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THREE DIMENSIONAL FINITE ELEMENT MODELING OF PAVEMENT SUBSURFACE DRAINAGE SYSTEMS

Yinhui Liu

University of Kentucky

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THREE DIMENSIONAL FINITE ELEMENT MODELING OF PAVEMENT SUBSURFACE DRAINAGE SYSTEMS

ABSTRACT OF DISSERTATION

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy in the College of Engineering at the University of Kentucky

By
Yinhui Liu

Lexington, Kentucky

Director: Dr. Kamyar C. Mahboub, Professor of Civil Engineering at University of Kentucky

Lexington, Kentucky

2005

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ABSTRACT OF DISSERTATION

THREE DIMENSIONAL FINITE ELEMENT MODELING OF PAVEMENT SUBSURFACE DRAINAGE SYSTEMS

Pavement subsurface drainage systems (PSDS) are designed to drain the entrapped water out of pavement. To investigate the effects of various factors on the performance of PSDS, three dimensional models were developed using the finite element method to simulate the unsaturated drainage process in pavement. The finite element models were calibrated using the field information on outflow, peak flow, layer saturations, and time to drain. Through a series of parametric analyses, the factors that significantly influence the performance of PSDS were screened out, and a set of recommendations were made to improve our current drainage practices.

The effects of pavement geometry on drainage were studied in this research. The analysis results indicate that edgdrain system can significantly improve the drainage efficiency of a pavement. The drainage performance of a pavement is mainly affected by the geometric factors that related to the edgdrain itself and the geometric factors related to the driving lanes have very limited effects.

To investigate the influences of the properties of various pavement materials, some physical-empirical equations were developed in this research. These equations were used to predict the material hydraulic properties from their grain-size distributions and aggregate/asphalt contents. The analysis results of the models with various material properties indicate that the use of permeable base is effective in improving the drainage ability of a pavement. The performance of PSDS is not only affected by material permeability but also by their water
retention ability. The pavement works as an integrated hydraulic system and the hydraulic compatibility of materials must be considered in the PSDS design.

The effects of climatic factors on pavement drainage were also studied in this research. A method was developed in this research to numerically describe the rainfall events. The analysis results of the models under various rainfall events indicate that rainfall duration is a more important parameter than the rainfall quantity in influencing the pavement drainage. Based on the analysis results, regression equations were developed for the estimation of pavement drainage. Finally, for design application purpose, a series of tables were included in this report to help with proper selected of pavement drainage options.

**KEYWORDS:** Pavement Subsurface Drainage, 3-D Finite Element Modeling, Unsaturated Soil Mechanics, Hydraulic Property Prediction, Rainfall Event Description

Yinhui Liu

July 14th, 2005
THREE DIMENSIONAL FINITE ELEMENT MODELING
OF PAVEMENT SUBSURFACE DRAINAGE SYSTEMS

By

Yinhui Liu

Kamyar C. Mahboub, Ph.D., PE
Director of Dissertation

Kamyar C. Mahboub, Ph.D., PE
Director of Graduate Study

July, 14th, 2005
Date
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DISSEMINATION

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for the degree of Doctor of Philosophy in the College of Engineering at the  
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Yinhui Liu  
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Lexington, Kentucky  
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DEDICATION

To My Parents: Weixin Liu and Shurong Yin

My Daughter: Ruth Wu
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CHAPTER 1.0 INTRODUCTION

1.1 The Importance of Pavement Drainage

Pavement is an important component of our transportation infrastructure. It is often designed to withstand a specified volume and weight of traffic for a specified number of years of service. However, the volume and weight of traffic is not the only factors that can cause damage to the pavement. Field data has shown that pavement damage can also be caused by climatic factors, and chief among various factors is water.

Water can enter the pavement through several pathways. These may include infiltration through surface, joints, cracks or roadside ditches; high groundwater; interrupted aquifers; and localized springs. If the water entering into the pavement is not removed in a timely manner, it will reside in the pavement and combining with traffic loads cause detrimental effects on the pavement. These effects can reduce pavement life, increase costs for maintenance, and increase pavement roughness. As indicated by Cedergren, “Damage during the periods with free water present in the structural sections was 100 to 200 times greater than when no free water was present” and “High repair and replacement costs on major highways and airfields are nearly always associated with heavy loads on sections containing free water” (Cedergren, 1974).

The detrimental effects induced by the entrapped water in the pavement include reducing the strength of unbounded granular materials and subgrade soils, causing pumping of fines, and inducing movement over swelling soils. Additionally, continuous contact with water also leads to stripping of asphalt pavement, and fatigue as well as “D” cracking of concrete pavement. In northern climates with a depth of frost penetration greater than the pavement thickness, high water table causes frost heave and the reduction of load-carrying capacity during the thaw periods (Huang, 1993). Heavy wheel loads can apply a water hammer type of force action on saturated bases and subbases. Water action can disintegrate cement-treated bases, and weaken base courses by rearranging the internal structure of fine-grained materials in aggregate mixtures (Cedergren, 1974). In short, the long-term presence of internal water can be responsible for a
majority of serious pavement problems, such as premature rutting, cracking, faulting, increased roughness, and decreased serviceability.

The detrimental effects of water can be minimized by preventing water from entering the pavement, removing entered water by providing subsurface drainage, or building the pavement strong enough to resist the combined effect of load and water. There are several ways to prevent water from entering the pavement structure, such as intercepting groundwater and sealing the pavement surface. However, after decades of various pavement sealing attempts, it has been learned that the present practices do not guarantee a high level of water tightness (Christopher et al., 1997). Even when the pavement is relatively new, significant amounts of water can enter into the pavements. Hagen and Cochran (1996) indicated that sealing the longitudinal and transverse joints of PCC may only temporarily reduce the rain inflow. They reported that after about two weeks, inflow resumed, although the joint sealant appeared to be intact.

Theoretically it might be considered possible to build a pavement sufficiently “stout” to withstand heavy traffic in the presence of water. Unfortunately, water damage persists even in thick pavements, and trying to live with excess water would not be a good option (Cedergren, 1974).

As water can not be prevented from entering into the pavement and pavement can not be constructed “stout” enough to endure the water-induced damage, the removal of the entered water becomes essential. This removal can be realized by providing a pavement subsurface drainage system (PSDS). In 1823, McAdam said “it is the native soil which really supports the weight of traffic; while it is preserved in a dry state it will carry the weight without sinking” (Ridgeway, 1982). Most historical road building documents included references to pavement drainage—such as: “There are just three things necessary for good roads, and they are drainage, drainage, and more drainage” (Cedergren, 1974).

Pavement subsurface drainage systems (PSDS) usually include a permeable base and an edgedrain. Water that has entered into the pavement can be carried to edge drains through a
permeable layer, and then drained out of pavement through outlet pipes. Proper drainage prevents the accumulation of free water in the pavement, thereby reducing the damaging effects of load and environment and maintaining the pavement service life. Results from the field studies on I-70 and I-77 in Ohio indicated that concrete pavement performance is directly related to drainage conditions, and where adequate drainage was provided for base, subbase, and shoulder, the pavement performed superbly even after 15 years (Cedergren, 1974). Based on documented case studies, Cedergren indicated that pavement life can be extended up to three times if adequate subsurface drainage systems are installed and maintained. Forsyth et al. reported a ratio of 2.4 to 1 for reduction of new crack formation in portland cement concrete (PCC) pavement with drainage, compared with pavements without drainage. Forsyth et al. also report at least a 33 percent increase in service life for asphalt pavements and a 50 percent increase for PCC pavement when drainage is provided. Ray and Christory (1997) observed premature pavement distress in an undrained pavement section in France, inferring a reduction in service life of nearly 70 percent, compared with the drained section in the same study. These facts indicate that the presence of an adequate subsurface drainage system in a pavement structure is a critical factor in pavement performance and service life.

1.2 Components of PSDS and Definition of Terms

The primary components of a Pavement Subsurface Drainage System (PSDS) include the pavement surface, a permeable base, a separator/filter layer, the subgrade, and the edge drains (Figure 1.1). The edge drains consist of trench, collection pipe, the outlet pipe and its headwall.
There are two types of pavement surface—flexible pavement or hot mix asphalt (HMA) surface, and rigid pavement or portland cement concrete (PCC). For HMA pavements, precipitation can infiltrate both from asphalt concrete and surface cracks. While for PCC pavement, surface water usually enters pavement through joints and cracks.

Permeable base is a free draining layer in the pavement designed to rapidly remove free water from the pavement. It is usually placed between the surface layer and a separator/filter layer. It may be aggregate or aggregate stabilized with either portland cement or asphalt. The aggregates used in the permeable layer usually are open graded aggregates with a permeability of more than 300 m/day (984 ft/day). The open graded aggregates are always stabilized to increase pavement structural strength. Although this added structural capacity is sometimes ignored in structural design, which has the effect of adding to the structural safety factor.

Separator/filter layer (aggregate or geotextile) is a geotextile or aggregate (subbase) layer separating a permeable base layer from the adjacent subgrade soil or dense graded aggregate (DGA) subbase. The purpose of this filter layer is to prevent clogging of the open drainage layer.
Edgedrain is a subsurface drain system usually located at the edge of the pavement (between the main lane and the shoulder) to intercept infiltrated water. There are two types of edgedrains: aggregate trench drain with geotextile filter pipe and prefabricated geocomposite edgedrain (PGED). In the aggregate trench drain, a rigid or flexible pipe conduit (usually perforated) is designed to collect and/or transport water out of the pavement. The PGED consists of drainage core covered with a geotextile. At the end of edgedrains, a lateral outlet pipe is designed to connect the edgedrain to the outlet, from which water is discharged. Beyond these components, a headwall is always constructed to protect the outlet.

The PSDS encompasses all pavement layers and the whole pavement structure serves as a hydraulic conduit for the entrapped water. This integrated relationship indicates that the conductivity of the pavement is highly dependent on the choice and geometry of various pavement layers.

1.3 Drainage Considerations in Pavement Structure Design

With the recognition of the importance of the pavement drainage system, it has become common to incorporate the drainage system into pavement design. In the 1993 AASHTO design guide which is used by a majority of states in the U.S. for pavement design, the treatment of drainage is through the use of drainage coefficients: \( m_2 \) and \( m_3 \), for asphalt pavements, and a drainage coefficient, called \( C_d \), for portland cement concrete pavements (AASHTO, 1993). These coefficients are determined by the drainage quality of the pavement material and percent time of pavement saturation. It is important to note that as with other 1993 AASHTO parameters, the drainage factors are not mechanical parameters.

The main structural benefit of providing a permeable base in the pavement system is to minimize the potential for occurrence of moisture related distresses such as faulting, pumping, and loss of support that leads to cracking. To account for this benefit, the pavement design models should directly consider the formation and propagation of these distresses over the life of the pavement. This can be accomplished through the implementation of mechanistic models.
and analyses in the design procedure. Currently a new design guide, NCHRP 1-37A, has been developed based on mechanistic-empirical principles (ARA, 2004).

The new design guide (NCHRP 1-37A) adopts a new approach for considering the effects of temperature and moisture regimes in the pavement design process. Under this new method, a procedure is used to preprocess the site-specific factors to develop climatic inputs (temperature and moisture) for the foundation and materials analyses and for pavement response analysis. Of the several procedures available to establish site-specific variations in moisture and temperature regimes, the FHWA's Integrated Climate Model (ICM/EICM) was selected and integrated within the new design guide. The ICM is a comprehensive environmental effect model that incorporates four primary sub-models: precipitation, infiltration and drainage (ID), climate-materials-structure (CMS), and frost heave and thaw settlement. The ICM is a one-dimensional coupled heat and moisture flow model for the analysis of pavement systems subjected to climatic effects, which directly relates structural design to environmental factors (Lytton et al., 1990).

1.4 Current Hydraulic Design and Analysis of PSDS

As mentioned earlier, pavement drainage is influenced by the structural design. Since PSDS consists of all layers of the pavement, there is no doubt that the drainage ability of a pavement is affected by the designed structure. Therefore, pavement structural design and pavement drainage design must be jointly performed.

The fundamental function of PSDS is to provide adequate capacity to ensure a smooth flow of water through the pavement system. For this to happen, the permeable base first should be designed to adequately handle the inflow due to infiltration, which itself is a function of the roadway geometry and surface conditions. Then the edgdrain trenches and pipes should be sized to efficiently handle the discharges from the permeable base. Finally, the outlet pipes and side ditches should be designed to carry the edgdrain discharges.
The Federal Highway Administration (FHWA) has suggested two basic methods for designing permeable bases: depth-of-inflow and time-to-drain (Mallela et al., 2000). Depth-of-inflow method is based on a uniform inflow rate to indicate the amount of water introduced through the pavement joints. This method requires that the outflow rate equal the inflow rate, a condition that can be established by the proper choice of a drainage layer. Calculations for this method are developed from Darcy's steady state equation. Time-to-drain method consists of two stages. For the first stage, the drainage layer is designed to carry the inflow rate under steady state conditions. The phreatic surface is assumed to be a triangle or parabola of zero head at the outlet and of full pavement thickness at the other edge. The layer must have a capacity of at least the inflow, so that the elevation of water can not exceed the thickness of the drainage layer. In the second stage, the rain is assumed to have ceased and the water is freely flowing out of the drainage layer in a non-steady-state condition. This approach selects a specific time interval for obtaining a specified degree of drainage for a saturated base to minimize pumping and erosion.

Although both methods have merit, the depth-of-inflow involves computation of exact amounts of infiltration into the pavement and can lead to conservative estimates of permeable base thicknesses (Richardson, 2001). Current practice is to use the time-to-drain method. The current recommendation is to provide a 100mm (4 inch) permeable base with an adequate permeability (minimum 300m/day (984 ft/day)) to meet the time-to-drain criteria. The final value of permeability is a function of the time required to drain a saturated base layer for a given roadway geometry. The AASHTO classification of permeable base quality is based on the time required to drain the base from a 100 percent saturation to a 50 percent saturation level and involves subjective ratings such as “excellent” (time to drain less than 2 hours), “good” (time to drain less than 1 day), “fair” (time to drain less than 7 days), and so on. In this report, a high-type pavement is expected to have an “excellent” quality of drainage. The FHWA computer program DRIP (Drainage Requirements In Pavements) has been developed to performing the time-to-drain design (Wyatt et al., 1998).

The primary purpose of a permeable base is to drain the water entering the pavement system as quickly as possible. Intuitively, the more open-graded a material, the higher its probability
of meeting the desired objective. However, the more open-graded a material, the lower its stability would be. To achieve an optimum design solution, the stability and drainability must be balanced to a certain level, and the background theory and criterion of structural response and hydraulic principle must be critically reviewed.

1.5 PSDS Research Status and Problem Statement

Before the advent of rational methods for designing pavements, there was an almost unanimous agreement among road builders that pavements should be kept in a well drained state. The traditional rational design methods put a greater weight on traffic than environmental factors. This resulted in drainage deficiencies in pavements constructed in the US during 1950 to 1970, which ultimately led to many premature failures (Cedergren, 1988). The short service life and unsatisfied service condition of these roads taught pavement researchers an important lesson that no matter what method is used in the design, water is always the greatest enemy of pavement (Cedergren, 1974). After realizing the importance of drainage to pavements, FHWA and state Departments of Transportation (DOTs) have devoted major efforts to researching drainage design and analyses. The research topics included the test and analysis of inflow, permeable base design and evaluation, material hydraulic parameter test and prediction, drainage system performance modeling, and the effectiveness evaluation of current drainage structures.

With recognizing the negative effects of entrapped water on pavement performance, various drainage practices have been designed and implemented into pavement constructions. An important task for pavement researchers is to evaluate the performance of these drainage practices and develop performance indicators that can be integrated with pavement design and pavement management practices. Recent field performance data on the effect of pavement drainage on pavement performance have been mixed. For example, a recent study of the Long-Term-Pavement-Performance (LTPP) database suggested that neither permeable bases nor edge drains were found to significantly reduce faulting of jointed concrete pavement. The overall conclusion was that incorporating drainage features into pavement did not necessarily result in improved pavement performance (Hall et al., 2003). These conclusions indicate that it is necessary to re-examine the whole pavement drainage design concept.
The primary purpose of the permeable base is to remove infiltration water as quick as possible. To meet the hydraulic requirements, materials used in permeable layer must have a high permeability, which can be reached by eliminating the fine content (passing No. 200 sieve). However, it has long been recognized that proper gradation and density are vital to the stability of granular materials. The highest permeability materials often are unstable under construction traffic; therefore, it is often desirable to use a more stable material, which is often dense and has a lower permeability. The key is to balance permeability and structural strength.

In LTPP program, the SPS1 and SPS2 projects have been designed to investigate the effectiveness of the PSDS, and statistical analyses have been conducted to quantify the effects of PSDS as well as traffic and climate factors (Hall et al., 2003). Pavement drainage performance is often masked by construction and maintenance variables. Therefore, parametric studies are not very effective through the use of field data. Consequently, computer modeling is often a better approach. However, a computer generated model must be calibrated using the field data. When pavement subsurface drainage is modeled, some important input factors must be examined and the critical flow pattern must be simulated. The input factors encompass the modeling of inflow, identification of material properties, and configuration of pavement geometries.

The major sources of pavement inflow of water are surface infiltration, groundwater seepage, and meltwater from ice lenses. When the water table is very low, surface infiltration is the only source of water flowing into the pavement. The design methods for PSDS usually assume that some percentage of the rainfall will infiltrate the pavement surface or joints. However, there is much discussion concerning the likely magnitude of infiltration and the effects of cracking and raveling along longitudinal joints on water infiltration. Existing models for the flow of water through pavement systems, such as the infiltration-drainage (ID) model in ICM/EICM model, usually consider only the infiltration through cracks in the pavement system, ignoring flow through the less-porous intact pavement (Lytton et al., 1990). Therefore, improvements in the measurement and modeling of the infiltration process are needed.

Permeability characterization of pavement materials is a major research topic. As mentioned earlier, the materials in a pavement may include treated or untreated dense or open
graded aggregate, soil, and HMA or PCC pavement. The coefficient of permeability of these materials ranged from as low as $10^{-6}$ m/day ($3.3 \times 10^{-6}$ ft/day) to $10^{5}$ m/day ($3.3 \times 10^{5}$ ft/day). Permeability is measured using Darcy’s Law under saturation condition. While one-hundred percent saturation is rarely reached in the field, the field permeability of materials is somewhat different than the lab test results. Some devices have been designed to test the in-situ permeability of the pavement (Lindly et al., 1995). The permeability test is time consuming and the test results are very sensitive to the apparatus conditions, size of test specimens, as well as operators. Currently, there are some regression models available for estimation of pavement material permeability.

The geometry of the pavement section is often complex and sometimes the construction deviates from design documents. It is difficult to accurately estimate the drainage paths for all sections of the roadway, and conservative assumptions must be made. In the current drainage analysis method the effects of pavement geometries have been considered. To maintain positive flow through the base, it is recommended by FHWA that the road section should be sloped as much as possible, with a recommended minimum cross slope of 2% (AASHTO, 1993). A minimum of 100 mm thickness is needed and the space between the edgerain outlets should be between 50-100 m (Christopher et al., 1997). When the pavement is considered as a whole hydraulic system, not only transverse slope but also the longitudinal slope should be included into the drainage design. The effects of geometry factors, such as: pavement slopes, thickness, length, as well as edgerain location, outlet slope, and pipe diameters, should be investigated and quantified.

Material characterization of a pavement starts with a model and some assumptions. For example, the FHWA time-to-drain method is a 2-D model developed from Darcy’s steady state equation with assumption that the shape of the preatic surface of flow is either a triangle or a parabola. This design method is based on the seepage flow analysis under saturated condition. However, recent studies suggest that the performance of PSDS can only be understood if unsaturated flow is considered (Birgisson et al., 2000). Furthermore, in current drainage analysis, the detrimental effect of water is only considered when the subgrade is fully saturated. However, unsaturated flow may have a significant impact on pavement drainage performance,
and design improvements may be realized if unsaturated flow is considered (Stormont et al., 2001).

For the modeling and analysis of a hydraulic system as complex as PSDS, finite element method can be a useful and powerful tool. Finite element method has been widely used in the modeling of groundwater flow in geotechnical engineering. Two equations govern the flow through porous media: Darcy’s law and continuity equation (mass conservation). The finite element model can incorporate both the saturated and unsaturated processes, and can include variable material properties. Currently, 2-D unsaturated analysis has been used in pavement drainage analysis (Hassan et al., 2001; Stormont et al., 2001). However, the scope of these analyses is very limited. The water flow in pavement is affected by factors such as: pavement geometry and edgedrain system, and unfortunately a 2-D model can not demonstrate the flow path along the longitudinal direction and the water content condition of different cross-sections. Therefore, a thorough research of the pavement drainage characteristics using a 3-D model is highly desirable. Until now, 3-D modeling of unsaturated groundwater flow has not been applied to pavement flow analysis problems because of demanding computer hardware and software requirements. Furthermore, to conduct the unsaturated flow analysis, water retention characteristics (Soil Water Characteristic Curve (SWCC) in unsaturated soil mechanics) are needed. However, at present time, there is very little SWCC data available for the materials used in pavement.

1.6 Objectives and Scope of Work

The objectives of this research are to explore the intrinsic mechanism that control the water flow in pavement; to examine the effects of various factors, such as geometry, inflow, and material properties; and to develop a tool that can be used for pavement drainage system performance prediction.

The 3-D finite element software for partial differential equation, otherwise known as: FlexPDE, will be used to analyze the drainage in pavement (FlexPDE, 2001). FlexPDE includes both steady state analysis and transient analysis, and it can handle both saturated as well
as unsaturated groundwater flow conditions. By conducting the unsaturated transient analyses, the drainage characteristics corresponding to various drainage design practices will be identified, and the quantifiable measures of pavement performance as related to drainage will be provided. Through a series of sensitivity analyses the factors that mask the effects of subsurface drainage system will be recognized and eliminated. Thereafter, a mathematical model describing the relationship between the hydraulic parameters and various influencing factors will be established.

To meet these objectives, the following tasks were developed:

- Develop 3-D finite element model by using the FlexPDE program
  - Identify governing equations for 3-D groundwater flow
  - Identify material properties and their estimations
  - Identify geometry parameters
  - Investigate boundary conditions and assumptions
  - Model calibration by using field data
- Research on the effects of geometry parameters
  - Select control conditions
  - Analyze the effect of pavement slopes (transverse, longitudinal, outlet pipe)
  - Analyze the effects of pavement width and length
  - Analyze the effects of layer thickness
  - Analyze the effects of trench and pipe
  - Effect of outlet quantity and positioning
- Research on effects of materials properties
  - Select and calibrate permeability prediction models for pavement materials
  - Select and calibrate SWCC prediction methods for pavement materials
  - Select and calibrate hydraulic conductivity prediction models for pavement materials
  - Investigate effect of material properties of each layer and trench.
- Develop a mathematical model for pavement drainage performance evaluation
  - Rainfall event characterization
- Analyze the effects of rainfall on pavement drainage
- Analyze the effects of pavement surface condition on pavement drainage
- Analyze the effects of pavement structure on drainage
- Create an indicator for pavement drainage evaluation
CHAPTER 2.0 FINITE ELEMENT MODELING OF PAVEMENT SUBSURFACE DRAINAGE SYSTEM

Modeling of flow through pavement with a numerical solution can be very complex because of the variability of material properties, geometries, and boundary conditions that can be selected. These complexities make it necessary to use some form of numerical method to analyze the water flow in pavement. Among the numerical methods, finite element method is a suitable and powerful approach for solving this complex problem.

2.1 Finite Element Method (FEM)

FEM is a powerful and general approach for solving partial differential equations. It has been used successfully to analyze complex engineering and physical systems. Typical areas of applications include structural analysis, thermal analysis for heat transfer, electromagnetic analysis, fluid analysis, etc. FEM consists of defining a solution that satisfies the partial differential equation on average over a finite element. Every element is connected to a neighboring element, and the field under study is analyzed by propagating the current values from one element to another through connection points. Formulation and application of FEM are considered to consist of eight basic steps:

a. Discretize and select element configuration;
b. Select approximation model or function;
c. Define gradient-unknown and constitutive relationships;
d. Derive element equations governing the behavior of the element;
e. Assemble element equations to obtain global or assemblage equations and introduce boundary conditions;
f. Solve for the primary unknowns;
g. Solve for derived or secondary quantities; and
h. Interpretation of results.
2.2 An Introduction to FlexPDE Program

The FlexPDE program is a scripted finite element model builder and numerical solver. This means that from a script written by the user, the FlexPDE program performs the operations necessary to turn a description of a partial differential equations system into a finite element model, solve the system, and present graphical output of the results. The FlexPDE program is also a problem solving environment, because it performs the entire range of functions necessary to solve partial differential equation systems: an editor for preparing scripts, a mesh generator for building finite element meshes, a finite element solver to find solutions, and a graphic system to plot results. FlexPDE has no pre-wired problem domain or equation list. The choice of partial differential equations is totally up to the user. The scripting language allows the user to describe the mathematics of the partial differential equations system and the geometry of the problem domain in a format similar to the way one might describe it to a co-worker (FlexPDE, 2001).

The FlexPDE can solve systems of first- or second-order partial differential equations in Cartesian or axisymmetric two-dimensional geometry or in three-dimensional Cartesian geometry. The system may be steady-state or time-dependent, or alternatively FlexPDE can solve eigenvalue problems. Steady-state and time-dependent equations can be mix in a single problem. Any number of simultaneous equations can be solved, subject to the limitations of the computer on which FlexPDE is executed. The equation can be linear or nonlinear. Nonlinear systems are solved by applying a modified Newton-Raphson iteration process. Any number of regions of differential material properties may be defined. Modeled variables are assumed to be continuous across material interfaces. Jump conditions on derivatives follow from the statement of the PDE system. FlexPDE is a fully integrated partial differential equation (PDE) solver, combining several modules to provide a complete problem solving system (FlexPDE, 2001):

a. A script editing module provides a full text editing facility and a graphic domain preview.

b. A symbolic equation analyzer expands defined parameters and relations, it performs spatial differentiation, and it symbolically applies integration by parts to reduce second
order terms to create symbolic Glerkin equations. It then differentiates these equations to form the Jacobian coupling matrix.

c. A mesh generation module constructs a triangular finite element mesh over an arbitrary two-dimensional problem domain. In three-dimensional problems, the 2D mesh is extruded into a tetrahedral mesh covering an arbitrary number of non-planar layers in the extrusion dimension.

d. A finite element numerical analysis module selects an appropriate solution scheme for steady-state, time-dependent or eigenvalue problems, with separate procedures for linear and nonlinear systems. Finite element basis may be either quadratic or cubic. In this research, quadratic finite element was used.

e. An error estimation procedure measures the adequacy of the mesh and refines the mesh wherever the error is large. The system iterates the mesh refinement and solution until a user-defined error tolerance is achieved. In this research, the spatial error tolerance was set to be 0.001 and the time error was set to be 0.01. To meet these error tolerances, approximately 10,000 nodes were generated for the PSDS modeling in this research.

f. A graphical output module accepts arbitrary algebraic functions of the solution and plots contour, surface, vector or elevation plots.

g. A data export module can write text reports in many formats, including simple table, full finite element mesh data.

A problem description script is a readable text file. The contents of the file consist of a number of sections, each identified by a header. The most frequently used sections are:

a. TITLE—a descriptive label for the output.
b. SELECT—user controls over the default behavior of FlexPDE
c. VARIABLES—here the dependent variables are named.
d. DEFINITIONS—useful parameters, relationships or functions are defined.
e. EQUATIONS—each variable is associated with a partial differential equation.
f. INITIAL VALUES—starting values for nonlinear or time-dependent problems.
g. BOUNDARIES—the geometry is described by walking the perimeter of the domain, stringing together line or arc segments to bind the figure.
h. PLOTS—the desired graphical outputs are listed. Plots may be any combination of CONTOUR, SURFACE, ELEVATION, or VECTOR plots.

This scripted form has many advantages. The script completely describes the equation system and problem domain, so there is no uncertainty about what equations are being solved, as might be the case with a fixed-application program. New variables, new equations or new terms may be introduced. Considering these advantages, the FlexPDE program was selected for the finite element modeling of the PSDS system. An example of the FlexPDE script that was developed for a pavement model is presented in Appendix A in this report.

2.3 Considerations in PSDS Modeling

Modeling of water flow through pavement with a numerical solution can be very complex because of the variability of material properties, geometries, and boundary conditions. The following problems must be considered for the PSDS modeling.

When simulating the pavement drainage flow, the first problem needed to be considered is determination of the governing equation. For groundwater flow, the Darcy’s law and the continuity equation are the two governing equations. In the past, the analyses related to groundwater have concentrated on saturated flow. However, recent studies suggest that the performance of PSDS can be better understood if unsaturated flow principles are considered (Birgisson et al., 2001). Thus, in the modeling, the governing equations in the form of unsaturated flow should be applied.

