisture content obtained from the embedded TDR probes are presented in the following sections.
3.7.1 Moisture Variation with Time

Gravimetric moisture from the instrumented pavement sections was recorded every 8 hours. The average daily gravimetric moisture contents (calculated as the average of the three 8-hours readings for the day) at the base, subbase, subgrade soil near the subbase, and subgrade soils at about 5 ft below the asphalt concrete layer on both driving and passing lanes are plotted with date in Figure 3.5 through Figure 3.8. The gravimetric water content calculated to be corresponding with 100 percent degree of saturation for each layer is also presented inside the box in the figures. An initial jump of the gravimetric moisture contents at the base, subbase and some of subgrade layers during the initial stages of data collection can be observed in the pavement sections with unbound drainable base and cement treated bound drainable base. The explanation of this
initial quick jump was attributed to the heavy rainfall immediately after the installation of the sensors before the asphalt concrete layers can be placed. For the pavement section built with asphalt treated bound drainable base materials, there was no recorded precipitation in the period between the TDR installation and asphalt concrete layer placement.

In general, it seems that time-dependent variation of the gravimetric moisture contents in the driving lanes was higher than those for the passing lanes. This could be related to the location of the shallow under-drain pipes. In the driving lanes, the shallow under-drain pipe was located about 22 feet away from the centerline of the road, while in the passing lane direction it was located 16 feet away. It seems that in the driving lane direction, more time is needed to drain the infiltrated water out of the pavement system compared with that for the passing lane.

The pavement sections built with unbound drainable base materials showed higher moisture variation in their base and subbase layers compared with the pavement sections built with bound drainable bases. The fact that bound drainable base has higher hydraulic conductivity than the unbound drainable base materials explains this observed difference. The degree of variation of moisture at the subgrade layer near the subbase was seen in Figure 3.7 to be the least in the pavement sections built with bound drainable base, thus confirming a superior performance of bound drainable base in preventing moisture migration into the top elevation of the subgrade soils.

Almost all the measured gravimetric moisture content was at the level less than the estimated gravimetric moisture content corresponding to 100 percent degree of saturation, with the exception at the pavement section built with ODOT 304 as base layer (i.e., no
drainable base material was used). This observation leads to a suspicion that due to less than satisfactory hydraulic conductivity, ODOT 304 may not be effectively draining the infiltrated water in a timely manner.

After spring 2007, sharp jumps in the measured gravimetric moisture contents can be noticed at both pavement sections built with ODOT 304 and ODOT 307-IA drainable base material. At ODOT 304 section, jumps occurred at driving lane at both base and subgrade layers, whereas at ODOT 307-IA section, jumps occurred at both driving and passing lane at the top elevation of the subgrade layer. Jumps in the measured moisture content in the subgrade soils at ODOT 307-IA section were unexpected since both base and subbase layers did not show the jump.

Equilibrium moisture content is defined as the final moisture content after which no significant variation in moisture content was observed. According to the results shown in Figure 3.5, it can be seen that all sections showed higher fluctuation in base layer moisture content for the early time period just after site construction, and then the moisture content for most of the sections moved towarded long-term equilibrium with little seasonal fluctuation. The equilibrium state was reached after almost one year of pavement in service. The exception to the above observation was noticed at the driving lane direction of ODOT 304 pavement section where no equilibrium state could be reached. For unbound drainable base materials, no equilibrium in moisture content was observed in the subbase and subgrade layers at both passing and driving lanes as shown in Figure 3.6 through Figure 3.8. On the other hand, bound drainable base materials (cement treated and asphalt treated) showed long-term equilibrium with little seasonal fluctuation.
Figure 3.5  Average daily water content measurements in Base Layer (a) driving lane (b) passing lane
Figure 3.6  Average daily water content measurements in subbase Layer (a) driving lane (b) passing lane
Figure 3.7 Average daily water content measurements in top subgrade layer (top 30 cm of subgrade layer) (a) driving lane (b) passing lane
Figure 3.8 Average daily water content measurements in bottom subgrade layer (at 180 cm from base surface) (a) driving lane (b) passing lane
3.7.2 Moisture Variation with Depth

The average of six years moisture content profiles with depth at four seasons are presented in Fig 3.9 through Figure 3.14 for six monitored pavement sections. It is apparent from these figures that the gravimetric moisture content in the late spring and early summer seasons are greater than those in the late fall and early winter seasons. The subgrade at shallow depth (top 12 inches of subgrade layer) exhibits higher gravimetric moisture content variations compared with base and subbase layers. Moisture content variations at the bottom of the subgrade (70 to 88 inches from the pavement surface) were much lower than those near the subgrade surface. This implies that moisture content near the subgrade surface is affected by surface water. No clear correlation between the moisture regimes under the pavement system and the type of the base materials could be observed in this study.
Figure 3.9 Average 6 year water content measurements versus depth for ODOT 304

Figure 3.10 Average 6 year water content measurements versus depth for ODOT 307-NJ
Figure 3.11  Average 6 year water content measurements versus depth for ODOT 307-IA

Figure 3.12  Average 6 year water content measurements versus depth for ODOT 307-CE
Figure 3.13  Average 6 year water content measurements versus depth for ODOT 308-asphalt treated

Figure 3.14  Average 6 year water content measurements versus depth for ODOT 306-cement treated
3.8 Statistical Analysis of Moisture Data

Moisture coefficient of variation (COV), soil materials degree of saturation, and moisture variation with precipitation will be discussed in the following sections.

3.8.1 Moisture Coefficient of Variation (COV)

The coefficient of variation is calculated as the ratio of the standard deviation to the mean, and it can be used as a statistic comparison of the degree of variation from one data set to another, even though the means of the two sets may be different. Using the data from the entire monitoring period (from October 2003 to September 2008), the mean and standard deviations of the moisture contents at each measurement point are calculated and presented in Tables 3.2 and 3.3. In general, large variation of the computed COV was observed in the base and subbase materials, whereas relatively small variation was observed at the top 12 inches of the subgrade layer. The smallest COV was actually observed at the bottom of the subgrade (i.e., 72 inches from the pavement surface). Both ODOT 306 cement treated base and ODOT 308 asphalt treated base materials showed the smallest COV values compared with the other base materials, particularly in the subgrade soils at the shallow depth. If using COV values at the shallow depth of the subgrade layer as a criterion of judging the effectiveness of drainable base in preventing moisture migration into subgrade, then the ranking from best to worst of the six base materials can be as follows: ODOT 306-cement treated base, ODOT 308-asphalt treated base, ODOT 307-CE, ODOT 307-NJ, ODOT 307-IA, and finally ODOT 304.
3.8.2 Soil Materials Degree of Saturation

In this section, more analysis of the moisture regime in the soils is presented, including the average degree of saturation, the percent of time when the moisture content is greater than or equal to the moisture content at full saturation, and the time of the year when full saturation occurs. The average degree of saturation is calculated according to the following equation:

\[
\text{Average degree of saturation \( \% \)} = \left( \frac{\theta_{g,\text{avg}} - \theta_{g,S=100\%}}{\theta_{g,S=100\%}} \right) \times 100\% \quad (3-1)
\]

where:

\( \theta_{g,S=100\%} \) = gravimetric water content at fully saturation

\( \theta_{g,\text{avg}} \) = Average 6 year gravimetric water content

The percent of time when the measured water content is greater than or equal to the water content at full saturation is calculated according to the following equation:

\[
\text{Percent of Time when } (\theta_g \geq \theta_{g,S=100\%}) = \left( \frac{N_{\theta_g \geq \theta_{g,S=100\%}}}{N_T} \right) \times 100\% \quad (3-2)
\]

where:

\( N_{\theta_g \geq \theta_{g,S=100\%}} \) = number of measurement points during the duration of the six year monitoring period when the measured water content is greater than or equal to the water content at full saturation.

\( N_T \) = total number of measurement points for TDR sensors in each base, subbase, and subgrade layers in six different pavement sections during the entire monitoring period.
The calculated average degree of saturation and the time percent of full saturation are shown in Tables 3.2 and 3.3 for driving and passing lanes, respectively. According to the results shown in Tables 3.2 and 3.3, base layer materials showed the lowest degree of saturation. This is expected since the materials in this layer are drainable with high hydraulic conductivity. Generally, the degree of saturation at driving lanes was higher than the degree of saturation in the passing lanes. None of the drainable base materials showed the measured water content exceeding the estimated water content at full saturation. This observation also holds true for subbase layers and subgrade bottom layers. The exception to the above observation was noticed at the top elevation of the subgrade layer underneath the ODOT 304 pavement section. In fact, the percent of saturation time for this section for this elevation was found to be 64 percent. The occurrence of full saturation was in the time period noted below: from May 2004 to October 2004, from June 2006 to October 2006, and from May 2007 to October 2008. A lesser severe percent of time (i.e., about 5 to 10 percent) when top elevation of subgrade was in full saturation can be observed in the pavement sections built with ODOT 307-NJ and ODOT 307-IA. For pavement section built with ODOT 307-NJ, full saturation of subgrade at its top elevation occurred in the following time period: from April 2004 to May 2004, and during the month of August 2007. For ODOT 307-IA, full saturation of top elevation of subgrade soils occurred in the following time period: from April 2008 to October 2008. Both cement treated and asphalt treated permeable base materials showed the lowest degree of saturation in the subgrade soils when compared to other pavement sections built with other types of drainable base materials.
<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>Pavement Section</th>
<th>Mean</th>
<th>St.Dev</th>
<th>COV</th>
<th>$\Theta_{g=S=100%}$</th>
<th>Saturatio -n, %</th>
<th>Time percent of fully saturation</th>
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</thead>
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<td>Saturation, %</td>
<td>Time percent of fully saturation</td>
</tr>
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<td>0.25</td>
<td>79.2</td>
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</table>
3.8.3 Moisture Content Variation with Precipitation

Statistical correlation analysis measures the relationship between two data sets that are scaled to be independent of the unit of measurements. In this section, the coefficient of correlation (r) was used to determine the relationship between soil moisture content and the amount of precipitations. The measured precipitation and moisture contents at different TDR locations were summarized on a monthly basis to determine if a correlation could exist between the amount of precipitation and the mean monthly moisture contents at different depths under different pavement sections. Table 3.4 shows the correlation analysis results for the six permeable base sections. It can be seen that, in general, there is relatively low (considering a perfect correlation coefficient to be 1.0) correlation between the amount of monthly rainfall and monthly mean moisture content at different depths. In fact, previous study conducted by Heydinger (2003) also concluded with the same observation.

Table 3.4  Correlation between base, subbase, and subgrade moisture contents and the amount of precipitation

<table>
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<th>Pavement Layer</th>
<th>Section</th>
<th>ODOT 304</th>
<th>ODOT 307-NJ</th>
<th>ODOT 307-IA</th>
<th>ODOT 307-CE</th>
<th>Asphalt Treated</th>
<th>Cement Treated</th>
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<td>D</td>
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<td>D</td>
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3.9 Summary and Conclusions

In this chapter, field measured moisture data for the regime underneath asphalt concrete layers of the flexible pavement sections built with different drainable base materials were analyzed to investigate the effect of drainable base material types on the subsurface moisture regimes. Base materials considered in this monitoring program consisted of both bound and unbound base courses. Unbound base materials included ODOT 304, ODOT 307NJ, 307IA, and 307CE. Bound base materials included asphalt treated, and cement treated base materials. Moisture contents were gathered by means of TDR probes embedded underneath the asphalt concrete layers up to about 6 ft deep. Due to rain storm occurred immediately after TDR probes installation and placing the asphalt concrete layers, an initial sharp jump in the gravimetric moisture contents at the base, subbase and some of subgrade layers was observed in some of pavement sections. In other pavement sections where no precipitation occurred prior to completion of placing asphalt concrete layers, there was no noticeable sharp jump of measured water content underneath the concrete.

Pavement sections built with cement treated and asphalt treated permeable base materials showed the lowest variation in the gravimetric moisture contents, especially at the top of the subgrade layer. From the consideration of minimizing the amount of moisture migrating toward the cohesive subgrade soils, then these two types of drainable base materials seemed to provide the best solution compared to other unbound base materials used in this study. Furthermore, using COV values at the top elevation of the subgrade soils as an evaluation index, the effectiveness of different drainable base materials to drain the infiltrated water can be ranked from the best to worst as follows:

Of particular interest to note here is that the moisture variation in the unbound drainable base layer was higher than that in the bound drainable base materials. After spring 2007, both ODOT 304 and ODOT 307-IA pavement sections showed an increasing trend in their measured moisture content at the top elevation of the subgrade layer. It seems that it is prudent to avoid using ODOT 304 and ODOT 307-IA as drainable base materials for poor performance in preventing moisture from migrating toward cohesive subgrade soils, particularly at the top elevation of the subgrade layer where pumping and fine migration toward subbase could be more pronounced.

Contrary to other field monitoring programs cited in the literature, there was no evidence of full saturation in base and subbase materials for the entire monitoring period. In some cases, particularly in ODOT 304 pavement section, subgrade soils at the top elevation of the subgrade layer experienced full saturation for 64 percent of the monitoring duration. Underneath passing lanes, subgrade layers at both ODOT 307-NJ and ODOT 307-IA sections experienced full saturation for about 5 and 10 percent of the monitoring duration. Both cement treated and asphalt treated permeable base materials showed the lowest degree of saturation among other base materials.

Finally, the notion that water regime will reach an equilibrium state after the pavement has been in service was evaluated in this monitoring project. Equilibrium water content was observed in the pavement sections built with bound base materials; nevertheless, continued fluctuation of water content was observed in the pavement built with the unbound base materials.
There does not seem to exist a simple correlation between monthly measured precipitation and monthly mean of moisture content at each TDR measurement points underneath the pavement. The complexity of the moisture flow in a flexible pavement system seemed to prohibit the development of any meaningful correlations between precipitation variation and moisture regimes.
CHAPTER IV
NUMERICAL INVESTIGATION OF SUBSURFACE DRAINAGE IN
FLEXIBLE PAVEMENT

4.1 Abstract

At the present time, the computer program DRIP has been used as a numerical analysis tool for design of subsurface drainage in a pavement system. The SEEP/W program, however, provides an alternative analysis tool for transient water flow analysis that can take into account of variation of hydraulic conductivity with water content of unbound base materials and complex pavement layers, geometry and dimension of a pavement section, and the use of subsurface drainage pipes. This chapter presents the results of a comprehensive parametric study of representative pavement subsurface drainage system using two-dimensional SEEP/W program. The parameters studied include the base hydraulic conductivity, subbase hydraulic conductivity, base layer thickness, pavement cross-slope, shoulder slope, drainage length, base water content at 100% saturation, base D60 gradation, and the location of edge drain. A statistically derived regression equation together with design charts are presented for estimating time to drain 50% of free water in a pavement system. Observations based on parametric analysis results and comparisons between DRIP analysis and SEEP/W analysis are provided for facilitate effective subsurface drainage design. Although saturated hydraulic conductivity of base
material is an important factor, other factors such as Soil-Water Characteristic Curve (SWCC) variation of hydraulic conductivity with water content (degree of saturation), pavement geometry and dimensions, use of subsurface drainage pipes are also important in controlling the effectiveness of a pavement subsurface drainage system.

4.2 Introduction

It has been recognized by experts such as Cedergren (1994) that premature structural and functional pavement failures can often be related to the length of time that excess moisture remains in the pavement system. The longer the duration is, the more pronounced water-induced pavement damage could be. Water-related problems in flexible pavement include reduction in strength of unbound base and subgrade, stripping of asphalt pavement, shrinkage and swelling of subgrade materials, frost heave, and thaw induced subgrade weakening. Thus, providing adequate subsurface drainage in a pavement system to remove the infiltrated moisture in a timely manner is an important design consideration. This is also supported by numerous researches on the importance of providing and maintaining positive drainage system within a pavement system. For example, Frosyth et al. (1987) stated that if the excess infiltrated water can be drained quickly, the pavement life can be extended by 33 percent for the flexible pavement system and 50 percent for the rigid pavement system. Cedergren (1987, 1989) reported that the pavement system with a good drainage system could last two to three times longer than those without a drainage system. Christopher and McGuffey (1997) estimated that free excess water can lead to reduction of the life expectancy of a pavement system by more than half. The importance of designing an effective drainage system in a
pavement was fully acknowledged by AASHTO Guide for Design of Pavement Structure (AASHTO, 1998) by incorporating the drainage factors in their design. The newly developed Mechanistic-Empirical Pavement Design Guide (MEPDG, 2004) accounts for drainage through the use of the FHWA microcomputer program (DRIP, 2002).

Most of the current drainage criteria have been developed on the basis of describing water flow in saturated conditions (Ariza and Birgisson, 2001). In reality, this could happen in a very limited circumstance and time duration. Most of the time, the water flow near the pavement surface occurs under unsaturated (or partially saturated) conditions where the hydraulic conductivity of the soils is not a constant value, but a function of the soil water content (Stormont and Zhou, 2005). Recently, Liang (2007) reported that most of the time the monitored flexible pavement sections had water contents less than the ones corresponding to the full saturation. According to Philip (1969), the amount of the infiltrated water is a function of materials permeability, gravity forces, and soil matric suction. In an unsaturated soil, the hydraulic conductivity is a function of water content (or soil suction). The higher the volumetric water content is, the higher the hydraulic conductivity becomes. Birgisson and Roberson (2000) articulated that the performance of drainage system can only be understood if unsaturated flow principles were considered. A better understanding of water movement in a flexible pavement system can only be achieved by using seepage analysis that adopts the principles of unsaturated flow.