Under the pavement surface, a certain thickness of treated or untreated aggregates is placed as base or subbase to endure the compressive load transferred from the surface. The soil below the subbase works as a bed for those upper layers. Thus, the materials used in a pavement range from fine clay soil to coarse as gravel, to cohesive as asphalt and portland cement concrete. In addition, due to the variability in construction conditions, material properties within each layer may also be heterogeneous and non-isotropic. To get an accurate estimation, the material properties of various pavement layers must be correctly estimated. Furthermore, when a
pavement is under unsaturated condition, the coefficient of permeability or hydraulic conductivity becomes a function of the negative pore-water pressure in the pavement. The pressure is the primary unknown and it needs to be determined, so iterative numerical techniques are required to match the computed pore-water pressure and the material property, which makes the solution highly non-linear.

For water flow modeling, geometry of pavement is an important influence factor. The longitudinal and transverse slopes of a pavement always change with the topography and highway safety requirements. The thickness of layers always changes with the designed traffic load or purpose of use. When the edge drain system is incorporated into the pavement, the water flow in the trench and collection pipe should be characterized. The collection pipe in the trench is placed in the longitudinal direction, which makes the water flow in pavement drainage system is no longer a 2-D flow, but rather a 3-D flow. Therefore, it is difficult to accurately estimate the drainage path for a pavement sections if only the 2-D model is applied. Compared to a 2-D model, the 3-D model can analyze a section with a realistic geometry and consequently provide a more accurate estimation of the field condition.

In addition to consideration of material properties and geometries, boundary condition is another factor that will increase the complexity of the drainage modeling. The boundary condition on pavement surface is the most important boundary condition because it directly related to the water inflow of pavement. However, the boundary condition on pavement surface will change with time. On the other hand, pavement surface is not always homogeneous because of cracks and joints. The cracks and joints make it very difficult to determine the exact boundary condition on pavement surface. The boundary condition at the end of the outlet pipe is an important factor that controls the outflow for a pavement. To simulate the free flow at this point, a pressure check must be made in the model.

2.4 Parameters and Governing Equations

In pavement drainage analyses, flow quantity and material water content is often considered to be the key parameters in quantifying pavement drainage ability. When unsaturated condition
is considered for the pavement, one more important parameter, pore-water pressure, should be introduced. Research in the last few decades has shown that even the flow of moisture in the unsaturated soil near the ground surface is directly related to the soil suction (negative water pressure). The prediction of pore-water pressure is important to the unsaturated analyses because this information is required for prediction of the volume change and shear strength associated with the flow of water or air.

The slow movement of water through porous media is commonly referred to as seepage or percolation. Seepage analyses involve the computation of the rate and direction of water flow and the pore-water-pressure distributions within the flow regime. Seepage problems are usually categorized as steady-state or unsteady state flow analyses. For steady state analyses, the hydraulic head and coefficient of permeability at any point in the soil mass remain constant with respect to time. For unsteady-state flow analyses, the hydraulic head (and possibly the coefficient of permeability) change with respect to time. Changes are usually in response to a change in the boundary conditions with respect to time.

The flow of water through a porous media is driven by a hydraulic head. The hydraulic head consists of three components, namely, the gravitational head, the pressure head, and the velocity head. The velocity head is negligible in comparison with the gravitational and the pressure heads (Fredlund et al., 1993). Thus, the hydraulic head at any point in the porous media can be expressed as

\[ h = y + \frac{u_w}{\rho_w g} = y + \frac{u_w}{\gamma_w} \]  

(Equation 2.1)

where

- \( h \) = hydraulic head,
- \( y \) = gravitational head,
- \( u_w \) = pore water pressure,
- \( \rho_w \) = density of water, and
\( \gamma_w \) = unit weight of water.

The heads expressed in Equation 2.1 have the dimension of length. Hydraulic head is a measurable quantity, the gradient of which causes flow through saturated or unsaturated media.

The flow of water in a saturated soil is commonly described using Darcy’s Law (Darcy, 1856). Darcy postulated that the rate of water flow through a soil mass was proportional to the hydraulic head gradient as follows:

\[
v_x = -k_x \frac{dh}{dx}\]

(Equation 2.2)

where

- \( v_x \) = flow rate of water,
- \( k_x \) = coefficient of permeability with respect to the water phase, and
- \( \frac{dh}{dx} \) = hydraulic head gradient in the x-direction.

Equation 2.2 can also be written for the y- and z-directions. The coefficient of proportionality between the flow rate of water and the hydraulic gradients is called the coefficient of permeability in x-direction, \( k_x \). The coefficient of permeability is relatively constant for a specific saturated soil. The negative sign in Equation 2.2 indicates that water flows in the direction of a decreasing hydraulic head.

Darcy’s law also applies to the flow of water through an unsaturated soil (Fredlund et al., 1993). Water can be visualized as flowing only through the pore space filled with water. The air-filled pores are nonconductive channels to the flow of water. Therefore, the air-filled pores in an unsaturated soil can be considered as behaving similarly to the solid phase, and the soil can be treated as a saturated soil having reduced water content (Childs, 1969). However, the coefficient of permeability in an unsaturated soil cannot generally be assumed to be constant. Rather, the coefficient of permeability (hydraulic conductivity) is a variable which is
predominantly a function of the water content or the pore-water pressure (matric suction) of the unsaturated soil. Figure 2.1 presents a curve showing a typical relationship between the coefficient of permeability and the pore-water pressure.

![Figure 2.1 A hydraulic Conductivity Function.](image1)

As the laboratory test of hydraulic conductivity under unsaturated condition is very complicated, techniques have been developed for predicting the hydraulic conductivity function from a soil-water characteristic function. The soil-water characteristic function describes the relationship between the pore-water pressure and the amount of water stored or retained in the soil structure as water flows through soil. Figure 2.2 is an illustration of this relationship.

![Figure 2.2 A Soil-water Characteristic Function.](image2)
The amount of water stored is usually specified as a ratio of the total volume. This ratio is known as the volumetric water content. In equation form:

\[ \theta_w = \frac{V_w}{V} \]  

(Equation 2.3)

where

\[ \theta_w = \text{volumetric water content}, \]
\[ V_w = \text{volume of water}, \] and
\[ V = \text{total volume}. \]

When the degree of saturation is 100%, the volumetric water content is equivalent to the soil porosity, \( n \), which is defined as the volume of voids divided by the total volume.

Fundamental to the formulation of a general seepage analysis is an understanding of the relationship between pore-water pressure and water content. In the soil-water characteristic curve (SWCC), the slope \( m_w \) represents the rate of change in the amount of water retained by the soil in response to a change in pore-water pressure, which can be expressed using the following function.

\[ m_w = -\frac{\Delta \theta_w}{\Delta(u_a - u_w)} \]  

(Equation 2.4)

where

\[ m_w = \text{slope of SWCC}, \] and
\[ u_a = \text{air pressure}. \]

For constant \( u_a \), Equation 2.4 becomes
By the chain rule in differentiation,

\[
\frac{\partial \theta_w}{\partial t} = \frac{\partial \theta_w}{\partial u_w} \frac{\partial u_w}{\partial t} = \left| m_w \right| \frac{\partial u_w}{\partial t}
\]

(Equation 2.6)

From the definition of hydraulic head (Equation 2.1), one can get the following:

\[
\frac{\partial h}{\partial t} = \frac{1}{\gamma_w} \frac{\partial u_w}{\partial t}
\]

(Equation 2.7)

Substituting Equation 2.7 into Equation 2.6, Equation 2.8 can be derived as:

\[
\frac{\partial \theta_w}{\partial t} = \left| m_w \right| \gamma_w \frac{\partial h}{\partial t}
\]

(Equation 2.8)

The governing partial differential seepage equations are derived in a manner consistent with the conservation of mass. The conservation of mass for a general seepage of an incompressible fluid dictates that the water content increase rate of an element must equal the net flux of that element. In other words, at any point in the soil mass the water content increase must equal to the water flow into that point minus the water flow out of that point. The quantity of flow of an incompressible fluid is expressed in terms of a flux, \( q \). Flux is equal to a flow rate, \( v \), multiplied by a cross-sectional area, \( A \). Figure 2.3 shows a cubical soil element with water flow in the \( x \), \( y \), \( z \) direction. The soil element has infinitesimal dimensions of \( dx \), \( dy \), and \( dz \). The flow rates, \( v_x \), \( v_y \), and \( v_z \), are assumed to be positive when water flows in the positive \( x \), \( y \), \( z \) directions.
Continuity for three-dimensional flow can be satisfied as follows:

\[
\left( v_x + \frac{\partial v_x}{\partial x} \right) dydzdt + \left( v_y + \frac{\partial v_y}{\partial y} \right) dzdxdt \\
+ \left( v_z + \frac{\partial v_z}{\partial z} \right) dxdydt - v_x dydzdt - v_y dzdxdt - v_z dxdydt = \frac{\partial \theta_w}{\partial t} dxdydzdt
\]  

(Equation 2.9)

This equation can be simplified as

\[
\left( \frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} \right) = \frac{\partial \theta_w}{\partial t}
\]  

(Equation 2.10)

Substituting Equation 2.8 into Equation 2.10, one can get the following:

\[
\left( \frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} \right) = |m_w| \gamma_w \frac{dh}{dt}
\]  

(Equation 2.11)

Applying Darcy’s Law to the continuity equation, one can get:
\[
\left\{ \frac{\partial}{\partial x} \left( k_x \frac{dh}{dx} \right) + \frac{\partial}{\partial y} \left( k_y \frac{dh}{dy} \right) + \frac{\partial}{\partial z} \left( k_z \frac{dh}{dz} \right) \right\} = |m_w| \gamma_w \frac{dh}{dt} \quad \text{(Equation 2.12)}
\]

where \( k_x, k_y, k_z \) represent hydraulic conductivities in \( x, y, z \) directions, respectively.

The Equation 2.12 (Fredlund et al., 1993; Hardin, 2001) is a general governing differential equation for seepage problems. It can be used to unsteady state analyses for both saturated and unsaturated conditions. In unsaturated, unsteady state analysis water head and water content change with time, and the coefficients of permeability always vary with coordinates and water content. Therefore, the unsaturated analysis regimes are heterogeneous. For the saturated condition, the slope of the SWCC is very small, which means the water content increase tends to be zero, and the coefficients of permeability are almost constant.

For steady state problems, the water head is constant respect to time, \( t \). Thus, \( \frac{dh}{dt} = 0 \) and Equation 2.12 reduces to the following:

\[
\left\{ \frac{\partial}{\partial x} \left( k_x \frac{dh}{dx} \right) + \frac{\partial}{\partial y} \left( k_y \frac{dh}{dy} \right) + \frac{\partial}{\partial z} \left( k_z \frac{dh}{dz} \right) \right\} = 0 \quad \text{(Equation 2.13)}
\]

If the coefficient of permeability varies with coordinates, the analysis regime is heterogeneous; otherwise, it is homogeneous. If the material has same coefficient of permeability in all directions, it is isotropic; otherwise, it is anisotropic. In this research, the isotropic material properties were applied. This means \( k_x = k_y = k_z \).

In this research, Equation 2.13 was applied to the steady-state analysis and Equation 2.12 was applied to the unsteady-state analysis when writing FlexPDE scripts. The outputs of steady-state analysis are usually used as initial conditions to the unsteady state analysis for the sake of convergence and stability of calculation.
### 2.5 Definition and Calculation of Material Properties

In the governing differential equation of seepage analysis, the coefficient of permeability and slope of SWCC of porous media, and unit weight of water are the material properties that control the characteristic of the water flow. The material properties of porous media related to the calculation of water content include: unit weight, dry density or apparent density, specific density, porosity or void ratio, volumetric or gravimetric water content or degree of saturation.

To simplify the model, the water flow is assumed to occur under normal temperatures, at which the unit weight of water has a constant value of 1000 kg/m$^3$ (62.4 lbs/ft$^3$) and the viscosity of water is very low. The effect of freeze is not considered in this research.

Unit weight, specific density and dry density of engineering materials are easy to be measured from regular laboratory test or predicted from documentation. The porosity, void ratio, volumetric water content, gravimetric water content, and degree of saturation can be derived from the following definitions and their relationships (Fredlund et al., 1993):

\[
n = \frac{V_{\text{void}}}{V_{\text{total}}} = 1 - \frac{\gamma_s}{\rho_s g} = \frac{e}{1 + e} \quad \text{(Equation 2.14)}
\]

\[
e = \frac{V_{\text{void}}}{V_{\text{solid}}} = \frac{n}{1 - n} = \frac{\rho_s g}{\gamma_s} - 1 \quad \text{(Equation 2.15)}
\]

\[
S = \frac{V_{\text{water}}}{V_{\text{void}}} = \frac{\theta_w}{n} \quad \text{(Equation 2.16)}
\]

\[
w = \frac{M_{\text{water}}}{M_{\text{solid}}} = \frac{Se}{G_s} \quad \text{(Equation 2.17)}
\]

where

\[
n = \text{porosity},
\]
e = void ratio,
S = degree of saturation,
θ_v = volumetric water content,
w = gravimetric water content,
γ_s = unit weight of solid,
ρ_s = dry density of solid, and
G_s = specific density of solid.

The coefficient of permeability (hydraulic conductivity) defined by Darcy’s Law is the most important parameter in the analysis of water flow in porous media. As mentioned before, the coefficient of permeability is a function of pore-water pressure and related to the water content directly.

Under saturated condition, all voids in the solid-phase have been filled by water and serve as a collection of conduits to water flow. Thus, the material has highest coefficient of permeability when saturated. Coefficients of permeability for various materials have been tested and cataloged by researchers (Richardson, 1997; Lindley, 1995; Randolph, 1996). Correlations have been developed to predict permeability of materials based upon its gradation, void content or void size distribution, and other parameters.

Laboratory tests for hydraulic conductivity under unsaturated condition are very complicated. Techniques have been developed for predicting the hydraulic conductivity function from a soil-water characteristic function. In the process of the conductivity-function prediction, the tested soil-water characteristic data is fitted by using several parameters first, and then these fitting parameters are substituted into some kind of equation to get the specific conductivity function for this specific material. In this regard, several empirical equations have been proposed to simulate the SWCC, such as Brooks and Corvey equation (Brook and Corvey, 1964), Van Genuchten equation (Van Genuchten, 1980), and Fledlund and Xing equation (Fredlund and Xing, 1994 (a)). Equations developed for the prediction of conductivity function from fitted SWCC include Campbell equation (Campbell, 1973), Van Genuchten closed-form
equation (Van Genuchten, 1980), and Lenog and Rahardjo equation (Lenog and Rahardjo, 1997 (b)). In this research, the Fledlund and Xing (1994 (a)) equation was selected for the SWCC fit and Leong and Rahardjo (1997(b)) equation was selected for the prediction of hydraulic conductivity under unsaturated conditions. These methods were selected because they provide a better fit for the SWCC and conductivity function for all soil. These curves keep smooth under a wide range of pore-water pressure. This property is very important for the convergence when solving finite element equations.

The Fledlund and Xing equation is expressed as follows (Fredlund et al., 1994(a)):

\[
\theta(\psi, a, n, m) = \theta_s \left\{ \frac{1}{\ln \left[ e + \left( \frac{\psi}{a} \right)^e \right]^m} \right\} 
\]

(Equation 2.18)

where

- \( \theta \) = volumetric water content,
- \( \psi \) = matric suction at the calculation point (kPa),
- \( \theta_s \) = saturated volumetric water content, which is equal to void ratio of solid,
- \( e \) = natural number, 2.71828, and
- \( a, n, m \) = fitting parameters.

The Leong and Rahardjo equation is expressed as (Leong et al., 1997(b)):

\[
k(\Psi) = k_s \left\{ \frac{1}{\ln \left[ e + \left( \frac{\Psi}{a} \right)^n \right]^m} \right\}^p
\]

(Equation 2.19)
where
\[ k = \text{coefficient of permeability or hydraulic conductivity}, \]
\[ \psi = \text{matric suction at the calculation point (kPa)}, \]
\[ k_s = \text{saturated coefficient of permeability}, \]
\[ e = \text{natural number, 2.71828}, \]
\[ a, n, m = \text{Fredlund and Xing fit parameters, and} \]
\[ p = \text{Leong parameter}. \]

2.6 Boundary Condition Identification

Specifying boundary conditions of a problem is one of the key components of a numerical analysis. Without proper boundary conditions, the finite element equation cannot be solved. Therefore, in order to obtain meaningful results, it is essential to thoroughly understand the physical significance of different types of boundary conditions and properly specify the boundary conditions of various elements.

Basically, there are three kinds of boundary conditions in the finite element method. The boundary condition that does not contain derivatives of the primary unknown are called forced boundary conditions. Forced boundary conditions are also known as geometric, Dirichlet type, or first kind boundary conditions. The boundary condition that contains derivatives of the unknown function is called natural boundary condition. The natural boundary conditions are also known as gradient, Neumann type, or second kind boundary conditions. If in a boundary more than one type of boundary condition is specified, then that boundary condition is called mixed or third kind boundary conditions (Desai et al., 2001). For the seepage problem, water head is a forced boundary condition and flux is a natural boundary condition. Either water head or flux must be specified at each node; otherwise, the finite element equation cannot be solved. For steady state analysis, at least one node must have a water head boundary condition. In the pavement drainage simulation, specific boundary conditions need to be considered for pavement surface, end of outlet pipe, and bottom of subgrade soil layer.
In pavement construction, the groundwater is often controlled by lowering the water table during subgrade preparation (Cedergren, 1974). For most pavements, water table is lowered to 3-7m below the pavement surface and thus groundwater is not a major source for the deterioration in such pavements (Huang, 1993). Therefore, only rainfall infiltration is simulated in this research as the source of water inflow into the pavement. During a rainfall event, the amount of water that falls on the pavement is a function of time, which has the same unit as flux. Thus, the boundary condition on pavement surface is expressed as a flux being a function of time.

The infiltration path of water is controlled by the surface type and surface condition of the pavement. In the case of uncracked asphalt pavements, water infiltrates into pavement through the pores within the asphalt layers, while in the case of cracked asphalt pavement and portland cement concrete pavements, most of water infiltrates through cracks and joints. When water infiltrates through the pore structure, the flow is driven mainly by the negative pore-water pressure (matric suction) in pavement surface; while when water infiltrates through cracks, the major driving force is gravitation. The effect of gravitation will last until water flow reaches the end of cracks or joints, after which the flow will be driven by the pore-water pressure in that layer.

Even though infiltration comes from the rainfall, it does not mean that all rainfall water can penetrate into the pavement structure. For a rainfall with certain intensity and duration, the amount of infiltration is influenced by the condition of pavement surface, such as: cracking, surface roughness, geometry, and others. It is also influenced by the effective porosity of the surface material. Under certain surface conditions, the infiltration rate of an uncracked pavement is controlled by the pore-water pressure and conductivity of pavement surface. If the pavement has a positive pore-water pressure, water tends to be pumped out from the pores and no water can infiltrate into the pavement. In another word, infiltration only occurs under a negative pore-water condition. The upper limit of infiltration flux is the maximum coefficient of permeability of the pavement surface because it is the maximum quantity of water that can be drained by the material for a unit of time. However this maximum value does not necessarily equal to the saturated permeability obtained from laboratory tests. For dense graded aggregates
with fines this upper limit is about 20% of the saturated permeability, while for open graded aggregate this value is about 50%-80% of their saturated permeability (Richardson, 1997). The permeability of asphalt concrete usually is similar to the dense graded aggregate.

Considering the physical laws influencing the infiltration of rainfall, the following conditional boundary conditions are applied to the pavement surfaces in this dissertation.

1. If pore-water pressure > 0, then flux = 0; else
2. For uncracked HMA surface, if rainfall intensity > upper limit of HMA permeability, then flux = rainfall intensity; else, flux = upper limit of HMA permeability;
3. For uncracked PCC surface, flux = 0, because permeability of PCC is very low;
4. For cracks or joints on HMA or PCC, if rainfall intensity > upper limit of permeability of base layer, then flux = rainfall intensity; else, flux = upper limit of permeability of base layer.

Here, it is assumed that the surface runoff of pavement can be drained within a short period of time and no accumulated water can form water head on the pavement surface. In highway design, the drainage of surface water runoff is usually completed in less than 10 minutes, which is much less than the time unit used in this research. Therefore, the assumption of not having backed up head on the surface is a reasonable one for this research.

Water entering into the pavement surface layer flows through base and subbase layers, and finally is collected by the collecting pipe and carried out pavement through outlet. The flow is driven by water head difference between pavement surface and the end of outlet pipe. At the end of outlet pipe, the flow is also controlled by the pore-water pressure. If pore-water pressure is negative, water will be held in the pipe and no water will flow out. If the pore-water pressure is positive, water will be forced to flow out. Therefore, the boundary condition at the end of outlet pipe is set as:

1. If pore-water pressure < 0, then flux = 0;
2. If pore-water pressure > 0, then flux = - pore water pressure multiplied by a pore water pressure multiplier. This multiplier is obtained as a result of field calibration.

In the models which were employed in this research, the collection pipe and the outlet pipe are treated as porous media with a very high permeability. The flow in a closed conduit is usually analyzed using energy equation, which consists of the following three components: the gravitational head, the pressure head, and the velocity head. But the water flow in the pipes is slow and the velocity head is still small compared to the gravitational head and pressure head. Therefore, the governing equation derived for a porous media is also applicable to the flow analysis in a collection pipe and outlet pipe.

The other boundary condition that needs to be specified is the bottom of the pavement model. During construction, the water table is often lowered to 3 to 7 m below the pavement. If the entire depth of soil above water table were to be modeled, the number of elements needed in the 3-D model would be exceedingly high. However, this problem can be solved by applying a free drain boundary condition at the bottom of model (Krahn, 2004). It is reasonable to assume that indicated that at some point beneath the subgrade soil, the water contents and pore-water pressures become constant with depth. When this is the case, the total head gradient within the soil is equal to unity and the downward flux is equal to the hydraulic conductivity at that point (Krahn, 2004). In this research, to reduce the number of elements the subgrade soil is only modeled to 0.5 m depth. Since this location is still far away from the water table, a free drain boundary at the bottom is useful for the estimation of water content in subgrade soil. In this research, the free drain boundary condition is expressed as flux = - conductivity of soil. In a transient analysis, the conductivity will be changed with time.

In the 3-D model used in this research, only the paved lanes were molded. To simplify the model, the sides of the pavement model were assumed to be impermeable, which means flux = 0.
2.7 Model Verification

Based upon the background knowledge of unsaturated flow and the specific considerations for pavement drainage, a 3-D model can be developed by writing a FlexPDE script. As with any finite element model of this type, computer generated data must be checked against field data for verification and calibration purposes. The model can be verified either by other calibrated model or field measurement. The Indiana DOT and Purdue University had performed a project to measure the PSDS performance and the detail information can be obtained from their project final report (Hassan et al., 1996). This report can provide all the information needed for an unsaturated transient flow analysis. This report included: gradation, dry unit weight, permeability and SWCC of pavement materials, rainfall events, pavement geometries, and drainage flow and moisture records corresponding to rainfall events.

The Indiana DOT project was conducted on a section of I-469 at Fort Wayne. Three sections were selected carefully from a four-lane divided highway, and among the four lanes only one lane was instrumented. In this research, section-1 and section-3 were used for model calibration. The components of the test sections are shown in Figure 2.4 and the material properties are listed in Tables 2.1 through 2.3.

```
25 mm (1 in.)
19 mm (1.5 in.)
19 mm (1.5 in.)
76 mm (3 in.)
228 mm (9 in.)
90 mm (3.5 in.)

Base #5C

25 mm (1 in.)
19 mm (1.5 in.)
19 mm (1.5 in.)
304 mm (12 in.)

Base #5C

216 mm (8.5 in.)

Base #5D

#53

Section-1

#11 Surface Over #9, #8 Binder

Section-3
```

Figure 2.4 Pavement Components of Test Sections in InDOT Project (Hassan and White, 1996) (Pavement layer designations are Indiana DOT designations).
The width of the instrumented lane was 3.66m (12 ft). Adjacent to the instrumented lane was a 0.6m (2 ft) shoulder. This shoulder width was the area covering the collector pipe trench. The length of the section-1 was 236m (780 ft) and the length of section-3 was 220m (736 ft). An outlet pipe was constructed at the end of each section. The test sections had a longitudinal slope of approximately 2%. The transverse slope was not indicated in the report. However, it was assumed to be 2% in the modeling. In the report, both rainfall and drainage data were recorded. Since the measurements were taken within half a year after construction, one may assume that pavement surface was crack free. Two of the three measured rainfall events were used for the model verification. Figures 2.5 to 2.7 illustrate the outflow comparisons of between model predictions and field measurements. The model predictions of water content of pavement section-3 under rainfall event-3 and their comparisons with field data are listed in Table 2.3.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Section-1</th>
<th>Section-3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Density (g/cm³)</td>
<td>Permeability (cm/s)</td>
</tr>
<tr>
<td>#11 surface</td>
<td>2.21</td>
<td>1.01e-4</td>
</tr>
<tr>
<td>#9 binder</td>
<td>1.789</td>
<td>8e-5</td>
</tr>
<tr>
<td>#8 binder</td>
<td>2.11</td>
<td>1e-4</td>
</tr>
<tr>
<td>#5C base</td>
<td>1.96</td>
<td>0.1297</td>
</tr>
<tr>
<td>#2 base</td>
<td>2.25</td>
<td>0.019319</td>
</tr>
<tr>
<td>#5D base</td>
<td>2.33</td>
<td>1.28e-4</td>
</tr>
<tr>
<td>#53 Coarse Agg.</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>#8 Trench Agg.</td>
<td>2.21</td>
<td>1.179627</td>
</tr>
<tr>
<td>Subgrade Soil</td>
<td>2.7</td>
<td>5.7E-07</td>
</tr>
</tbody>
</table>
Table 2.2 Material Moisture Suction Relationships

<table>
<thead>
<tr>
<th>Material</th>
<th>Pressure (bars)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>Soil Section-1</td>
<td>0.43</td>
</tr>
<tr>
<td>Soil Section-3</td>
<td>0.39</td>
</tr>
<tr>
<td>#11 surface</td>
<td>0.016</td>
</tr>
<tr>
<td>#9 binder</td>
<td>0.055</td>
</tr>
<tr>
<td>#8 binder</td>
<td>0.031</td>
</tr>
<tr>
<td>#5C base</td>
<td>0.164</td>
</tr>
<tr>
<td>#2 base</td>
<td>0.056</td>
</tr>
<tr>
<td>#5D base</td>
<td>0.02</td>
</tr>
<tr>
<td>#53 aggregate</td>
<td>0.279</td>
</tr>
<tr>
<td>#8 aggregate</td>
<td>0.566</td>
</tr>
</tbody>
</table>

Note: 1 bar = 100 kPa

Comparison of Outflow of Section-3 under Rainfall Event-3

Figure 2.5 Comparison of Outflow Curves for Section-3 under Rainfall Event-3.
Figure 2.6 Comparison of Outflow Curves for Section-3 under Rainfall Event-1.

Figure 2.7 Comparison of Outflow Curves for Section-1 under Rainfall Event-3.
Table 2.3 Initial Degree of Saturation of Section-3

<table>
<thead>
<tr>
<th>Layer</th>
<th>Degree of Saturation</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Predicted (1m (3 ft) from outlet)</td>
<td>Observed (25.4mm (1 in.) deep below top of subgrade)</td>
</tr>
<tr>
<td>Subgrade</td>
<td>88% -100%</td>
<td>99%</td>
</tr>
<tr>
<td>Filter</td>
<td>22% - 48%</td>
<td>26% - 32%</td>
</tr>
<tr>
<td>Base</td>
<td>0.2% - 0.47%</td>
<td>&lt;1%</td>
</tr>
</tbody>
</table>

The comparison of the predicted and measured data indicates that the unsaturated 3-D model used in this study did a good job of simulating field conditions.

2.8 Sensitivity Analysis of Model Parameters

In the 3-D finite element model used in this study, the collection pipe in trench was assumed to be a porous media with a high permeability. This permeability was kept constant with time and did change with water content. In the process of model calibration and verification, the value of collection pipe permeability was adjusted to 2000 m/hour to get a result comparable with the field measurement.