Transient flow in unsaturated soils is a complex problem since the moisture flow is driven in part by suction gradients, which can result in upward or lateral flow in some cases. The conventional pavement drainage system has been designed based on the concept to intercept saturated flow driven by gravity. Rabab’ah and Liang (2007) used
SEEP/W (GEO-SLOPE, 2004) to model pavement sections that were instrumented to investigate the influences of different types of drainable base materials on moisture regimes in pavement layers and subgrade soils. Comparison between observed and predicted behavior were generally favorable, indicating the capability of SEEP/W with respect to simulating unsaturated, transient, two dimensional flow in the layered pavement system. They pointed out that the soil water characteristic curves (SWCC) and the moisture content dependent hydraulic conductivity functions along with the layering and geometry of the pavement, dictate the moisture regime in a pavement system.

In the past, the design of sub-surface drainage layer calls for a drainable base material with a minimum permeability of 0.35 cm/sec, a nominal thickness of 100 mm (4 inches), and a longitudinal grade of 2 % to meet the time-to-drain requirement (Stormont and Zhou, 2001). In this paper, SEEP/W is used as a numerical analysis tool to simulate the water flow through a pavement system. The objectives of this chapter include the following: (1) performing a parametric study to evaluate the effect of different design parameters of a flexible pavement system on the performance of sub-surface drainage, (2) developing a new predictive equation to estimate the time required to drain 50 percent of water from the drainage layer based on parametric SEEP/W analysis results, 3) providing general guidelines for designing effective sub-surface drainage in a flexible pavement.

4.3 SEEP/W-Groundwater Seepage Analysis Program

SEEP/W is a 2-Dimensional, finite element modelling software package for simulating pore-water movement and pore pressure distribution within porous materials, such as soil and rock.
The governing differential equation, Richards’ equation, for 2-dimensional saturated/unsaturated flow is used by SEEP/W.

\[
\frac{\partial}{\partial x} \left( k_x \frac{\partial H}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial H}{\partial y} \right) + Q = \frac{\partial \Theta}{\partial t}
\]  (4-1)

where:

\( h \) = total head

\( k_x \) = hydraulic conductivity in x-direction

\( k_y \) = hydraulic conductivity in y-direction

\( Q \) = applied boundary flux

\( \Theta \) = volumetric water content

\( t \) = time

Rechiard’s equation states that the difference between the flow entering and leaving an element volume at a point in time is equal to the change in the volumetric water content. Under a steady state flow condition, the right part of Richards’ equation would become zero.

4.4 DRIP (Drainage Requirement in Pavement) Computer Program

The computer program, DRIP 2.0 (2002), was developed by the United States Department of Transportation and Federal Highway Administration (FHWA) for the design and analysis of pavement subsurface drainage. The computer code is an implementation of the FHWA’s Highway Subsurface Drainage Manual prepared by Moulton (1980). DRIP 2.0 can be used to perform drainage design for flexible and rigid pavements and retrofit edge drains, to calculate the time-to-drain and depth of flow in the
drainage layer, and to perform design for separator layers, geotextiles, edge drains, and
geocomposite fin drains. The DRIP program is organized into six input modules:
roadway geometry, inflow, permeable base, edge drain, separator layer, and gradation
analysis. It is designed to provide the user with three essential outputs: permeable base
thickness, edge drain outlet spacing, and separator layer-to-subgrade compatibility. In
DRIP program, drainage analysis can be set either as the depth-of-flow method or the
time-to-drain method. However, according to Hassan and White (2001), the
recommended approach for hydraulic design of permeable base layers is the time-to-drain
approach. DRIP utilizes two methods to calculate the time-to-drain in the drainage layer
of a pavement system: Barber and Sawyer method (1952) and Casagrande and Shannon
method (1952). The Barber and Sawyer (1952) equation is given below.

\[
t = T \times \left[ \frac{n_e L R^2}{k H} \right]
\]

(4-2)

where:

\( t = \) time for drainage to reach a percentage of drain, \( U \)

\( n_e = \) effective porosity of the drainage layer

\( L_R = \) resultant length of the drainage path

\( k = \) permeability of the drainage layer

\( H = \) thickness of the drainage layer

\( T = \) time factor, calculated as:

For \( 0.5 \leq U \leq 1.0 \)
\[ T = 0.5S_R - 0.48S_R^2 \log(1 + 2.4/S_R) + 1.15S_R \log \left( \frac{S_R - US_R + 1.2}{(1-U)(S_R + 2.4)} \right) \] (4-3)

For \( 0 \leq U \leq 0.5 \)

\[ T = US_R - 0.48S_R^2 \log(1 + 4.8U/S_R) \] (4-4)

where:

\( U \) = percent drainage (expressed as a fraction).

\( S_R \) = resultant slope of the drainage path

Casagrande and Shannon method (1952) uses the same equation in the above, but the time factor (T) is determined based on the following equations:

For \( U > 0.5 \)

\[ T = \left( 1.2 - 0.4/S_R^{1/3} \right) \left[ S_R - S_R \ln \left( \frac{S_R + 1}{S_R} \right) + S_R \ln \left( \frac{2S_R - 2US_R + 1}{(2-U)(S_R + 1)} \right) \right] \] (4-5)

For \( U > 0.5 \)

\[ T = \left( 1.2 - 0.4/S_R^{1/3} \right) \left[ 2US_R - S_R^2 \ln \left( \frac{S_R + 2U}{S_R} \right) \right] \] (4-6)

To calculate time-to-drain using DRIP software, the following inputs are required: roadway geometry (cross-slope, longitudinal slope, lane width), permeable base thickness, effective porosity of permeable base, and permeable base hydraulic conductivity. The AASHTO design standard based on time-to-drain criterion rates the quality of drainage of a permeable base from “Excellent” to “Poor”. “Excellent” rating is given to the base materials that have the capability to drain 50 percent of the drainable water in less than two hours. “Good”, “Fair”, “Poor” and “Very Poor” ratings are given
to the base materials where the time required to drain 50 percent of drainable water in 1
day, 7 days, 1 month, and not drain, respectively. In the time-to-drain approach, DRIP
makes the following assumptions: the drainable base is fully saturated at the time of
drain, no water will enter the system after it is saturated, and the hydraulic conductivity
of the drainable base is constant.

4.5 Pavement Model Used in Numerical Study

Since it is not possible to conduct simulations that would encompass every
combination of materials, geometries and boundary conditions relevant to all possible
drainage configurations, a typical pavement section similar to that studied by Rabab’ah
and Liang (2007) is adopted as the baseline pavement section in the present study. The
baseline pavement section is shown in Figure 4.1, with a 3.65 m (12 feet) wide driving
lanes and 3.0 m (10 feet) and 1.22 m (4 feet) of shoulders for driving lane and passing
lane, respectively. The pavement surface is sloped at 1.6% in the transverse direction.
The pavement shoulder is sloped at 4%. Both shallow and deep perforated drainage pipes
in gravel trench were used for collecting subsurface water.
4.6 Baseline Model Materials

The baseline geometry model shown in Figure 4.1 includes four types of materials: the asphalt concrete layer, a base layer, a subbase layer, and the subgrade soil. The properties for different base materials are provided in Table 4.1. In the current numerical study, the asphalt concrete layer was not modeled, as the primary attention was on base, subbase, and subgrade soils. Aggregate gradations and the corresponding soil water characteristic curves (SWCC) for six different types of drainable base materials, as measured by Rabab’ah and Liang (2007), are shown in Figure 4.2 and Figure 4.3, respectively. The hydraulic conductivity of each base material and its variation with water content were estimated using Green and Corey (1971) predictive equation. The subbase layer was typical dense graded materials (ODOT 304), while the subgrade soil was silty clay.
Table 4.1  Saturated Hydraulic Conductivity of Different Permeable Base Materials

<table>
<thead>
<tr>
<th>Material</th>
<th>k (cm/s)</th>
<th>k (ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>304 – Fine</td>
<td>0.073</td>
<td>206</td>
</tr>
<tr>
<td>304 – Median</td>
<td>0.5</td>
<td>1417</td>
</tr>
<tr>
<td>304 – Coarse</td>
<td>1.92</td>
<td>5443</td>
</tr>
<tr>
<td>No. 57</td>
<td>9.37</td>
<td>26563</td>
</tr>
<tr>
<td>307 – NJ Fine</td>
<td>0.788</td>
<td>2234</td>
</tr>
<tr>
<td>307 – NJ Median</td>
<td>1.349</td>
<td>3824</td>
</tr>
<tr>
<td>307 – IA Fine</td>
<td>0.308</td>
<td>873</td>
</tr>
<tr>
<td>307 – IA Median</td>
<td>0.803</td>
<td>2277</td>
</tr>
<tr>
<td>307 – CE Fine</td>
<td>0.937</td>
<td>2654</td>
</tr>
<tr>
<td>307 – CE Median</td>
<td>1.307</td>
<td>3703</td>
</tr>
<tr>
<td>Cement Stabilized</td>
<td>8.94</td>
<td>25345</td>
</tr>
<tr>
<td>Asphalt Stabilized</td>
<td>8.84</td>
<td>25061</td>
</tr>
<tr>
<td>A-6a Subgrade</td>
<td>5.3 x 10^-8</td>
<td>1.5 x10^-4</td>
</tr>
</tbody>
</table>

Figure 4.2  Gradation curves for ODOT 307, ODOT 304 and No. 57 base course materials
4.7 SEEP/W Model Validation

The validation of the SEEP/W numerical model was previously established by Rabab’ah and Liang (2007). In their validation study, the pertinent materials properties were determined from laboratory test results while the precipitation data was those obtained from field weather station. In their study, the SEEP/W model was shown to be capable of predicting the moisture content variations within the pavement system with fairly acceptable level of accuracy compared to field moisture data measured by TDR. It should be pointed out that the SEEP/W model by Rabab’ah and Liang was calibrated by following three calibration steps. First, they used Fredlund and Xing (1994) empirical SWCC equation to generate the SWCC according to dry density data reported for the monitored pavement sections (Liang, 2007). Second, they assumed the air entry values according to Fredlund and Xing (1994) empirical equation. Finally, they made
assumption about percentage of measured precipitation as influx in the numerical simulation. Nevertheless, the capability of SEEP/W to model transient water flow conditions for a long period of time was established.

4.8 Current SEEP/W Model

Figure 4.4 shows the finite element mesh representation of the baseline pavement section given in Fig. 1, in which 8-node quadrilateral elements were used for representing each layer of the pavement, except in the deep subgrade zone where triangular elements were used. At the bottom of the subgrade soil and along the side of the pavement structure, the infinite elements were assigned. Infinite element is used to represent the soil which extends to infinity without actually extending the mesh (SEEP/W, 2004). The boundaries around the drainage pipes were modeled by assigning the total pressure head equal to zero.
Figure 4.4 Pavement Finite Element Mesh and Boundary Conditions of the model pavement (a) entire cross-section (b) detailed mesh near subsurface drain pipes

4.9 Parametric Study using SEEP/W Program

The purpose of the parametric study using SEEP/W program is to identify the effects of different design parameters on pavement subsurface drainage performance under unsaturated conditions. Among those parameters investigated are permeable base hydraulic conductivity, base layer thickness, subbase hydraulic conductivity, use of shallow or deep underdrain pipes, pavement cross-slope, shoulder slope, drainage length, the water content of drainable base materials at 100% saturation, and D60 of the
drainable base materials. The range of values for these parameters in the parametric study is summarized in Table 4.2. The initial water content in the drianble base was assigned to correspond to full saturation so as to match the initial condition assumed in DRIP analysis. Then, transient flow analysis was conducted to estimate the time required to drain free water to reach 50% saturation. In assessing the influence of each design parameter, time-to-drain 50 percent of the water is used as an evaluation criterion.

### Table 4.2 Input parameters of interest to be used in the parametric analysis

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Baseline values</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>base hydraulic conductivity, cm/s</td>
<td>1.35</td>
<td>0.01 - 10</td>
</tr>
<tr>
<td>base layer thickness, m</td>
<td>0.1</td>
<td>0.1 – 0.3</td>
</tr>
<tr>
<td>subbase hydraulic conductivity, cm/s</td>
<td>0.5</td>
<td>0.000001 – 10</td>
</tr>
<tr>
<td>Location of underdrain pipes</td>
<td>Both shallow and deep underdrain pipes</td>
<td>Shallow or deep</td>
</tr>
<tr>
<td>Base coarse cross-slope, %</td>
<td>1.6</td>
<td>1 - 8</td>
</tr>
<tr>
<td>pavement shoulder-slope, %</td>
<td>4</td>
<td>2 - 8</td>
</tr>
<tr>
<td>drainage length, m</td>
<td>8</td>
<td>2 – 18</td>
</tr>
<tr>
<td>Drainable base volumetric moisture contents at fully saturation, %</td>
<td>25</td>
<td>15 - 45</td>
</tr>
<tr>
<td>D_60 of the base materials, cm</td>
<td>1.143</td>
<td>0.2 – 1.8</td>
</tr>
</tbody>
</table>

In conducting parametric FEM simulation study, individual parameter was varied within the range shown in Table 4.2 while the other parameters were kept at the baseline values. The SEEP/W analysis results are presented in Figure 4.5 to Figure 4.14.

Figure 4.5 shows the effects of the permeability at full saturation of base materials on the time to drain. As expected, the analysis results show that the higher the saturated hydraulic conductivity, the faster the drainage. Figure 4.6 shows the effect of varying drainable base layer thickness while keeping other design parameters constant at the
baseline values. A linear relationship can be observed. Figure 4.7 presents the effects of subbase layer permeability, while keeping other design parameters at the baseline values. It shows that saturated hydraulic conductivity of subbase layer also played a role in controlling time to drain. Figure 4.8 shows the relationship between the time to drain and the variation of the cross slope of the base course, while keeping other design parameters constant. It seems that the cross slope greater than 6% would not produce significant benefits in the drainage efficiency.

Figure 4.9 presents the relationship between the time to drain and the shoulder slope. The relationship may be characterized as decaying exponential function; nevertheless, as expected, the steeper the shoulder slope, the faster the drainage.

The influence of drainage length on time to drain is presented in Figure 4.10. The drainage length is defined as the longest distance traveled by the water to reach the edge drain. It is computed based on the width, longitudinal slope and transverse slope of the base layer as given in Equation 4-7. Figure 4.10 shows the computational results of SEEP/W for different drainage length, while keeping other parameters constant.

\[
L = W \sqrt{1 + \left(\frac{S}{S_x}\right)^2}
\]

(4-7)

where:

\( L \) = drainage length

\( W \) = base layer width

\( S \) = longitudinal slope

\( S_x \) = transverse slope
Figure 4.11 shows the effects of the volumetric water content of drainable base materials at full saturation ($\Theta_{v@S=100\%}$) on pavement drainage performance. The initial value of the water content corresponding to 100 percent saturation of the drainable base does not seem to play a major role in influencing the drainage time.

D60 was considered to be an important base material gradation parameter that controls the shape of the soil water characteristic curves (SWCC). Fredlund and Xing (1994) equation was used to generate several different SWCCs based on different D60 values. Figure 4.12 shows that time to drain decreases with an increase in D60 values.

The amount of the fines in the drainable base materials (i.e., percent passing 200 sieve) was an important material gradation parameter. The water storage capacity of the soil is greatly affected by the amount of fines in the soil. Using Fredlund and Xing (1994) equation, different SWCCs were generated for different percentages passing 200 sieve. Figure 4.13 shows the amount of fines played an important role in controlling drainage performance.

The effects of using shallow and deep underdrain pipes were studied for three cases: Case 1 represents the case of using both shallow and deep underdrain pipes, Case 2 represents the case of using just the shallow underdrain pipes, and Case 3 represents the case using only deep underdrain pipes. The shallow underdrain pipes were used to collect the water from the base layer, while the deep underdrain pipes were used to collect the water from the subbase layer. Figure 4.14 shows that shallow underdrain pipes provide more advantages in draining water out of the pavement system than do the deep underdrains.
Figure 4.5 Effect of hydraulic conductivity of permeable base materials on drainage performance

Figure 4.6 Effect of permeable base layer thickness on pavement drainage performance
Figure 4.7 Effect of hydraulic conductivity of subbase materials on pavement drainage performance

Figure 4.8 Effect of pavement cross-slope on pavement drainage performance
Figure 4.9 Effect of pavement shoulder slope on pavement drainage performance

Figure 4.10 Effect of drainage length on pavement drainage performance
Figure 4.11 Effect of permeable base water content at 100 percent saturation on pavement drainage performance

Figure 4.12 Effect of permeable base D_{60} gradation on pavement drainage performance
Figure 4.13 Effect of permeable base fine content on pavement drainage performance

Figure 4.14 Effect of shallow/deep edge drains on pavement drainage performance
4.10 Development of Predictive Equation of $T_{50}$ (Time-to-Drain 50 %)

The current equations for predicting $T_{50}$ were developed by Barber and Sawayer (1952) and Casagrade and Shannon (1952) with a basic assumption that base layer remains fully saturated during drainage. Furthermore, it is essentially a one-dimensional seepage analysis. The numerical parametric study results presented in the previous section, together with numerous random runs with the parameters varied randomly, were used for statistical regression analysis to develop a new predictive equation to relate the time-to-drain 50 percent of the infiltrated water with those pertinent parameters.

The relative importance of each parameter is calculated and presented in Table 4.3. The order of importance of the investigated parameters affecting $T_{50}$ can be ranked from high to low as follows: k, D60, ks, H, S, and L (notations defined in Equation 4-8). For the purpose of nonlinear regression, $T_{50}$ was calculated according to Equation 4-8.

$$T_{50} = \psi_1(K)\psi_2(H)\psi_3(S)\psi_4(L)\psi_5(D_{60})\psi_6(Ks)$$

(4-8)

where:

- $T_{50}$ = time-to-drain 50 percent of the infiltrated water
- $L$ = drainage Length, m
- $S$ = pavement cross-slope
- $k$ = permeable base hydraulic conductivity, cm/ses
- $H$ = permeable base layer thickness, m
- $D_{60}$ = sieve opining size where 60 percent of the aggregate pass through, mm
- $ks$ = subbase hydraulic conductivity, cm/sec
The solution obtained from the regression analysis is presented for convenience in design charts in Figure 4.15. These charts can be used together to find $T_{50}$ in accordance with the following equation:

$$T_{50} = 1.041 \psi(K, H) \psi(Ks, S) \psi(D_{60}, L)$$

(4-9)

R-squared was found to be 0.93 for the regression line between the empirically predicted $T_{50}$ using Equation 4-9 and the FEM calculated $T_{50}$ as shown in Figure 4.16. This R-squared value is considered excellent.
Figure 4.15 Design charts for estimating $T_{50}$

\[ T_{50} = z \left[ (a+bH)K_e K_s D^{d} S^{e} L^{f} D_{60}^{g} \right] \]

<table>
<thead>
<tr>
<th>Regression Constant</th>
<th>Constant Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>1.496</td>
</tr>
<tr>
<td>b</td>
<td>-4.647</td>
</tr>
<tr>
<td>c</td>
<td>-0.335</td>
</tr>
<tr>
<td>d</td>
<td>-0.079</td>
</tr>
<tr>
<td>e</td>
<td>-0.398</td>
</tr>
<tr>
<td>f</td>
<td>0.284</td>
</tr>
<tr>
<td>g</td>
<td>-0.526</td>
</tr>
<tr>
<td>z</td>
<td>1.041</td>
</tr>
</tbody>
</table>
Figure 4.16  SEEP/W calculated time-to-drain versus predicted time-to-drain

Table 4.3 Sensitivity analysis of $T_{50}$

| Parameter | $T_{50\text{max}}$ | $T_{50\text{min}}$ | $|T_{50\text{max}} - T_{50\text{min}}|$ | $\frac{|T_{50\text{max}} - T_{50\text{min}}|}{\sum (T_{50\text{max}} - T_{50\text{min}})} \times 100\%$ |
|-----------|------------------|------------------|-------------------------------|-------------------------------------------------|
| K         | 4.6              | 0.125            | 4.475                         | 38.21                                           |
| H         | 0.831            | 0.059            | 0.831                         | 7.10                                            |
| S         | 1.17             | 0.5              | 0.67                          | 3.58                                            |
| L         | 0.99             | 0.64             | 0.35                          | 2.99                                            |
| D60       | 3.92             | 0.003            | 3.917                         | 33.44                                           |
| Ks        | 1.97             | 0.57             | 1.4                           | 11.95                                           |
| $\sum$    |                  |                  | 11.712                        | 100                                             |
4.11 Comparison between DRIP and SEEP/W

In this section, analysis results in terms of $T_{50}$ using the DRIP program and the SEEP/W program for the assigned pavement geometry and material properties are compared. For meaningful comparison, the initial water content of base, subbase and subgrade layers were assigned to correspond to full saturation.

The analysis results for the typical pavement section with baseline material properties using the DRIP 2.0 program and SEEP/W program are given in Table 4.4. Generally, for unbound base materials, time-to-drain predicted by SEEP/W under unsaturated condition ($T_{50}$) is higher than the time-to-drain predicted by DRIP under fully saturated conditions. In unsaturated flow analysis, as the degree of saturation become less than 100 percent during drainage process, there is more ability of the soil to store water. This is particularly true for base materials that contain more fines. As a result, more time is needed to drain the water out of the fine graded base materials. For median and coarse graded materials, the difference of time-to-drain between saturated and unsaturated flow analysis becomes less pronounced.

4.12 Effective Sub-surface Drainage Design

Based on the results of the numerical parametric study using SEEP/W program, the following comments may be made for effective sub-surface drainage in flexible asphalt pavements.

- Preferably, SEEP/W program should be used as an analysis tool to evaluate the pavement subsurface drainage design. The use of DRIP program to estimate time to drain could be unconservative, since it uses the one-dimensional water flow in
a fully saturated condition. A more representative condition for estimating time to 
drain should be transient flow in two dimension, where hydraulic conductivity 
varies as a function of water content (or degree of saturation) of the base materials.

- Specifying a minimum hydraulic conductivity value for the base materials as a 
  sole controlling factor would not be sufficient to ensure adequate drainage. 
  Factors such as soil-water characteristic curve (SWCC) and hydraulic 
  conductivity as a function of water content, as well as pavement geometry (width 
  of the road, drainage distance to underdrain pipes, longitudinal and transverse 
  slopes of base and shoulder, and permeable layer thickness) are also important.

- Specifying 100 mm (4 inches) of unbound drainable base materials for 
  constructability is generally adequate, as increase of the thickness of drainable 
  base layer does not seem to provide much more benefit in promoting drainage.

- The hydraulic conductivity of the subbase material needs to be selected optimally 
  to ensure that water would flow predominantly laterally in the base layer, rather 
  than going downward into subbase.

- The fine content (passing 200 sieve) of base materials preferably should be less 
  than or equal to 2 percent for good pavement drainage.

- Shallow underdrain pipes appeared to facilitate subsurface drainage better than 
  the deep underdrain pipes.
<table>
<thead>
<tr>
<th>Materials</th>
<th>Time-to-drain-(50% saturation criteria)-hours</th>
<th>Unsaturated Time-to-drain-(50% saturation criteria)-hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>304-Fine</td>
<td>2.75-Good</td>
<td>5 days- Fair</td>
</tr>
<tr>
<td>304-Median</td>
<td>0.59-Excellent</td>
<td>1.35- Excellent</td>
</tr>
<tr>
<td>304-Coarse</td>
<td>0.17-Excellent</td>
<td>0.55- Excellent</td>
</tr>
<tr>
<td>307 – NJ Fine</td>
<td>0.75-Excellent</td>
<td>10.04-Good</td>
</tr>
<tr>
<td>307 – NJ Median</td>
<td>0.47-Excellent</td>
<td>0.85- Excellent</td>
</tr>
<tr>
<td>307 – IA Fine</td>
<td>1.58-Excellent</td>
<td>7.1-Good</td>
</tr>
<tr>
<td>307 – IA Median</td>
<td>0.6-Excellent</td>
<td>1.20- Excellent</td>
</tr>
<tr>
<td>307 – CE Fine</td>
<td>0.72-Excellent</td>
<td>4.6-Good</td>
</tr>
<tr>
<td>307 – CE Median</td>
<td>0.53-Excellent</td>
<td>0.57- Excellent</td>
</tr>
<tr>
<td>No. 57</td>
<td>0.12-Excellent</td>
<td>0.05-Excellent</td>
</tr>
<tr>
<td>Cement Stabilized</td>
<td>0.14-Excellent</td>
<td>0.06-Excellent</td>
</tr>
<tr>
<td>Asphalt Stabilized</td>
<td>0.14-Excellent</td>
<td>0.06-Excellent</td>
</tr>
</tbody>
</table>

4.13 Summary and Conclusions

In this chapter, a transient analysis using SEEP/W program was conducted to simulate water flow in a flexible pavement system. The soil water characteristic curves (SWCC) for several typical base, subbase, and subgrade soils, together with Fredlund and Xing (1994) empirical equation for hydraulic conductivity functions, were determined as input in the SEEP/W parametric studies.

The results of the transient SEEP/W numerical simulations revealed that specifying a minimum hydraulic conductivity value for the base materials as a sole controlling factor would not be sufficient to ensure effective subsurface drainage. Factors such as soil water characteristic curve (SWCC) and hydraulic conductivity variations with degree of saturation, pavement geometry (width of the road, distance between the edge of the
pavement to the edge drain, longitudinal and transverse slopes, and permeable layer thickness) are also important.

The numerical study has revealed some important observations that can help designers to achieve effective underdrain system for a flexible pavement system. The effect of the base layer thickness on the time-to-drain was found insignificant. As a result, 100 mm (4 inches) for constructability was adequate. A subbase hydraulic conductivity of 0.01 cm/sec is considered to be sufficient enough to increase the efficiency of the base layer to drain the infiltrated water laterally by reducing the amount of water infiltrated downward. The typical values of the cross-slope and shoulder slope (1.6 % and 4 %) were found to be effective in producing a good under-drain system under unsaturated conditions. The effect of the shallow edge drains on the drainage performance was found more pronounced than the effect of the deep edge drains. However, the deep edge drains should be used to take care of the infiltrated water to the subbase layer and to reduce the chance of the infiltrated water to reach the subgrade layer. As expected, it was found that the higher the degree of saturation of the drainable base materials, the faster the drainage. The amount of fines in the permeable base materials is critical in controlling the time-to-drain. The numerical study results suggest limiting the amount of fines (passing 200 sieve) in the drainable base materials to be within 2 percent by weight.

Generally, for unbound base materials, time-to-drain predicted by SEEP/W under unsaturated condition ($T_{50}$) is higher than the time-to-drain predicted by DRIP under fully saturated conditions. For practical use, a new predictive equation along with design charts for estimating the time required to drain 50% saturation were developed by utilizing the results of SEEP/W analysis. $T_{50}$ is calculated as product of multiple empirical functions.
of the following parameters \( K, D_{60}, K_s, H, S, \) and \( L \). The predictive equation could be used to design a flexible pavement system for effective subsurface drainage.
CHAPTER V

PREDICTING STRESS AND MOISTURE DEPENDENT RESILIENT MODULUS OF COHESIVE SOILS

5.1 Abstract

Resilient modulus is an important factor in both flexible and rigid pavement design and analysis. It has widely been recognized by the pavement agencies as a good property that describes the stress-dependent elastic modulus of different soil materials. Subgrade resilient modulus is shown to be affected by the in-situ state of stresses and moisture contents. Several models have been developed to predict the subgrade resilient modulus. However, most of these models are stress dependent. In this chapter, a new predictive model is proposed to characterize the resilient modulus of cohesive soils using both octahedral stresses and soil suction concepts. To validate the accuracy of the new proposed model, resilient modulus conducted in the laboratory and gathered from the literature were used. Validation process revealed that the proposed model performs well and provides advantages over the other models in which it provides better prediction accuracy as well as reducing the number of soil specimens needed to determine the regression coefficient in the model. A comprehensive statistical analysis was carried out to relate the proposed model k-coefficients with the physical properties of different subgrade soils. Statistical analysis indicated that the correlations between the model
parameters and the soil properties yield best results. The validity of the developed k-coefficient models to predict the resilient modulus was examined. A good agreement between the measured and the predicted resilient modulus was observed.

5.2 Introduction and Background

The resilient modulus of cohesive soils has widely been adopted in the Mechanistic-Empirical Design Approach (NCHRP 2004) as an important material parameter for design and analysis of flexible pavements. There are three approaches for determining the resilient modulus of cohesive soils: (1) conducting Repeated Triaxial Load (RTL) laboratory tests on either undisturbed or re-constituted soil samples; (2) back-calculation of modulus by conducting in-situ non-destructive tests such as falling weight deflectometer test; and (3) empirical prediction using correlations with soil physical properties. The former two approaches are very complex, requiring not only highly trained personnel to conduct the tasks but also substantial amount of time and resources in carrying out the data interpretations. The third approach is a much simpler one as long as it can adequately capture the effects of influencing factors such as state of stress, moisture content, and basic physical properties.

From numerous literature, such as most recent ones by Lekarp et al. (2000) and George (2004), it has been well established that resilient modulus of cohesive soils depends upon factors such as stress state, moisture, and physical properties (e.g., density, gradation, mineralogy, morphology, of soil particles). Among the many early model development work for incorporating stress effects on resilient modulus of cohesive soils, Seed et al. (1967) proposed that the resilient modulus can be linearly related to the logarithmic value
of bulk stress, while Thompson and Robnett (1976) proposed a bilinear function of the deviatoric stress model. In more recent research, Moossazadeh and Witczak (1981) proposed a linear relationship with the logarithmic value of deviator stress. AASHTO (1993) design approach recommends the use of bulk stress and deviatoric stress to predict resilient modulus of sands and clays, respectively.

Recognizing that the bulk stress model could not account for shear stresses and shear strains developed in subgrade layers in pavement due to traffic loading and that the deviator stress model could not capture the influence of confining pressure on resilient modulus accurately, several noted researchers (e.g., Witczak and Uzen 1988, Mohammad et al. 1995, Mohammad et al. 1999, and Andrei 1999) proposed to combine both bulk and deviatoric stresses in the resilient modulus predictive equations. The underlying supporting rationales for their approach were observations that the resilient modulus of the cohesive soils increases as the confining pressure increases and decreases as the deviator stress increases. Recently, the Mechanistic-Empirical pavement design approach (MEPDG, 2004) adopted a “generalized constitutive model”, similar to the one proposed by Witczak and Uzen (1988), to predict the resilient modulus of cohesive soils. The generalized resilient constitutive model is presented as shown in Equation 5-1:

\[
\frac{M_R}{p_a} = k_1 \left( \frac{\theta}{p_a} \right)^{k_2} \left( \frac{\tau_{oct}}{p_a} + 1 \right)^{k_3}
\]  

(5-1)

where \(M_R\) = resilient modulus; \(\theta\) = bulk stress = \(\sigma_1 + \sigma_2 + \sigma_3\); \(\sigma_1\) = major principal stress; \(\sigma_2\) = intermediate principal stress; \(\sigma_3\) = minor principal stress; \(\tau_{oct}\) = octahedral shear stress = \(\sqrt{2/3} (\sigma_1 - \sigma_3)\); Pa = atmospheric pressure (101.325 kPa); and \(k_1, k_2, k_3\) are regression constants.
The effects of moisture content on resilient modulus of cohesive soils has been addressed by many noted researchers in the field, such as Mohammad et al. (1995), Romero and Pamukcu (1996), Fredlund et al. (1977), Yuan and Nazarian (2003), Wolfe and Butalia (2004). The general trend is that the resilient modulus of cohesive soils decreases as the moisture content increases. In the MEPDG, the effect of moisture on resilient modulus was addressed by referencing to resilient modulus of soils at optimum water content. Recently, the framework of unsaturated soil mechanics has been adopted by Yang et al. (2005) and Liang et al. (2008) in developing resilient modulus model for soils with different moisture content. Yang et al. (2005) proposed a predictive resilient modulus model using both deviatoric stresses and matric suction state variables. The model is presented in Equation 5-2:

\[ M_R = k_1\left(\sigma_d + \chi_w \psi_m\right)^{k_2} \] (5-2)

where \( \sigma_d \) is the deviator stress (\( \sigma_1 - \sigma_3 \)), \( \chi_w \) is the Bishop’s effective stress parameter (\( \chi_w = 0 \) for dry soils, \( \chi_w = 1 \) for saturated soils), \( \psi_m \) is the matric suction, and \( k_1, k_2 \) are the regression coefficients.

Liang et al. (2008) modified the generalized resilient modulus predicative model adopted by MEPDG by incorporating the soil matric suction concept. The modification has been made by adding the matric soil suction to the bulk stress component as presented in Equation 5-3:

\[ M_R = K_1 P_d \left(\frac{\theta + \chi_w \psi_m}{P_a}\right)^{K_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{K_3} \] (5-3)
where $\chi_w$ is the Bishop’s effective stress parameter which can be approximated by the equation proposed by Khalili and Khabbaz (1998), and $k_1$, $k_2$, $k_3$ are regression constants.

Liang et al. model has been re-evaluated based on new data for A-4 and A-6 soils obtained in this study and found to over predict the resilient modulus of cohesive soils. This was contributed to the fact that adding the matric suction component to the bulk stress term, which in turn will increase the confining pressure and result in an increase of the predicted resilient modulus.