Another parameter that influences the convergence of the finite element model is the pore water pressure multiplier which was described in the boundary condition section of this report. This is a pressure review boundary condition in which the flow of water is controlled by the pressure difference between the inside and the outside of the outlet. When inside pressure is higher than outside pressure water will be pushed out. The amount of the exiting water is calculated as pore water pressure multiplied by a constant. This is an important parameter that controls the convergence of the model. If this value is very high, water will be drain out very quickly and leave a vacuum in the pavement. This vacuum translates into a high negative pore water pressure which will make the convergence of the finite element model impossible. On the other hand, if this value is too low, water can not drain out in time. The accumulated water in the pore will induce a very high positive pore-water pressure and will make the matrix become singular again. Therefore, the key role of the pore water pressure multiplier must be thoroughly understood.
To investigate the contributions of collection pipe permeability and the pore water pressure multiplier constant, sensitivity analyses have been conducted on a model of Section-3 in the InDOT’s project. To save computing time, only a 10-meter long (30 ft) pavement was simulated. In the sensitivity analysis of the collection pipe permeability, the coefficient of permeability was the only parameter that has been changed and the pore water pressure multiplier was set as 100 for the 10 m-model. On the other hand, a separate sensitivity analysis was conducted to check the effect of collection pipe. In this analysis the permeability of the collection pipe was set to be 100 for the 10-meter long model. The results of this sensitivity analysis are shown in Figures 2.8 and 2.9.

![Sensitivity Analysis of Collection Pipe Permeability: Outflow vs. Time.](image-url)
Figure 2.8 indicates that with increasing the permeability of the collection pipe, less time will be needed to drain out the infiltration water. For a given pore water pressure multiplier, there is an upper limit of the effective permeability for the collection pipe. Above this upper limit the increase of permeability has little effect on the outflow. The value of the upper limit will change with the components and geometries of pavement, the pore water pressure multiplier, and the amount of inflow. In contrast to the outflow curve, in Figure 2.8, the total outflow that drained during a certain rain event did not change with the permeability of collection pipe.
Figure 2.9 Sensitivity Analysis of Pore Water Pressure Multiplier.

The sensitivity analysis of pore water pressure multiplier indicates that this factor has a significant effect on the convergence as well as the drainage characteristic of a model. For the pavement in this sensitivity analysis, a smooth flow can be obtained with a pore water pressure multiplier below 1000. Above this level, the model either cannot converged or cannot obtain reasonable results. The pore water pressure multiplier also has little effect on the total flow through a rainfall event. The total flow will increase by 7.6% when the pore water pressure multiplier changes from 100 to 500. Therefore, during model calibration the pore water pressure multiplier needs to be adjusted to reflect field measurement. For the 10-meter long models used in this research, the pressure multiplier was set as 100, which is the value that was calibrated by the InDOT section-1 and section-3 models.

2.9 Analysis Solutions of Unsaturated 3-D Finite Element Model

An advantage of the finite element method is to provide solutions for the entire domain. Using the 3-D pavement drainage model, the following parameters can be obtained: volumetric water content, gravitational water content, degree of saturation, pore-water pressure, water head,
and flux. The total flow can be calculated by integrating flux over a specific area or over a specific domain. By applying some visualization techniques, one can see the drainage process of a pavement model.

Figures 2.10 through 2.15 present some analysis results for the 10-meter long (30 ft) section-3 and section-1 models. These analyses were conducted under rainfall Event-3 in InDOT’s record (Hassan et al., 2000). Figure 2.10 shows the pore-water pressure distribution within the pavement models at time = 6 hours, which is the time that the peak flow occurred for both section-1 and section-3. From Figure 2.10 one can see that the pore-water pressure becomes more negative with the increase of the pavement elevation and the distance to the outlet pipe. Figure 2.11 shows the total water head distribution within the pavement models. At time = 6 hours, the total head in section-1 was more uniform than that in section-3, which means at this time the drainage in section-1 was close to cessation but the drainage in section-3 was not. The distribution of degree of saturation within the pavement models are presented in Figure 2.12. From this figure one can see that at time = 6 hours both section-1 and section-3 had high saturation in subgrade (>95%) and surface (>80%), and had low saturation in HMA base (<10%). The saturation in filter layers was different for these two sections because different materials had been used. The filter saturation in section-1 is very high (>90%) but that in section-3 is moderate (60%). The history of the moisture variation in pavement layers was presented in Figures 2.13 through 2.15. Figure 2.13 shows the saturation history of subgrade at coordinate (x=0, y=10, z=0.7), which is located at the left back corner of the model (refer to the coordination labels in Figures 2.10 through 2.12). The analysis results indicated that the water content in subgrade soil is very high, and it is rarely affected by rainfall infiltration. Figure 2.14 shows the saturation history of the HMA base, which indicated that the HMA base is always under an unsaturated condition and the influence of rainfall is very limited. Figure 2.15 presents the saturation history of pavement surface. This figure shows again that pavement surface is always under unsaturated condition. However, the degree of saturation for surface is much higher than the HMA base even though HMA base is under the pavement surface.
Figure 2.10.a Pore-Water Pressure in Pavement Model: Section-3 at Time=6 hour.

Figure 2.10.b Pore-Water Pressure in Pavement Model: Section-1 at Time=6 hour.
Figure 2.11.a Total Water Head in Pavement Model: Section-3 at Time=6 hour.

Figure 2.11.b Total Water Head in Pavement Model: Section-1 at Time=6 hour.
Figure 2.12.a Degree of Saturation of Pavement Model: Section-3 at Time=6 hour.

Figure 2.12.b Degree of Saturation of Pavement Model: Section-1 at Time=6 hour.
Figure 2.13 Degree of Saturation History of Pavement Subgrade.

Figure 2.14 Degree of Saturation History of HMA Base.
In addition to the analysis results that were shown in Figures 2.10 through 2.15, there are other parameters that can represent the characteristics of the pavement drainage system, such as: total outflow or outflow history (Figures 2.5 through 2.7), flow path, peak flow, and time-to-drain. Currently, time-to-drain 95% of inflow is set as a criterion for the evaluation of the pavement drainage performance. The peak flow is used for the design of the collection pipe diameter. Therefore, in this research, the total outflow, outflow history, peak flow, time-to-drain 95%, and the degree of saturation will be calculated and used for the pavement performance evaluation.
CHAPTER 3.0 INFLUENCE OF PAVEMENT GEOMETRY ON SUBSURFACE DRAINAGE

3.1 Pavement Geometries and Subsurface Drainage

Water flow through pavement is driven by hydraulic head that is consisted of gravitational head and pressure head. The gravitational head is the difference between the elevation of inflow and outflow points. Therefore, geometry factors of a pavement will affect the drainage characteristics. In current pavement drainage design and analysis methods, the pavement transverse slope, width, and drainage layer thickness are considered in determining the drainage capability and time-to-drain of the drainage system, such as Equation 3.1 (Barber et al., 1952) and Equation 3.2 (Casagrande et al., 1952).

\[ q = kH \left( S + \frac{H}{2L} \right) \] \hspace{1cm} \text{(Equation 3.1)}

where

- \( q \) = discharge capacity of the drainage layer,
- \( k \) = permeability of the discharge layer,
- \( S \) = transverse slope of the drainage layer,
- \( H \) = thickness of drainage layer, and
- \( L \) = width of drainage layer.

\[ t_{50} = \frac{n_e L^3}{2k(H + SL)} \] \hspace{1cm} \text{(Example 3.2)}

where

- \( t_{50} \) = the time for 50% drainage, and
- \( n_e \) = effective porosity.
In these equations, pavement transverse slope is considered to be an important factor that affects the drainage of pavement. To ensure that the entering water can be drained efficiently, AASHTO requires a minimum of 2% transverse slope must be designed for the pavement, and to ensure safety, the transverse slope of driving lane cannot beyond 4% (AASHTO, 1993).

In the pavement subsurface drainage system (PSDS), a system of longitudinal collectors is generally used to remove the free water from the drainage layer. The collection system consists of a set of perforated, slotted, or open-jointed pipes that are used to remove water from the pavement structure and to convey it to suitable outlets outside the roadway area. The design of such systems should consider the type of pipe, the location and depth of collectors and their outlets, the slope and size of the collector pipes, and the provision for adequate filter protection for the pipes. In the current practice, the collector pipes are usually placed in a trench near pavement shoulder. The longitudinal roadway grades or the cross slopes usually govern the slopes of the collector pipes. The pipes are simply set at a constant depth below the roadway surface. However, practical construction and operational factors dictate that slopes of collector pipes be not less than 1% for smooth bore pipes and 2% for corrugated pipes. Thus, in areas where the longitudinal grades or cross slopes are nearly flat, it may be necessary to steepen the grade of collector pipes to meet these minimum requirements. Minimum recommended pipe diameters are 76mm (3 inches) for PVC pipes and 102mm (4 inches) for all other pipes (Cedergren et al., 1972)

The discharge capability $Q$ of collector pipes can be calculated by Manning’s formula for channel flow.

$$Q = 86400 \left( \frac{1.486}{n} AR^{2.3} S^{1/2} \right)$$

(Equation 3.3)

where

$n =$ roughness coefficient,

$A =$ area of pipe,

$R =$ hydraulic radius of pipe, and
\[ S = \text{slope of pipe.} \]

The hydraulic radius is a ratio between flow area and wetted perimeter. When flow in the pipe is full, \( R = \frac{D}{4} \), then Equation 3.3 becomes

\[ Q = \frac{53}{n} S^{0.5} D^{2.667} \quad \text{(Equation 3.4)} \]

Thus, the required pipe diameter \( D \) can be calculated using Equation 3.5 (Huang, 1993)

\[ D = \left( \frac{nQ}{53S^{0.5}} \right)^{0.375} \quad \text{(Equation 3.5)} \]

Equation 3.5 indicates that the pipe diameter depends on the pipe slope and amount of discharge. Since there is no 3-D model in use for the analysis of drainage outflow, the term \( Q \) usually computed by:

\[ Q = qL \quad \text{(Equation 3.6)} \]

where

\[ q = \text{lateral flow, and} \]
\[ L = \text{the distance between outlets.} \]

These design and analysis methods for pavement drainage systems are based on a 2-D flow analyses, thus the lateral flow in pavement and longitudinal flow in collector pipe are treated as two hydraulic processes. In the real world, the PSDS is one integrated hydraulic system. The hydraulic components in PSDS are related and reciprocal. The outlet pipe will not only affect the flow in collector pipe but also affect the water distribution in pavement layers. There are many uncertainties surrounding the effectiveness of the edgedrain systems. Therefore, a clear
understanding of the contributions of drainage system is very important in evaluating the performance of PSDS, and consequently in improving the pavement design quality.

In the past, the problem of pavement drainage could not be fully analyzed in an integrated 3-D fashion due to complexities involved. In this research, the influence of various geometric factors of PSDS was studied using a 3-D finite element model. In these analyses, the 10-meter long model of InDOT section-3 was used as the basic model and the material properties were kept unchanged during the analysis. The rainfall event-3 recorded in InDOT project was used as input for all of the analyses. When one geometry parameter was set as objective parameter, the other parameters were kept unchanged.

3.2 Effect of Edge Drain on Pavement Drainage Characteristics

Early edge drain systems were constructed by cutting a trench, backfilling it with a free draining aggregate, and providing an outlet for water to exit the system. In later varieties round perforated pipe was placed in the porous backfill, which increased the discharge rate of the system. Panel edge drains were introduced during the mid-1980s. Pavement edge drains are designed to collect water from throughout the pavement system and discharge it into the drainage ditch. In order to determine the effectiveness of pavement edge drains, pavement models with edge drains were compared to those without edge drains. The pressure review boundary condition is applied to the base layer and surface layer along the right edge of the pavement models. The pressure review boundary condition was as follows:

1. If pore-water pressure < 0, then flux = 0;
2. If pore-water pressure >0, then flux = - u multiplied by a pore water pressure multiplier, where u = pore water pressure.

The comparisons between the models with edge drains and the models without edge drains are presented in Figures 3.1 through 3.3. Figure 3.1 shows the analysis results of pavement outflow of the models. Figures 3.2 and 3.3 present the comparison of the subgrade saturation and base saturation of the models respectively. From the results of these analyses one can see
that the pavement with edge drain can carry more water out of the pavement and can significantly reduce water content within the pavement layers.

![Effect of Edgedrain on Outflow](image1.png)

**Figure 3.1** Comparison of Outflow between Models with Edgedrain and Models without Edgedrain.

![Effect of Edgedrain on Subgrade Saturation](image2.png)

**Figure 3.2** Comparison of Subgrade Saturation between Models with Edgedrain and Models without Edgedrain at Location (x=0, y=10, z=0.7).
3.3 Effect of Pavement Slopes on Pavement Drainage Characteristics

In current design methods, both transverse and longitudinal slopes are considered to be important factors that influence the characteristics of pavement drainage. In pavement design, the driving lanes normally have a 2% to 4% transverse slope and incline to the side with edge drain. If edge drains were constructed on both sides, pavement may be designed to have a crown cross profile. The longitudinal roadway grade usually changes with the area topography. The slope of collector pipe is governed by the longitudinal grade because it is usually simply set as a constant depth below the roadway surface. The transverse slope of shoulder usually constructed with a 4% to 6% slope for drainage purposes. The outlet pipes are also set at a constant depth below the surface of shoulder, so its slope is often governed by shoulder slope. In the following section, the contributions of transverse slope, longitudinal slope, and shoulder slope were studied.
3.3.1 Effect of Transverse Slope

In this analysis, the following three levels of transverse slope were examined: 0%, 2%, and 4%. The analysis results are listed in Figure 3.4.a and Table 3.1.

![Effect of Transverse Slope](image)

**Figure 3.4.a Comparison of Outflow for Section-3 with Different Transverse Slope.**

**Table 3.1 Comparison of Total Outflows for Pavements with Different Transverse Slopes**

<table>
<thead>
<tr>
<th>Transverse Slope</th>
<th>0%</th>
<th>2%</th>
<th>4%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Outflow m³</td>
<td>0.1335</td>
<td>0.1343</td>
<td>0.1352</td>
</tr>
<tr>
<td>(ft³)</td>
<td>(4.714)</td>
<td>(4.742)</td>
<td>(4.775)</td>
</tr>
</tbody>
</table>

The analysis results showed that for the one-lane pavement with edgerain the subsurface drainage performance is almost not affected by the transverse slope of the driving lane. This result is much different from the current design concept for the saturated flow analysis. The transverse slope which has been emphasized in pavement drainage design is important to surface drainage but hardly has any contribution to the subsurface drainage in the pavement. To check this conclusion, the 10-meter long model of section-1 in InDOT’s project was analyzed, and the analysis results were presented in Figure 3.4.b. Section-1 is a pavement with a high drainage capability. The analysis results verified that the subsurface drainage performance is almost not affected by the transverse slope of the driving lane.
To represent actual contribution practices, longitudinal slope was set to be the collector pipe slope in the model. By using the actual practices in pavement construction as a guide, the pavement models in study included the following longitudinal slopes: 0%, 2%, 4%, 6%, 8% and 12%. The results of these analyses are presented in Figure 3.5 and Table 3.2. The results indicate that the longitudinal slope of collector pipe has a significant effect on pavement drainage characteristics. The higher the longitudinal slope the quicker the drainage and the more water will flow. The relationships between longitudinal slope and pavement drainage characteristics are shown in Figures 3.6 through 3.8. From the regression results demonstrated in these figures one can see that the changes of total outflow and peak flow are proportional to the change of the longitudinal slope. Thus, these parameters can be estimated using the linear equations shown in these figures, respectively. It also demonstrated that the time-to-drain 95% of outflow can be estimated using a second-order regression equation. These regression equations were developed based upon the 10-meter long section-3 model. The constants in the regression equations may be different for other pavement structures, but the basic relationship between the longitudinal slope and the pavement drainage characteristics will be the same.

Figure 3.4.b Comparison of Outflow for Section-1 with Different Transverse Slope.

3.3.2 The Effect of Longitudinal Slope

To represent actual contribution practices, longitudinal slope was set to be the collector pipe slope in the model. By using the actual practices in pavement construction as a guide, the pavement models in study included the following longitudinal slopes: 0%, 2%, 4%, 6%, 8% and 12%. The results of these analyses are presented in Figure 3.5 and Table 3.2. The results indicate that the longitudinal slope of collector pipe has a significant effect on pavement drainage characteristics. The higher the longitudinal slope the quicker the drainage and the more water will flow. The relationships between longitudinal slope and pavement drainage characteristics are shown in Figures 3.6 through 3.8. From the regression results demonstrated in these figures one can see that the changes of total outflow and peak flow are proportional to the change of the longitudinal slope. Thus, these parameters can be estimated using the linear equations shown in these figures, respectively. It also demonstrated that the time-to-drain 95% of outflow can be estimated using a second-order regression equation. These regression equations were developed based upon the 10-meter long section-3 model. The constants in the regression equations may be different for other pavement structures, but the basic relationship between the longitudinal slope and the pavement drainage characteristics will be the same.
Figure 3.5 Effect of Longitudinal Slope on Pavement Drainage Characteristics.

Table 3.2 Comparison of Total Outflow and Time-to-drain of Pavements with Different Longitudinal Slopes

<table>
<thead>
<tr>
<th>Longitudinal Slope</th>
<th>0%</th>
<th>2%</th>
<th>4%</th>
<th>6%</th>
<th>8%</th>
<th>12%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Outflow m³ (ft³)</td>
<td>0.1221 (4.312)</td>
<td>0.1343 (4.742)</td>
<td>0.1377 (4.863)</td>
<td>0.1403 (4.955)</td>
<td>0.1419 (5.011)</td>
<td>0.1496 (5.283)</td>
</tr>
<tr>
<td>95% Time-to-drain</td>
<td>33</td>
<td>27</td>
<td>23</td>
<td>21</td>
<td>19.5</td>
<td>18</td>
</tr>
<tr>
<td>Peak Flow m³ (ft³)</td>
<td>0.00731 (0.257)</td>
<td>0.0132 (0.466)</td>
<td>0.0166 (0.586)</td>
<td>0.0189 (0.667)</td>
<td>0.0209 (0.738)</td>
<td>0.025 (0.883)</td>
</tr>
</tbody>
</table>
Figure 3.6 Effect of Longitudinal Slope on Total Flow.

\[ y = 0.1472x + 0.1314 \]
\[ R^2 = 0.9833 \]

Figure 3.7 Effect of Longitudinal Slope on Time-to-drain 95% of Outflow.

\[ y = 1294.6x^2 - 273.9x + 32.496 \]
\[ R^2 = 0.9906 \]
In addition to influencing outflow and time-to-drain, longitudinal slope has an important influence on water content in pavement layers. Figures 3.9 and 3.10 show the comparison of saturation of subgrade and HMA base in pavements with 0% and 6% longitudinal slope respectively. From these figures one can see that the saturation of subgrade is very high for a pavement model with 0% longitudinal slope model and the saturation along the longitudinal direction is almost uniform. On the other hand, for the pavement model with 6% longitudinal slope, the saturation is not uniform. After drainage, the degree of saturation in the pavement model with 6% longitudinal slope is much lower than that in the pavement model with 0% longitudinal slope. Thus, having an adequate longitudinal slope in pavement is important.

Figure 3.8 Effect of Longitudinal Slope on Peak Flow.
As mentioned before, in the models that used for these analyses the longitudinal slope of the driving lanes were combined with the slope of the collection pipe. Thus, further investigation needs to be conducted to clarify the contribution of these two factors. For this
purpose, the models with edgedrain were compared to the models without edgedrain. The longitudinal slope levels of 2% and 6% were applied to the models respectively. The analysis results were presented in Figures 3.11 and 3.12.

From Figures 3.11 and 3.12 one can see that the longitudinal slope of driving lanes is not the critical factor that affects the drainage performance of a pavement. The critical factor that contributes to the pavement drainage performance is the slope of the collection pipe that placed in the edgedrain. For pavement without edgedrain, the drainage capability is always low even though the longitudinal slope of its driving lane is high. This analysis result shows again that including an edgedrain is very important in improving the drainage performance for a pavement.

![Effect of Longitudinal Slope on Pavements with and without Edgedrain](image)

**Figure 3.11 Effect of Longitudinal Slope on Pavements with and without Edgedrain.**
3.3.3 The Effect of Outlet Pipe Slope

Outlet pipe slope is usually governed by the shoulder slope. In pavement design, it is often required that the outlet pipe slope should not be less than 4%. To investigate the effect of outlet pipe, three slope levels, 2%, 4%, and 6%, were examined using the 3-D finite element pavement model. The results are shown in Figure 3.13, and the relationships between the outlet pipe slope and the pavement drainage characteristics are shown in Figures 3.14 to 3.16. These results illustrate that outlet pipe slope is an important factor that influences the drainage characteristics of the pavement. The outlet pipe slope demonstrates the same effect patterns with the longitudinal slope. That is the total flow and peak flow has linear relationship with the outlet pipe slope and the time-to-drain has a second-order relationship with the outlet pipe slope. As shown in Table 3.3, when outlet pipe slope was increased from 0% to 2% the time-to-drain can be decreased by 7 to 9 hours. When longitudinal slope was increased from 2% to 4%, the time-to-drain only decreased by 4 hours. Thus, one can see that the effect of shoulder slope (outlet pipe slope) is more significant than the longitudinal slope of pavement. This means that positive drainage can be realized only by increasing the pavement outlet pipe slope, which is very easy to achieve in the field.
Figure 3.13 Effect of Shoulder Slope (Outlet Pipe Slope) on Pavement Drainage.

Table 3.3 Comparison of Total Flow and Time-to-drain of Pavements with Different Shoulder Slope

<table>
<thead>
<tr>
<th>Shoulder Slope</th>
<th>Outlet Pipe Slope</th>
<th>Total Outflow (m³)</th>
<th>Time-to-drain (95%)</th>
<th>Peak Flow (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2%</td>
<td>4%</td>
<td>6%</td>
<td></td>
</tr>
<tr>
<td>Outlet Pipe Slope</td>
<td>2%</td>
<td>4%</td>
<td>6%</td>
<td></td>
</tr>
<tr>
<td>Total Outflow m³</td>
<td>0.1253 (4.425)</td>
<td>0.1357 (4.792)</td>
<td>0.1390 (4.909)</td>
<td></td>
</tr>
<tr>
<td>Time-to-drain (95%)</td>
<td>34</td>
<td>25</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td>Peak Flow m³</td>
<td>0.0786 (0.278)</td>
<td>0.0132 (0.466)</td>
<td>0.0182 (0.642)</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.14 Effect of Outlet Pipe Slope on Total Flow.

Figure 3.15 Effect of Outlet Pipe Slope on Time-to-drain of 95% outflow.
3.4 Effect of Distance between Outlets

In current pavement drainage practice, the distance between outlets is used for the calculation of pipe capacity under the assumption that the total outflow is proportional to the length of pavement to be drained. In most states in the US, the most common outlet spacing interval is 75m (246 ft). This distance is used for the convenience of maintenance. To investigate the influence of outlet spacing, a 3-D finite element model was used to examine the effect of outlet spacing at the following distance: 30m (98.4 ft), 50m (164 ft), 75m (246 ft), 100m (328 ft), and 150m (492 ft) long. To shorten the analysis time, the permeability of pipe was increased with the length. In the 10-meter long model the pipe permeability was set as 100 m/hr (328ft/hr) to obtain a comparable outflow curve with the full length model of the test section. With the increase of pavement length, more water needs to be drained in the pipe, and the pipe permeability needed to be increased to ensure that the entering water can be drained out within the analysis time duration. The analysis results are illustrated in Figures 3.17 through 3.19. These results indicate that the total outflow and peak flow increase with the pavement length. Both total flow and peak flow have a positive linear relationship with the pavement
Therefore, the earlier assumption that drainage capability of a pavement is equal to the multiplication of lateral flow and outlet distance is a reasonable one.

Figure 3.17 Effect of Pavement Length on Drainage Characteristics.

Figure 3.18 Effect of Pavement Length on Total Outflow.
3.5 The Effect of Pavement Width

Pavement width is another factor that will influence the total flow of water in the pavement. In the current practices, pavement edge drains are often designed to handle drainage for two or three driving lanes with a shoulder. Obviously, adding lanes will increase the quantity of water to be drained. This study examined the nature of relationship between the number of the pavement lanes and water outflow. The 10-meter long section-3 model was modified and used for the analysis. This modification only included the increase of the number of lanes while other parameters were unchanged. The results of the analyses are illustrated in Figures 3.20 and 3.22. The analysis results indicated that increasing the number of lanes will add more water to drain. The total outflow and peak flow have a linear relationship with the number of lanes. There is an intercept in the regression model presented in Figure 3.20. This intercept can be viewed as the contribution of pavement shoulder.

Figure 3.19 Effect of Pavement Length on Peak Flow.
Figure 3.20 Effect of Pavement Width on Drainage Characteristics.

Figure 3.21 Effect of Pavement Width on Total Outflow.
Figure 3.22 Effect of Pavement Width on Peak Flow.

3.6 The Effect of Lane Slopes and Edgedrain Numbers

In current pavement drainage practices, the typical profile of a pavement cross-section is a two to four-lane pavement with edge drains on each side. The driving lanes can be crowned or inclined to one side according to the safety and super elevation requirements of highway design. In pavement design, one may assume that the water entering in a driving lane will be drained by the nearest edgedrain. This assumption is true if the driving lanes are isolated from one another. However, in the field pavement lanes work together as an integrated hydraulic system during the drainage. Using the 3-D finite element model, the effect of edgedrain system was investigated. The models are listed in Table 3.3. These models were designed with different number of lanes, lane slope, and number of edgedrain. The analysis results are presented in Figures 3.23 to 3.25.
Table 3.4 Models with Different Slopes and Edge Drains

<table>
<thead>
<tr>
<th>Model Name</th>
<th>No. of Lanes</th>
<th>No. of Edge Drain</th>
<th>Lane 1 Trans. Slope</th>
<th>Lane 2 Trans. Slope</th>
<th>Lane 3 Trans. Slope</th>
<th>Lane 4 Trans. Slope</th>
<th>Model Cross Section Diagram</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-Lane-1</td>
<td>2</td>
<td>1</td>
<td>-2%</td>
<td>-2%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2-Lane-2</td>
<td>2</td>
<td>1</td>
<td>2%</td>
<td>-2%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2-Lane-3</td>
<td>2</td>
<td>2</td>
<td>2%</td>
<td>-2%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2-Lane-4</td>
<td>2</td>
<td>2</td>
<td>-2%</td>
<td>-2%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>3-Lane-1</td>
<td>3</td>
<td>2</td>
<td>2%</td>
<td>-2%</td>
<td>-2%</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>3-Lane-2</td>
<td>3</td>
<td>2</td>
<td>-2%</td>
<td>-2%</td>
<td>-2%</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>3-Lane-3</td>
<td>3</td>
<td>1</td>
<td>2%</td>
<td>-2%</td>
<td>-2%</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>3-Lane-4</td>
<td>3</td>
<td>1</td>
<td>-2%</td>
<td>-2%</td>
<td>-2%</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>4-Lane-1</td>
<td>4</td>
<td>2</td>
<td>2%</td>
<td>2%</td>
<td>-2%</td>
<td>-2%</td>
<td></td>
</tr>
</tbody>
</table>

Effect of Edgedrain and Lane Slopes

Figure 3.23 Effect of Edgedrain and Lane Slope on Pavement Drainage Characteristics (2-Lane and 4-Lane Models).
Figure 3.23 shows the total outflow for various lane and edgdrain combinations. In the 2-lane models, the 2-lane-1 and 2-lane-2 model have only one edge drain on the right side. In the 2-lane-2 model left lane was inclined to left and right lane was inclined to right; while in 2-lane-1 model both of the two lanes was inclined to right. The 2-lane-3 model was a model that had the same lane slope as the 2-lane2 model but having edge drain on both sides. Thus, the 2-lane-3 was symmetric about the center line. The 2-lane-4 model had the same lane slope as the 2-lane-1 model, but had an extra edge drain on the left side. The 4-lane model was a model with four driving lanes and two edge drains. The four driving lanes symmetrically sloped to each side. The results reported in Figure 3.23 indicates that among the 2-lane pavement alternatives, 2-lane-3 model can drain more entering water and has a shorter drainage time. As shown in Table 3.5, comparing to the 2-lane-1 model, the 2-lane-3 model can remove extra 13% water. The 2-lane-4 model can drain more water than a pavement with only one edge drain, but it cannot drain water more quickly. The water flow in symmetric 4-lane model is also symmetric. The total outflow of each edgdrain almost equals to the water removed by the 2-lane1 model.

![Effect of Lane Slopes on Two Edgedrain Pavement](image)

Figure 3.24 Effect of Lane Slope on Pavement Drainage Characteristics of 2-Lane models.
Table 3.5 Total Outflow of 2-Lane Models and 4-Lane Model

<table>
<thead>
<tr>
<th></th>
<th>2-lane-1</th>
<th>2-lane-2</th>
<th>2-lane-3 Total</th>
<th>2-lane-4 Total</th>
<th>4-lane-1 Left Edge</th>
<th>4-lane-1 Right Edge</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Outflow m³ (ft³)</td>
<td>0.2249 (7.94)</td>
<td>0.2242 (7.92)</td>
<td>0.2533 (8.95)</td>
<td>0.2402 (8.48)</td>
<td>0.2289 (8.08)</td>
<td>0.2289 (8.08)</td>
</tr>
<tr>
<td>Ratio to Outflow of 2-lane-1</td>
<td>100%</td>
<td>100%</td>
<td>113%</td>
<td>107%</td>
<td>102%</td>
<td>102%</td>
</tr>
</tbody>
</table>

The water quantity removed by each edgdrain in 2-lane model is illustrated in Figure 3.24. This illustration indicates that in the symmetric models the outflow at each edgdrain is the same; while in the models with all lanes sloped to one side most of the water was drained through the lower edgdrain. The edgdrain on the higher side is almost none participant.

The analysis results of 3-lane models are shown in Figure 3.25. The 3-lane-1 and 3-lane-2 models had two edge drains, while the 3-lane-3 and 3-lane-4 models only had one edge drain at right side. The 3-lane-1 and 3-lane-3 models had one lane inclined to the left and the other two lanes inclined to the right; while in 3-lane-2 and 3-lane-4 models all three lanes inclined to the right side. In these cases the lane number is odd, no model is symmetric.
The analysis results of 3-lane models indicated that if the pavement profile is not symmetric, only one edgedrain is the active participant in removing water. The edgedrain on the higher side carries very little water. In the design that all three lanes inclined to one side, more water can be removed, and can be removed quickly.

3.7 The Effect of Layer Thickness

Pavement drainage is influenced by the pavement layer thickness. To ensure the pavement has enough drainage capability, AASHTO suggested a minimum of 100mm (4 inches) thick drainage layer should be used (AASHTO, 1993). While, a statistical analysis of the LTPP data indicated that the thickness of drainage layer does not show any significant effect on pavement drainage (Hall et al., 2003). This analysis was conducted using the LTPP data in SPR1 and SPR2 program, which were designed specially to study the pavement drainage problem using a factorial experiment design method. However, during the construction process, pavement layer thicknesses were changed, which made statistical analysis very difficult. To address this issue, a 3-D finite element model was used to quantify the effect of pavement layer thickness on pavement drainage.

3.7.1 Surface Thickness

It is well know that the surface permeability has a significant effect on pavement drainage, but the effect of surface thickness is still not well understood. This analysis is designed to study the effect of pavement surface thickness. In this analysis, the 10-meter long section-3 model was modified by increasing its surface thickness to 178mm (7 inches). The analysis results are shown in Figure 3.26.
One can conclude from this analysis that the thickness of pavement surface, with the range of study, does not have any effect on the drainage characteristics. The two outflow curves for 100mm (4 inches) and 178mm (7 inches) surface completely overlap. It is important to note that this analysis assumed a constant asphalt permeability regardless of lift thickness. However, a thick lift often produces a lower density, which is corresponding to a higher permeability. The analysis of the asphalt layer with a permeability gradient was outside of the scope of this study.

3.7.2 Base Thickness

In order to get a better understanding of the effect of base layer thickness, the 10-meter long section-3 model was modified by changing the base thickness to 100mm (4 inches) and 200mm (8 inches). The analysis results of these models are shown in Figure 3.27. From the analysis results one can see that the thickness of base, within the range of study, does not have any effect on drainage.
3.8 The Effect of Collection Pipe and Outlet Pipe Diameter

In pavement drainage design, the diameter of collector pipe is designed by using Equation 3.5, which is a function of peak outflow of pavement and collector pipe slope. Conversely, for a certain slope, the peak outflow also should be affected by pipe diameter. To verify this observation, the 3-D model with 100mm (4 inches) and 150mm (6 inches) collector pipe were analyzed separately. Figure 3.28 shows the effect of collector pipe on pavement drainage character of the 10-meter long model. From this figure one can see that the pipe diameter does play an important role on the drainage of pavement. In this figure, the increase of pipe diameter tends to decrease the peak flow and increase the time-to-drain. But the results are different for the 75-meter long model. Through the results presented in Figure 3.29, one can see that the effect of pipe diameter is just opposite to the results for 10-meter long model. For 75-meter long model, pavement with 100 mm (4 inches) pipe has lower peak value and longer drainage time than pavement with 150 mm (6 inches) pipe. This phenomenon was induced by the larger flow capability of the larger pipe, which can be presented by the following equation:

\[ q_c = k \frac{D^2}{4} \]  

(Equation 3.7)
where

\[ q_c = \text{pipe flow capacity}, \]
\[ k = \text{pipe permeability}, \]
\[ D = \text{pipe diameter}. \]

The pipe flow capacity is determined by the pipe diameter provided that \( k \) is constant. The flow capacity of thin pipe is smaller than that of thick pipe. For the 10-meter long model, the amount of collected water was smaller than the flow capacity of 100mm (4 inches) pipe and could be carry out without any congestion. The increase of pipe diameter could not increase the water flow but decreased the suction in the system, which made the drainage slowing down. However, in the long pavement more water was collected into the collection pipe and the water amount exceed the capability of 100mm (4 inches) pipe, which induced congestion in the pipe. The increase of the pipe diameter would increase the flow capability of the system, thus, for 75-meter long model the drainage capability was improved by using 150mm (6 inches) pipe.

![Figure 3.28 Effect of Pipe Diameter on Drainage of 10-meter Long Model](image.png)

**Figure 3.28 Effect of Pipe Diameter on Drainage of 10-meter Long Model.**
3.9 Summary of Analyses and Recommendations

The analyses results for the influence of various geometry parameters indicated that the current edgdrain practice is effective for pavement subsurface drainage. Among the geometry parameters, the parameters that were related to the geometry of collection pipe and outlet pipe have the most significant effect on pavement drainage performance. These parameters include pavement longitudinal slope (collection pipe longitudinal slope), shoulder slope (outlet pipe slope), and pipe diameters. The transverse slope of driving lanes does not affect the pavement drainage as much as one might have guessed, and the pavement layer thickness effect is negligible.

Based upon the analyses of this section, the following strategies are recommended for the pavement drainage system geometry design:

1. To provide positive drainage to pavement, edge drain system should be designed properly.
2. The longitudinal slope of collection pipe should be designed as deep as possible, and a minimum of 2% longitudinal slope is recommended.

![Figure 3.29 Effect of Pipe Diameter on Drainage of 75-meter Long Model.](image)
3. The slope of outlet pipe should be designed as deep as possible, and a minimum of 4% slope is recommended.

4. The collection pipe and outlet pipe diameter should be designed in an optimized range. Large pipe diameter does not necessarily lead to effective drainage. The estimation of peak flow should include the effects of rainfall, surface condition, as well as the distance between outlets.

5. For two lane or four lane pavements, if two edge drains are designed, a symmetric crown cross section should be used. If the cross section cannot be designed symmetrically, then only one edge drain needs to be placed at the lower side.

6. For three lane pavement, only one edge drain should be designed at the lower side.

7. The transverse slope and layer thickness do not contribute to drainage within the range of this study.
CHAPTER 4.0 PAVEMENT MATERIAL HYDRAULIC PROPERTY DETERMINATION

4.1 Pavement Material Properties in PSDS Modeling

In the finite element analysis, the drainage characteristic of a pavement is determined by boundary conditions, geometry, as well as material properties. Material properties are introduced into the governing formulation by the continuity equation. The constitutive law is one of the most vital parts of finite element analysis. However, unless it is defined to reflect the precise behavior of a material or a system, the results from the analysis can be of very little significance.

For a seepage problem, the governing constitutive law is Darcy’s law. In this simple linear relationship, the coefficient of permeability is the parameter that reflects the hydraulic property of materials. The coefficient of permeability is defined as the rate of discharge of water under conditions of laminar flow through a unit cross sectional area of a porous media under a unit hydraulic gradient. The units of the permeability coefficient are expressed in cm/sec and ft/hr in this report. It is well known that the permeability of a material is not constant but varies with its matric suction or its saturation condition. Researchers have reported that unsaturated permeability can be expressed as saturated permeability adjusted by a function of water content. The saturated permeability of a material can be easily determined by laboratory tests or field tests. It also can be predicted through rational or statistical models. Water content is a parameter that reflects the saturation condition of a material. The relationship between matric suction and water content is called Soil-Water Characteristic Curve (SWCC). In the governing equation for transient analysis, the slope of SWCC is also an important material property that determines the drainage characteristic of the seepage domain. This property is brought into the governing equation by the continuity equation of mass conservation and is also determined by the SWCC. Therefore, to conduct the unsaturated finite element modeling of the pavement drainage, the saturated permeability and the SWCC are the two important hydraulic properties that must be determined for pavement materials.
Most pavements are made of several layers. The materials that are used in pavement include HMA, PCC, dense-graded or open-graded coarse aggregates, stabilized or unstabilized base, and subgrade soils. For subgrade soils, there is a good body of hydraulic knowledge in the field of geotechnical engineering. However, the information on hydraulic properties of various pavement materials that are constructed on top of the subgrade is rather limited. Currently, saturated permeability is the only hydraulic property that is evaluated by most researchers. Although the importance of unsaturated flow has been realized for some time, there have been only a few researches for investigation of the SWCC and the unsaturated hydraulic conductivity of pavement materials into consideration (Hassan et al., 1996 (a); Cote et al., 2003).

One of the objectives of this research was to compare the effectiveness of the current drainage practices. Therefore, the hydraulic properties of both saturated permeability and SWCC must be obtained for various pavements. Since the laboratory data for unsaturated hydraulic properties of some materials are not widely available, an estimation method needed to be developed for this research.

4.2 Saturated Permeability

Pavement materials are very diverse; however, their microstructure serves as a common thread. From materials as dense as HMA and PCC to that as loose as gavel, they are all aggregated from granular particles. The differences between these materials are their grain size, grain distribution, and their bound or unbound nature. Within these structures, the coarse particles form a skeleton. The voids between the skeleton are filled by the fine particles, air, and/or a binder. When exposed to water, these materials can be considered as either a two-phase media (solid and fluid) or a three phase media (solid, fluid, and air). Under the saturated condition, the voids are fully filled with water, while under unsaturated conditions the voids are only partially filled with water.

The water flows through the connected pores in the porous media. While, it is observed that water only flows through the pores that are filled with water. The pores filled with air do
not serve as a good conduit for water flow. Under saturated condition, all effective pores are filled with water; therefore, the conductivity reaches its maximum level.

4.2.1 Influencing Factors

Under the saturated condition, the permeability of granular material is a function of pore size distribution, pore continuity, and pore shape. These are affected by grain size distribution, particle shape, and relative density. The permeability is also a function of specimen mineralogical composition and the viscosity, unit weight, and chemical composition of the fluid. In the testing of coarse-grained granular materials under normal circumstance, tap water is used as the permeant, and the interaction of particle mineralogical composition and water chemistry is considered negligible. Thus, in a practical sense, the significant variables are particle size gradation, particle shape, degree of saturation, relative density, mineralogical composition of fines and permeant temperature.

Porosity is defined as the ratio of volume of voids to total volume. It is related roughly to hydraulic conductivity and is used in various flow algorithms. Porosity is a function of relative density, specific gravity, and particle shape. Upon compaction to a given relative density, a more angular material will have a greater porosity and, most likely, a higher hydraulic conductivity (Loudon 1953). In general, hydraulic conductivity increases with a more open gradation, a more angular particle shape, a higher degree of saturation, a lower relative density, a higher temperature (lower permeant viscosity), and a higher porosity (Richardson, 1997).

4.2.2 Testing Methods

There are two common methods for measuring hydraulic conductivity in the laboratory: one in which the flow head is constant, and alternately, one in which the head falling (Richardson, 1997). The constant head method is usually applicable to materials with hydraulic conductivities greater than 0.001cm/s (2.83 ft/day). In constant head tests, the flow is measured. For low hydraulic conductivity materials, the flow is rather slow and the falling head method is more appropriate.
Two different permeameters are available for testing the hydraulic conductivity of pavement materials: rigid wall and flexible wall. Rigid wall permeameters are generally less costly and easier to operate, and can handle relatively large flow rates. The major disadvantages are potential leakage along the permeameter wall/specimen interface, the large sample size required, the difficulties in specimen saturation, and the limitation in available head that can be applied. The major advantages of flexible wall permeameters are the ability to seal the permeameter wall/specimen interface, the ability a back-pressure to saturate the specimen, and the ability to apply larger heads. Disadvantages of flexible wall permeameters are their rather complex operation, and higher cost. In general, the constant head test with rigid wall permeameter can be used successfully to measure permeability of open-graded specimens. Constant head test with a flexible permeameter is more suitable for testing of dense-graded specimens.

4.2.3 Permeability Estimation Methods

To save time, sometimes it is preferred to determine the saturated permeability through statistical models or rational equations. The well-known Kozeny-Carman equation (Kozeny, 1927; Carman, 1956) is widely used for the prediction of permeability in saturated soils:

\[
k = \frac{1}{k_0 T^2 S_0^2 (1 - n_e)^2} n_e^3 \tag{Equation 4.1}
\]

where

- \( k \) = permeability in saturated soil,
- \( k_0 \) = pore shape factor \( \approx 2.5 \),
- \( T \) = tortuosity factor \( \approx \sqrt{2} \),
- \( S_0 \) = the surface area per unit volume of particles, and
- \( n_e \) = effective porosity.
This equation is based upon the assumptions that particles are approximately uniform and larger than 1μm (3.93×10^{-5} inches) and that flow is laminar. This has worked well for sands; however, it is inadequate for clays because of the effect of fine-grained soils (Mitchell, 1976).

Moulton suggested a model that predicts permeability from grain size, content of fine particles, and porosity.

\[ k = \frac{6.214 \times 10^5(D_{10})^{1.478}(n)^{6.654}}{(P_{200})^{0.597}} \]  
(Equation 4.2)

where

\[ k = \text{permeability in saturated soil}, \]
\[ n = \text{porosity}, \]
\[ P_{200} = \text{portion of fines passing sieve #200 (0.075mm)}. \]

This equation can only be used for aggregates that have fine grains passing sieve #200. For open graded aggregates with no fines, the equation is invalid. In addition, a study by Richardson (1997) reported that this equation underestimated the coefficient of permeability by three to four times.

After a close examination of the problems existing in current permeability testing methods, Richardson (1997) developed four regression equations to predict the saturated permeability for both dense-graded as well as open-graded aggregates. These equations are regressed using 106 data sets from his own experiments and the data in the literature.

The first equation is as follows:

\[ \log k = 3.062 + 6.400\log n + 1.905\log D_{10} \]  
(Equation 4.3)

where

\[ k = \text{permeability in saturated soil}, \]
\[ n = \text{porosity, and} \]
\[ D_{10} = \text{size the represents 10\% passing (mm).} \]

This covers a wide range of hydraulic conductivity: \(4.7 \times 10^{-5}\) to \(8.8\text{cm/s (0.02 to 24,940 ft/day).}\)

The second equation is as follows:

\[
k = -2.873 + 23.923n + 1.005D_{10} - 0.107P_{3/8} - 0.214P_{50} + 0.218P_{16} \quad \text{(Equation 4.4)}
\]

where

\[
P_{3/8} = \text{percent passing 9.5 mm (3/8 in.) sieve,}
\]
\[
P_{16} = \text{percent passing #16 sieve, and}
\]
\[
P_{50} = \text{percent passing #50 sieve.}
\]

This equation was developed for open-graded materials with hydraulic conductivity greater than \(0.1\text{cm/s (283 ft/day).}\)

For permeability ranging from \(0.1\) to \(1\text{cm/s (283 to 2835 ft/day)}\)

\[
k = -0.024 + 5.573n - 0.024P_{3/8} + 0.004P_{8} \quad \text{(Equation 4.5)}
\]

\[
k = 7.137 + 12.521n + 0.411D_{10} - 0.192P_{3/8} \quad \text{(Equation 4.6)}
\]

In asphalt pavements, the base layer is usually designed as a drainage blanket to increase the drainage capability of the pavement. The drainage blankets are usually constructed using open-graded aggregates in which the fine grains were removed to increase the permeability. However, open graded permeable materials may have lower stability. To improve the stability of the base layers, the open graded aggregates are always treated with 2\% to 3\% asphalt or portland cement. Through a series of laboratory test, a regression model for the prediction of asphalt-treated aggregate permeability was developed as follows (Lindly et al., 1994):
\[ k = 0.3 - 0.088 \times AC + 0.034 \times AIR - 0.0337P_8 \]  
(Equation 4.7)

where

\[ k \] = permeability in saturated aggregate,

\[ AC \] = percent asphalt cement by total weight,

\[ AIR \] = percent air voids by total volume of sample, and

\[ P_8 \] = percent by weight passing sieve #8.

This regression equation was based on 38mm (1.5 inches) top size aggregates and 2% to 3% asphalt stabilization. This equation is not suitable for dense-graded bases because only open gradations were tested in the research. Thus, Equation 4.7 can not be used for asphalt concrete surface course.

Based on the Kozeny-Carman equation (Kozeny, 1927; Carman, 1956) and a series of experimental findings, Masad et al. (2004) produced an empirical equation to predict the permeability for asphalt concrete and HMA base. The equation is

\[ k = \frac{0.098V^m_a}{S_{agg}} \]  
(Equation 4.8)

where

\[ k \] = permeability in saturated HMA,

\[ V_a \] = the total percent air voids in an asphalt mix,

\[ m \] = constants determined from data fitting, and

\[ S_{agg} \] = total surface area of aggregate (calculated from gradation).

By applying Equation 4.8 to various HMA permeability data, it was found by Masad et al. that m-value varies within a small range between 4.6 to 6.9 (Masad et al., 2004). In this equation, the surface area of aggregates was measured using the procedure recently developed by
Only the gradation and density of the aggregates are needed to determine the specific surface area \( S_{agg} \) assuming that the particles are of cubical shape. For each sieve, the weight and average particle size are used to calculate the number of particles with cubical shape existing on that sieve. The surface area of these particles is calculated in units of \( \text{m}^2/\text{kg} \). This value is multiplied by density to determine the surface area in units of \( 1/\text{m} \). Finally, the surface areas of all sieves are added to get the total surface area for the entire aggregate. The procedure does not address the influence of texture and particle shape on surface area. However, it is based on a rational analysis, and as such is considered an improvement over the current method used in the practice for estimating surface area.

### 4.2.4 Improved Permeability Models

The coefficient of permeability for various pavement materials can be estimated using the models that were presented earlier. The amount and size of effective pores as well as the total surface area are related to the gradation. Hence, there must be a common relationship between the saturated permeability and the gradation of the aggregates. The equation derived from Poiseuille’s law established this relationship as follows (Mitchell, 1976):

\[
k = C_s V_s^2 \left( \frac{\gamma_w}{\mu} \right) \frac{1}{S_0^2 \ (1-n_e)^2} \ n_e^3 \ S^3
\]

(Equation 4.9)

where

- \( k \) = permeability in saturated soils or aggregates,
- \( C_s \) = a shape coefficient,
- \( V_s \) = volume of solids,
- \( S_0 \) = the surface area per unit volume of particles,
- \( \gamma_w \) = unit weight of permeant,
- \( \mu \) = viscosity,
- \( n_e \) = effective porosity, and
- \( S \) = degree of saturation.
For the case of full saturation, \( S=1 \), and \( C_s=1/k_0T^2 \), and the Equation 4.9 becomes the well-known Kozeny-Carman equation (Kozeny, 1927; Carman, 1956). In this equation, if \( C_s \) is taken to be a composite shape factor, and \( V_0^2/S_0^2 \) is interpreted as a representative grain size \( D_s \), then

\[
k = C_sD_s^2 \left( \frac{\gamma_w}{\mu} \right) \frac{n_e^3}{(1-n_e)^2} S^3 \quad \text{(Equation 4.10)}
\]

If 20 °C water is used as permeant, \( C_s \frac{\gamma_w}{\mu} \) is constant and can be represented by constant \( C \). The \( C \)-parameter will be further described in this section. Under fully saturated condition, the Equations 4.9 and 4.10 may be written as:

\[
k = CD_s^2 \frac{n_e^3}{(1-n_e)^2} \quad \text{(Equation 4.11)}
\]

This equation is easier to use compared to the Kozeny-Carman equation because the parameter of specific surface area is represented by grain size of the aggregates, which can be determined easily from aggregate gradation data.

1. **Determination of Effective Porosity**

Effective porosity is the ratio of the volume of voids that can be drained under gravity to total volume of the material. The water that is essentially undrainable is water that is held in the pores by capillary action or adhered to the aggregate particle surfaces. Smith et al. (1964) have shown that for open-graded materials, the effective porosity can be close to the total porosity. However, for dense-graded base materials, the effective porosity may be much smaller than the total porosity. This conclusion was verified by the experimental data reported by Randolph et al (Randolph et al., 1996). They indicated that for fine gradations of DGA (Ohio No. 310 and 304), as little as one-quarter of total porous space can drain; while for coarse
gradation of No. 57 as much as 85% of total pore space can drain. This result is very close to the test data reported by Richardson (1997), who reported 68% and 27% for open and dense graded aggregate, respectively.

The equation for effective porosity is essentially the traditional total porosity equation with a term that serves as an adjustment factor to account for the undrainable water

\[ n_e = 1 - \frac{\gamma_d}{G_s \gamma_w} (1 + G_s \omega_e) \]  

(Equation 4.12)

where

- \( \gamma_d \) = compacted dry unit weight,
- \( \gamma_w \) = unit weight of water,
- \( G_s \) = apparent specific gravity, and
- \( \omega_e \) = undrained moisture content.

Equation 4.12 is not very practical due to the fact that the undrained residual moisture content is hard to measure. While, through a comparison with the measured effective porosities, one can see that the calculation method for \( n_f \) described by Cote and Konard (2003) can be used for this determination.

\[ n_f = \frac{n}{n + \frac{(1-n)F}{100}} \]  

(Equation 4.13)

where

- \( n \) = porosity,
- \( n_f \) = the portion of void of coarse grains that not filled by fines, and
- \( F \) = percent passing of sieve #200 or content of fines.
One can define the coefficient of effective porosity to be \( f = \frac{n_e}{n} \). The relationship between the calculated \( n_f \) and the measured \( \frac{n}{n} \) reported by Richardson (1997) is illustrated in Figure 4.1. From this figure one can see that the coefficient of effective porosity \( f \) is highly related with \( n_f \), so that the effective porosity can be estimated using \( n_f \).

**Figure 4.1 Relationship between \( n_f \) and Ratio of Effective Porosity Derived Using Richardson’s Laboratory Data (Richardson, 1997).**

Based upon the work of Cote and Konard (2003) one can estimate the effective porosity as:

\[
\frac{n_e}{n} = \frac{n^2}{n + \frac{(1 - n)F}{100}}
\]

(Equation 4.14)

This equation is suitable for both untreated aggregate as well as treated aggregates, like HMA base by considering the asphalt content as a part of fines.
2. **Determination of Representative Grain Size**

The representative grain size can be determined from aggregate gradation using the following procedure.

- Change the Cumulative-Distribution-Function (CDF) curve of aggregate gradation into Possibility-Density-Function (PDF) curve,
- Calculating the average grain size $\bar{d}_i$ using following equation:
  \[
  \bar{d}_i = \frac{d_i + d_{i+1}}{2}
  \]  
  (Equation 4.15)
  where
  $d_i$ = sieve size of $i^{th}$ fraction of the aggregate, and
- Calculating representative grain size using following equation:
  \[
  D_s = \sum_{i=1}^{n} \Delta g_i \bar{d}_i
  \]  
  (Equation 4.16)
  where
  $\Delta g_i$ = the weight of the material of $i^{th}$ fraction in terms of total weight.

By introducing the representative grain size, the new permeability prediction method accounts for the contribution of all grain sizes instead of one or two grain sizes used in other methods. To get a more reasonable representation of the total grain sizes, the interval between the sieve sizes should be as small as possible. This requirement can be satisfied by implementing a fitting equation to the gradation curve. The Fredlund’s unimodal and bimodal equations can be used to produce a good fit to grain-size distribution data, and provide a continuous fit of the entire grain-size distribution curve including the coarse and fine extremes (Fredlund et al., 2000).
\[ P_g(d) = \frac{1}{\ln\left(\exp\left(1 + \left(\frac{a_{gr}}{d}\right)\right)^{m_{gr}}\right)} \left\{ 1 - \left[ \ln\left(1 + \frac{d_r}{d}\right) \right]^{-\gamma} \right\} \]  
(Equation 4.17)

where

\[ a_{gr} = \text{parameter related to the initial breaking point of the curve}, \]

\[ n_{gr} = \text{parameter related to the steepest slope of the curve}, \]

\[ m_{gr} = \text{parameter related to the shape of the fines portion of the curve}, \]

\[ d_r = \text{parameter related to the amount of fines in a soil}, \]

\[ d = \text{diameter of any particle size under consideration, and} \]

\[ d_m = \text{diameter of the minimum allowable size particle}. \]

The bimodal equation of the grain-size distribution can be written as follows:

\[ P_g(d) = \left[ w \left\{ \frac{1}{\ln\left(\exp\left(1 + \left(\frac{a_{bi}}{d}\right)^{n_{bi}}\right)^{m_{bi}}\right)} \right\} + (1 - w) \left\{ \frac{1}{\ln\left(\exp\left(1 + \left(\frac{j_{bi}}{d}\right)^{k_{bi}}\right)^{l_{bi}}\right)} \right\} \right] \left\{ 1 - \left[ \ln\left(1 + \frac{d_{rbi}}{d}\right) \right]^{-\gamma} \right\} \]  
(Equation 4.18)

where

\[ a_{bi} = \text{parameter related to the initial breaking point of the curve}, \]

\[ n_{bi} = \text{parameter related to the steepest slope of the curve}, \]

\[ m_{bi} = \text{parameter related to the shape of the fines portion of the curve}, \]

\[ j_{bi} = \text{parameter related to the second breaking point of the curve}, \]

\[ k_{bi} = \text{parameter related to the second steepest slope of the curve}, \]

\[ l_{bi} = \text{parameter related to the second shape of the fines portion of the curve}, \]

\[ d_{rbi} = \text{parameter related to the amount of fines in a soil}, \]
\[ d = \text{diameter of any particle size under consideration,} \]
\[ d_m = \text{diameter of the minimum allowable size particle, and} \]
\[ w = \text{the weighting factor indicating the ratio of the overall sample that constitutes} \]
\[ \text{the coarse fraction.} \]

The parameters of the Fredlund’s unimodal and bimodal (Fredlund et al., 2000) can be determined using a fitting algorithm. After the gradation curve fitting, a cumulative-distribution-function (CDF) can be obtained, which can be transferred into probability-density-function (PDF). Then, the grain size of aggregates can be divided into small intervals and the representative grain size can be calculated using equations 4.15 and 4.16. To demonstrate this, aggregate gradation data and their fitted distribution curves using Fredlund’s unimodal are shown in Figure 4.2.

![Aggregate Gradation Curve Fitting Using Fredlund's Unimodal](image)

**Figure 4.2 Fitted Aggregate Gradation Curves Using Fredlund Unimodal.**
3. Determination of Dummy Constant C-Parameter

In addition to $n_e$ and $D_s$, the C-parameter must be determined in Equation 4.11. As mentioned earlier, under saturated condition C-parameter is constant, and can be determined with statistical regression. The total 75 datasets published by Richardson (1997), Crovetti et al. (1991), and Randolph et al. (1996) were used for this determination. The regression results are presented in Figure 4.3, from which one can see that the C-parameter is 0.279cm/s/cm² with $R^2$ of 0.9179.

\[
y = 0.2791x \quad R^2 = 0.9179
\]

![Measured Permeability vs. $D_s^2n_e^3/(1-n_e)^2$](image)

**Figure 4.3 Determination of C-parameter in Equation 4.11.**

Hence, for permeability ranging from 0 to 30cm/s (0 to 80000 ft/day), the saturated permeability can be predicted using Equation 4.19.

\[
k = 0.279D_s^2 \frac{n_e^3}{(1-n_e)^2} \quad \text{(Equation 4.19)}
\]

The comparison between this new method and Richardson’s relationship (Equation 4.3) and Moulton’s relationship (Equation 4.2) is shown in Figure 4.4. The comparison results indicate
that the accuracy of the permeability prediction for aggregates with permeability higher than 0.1 cm/s (283 ft/day) is improved by using this new method. Other methods tend to underestimate the permeability at this range. The Richardson’s first relationship is good for the prediction of permeability less than 0.1 cm/s (283 ft/day); while the new equation tends to overestimate the permeability for this range.