In this chapter, both the octahedral normal stress ($\sigma^{\text{oct}} = (\sigma_1 + \sigma_2 + \sigma_3)/3$), and octahedral shear stress ($\tau^{\text{oct}} = \sqrt{2}/3 (\sigma_1 - \sigma_3)$), will be used in a new predictive model, taking into account of the argument that they represent the best of actual stresses imposed by the traffic loading (Houston et al. 1992, Mohammad et al. 1999, Wolfe and Butali, 2004). In addition, soil suction concept will be incorporated in the new model. Therefore, the primary objectives of this paper are twofold: (a) to develop a new predictive model that can overcome the shortcoming experienced by previous models by utilizing the concept of soil suction and by adopting both octahedral normal states and octahedral shear stress in the prediction model, and (b) to develop empirical relationships between the basic soil physical properties and the regression constants in the model. Ultimately, by achieving the above two objectives, a more versatile and accurate semi-empirical approach for estimating resilient modulus of cohesive soils will be available for practical use.
5.3 Resilient Modulus Data

For generating high quality resilient modulus data of cohesive soils, the Material Testing System (MTS) Closed-Loop servo Hydraulic System, Model 810 was used for repeated load triaxial test following AASHTO T307 procedure. The test samples were contained in Shelby tubes retrieved from the field. Several ASTM standard tests were conducted to determine the index and physical property tests that were performed on the subgrade soils include: soil particle-size analysis (ASTM D 422-90), soil classifications (ASTM D 2487-93), and soils Atterberg Limits (ASTM D 4318-84). The material index and physical properties and the soil resilient modulus values are shown in Table 5.1 and 5-2, respectively. For each material type and moisture content, three samples were prepared and tested for the mean resilient modulus values as presented in Table 5.2. The variation in resilient modulus results with octahedral normal stresses, octahedral shear stresses and moisture content are shown in Figures 5.1 and 5.2 for A-4 and A-6 subgrade soils, respectively. From these figures, it can be observed that an increase in moisture content and octahedral shear stress will decrease the resilient modulus. The effect of moisture content on resilient modulus depends on the level of the deviatoric stresses. When the deviatoric stress increases the effect of moisture content on subgrade cohesive soils decreases.
Table 5.1 Summary of the particle-size analysis, Atterberg’s limits, and compaction properties for subgrade materials (after Khasawneh, 2005)

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>% Passing for Sieve No.</th>
<th>Atterberg’s Limits</th>
<th>Max Dry Density, pcf</th>
<th>Optimum Water Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. 4</td>
<td>No. 10</td>
<td>No. 40</td>
<td>No. 100</td>
</tr>
<tr>
<td>A-4</td>
<td>94.7</td>
<td>88.2</td>
<td>75.7</td>
<td>65.5</td>
</tr>
<tr>
<td>A-6</td>
<td>94.1</td>
<td>88.3</td>
<td>75.1</td>
<td>68.8</td>
</tr>
</tbody>
</table>

Table 5.2 Resilient Modulus Results in ksi of subgrade materials (after Khasawneh, 2005)

<table>
<thead>
<tr>
<th>Stress (ksi)</th>
<th>Material (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>σ₃, psi</td>
<td>A-4a @ OMC</td>
</tr>
<tr>
<td>6.05</td>
<td>2.04</td>
</tr>
<tr>
<td>6.05</td>
<td>4.08</td>
</tr>
<tr>
<td>6.05</td>
<td>6.18</td>
</tr>
<tr>
<td>6.05</td>
<td>8.11</td>
</tr>
<tr>
<td>6.05</td>
<td>9.98</td>
</tr>
<tr>
<td>4.01</td>
<td>2.10</td>
</tr>
<tr>
<td>4.01</td>
<td>4.03</td>
</tr>
<tr>
<td>4.01</td>
<td>6.14</td>
</tr>
<tr>
<td>4.01</td>
<td>8.11</td>
</tr>
<tr>
<td>4.01</td>
<td>10.05</td>
</tr>
<tr>
<td>2.01</td>
<td>2.03</td>
</tr>
<tr>
<td>2.01</td>
<td>4.05</td>
</tr>
<tr>
<td>2.01</td>
<td>6.05</td>
</tr>
<tr>
<td>2.01</td>
<td>8.03</td>
</tr>
<tr>
<td>2.01</td>
<td>10.03</td>
</tr>
</tbody>
</table>
Figure 5.1  Effect of moisture content and stress state on resilient modulus for A-4a soil

Figure 5.2  Effect of moisture content and stress state on resilient modulus for A-6a soil
5.4 Soil Suction

The behaviour of unsaturated soils is highly dependent upon soil suction, which in turn is influenced by soil moisture content for a given soil. Soil suction is comprised of two components: matric suction and osmotic suction. In most engineering problems where the matric suction governs the behaviour of unsaturated soils, the osmotic suction is not significant and commonly not determined. Numerous devices and methods can be used to measure the soil suction. Among them, the filter paper method specified in ASTM D 5298-94 (2000) has the following advantages over the others: (1) it is a relatively simple and inexpensive test method that covers the full range of suction, and (2) it is the only method that is capable of measuring the total suction and the matric suction simultaneously (Fredlund and Rahardjo 1993). The filter paper method is based on an assumption that a filter paper will reach equilibrium with respect to moisture flow within a soil. After achieving equilibrium, the water contents of the soil and filter paper are measured and the suction in the soil is calculated using the typical calibration curve shown in Figure 5.3.

The relationship between water content and the associated matric suction can be empirically described by the soil water characteristic curve (SWCC), which is defined as the water storage capacity of a soil at a given soil suction. Fredlund and Xing (1994) proposed an empirical equation for representing the SWCC that was found to be in good agreement with the extended database. In fact, MEPDG adopts Fredlund and Xing (1984) empirical equation. Figure 5.4 shows a typical soil water characteristic curve, displaying the relationship between volumetric water content and soil suction. The air-entry value of the soil is the matric suction at which air starts entering the largest pores in the soil. It can
be determined by drawing a tangent line from the first inflection point. The curve in the low-suction range can be approximated by another line. The air-entry value can be approximated as the ordinate of the point at which the two lines intersect, as illustrated in Figure 5.4. Residual water content is the water content of the soil when a large amount of suction pressure is required to remove the additional water from the initially saturated soil. A consistent way to define the residual water content is also shown in Figure 5.4. The shape of the soil-water characteristic curve is highly dependent on the material type. Fine-grained soils (such as clays) generally have higher matric suction than coarse-grained soils, while loose clays can undergo large volumetric changes as a result of changes in suction (Heath et al. 2004).

![Figure 5.3 Typical Filter Paper Calibration Curve](image)

\[
\psi_t = 5.327 - 0.0779 \psi_{fp}
\]

\[
\psi_t = 2.412 - 0.0135 \psi_{fp}
\]
In this chapter, the SWCCs for A-4 and A-6 soils were developed using the filter paper method using Whatman No. 42 filter paper (ash-free quantitative Type II with a diameter of 5.5 cm). The measured SWCCs, together with the SWCCs predicated by Fredlund and Xing equation, are depicted in Figures 5.5 and 5.6 for A-4 and A-6 soils, respectively. It can be seen that the Fredlund and Xing equation is in good agreement with the experimental data.
Figure 5.5 Measured and predicted matric suction for A-4 soil (after Liang et al., 2008)

Figure 5.6 Measured and predicted matric suction for A-6 soil (after Liang et al., 2008)
5.5 Cohesive Soils Resilient Modulus Proposed Model

The proposed model for predicting the cohesive soil resilient modulus under different moisture contents, takes the form given in Equation 5-4.

\[ M_R = k_1 P_a \left( \frac{\sigma_{oct} + \chi_w \Psi_m}{P_a} \right)^{k_2} \left( \frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \]  

(5-4)

where \( M_R \) is the resilient modulus, \( \sigma_{oct} \) is the octahedral normal stresses = \((\sigma_1 + \sigma_2 + \sigma_3)/3\), \( \tau_{oc} \) is the octahedral shear stresses, \( \Psi_m \) is the matric suction, \( \chi_w \) is the Bishop’s parameter, \( P_a \) is the atmospheric pressure, and \( k_1, k_2, k_3 \) are regression coefficients.

The matric suction in the model can be measured using filter paper method. Regarding Bishop’s parameter (\( \chi_w \)), some efforts have been made to develop the model of \( \chi_w \). Khalili and Khabbaz (1998) found that the relationship between \( \chi_w \) and the martic suction is linear in a log-log space at the suction level above the air entry value. Khalili and Khabbaz (1998) model for \( \chi_w \) is given in Equation 5-5.

\[ \chi_w = \begin{cases} \left( \frac{\Psi_a}{\Psi_m} \right)^{0.55} & \text{When } \Psi_m \geq \Psi_a \\ 1 & \text{When } \Psi_m < \Psi_a \end{cases} \]  

(5-5)

where \( \Psi_a \) is the air entry value (matric suction where air starts to enter the largest pores of soils, and \( \Psi_m \) is the matric Suction.
It is important to note that the model developed by Khalili and Khabbaz (1998) is based on static triaxial tests, while resilient modulus is measured under dynamic loading. Thus, this may raise a question about the accuracy of using $X_w$ in the predictive equation for the cohesive soils. Also, the proposed model assumed that the value of Bishop’s parameter and matric suction are the same regardless of the level of the deviatoric stresses applied. Despite these concerns, later in this paper, the proposed mode was shown to provide good prediction accuracy compared with other predictive models.

5.6 Validation of the Proposed Model

The data set presented previously was used for validating the proposed predictive model. The values of matric suction and Bishop’s parameters at the optimum and at 2% above the optimum moisture contents are presented in Table 5.3. For comparison purpose, both Liang, et al (2008) model and and ME models are also used to predict the resilient modulus. Regression analysis using the SPSS statistical was performed to determine the regression coefficients of the proposed model and they are given in Table 5.4.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Moisture Condition</th>
<th>Water Content, %</th>
<th>Matric Suction, $\Psi_m$ psi</th>
<th>Bishop’s parameter, $X_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-4a</td>
<td>OMC</td>
<td>14.2</td>
<td>55</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td>OMC+2 %</td>
<td>16.2</td>
<td>22</td>
<td>0.48</td>
</tr>
<tr>
<td>A-6a</td>
<td>OMC</td>
<td>16.5</td>
<td>51</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>OMC+2 %</td>
<td>18.5</td>
<td>22</td>
<td>0.65</td>
</tr>
</tbody>
</table>
Table 5.4  Regression coefficient for subgrade resilient modulus models

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Resilient Modulus Model</th>
<th>K₁</th>
<th>K₂</th>
<th>K₃</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-4a</td>
<td>Proposed</td>
<td>762.946</td>
<td>0.592</td>
<td>2.393</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>Liang 2008</td>
<td>640.942</td>
<td>0.508</td>
<td>2.602</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td>ME</td>
<td>915.787</td>
<td>0.150</td>
<td>2.332</td>
<td>0.78</td>
</tr>
<tr>
<td>A-6a</td>
<td>Proposed</td>
<td>88.601</td>
<td>1.186</td>
<td>1.847</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td>Liang 2008</td>
<td>240.116</td>
<td>0.868</td>
<td>2.501</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>ME</td>
<td>474.49</td>
<td>0.149</td>
<td>1.926</td>
<td>0.51</td>
</tr>
</tbody>
</table>

Predictions by the proposed model, the Liang et al. (2008), and ME model are shown in Figure 5.7 and 5.8 for A-4 and A-6 cohesive soils, respectively. From Figure 5.7, it can be seen that the proposed model has the lowest scatter around the 45 degree line with the highest coefficient of determination (R-squared = 0.92). Moreover, the improvement in resilient modulus prediction by using the proposed model is more clearly seen in Figure 5.8. Indeed, the proposed model predicted the subgrade resilient modulus of A-6 soils much better than the other models. A high coefficient of determination (R-squared = 0.97) and less scatter around the 45 degree line were achieved. This excellent performance of the predictive equation could be attributed to the fact that A-6 soils contain more fines (clay materials) than A-4 soils, thus resulting in higher matric suction. Compared with predictions by Liang et al. (2008) and ME model, a significant improvement in the prediction capability can be observed, especially for soils with high amount of clay contents. The proposed model eliminates the tendency of overprediction by Liang et al. (2008) model. The proposed model has an advantage over the MEPDG equation in that it can predict resilient modulus at different stress states and water contents. It should be noted that the regression constants for soils are independent of moisture contents.
To further validate the proposed model, resilient modulus data reported by Mao (1995), Wolfe and Butalia (2004) and Masada and Sargand (2002) were used. Predictions of subgrade resilient modulus by the proposed model are compared with the measured resilient modulus in Figures 5.9, 5.10 and 5.11 for A-7-6, A-6 and A-4 cohesive subgrade soils, respectively. It should be noted that for data without direct measurements of soil suction, the Fredlund and Xing (1994) empirical equation was used to determine the soil water characteristic curve (SWCC) from which the soil suction can be measured at different levels of volumetric water content. From Figures 5.9, 5.10 and 5.11, it can be seen that the proposed model predicts very well of the resilient modulus of cohesive subgrade soils. The coefficients of determination (R-squared) of A-7-6, A-6 and A-4 are 0.90, 0.89, and 0.87, respectively. The accuracy of the proposed model in predicting resilient modulus of cohesive soils could be further improved by conducting tests to determine SWCC, rather than using Fredlund and Xing empirical equation.
Figure 5.7 Predicted versus measured resilient modulus for A-4 soil

Figure 5.8 Predicted versus measured resilient modulus for A-6 soil
Figure 5.9 Comparison between predicted and measured resilient modulus using proposed model for A-7-6 subgrade cohesive soils

Figure 5.10 Comparison between predicted and measured resilient modulus using proposed model for A-6 subgrade cohesive soils
5.7 Regression Models for k-coefficients

In the past, numerous researchers have developed models for relating model regression constants to soil properties, such as the work by Santha (1994), Von Quintus and Killingsworth (1998), Mohammad et al. (1999), Yau and Von Quintus (2002), Dai and Zollars (2002). According to the study by Mohammad et al., (1999), good fit can be achieved only to those specific soil types.

In this section, a comprehensive statistical analysis has been conducted to develop regression models to predict the k-coefficients in the proposed model from basic soil properties. For this purpose, subgrade soil properties obtained from standard laboratory tests by Wolfe and Butalia (2004) were utilized. Table 5.5 presents the physical
properties of A-4, A-6 and A-7-6 AASHTO classified subgrade materials. The SPSS software was employed for obtaining the statistical analysis results. A nonlinear regression has been conducted between the measured resilient modulus and the corresponding values of octahedral normal stress, octahedral shear stress and soil matric suction to get the values of $k_1$, $k_2$ and $k_3$ for each sample. The soil matric suction at a particular water content was obtained from the soil water characteristic curves (SWCC) developed in this study based on Fredlund and Xing (1994) equation. Figures 5.12, 5.13 and 5.14 show the SWCC for A-4, A-6 and A-7-6 subgrade soils, respectively. After obtaining the values of $k_1$, $k_2$ and $k_3$ for each test specimen of a soil type, a second regression analysis was carried out to relate these k-coefficients with the soil physical properties to obtain a set of $k_1$, $k_2$ and $k_3$ prediction models. A stepwise regression analysis was performed to identify the important physical properties that affect the prediction of the resilient modulus coefficients. A set of simple linear models was first generated using the backward and forward stepwise regression methods available in SPSS program. From this set of models, a model which had a higher value of R-squared and a small variance inflation factor (VIF) was selected. VIF needs to be less than 10 to avoid the multicollinearity problems which may cause problems in the estimation (Chatterjee and Price, 1977).
Table 5.5 Material physical properties standard laboratory test results (after Wolfe and Butalia, 2004)

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<th>Soil Type</th>
<th>Sample Name</th>
<th>Sample Condition</th>
<th>PI</th>
<th>LL</th>
<th>MC, %</th>
<th>MCR</th>
<th>DD, pcf</th>
<th>DDR</th>
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Figure 5.12  Soil Water Characteristic Curves (SWCC’s) for A-4 subgrade soils

Figure 5.13  Soil Water Characteristic Curves (SWCC’s) for A-6 subgrade soils
The prediction models developed from the second regression analysis are presented in Equation 5-14 through Equation 5-22.

For A-4 subgrade soil:

\[ k_1 = 3058.622 + 81.194.Pi - 18.111.DD - 85.204.MC + 2.029.Gravel\% \]

\[ (R^2 = 0.84, \text{ Adjusted } R^2 = 0.71) \] \hspace{1cm} (5-6)

\[ k_2 = -26.152 + 0.065.DD + 21.605.DDR - 0.018.Gravel\% - 1.593. MCR \]

\[ (R^2 = 0.79, \text{ Adjusted } R^2 = 0.67) \] \hspace{1cm} (5-7)

\[ k_3 = 34.211 - 0.613.Pi - 33.761.DDR + 0.193.MC \]

\[ (R^2 = 0.77, \text{ Adjusted } R^2 = 0.62) \] \hspace{1cm} (5-8)
For A-6 subgrade soil:

\[
k_1 = -26476.1 + 13.560\cdot PI + 204.446\cdot DD + 125.416\cdot MC + 13.56\cdot P200
\]

\[R^2 = 0.75, \text{ Adjusted } R^2 = 0.60\] \hspace{1cm} (5-9)

\[
k_2 = 0.033 + 0.480\cdot LL - 0.616\cdot PI + 0.071\cdot Gravel\% - 7.434\cdot MCR
\]

\[R^2 = 0.83, \text{ Adjusted } R^2 = 0.72\] \hspace{1cm} (5-10)

\[
k_3 = 114.431 - 0.968\cdot DD + 0.172\cdot Gravel\% - 11.098\cdot MCR + 0.416\cdot MC
\]

\[R^2 = 0.88, \text{ Adjusted } R^2 = 0.76\] \hspace{1cm} (5-11)

For A-7-6 subgrade soil:

\[
k_1 = -61.234 - 1.326\cdot PI + 4.854\cdot MC + 16.729\cdot Gravel\%
\]

\[R^2 = 0.87, \text{ Adjusted } R^2 = 0.71\] \hspace{1cm} (5-12)

\[
k_2 = -77.859 + 0.487\cdot PI + 0.845\cdot DD - 16.271\cdot MCR - 3.645\cdot Gravel\%
\]

\[R^2 = 0.91, \text{ Adjusted } R^2 = 0.78\] \hspace{1cm} (5-13)

\[
k_3 = 76.061 - 0.512\cdot P200 - 0.287\cdot PI - 0.803\cdot MDD + 61.216\cdot DDR
\]

\[R^2 = 0.78, \text{ Adjusted } R^2 = 0.64\] \hspace{1cm} (5-14)

where PI is the plastic index, LL is the liquid limit, MC is the moisture content, MCR is the moisture content ratio (MC/MC_{opt}), DD is the dry density, DDR is the dry density ratio (DD/DD_{max}), and P_{200} is the percent of material passing # 200 sieve. Note that the unit used for the resilient modulus and the stresses in Equation 5-4 is psi.