Equation 4.11 is applicable to both untreated materials as well as asphalt treated materials. In fact, the Masad’s equation (Equation 4.8) has already shown the validity of Kozeny-Carman equation (Equation 4.1) (Masad et al., 2004). In the case of treated aggregates, the effect of asphalt content should be included in the calculation of the effective porosity. The asphalt cement in HMA will fill the voids and form a film on the surface of aggregates, which tends to reduce the effective porosity of the aggregates. Therefore, asphalt can be simulated as fines in the calculation of the effective porosity and Equation 4.14 can be modified as follows:

\[ n_f = \frac{n}{n + \frac{(1-n)(F + A)}{100}} \]  

(Equation 4.20)

Figure 4.4 Comparison of Permeability Prediction Methods.
where
\[
\begin{align*}
  n &= \text{porosity}, \\
  n_f &= \text{the portion of void of coarse grains that not filled by fines}, \\
  F &= \text{percent passing of sieve #200 or content of fines, and} \\
  A &= \text{percent of asphalt cement}.
\end{align*}
\]

The treated base and HMA datasets published by Hassen and White (2000), and Lindley (1994) were used to determine the \(C\)-parameter in Equation 4.11. The regression results showed that the permeability of these materials can be calculated using following regression equation

\[
k = 0.358D_s^2 \frac{n_e^3}{(1-n_e)^2}, \quad R^2 = 0.9937 \tag{Equation 4.21}
\]

The comparison of measured and predicted permeability is shown in Figure 4.5.

![Comparison of Measured and Predicted Permeability of Treated HMAs](image-url)
4.3 Soil-Water Characteristic Curve

The water content in an unsaturated soil is a function of soil suction. This relationship between the water content in a soil and the suction can be expressed in a plot which is known as the soil-water characteristic curve. This curve is more commonly referred to as a soil-water retention curve in soil sciences. The soil-water characteristic curve of a soil can be measured using a pressure plate device in the laboratory. Using the axis-translation technique, air pressure above atmospheric is applied to the soil specimen while the water pressure is kept at a lower level. The difference between the air and water pressures is known as matric suction. The water content of the soil specimen at various matric suction levels can be determined and a soil-water characteristic curve is obtained. A typical SWCC is illustrated in Figure 2.2.

4.3.1 SWCC Fitting

In the finite element analysis, the SWCC must be smooth or the slope of SWCC must be continuous because the slope of SWCC is the material property presented in the governing equation. Thus the measured SWCC needs to be expressed in an equation from which ensures its continuity of its slope. Over the years a number of equations have been suggested for the SWCC. Among these equations, the relationship suggested by Fredlund and Xing (1994(a)) gave the best fit and was developed to obtain a smooth function over the complete range of negative pore-water pressure levels (Leong et al., 1997 (a)).

The Fredlund and Xing (1994(a)) method is a closed-form solution that can be used to develop the volumetric water content function for all possible negative pressures between 0 and $10^6$ kPa. The governing equation of Fredlund and Xing is as follows:

$$
\theta(\psi, a, n, m) = \theta_e C(\psi) \frac{1}{\ln \left[ e + \left( \frac{\psi}{a} \right)^n \right]^m}
$$

(Equation 4.22)
\[ C(\psi) = \begin{cases} \ln \left( \frac{1 + \frac{\psi}{\psi_r}}{1 + \frac{10^6}{\psi_r}} \right) \\ 1 - \ln \left( \frac{1 + \frac{\psi}{\psi_r}}{1 + \frac{10^6}{\psi_r}} \right) \end{cases} \]  

(Equation 4.23)

where
\[
\begin{align*}
\theta &= \text{volumetric water content,} \\
\psi &= \text{matric suction at the calculation point,} \\
\psi_r &= \text{matric suction at the residual water content,} \\
\theta_s &= \text{saturated volumetric water content, which equal to void ratio of solid,} \\
e &= \text{natural number, 2.71828, and} \\
a, n, m &= \text{fitting parameters.}
\end{align*}
\]

In this equation, the parameter \( a \) (with a unit of kPa) is closely related to the air entry value. Air entry value is a suction level at which air begin to enter the voids. In general, the value for the parameter \( a \) would be higher than the air entry value. However, for small values of \( m \), the air-entry value can be used for parameter \( a \). The parameter \( n \) controls the slope of the SWCC and its maximum value is approximately equal to \( a \). In this equation, \( \theta \) becomes equal to \( \theta_s \) when the suction is zero, and \( \theta \) becomes zero when the suction goes to infinity. It is also possible to use the degree of saturation for curve fitting, since the degree of saturation varies from 0 to 1. Gravimetric water content can be similarly normalized for curve-fitting purposes (Fredlund et al., 1994(a)). Fredlund et al. suggested a correction factor \( C(\psi) \) to force water content equal to 0 at suction of \( 10^6 \) kPa. Leong et al. (1997 (a)) has shown that this equation is robust. This method is only functional if one knows values for \( a, n \) and \( m \).

### 4.3.2 SWCC Estimation from Grain Size Distribution

The Fredlund and Xing (1994(a)) method was not intended to predict a volumetric water content function from grain-size curve, but it was developed to obtain a smooth function for the measured SWCC. However, for pavement materials, very few SWCC data are available...
because of the variability of the material properties and the high costs associated with direct measurement of SWCC. To perform the unsaturated analysis for a material, the SWCC must be estimated from the gradation curve.

Through a comprehensive laboratory investigation on the hydraulic characteristics of granular base-courses, Cote and Konrad (2003) proposed a methodology for the assessment of the SWCC for base layer aggregates. This methodology was developed based on the Brooks and Corey (1964) model, in which \( \theta_r \) is set to 0. The Brooks and Corey equation is

\[
\frac{\theta - \theta_r}{\theta_s - \theta_r} = \begin{cases} 
1 & \text{if } \psi < \psi_a \\
\left( \frac{\psi_a}{\psi} \right)^{\lambda} & \text{if } \psi \geq \psi_a
\end{cases}
\]  
(Equation 4.24)

where

\( \theta_r \) = residual water content,
\( \theta_s \) = saturated water content = effective porosity,
\( \psi_a \) = air entry value (kPa), and

\[ \lambda = \frac{\Delta \log(\theta)}{\Delta \log(\psi)} \] = the logarithmical slope of the SWCC.

For coarse aggregates, the residual water content is assumed to be 0, thus Equation 4.24 becomes

\[
\theta = \begin{cases} 
\theta_s & \text{if } \psi < \psi_a \\
\theta_r \left( \frac{\psi_a}{\psi} \right)^{\lambda} & \text{if } \psi \geq \psi_a
\end{cases}
\]  
(Equation 4.25)
The $\psi_a$ and $\lambda$ for coarse aggregates can be determined using the following experimental equations provided by Cote et al (2003).

\[
\log(\psi_a) = 3.92 - 5.19n_f \tag{Equation 4.26}
\]

\[
\lambda = 0.358 - 0.021\left(n_f^{0.65}S_{sf}\right) \tag{Equation 4.27}
\]

where

\[
S_{sf} = \text{specific surface area of the fines fraction (m}^2/\text{g}).
\]

The specific surface area (m$^2$/g) of materials can be calculated using as follows:

\[
S_{sf} = \frac{6}{1000\rho_s} \sum_{i=1}^{n} \frac{\Delta g_i}{d_i} \tag{Equation 4.28}
\]

where

\[
\rho_s = \text{density of solid (kg/m}^3).\]

The Cote and Konrad (2003) methodology is developed based on well-graded dense aggregates, thus it must be checked before being applied to open graded aggregates. In this study, the measured SWCC of coarse sand #8 and coarse aggregate #53 in InDOT project (Hassan et al., 1996(a)) were used to verify this. The calculated and measured parameters of these materials are shown in Table 4.1 and the measured and predicted SWCC were illustrated in Fig. 4.6.

### Table 4.1 Brook and Corey Parameters of Coarse Aggregate

<table>
<thead>
<tr>
<th>Material</th>
<th>$n_f$</th>
<th>Calculated $\psi_a$</th>
<th>Fitted $\psi_a$</th>
<th>Calculated $\lambda$</th>
<th>Fitted $\lambda$</th>
</tr>
</thead>
<tbody>
<tr>
<td>#8 Agg.</td>
<td>0.98</td>
<td>0.068</td>
<td>0.071</td>
<td>0.337</td>
<td>0.497</td>
</tr>
<tr>
<td>#53 Agg.</td>
<td>0.96</td>
<td>0.087</td>
<td>0.080</td>
<td>0.223</td>
<td>0.242</td>
</tr>
</tbody>
</table>
The data in Table 4.1 indicated that the predicted entry values for both materials are very close to the values regressed using the measured SWCC; while the grain size distribution parameter $\lambda$ tends to be underestimated for uniform sand. This is because the open-graded aggregates contain more large pores, from which water can be drained out easier. This makes the SWCC become steeper, and the residual water will be smaller than the dense-graded aggregates. Thus, if the Cote and Konard method is used for the prediction of SWCC of open-graded aggregates, the $\lambda$ calculated from Equation 4.27 should be multiplied by a factor ranging from 1 to 2.

The Cote and Konard (2003) method was developed for the estimation of SWCC in untreated base layer aggregates in pavement. To extend the application of this method to asphalt treated base layer aggregates and HMA surfaces, the measured SWCC data in InDOT project were used to determine the Brooks and Corey parameters in this study. For this purpose, the $n_f$ defined by Cote and Konard was modified by considering the asphalt cement as fines (Equation 4.20).
The measured SWCC of the materials listed in Table 4.2 were fitted to Brooks and Corey equation and the modified $n_f$ was used as independent variable. The properties and the fitted Brooks and Corey parameters of asphalt treated aggregates are listed in Table 4.2, and illustrated in Figures 4.7 and 4.8.

### Table 4.2 Material Properties and Brooks and Corey Parameters of HMA

<table>
<thead>
<tr>
<th>Material Name</th>
<th>IN#5C base</th>
<th>IN #2 base</th>
<th>IN #5D base</th>
<th>IN #11 surface</th>
<th>IN #8 surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine Content (%)</td>
<td>4.4</td>
<td>4.1</td>
<td>4.5</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Asphalt (%)</td>
<td>4.1</td>
<td>2.3</td>
<td>4.3</td>
<td>4.5</td>
<td>4.1</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.0</td>
<td>2.2</td>
<td>2.3</td>
<td>2.2</td>
<td>2.2</td>
</tr>
<tr>
<td>$V_{Asphalt}$</td>
<td>0.061</td>
<td>0.052</td>
<td>0.100</td>
<td>0.122</td>
<td>0.088</td>
</tr>
<tr>
<td>$V_{Aggregate}$</td>
<td>0.715</td>
<td>0.820</td>
<td>0.832</td>
<td>0.782</td>
<td>0.772</td>
</tr>
<tr>
<td>$V_{air}$</td>
<td>0.224</td>
<td>0.129</td>
<td>0.069</td>
<td>0.096</td>
<td>0.140</td>
</tr>
<tr>
<td>$n_f$</td>
<td>0.816</td>
<td>0.733</td>
<td>0.486</td>
<td>0.529</td>
<td>0.696</td>
</tr>
<tr>
<td>$\psi$ (kPa)</td>
<td>0.033693</td>
<td>0.0253</td>
<td>4.69E-16</td>
<td>1.49E-23</td>
<td>9.84E-08</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>0.5556</td>
<td>0.7691</td>
<td>0.0368</td>
<td>0.0361</td>
<td>0.114</td>
</tr>
<tr>
<td>$\theta_s$</td>
<td>0.164</td>
<td>0.056</td>
<td>0.02</td>
<td>0.016</td>
<td>0.031</td>
</tr>
</tbody>
</table>

![Aggregate Gradation of HMAs](image)

**Figure 4.7 Aggregate Gradations of Asphalt Treated Base and HMA Surface.**
From the data listed in Table 4.2 and Figure 4.9 one can see that the air entry value (AEV) of HMA with small grains and high asphalt content is close to 0 and thus the water content will drop down to a low level under a very small suction. This is because in the HMA most of the aggregate grains are coated by the asphalt binder. The contact angle between water and asphalt is higher than 90, so it is very easy for air to enter into even small pores. While, for HMA bases with large grains and less asphalt binder, there is not a continuous asphalt film, and the air entry value is higher and near the air entry value for untreated aggregates. Based on this analysis, the air entry values of HMAs were regressed with $\frac{n_f}{A}$, where $A$ is the asphalt content. The relationship between $\log \psi_a$ and $\frac{n_f}{A}$ is shown in Figure 4.9.
Figure 4.9 Regression Curve for Air Entry Value Prediction for HMAs.

Thus, the air entry value $\psi_a$ of asphalt treated base and HMA surface can be predicted using the following equation:

$$\log\psi_a = -692.5\left(\frac{n_f}{A}\right)^2 + 372.3\left(\frac{n_f}{A}\right) - 50.47$$  \hspace{1cm} (Equation 4.29)

Similar to the air entry value, $\lambda$ of HMA will be affected by the grain size distribution and asphalt content. The relationship between $\lambda$ and $\frac{n_f}{A}$ was illustrated in Figure 4.10 and the regression equation for $\lambda$ prediction is as follows:

$$\lambda = 12.70\left(\frac{n_f}{A}\right)^{2.5857}$$  \hspace{1cm} (Equation 4.30)
Figure 4.10 Regression Curve for $\lambda$ Prediction for HMAs.

After observing the measured SWCC of HMA bases and HMA surface one can see that the maximum water content at zero suction is not equal to the porosity of the material. This means that not all pores in the material is effective for water flow, and the $\theta_s$ parameter need to be evaluated. By contrasting the SWCC data with effective porosity $n_f$, one can see that they are highly correlated and their relationship are illustrated in Figure 4.11.

The maximum volumetric water content can be estimated using the following equation:

$$\theta_s = 6.694 \left(n \times n_f\right)^2 - 0.47 \left(n \times n_f\right) + 0.026$$

(Equation 4.31)

By applying these regression equations, the SWCC of asphalt treated bases can be estimated through their gradation and compositions.
4.4 Determination of Unsaturated Permeability (Hydraulic Conductivity)

The unsaturated permeability can be measured either by a direct method or an indirect method. The direct measurement of unsaturated permeability is a very time-consuming process. The duration of the test increases as the water content in the soil decreases. In direct measurements there are steady-state and unsteady-state methods (Fredlund et al., 1993). In the steady-state method, a matric suction is first imposed on a soil specimen using the axis-translation technique. At equilibrium, denoted by constant water content, a hydraulic gradient is then imposed across the soil specimen. The flow rate is measured and the permeability is obtained via Darcy’s law. Using the unsteady-state method or instantaneous profile method, a cylindrical soil specimen is subjected to a continuous flow of water from one end. The hydraulic head gradient and the flow rate at various points along the specimen are computed by monitoring water content and pore-water pressure at these points. In indirect measurement the water content of the soil specimen at various matric suction levels is determined. The permeability is then inferred from the soil-water characteristic curve using a
regression model. Thus, it is more desirable to determine the unsaturated permeability through mathematical models.

In the current literature, three types of models are available for soil unsaturated permeability estimation. They are empirical equations, macroscopic models, and statistical models. The degree of sophistication increases from the empirical equation to the statistical models. Empirical equations are derived from laboratory permeability data and regression equations. For example, Brooks and Corey equation (1964) is a widely used empirical equation, which is expressed as follows:

\[ k_w = k_s \left( \frac{\psi_d}{\psi} \right)^\eta = k_s \left( \frac{\theta}{\theta_s} \right)^\delta \]  

(Equation 4.32)

where \( \eta \) and \( \delta \) are experimental constants and may be calculated from the following:

\[ \eta = 2 + 3\lambda \]  

(Equation 4.33)

\[ \delta = \frac{2 + 3\lambda}{\lambda} \]  

(Equation 4.30)

The macroscopic models are derived using analytical method under the assumption of similarity between laminar flow (microscopic level) and flow through porous media (macroscopic level). The statistical models are the most rigorous models for permeability functions. In these models, the permeability function is derived from the soil-water characteristic curve assuming that the porous media consists of a set of randomly distributed interconnected pores. Statistical models and the closed form models derived from statistical models are becoming popular in unsaturated permeability analyses. Examples are: Mualem’s method (1976), Van Genuchten’s method (1980) and Fredlund and Xing method (Fredlund et al., 1994 (b)).
The statistical methods can provide good estimations for hydraulic conductivity. However, these models are rather complicated. Leong and Rahadjor (1997(b)) have illustrated that the statistical model can be transformed into a macroscopic model and then to an empirical equation. Leong and Rahadjor (1997(b)) found that there are two common characteristics in the empirical permeability functions. First, the permeability curve of a material is similar to its soil-water characteristic curve. The similarity in shape between permeability function and SWCC is not surprising since water only flows through the water phase in the soil. Second, the permeability under unsaturated condition can be expressed using the following relationship:

\[ k_r = k_s \Theta^p \]  

(Equation 4.35)

where

- \( k_r \) = relative coefficient of permeability = \( \frac{k_w}{k_s} \),
- \( k_s \) = saturated permeability,
- \( \Theta \) = normalized water content = \( \frac{\theta_w - \theta_r}{\theta_s - \theta_r} \),
- \( \theta_r \) = water content at calculation point,
- \( \theta_r \) = residual water content,
- \( \theta_s \) = saturated water content = effective porosity, and
- \( p \) = a constant for a specific soil.

As \( \Theta \) parameter is expressed as a function of suction \( \psi \), \( k \) parameter also can be expressed as a function of suction, \( \psi \). Leong and Rahadjor (1997(b)) compared the relationship of \( \Theta \) and \( \psi \), and showed that Fredlund’s equation (Equation 4.22) fits the experimental SWCC well and robust when the correction factor equals to 1 (Leong and Rahadjor, 1997(b)). Thus, Equation 4.32 can be expressed as follows:
In this equation, \( p \) is a constant that needs to be determined through an analysis of experimental data; while other parameters can be determined by Fredlund and Xing fit of SWCC (Leong et al., 1997 (b)). Applying this equation to soil database, it was found that the back-calculated \( p \) value concentrated within the following range: 1 to 8.8 and most frequently occurred at 3.0 (SoilVision Systems Ltd., 2004). Comparing the calculated results of Leong and Rahadjo’s equation to that of Fredlund’s statistical model (Fredlund et al, 1994(b)), which can predict conductivity without any experimental data, \( p \) value of 4.0 is appropriate for both of treated and untreated aggregates. This procedure will simplify the effort needed for determining the unsaturated hydraulic conductivity.

4.5 Hydraulic Property Estimation for Finite Element Modeling

The materials used for pavement construction include HMA, PCC, coarse aggregate, sand and soil. At current stage, the hydraulic properties are only well-established for soil; while the hydraulic properties, especially SWCC and unsaturated conductivity, for other materials need to be estimated from their grain-size distribution or gradation. Based upon the methodology introduced in this section, the estimation procedure is as follows:

Step 1 – collect material property data for gradation, dry density, specific gravity, unit weight, etc.. Fit the gradation curve using Fredlund unimodel or bimodal (Fredlund et al., 2000). Calculate representative grain size, specific surface, and \( D_{10} \) using the probability-density-function (PDF).

Step 2 – If the measured saturated permeability data are not available, the appropriate model will be applied to predict the saturated permeability.
Step 3 – If the measured SWCC data are not available, the Cote and Konard method (2003) or the modified Cote and Konard method proposed in this research will be used for the prediction of SWCC.

Step 4 – Fit the measured or predicted SWCC using Fredlund and Xing (1994 (a)) equation to get a smooth material curve for finite element analysis.

Step 5 – Estimate the unsaturated permeability using Leong and Rahardjo (1997(b)) equation with the parameters determined in Fredlund and Xing fit and a proper p value.

This procedure will be applied to both of unbounded materials (coarse aggregates, sands, soils) and bounded materials (HMA, or treated aggregates). Since the permeability of PCC and cement treated dense-graded aggregates is very low and its change with suction is negligible, thus the PCC surface and cement treated dense graded aggregates was treated as impermeable in the finite element model.
CHAPTER 5.0 INFLUENCE OF MATERIAL PROPERTIES ON PAVEMENT SUBSURFACE DRAINAGE PERFORMANCE

5.1 Hydraulic Properties of Pavement Materials

To ensure the pavement subsurface drainage system is functioning properly, proper materials must be used. The materials used in a pavement usually include: subgrade soils, untreated dense-graded or open-graded aggregates or sands, asphalt or cement treated dense-graded or open-graded aggregates, asphalt or portland cement surface layer, and plastic or steel pipes and geotextiles.

The hydraulic properties of various types of soils and sands have been well established. The permeability and soil water characteristic curve (SWCC) of all kinds of soils can be obtained from various soil databases. According to their ingredients and physical properties, soils can be classified into several types, such as silt, clay, till, loam etc.. The saturated permeability of soils can range from $10^{-8}$ to $10^{-5}$cm/s (0.0000283 to 0.0283 ft/day). The SWCC of soils varies with the fine content and their chemical/physical properties. To investigate the effect of subgrade soil on PSDS performance, three typical soils were selected for the analyses. They are the three soil samples presented in InDOT’s project (Hassan et al., 1996 (a)). The hydraulic properties of those soils are listed in Table 5.1 and Figures 5.1 and 5.2.
### Table 5.1 Gradation and Properties of Soils

<table>
<thead>
<tr>
<th>Sieve Size (mm (inches))</th>
<th>IN Soil Sample 1.2</th>
<th>IN Soil Sample 2.1</th>
<th>IN Soil Sample 3.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.05 (3/4)</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>12.7 (1/2)</td>
<td>100</td>
<td>100</td>
<td>95.6</td>
</tr>
<tr>
<td>9.525 (3/8)</td>
<td>100</td>
<td>99.3</td>
<td>95.9</td>
</tr>
<tr>
<td>4.75 (No.4)</td>
<td>99.9</td>
<td>98.7</td>
<td>92.7</td>
</tr>
<tr>
<td>2.35 (No.8)</td>
<td>99.3</td>
<td>97.3</td>
<td>85.2</td>
</tr>
<tr>
<td>0.6 (No.30)</td>
<td>98.4</td>
<td>95.3</td>
<td>78.9</td>
</tr>
<tr>
<td>0.3 (No.50)</td>
<td>95.5</td>
<td>88.9</td>
<td>59.7</td>
</tr>
<tr>
<td>0.15 (No.100)</td>
<td>92.2</td>
<td>81.6</td>
<td>39.1</td>
</tr>
<tr>
<td>0.075 (No.200)</td>
<td>87.6</td>
<td>75.3</td>
<td>25.9</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.75</td>
<td>2.76</td>
<td>2.72</td>
</tr>
<tr>
<td>Permeability (cm/s (ft/day))</td>
<td>5.7E-07 (1.6E-3)</td>
<td>7.73E-08 (2.19E-4)</td>
<td>7.25E-06 (2.06E-2)</td>
</tr>
<tr>
<td>$\Theta_s$</td>
<td>0.43</td>
<td>0.42</td>
<td>0.39</td>
</tr>
<tr>
<td>$a_f$</td>
<td>2500.0</td>
<td>618.7</td>
<td>602.0</td>
</tr>
<tr>
<td>$n_f$</td>
<td>0.293</td>
<td>0.202</td>
<td>1.065</td>
</tr>
<tr>
<td>$m_f$</td>
<td>0.955</td>
<td>1.687</td>
<td>5.428</td>
</tr>
</tbody>
</table>

**Gradation of Soils**

![Gradation of Soils](image)

**Figure 5.1 Gradations of Soils (Hassan and White, 1996(a)).**
The term dense or well-graded aggregate refers to an aggregate with gradation that is near to its maximum density. The researches have indicated that the permeability of aggregates is highly related to the content of fines. Therefore, a dense-graded aggregate usually has a low saturated permeability. The gradation and material properties of two typical dense graded aggregates are shown in Table 5.2. In response to deleterious effect of fines on aggregate permeability, open-graded aggregates are used to improve the drainage capability of the pavement. An open-graded aggregate is often used in a drainage blanket to help with pavement drainage. In current practices, the typical open gradations include PenOGA (Highlands and Hoffman, 1988), NJ Mix (Richardson, 1997), AASHTO #57 (AASHTO, 1993), and AASHTO #67 (AASHTO, 1993). The gradation and saturated permeability of some dense or open graded aggregates are presented in Table 5.2 and Figure 5.3. The SWCC of these materials, which illustrated in Figure 5.4, were estimated using the procedures presented in Chapter 4.0. Open graded aggregates or coarse sands are also used as backfill material for the edgedrain trenches.
Table 5.2 Material Property and Gradation of Untreated Aggregates (Highlands and Hoffman, 1988, Richardson, 1997, Randolph, 1996, Hassan and White, 1996)

<table>
<thead>
<tr>
<th>Sieve Name mm (inches)</th>
<th>No.57 PANo.2B</th>
<th>PA High Perm.</th>
<th>IN #8 Agg.</th>
<th>NJ Mix</th>
<th>PenOGS</th>
<th>IN #53 Agg.</th>
<th>PA No.2A DGA</th>
<th>MoDOT DGA</th>
</tr>
</thead>
<tbody>
<tr>
<td>50.8 (2)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>38.1 (3/2)</td>
<td>100</td>
<td>98</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>98</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>25.4 (1)</td>
<td>94</td>
<td>88</td>
<td>100</td>
<td>97.5</td>
<td>100</td>
<td>80-100</td>
<td>88</td>
<td>100</td>
</tr>
<tr>
<td>19.05 (3/4)</td>
<td>66</td>
<td>72</td>
<td>100</td>
<td>87</td>
<td>82</td>
<td>70-90</td>
<td>80</td>
<td>90</td>
</tr>
<tr>
<td>12.7 (1/2)</td>
<td>30</td>
<td>44</td>
<td>100</td>
<td>70</td>
<td>60</td>
<td>55-80</td>
<td>65</td>
<td>75</td>
</tr>
<tr>
<td>9.525 (3/8)</td>
<td>15</td>
<td>33</td>
<td>95</td>
<td>63</td>
<td>50</td>
<td></td>
<td>55</td>
<td>67.5</td>
</tr>
<tr>
<td>4.75 (No.4)</td>
<td>4</td>
<td>11</td>
<td>75</td>
<td>47.5</td>
<td>30</td>
<td>35-60</td>
<td>37</td>
<td>50</td>
</tr>
<tr>
<td>2.35 (No.8)</td>
<td>1</td>
<td>8</td>
<td>42</td>
<td>15</td>
<td>18</td>
<td>25-50</td>
<td>28</td>
<td>40</td>
</tr>
<tr>
<td>1.18 (No.16)</td>
<td>0</td>
<td>5.5</td>
<td>5</td>
<td>4</td>
<td>7</td>
<td></td>
<td>25</td>
<td>32</td>
</tr>
<tr>
<td>0.6 (No.30)</td>
<td>0</td>
<td>5.5</td>
<td>2</td>
<td>3</td>
<td>5.8</td>
<td></td>
<td>22</td>
<td>27.5</td>
</tr>
<tr>
<td>0.4 (No.40)</td>
<td>0</td>
<td>5</td>
<td>2</td>
<td>2.75</td>
<td>5</td>
<td></td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>0.3 (No.50)</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>2.5</td>
<td>4</td>
<td>12-30</td>
<td>18</td>
<td>22</td>
</tr>
<tr>
<td>0.15 (No.100)</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>3</td>
<td></td>
<td>8</td>
<td>16</td>
</tr>
<tr>
<td>0.075 (No.200)</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>2</td>
<td>0-10</td>
<td>4</td>
<td>9</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>1.65</td>
<td>1.76</td>
<td>1.26</td>
<td>1.92</td>
<td>2.03</td>
<td>2.3</td>
<td>2.10</td>
<td>2.19</td>
</tr>
<tr>
<td>Permeability (cm/s (ft/day))</td>
<td>12.27</td>
<td>5.38</td>
<td>1.18</td>
<td>0.28</td>
<td>0.24</td>
<td>0.00355</td>
<td>0.00043</td>
<td>0.00008</td>
</tr>
<tr>
<td>$\Theta_s$</td>
<td>0.382</td>
<td>0.340</td>
<td>0.566</td>
<td>0.279</td>
<td>0.224</td>
<td>0.279</td>
<td>0.186</td>
<td>0.129</td>
</tr>
<tr>
<td>$a_t$</td>
<td>0.116</td>
<td>0.117</td>
<td>0.070</td>
<td>0.118</td>
<td>0.305</td>
<td>0.080</td>
<td>0.712</td>
<td>5.332</td>
</tr>
<tr>
<td>$n_t$</td>
<td>1.883</td>
<td>1.847</td>
<td>1.041</td>
<td>1.840</td>
<td>1.378</td>
<td>0.447</td>
<td>1.338</td>
<td>1.418</td>
</tr>
<tr>
<td>$m_t$</td>
<td>1.312</td>
<td>1.304</td>
<td>2.038</td>
<td>1.302</td>
<td>1.092</td>
<td>1.463</td>
<td>0.935</td>
<td>0.720</td>
</tr>
</tbody>
</table>
Figure 5.3 Gradations of Untreated Aggregates.