Figures 5.15, 5.16 and 5.17 compare the measured k-coefficients, obtained by performing a nonlinear regression analysis on the measured resilient modulus, to those predicted using the models shown in Equations 5-6 through 5-14. From these figures, it can be seen that there is a good agreement between the measured and predicted values.
The predicted resilient modulus obtained from the proposed model, Equation 5-4, by substituting the predicted values of k-coefficients (k1, k2 and k3) is compared with the measured resilient modulus in Figure 5.18. A good agreement between the measured and predicted resilient modulus can be observed.
Figure 5.15 Comparison between predicted and measured k-coefficients for A-4 subgrade soils (a) $k_1$ (b) $k_2$ (c) $k_3$
Figure 5.16 Comparison between predicted and measured k-coefficients for A-6 subgrade soils (a) $k_1$ (b) $k_2$ (c) $k_3$
Figure 5.17 Comparison between predicted and measured k-coefficients for A-7-6 subgrade soils (a) $k_1$ (b) $k_2$ (c) $k_3$
Figure 5.18  Predicted resilient modulus versus laboratory resilient modulus (a) A-4 soils (b) A-6 soils (c) A-7-6 soil
5.8 Summary and Conclusions

Based on the experimental data conducted in this study for A-6 and A-4 cohesive soils, a new model was proposed to predict the resilient modulus of subgrade cohesive soils. In this model, both octahedral normal and shear stresses as well as soil matric suction were used. It is shown that by incorporating the octahedral normal and shear stresses, the actual stresses in the field can be represented. Further, the use of soil matric suction significantly enhanced the capability of the predictive equations to capture resilient modulus of soils at different levels of moisture contents. The accuracy of the proposed model was best for A-7-6 and A-6 subgrade cohesive soils. The model also works well for A-4 subgrade soils. Comparing with Liang et al. (2008) model, the new proposed model provides a significant enhancement to the cohesive subgrade resilient modulus predictions, mainly for materials with higher amount of clays. Comparing with the empirical model adopted in the MEPGD, the proposed model provides better accuracy in predicting the resilient modulus of cohesive soils. The proposed model has an added advantage over the empirical model adopted in the new 2002 design guide, in which it can be used to predict resilient modulus at different stress states and moisture contents. Regression models to predict the k-coefficients in the proposed model from basic soil properties were developed. High coefficients of determination (R-squared) were observed between the proposed model k-coefficients and soil physical properties. The validity of the developed k-coefficient models to predict the resilient modulus was examined. A good agreement between the measured and the predicted resilient modulus was observed.
CHAPTER VI

VALIDATION OF THE NEW ENHANCED INTEGRATED CLIMATIC
MODEL (EICM VERSION 3.2) FOR THE ATB-90 OHIO DRAINABLE BASE
SECTIONS AT I-90

6.1 Abstract

In this chapter, the prediction capability of the new EICM (version 3.2) has been evaluated by comparing the predicted values of the temperature, moisture, and frost depth with those predicted from a previous version (EICM version 3.0) and those collected from I-90 site over a period of six years. Also, sensitivity analyses were conducted to evaluate the effect of various input parameters on EICM temperature and moisture predictions. As a conclusion, the new EICM (version 3.2) has better prediction capability compared with the previous EICM (version 3.0). As expected, the ability of the new EICM to predict the moisture content in the granular bases has been greatly improved. Also, the temperature prediction capability has been improved; however, some variations still exist between the predicted and the measured temperature especially at the surface of the pavement structure. Based on the default upper and lower limits of water freezing temperatures (32 and 30.2 ºF, respectively), no agreement was found between the predicted and the measured frost penetration depths. However, the disagreement has been solved by setting the upper and lower freezing temperature limits to 12 ºF and 10 ºF,
respectively. Results from sensitivity analyses indicated that new EICM exhibit high to moderate sensitivity to the surface short-wave absorptivity and low sensitivity to the asphalt thermal conductivity and heat capacity. For moisture predictions, the analysis indicated that the new EICM exhibit high sensitivity to the soil porosity and fines contents. Based on the sensitivity analysis, pavement sections were calibrated. The calibrated pavement sections were found in a good agreement with the measured pavement temperatures and moisture contents.

6.2 Introduction

Materials used in constructing pavements are highly influenced by the climatic conditions; consequently, a sound pavement design entails the need for an ability to estimate or predict the seasonal variations of pavement material properties. The major factors that may be influenced by the climatic conditions are moisture, temperature, and freeze-thaw cycles experienced by pavement materials. In the new mechanistic-empirical pavement design guide (Design Guide, 2002), the effect of moisture and temperature variations on the pavement materials characteristics was taken into account through the use of a climatic modeling computer program called the Enhanced Integrated Climatic Mode (EICM).

The EICM is a one-dimensional coupled heat and moisture flow model initially developed for the FHWA and later adapted by the new mechanistic-empirical pavement design guide (MEPDG) developed under NCHRP Project 1-37A. The EICM program has undergone several revisions since its initial development. The latest improvement on the EICM incorporates the concept of Thornthwaite Moisture Index (TMI) and employs
improved models under the development effort of NCHRP Project 9-23, reported in NCHRP Report 602 by Zapata and Houston (2008).

In the past, there have been numerous studies devoted to validation of EICM in terms of its ability to predict moisture, temperature, and frost depth within the pavement structure; nevertheless, most of these studies were aimed at EICM version 3.0 and older version. For example, Birgisson et al. (2000) used ICM model to predict seasonal variations in temperature, moisture content, and layer moduli at two flexible pavement sections at the Minnesota Research Project (Mn/ROAD) site. They observed a fairly close trend between the predicted and measured moisture contents, except during spring thaw period where the ICM model failed to predict sudden increase in volumetric moisture content. Richter and Witczak (2001) used data collected from the Long-Term Pavement Performance Seasonal Monitoring Program (LTPP SMP) sites to evaluate the predictive capability of ICM for the volumetric moisture content. They concluded that ICM Version 2.1 failed to predict the measured while the ICM Version 2.6 predicted well. Heydinger (2003) studied the climatic effects on pavement using the Enhanced Integrated Climatic Mode (EICM) Version 3.0. A calibrated EICM models were developed for pavement sections 390104 and 390304 for the year 2000. The calibrated models predict higher volumetric moisture content than the default models but very little seasonal variation of moisture content. Nevertheless, the predicted temperature profiles were significantly improved. Ahmad ea al. (2005) evaluated EICM Version 3.0 using New Jersey monitoring site data. It was shown that a strong, consistent correlation could not be established between EICM prediction and field measurement, especially for the surface layers. Quintero N. M. (2007) evaluation study of EICM 3.2 based on the Ohio SHRP
Test Road data indicated underprediction of moisture content. Temperature profile prediction was erratic. The frost penetration depth can occasionally be matched; however there was not a consistent trend. Zapata and Houston (2008) evaluated, calibrated, and validated the moisture predictive capability of the EICM. Data from the LTPP SMP and other relevant field experiment were used. The statistical analysis of the test results of the EICM validation confirmed that all its component models, especially the suction model, were in need of improvement and calibration. The improved EICM was incorporated in Version 1.0 of the MEPDG software developed under NCHRP Project 1-40D.

The main objectives of this study are: (a) to evaluate the predictive capability of the EICM Version 3.2 by comparing the moisture, temperature and frost depth predictions with the field measured data collected from I-90 in Ashtabula County, Ohio; (b) to conduct a sensitivity analysis to evaluate the effects of several key parameters on EICM Version 3.2 predictions; (c) to conduct a comparison between EICM Version 3.2 and EICM Version 3.0 to quantify the improvement by EICM Version 3.2 using I-90 as a case study.

6.3 Enhanced Integrated Climatic Model (EICM Version 3.2)

The EICM is a one-dimensional coupled heat and moisture flow model initially developed for the FHWA and adapted for use in the new mechanistic-empirical pavement design guide (MEPDG) developed under NCHRP Project 1-37A. The EICM program has undergone several revisions since its initial development. The latest version of the EICM incorporates the concept of Thornthwaite Moisture Index (TMI) and incorporates improved models developed under NCHRP Project 9-23. The improved version of the
EICM (Version 3.2) was incorporated in Version 1.0 of the MEPDG software developed in NCHRP Project 1-40D. As a model input, EICM (version 3.2) requires:

- Model analysis parameters: the exact date and duration of the analysis period
- Climatic data inputs: temperature, wind speed, solar radiation, precipitation, humidity and water table depth.
- Material properties: thermal conductivity, heat capacity, and total unit weight
- Pavement structure: select layers type, thickness and number of element.
- Base course moisture models: Thornthwaite Moisture Index (TMI) or TTI Infiltration and Drainage Model.
- Initial temperature and water content profile: generate initial temperature and select the output nodes.

One of the main outputs of the EICM is a set of adjustment factors for unbound materials layers that account for the effects of environmental parameters and conditions such as moisture content changes, freezing, thawing, and recovery from thawing. These adjustment factors are used to compute the composite environmental adjustment factor (Fenv), which then is used by the MEPDG to modify the unbound materials resilient modulus as a function of location and time. Other important outputs of the EICM include the predictions of in situ temperature and moisture profiles. In the MEPDG, the predicted temperature profile through the asphalt layer is used in both fatigue and permanent deformation prediction models; whereas the predicted moisture profile is used in the permanent deformation model for the unbound materials. Also, the EICM computes and predicts the following data throughout the pavement profile: resilient modulus adjustment
factors, pore water pressure, frost and thaw depth, frost heave and drainage performance. The time period for which this data is generated can range from one day to one year. The time interval for the data output can range from 6 minutes to 12 hours. The EICM requires the following climatic data inputs: daily maximum and minimum temperatures, cloud cover, wind speed and precipitation amounts.

6.4 Thronthwaite Moisture Index (TMI)

The EICM Version 3.2 incorporates a new approach to calculate the equilibrium suctions in the field. This approach is based on the use of the Thronthwaite Moisture Index (TMI) instead of water table depth in the suction model to calculate the equilibrium moisture content. The TMI is a useful indicator of the supply of water (precipitation) in an area relative to the demand for water under prevailing climatic conditions (potential evapotranspiration), according to work by Thorthwaite and Mather (1955) and Mather (1978). It has been found that the new suction model greatly improves the prediction of the equilibrium moisture content, especially for granular bases. The TMI is calculated as follows:

\[
TMI = 75*(P/PE - 1) + 10
\]  

(6-1)

where: TMI = Thorthwaite Moisture Index, P = Annual Precipitation (cm), and PE = Adjusted Potential Evaporanspiration (cm).

6.5 I-90 Field Monitoring Data

The data used in this study belong to the ATB-90 project, located at I-90 in Ashtabula County, Ohio. The ATB-90 research project is a field instrumentation and monitoring
research sponsored by the Ohio Department of Transportation (ODOT) and Federal Highway Administration (FHWA) to study the effectiveness of different drainable base materials in actual service conditions. The instrumentation installed at the monitoring sites includes moisture sensors, temperature probes, frost depth probes and two on-site weather stations. The moisture measurements were the Time Domain Reflectometry (TDR) system manufactured by Campbell Scientific Inc., Logan, UT. The temperature probes (TP 101), manufactured by Measurements Research Corporation (MRC), were used to monitor the temperature profiles in different pavement layers and subgrade soils. Frost depth probes, developed by the U.S. Army Corps of Engineers Cold Region Research and Engineering Laboratory (CRREL), were used to monitor frost occurrence through the pavement system. Two weather stations with the capability of monitoring solar radiation, air temperature, wind speed, wind direction, and rainfall were installed at the project site to collect the climatic data.

In this study, the data collected at a total of six monitored pavement sections were used in evaluating the Enhanced Integrated Climatic Model (EICM Version 3.2). Four test sections were constructed with unbound drainable base materials (ODOT 304, ODOT 307_NJ, ODOT 307_IA, and ODOT 307_CE) and two with bound materials (ODOT 308-asphalt treated and ODOT 306-cement treated base materials). The TDR probes were placed at ten different depths in the unbound drainable bases pavement cross-sections and at nine different depths in the bound drainable bases pavement cross-sections. The temperature probes (thermistor probes) were essentially a stainless steel metal rod on the top connected with a 6 foot long, acrylic pipe, wherein 15 thermistors were embedded inside for temperature measurements. The theory of the MRC probes is that any slight
temperature changes will create major variation in the resistance values of the thermistor. Knowing the change in resistance, Equation 6-2 can be used to determine the pavement temperature.

\[
\frac{1}{T} = c_1 + c_2 \ln R + c_3 (\ln R)^3
\]  

(6-2)

where \( T = \) absolute temperature in Kelvin, \( R = \) resistance in Ohms, and \( c_1, c_2 \) and \( c_3 = \) calibration constants for individual thermistors. The frost depth probe consists of 36 metal wire electrodes, with 2-inch spacing, mounted on a solid PVC pipe 73-inch long. Only three monitoring sites were installed with frost probe: ODOT 307 (1A), ODOT 308 (asphalt treated), and ODOT 306 (cement treated) sections. The frost depth profiles were measured on daily basis in those sections. The laboratory calibration of the resistivity probes showed that the values of electrical resistivity of 2000 mV or less indicate the freezing of the materials surrounding the probe (Al-Akhras, 2004).

6.6 Validation of the EICM (Version 3.2)

For the validation study, EICM version 3.2 and 3.0 were used to predict the temperature, moisture and frost depth profiles at each monitored pavement section. The EICM predictions were carried out with the data from ATB-90 project site. The input values for EICM models are presented in Table 6.1 and 6.2.
### Table 6.1 Asphalt cement material properties

<table>
<thead>
<tr>
<th>Properties</th>
<th>Range</th>
<th>Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of layer (in)</td>
<td>---</td>
<td>15.25</td>
</tr>
<tr>
<td>Thermal conductivity asphalt Btu/(ft)(hr)(°F).</td>
<td>0.44 to 0.81</td>
<td>0.63</td>
</tr>
<tr>
<td>Shortwave absorptivity for fresh asphalt (black)</td>
<td>0.9-0.98</td>
<td>0.95</td>
</tr>
<tr>
<td>Heat capacity of asphalt Btu/(lb)(°F)</td>
<td>0.22 to 0.40</td>
<td>0.32</td>
</tr>
<tr>
<td>Total unit weight of asphalt (pcf)</td>
<td>---</td>
<td>148</td>
</tr>
</tbody>
</table>

### Table 6.2 Base, Subbase and Subgrade material properties

<table>
<thead>
<tr>
<th>Property</th>
<th>307-NJ</th>
<th>307-IA</th>
<th>307-CE</th>
<th>306-cement Treated</th>
<th>304-Asphalt Treated</th>
<th>304-Subbase</th>
<th>A-6-a Subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness, inches</td>
<td>4</td>
<td>4</td>
<td>6</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>12</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.22</td>
<td>0.19</td>
<td>0.23</td>
<td>0.37</td>
<td>0.16</td>
<td>0.37</td>
<td>0.3</td>
</tr>
<tr>
<td>Specific Gravity, lb/ft³</td>
<td>2.6</td>
<td>2.6</td>
<td>2.6</td>
<td>2.6</td>
<td>2.59</td>
<td>2.6</td>
<td>2.6</td>
</tr>
<tr>
<td>Saturated permeability (ft/hr)</td>
<td>159</td>
<td>95</td>
<td>154</td>
<td>1056</td>
<td>59</td>
<td>1044</td>
<td>226</td>
</tr>
<tr>
<td>dry unit weight</td>
<td>127</td>
<td>128</td>
<td>127</td>
<td>109</td>
<td>130</td>
<td>108</td>
<td>122</td>
</tr>
<tr>
<td>Percent passing #4 sieve %</td>
<td>47</td>
<td>40</td>
<td>28</td>
<td>5</td>
<td>45</td>
<td>5</td>
<td>33</td>
</tr>
<tr>
<td>plasticity index</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Percent passing #200 sieve %</td>
<td>1.2</td>
<td>0.7</td>
<td>1.18</td>
<td>0.5</td>
<td>6.5</td>
<td>0.5</td>
<td>4.4</td>
</tr>
<tr>
<td>Diameter D60 (mm)</td>
<td>12.2</td>
<td>11.9</td>
<td>16.1</td>
<td>16</td>
<td>11.9</td>
<td>16</td>
<td>22.4</td>
</tr>
<tr>
<td>Initial volumetric water content</td>
<td>17.9</td>
<td>6.7</td>
<td>6.8</td>
<td>8</td>
<td>11.2</td>
<td>8.3</td>
<td>(12.65, 10.7, 8.6, 18, 12 and 18)*</td>
</tr>
</tbody>
</table>


** Subgrade initial volumetric water content underneath 307-NJ, 307-IA, 307-CE, Cement treated, and asphalt treated respectively.
6.6.1 Moisture Data Comparison

The moisture content predicted by EICM Version 3.2 and EICM Version 3.0 is compared with the moisture data collected at ATB-90 test sections in Figures 6.1 through 6-6 for each monitored section respectively. The seasonal representative moisture contents are the values of the average of daily moisture contents over the month of July-2007, October-2007, January-2008, and April-2008. Comparisons between EICM Version 3.2, EICM Version 3.0 and field measured moisture content for unbound drainable base material sections are shown in Figures 6.1 through 6.4. From these figures, it can be seen that the predictions from EICM Version 3.2 are in much better agreement with the field measured values than the predictions obtained from EICM Version 3.0. The improved prediction by EICM 3.2 can be seen clearly in the base and subbase layers. In the subgrade layer, both EICM Version 3.2 and EICM Version 3.0 provide close predictions. However, EICM Version 3.2 shows better prediction trend by following the measured data profiles. Also, it can be seen that both EICM Version 3.2 and EICM Version 3.0 under-predict the moisture content in base and subbase materials and over-predict the moisture content in subgrade materials. Furthermore, it can be seen that the moisture distributions with depth from season to season at each section are almost the same. Figure 6.5 and 6.6 show the moisture profile comparison for bound drainable base material sections. It can be seen that the new EICM Version 3.2 can provide a close match with measured in both base and subbase materials. However, there is some discrepancy in predicted and measured moisture content in the subgrade soils.

The time history for the measured and predicated moisture in base, subbase, and subgrade for ODOT 307-NJ (unbound base) and ODOT 308 asphalt cement treated
(bound) sections are shown in Figures 6.7 through 6.11. Figure 6.7 shows both measured and predicted moisture content in base layer at ODOT 307-NJ section during the period between December 2003 and September 2008. It can be seen that EICM Version 3.2 predicts better than EICM Version 3.0 does. Furthermore, EICM Version 3.2 predicts the seasonal moisture variation very well.

Both measured and predicted moisture content in subbase and subgrade for ODOT 307-NJ section are shown in Figure 6.8 and 6.9, respectively. In the subbase layer, the EICM Version 3.2 predicted much better than EICM Version 3.0 did. EICM Version 3.2 predicted that volumetric moisture content eventually reaches a stable value of about 0.21. In the subgrade layer, EICM Version 3.2 predicted higher moisture content than those measured; whereas EICM 3.0 tended to underpredict. Constant values of about 0.37 and 0.30 for moisture contents were predicted by EICM Version 3.2 and EICM Version 3.0, respectively.

Both measured and predicted moisture in subbase and subgrade at ODOT 308 asphalt treated section are shown in Figure 6.10 and Figure 6.11, respectively. Again, EICM Version 3.2 predicts much better than EICM Version 3.0 does. For subgrade layer, both EICM Version 3.2 and EICM Version 3.0 predict reaching a stable moisture content at about 0.37.

A comparison of prediction accuracy of two EICM versions is made by calculating the coefficient of determination, R-squared between the measured and the predicted moisture contents at different pavement sections. The computed values for R-squared are presented in Table 6.3. It can be seen that EICM Version 3.2 predicts better than the EICM Version 3.0. In general, the predictive capability of the EICM models is better in the asphalt
pavement sections built with unbound base layers. In the case of the pavement section built with cement treated base layer, R-squared for EICM Version 3.2 and EICM Version 3.0 are 0.42 and 0.06, respectively. This means that both EICM models are incapable of predicting the moisture contents with an acceptable accuracy. This conclusion agrees with the finding obtained by Zapata and Houston (2008) in that EICM is not capable of dealing with sections built with lime-treated or cement-treated aggregate materials.
Figure 6.1 Moisture content profiles for ODOT 304 pavement section
Figure 6.2  Moisture content profiles for ODOT 307-NJ drainable base section
Figure 6.3 Moisture content profiles for ODOT 307-IA drainable base section
Figure 6.4 Moisture content profiles for ODOT 307-CE drainable base section
Figure 6.5 Moisture content profiles for ODOT 306-Cement treated drainable base section
Figure 6.6  Moisture content profiles for ODOT 308-Asphalt treated drainable base section
Figure 6.7 Moisture profile for ODOT 307_NJ drainable base section (base layer)

Figure 6.8 Moisture profile for ODOT 307_NJ drainable base section (Subbase layer)
Figure 6.9  Moisture profile for ODOT 307_NJ drainable base section (Subgrade layer)

Figure 6.10 Moisture profile for ODOT 308_asphalt treated base section (subbase layer)
Figure 6.11  Moisture profile for ODOT 308 asphalt treated base section (subgrade layer)

Table 6.3  Coefficient of determination, R-squared, between predicted and measured moisture contents

<table>
<thead>
<tr>
<th>Pavement Section</th>
<th>R-squared</th>
<th>EICM Version 3.2</th>
<th>EICM Version 3.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>ODOT 304</td>
<td>0.90</td>
<td>0.89</td>
<td></td>
</tr>
<tr>
<td>OCOT 307 (CE)</td>
<td>0.97</td>
<td>0.62</td>
<td></td>
</tr>
<tr>
<td>ODOT 307 (IA)</td>
<td>0.74</td>
<td>0.56</td>
<td></td>
</tr>
<tr>
<td>ODOT 307 (NJ)</td>
<td>0.89</td>
<td>0.72</td>
<td></td>
</tr>
<tr>
<td>ODOT 308 (asphalt treated)</td>
<td>0.76</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>ODOT 306 (cement treated)</td>
<td>0.42</td>
<td>0.06</td>
<td></td>
</tr>
</tbody>
</table>
6.6.2 Temperature Data Comparison

Comparisons between models-predicted and field-measured pavement temperature profiles for ATB-90 sections are presented in this section from Figures 6.12 to 6.17, with each figure for each monitored pavement section. From these figures, it can be seen that the capability of the EICM Version 3.2 to predict surface, base, subbase and subgrade temperature profiles is relatively better than EICM Version 3.0. Generally, both EICM Version 3.2 and EICM Version 3.0 over predict the pavement temperature profiles. Noticeable differences between the measured and predicted temperatures were found close to the pavement surfaces. However, these differences become smaller and smaller with depth. The accuracy of temperature prediction varies from season to season. The smallest difference between the measured and predicted surface temperature was found in the month of October, whereas the largest difference was found in the month of July. For EICM Version 3.2, the differences between the predicted and the measured surface temperatures are in the range of (0.3 – 1.2 ºF) and (8.3 – 11.9 ºF) in October and in July, respectively. For EICM Version 3.0, the differences between the predicted and the measured surface temperatures are in the range of (2.9– 4.3 ºF) and (13.9 – to 18.1 ºF) in October and in July, respectively. To visually investigate the effect of different types of base materials on pavement temperature distribution, the measured temperature at different sections were plotted together in Figure 6.18. From this figure, it can be seen that the difference between the measured pavement temperature profiles at different pavement sections with different underlying drainable base materials are small.
The time trends for the measured and predicated base, subbase and subgrade temperature profiles for ODOT 304 and ODOT 308 (asphalt cement) are shown in Figures 6.19 through 6.26. The results drawn from these figures support the conclusions made earlier. We can see the difference between the predicted and the measured surface temperature as presented in Figure 6.19 and Figure 6.23 for ODOT 304 and ODOT 308 asphalt treated section, respectively. For ODOT 304 section, there is a general agreement between the measured and predicted pavement temperatures at base, subbase and subgrade layers as shown in Figures 6.20, 6.21 and 6.22. The same agreement was also shown in ODOT 308 asphalt treated section as presented in Figures 6.24, 6.25, and 6.26.
Figure 6.12 Temperature, ODOT 304 pavement section
Figure 6.13 Temperature, ODOT 307-IA drainable base section
Figure 6.14  Temperature, ODOT 307-NJ drainable base section
Figure 6.15 Temperature, ODOT 307-CE drainable base section
Figure 6.16 Temperature, ODOT 308-Asphalt treated drainable base section
Figure 6.17 Temperature, ODOT 306 –cement treated drainable base section
Figure 6.18 Temperature distributions with depth at different pavement sections
Figure 6.19  Temperature profile for ODOT 304 pavement section (Surface layer)

Figure 6.20  Temperature profile for ODOT 304 pavement section (base layer)
Figure 6.21 Temperature profile for ODOT 304 pavement section (subbase layer)

Figure 6.22 Temperature profile for ODOT 304 pavement section (subgrade layer)
Figure 6.23 Temperature profile for ODOT 308 asphalt treated base section (Surface Layer)

Figure 6.24 Temperature profile for ODOT 308 asphalt treated base section (base layer)
Figure 6.25 Temperature profile for ODOT 308 asphalt treated base section (subbase layer)

Figure 6.26 Temperature profile for ODOT 308 asphalt treated base section (subgrade layer)
6.6.3 Frost Depth Comparison

The comparison of the predicted and measured frost depth was carried out for the time interval from the beginning of the monitoring period (September 2003) to September 2008. The comparisons at three monitored sites are shown in Figures 6.27, 6.28, and 6.29 for ODOT 307-IA (unbound base), ODOT 308 (asphalt treated base), and ODOT 306 (cement treated base), respectively. From these figures, it can be seen that CRREL electrical resistivity probes recorded no freezing action at any depth over the entire monitoring period at all pavement sections. On the other hand, both EICM Version 3.2 and EICM Version 3.0 predicted frost penetration during the monitoring period. The absence of the freezing actions in the field confirms the non-zero moisture contents measured by the TDR, as mentioned earlier. Frost predictions occurred yearly between December and March. Table 6.4 shows the maximum frost penetration depths predicted by EICM models and measured by CRREL electrical resistivity probes. The EICM Version 3.2 predicted a maximum frost depth of 26 inches, 29 inches, and 29 inches for ODOT 307-IA, ODOT 308 (asphalt treated), and ODOT 306 (cement treated) section, respectively. In comparison, The EICM Version 3.0 predicted a maximum frost depth of 24 inches, 23 inches, and 22 inches for ODOT 307-IA, ODOT 308 (asphalt treated), and ODOT 306 (cement treated) section, respectively. The discrepancy between the measured and predicted frost depths could be explained by the influence exerted on the measurement sensors due to the salinity (salt as de-icing agent in winter maintenance program) of the water in the pavement structure. For clean water, frost occurs when the pavement temperature drops below 32°F. For salted water, the freezing temperature is a function of salt concentration. The higher the concentration is, the lower the freezing
temperature. To incorporate the salinity affects on frost depth predictions, the upper and lower temperature limits of freezing range have to be adjusted inside the EICM program. It was found that by setting the upper freezing temperature at 12 °F and the lower freezing temperature at 10 °F, no frost depth was predicted in the pavement sections under this study.

Figure 6.27  Frost penetration depth and pavement temperature profiles for ODOT 307 – IA drainable base section
Figure 6.28 Frost penetration depth and pavement temperature profiles for ODOT 308 – asphalt treated drainable base section

Figure 6.29 Frost penetration depth and pavement temperature profiles for ODOT 306 – cement treated drainable base section
Table 6.4 Maximum frost penetration depths (upper freezing temperature = 32 °F, lower freezing temperature = 30.2 °F)

<table>
<thead>
<tr>
<th>Pavement Section</th>
<th>Max. Frost Penetration Depth, inches</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Measured</td>
</tr>
<tr>
<td>ODOT 307 (IA)</td>
<td>0</td>
</tr>
<tr>
<td>ODOT 308 (asphalt treated)</td>
<td>0</td>
</tr>
<tr>
<td>ODOT 306 (cement treated)</td>
<td>0</td>
</tr>
</tbody>
</table>

6.7 Sensitivity Analysis

In this study, sensitivity analyses were conducted to evaluate the effect of various input parameters on EICM Version 3.2 temperature and moisture predictions.

6.7.1 Sensitivity Analysis of Temperature Predictions

The main thermal properties affecting heat transport in a pavement structure are as follows: surface short-wave absorptivity, thermal conductivity, and heat capacity. Due to the lack of laboratory information on these input parameters for the materials at ATB-90 site, typical values recommended by M-E design guide were used. In this study, the EICM predictions showed some discrepancy between the measured and the predicted pavement temperatures, in particular at the pavement surface. As a result, a sensitivity analysis was conducted to evaluate the effect of changing pavement thermal properties on temperature predictions. The thermal property values were changed over a range recommended by the M-E design guide as shown in Table 6.5. The pavement section built with ODOT 304 base material is used for the sensitivity analysis. The results of the sensitivity analysis are shown in Figures 6.30, 6.31 and 6.32. Figure 6.30 shows the effect
of changing the pavement surface short-wave absorptivity over the range recommended by the M-E design guide. From this figure, it can be seen that the surface short-wave absorptivity has a significant effect on temperature predictions. The predicted temperature increases as the surface short-wave absorptivity increases. As expected, the highest effect of the pavement surface short-wave absorptivity is observed in the summer season, while the lowest in the winter season. Also, from the same figure, it can be seen that with surface short-wave absorptivity equal to 0.7, which is out of the recommended range for asphalt materials, the pavement temperature predictions are close to the measured pavement temperatures. The effects of the asphalt thermal conductivity and heat capacity on pavement temperature predictions, as shown in Figures 6.31 and 6.32, are not significant. Therefore, the typical values of 0.67 Btu/(Ib)(°F) and 0.22 Btu/(Ib)(°F) for thermal conductivity and heat capacity were used. Calibrated models were developed for ODOT 307-NJ and ODOT 306 (asphalt treated) sections by adjusting the surface short-wave absorptivity to 0.7. As shown in Figures 6.33 and 6.34, the temperature predictions from the calibrated model are in much better agreement with the field measured values, especially at the summer season.

Table 6.5 Pavement thermal properties

<table>
<thead>
<tr>
<th>Thermal Input Parameter</th>
<th>Typical Value</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface short-wave absorptivity</td>
<td>0.85</td>
<td>0.8 – 0.9 (weathered asphalt, Gray)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.9 – 0.98 (Fresh asphalt, Black)</td>
</tr>
<tr>
<td>Thermal conductivity of asphalt</td>
<td>0.67Btu/(Ib)(°F)</td>
<td>0.44 – 0.8</td>
</tr>
<tr>
<td>Heat capacity of asphalt</td>
<td>0.22Btu/(Ib)(°F)</td>
<td>0.22 – 0.44</td>
</tr>
</tbody>
</table>
Figure 6.30 Effect of pavement surface short-wave absorptivity on EICM 3.2 temperature predictions
Figure 6.31 Effect of pavement thermal conductivity on EICM 3.2 temperature predictions
Figure 6.32 Effect of pavement heat capacity on EICM 3.2 temperature predictions
Figure 6.33 Temperature profiles for ODOT 307-NJ drainable base section
Figure 6.34 Temperature profiles for ODOT 308-Asphalt treated drainable base section
6.7.2 Sensitivity Analysis of Moisture Prediction

Despite the improvements made in the moisture predictions using EICM 3.2, some discrepancies between the predicted and the measured moisture contents still exist. According to Heydinger (2003), EICM program is sensitive to the soil porosity and fine contents when predicting volumetric water content. The predicted volumetric water contents increase proportionally to the increases in porosity and fine contents. In this section, the above findings were confirmed as shown in Figures 6.36 and 6.37. From these figures, it can be seen that both base porosity and amount of fines can affect the base water content to a certain limit; after which, no significant effect can be noticed. In comparison, more pronounced effects can be seen on the subbase water contents, while less effects can be noticed on the subgrade water content. As a result, base, subbase and subgrade porosity values were changed to obtain the optimum combination that brings the predicted moisture profiles close to the measured ones. Also, the amounts of fines were altered to enhance the moisture predictions. Porosity values and the percent passing #200 sieve used in this study are shown in Tables 6.6 and 6.7, respectively.

Calibrated models were developed for ODOT 304, ODOT 307 (NJ, IA and CE), cement treated and asphalt treated sections by varying the porosity and fine contents of base, subbase and subgrade soils. The calibrated predictions are shown in Figures 6.1 through 6.6. From these figures, it can be seen that the predictions from the calibrated model are in much better agreement with the field measured values than the predictions obtained from the default data. Generally, base and subbase volumetric water contents were increased by increasing their porosity and fine contents, while subgrade volumetric water contents were decreased by decreasing their porosity.
Figure 6.35 Effect of base porosity on EICM 3.2 moisture predictions

Figure 6.36 Effect of base amount of fines (Passing sieve #200) on EICM 3.2 moisture predictions
Table 6.6 Default and calibrated input values for porosity

<table>
<thead>
<tr>
<th>Pavement Section</th>
<th>Porosity (n-values)</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base</td>
<td>Subbase</td>
<td>Subgrade</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Default</td>
<td>Calibrated</td>
<td>Default</td>
<td>Calibrated</td>
<td>Default</td>
</tr>
<tr>
<td>ODOT 304</td>
<td>0.16</td>
<td>0.18</td>
<td>0.3</td>
<td>0.33</td>
<td>0.38</td>
</tr>
<tr>
<td>ODOT 307-NJ</td>
<td>0.22</td>
<td>0.25</td>
<td>0.3</td>
<td>0.32</td>
<td>0.38</td>
</tr>
<tr>
<td>ODOT 307-IA</td>
<td>0.19</td>
<td>0.2</td>
<td>0.3</td>
<td>0.35</td>
<td>0.38</td>
</tr>
<tr>
<td>ODOT 307-CE</td>
<td>0.23</td>
<td>0.25</td>
<td>0.3</td>
<td>0.32</td>
<td>0.38</td>
</tr>
<tr>
<td>Asphalt treated</td>
<td>0.33</td>
<td>0.35</td>
<td>0.3</td>
<td>0.33</td>
<td>0.38</td>
</tr>
<tr>
<td>Cement treated</td>
<td>0.33</td>
<td>0.33</td>
<td>0.3</td>
<td>0.31</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Table 6.7 Default and calibrated input values for % Passing #200 sieve

<table>
<thead>
<tr>
<th>Pavement Section</th>
<th>% Passing #200 sieve</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base</td>
<td>Subbase</td>
<td>Subgrade</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Default</td>
<td>Calibrated</td>
<td>Default</td>
<td>Calibrated</td>
<td>Default</td>
</tr>
<tr>
<td>ODOT 304</td>
<td>6.5</td>
<td>13</td>
<td>4.4</td>
<td>7.5</td>
<td>69</td>
</tr>
<tr>
<td>ODOT 307-NJ</td>
<td>1.2</td>
<td>10</td>
<td>4.4</td>
<td>7.5</td>
<td>69</td>
</tr>
<tr>
<td>ODOT 307-IA</td>
<td>0.7</td>
<td>10</td>
<td>4.4</td>
<td>7.5</td>
<td>69</td>
</tr>
<tr>
<td>ODOT 307-CE</td>
<td>1.18</td>
<td>5</td>
<td>4.4</td>
<td>4.4</td>
<td>69</td>
</tr>
<tr>
<td>Asphalt treated</td>
<td>0.5</td>
<td>1.8</td>
<td>4.4</td>
<td>4.4</td>
<td>69</td>
</tr>
<tr>
<td>Cement treated</td>
<td>0.5</td>
<td>1.5</td>
<td>4.4</td>
<td>4.4</td>
<td>69</td>
</tr>
</tbody>
</table>

6.8 Summary and Conclusions

Temperature, moisture, and frost depth data gathered from the ATB-90 sites, Ashtabula County, Ohio, were analyzed to evaluate the predictive capability of EICM Version 3.2 for Ohio climatic conditions. EICM Version 3.0 was also used to contrast the
improvements made to EICM Version 3.2. Based on the findings reported herein, it can be concluded that EICM Version 3.2 were substantially improved, especially for the prediction of the volumetric moisture content in the base and subbase layers. EICM Version 3.2 showed an improvement in the moisture predictions, especially at base and subbase layers, while in the subgrade layer both EICM Version 3.2 and EICM Version 3.0 provided close predictions. Generally, the EICM Version 3.2 under-predicts the moisture content in the base and subbase layers, and over-predicts the moisture content of the subgrade layer.

The prediction capability of the EICM models is better in the pavement sections built with unbound base layers than in pavement sections built with bound base layer. For cement treated base, EICM model was incapable of predicting the moisture contents with an acceptable accuracy. The effect of the salinity of the infiltrated water during the winter season seems to have affected the capability of the EICM models to mimic what is going on in the field. To incorporate the salinity affects on frost depth predictions, the upper and lower temperature limits of freezing range have to be adjusted inside the EICM program. It was found that by setting the upper and lower freezing temperature limits at 12 °F and 10 °F, respectively, the predicted and measured frost depths were matched for the pavement sections under this study.