Figure 5.4 Predicted SWCC of Untreated Aggregates.
As mentioned earlier, to increase the stability of a pavement, aggregate layers are often treated with asphalt or portland cement. The treated aggregate usually has a lower permeability than the untreated aggregates. The material properties and aggregate gradations of some asphalt treated base materials typically used in pavement construction are listed in Table 5.3 and Figures 5.5 and 5.6. The SWCC of such materials are estimated using procedures and equations presented in Chapter 4.0. Hot mix asphalt surface layers are often dense and rather impermeable when compared to asphalt treated bases. The typical permeability of asphalt concrete is $10^{-4}$ cm/s (0.283 ft/day). Cooley et al. (2001) reported that the critical permeability of asphalt concrete is about $10^{-3}$ cm/s (2.83 ft/day). In this research, the asphalt surface course IN#11 and binder/surface course IN#8 presented in InDOT project (Hassan et al., 1996(a)) were selected as asphalt surface alternatives. These two materials have similar permeability but different SWCC. The material properties are listed in Table 5.3 and Figures 5.5 and 5.6.
Table 5.3 Material Property and Gradation of Asphalt Treated Aggregates (Highlands and Hoffman, 1988, Randolph, 1996, Hassan and White, 1996)

<table>
<thead>
<tr>
<th>Sieve Size (mm (inches))</th>
<th>No.57 (PANo.2B)</th>
<th>PA ATM</th>
<th>NJ Mix</th>
<th>IN#5C (MnPASB)</th>
<th>IN #2</th>
<th>IN #5D</th>
<th>IN #11 HMA Surf.</th>
<th>IN #8 AC Surf.</th>
</tr>
</thead>
<tbody>
<tr>
<td>50.8 (2)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>38.1 (3/2)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>68.9</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>25.4 (1)</td>
<td>94</td>
<td>98</td>
<td>94</td>
<td>88.5</td>
<td>39.3</td>
<td>95.4</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>19.05 (3/4)</td>
<td>66</td>
<td>85</td>
<td>79.9</td>
<td>61.8</td>
<td>28.4</td>
<td>79.8</td>
<td>100</td>
<td>92.5</td>
</tr>
<tr>
<td>12.7 (1/2)</td>
<td>30</td>
<td>56</td>
<td>61.8</td>
<td>35.7</td>
<td>20.3</td>
<td>67.5</td>
<td>100</td>
<td>68</td>
</tr>
<tr>
<td>9.525 (3/8)</td>
<td>15</td>
<td>45</td>
<td>51.7</td>
<td>27.5</td>
<td>17.5</td>
<td>60</td>
<td>85</td>
<td>55</td>
</tr>
<tr>
<td>4.75 (No.4)</td>
<td>4</td>
<td>16</td>
<td>33.9</td>
<td>15.3</td>
<td>11.7</td>
<td>45</td>
<td>62.5</td>
<td>35</td>
</tr>
<tr>
<td>2.35 (No.8)</td>
<td>2</td>
<td>6</td>
<td>17.5</td>
<td>12.4</td>
<td>9.7</td>
<td>35</td>
<td>45</td>
<td>25</td>
</tr>
<tr>
<td>1.18 (No.16)</td>
<td>0</td>
<td>0</td>
<td>5.2</td>
<td>10.5</td>
<td>8.6</td>
<td>25</td>
<td>35</td>
<td>20</td>
</tr>
<tr>
<td>0.6 (No.30)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>8.9</td>
<td>7.4</td>
<td>17.5</td>
<td>25.5</td>
<td>15.5</td>
</tr>
<tr>
<td>0.4 (No.40)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>7</td>
<td>6</td>
<td>13</td>
<td>22</td>
<td>13.5</td>
</tr>
<tr>
<td>0.3 (No.50)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>5.4</td>
<td>5.5</td>
<td>11</td>
<td>18</td>
<td>12</td>
</tr>
<tr>
<td>0.15 (No.100)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>5.3</td>
<td>3.9</td>
<td>7</td>
<td>12</td>
<td>8</td>
</tr>
<tr>
<td>0.075 (No.200)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>3.4</td>
<td>3.1</td>
<td>3.5</td>
<td>4</td>
<td>3</td>
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<tr>
<td>Asphalt (%)</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.1</td>
<td>2.3</td>
<td>5.3</td>
<td>5.5</td>
<td>5.1</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>1.59</td>
<td>1.81</td>
<td>2.16</td>
<td>1.97</td>
<td>2.24</td>
<td>2.32</td>
<td>2.21</td>
<td>2.15</td>
</tr>
<tr>
<td>Permeability (cm/s (ft/day))</td>
<td>11</td>
<td>2.28</td>
<td>0.21</td>
<td>0.12</td>
<td>0.016</td>
<td>0.00014</td>
<td>0.00010</td>
<td>0.00011</td>
</tr>
<tr>
<td>θₙ</td>
<td>0.374</td>
<td>0.289</td>
<td>0.077</td>
<td>0.164</td>
<td>0.041</td>
<td>0.018</td>
<td>0.019</td>
<td>0.043</td>
</tr>
<tr>
<td>aₙ</td>
<td>0.017</td>
<td>0.048</td>
<td>0.572</td>
<td>0.866</td>
<td>0.024</td>
<td>1025.400</td>
<td>5.260</td>
<td>2.485</td>
</tr>
<tr>
<td>nₙ</td>
<td>2.187</td>
<td>1.921</td>
<td>1.522</td>
<td>1.525</td>
<td>1.273</td>
<td>0.410</td>
<td>7.000</td>
<td>5.999</td>
</tr>
<tr>
<td>mₙ</td>
<td>1.330</td>
<td>1.379</td>
<td>1.327</td>
<td>1.105</td>
<td>1.827</td>
<td>1.360</td>
<td>0.930</td>
<td>0.059</td>
</tr>
<tr>
<td>hₙ</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
<td>3000</td>
<td>7.260</td>
<td>0.053</td>
</tr>
</tbody>
</table>
Figure 5.5 Aggregate Gradations of Treated Aggregate Base and HMA Surface.

Figure 5.6 SWCC of Treated Aggregate Base and HMA Surface.
In this chapter, the effect of various pavement materials on the performance of PSDS will be investigated. To simplify the analysis while keeping it relevant, all of the analyses in this chapter were based upon a model that was designed for the SPS project of the Long-Term Performance Program (LTPP). This model consisted of a 0.5m (20 inches) subgrade layer, a 100mm (4 inches) filter layer, two 100mm (4 inches) base layers, and a 100mm (4 inches) surface layer. At the right side of this model is a 0.6m (2 ft)-edgedrain, in which a 100mm (4 inches) diameter-collector pipe is placed at the bottom. The longitudinal and transverse slopes are 2% and the slope of the outlet pipe is 4%. For all of the analyses, a 76.2mm (3 inches) rainfall lasting for 6 hours was used as input, and the flow at the end of outlet pipe and the degrees of saturation for each layer were recorded for comparison.

### 5.2 Effects of Base and Subbase Material Properties

Historically, dense graded aggregates have been used in base and subbase applications, primarily to keep the cost low. The recognition that good subsurface drainage can extend the life of a pavement has led to a greater use of permeable bases. In current design procedures, the permeability of the base and/or subbase is usually considered to be the most important drainage factors. Among current drainage practices, the permeability of drainable bases range from 0.01 to 10 cm/s (2.83 to 2830 ft/day). To compare the effectiveness of various base materials, a series of finite element models were designed and listed in Table 5.4. These models have the same geometry, initial condition, and boundary conditions. The materials used in the layers other than the base and subbase layers are the same too. The base layer and subbase layer are both 100mm (4 inches) thick. In the model MP1 to MP5, the same material was applied to both base and subbase layers; while in the model MP6 to MP13, the base and subbase take turns in receiving different materials. Among these models, MP1 was a pavement without a permeable base and subbase. Models MP2 and MP3 consisted of permeable base with permeability around 0.01cm/s (2.83 ft/day). Models MP4 and MP5 consisted of permeable base with permeability greater than 5cm/s (1415 ft/day). The analysis results were shown in Figures 5.7 to 5.12.
### Table 5.4 Models for Analyses of Base and Subbase Effect

<table>
<thead>
<tr>
<th>Model</th>
<th>subgrade</th>
<th>Filter</th>
<th>Subbase</th>
<th>Base</th>
<th>Surface</th>
<th>Trench</th>
</tr>
</thead>
<tbody>
<tr>
<td>MP1</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP2</td>
<td>IN Sample 2.1</td>
<td>PA No.3A DGA</td>
<td>IN#2</td>
<td>IN#2</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP3</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP4</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>PA ATPM.</td>
<td>PA ATPM.</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP5</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>Untreated No. 57</td>
<td>Untreated No. 57</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP6</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>IN#5C</td>
<td>IN#2</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP7</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>Untreated NJ Mix</td>
<td>IN#2</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP8</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>PA ATPM.</td>
<td>IN#2</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP9</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>Untreated No. 57</td>
<td>IN#2</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP10</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>IN#2</td>
<td>IN#5C</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP11</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>IN#2</td>
<td>Untreated NJ Mix</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP12</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>IN#2</td>
<td>PA ATPM.</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
<tr>
<td>MP13</td>
<td>IN Sample 2.1</td>
<td>PA No.2A DGA</td>
<td>IN#2</td>
<td>Untreated No. 57</td>
<td>IN #11 Surface</td>
<td>IN #8 Agg</td>
</tr>
</tbody>
</table>

### Table 5.5 Drainage Characteristics of Base and Subbase Models

<table>
<thead>
<tr>
<th>Model</th>
<th>MP1</th>
<th>MP2</th>
<th>MP3</th>
<th>MP4</th>
<th>MP5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time to 95% drain (hour)</td>
<td>37</td>
<td>14</td>
<td>16</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td>Total Infiltration m³ (ft³)</td>
<td>0.772 (27.2)</td>
<td>0.692 (24.4)</td>
<td>0.849 (30.0)</td>
<td>0.601 (21.2)</td>
<td>0.857 (30.2)</td>
</tr>
<tr>
<td>Peak Flow m³/hr (ft³/hr)</td>
<td>0.054 (1.91)</td>
<td>0.082 (2.89)</td>
<td>0.090 (3.18)</td>
<td>0.077 (2.72)</td>
<td>0.094 (3.32)</td>
</tr>
</tbody>
</table>
From Figure 5.7 and Table 5.5 one can see that the use of permeable base can improve pavement drainage significantly. The pavement with impermeable base (PA No.2A DGA) needs 37 hours to drain 95% of outflow; while the time to drain 95% with permeable bases are just 14 to 16 hours. Compared to the pavement with impermeable base, the pavement with permeable base have adequate drainage capacity for the sever rainfall that used in these analyses (76.2mm (3 inches), 6 hour rainfall). Generally, the total drainage and peak flow increase with the base permeability. However, this increase becomes less significant as base permeability reaches an upper limit. For example, when base permeability increases from 2.27 to 12.27 cm/s (6435 to 34781 ft/day), the pavement drainage characteristics exhibit very little change (Figure 5.7). In addition, the drainage is not only affected by the permeability but also by the water retention of the base materials. PA ATPM has a higher permeability than IN#2 and IN#5C, but the pavement with PA ATPM base exhibits a lower outflow and peak flow because PA ATPM has a low water retention ability than the other two materials.

Figure 5.8 shows the effect of base and subbase material combination on drainage system outflow. From this figure one can see that replacement of subbase with IN#2 (Hassan et al., 1996) will change the drainage characteristics of the pavements. If IN#5C (Hassan et al., 1996)
or No.57 aggregates (AASHTO, 1993) are used as drainage base, the replacement of subbase tends to decrease the total outflow and peak flow. However, when PA ATPM (Highlands et al., 1988) is used as a drainage base, the replacement of subbase tends to increase the total infiltration and peak flow. This means that the combination of the base and subbase can change the pore-water pressure distribution within the pavement and thus affect the drainage characteristics. Therefore, the drainage characteristics are not only affected by the permeability of the layers but also by the water retention ability of these layers.

**Figure 5.8 Effects of Base and Subbase Combinations on Outflow.**

In current drainage practices, the use of a drainage blanket is very popular. Some researchers recommended that the drainage blanket should be placed right below the surface to improve the efficiency of the PSDS (Cedergren, 1974). However, if the drainage blanket is not treated with asphalt or Portland cement, this may pose pavement stability problems. The effect of drainage layer position on outflow is shown in Figure 5.9. From these comparisons one can see that the position of drainage blanket does not affect the drainage characteristics significantly.
Other than infiltration and outflow, the saturation of pavement layers is another important parameter which is used to evaluate the performance of the PSDS. The saturation conditions of pavement with different base and subbase are shown by Figures 5.10 through 5.12. The results of these analyses indicated that the base and subbase material properties have very little effect on the saturation of subgrade, but have a significant effect on the saturation of base and surface layers. The application of a permeable base can significantly reduce the degree of saturation of surface and base, and thus can reduce the detrimental effects of water on those components. From these figures one also can see that the degree of saturation in a layer is not uniform during the drainage process. The water content at the location close to outlet pipe is often higher than the location far away from the outlet pipe.
Figure 5.10.a Effect of Base and Subbase Material on Subgrade Saturation: at Location (x=0, y=10, z=0.7)—Away from Outlet.

Figure 5.10.b Effect of Base and Subbase Material on Subgrade Saturation: at Location (x=3.6, y=1, z=0.448)—Close to Outlet.
Effect of Base and Subbase Material on Base Saturation at Location (x=0, y=10, z=1.1)

Figure 5.11.a Effect of Base and Subbase Material on Base Saturation: at Location (x=0, y=10, z=1.1)—Away from Outlet.

Effect of Base and Subbase Material on Base Saturation at Location (x=3.6, y=1, z=0.848)

Figure 5.11.b Effect of Base and Subbase Material on Base Saturation: at Location (x=3.6, y=1, z=0.848)—Close to Outlet.
Figure 5.12.a Effect of Base and Subbase Material on Surface Saturation: at Location (x=0, y=10, z=1.2)—Away from Outlet.

Figure 5.12.b Effect of Base and Subbase Material on Surface Saturation: at Location (x=3.6, y=1, z=0.948)—Close to Outlet.
5.3 Effects of Filter/Separator Material Properties

Filter or separator layer is designed to separate a permeable base layer from an adjacent soil containing fines. This is done to protect the permeable material from clogging up. To block the transport of fines in soils, the materials used in filter layers must offer a low permeability and high stability at high saturation. The filter layers are usually constructed with dense-graded aggregate or geotextile. To increase pavement stability, the dense graded aggregate filters are often stabilized by using asphalt or portland cement, such as: IN #5D listed in Figure 5.5. Geotexiles are very popular in filter applications. Geotexitiles are thin fabrics and with high permeability and low water retention ability, so that their direct effect on pavement drainage will be very small. This has been verified in the 2-D PSDS analysis of Stormont’s report (Stormont, 2001). Thus, in this research, only the effect of dense graded aggregate filter (treated or untreated) is considered. The models designed for the analyses of filter effects on pavement drainage are shown in Table 5.5. The analysis results are illustrated in Figures 5.13 through 5.15. In these figures, the base and subbase for models MP3, MP14 and MP15 was IN#5C (Hassan et al., 1996); the base and subbase of models MP5, MP16 and MP17 was untreated No.57 aggregate. The three materials which were selected for filter layer were IN#53 aggregate, PA No.2A DGA, and IN#5D asphalt-treated DGA.

Table 5.6 Models for Analyses of Filter/Separator Effect

<table>
<thead>
<tr>
<th>Model</th>
<th>subgrade</th>
<th>Filter</th>
<th>Base-1</th>
<th>Base-2</th>
<th>Surface</th>
<th>Trench</th>
</tr>
</thead>
<tbody>
<tr>
<td>MP14</td>
<td>IN#11 Surface</td>
<td>IN #53 coarse Agg.</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN Sample 2.1</td>
<td>IN #8 Agg.</td>
</tr>
<tr>
<td>MP15</td>
<td>IN#11 Surface</td>
<td>IN #5D</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN Sample 2.1</td>
<td>IN #8 Agg.</td>
</tr>
<tr>
<td>MP16</td>
<td>IN#11 Surface</td>
<td>IN #53 coarse Agg.</td>
<td>UnTr No. 57</td>
<td>UnTr No. 57</td>
<td>IN Sample 2.1</td>
<td>IN #8 Agg.</td>
</tr>
<tr>
<td>MP17</td>
<td>IN#11 Surface</td>
<td>IN #5D</td>
<td>UnTr No. 57</td>
<td>UnTr No. 57</td>
<td>IN Sample 2.1</td>
<td>IN #8 Agg.</td>
</tr>
</tbody>
</table>
The results shown in Figure 5.13 indicate that the effect of filter layer on pavement drainage is highly dependent on the base and subbase materials. When IN#5C is used as base and subbase layers, the effect of filter material is not significant. However, when No.57 aggregate is used as base and subbase, the effect of filter material is significant, and the use of materials with lower permeability tends to reduce the rainfall infiltration and shorten the time-to-drain. This phenomenon points out that the performance of drainage system is affected by the compatibility of the layer materials. It is also demonstrated that the drainage performance is not only affected by the material permeability but also by the water retention of the materials.

Among the three selected materials for filter layer, the asphalt treated DGA tended to absorb least rainfall water and drain the infiltration water out most quickly. However, this dose not mean that IN#5D is the best material that can be used as a filter layer because it brings with it the highest filter saturation. From Figure 5.14 one can see that the degree of saturation of filter layer in models MP15 and MP17 are always as high as 95%, which can cause potential pavement performance problems. The analysis results of the filter effect also showed that the material properties of filter layer do not have any effect on the saturation of subgrade, base, and surface layers.
Effect of Filter Material on Filter Saturation at Location (x=0,y=10,z=0.9)

Figure 5.14 Effects of Filter Material Properties on Saturation of Filter Layer.

Effect of Filter Material on Base Saturation at Location (x=0,y=10,z=1.1)

Figure 5.15 Effects of Filter Material Properties on Saturation of Base Layer.
5.4 Effects of Surface Material Properties

The surface material properties are important factors that affect the performance of PSDS because the surface infiltration phenomenon. The effect of SWCC of surface materials will be investigated in this section. In the available InDOT data, IN#11 surface and IN#8 binder course (Hassan et al., 1996) have the same permeability but different SWCC because of the different aggregate gradation. From Figure 5.6 one can see that the IN#8 binder course has higher water retention capability than IN#11 surface. These two materials were selected as alternative surface materials for the study of the effect of SWCC. The models that were designed for this comparison are listed in Table 5.7. The results of analyses are illustrated in Figures 5.16 through 5.18. In these figures, the base and subbase material for models MP1 and MP18 was PA No.2A DGA; the base and subbase material of models MP3 and MP19 was IN#5C; and the base and subbase material of models MP5 and MP20 was untreated No.57 Aggregate.

Table 5.7 Models for Analyses of Surface Material Effect

<table>
<thead>
<tr>
<th>Model</th>
<th>subgrade</th>
<th>Filter</th>
<th>Base-1</th>
<th>Base-2</th>
<th>Surface</th>
<th>Trench</th>
</tr>
</thead>
<tbody>
<tr>
<td>MP18</td>
<td>IN#8 Surface</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>IN Sample 2.1</td>
<td>IN#8 Agg.</td>
</tr>
<tr>
<td>MP19</td>
<td>IN#8 Surface</td>
<td>PA No.2A DGA</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN Sample 2.1</td>
<td>IN#8 Agg.</td>
</tr>
<tr>
<td>MP20</td>
<td>IN#8 Surface</td>
<td>PA No.3A DGA</td>
<td>UnTr No. 57</td>
<td>UnTr No. 57</td>
<td>IN Sample 2.1</td>
<td>IN#8 Agg.</td>
</tr>
</tbody>
</table>

The analysis results shown in Fig 5.18 indicate that the SWCC of surface material does have an effect on the pavement drainage characteristics even though this effect is not very apparent. For a pavement with DGA or No.57 Aggregate as a drainage layer, the substitute of the surface material IN#11 with IN#8 increases the total flow and peak flow of the PSDS. Since the residual water content of IN#8 binder is higher than that of IN#11 surface, the surface degree of saturation after rainfall event is increased significantly when IN#8 is used. This may contribute to the deterioration of the surface. The saturation of base is not affected by the surface material SWCC if a permeable base is used.
Effect of Surface Material on Outflow

Figure 5.16 Effects of Surface Material Properties on Outflow.

Effect of Surface Material on Surface Saturation
at Location (x=0, y=10, z=1.2)

Figure 5.17 Effects of Surface Material Properties on Surface Saturation.
5.5 Effects of Sugrade Soil Properties

Subgrade soil is an important component of a pavement because this layer serves as the foundation for the pavement. Unlike other pavement components, subgrade soil properties are dictated by the geographical factor of the pavement location, and these factors cannot be changed as easy as other pavement materials. The effects of soil on PSDS performance need to be studied for drainage design purpose. The models for this analysis shown in Table 5.8 and the analysis results are shown in Figures 5.19 through 5.22. In these analyses, the soils of IN Sample 1.2, IN Sample 2.1, and IN Sample 3.1 (Hassan et al., 1996) were selected to predict various subgrade soil conditions. When soil suction was less than 10 kPa, Sample 1.2 showed moderate permeability and low SWCC slope; Sample 2.1 showed low permeability and high SWCC slope; and Sample 3.1 showed high permeability and low SWCC slope. This means that the Sample 3.1 has a higher hydraulic conductivity than other soils and the Sample 2.1 has a lower water retention capability than other soils. The pavement models used in this part included the following base materials: PA No.2A DGA, IN#5C and untreated No.57 aggregate.
### Table 5.8 Models for Analyses of Subgrade Soil Effect

<table>
<thead>
<tr>
<th>Model</th>
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<th>Base-1</th>
<th>Base-2</th>
<th>Surface</th>
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<td>MP21</td>
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<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>IN Sample 1.2</td>
</tr>
<tr>
<td>MP22</td>
<td>IN #11</td>
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<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>IN Sample 3.1</td>
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<td>MP23</td>
<td>IN #11</td>
<td>PA No.2A DGA</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN Sample 1.2</td>
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<tr>
<td>MP24</td>
<td>IN #11</td>
<td>PA No.2A DGA</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN Sample 3.1</td>
</tr>
<tr>
<td>MP25</td>
<td>IN #11</td>
<td>PA No.3A DGA</td>
<td>UnTr No. 57</td>
<td>UnTr No. 57</td>
<td>IN Sample 1.2</td>
</tr>
<tr>
<td>MP26</td>
<td>IN #11</td>
<td>PA No.3A DGA</td>
<td>UnTr No. 57</td>
<td>UnTr No. 57</td>
<td>IN Sample 3.1</td>
</tr>
</tbody>
</table>

#### Figure 5.19 Effects of Subgrade Soil Properties on Outflow.

From Fig. 5.19 one can see that the permeability of subgrade is important to the drainage of pavement with a permeable base, the higher the subgrade permeability the lower the peak flow and total outflow. However, the effect of subgrade permeability on total outflow of pavements with an impermeable base is not apparent. The time to drain of pavements tends to decrease with the increase of subgrade permeability. But the decrease is very limited for pavement with impermeable base. The effects of subgrade material on saturation of pavement layers are shown in Figures 5.20 through 5.23. From these figures one can see that the saturation of subgrade is mainly determined by the SWCC of subgrade soil. The Sample 2.1 showed the highest SWCC slope within 100 kPa and thus the pavement subgrade experienced the lowest degree of saturation. The degree of saturation of Samples 1.2 and 3.1 are much higher even...
though they have higher permeability. The subgrade soil hydraulic properties can also influence the saturation of base and surface layers, especially for the pavements with impermeable bases. The decrease of permeability of subgrade soil tends to increase the degree of saturation of base and surface layers.

Figure 5.20 Effects of Subgrade Soil Properties on Subgrade Saturation.

Figure 5.21 Effect of Subgrade Soil Properties on Base Saturation.
5.6 Effects of Trench Material Properties

The field observations showed that the use of an edgdrain system can improve the pavement drainage capability significantly (Allen et al., 1991). The edgdrain system usually consists of a trench, a collection pipe, and an outlet pipe and its accessories. The trench is a component of the pavement drainage system that is designed to carry water to the collection pipe. It usually is filled with aggregate materials wrapped with geotextiles. In the current practice, the most frequently used trench backfill material is coarse sands, such as IN#8 Aggregate as shown in Table 5.2. However, a research project in Kentucky has shown that using AASHTO #57 aggregate as trench backfill can improve the drainage capacity of pavements (Allen et al., 1991). The effects of trench backfill are discussed in this section. As mentioned earlier, the geotextile is a thin fabric and has higher permeability than the surrounding materials, so that it has very limited effect on the drainage of pavement. To simplify the model, the geotextile was not included in the 3-D model. The models listed in Table 5.9 were designed to study the influence of trench materials. Three materials, IN#8 Aggregate, No.57 Aggregate, and IN#53...
Aggregate, were selected as the trench backfill alternatives. Among these three, the No. 57 aggregate has highest permeability and IN#53 has highest water retention capability. The analysis results are shown in Figures 5.23 through 5.25. In these figures, the base material of MP1, MP27, and MP28 was PA No.2A DGA; the base material of MP3, MP29, and MP30 was IN#5C; and the base material of MP5, MP31, and MP32 was UnTr No. 57.

Table 5.9 Models for Analyses of Trench Material Effect

<table>
<thead>
<tr>
<th>Model</th>
<th>subgrade</th>
<th>Filter</th>
<th>Base-1</th>
<th>Base-2</th>
<th>Surface</th>
<th>Trench</th>
</tr>
</thead>
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<td>IN #11</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>IN Sample 2.1</td>
<td>IN Sample 2.1</td>
</tr>
<tr>
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<td>IN #11</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>PA No.2A DGA</td>
<td>IN Sample 2.1</td>
<td>UnTr No. 57</td>
</tr>
<tr>
<td>MP29</td>
<td>IN #11</td>
<td>PA No.2A DGA</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN Sample 2.1</td>
<td>IN Sample 2.1</td>
</tr>
<tr>
<td>MP30</td>
<td>IN #11</td>
<td>PA No.2A DGA</td>
<td>IN#5C</td>
<td>IN#5C</td>
<td>IN Sample 2.1</td>
<td>UnTr No. 57</td>
</tr>
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<td>MP31</td>
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<td>PA No.3A DGA</td>
<td>UnTr No. 57</td>
<td>UnTr No. 57</td>
<td>IN Sample 2.1</td>
<td>IN Sample 2.1</td>
</tr>
<tr>
<td>MP32</td>
<td>IN #11</td>
<td>PA No.3A DGA</td>
<td>UnTr No. 57</td>
<td>UnTr No. 57</td>
<td>IN Sample 2.1</td>
<td>UnTr No. 57</td>
</tr>
</tbody>
</table>

The analysis results indicate that the effect of trench material on pavement drainage is highly dependent on the hydraulic properties of base and subbase materials. Considering the extreme cases, for a pavement with impermeable base, the effect of trench backfill is not significant. However, for a pavement with a permeable base, the trench material influences the drainage characteristics. Generally the increase of the trench permeability will reduce the total outflow and peak flow, but the time to drain is not influenced by the trench. The analysis results also showed that the trench material properties have no effect on the saturation of various pavement layers. The effect of trench material on subgrade and base saturation is shown in Figures 5.24 and 5.25 respectively. In these models, the trench that was backfilled by IN#53 coarse aggregate tends to drain more water than the other trench materials. However, this dose not mean that the IN#53 coarse aggregate is a good choice for the trench. Since the time-to-drain and layer saturations are not affected by the trench, the increase of outflow and peak flow means more surface water was sucked into the pavement when fine material is filled into the trench. This will increase the adequacy requirement of the drainage system. Therefore, for pavement
with permeable base, coarse materials or materials with high permeability and low water retention should be used as trench material. Fine materials can be used for the pavement with an impermeable base.

**Figure 5.23 Effect of Trench Material Property on Outflow**

**Figure 5.24 Effect of Trench Material on Subgrade Saturation**
Effect of Trench Material on Base Saturation at Location (x=0,y=10,z=1.1)

Figure 5.25 Effect of Trench Material on Base Saturation

5.7 Summary of Analysis Results and Recommendations

The material property sensitivity analyses conducted in this section indicate that the use of a permeable base can significantly improve the drainage capability of pavements. Generally, the outflow and peak flow of the pavement increase with the permeability of base material. However, this increase becomes less significant when base permeability is above 0.1 cm/s (283 ft/day). In addition, this trend may be inversed if the water retention ability of the material with a high permeability is much lower than the material with a low permeability. Pavement drainage performance is not influenced by the position of the permeable layer, but influenced somewhat by the combination of the base and subbase layer.