Results from sensitivity analyses indicated that EICM Version 3.2 exhibit high to moderate sensitivity to the surface short-wave absorptivity and low sensitivity to the asphalt thermal conductivity and heat capacity. For moisture predictions, the sensitivity analysis indicated that the EICM 3.2 exhibit high sensitivity to the soil porosity and fines contents. Based on the sensitivity analysis, values of the input parameters to EICM
Version 3.2 were adjusted and calibrated. The calibrated EICM models were found to predict well pavement temperatures and moisture contents.
CHAPTER VII
PERFORMANCE PREDICTION BASED ON MECHANISTIC-EMPIRICAL
MODEL FOR DIFFERENT DRAINABLE BASE MATERIALS

7.1 Abstract

In this chapter, the impact of base material types on pavement performance that the pavement will exhibit during its life and on the selection of HMA layer thicknesses has been evaluated using the latest version of the MEPDG software (version 1.0). The base materials studied include ODOT 304, ODOT 307 (NJ, IA, and CE), ODOT 308 (asphalt treated), and ODOT 306 (cement treated). The performance predictions analyzed include rutting, IRI, alligator cracking, longitudinal cracking, and low-temperature transverse cracking. The results showed that different base materials had little to no affects on pavement performance. This was contributed to the high HMA thickness (15.25 inches) which conceals the effect of using different base materials on pavement performance. HMA thicknesses of 12 and 10 inches were found adequate for pavement sections built with ODOT 304, ODOT 307 (NJ, IA, and CE) and asphalt treated base materials over A-6a and A-1b subgrade soils, respectively. The HMA thickness was reduced to 11 and 9 inches for the pavement section built with the cement treated base over A-6a and A-1b subgrade soils, respectively. Cement treated base was seemed to work the best, from providing effective drainage and reducing thickness requirement of asphalt layer.
7.2 Introduction

Until recently, most state highway agencies used the 1993 AASHTO Guide for Design of Pavement Structures (1) as a primary pavement design tool. This procedure adopts an empirical approach based on measurements conducted on limited number of pavement sections in Ottawa, Illinois in the late 1950s and early 1960s. Since then, factors such as traffic loads had increased and better understanding of weather conditions on pavement material behaviors had emerged, thus providing an impetus to developing a new design procedure to address new design concept and computational powers afforded by highly sophisticated analysis algorithms.

In recognition of the limitations of the earlier Guides, AASHTO initiated an effort to develop an improved design approach. The research work was initiated in 1997 and spanning a decade under two major research projects: National Cooperative Highway Research Program (NCHRP) (2) Project 1-37A and 1-40. The end product of these research efforts are the development of a mechanistic-empirical pavement design guide (MEPDG) and the accompanying software (3). Unlike empirical pavement design methods (1993 AASHTO Guide), the new proposed MEPDG combines both empirical and mechanistic procedures in determining required pavement thickness for a given set of design inputs. The mechanistic model is based on elementary physics and determines pavement response to the wheel loads or environmental condition in terms of stress, strain, and displacement. The empirical part of the design uses the pavement response to predict the life of the pavement on the basis of actual field performance (4). Comparing with 1993 AASHTO pavement design, the new MEPDG requires comprehensive material, traffic and climatic inputs to predict pavement performance. However, the
MEPDG procedure has built-in adequate flexibility to enable agencies to adopt it in a manner that is commensurate with the resources available to them at any given time. The flexibility is afforded by the three hierarchal input levels, nationally calibrated performance models, and nationally established default input values for several parameters. The three hierarchal MEPDG levels allow the designer flexibility in selecting the design inputs based on the importance of the project and the available information. Level 1 provides the highest level of reliability. The input data is obtained from direct testing on the actual material in question. Level 2 provides an intermediate level of reliability. It is used when direct test results are not available, but results from other tests are available and a relationship exists between them. Level 3 provides the lowest level of reliability and is intended for designing low volume roads.

The use of drainable base in a flexible pavement has been a widely adopted practice to ensure that no excessive water remains within pavement layers to cause early distresses. Currently, Ohio Department of Transportation (ODOT) accepts several types of materials specifications for use as drainable base materials. They are ODOT 307 (NJ, IA, and CE), ODOT 308 (asphalt treated), and ODOT 306 (cement treated). To evaluate the effectiveness of these drainable base materials in preventing excessive moisture migration and staying in different layers of the flexible pavement, a field monitoring program has been ongoing for about six years in a totally rebuilt I-90 pavement sections in Ashtabula County, Ohio. Several papers have been published to document the monitoring results (Liang et al. 2006, Rabab’ah and Liang 2007, Rabab’ah and Liang 2008). However, the as-built pavement sections using six different drainable base materials had not show any differences in distress due to relatively young life of the
pavements. There is a desire to evaluate the potential performance differences between the pavement sections built with six different drainable base materials.

The main objectives of this study are twofold. First, the impact of using six different drainable base materials on the long-term performance of the as-built pavement sections will be evaluated using the latest version of MEPDG software. Second, the influence of different types of drainable base materials and subgrade soils on the required HMA layer thickness to yield the same level of performance will be determined using the new version of MEPDG software. Details of input for material properties, traffic loads, and environmental conditions will be described. The MEPDG analysis results will be used to provide insights on the relative advantage of the drainable base materials evaluated in this study.

7.3 Project Location and Site Description

Six completely reconstructed asphalt pavement sections on I-90, each built with one type of drainable base materials accepted by ODOT (i.e., cement treated, asphalt-treated, ODOT 307-NJ, ODOT 307-IA, ODOT 307-CE, and ODOT 304 ), were comprehensively instrumented to continuously monitor the moisture content, temperature, frost depth, and weather condition at each pavement section. Details of the instrumentation work and the monitored data can be accessed in Liang (2007). The cross section of the as-built pavement sections is shown in Figure 3.1. Each section of the pavement is 500 ft long, with 4-inch thick drainable base in each section except for the section built with ODOT 307-NJ where 6-inch was adopted. All the base types were placed over a 12 inches ODOT 304 subbase layer.
7.4 Material Properties for MEPDG Analysis

Material characterization for hot mix asphalt (HMA), base, subbase, and subgrade layers required for the mechanistic-empirical design approach are presented in this section.

7.4.1 Subgrade Material Properties

The subgrade at the site consisted of either granular sandy soils (A-1b) or silty clay soils (A-6a). The material properties used in the MEPDG analyses are summarized in Table 7.1.

7.4.2 Base/Subbase Course Materials

The gradation curves for ODOT 304, No. 57, 307-IA, 307-CE, and 307-NJ base materials are shown in Figure 7.1. The cement treated base material was made by compacting AASHTO #57 stones with cement at 250 lb per cubic yard at water cement ratio (w/c) of 0.36. The asphalt treated base was made by compacting AASHTO #57 stones with 1.5 to 3.5 percent by weight of PG 64-22 asphalt binder. The subbase material was ODOT 304. The material properties used in the MEPDG analyses were determined from a comprehensive series of laboratory tests by Liang (2007) and they are summarized in Table 7.2.
Table 7.1 Material properties used for the MEPDG analyses for Subgrade soil

<table>
<thead>
<tr>
<th>Item</th>
<th>AASHTO classification of soil</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A-6a</td>
</tr>
<tr>
<td>Poisson's ratio*</td>
<td>0.35</td>
</tr>
<tr>
<td>Coeff. of lateral pressure, $K_0^*$</td>
<td>0.5</td>
</tr>
<tr>
<td>Resilient modulus (psi)</td>
<td>7,500</td>
</tr>
<tr>
<td>Plasticity Index, PI</td>
<td>12.3</td>
</tr>
<tr>
<td>Liquid Limits, LL</td>
<td>30.8</td>
</tr>
<tr>
<td>Passing #200 sieve (%)</td>
<td>68.8</td>
</tr>
<tr>
<td>Passing #4 sieve (%)</td>
<td>94.1</td>
</tr>
<tr>
<td>$D_{50}$ (in)</td>
<td>0.00126</td>
</tr>
<tr>
<td>Maximum dry unit weight</td>
<td>113</td>
</tr>
<tr>
<td>Specific gravity of solids, $G_s$</td>
<td>2.70</td>
</tr>
<tr>
<td>Saturated hydraulic conductivity, ft/hr</td>
<td>$6.5 \times 10^{-6}$</td>
</tr>
<tr>
<td>Optimum gravimetric water content (%)</td>
<td>16.2</td>
</tr>
</tbody>
</table>

Figure 7.1 Gradation curves for ODOT 304 and unbounded base course materials
Table 7.2  Material properties used for the MEPDG analyses for Granular aggregate base

<table>
<thead>
<tr>
<th>Strength properties</th>
<th>ODO 304</th>
<th>ODOT 307-CE</th>
<th>ODOT307-IA</th>
<th>ODOT307-NJ</th>
<th>ODOT 306-Cement</th>
<th>ODOT 308-Asphalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resilient modulus (psi)</td>
<td>46600</td>
<td>46400</td>
<td>40800</td>
<td>43000</td>
<td>258900</td>
<td>113700</td>
</tr>
<tr>
<td>Poisson's ratio*</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Coeff. of lateral pressure, K_o*</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Plasticity Index, PI</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Passing #200 sieve (%)</td>
<td>0</td>
<td>1.0</td>
<td>3</td>
<td>1.0</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Passing #4 sieve (%)</td>
<td>7.5</td>
<td>27.5</td>
<td>40.3</td>
<td>2.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>D_60 (in)</td>
<td>0.858</td>
<td>0.527</td>
<td>0.404</td>
<td>0.320</td>
<td>0.614</td>
<td>0.614</td>
</tr>
<tr>
<td>Maximum dry unit weight (pcf)</td>
<td>130</td>
<td>125</td>
<td>130</td>
<td>127</td>
<td>107</td>
<td>107</td>
</tr>
<tr>
<td>Specific gravity of solids, G_s</td>
<td>2.6</td>
<td>2.6</td>
<td>2.6</td>
<td>2.6</td>
<td>2.64</td>
<td>2.64</td>
</tr>
<tr>
<td>Saturated hydraulic conductivity</td>
<td>59</td>
<td>154</td>
<td>95</td>
<td>159</td>
<td>1056</td>
<td>1044</td>
</tr>
<tr>
<td>Optimum gravimetric water content (%)</td>
<td>3</td>
<td>2.0</td>
<td>2.5</td>
<td>2.0</td>
<td>2.5</td>
<td>2.5</td>
</tr>
</tbody>
</table>

* MEPDG Default Values

7.4.3 Hot Mix Asphalt (HMA)

The asphalt concrete layer consisted of two lifts of Asphalt concrete (ODOT 302), an intermediate course layer (Item 858, Type A.) and a surface course layer (Item 858, Type A.). The total thickness of the asphalt concrete layer was 15.25 inches. The material properties of asphalt concretes were extensively tested by Liang (2007) and are summarized in Table 7.3.

7.5 Climatic Inputs

The influences of climate on pavement performances are fully considered by incorporating the Enhanced Integrated Climatic Model (EICM) in the new MEPDG. The EICM is a one-dimensional coupled heat and moisture flow computer program that simulates changes in the behavior and characteristics of pavement and subgrade materials in conjunction with climatic conditions over several years of operation. Two major types
of inputs are required by EICM. The first type is related to the weather information data obtained from the weather station near the project site. These data include: solar radiation, air temperature, wind speed, wind direction, and rainfall gathered over six years of monitoring. The second information is related to the groundwater table depth which needs to be entered manually to the MEPDG software. Since there is no high groundwater table observed in the preliminary soil investigations at the instrumentation sections at both sites (Liang, 2007), the groundwater table depth in this study was kept constant at 15 ft below the pavement surface for all analyses. The exact location in terms of longitude and latitude and elevation is as follows:

- Latitude: 41.31
- Longitude: -81.41
- Elevation: 606-ft.
Table 7.3 Material properties used for the MEPDG analyses for Asphalt concrete

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Superpave HMA Mix Surface Course Item 442, Type A, 12.5mm</th>
<th>Superpave HMA Mix Intermediate Course Item 442, Type A, 19mm</th>
<th>Marshall Mix Bituminous Aggregate Base Course (Item 302)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference temperature (°F)*</td>
<td>70</td>
<td>70</td>
<td>70</td>
</tr>
<tr>
<td>Poisson's ratio*</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>Air voids (%) **</td>
<td>5.5</td>
<td>5.5</td>
<td>9.5</td>
</tr>
<tr>
<td>Effective binder content (%) ***</td>
<td>11.1</td>
<td>9.6</td>
<td>8.7</td>
</tr>
<tr>
<td>Total unit weight (pcf)</td>
<td>145</td>
<td>145</td>
<td>140</td>
</tr>
<tr>
<td>Cumulative % Retained 3/4 inch sieve</td>
<td>0</td>
<td>1</td>
<td>31</td>
</tr>
<tr>
<td>Cumulative % Retained 3/8 inch sieve</td>
<td>5</td>
<td>24</td>
<td>61</td>
</tr>
<tr>
<td>Cumulative % Retained #4 sieve</td>
<td>24</td>
<td>43</td>
<td>74</td>
</tr>
<tr>
<td>% Passing #200 sieve</td>
<td>4.1</td>
<td>4</td>
<td>3.9</td>
</tr>
<tr>
<td>Thermal conductivity asphalt (BTU/hr-ft-°F)*</td>
<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
</tr>
<tr>
<td>Heat capacity asphalt (BTU/lb-°F)*</td>
<td>0.23</td>
<td>0.23</td>
<td>0.23</td>
</tr>
<tr>
<td>Binder grade</td>
<td>PG 70-22M</td>
<td>PG 64-28</td>
<td>PG 64-22</td>
</tr>
</tbody>
</table>

* MEPDG Default Values.

** Typical air voids based on field compaction specifications and typical field densities obtained from ODOT.

*** Estimated based on VMA and estimated in-place air voids etc.
7.6 Traffic Data

Traffic data is one of the key data elements required for the structural design/analysis of pavement structures. It is required for estimating the loads that are applied to a pavement and the frequency with which those given loads are applied throughout the pavement’s design life. Some of the traffic data in this study were site specific; others were extracted from Ohio LTPP sections. In this study LTPP section 39_5003, located on US-20 in Lorain County, was used. The reason behind the selection of this section to represent the traffic characteristics in this study is its proximity to ATB-90 project site. Table 7.4 summarizes the traffic data used in the MEPDG. Figures 7.2 through 7.7 present the traffic inputs to the new MEPDG.

<table>
<thead>
<tr>
<th>Traffic Input</th>
<th>Description</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADT</td>
<td>21000</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>Truck %</td>
<td>0.45</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>AADTT</td>
<td>9400</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>Truck Design Direction Distribution Factor</td>
<td>50 %</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>Truck Design Lane Distribution Factor</td>
<td>90 %</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>Vehicle Operational Speed</td>
<td>70 mph</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>Functional Classification</td>
<td>Rural Interstate</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>Truck growth factors.</td>
<td>1.55% (Compound)</td>
<td>Site Specific (ATB-90)</td>
</tr>
<tr>
<td>Vehicle (truck) class distribution.</td>
<td>See Figure 7.2</td>
<td>LTPP section 39_5003</td>
</tr>
<tr>
<td>Axle load distribution factors</td>
<td>See Figures 7.3, 7.4, 7.5, and 7.6</td>
<td>LTPP section 39_5003</td>
</tr>
<tr>
<td>Truck lateral distribution factor</td>
<td>See Figure 7.7</td>
<td>LTPP section 39_5003</td>
</tr>
<tr>
<td>Wheel base configurations and tire characteristics and inflation pressure</td>
<td>MEPDG Default</td>
<td>MEPDG Default</td>
</tr>
</tbody>
</table>

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Figure 7.2  AADTT distribution by vehicle class for baseline new HMA pavement section (obtained from LTPP ID 39_5003)

Figure 7.3  Single axle distribution for each truck class (averaged over all months) for baseline new HMA pavement section (obtained from LTPP ID 39_5003)
Figure 7.4 Tandem axle distribution for each truck class (averaged over all months) for baseline new HMA pavement section (obtained from LTPP ID 39_5003)

Figure 7.5 Tridem axle distribution for each truck class (averaged over all months) for baseline new HMA pavement section (obtained from LTPP ID 39_5003)
Figure 7.6 Quad axle distribution for each truck class (averaged over all months) for baseline new HMA pavement section (obtained from LTPP ID 39_5003)

Figure 7.7 Lateral truck wander and mean number axles/truck for baseline new HMA pavement section (obtained from LTPP ID 39_5003)
7.7 Sensitivity Analysis

Sensitivity analysis is the process of varying model input parameters over a practical range and observing the relative change in model response. The input parameters included in the sensitivity analysis are: base type, HMA layer thickness, and subgrade type. The latest version of the MEPDG software (version 1.0) is used in the sensitivity analysis study. The input parameters of interest to be used in the sensitivity analyses are presented in Table 7.5.

### Table 7.5 Input parameters of interest to be used in the sensitivity analysis

<table>
<thead>
<tr>
<th>MEPDG Input Parameter</th>
<th>Levels of Input (*indicates the baseline representative design)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA thickness</td>
<td>4, 6, 8, 10, 12, 14, 15.25*, and 16 inches</td>
</tr>
<tr>
<td>Base type</td>
<td>▪ Dense graded aggregate base course (Item 304)*</td>
</tr>
<tr>
<td></td>
<td>▪ Cement treated free draining base (Item 306)</td>
</tr>
<tr>
<td></td>
<td>▪ Asphalt treated free draining base (Item 308)</td>
</tr>
<tr>
<td></td>
<td>▪ Aggregate base course (Item 307-NJ)</td>
</tr>
<tr>
<td></td>
<td>▪ Aggregate base course (Item 307-IA)</td>
</tr>
<tr>
<td></td>
<td>▪ Aggregate base course (Item 307-CE)</td>
</tr>
<tr>
<td>Subbase Type</td>
<td>▪ Subbase course material (Item 304)*</td>
</tr>
<tr>
<td>Subgrade type</td>
<td>▪ Sandy granular soil (A-1b) with top 12-in compacted</td>
</tr>
<tr>
<td></td>
<td>▪ Silty clay soils (A-6a)* with top 12-in compacted</td>
</tr>
</tbody>
</table>

7.7.1 Base Type Impact on Pavement Performance

Sensitivity analysis was conducted by varying the base material characteristics of the baseline pavement section (ATB-90 pavement section) to determine how changes to the base materials influence the prediction of the following key HMA pavement distress: rutting, alligator cracking, roughness (International Roughness Index, IRI), longitudinal cracking, and low temperature transverse cracking.
A summary of the relative effect of the base type on all distress/IRI is presented in Table 7.6 which shows that base type has little effect on pavement rutting, alligator cracking and IRI. Transverse cracking and longitudinal cracking were not affected by the base type. These observations were expected due to the high HMA thickness (15.25 inches) used at ATB-90 project. The expected life span seems to be greater than 20 years in all pavement sections.

Table 7.6 Relative effects of base types on pavement distresses and IRI

<table>
<thead>
<tr>
<th>Distress/IRI</th>
<th>Effect of Base Type on Distress/IRI</th>
<th>Expected Life Span, Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rutting</td>
<td>Low</td>
<td>&gt; 20</td>
</tr>
<tr>
<td>Alligator cracking</td>
<td>Low</td>
<td>&gt; 20</td>
</tr>
<tr>
<td>IRI</td>
<td>Low</td>
<td>&gt; 20</td>
</tr>
<tr>
<td>Transverse cracking</td>
<td>None</td>
<td>&gt; 20</td>
</tr>
<tr>
<td>Longitudinal cracking</td>
<td>None</td>
<td>&gt; 20</td>
</tr>
</tbody>
</table>

7.7.2 Base Type Impact on HMA Layer Thickness Selection

In this section, the influence of varying base materials on HMA layer thickness was evaluated over two subgrade soils, namely A-6a and A-1b. To this end, a sensitivity analysis was conducted, at each base material type, by varying the HMA thickness over a range of 4 to 16 inches and observing the MEPDG distress/IRI outputs. Figures 7.8 through 7.17 present the pavement distress/IRI for pavement sections constructed with different base materials at different HMA layer thicknesses. Accumulated stress/IRI outputs over a design life span equal to 20 years was used to obtain the previous figures. As shown in these figures, the effect of base materials on pavement performance is more pronounced in pavement sections constructed with thinner HMA layers. For high HMA
thicknesses, little to no effect on pavement performance was observed by changing the base materials. Among base materials, the cement treated base seemed to work the best by reducing the asphalt layer thickness required to satisfy most of the performance criteria.

![Figure 7.8 Plot of HMA thickness versus rutting showing the effect of base type (A-6a Subgrade soils)](image-url)
Figure 7.9  Plot of HMA thickness versus Alligator cracking showing the effect of base type (A-6a Subgrade soils)

Figure 7.10  Plot of HMA thickness versus International Roughness Index (IRI) showing the effect of base type (A-6a Subgrade soils)
Figure 7.11 Plot of HMA thickness versus longitudinal cracking showing the effect of base type (A-6a Subgrade soils)

Figure 7.12 Plot of HMA thickness versus transverse cracking showing the effect of base type (A-6a Subgrade soils)
Figure 7.13  Plot of HMA thickness versus rutting showing the effect of base type (A-1b Subgrade soils)

Figure 7.14  Plot of HMA thickness versus Alligator cracking showing the effect of base type (A-1b Subgrade soils)
Figure 7.15  Plot of HMA thickness versus International Roughness Index (IRI) showing the effect of base type (A-1b Subgrade soils)

Figure 7.16  Plot of HMA thickness versus longitudinal cracking showing the effect of base type (A-1b Subgrade soils)
Table 7.7 summarized the relative effect of HMA thickness on all distress and IRI. Information presented in Table 7.7 shows that rutting is highly influenced by HMA thickness; whereas, alligator cracking, longitudinal cracking and IRI were all highly affected by HMA thickness less than 8 inches. Low temperature transverse cracking was not affected by HMA thickness. This is expected since low temperature transverse cracking is only influenced by the asphalt concrete (AC) material properties with respect to the climatic conditions.

Tables 7.8 and 7.9 summarize the results of the sensitivity analysis for all cross-sections constructed over A-6a and A-1b subgrade soils, respectively. Performance criteria limits, as presented in Table 7.8, were assumed to be the same as the default
values provided by the new MEPDG. From Table 7.8, it can be seen that the total rutting is the key distress which controls the thickness of the HMA layer constructed over A-6a subgrade soil. To satisfy the pavement performance rutting criteria (0.75 inches), a 12 inch HMA layer thickness was found sufficient for ODOT 304, ODOT 307 (NJ, IA and CE), and asphalt treated base materials, while 11 inches were needed for cement treated base. As compared to other base materials, cement treated base required the smallest HMA thickness to fulfill the pavement performance criteria. This was attributed to the high resilient modulus values of the cement treated base layer. From Table 7.9, it can be seen that the longitudinal cracking is the key distress which controls the thickness of the HMA layer constructed over A-1b subgrade soil. To satisfy the longitudinal cracking performance criteria (1000 ft/mile), 10 inches HMA layer thickness was required for ODOT 304, ODOT 307 (NJ, IA and CE), and asphalt treated base materials, while 9 inches were needed for cement treated base. In general, cement treated base required the smallest HMA thickness to fulfill the pavement performance criteria compared with other base materials.

Table 7.7 Relative effect of HMA thickness on pavement distresses and IRI

<table>
<thead>
<tr>
<th>Distress/IRI</th>
<th>Effect of HMA thickness on Distress/IRI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rutting</td>
<td>High</td>
</tr>
<tr>
<td>Alligator cracking</td>
<td>High (HMA thickness &lt; 8 inches)</td>
</tr>
<tr>
<td>IRI</td>
<td>High (HMA thickness &lt; 8 inches)</td>
</tr>
<tr>
<td>Transverse cracking</td>
<td>None</td>
</tr>
<tr>
<td>Longitudinal cracking</td>
<td>High (HMA thickness &lt; 8 inches)</td>
</tr>
</tbody>
</table>
Table 7.8 Hot Mix Asphalt (HMA) layer thicknesses constructed over A-6 subgrade soils

<table>
<thead>
<tr>
<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Rutting (in)</td>
<td>0.75</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>12</td>
<td>11</td>
</tr>
<tr>
<td>Alligator Cracking (%)</td>
<td>25</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>7</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Longitudinal Cracking (ft/mi)</td>
<td>1000</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>Transverse Cracking (ft/mi)</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>IRI (in/mi)</td>
<td>172</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>5.5</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 7.9 Hot Mix Asphalt (HMA) layer thicknesses constructed over A-1b subgrade soils

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Rutting (in)</td>
<td>0.75</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Alligator Cracking (%)</td>
<td>25</td>
<td>7</td>
<td>7</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td>4</td>
</tr>
<tr>
<td>Longitudinal Cracking (ft/mi)</td>
<td>1000</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>Transverse Cracking (ft/mi)</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>IRI (in/mi)</td>
<td>172</td>
<td>5.5</td>
<td>5.5</td>
<td>5.5</td>
<td>5.5</td>
<td>5</td>
<td>4</td>
</tr>
</tbody>
</table>
7.8 Summary and Conclusions

In this study, the impact of using different base materials on pavement performance and asphalt layer thickness has been evaluated using the latest version of the MEPDG software (version 1.0). As a result, the following conclusions were made:

- The performance of pavement structures at ATB-90 project was found insensitive to the type of drainable base materials. This was attributed to the large HMA thickness (15.25 inches) adopted by ODOT at this site, which conceal the effect of different base materials.

- For pavement structures constructed over A-6a and A-1b subgrade soils, the total rutting and longitudinal cracking were found to be the key distress that controls the HMA layer thickness, respectively.

- For pavement sections constructed with ODOT 304, ODOT 307 (NJ, IA, and CE) and asphalt treated base materials, 10 inches and 12 inches of HMA layer thickness were found adequate for pavement sections constructed over A-1b and A-6a subgrade soils, respectively. For pavement section built with cement treated base, 9 inches and 11 inches of HMA layer thickness were found adequate to fulfill the performance criteria.

- Cement treated base seemed to work the best, from providing effective drainage and reducing the asphalt layer thickness to satisfy the pavement performance criteria.
8.1 Summary

Towards the objective of evaluating the effectiveness of Ohio drainable base materials under in-service conditions, extensive filed instrumentations, laboratory works, and numerical simulations have been carried out. A detailed literature review was performed to study the influence of the environmental conditions and pavement drainage systems on pavement long-term performance. Moisture regimes within different pavement sections constructed with different permeable base materials were studied. Accordingly, the impact of using different permeable base materials on moisture variations on unbound layers was examined. The higher the variation is, the lower the effectiveness to drain the infiltrated water in timely manner.

2D FE models simulating both saturated and unsaturated water flow within pavement structures using SEEP/W program was established to better understand the performance of conventional drainage system. A comprehensive parametric study on a representative pavement subsurface drainage system was conducted to exam the effect of different parameters, such as, hydraulic conductivity, pavement geometry (width of the road, distance between the edge of the pavement to the edge drain, transverse slopes, and permeable layer thickness), and the location of edge drain. Based on the results of the FE
parametric study, an empirical equation is proposed that can be used to estimate the time to drain 50% of the infiltrated water. For simplicity, design charts were also presented to estimate the time of 50% saturation.

A new predictive model was proposed to characterize the resilient modulus of cohesive soils using both octahedral stresses environment and soil suction concepts. The accuracy of the new proposed model is validated against laboratory resilient modulus data as well as data gathered from the literature. In addition, a comprehensive statistical analysis was carried out to relate the proposed model k-coefficients with the physical properties of different subgrade soils. Both proposed model and k-coefficients were shown good agreement with the measured resilient modulus.

To validate the capability of the Enhanced Integrated Climatic Model (EICM) software to predict temperature, moisture and frost depth data, environmental data gathered over a period of six years of continuous monitoring at the ATB 90 project site were used. In situ material properties and climatic boundary conditions were also utilized by the EICM software as input parameters. Sensitivity analyses were conducted to evaluate the effect of various input parameters on EICM predictions. The main parameters included in the sensitivity analysis are materials thermal properties (surface short-wave absorptivity, thermal conductivity, and heat capacity), materials porosity, and fine contents (percent passing #200 sieve).

The impact of using different base materials on pavement performance and on the selection of the asphalt layer thickness were evaluated using the latest version of the Mechanistic-Empirical Pavement Design Guide (MEPDG Version 1.0). Pavement performance was analyzed based on pavement rutting, cracking, and roughness (IRI). Site
specific materials, traffic, and climatic data were used whenever it possible. The effect of subgrade strength on pavement performance was also investigated.

8.2 Conclusions

The specific remarks or statements from the work conducted in this research can be mentioned briefly as follows:

- Based on the moisture data monitored over a period of six years, pavement sections built with cement treated and asphalt treated permeable base materials have shown the lowest variation in the gravimetric moisture contents, especially at the top of the subgrade layer. On the other hand, unbound drainable base materials have shown higher seasonal variations in the gravimetric moisture contents.

- Among the base materials used by Ohio State, ODOT 304 and ODOT 307-IA had experienced an increasing trend in their measured moisture content at shallow subgrade depth. Therefore, it is not recommended to use either ODOT 304 or ODOT 307-IA as a drainable base materials in the subsurface drainage design.

- There was no evidence of full saturation in base and subbase materials. However, at shallow subgrade ODOT 304, ODOT 307-IA and ODOT 307-NJ experienced fully saturation conditions for almost 64, 10 and 5 percent of the time, respectively.

- Attempts to correlate the amount of rainfall accumulated over one month with the moisture content were unsuccessful.
According to the COV at the top elevation of the subgrade soils, the effectiveness of different permeable base materials to drain the infiltrated water can be ranked from the best to worst as follows: ODOT 306-cement treated base, ODOT 308-asphalt treated base, ODOT 307-CE, ODOT 307-NJ, ODOT 307-IA, and finally ODOT 304.

The results of the transient SEEP/W numerical simulations revealed that specifying a minimum hydraulic conductivity value for the base materials as a sole controlling factor would not be sufficient to ensure adequate drainage. Factors such as Soil-Water Characteristic Curves (SWCC’s), and hydraulic conductivity variations with degree of saturation, pavement geometry (width of the road, distance between the edge of the pavement to the edge drain, longitudinal and transverse slopes, and permeable layer thickness) are also important.

Specific to ATB-90 pavement design, the saturated hydraulic conductivity of the base materials need to be 1275 ft/day or higher. Base thickness was found insignificant; therefore, 4 inches for constructability was found adequate. A subbase hydraulic conductivity of 28 ft/day is considered to be sufficient enough to increase the efficiency of the base layer to drain the infiltrated water laterally by reducing the amount of water infiltrated downward. The typical values of the cross-slope and shoulder slope (1.6 % and 4 %) were found to be effective in producing a good under-drain system under unsaturated conditions.
The effect of the shallow edge drains on the drainage performance was found more pronounced than the effect of the deep edge drains. However, the deep edge drains should be used to take care of the infiltrated water to the subbase layer and to reduce the chance of the infiltrated water to reach the subgrade layer.

The amount of fines in the permeable base materials is critical in controlling the time-to-drain. The numerical study results suggest limiting the amount of fines (passing 200 sieve) to be within 2 percent by weight.

Based on the parametric study, a new predictive equation along with design charts for estimating the time required to drain 50% saturation were developed.

A new model was proposed to predict the resilient modulus of subgrade cohesive soils using both octahedral stresses and soil matric suction concepts. It was shown that the use of the octahedral stresses and soil matric suction significantly affect the resilient modulus prediction capabilities at different levels of stresses and moisture contents. The accuracy of the new proposed model was validated against laboratory resilient modulus data as well as data gathered from the literature.

Regression models to predict the resilient modulus proposed model k-coefficients from basic soil properties using the multiple linear regression technique available is SPSS program were developed. Statistical analysis indicates that the correlations between model parameters and soil properties yield best results. The validity of the developed k-coefficient models to predict the resilient modulus was examined. A good agreement between the measured and the predicted resilient modulus was observed.
- Base on the environmental data gathered from ATB-90 sites, it was shown that the revisions made in the transition from EICM Version 3.0 to EICM Version 3.2 have substantially enhanced the prediction capability of the model, especially the prediction of the volumetric moisture content in the base and subbase layers.
- The prediction capability of the EICM models is better in the pavement sections built with unbound base layers.
- The salinity of the infiltrated water during the winter season seems to have the main effect on the capability of the EICM models to mimic what is going on in the field. Both upper and lower temperature limits of freezing range have to be adjusted inside the EICM program to get close match in predictions. It was found that by setting the upper and lower freezing temperature limits at 12 °F and 10 °F, respectively, the predicted and measured frost depths could be matched for the pavement sections under this study.
- Based on the sensitivity analysis, EICM Version 3.2 exhibits high to moderate sensitivity to the surface short-wave absorptivity and low sensitivity to the asphalt thermal conductivity and heat capacity. For moisture predictions, the analysis indicated that the EICM 3.2 exhibits high sensitivity to the soil porosity and fines contents.
- Based on the sensitivity analysis, pavement sections were calibrated. The calibrated pavement sections were found in a good agreement with the measured pavement temperatures and moisture contents.
- Based on the MEPDG, the performance of pavement structures at ATB-90 project was found insensitive to the type of drainable base materials. This was attributed
to the large HMA thickness (15.25 inches) adopted by ODOT at this site, which conceals the effect of different base materials.

- For pavement structures constructed over A-6a and A-1b subgrade soils, the total rutting and longitudinal cracking were found to be the key distress that controls the HMA layer thickness, respectively.

- For pavement sections constructed with ODOT 304, ODOT 307 (NJ, IA, and CE) and asphalt treated base materials, 10 inches and 12 inches of HMA layer thickness were found adequate for pavement sections constructed over A-1b and A-6a subgrade soils, respectively. For pavement section built with cement treated base, 9 inches and 11 inches of HMA layer thickness were found adequate to fulfill the performance criteria.

- Cement treated base seemed to work the best, from providing effective drainage and reducing the asphalt layer thickness to satisfy the pavement performance criteria.

8.3 Recommendations for Future Studies

- ATB-90 pavement sections have been in service for almost six years. From environmental data gathered over this time span, many useful observations were made. Nevertheless, it is highly recommended to keep monitoring these sections to determine the long-term performance of drainable base materials.

- ODOT 304 and ODOT 307-IA are performing poorly under the HMA pavement sections at I-90 project site. It appears that these materials lose their drainage efficiency under traffic loading. Since other satisfactory materials are available
for use under flexible pavements, the use of ODOT 304 and ODOT 307-IA base should be ceased.

- The empirical model developed during this research was shown to make significant improvements in the predictions of resilient modulus over existing models. However the size of the study that led to the model was quite small. Additional test data will allow a widening of the range of the material properties and the incorporation of additional materials in the model.

- Soil Water Characteristic Curves (SWCC’s) and hydraulic conductivity functions are essential factors in modeling water flow under unsaturated conditions. Therefore, it is recommended to establish a database that contains these functions for different soil types for future uses, especially in the implementation of the Mechanistic-Empirical Design Guide (MEPDG).

- Since the use of DRIP program to estimate time to drain could be unconservative, it is recommended to use SEEP/W program as an analysis tool to evaluate the pavement subsurface drainage design effectiveness.

- EICM showed very little seasonal variation in the predicted moisture content. Hence, there is a need to improve the prediction capability of the EICM to capture the seasonal changes in soils moisture contents.

- The major set back of the MEPDG procedure is that the software takes 40 minutes for each run and hence setting the time constraints. Therefore, future work should be done to reduce the running time of the MEPDG software.
REFERENCES