The peak flow of a pavement is mainly controlled by the material properties of surface, base, subbase, filter layers and trench. The effect of subgrade soil on peak flow is very limited. The time-to-drain is mainly controlled by the material properties of base, subbase, and subgrade soil. Trench materials have very limited effect on time-to-drain and layer saturations. The increase
of fines in trench material will increase the surface water infiltration and add burden to the drainage system.

When permeable base is used, the base layer was always under the unsaturated condition. The saturation condition of layers is mainly controlled by the water retention ability of the materials. The water retention of a material in one layer may influence the saturation of other layers. For example, the water retention of subgrade soil affects saturation conditions of all layers of a pavement.

From these analyses, one can see that the performance of PSDS is not determined by one or two components but by the combination of the entire system. The material of a component may be effective in one pavement structure but not necessarily in other structural arrangements. This makes the design of pavement drainage a complex task. To provide a positive drainage, the following recommendations should be considered for the material selection.

1. A permeable material with a permeability higher than 0.1cm/s (283 ft/day) should be used for the base or subbase layer.
2. To increase the drainage efficiency, one may use a filter layer with low void ratio.
3. Trench material should be selected based on the gradation of the adjacent materials, and the permeability of trench materials should not less than 1 cm/s (2830 ft/day).
4. The pavement should be constructed with materials that have low water retention ability.
5. For the HMA surface, the water retention property should be balanced with its permeability.
CHAPTER 6.0 PAVEMENT SUBSURFACE DRAINAGE ADEQUACY ESTIMATION

6.1 Pavement Drainage Design and Performance Evaluation

It is important to quantify the effectiveness of various pavement drainage schemes. Current practice is to use the time-to-drain method for the evaluation of drainage adequacy. The current recommendation is to provide a permeable base with an adequate permeability to meet the time-to-drain criteria (Christopher et al., 1997). The final value of permeability is a function of the time required to drain a saturated base layer for a given roadway geometry. The AASHTO classification of permeable base quality is based on the time required to drain the base from a 100 percent saturated to a 50 percent level and involves subjective ratings such as “excellent” (time to drain less than 2 hours), “good” (time to drain less than 1 day), “fair” (time to drain less than 7 days), and so on (AASHTO, 1993). A high-type pavement is expected to have an “excellent” quality of drainage. The FHWA computer program DRIP (Drainage Requirements In Pavements) is developed to performing the time-to-drain design (Wyatt et al., 1998).

This method is totally based on the saturated steady-state hydraulic analyses even though an unsaturated term has been used in the calculation of time-to-drain. However, the field observations and the analysis results of this research indicate that the permeable bases are always under unsaturated condition. The process that drains the base from 100% saturation to 50% saturation may rarely occur in the actual pavement. In addition, according to the analysis results in this research, the saturation of pavement layers is mainly controlled by the material’s water retention ability, and a quick drainage does not guarantee a low degree of saturation within the pavement layers. Therefore, the current time-to-drain criteria may not provide a suitable criterion for the pavement drainage ability.

A major component of a pavement drainage assessment program should be the determination of the inflow quantity that needs to be drained. In the newly proposed design method, the inflow quantity was calculated from the precipitation rate of that area (Huang, 1993).
The rainfall rate used in this design method is a 1-hour, 1-year frequency precipitation rate. This is the maximum rainfall in 1-hour that can be expected to occur on the average once a year, which is considered to be a critical condition for the pavement drainage ability.

As mentioned earlier, the amount of surface water that can enter into a given pavement will vary with the age and condition of that pavement; the width and spacing of cracks, joints, and so forth; and the porosity and permeability of the pavement material. Also, pavement maintenance operations can influence the water infiltration. The variation of these factors makes the estimation of infiltration very complicated. When there is no sophisticated modeling technique available, the only reasonable procedure appears to be to estimate surface water infiltration on the basis of a design precipitation rate. It was suggested by FHWA Guidelines (FHWA, 1973) that the design precipitation rate (1-hour, 1-year frequency rate) be multiplied by a coefficient between 0.5 and 0.67 for PCC pavements and 0.33 to 0.5 for asphalt pavements (Cedergren, 1974). Based on field test, Ridgeway found that the amount of infiltration can be related directly to cracking and suggested that an infiltration rate of 0.22 m$^3$/day/m (2.4 ft$^3$/day/ft) of crack inflow be used for design (Ridgeway, 1976). Minnesota DOT field test data reported that the infiltration rate for PCC pavement is typically 25 to 40 percent on average (Hagen, 1996). Rainwater et al. monitored the water flow following precipitation events in asphalt pavement. It was observed from these results that there is an upper limit to the rainfall intensity that can affect the infiltration of the pavement. A rainfall with intensity higher than this upper limit is not likely to increase the infiltration, because surplus rainfall is only handled through surface runoff (Rainwater, 2001).

The estimation of infiltration provides designers a rough approximation of the inflow for drainage design. However, pavement drainage adequacy can not be guaranteed if only such a rough estimation is used for all pavements under all climatic conditions. Under saturated steady-state conditions no factors relevant to time can be incorporated into the analyses. However, field observations had indicated that the duration of rainfall is a more critical factor than the intensity (Ridgeway, 1976). To estimate the infiltration quantities accurately, a practical way need to be developed including both the rainfall intensity and the duration in the
hydraulic analyses. Also the effects of pavement surface, climate, and material properties should be considered.

In order to develop a mathematical model, the influence of rainfall events is numerically described in this chapter. The effect of rainfall intensity, duration, and distribution on pavement drainage was investigated. Also, statistical experiments were designed and analyzed, through which a mathematical model for drainage performance prediction was developed.

6.2 Characterization of Rainfall Event

Field observations have shown that the infiltration quantities directly related to the rainfall intensity and duration (Ridgeway, 1976). In the numerical modeling of pavement drainage in this study, rainfall was viewed as surface boundary condition. In the surface boundary condition, rainfall was represented as a distribution of rainfall density over its duration. Thus, all characteristics of a rainfall event need to be reviewed and quantified for proper modeling of the PSDS.

6.2.1 Characteristics of Rainfall Events

Precipitation occurs when a mass of air is cooled to the dew point, at which point the water vapor in the air condenses into drops and falls as rain. There are four mechanisms for the formation of rainfall: frontal storms, convection storms, orographic precipitation, and tropical cyclones. Frontal storms persist for several days, and can cross the entire continent, causing precipitation over large areas. Warm fronts generally cause low intensity and long duration storms, while cold fronts have higher intensities and shorter durations. Convective Storms also called thunderstorms. They are short-lived (1-2 hours) storms that typically produce high intensity rainfall. The thunderstorms are triggered by unequal heating of air near the ground surface. Orographic storms are caused when wind moves moist air over a mountainous region, thereby increasing the altitude and reducing the temperature of the air mass. Tropical Cyclones are also called hurricanes; these storms are formed over oceans with warm surface temperatures. Hurricanes and tropical depressions often come ashore in the Southeast region of the US.
When they do, they bring with them large volumes as well as high intensities of rainfall (Potter, 2003).

Generally, precipitation varies over both space and time. The time variability in precipitation occurs at many temporal scales. It varies from year to year (due to climatic changes), from month to month (due to seasonal changes), and from minute to minute during a rainstorm. In the pavement design and analyses, precipitation is usually referred in monthly scale and described by monthly average rainfall amount, dry and wet days, and numbers of thunderstorms in ICM/EICM precipitation model (Lytton et al., 1990). The month temporal scale is adequate if only monthly average of pavement moisture is needed. However, for pavement drainage system design, the monthly scale becomes too large to be suitable. Because pavement drainage usually occurs during several hours to several days, an hourly scale is more suitable for this type of problems.

Rainfall intensity is the instantaneous rate of rainfall expressed in terms of depth of water/time, such as mm/hour. Typically, a low intensity rainfall is less than 5mm/hr (0.2 in./hr); a medium intensity rainfall is between 5 and 25mm/hr (0.2 to 1 in./hr); and a high intensity rainfall is greater than 25mm/hr (1 in./hr). An extreme intensity is 100mm/hr (4 in./hr). Rainfall duration is the time interval that the storm lasts. Normally, the greater the intensity of a rainfall, the less likely it is to last a long time. Long duration storms tend to have relatively low intensities, and high intensity storms tend to have short durations.

The temporal variations of rainfall usually were summarized using an intensity-duration-frequency (IDF) curve, which is a plot of the rainfall intensity for various durations. Rainfall probabilities are often described by recurrence intervals. A two-year storm has an average return interval of two years, and a twenty-five year storm has an average return interval of twenty-five years. In any given year, the probability of a two-year storm is 50%, and the probability of a twenty-five year storm is 4%. The probability of occurrence in any year is the reciprocal of the return period, and vice versa. Given enough rainfall observations, a specific intensity-duration curve can be calculated for different frequencies, or return periods. One cannot determine the probability of a precipitation volume or intensity without specifying a time
interval for the storm. For instance, a 25 mm/hr storm lasting two hours (50 mm total) has a return period of two years, but a 25 mm/hr storm lasting four hours (100 mm total) has a return period of almost twenty-five years. Figure 6.1 illustrate an IDF curve that developed for Louisville, Kentucky (Dupont et al., 1999). In this figure, time of concentration (T<sub>C</sub>) refers to rainfall duration, and T is the recurrence interval of a rainfall.

![Figure 6.1 An Example of Rainfall IDF Curve (courtesy, KTC report 00-18-SPR-178-98) (1 inch = 25.4mm).](image)

Using the IDF curve, one can get the critical rainfall intensity for a given duration and reoccurrence. The intensity obtained from IDF curve is the average rainfall intensity over the rainfall duration. While during an actual rainfall event the intensity is not constant but varies from hour to hour. Therefore, rainfall intensity distribution influences pavement drainage performance. In the PSDS model developed in this research, actual rainfall event is used as a boundary condition that varies with time. When the quantity relationship between rainfall distribution and pavement drainage is concerned, a mathematical presentation of rainfall distribution is required.

6.2.2 Rainfall Event Simulation

Rainfall intensity distribution is the pattern of rainfall distribution within the duration of a rainfall event. It should not be confused with the historical record of average precipitation
distribution. In general, rainfalls are usually distributed through their duration with a bell shape pattern or a combination of several bell shapes. The bell-shape may skews to the left or to the right due to individual storm characteristics. There are three typical patterns for the rainfall intensity distribution. Thunderstorms usually have a short duration and the peak intensity often appears at the very beginning of the rainfall duration. Thus, the distribution curve of this type of rainfall often skews to left. Hurricanes and cold frontal storms usually have moderate duration. Their distribution may consist of one or two peaks and the distribution curve may skew to the left or to the right. Warm frontal storms have a low intensity and a long duration. Their intensity curve usually consists of several low peaks. Examples of these typical distributions are listed in Figure 6.2.

![Recorded Rainfall Events](image)

**Figure 6.2 Rainfall Event Records:** Event #1 with high intensity and short duration; Event #2 with low intensity and long duration; Event #3 with moderate intensity and duration; Event #4 with low intensity and moderate duration.

Considering the mechanism of rainfall occurrence, storms may be viewed as arrival problems that may be described by the Poisson random variable (Serrano, 2001). Therefore, one can state a hypothesis that rainfall data comes from a Poisson population. The Poisson random variable (Young, 1962) is given by
\[
f_n(n) = \frac{a^ne^{-a}}{n!}, a = Np, \ n = 0,1,2,3,4... \]  \hspace{1cm} \text{(Equation 6.1)}

where

\[p = \text{probability of success in one trial},\]
\[N = \text{number of independent trails, and}\]
\[n = \text{number of successes}.\]

The Poisson distribution function is an approximation of the binomial distribution when \(N \sim \infty\) and \(p\) is very small. It is an efficient means of calculating the probability of \(n\) success using only one variable. In addition, the Poisson random variable remains a good approximation to the Binomial distribution even when the trails are not independent, provided that their dependence is weak (Serrano, 2001). The mean and the variance of the Poisson random \(n\) are found to be equal to \(a\) (Young, 1962). This property provides one with a very convenient tool for the simulation of rainfall event.

When Poisson distribution is applied to rainfall event simulation, the success number, \(n\), is corresponding to the rainfall duration, while the frequency, \(f\), is corresponding to the rainfall intensity. Considering the special properties of rainfall event, the following steps need to be taken to get a good fit for the measured rainfall data.

1. Observe the rainfall data. If it has more than one peak, divide it to segments to ensure that each segment only has one peak. Count the time units within each segment from 0.

2. Calculate the total rainfall amount of part one and divide rainfall amount at each time unit by this calculated value. Calculate the term “\(a\)” using Equation 6.2 (\(a\) equals the mean of \(n\)).

\[a_1 = \sum_{0}^{N} nf_n^1 \]  \hspace{1cm} \text{(Equation 6.2)}

where

\[a_1 = \text{parameter } a \text{ of segment one},\]
\[ N_i = \text{duration of segment one} \]
\[ f_{n1} = \text{rainfall amount ratio at time } n \text{ within segment one} \]

3. Substitute \( a_1 \) into Equation 6.1 to get the predicted rainfall distribution of segment one and multiply the predicted distribution by the total rainfall of segment one.

4. Repeat the same process to other segments and put the calculated distributions together in sequence.

The recorded rainfall in Figure 6.2 was used for the test of rainfall distribution hypothesis and the check of prediction correlation. The hypothesis is tested by Chi-square (Serrano, 2001), and the test statistic can be calculated using Equation 6.6.

\[
R = \sum_{i=1}^{N} \frac{(f_i - e_i)^2}{e_i} \tag{Equation 6.3}
\]

where

- \( N = \text{total rainfall duration} \),
- \( f = \text{rainfall intensity from rainfall data} \), and
- \( e_i = \text{rainfall intensity from prediction Equation} \).

The correlation relationship between rainfall data and the prediction value were checked using Pearson’s r-value. The test and simulation results are listed in Table 6.1 and Figure 6.3. From Table 6.1 one can see that for all of the four rainfall events, \( \chi^2_{N,0.05} > R \), which demonstrates that the rainfall events come from Poisson population and the hypothesis for rainfall distribution can be accepted. The Pearson’s r-values of all four rainfall events are greater than 0.75. Combining the information shown in Figure 6.3, one can say that the simulation developed in this research can get a good fit for the actual rainfall data.
Table 6.1 Rainfall Simulation Statistical Results

<table>
<thead>
<tr>
<th>Rainfall Event</th>
<th>Degrees of freedom</th>
<th>$R$</th>
<th>$\kappa^2_{N, 0.95}$</th>
<th>Pearson’s r-value</th>
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<tbody>
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<td>1</td>
<td>6</td>
<td>0.4786</td>
<td>1.635</td>
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<tr>
<td>2</td>
<td>13</td>
<td>0.3993</td>
<td>5.892</td>
<td>0.899</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>0.3964</td>
<td>6.325</td>
<td>0.855</td>
</tr>
<tr>
<td>4</td>
<td>8</td>
<td>0.3360</td>
<td>2.733</td>
<td>0.782</td>
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</tbody>
</table>

Figure 6.3.a Rainfall Event Simulation: Event #1 with high intensity and short duration.

Figure 6.3.b Rainfall Event Simulation: Event 2# with low intensity and long duration.
Figure 6.3.c Rainfall Event Simulation: Event #3 with moderate intensity and duration.

Figure 6.3.d Rainfall Event Simulation: Event #4 with low intensity and moderate duration.

For pavement drainage design, designers are more concerned about the impact of the rainfall on the pavement. However, due to the geographical variability of precipitation,
different locations may produce different critical conditions for the same pavement. Thus, a practical mathematical Equation that can be used to generate any rainfall event is very desirable for pavement drainage. Based upon the simulation method described in this section, Equations 6.4 and 6.5 can be used to create rainfall data with any intensity, duration, and distribution.

\[ I(a, Q, t) = \frac{Qa^{(t-1)}e^{-a}}{C(t-1)!}, \quad t = 1, 2, 3, ..., T \]  \hspace{1cm} \text{(Equation 6.4)}

\[ C = \sum_{s=0}^{T-1} \frac{s a^s e^{-a}}{s!}, \quad s = 0, 1, 2, ..., T - 1 \]  \hspace{1cm} \text{(Equation 6.5)}

where

- \( a \) = Poisson parameter,
- \( Q \) = total rainfall quantity during the rainfall event,
- \( T \) = rainfall duration,
- \( e \) = natural number,
- \( t \) = time, and
- \( C \) = summation of frequency for given \( T \) and \( a \).

This equation was developed based upon Poisson distribution equation with the term “\( a \)” as independent variable and “\( C \)” as a normalization factor. The summation of Poisson distribution within the given time interval equals to unity only under condition that the term “\( a \)” equals the mean of the time (Young, 1965). When \( a \) is treated as an independent variable, the summation of frequency within the given time interval is no longer equals to unity and the distribution need to be normalized to ensure that the total rainfall quantity of the generated model equals the given quantity.

An example of the generated rainfall is listed in Table 6.2 and Figure 6.4. From the IDF curve for the bluegrass in Kentucky, the 6-hour, 2-year rainfall intensity is 13.2mm/hr (0.52 in./hr). Thus, one can get \( Q = 6 \times 13.2 = 79.25 \text{ mm (3.12 in.)} \) and \( T = 6 \text{ hour} \). The rainfall intensity distribution data can be generated for different values of \( a \) using Equations 6.4 and 6.5.
### Table 6.2 Example of Generated Rainfall (mm)

<table>
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<tr>
<th>a</th>
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<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
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<td>48.07</td>
<td>24.03</td>
<td>6.01</td>
<td>1.00</td>
<td>0.13</td>
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<td>0</td>
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<td>0</td>
</tr>
<tr>
<td>1.5</td>
<td>0</td>
<td>17.76</td>
<td>26.64</td>
<td>19.98</td>
<td>9.99</td>
<td>3.75</td>
<td>1.12</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>10.91</td>
<td>21.81</td>
<td>21.81</td>
<td>14.54</td>
<td>7.27</td>
<td>2.91</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
<td>0</td>
<td>6.79</td>
<td>16.98</td>
<td>21.22</td>
<td>17.68</td>
<td>11.05</td>
<td>5.53</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>4.31</td>
<td>12.92</td>
<td>19.38</td>
<td>19.38</td>
<td>14.54</td>
<td>8.72</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>1.85</td>
<td>7.39</td>
<td>14.79</td>
<td>19.72</td>
<td>19.72</td>
<td>15.78</td>
<td>0</td>
</tr>
</tbody>
</table>

**Generated Rainfall Intensity Distribution**

![Figure 6.4 Computer Generated Rainfall Distributions for Different Poisson Parameters.](image-url)
Figure 6.5.a Sensitive Analysis of Rainfall Generation Equation to Parameter $a$ and $T$: $a = 1$.

Figure 6.5.b Sensitive Analysis of Rainfall Generation Equation to Parameter $a$ and $T$: $a = 4$. 
Figure 6.5.c Sensitive Analysis of Rainfall Generation Equation to Parameter $a$ and $T$: $a=8$.

Figure 6.5 shows that for a given duration, the peak time of rainfall will be more shifted to the right as the $a$-parameter increase. For a given $a$-parameter, the peak time of rainfall will become more shifted to the right as the duration $T$ increase until $T$ reaches a limiting value, $T_L$. Above $T_L$, the increase of $T$ will not have a significant effect on the rainfall distribution. For different values of $a$-parameter, the $T_L$ is different. The smaller the $a$-parameter, the lower the $T_L$. For instance, in Fig. 6.4, when $a=1$, $T_L \approx 4$; when $a=4$, $T_L \approx 10$; and, when $a=8$, $T_L \approx 16$. This property of the rainfall distribution Equation can be explained by the following analysis.

Poisson Equation 6.1 has the property that for any $a$, when $N \sim \infty$, the summation of frequency equals to 1 (Young, 1965), which means that the Poisson Equation is converged. In
another word, the term $\frac{a^e e^{-a}}{n!}$ tends to disappear when the term $n$ gets to be very large. Then for a given $a$, there must be a $t=T_L$ satisfying the following

$$\frac{a^{(r-1)} e^{-a}}{(r-1)!} \leq \delta$$

(Equation 6.6)

Where $\delta$ is a small number, and when it is very small it can be ignored. Therefore, the items corresponding to $t>T_L$ can be deleted from the calculation of $C$. That means when $T=T_L$, then $C \sim 1$. Hence one would have the following relationship:

$$\frac{a^{(T_k-1)} e^{-a}}{C(T_k - 1)!} = \frac{a^{(T_k-1)} e^{-a}}{(T_k - 1)!} \leq \delta$$

(Equation 6.7)

$$(T_k - 1) \ln a - a - \ln [(T_k - 1)!] \leq \ln \delta$$

(Equation 6.8)

$$(T_k - 1) \ln a - \sum_{k=1}^{T_k-1} \ln k \leq \ln \delta + a$$

(Equation 6.9)

As an example, the $T_L$ values for some $a$ and $\delta$ are listed in Table 6.3.

<table>
<thead>
<tr>
<th>$\delta$</th>
<th>0.01</th>
<th>0.1</th>
<th>0.5</th>
<th>1</th>
<th>1.5</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
<td>3</td>
<td>4</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td>9</td>
<td>11</td>
<td>13</td>
<td>15</td>
<td>17</td>
<td>19</td>
</tr>
<tr>
<td>0.05</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>5</td>
<td>6</td>
<td>8</td>
<td>9</td>
<td>10</td>
<td>11</td>
<td>13</td>
</tr>
</tbody>
</table>

Similarly, for a given $T$, there is a limit of $a$, $a_L$, below which the effect of $a$-parameter is not significant. This means if $a$-parameter is lower than this limit, the rainfall cannot last for $T$. Then Equation 6.9 can be revised as follows:
\[(T-1)\ln a_L - a_L \leq \ln \delta + \sum_{k=1}^{T-1} \ln k \]  
(Equation 6.10)

The \(a_L\) values for some \(T\) and \(\delta\) were calculated using Equation 6.10 and listed in Table 6.4

<table>
<thead>
<tr>
<th>(\delta)</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>12</th>
<th>16</th>
<th>20</th>
<th>24</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001</td>
<td>0.001</td>
<td>0.046</td>
<td>0.194</td>
<td>0.76</td>
<td>1.58</td>
<td>2.56</td>
<td>6.65</td>
<td>6.08</td>
<td>8.73</td>
<td>11.53</td>
<td>15.93</td>
</tr>
<tr>
<td>0.05</td>
<td>0.05</td>
<td>0.38</td>
<td>0.91</td>
<td>2.24</td>
<td>6.78</td>
<td>5.44</td>
<td>7.18</td>
<td>10.83</td>
<td>14.62</td>
<td>18.51</td>
<td>24.50</td>
</tr>
</tbody>
</table>

This is summarized using regression models which are presented in Figures 6.6 and 6.7.

**Maximum \(T\)-Parameter to Have an Effect on Rainfall Distribution**

\[ y = -0.1184x^2 + 2.8503x + 3.8157 \]
\[ R^2 = 0.9932 \]

\[ y = -0.0601x^2 + 1.7228x + 2.9883 \]
\[ R^2 = 0.9948 \]

Figure 6.6 Limit of \(T\)-Parameter to Have an Effect on Rainfall Distribution at Given \(a\)-Parameter.
6.3 Effect of Rainfall Characteristics on Pavement Drainage

A rainfall can be described quantitatively through parameters indicating intensity, duration and distribution. Subsequently, the effect of rainfall on the performance of pavement drainage system can be quantified through these parameters.

To closely examine the effects of rainfall on pavement drainage, a series of rainfall data were analyzed using the finite element model developed in this research. Through the analysis results, the quantitative relationship between rainfall parameters and pavement infiltration was developed.

For the analyses conducted in this stage the 10-meter long model of section-3 in the Indiana DOT project was the only pavement model that was used. The pavement drainage ability was
represented by the infiltration rate, peak flow and time to drain 95% infiltration. The infiltration rate refers to the ratio between the total outflow and the total rainfall on the pavement. This parameter has been used by some researchers for the evaluation of pavement drainage system (Hassan et al., 2000).

6.3.1 Effect of Rainfall Quantity

The objective of this analysis is to find a mathematical relationship between the rainfall quantity and the infiltration into the pavement. Conventionally, rainfall is described by intensity and duration. The rainfall intensity that can be obtained from the literature, and it is often the average of rainfall amount over a unit time during the rainfall event. Therefore, the rainfall quantity in Equation 6.4 can be obtained by multiplying the average intensity by its duration. One can observe from the IDF curves for most areas within the US, 1-year 12-hour total rainfall is no more than 100mm (4 in.) in total. Thus the highest rainfall quantity was set as 100mm (4 in.) in the selection of treatment levels in this analysis. To better understand the general effect of rainfall quantity of rainfalls with various distribution and duration, a factorial statistical experiment was designed, called experiment #1. In this experiment, seven levels were assigned to the independent variable $Q$. They were 5.08mm (0.2 in.), 12.7mm (0.5 in.), 25.4mm (1 in.), 38.1mm (1.5 in.), 50.8mm (2 in.), 76.2mm (3 in.), and 101.6mm (4 in.). Three rainfalls with different durations and distributions were designed as blocks. The blocks are shown in Figure 6.5. Rain #1 was a rainfall with high concentration and short duration. Rain #2 had moderate concentration and duration. Rain #3 had low concentration and long duration. The seven treatments of rainfall quantity were repeated to the three rainfalls. Thus 21 treatments were obtained totally in this experiment. The finite element modeling outputs of experiment #1 is listed in Tables 6.5 and 6.6 and Figures 6.8 through 6.11.
Figure 6.8 Rainfall Designed for Statistical Experiment #1.

Figure 6.9.a Effect of Rainfall Quantity on Outflow at End of Outlet: under Rain #1 (\(a=0.03, T=2\)).
The outflow curves in Figures 6.9 showed that the rainfall quantity had little influence on the time-to-drain provided that the distribution and duration were not changed. When rainfall quantity increased, the outflow initially increased and then decreased, but the time to drain remained almost constant.
has a high concentration and low duration, its effect on the peak flow is not significant; while this effect increases with the increase of rain duration.

Table 6.5 Total Outflow (m³ (ft³)) of Pavement Drainage Model

<table>
<thead>
<tr>
<th>Rain Event</th>
<th>Rainfall Quantity (mm (inches))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.08 (0.2)</td>
</tr>
<tr>
<td>Rain #1</td>
<td>0.0556 (1.96)</td>
</tr>
<tr>
<td>Rain #2</td>
<td>0.149 (5.26)</td>
</tr>
<tr>
<td>Rain #3</td>
<td>0.198 (6.99)</td>
</tr>
</tbody>
</table>

Figure 6.10 Relationship between Total Infiltration and Rainfall Quantity.
The total infiltrations from the total outflow during the entire drainage period are shown in Table 6.5 and Figure 6.10. In general, the total pavement infiltration increases with the total rainfall amount until it reaches a maximum value. Rainfall with a quantity higher than this limit can not increase the pavement infiltration amount. The extra rainfall only can increase runoff on the pavement surface.

### Table 6.6 Infiltration Rate of Pavement Drainage Model

<table>
<thead>
<tr>
<th>Rain Event</th>
<th>Rainfall Quantity (mm (inches))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.08 (0.2)</td>
</tr>
<tr>
<td>Rain #1</td>
<td>0.257</td>
</tr>
<tr>
<td>Rain #2</td>
<td>0.689</td>
</tr>
<tr>
<td>Rain #3</td>
<td>0.915</td>
</tr>
</tbody>
</table>

### Figure 6.11 Relationship between Infiltration Rate and Rainfall Quantity

The Table 6.6 and Figure 6.11 present the infiltration rate, which equals to the total outflow divided by the total rainfall on the pavement section. The infiltration rate is influenced by the rainfall quantity as well as rainfall distribution and duration. Regression Equations of
infiltration rate vs. rainfall quantity are obtained using statistical analysis for the three rainfalls described earlier.

For Rain #1 with \( a = 0.03 \) and \( T = 2 \), the regression Equation is as follows

\[
i = 0.1509Q^{-0.8591}, \quad R^2 = 0.9954 \quad \text{(Equation 6.11)}
\]

where

- \( i \) = infiltration rate
- \( i = \text{total infiltration divided by pavement area and total rainfall quantity, and} \)
- \( Q = \text{total rainfall.} \)

For Rain #2 with \( a = 1.3 \) and \( T = 6 \), the regression Equation is as follows

\[
i = 0.3929Q^{-0.8713}, \quad R^2 = 0.9994 \quad \text{(Equation 6.12)}
\]

For Rain #3 with \( a = 4 \) and \( T = 12 \), the regression Equation is as follows

\[
i = 0.5836Q^{-0.7743}, \quad R^2 = 0.9954 \quad \text{(Equation 6.13)}
\]

The regression Equations showed that the infiltration rates have a power relationship with rainfall quantity and the parameters in the power Equations vary with the distribution and duration of rainfall. Thus, the relationship between \( Q \) and \( i \) can be represented as follows

\[
i = kQ^m \quad \text{(Equation 6.14)}
\]

where \( k \) and \( m \) are regression parameters that depend on rainfall distribution and duration as well as pavement surface permeability and cracking condition. If one were to defined \( I = \text{infiltration per unit area, which is a commonly used infiltration rate in pavement inflow methodologies, then one would get the following} \)
Table 6.7 Peak Flow (m$^3$/hr (ft$^3$/hr)) of Pavement Drainage Model

<table>
<thead>
<tr>
<th>Rain Event</th>
<th>Rainfall Quantity (mm (inches))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.08 (0.2)</td>
</tr>
<tr>
<td>Rain #1</td>
<td>0.0057 (2.01)</td>
</tr>
<tr>
<td>Rain #2</td>
<td>0.0140 (0.49)</td>
</tr>
<tr>
<td>Rain #3</td>
<td>0.0167 (0.59)</td>
</tr>
</tbody>
</table>

Effect of Rainfall Quantity on Peak Flow

Figure 6.12 Effect of Rainfall Quantity on Peak Flow.

Table 6.7 and Figure 6.12 illustrate the influence of rainfall quantity on the peak flow during the pavement drainage. The regression Equations presented on this figure indicates that the rainfall duration has a significant effect on the peak flow. One can conclude that for a given rainfall quantity, the longer the rainfall lasts, the higher the peak flow. The peak flow is also
influenced by the rainfall quantity, but this influence is very limited. In summary, the peak flow can be estimated by using the following Equation:

\[ q = k' \ln Q + n \]  

(Equation 6.16)

where

- \( q \) = peak flow at end of outlet pipe (m³/hr),
- \( k' \), \( n \) = regression parameter, and
- \( Q \) = rainfall quantity (mm).

As in Equation 6.13, the regression parameter \( k' \) depends on rainfall distribution and duration as well as pavement surface permeability and cracking condition.

**Table 6.8 Time-to-Drain (hours) of Pavement Drainage Model**

<table>
<thead>
<tr>
<th>Rain Event</th>
<th>Rainfall Quantity (mm (inches))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.08 (0.2)</td>
</tr>
<tr>
<td>Rain #1</td>
<td>23</td>
</tr>
<tr>
<td>Rain #2</td>
<td>24</td>
</tr>
<tr>
<td>Rain #3</td>
<td>25</td>
</tr>
</tbody>
</table>

Table 6.8 list the time to drain 95% of the pavement models. From this table on can see that rainfall duration and quantity almost have no effect on time to drain. The time to drain depends more on the pavement structure and the edge drain systems.

**6.3.2 Effect of Rainfall Duration and Distribution**

The pavement infiltration analyses presented in this report indicated that the rainfall distribution and duration have a significant effect on the infiltration of pavement. A further investigation of this effect is needed for better quantification of pavement infiltration. The factorial statistical experiment, Experiment #2, was designed to accomplish this. The generated rainfalls used in this analysis are listed in Table 6.8. Here, the rainfall quantity \( Q \) is constant,
and duration and distribution parameter are independent variables. The analyses results are presented in Figures 6.13 and 6.14 and Table 6.9.

Table 6.9 Rainfall Data and Infiltration of Experiment #2

<table>
<thead>
<tr>
<th></th>
<th>Q (mm (inches))</th>
<th>a</th>
<th>T</th>
<th>i</th>
<th>Peak Flow (m$^3$/hr (ft$^3$/hr))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rain #1</td>
<td>50.8 (2)</td>
<td>0.03</td>
<td>2</td>
<td>0.0972</td>
<td>0.0087 (0.307)</td>
</tr>
<tr>
<td>Rain #2</td>
<td>50.8 (2)</td>
<td>1.3</td>
<td>6</td>
<td>0.2418</td>
<td>0.0187 (0.660)</td>
</tr>
<tr>
<td>Rain #3</td>
<td>50.8 (2)</td>
<td>4</td>
<td>11</td>
<td>0.4345</td>
<td>0.0257 (0.907)</td>
</tr>
<tr>
<td>Rain #4</td>
<td>50.8 (2)</td>
<td>1</td>
<td>1</td>
<td>0.0579</td>
<td>0.0005 (0.017)</td>
</tr>
<tr>
<td>Rain #5</td>
<td>50.8 (2)</td>
<td>1</td>
<td>2</td>
<td>0.0976</td>
<td>0.0087 (0.307)</td>
</tr>
<tr>
<td>Rain #6</td>
<td>50.8 (2)</td>
<td>1</td>
<td>4</td>
<td>0.1788</td>
<td>0.0149 (0.526)</td>
</tr>
<tr>
<td>Rain #7</td>
<td>50.8 (2)</td>
<td>1</td>
<td>6</td>
<td>0.2263</td>
<td>0.0178 (0.628)</td>
</tr>
<tr>
<td>Rain #8</td>
<td>50.8 (2)</td>
<td>2</td>
<td>2</td>
<td>0.0976</td>
<td>0.0087 (0.307)</td>
</tr>
<tr>
<td>Rain #9</td>
<td>50.8 (2)</td>
<td>4</td>
<td>4</td>
<td>0.1788</td>
<td>0.0149 (0.526)</td>
</tr>
<tr>
<td>Rain #10</td>
<td>50.8 (2)</td>
<td>8</td>
<td>2</td>
<td>0.2599</td>
<td>0.0197 (0.696)</td>
</tr>
<tr>
<td>Rain #11</td>
<td>50.8 (2)</td>
<td>6</td>
<td>9</td>
<td>0.3815</td>
<td>0.0248 (0.876)</td>
</tr>
<tr>
<td>Rain #12</td>
<td>50.8 (2)</td>
<td>2</td>
<td>2</td>
<td>0.1451</td>
<td>0.0122 (0.431)</td>
</tr>
<tr>
<td>Rain #13</td>
<td>50.8 (2)</td>
<td>8</td>
<td>6</td>
<td>0.1972</td>
<td>0.0158 (0.558)</td>
</tr>
<tr>
<td>Rain #14</td>
<td>50.8 (2)</td>
<td>8</td>
<td>9</td>
<td>0.2897</td>
<td>0.0209 (0.738)</td>
</tr>
<tr>
<td>Rain #15</td>
<td>50.8 (2)</td>
<td>8</td>
<td>12</td>
<td>0.3976</td>
<td>0.0251 (0.886)</td>
</tr>
<tr>
<td>Rain #16</td>
<td>50.8 (2)</td>
<td>8</td>
<td>18</td>
<td>0.5538</td>
<td>0.0281 (0.992)</td>
</tr>
<tr>
<td>Rain #17</td>
<td>50.8 (2)</td>
<td>0.01</td>
<td>2</td>
<td>0.0829</td>
<td>0.0074 (0.261)</td>
</tr>
<tr>
<td>Rain #18</td>
<td>50.8 (2)</td>
<td>0.5</td>
<td>2</td>
<td>0.0971</td>
<td>0.0087 (0.307)</td>
</tr>
<tr>
<td>Rain #19</td>
<td>50.8 (2)</td>
<td>2</td>
<td>2</td>
<td>0.0971</td>
<td>0.0087 (0.307)</td>
</tr>
<tr>
<td>Rain #20</td>
<td>50.8 (2)</td>
<td>6</td>
<td>2</td>
<td>0.0971</td>
<td>0.0087 (0.307)</td>
</tr>
<tr>
<td>Rain #21</td>
<td>50.8 (2)</td>
<td>0.5</td>
<td>4</td>
<td>0.1755</td>
<td></td>
</tr>
<tr>
<td>Rain #22</td>
<td>50.8 (2)</td>
<td>2</td>
<td>6</td>
<td>0.2599</td>
<td>0.0197 (0.696)</td>
</tr>
<tr>
<td>Rain #23</td>
<td>50.8 (2)</td>
<td>6</td>
<td>6</td>
<td>0.2166</td>
<td>0.0170 (0.600)</td>
</tr>
<tr>
<td>Rain #24</td>
<td>50.8 (2)</td>
<td>2</td>
<td>8</td>
<td>0.3029</td>
<td></td>
</tr>
<tr>
<td>Rain #25</td>
<td>50.8 (2)</td>
<td>6</td>
<td>12</td>
<td>0.4451</td>
<td></td>
</tr>
</tbody>
</table>
Figure 6.13.a Effect of Rainfall Duration on Outflow at the End of Outlet: under Rain #4 to #7 ($Q = 50.8\text{mm (2 in.)}, \ a = 1$);

Figure 6.13.b Effect of Rainfall Duration on Outflow at the End of Outlet: under Rain #13 to #18 ($Q = 50.8\text{mm (2in.)}, \ a = 8$).
Figure 6.14.a Effect of Rainfall Distribution Parameter $a$ on Outflow at End of Outlet:
Rain with $Q = 50.8\text{mm (2 in.)}$ and $T = 2\text{ hours}$.

Figure 6.14.b Effect of Rainfall Distribution Parameter $a$ on Outflow at End of Outlet:
Rain with $Q = 50.8\text{mm (2 in.)}$ and $T = 2\text{ hours}$.
From Figure 6.13 one can see that rainfall duration has a significant effect on the pavement drainage. The time-to-drain and peak outflow are both related to rainfall duration. For given $Q$ and $a$, the time-to-drain, peak flow, as well as the total infiltration will increase with rain duration. On the other hand, the effect of $a$-parameter is not significant. The $a$-parameter only has a small effect on the total infiltration, and it hardly has any effect on the time-to-drain and peak flow. The relationships between rainfall duration and infiltration rate and peak flow are illustrated in Figure 6.15. The relationships between rainfall $a$-parameter and infiltration rate and peak flow are illustrated in Figure 6.16.

\[ y = -0.0002x^2 + 0.0163x + 0.0053 \]

\[ R^2 = 0.9744 \]

**Figure 6.15.a Relationship between Rainfall Duration and Infiltration Rate.**
Figure 6.15.b Relationship between Rainfall Duration and Peak Flow.

Figure 6.16.a Relationship between Rainfall $a$-Parameter and Infiltration Rate.
Figure 6.16.b Relationship between Rainfall $a$-Parameter and Peak Flow.

Figure 6.15 shows that both infiltration rate and peak flow have strong correlation with rainfall duration. Figure 6.16 show a weak correlation between $a$-parameter and infiltration rate. The regression model for the relationship between rainfall duration, $T$, and infiltration rate, $i$, is as follows

$$i = -0.0002T^2 + 0.016T + 0.0053, \quad R^2 = 0.9744 \quad \text{(Equation 6.17)}$$

The regression model for the relationship between peak flow and rainfall duration, $T$, is as follows

$$q = 0.00944 \ln T + 0.0016, \quad R^2 = 0.9702 \quad \text{(Equation 6.18)}$$

If the total rainfall, $Q$, is treated as a variable in Equations 6.17 and 6.18, the general regression equation for infiltration rate of uncracked pavement can be written as follows

$$i = k(T)Q^{m(T)} \quad \text{(Equation 6.19)}$$
and the general regression equation for peak flow of uncracked pavement can be written as follows

\[ q = k'(T) \ln Q + n \]  

(Equation 6.20)

where \( k, m, k' \) and \( n \) are function of \( T \). To determine the parameters of equations 6.19 and 6.20, another factorial statistical experiment is designed. The treatments levels for these variables are listed in Table 6.10, and the analysis results are shown in Figures 6.17 through 6.22.

### Table 6.10 Experiment #3 Design for Factorial Analysis on Infiltration

<table>
<thead>
<tr>
<th>Variable</th>
<th>Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T ) (hour)</td>
<td>1 ((a=1))</td>
</tr>
<tr>
<td>( Q ) mm (inches)</td>
<td>5.08 ((0.2))</td>
</tr>
</tbody>
</table>

### Rainfall Quantity \( Q \) vs. Infiltration Rate

![Rainfall Quantity vs. Infiltration Rate](image)

**Figure 6.17 Relationship Between Rainfall Quantity and Infiltration Rate For Various \( T \) Value.**
Figure 6.17 shows that for all rainfall durations the infiltration rate and rainfall quantity have a power relationship as explained by Equation 6.14. The regression parameters and Pearson’s R-squared for each $T$ are listed in Table 6.11.

Table 6.11 Statistical Regression Results for Infiltration Rate

<table>
<thead>
<tr>
<th>$T$</th>
<th>$k$</th>
<th>$m$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.1541</td>
<td>-0.9997</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>1.9482</td>
<td>-0.9999</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>2.3454</td>
<td>-0.8956</td>
<td>0.9966</td>
</tr>
<tr>
<td>6</td>
<td>3.2407</td>
<td>-0.8858</td>
<td>0.9981</td>
</tr>
<tr>
<td>9</td>
<td>3.714</td>
<td>-0.8186</td>
<td>0.9951</td>
</tr>
<tr>
<td>12</td>
<td>4.4775</td>
<td>-0.8559</td>
<td>0.9888</td>
</tr>
<tr>
<td>18</td>
<td>3.7254</td>
<td>-0.7393</td>
<td>0.9623</td>
</tr>
</tbody>
</table>

Figure 6.18 Relationship between $k$-Parameter and Rainfall Duration $T$. 

Rainfall Duration $T$ vs. Regression $k$-Parameter

$y = -0.0205x^2 + 0.5393x + 0.7102$

$R^2 = 0.9753$
Figure 6.19 Relationship between $m$-Parameter and Rainfall Duration $T$.

Figures 6.18 and 6.19 showed the relationship between regression parameters and rainfall duration $T$. $k$ and $m$ can be predicted using following regression equations.

\[
k(T) = -0.0205T^2 + 0.5393T + 0.7102, R^2 = 0.9753 \quad \text{(Equation 6.21)}
\]

\[
m(T) = -0.0005T^2 + 0.0239T - 1.0186, R^2 = 0.8955 \quad \text{(Equation 6.22)}
\]

Thus, Equation 6.14 becomes

\[
i = \left( -0.0205T^2 + 0.5393T + 0.7102 \right)Q^{(-0.0005T^2 + 0.0239T - 1.0186)} \quad \text{(Equation 6.23)}
\]

Then the rainfall infiltration per unit area can be calculated as
\[ I = \left( -0.0205T^2 + 0.5393T + 0.7102 \right) Q^{\left(-0.0005T^2 + 0.0239T - 1.0186 \right)} \]

(Equation 6.24)

\[ = \left( -0.0205T^2 + 0.5393T + 0.7102 \right) Q^{\left(-0.0005T^2 + 0.0239T - 0.0186 \right)} \]

Figure 6.20 Relationship Between Rainfall Quantity and Peak Flow For Various \(T\) Value.

Figure 6.20 shows that for all rainfall durations the peak flow and rainfall quantity have a log relationship as explained by Equation 6.16. The regression parameters and Pearson’s R-squared for each \(T\) are listed Table 6.12.

<table>
<thead>
<tr>
<th>(T)</th>
<th>(k')</th>
<th>(n)</th>
<th>(R^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0003</td>
<td>0.0038</td>
<td>0.6178</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>0.0087</td>
<td>0.5087</td>
</tr>
<tr>
<td>4</td>
<td>0.0015</td>
<td>0.009</td>
<td>0.8635</td>
</tr>
<tr>
<td>6</td>
<td>0.0018</td>
<td>0.0124</td>
<td>0.9476</td>
</tr>
<tr>
<td>9</td>
<td>0.003</td>
<td>0.0132</td>
<td>0.8963</td>
</tr>
<tr>
<td>12</td>
<td>0.0025</td>
<td>0.0156</td>
<td>0.7636</td>
</tr>
<tr>
<td>18</td>
<td>0.0048</td>
<td>0.0099</td>
<td>0.9141</td>
</tr>
</tbody>
</table>
Rainfall Duration $T$ vs. Regression $k'$-Parameter

$$y = 0.0015\ln(x) - 0.0004$$

$R^2 = 0.8415$

Figure 6.21 Relationship between $k'$-Parameter and Rainfall Duration $T$.

Rainfall Duration $T$ vs. Regression $n$-Parameter

$$y = -1\times10^{-04}x^2 + 0.0021x + 0.0029$$

$R^2 = 0.9089$

Figure 6.22 Relationship between $n$-Parameter and Rainfall Duration $T$.  

172
Figures 6.21 and 6.22 showed the relationship between regression parameters and rainfall duration $T$. $k'$ and $n$ can be predicted using following regression equations.

\[ k'(T) = 0.0015\ln T - 0.0004, \quad R^2 = 0.8415 \]  
\[ (Equation \ 6.25) \]

\[ n(T) = -0.0001T^2 + 0.0021T + 0.0029, \quad R^2 = 0.9089 \]  
\[ (Equation \ 6.26) \]

Thus, Equation 6.16 becomes

\[ q = (0.0015\ln T - 0.0004)\ln Q + (-0.0001T^2 + 0.0021T + 0.0029) \]  
\[ (Equation \ 6.27) \]

The regression Equations 6.24 and 6.27 are based upon the analysis of an uncracked pavement with a surface permeability of $10^{-4}$ cm/s (0.283 ft/day). Therefore the coefficients in this equation may be changed with the surface properties or layer components of pavement. To evaluate the effect of the influence of those factors, further analyses were conducted in the following sections.

### 6.4 Effect of Pavement Surface Permeability on Infiltration

Cedergren indicated that the amount of surface infiltration that can enter into a structural section is controlled by the lesser of: (a) the amount allowed by the effective permeability of the wearing course, or (b) the amount of supply (design precipitation rate) (Cedergren, 1974). To establish a relationship for the effect of permeability on infiltration given various rainfalls, a factorial experiment was designed and conducted. The rainfall #1 through #3 in Experiment #2 with quantities of 5.08mm, 25.4mm, and 76.2mm were entered into the model with surface permeability of $10^{-5}$, $10^{-4}$, $10^{-3}$, and $10^{-2}$ cm/s (0.0283, 0.283, 2.83, and 28.3 ft/day). The treatment levels of variables are listed in Table 6.13. The analysis results are illustrated in Figure 6.23.
Table 6.13 Experiment Design for Surface Permeability Effect on Infiltration

<table>
<thead>
<tr>
<th>Variable</th>
<th>Variable Levels</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall Q (mm (inches))</td>
<td>5.08 (0.2)</td>
</tr>
<tr>
<td>T (hour)</td>
<td>2</td>
</tr>
<tr>
<td>Surface Perm. (cm/s (ft/day))</td>
<td>10⁻³ (0.0283)</td>
</tr>
</tbody>
</table>

Model Infiltration Rate vs. Surface Permeability

Figure 6.23 Effect of Pavement Surface Permeability on Pavement Drainage.

From Figure 6.23 one can see that the infiltration rate of pavement increases with the surface permeability, and the increased infiltration varies with the rainfall quantity and duration. To estimate the infiltration rate of pavement with surface permeability other than 0.0001 cm/s (0.283 ft/day), Equation 6.23 needs to be multiplied by a coefficient. This coefficient should be a function of surface permeability, rainfall quantity and duration. Thus, Equation 6.23 becomes:

\[ i = C\left(-0.0205T^2 + 0.5393T + 0.7102\right)Q^{\left(-0.0005T^2+0.0239T-1.0186\right)} \]  
(Equation 6.28)
where C represents the coefficient related to surface permeability. The reference values of C for various rainfalls are listed in Table 6.14.

### Table 6.14 Reference Values of C

<table>
<thead>
<tr>
<th>Surface Permeability cm/s (ft/day)</th>
<th>Rain Events</th>
<th>Q=5.08mm (0.2 in.)</th>
<th>Q=25.4mm (1 in.)</th>
<th>Q=76.2mm (3 in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T=2</td>
<td>T=6</td>
<td>T=11</td>
<td>T=2</td>
</tr>
<tr>
<td>$10^{-5}$ (0.0283)</td>
<td>0.11</td>
<td>0.13</td>
<td>0.15</td>
<td>0.11</td>
</tr>
<tr>
<td>$10^{-4}$ (0.283)</td>
<td>0.73</td>
<td>0.92</td>
<td>0.83</td>
<td>1.01</td>
</tr>
<tr>
<td>$10^{-3}$ (2.83)</td>
<td>2.85</td>
<td>1.33</td>
<td>0.91</td>
<td>8.64</td>
</tr>
<tr>
<td>$10^{-2}$ (28.3)</td>
<td>2.85</td>
<td>1.33</td>
<td>0.91</td>
<td>13.63</td>
</tr>
</tbody>
</table>

The values in Table 6.14 were calculated from dividing the model infiltration rate of models with various surface permeability values by the infiltration rates calculated by Equation 6.23. Equation 6.23 is used to calculate the infiltration rate of pavement with surface permeability of 0.0001cm/s (0.283 ft/day). The correlation between the model analysis results and the predicted value of the regression equation 6.23 is shown in Figure 6.24.

### Figure 6.24 Correlation between Equation 6.23 and 3-D Model Results.
Figure 6.24 indicates that the regression Equation 6.23 does a good job in predicting the pavement infiltration rate from rainfall parameters. Multiplying this equation by a proper C factor, one can get the predicted infiltration rate for pavement with various values of surface permeability.

6.5 Effect of Pavement Structure on Infiltration

Pavement layers below the surface layer do contribute to pavement drainage. To investigate this phenomenon, rainfall infiltration results of section-1 in InDOT project, model MP3 and model MP5 in Chapter 5.0 was compared with the results of section-3. To conduct this comparison, the rainfalls #1 through #3 in Experiment #2 with quantities of 5.08mm (0.2 inches), 25.4mm (1 inches), and 76.2mm (3 inches), respectively, were entered into the finite element models of both pavement sections. The outflow results were plotted in Figures 6.25 and their total infiltrations are shown in Figure 6.26.

![Comparison between Outflow of Section-1 and Section-3](image)

Figure 6.25.a Effect of Pavement Structure and Rainfall on Pavement Drainage: Comparison between Section-1 and Section-3.
Figure 6.25.b Effect of Pavement Structure and Rainfall on Pavement Drainage: Comparison between MP3 and Section-3.

Figure 6.25.c Effect of Pavement Structure and Rainfall on Pavement Drainage: Comparison between Section-3 and MP5.
Comparison of Total Infiltration between Two Pavement Structures

- **Section-1**
- **Section-3**

**Figure 6.26 Effect of Pavement Structure on Total Infiltration**

Correlation between Infiltration Rates of Pavement Structures

**Figure 6.27 Correlation between Infiltration Rate of Pavement Structures**
From Figures 6.25 through 6.27 one can see that the outflows of section-1, MP3 and MP5 have high concentration and short drainage time than section-3. Under the same rainfall condition, the total outflows of different structures are different. However, the differences of total outflow between these sections are linearly correlated. Figures 6.26 and 6.27 show the relationship between infiltration rates of these sections. Figure 6.27 indicates that the infiltration rates of the two sections are highly correlated, and there is a constant ratio between the infiltration rates of the structures. This constant ratio is independent of the rainfalls, which means that the infiltration rate of other type of pavement can be predicted by using the following linear equation:

\[ i = aC \left( -0.02057^2 + 0.5393T + 0.7102 \right)Q \left( -0.00057^2 + 0.0239T - 1.0186 \right) + b \]  
(Equation 6.29)

where \( a \)-parameter and \( b \)-parameter are constants that reflect the effect of pavement structure on infiltration rate. The values of \( a \)-parameter and \( b \)-parameter for pavement Section-1, Section-3, MP3 and MP-5 are listed in Table 6.15.

<table>
<thead>
<tr>
<th></th>
<th>Section-3</th>
<th>Section-1</th>
<th>MP3</th>
<th>MP-5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a )-Parameter</td>
<td>1</td>
<td>1.3016</td>
<td>-0.2312</td>
<td>2.2874</td>
</tr>
<tr>
<td>( b )-Parameter</td>
<td>0</td>
<td>-0.0306</td>
<td>0.4530</td>
<td>0.1876</td>
</tr>
</tbody>
</table>

### 6.6 Effect of Pavement Cracking and Surface Deterioration on Infiltration

Cracking is often considered to be a major pathway for surface water infiltration. The ICM model accounts for the infiltration through cracks in the pavement system, while ignoring flow through the less-porous intact pavement (Lytton, 1990). Ridgeway (1976) indicated that the amount of infiltration can be related directly to cracking and suggested that an infiltration \( I_c \) of 0.22m³/day/m (2.4 ft³/day/ft) of crack be used for design. The infiltration per unit area \( q_c \) can expressed as
\[ q_i = I_c \left( \frac{N_c}{W_p} + \frac{W_c}{W_p C_s} \right) + k_p \]  

(Equation 6.30)

where

- \( I_c \) = crack infiltration rate,
- \( N_c \) = number of cracks per unit area,
- \( W_p \) = width of pavement subject to infiltration,
- \( W_c \) = length of transverse cracks or joints, and
- \( k_p \) = the rate of infiltration (m\(^3\)/m\(^2\)/hr) through uncracked pavement surface.

This equation was developed based upon field measurements in Connecticut. However, this method fails to consider the effect of rainfall in the calculation of the pavement infiltration. Thus, the results obtained by using this equation are much lower than that what was obtained by Cedergren’s method in areas with heavy precipitation (Huang, 1993). To better understand the relationship between cracking and rainfall infiltration, the following analyses were designed and conducted.

Since the occurrence of cracks on pavement surface is complex and unpredictable, the models in Figure 6.28 were designed for the modeling of cracking. In Figure 6.28.a, two longitudinal and two transverse linear cracks are shown. The variations of crack width were listed in Table 6.16. In Figure 6.28.b, a square punch out was simulated in the model to better understand infiltration caused by surface damage. The analysis results are shown in Figures 6.29 and 6.30.
Figure 6.28.a Plan View of Pavement: Position of Cracks.

Figure 6.28.b Plan View of Pavement Deteriorations: Position of Punch Out.

Table 6.16 Treatment Levels for Cracking Effects on Infiltration

<table>
<thead>
<tr>
<th>Crack Width cm (inches)</th>
<th>0.625 (0.25)</th>
<th>1.25 (0.5)</th>
<th>2.5 (1)</th>
<th>5 (2)</th>
<th>10 (4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area Ratio of Cracking</td>
<td>0.0042</td>
<td>0.0084</td>
<td>0.0167</td>
<td>0.0335</td>
<td>0.067</td>
</tr>
</tbody>
</table>
The analysis results indicated that the total infiltration through the rainfall duration has a positive relationship with the cracking area and this relationship is almost linear. However, the slope of infiltration vs. cracking curve for different rainfalls changes, which means that the
The term $I_c$ in Equation 6.30 (Ridgway, 1976) is not constant with rainfall intensity and durations. A rainfall with a long duration and high quantity tends to have a higher infiltration per unit area of pavement. Therefore, one can conclude that pavement cracking effect on pavement infiltration is more pronounced under a long-term heavy rainfall. In addition, the results summarized in Figure 6.30 indicate that punch out tends to infiltrate more surface water than linear cracking. This is an important observation, and it must be considered in pavement maintenance strategies.

Through the regression of the analysis results in Figure 6.29, the following equation is developed for the estimation of the infiltration of cracked pavement.

$$i_{crack} = i + \frac{0.1271 \cdot 0.0132}{76.2 - 5.08} \cdot p = i + 0.0016p \quad \text{(Equation 6.31)}$$

where

$i =$ infiltration rate of uncracked pavement, which can be calculated from Equation 6.29, and

$p =$ cracking ratio by area.

### 6.7 Summary of Analysis Results and Recommendations

In this chapter, the effects of rainfall condition on pavement drainage were investigated. The analysis results indicated that both rainfall quantity and duration significantly affect the inflow and peak flow of water through pavement. Compare with rainfall quantity, rainfall duration plays a more important role on pavement drainage. Pavement infiltration is controlled by the pavement surface condition. The increase of surface permeability and cracking will significantly increase the inflow of water into a pavement.

Through a series of statistical experiments, a mathematical model for estimation of pavement infiltration was developed. This model can be used for predicting inflow under various rainfall conditions, surface conditions, and it is applicable for various pavement conditions.
structures. A mathematical model for the estimation of peak flow was also developed using the statistical analyses.
CHAPTER 7.0 CONCLUSIONS AND RECOMMENDATIONS

Water is the leading factor that induces deterioration in pavement structures. To reduce the detrimental effects of entrapped water, pavement subsurface drainage systems (PSDS) are designed to drain the entrapped water out of pavement. The current practice of pavement drainage is often consisted of a permeable base and an edgedrain system. Currently, the adequacy of this type of drainage system is evaluated using a method developed by FHWA, called time-to-drain method. This design method was developed based on the saturated seepage analysis for a 2-D pavement cross-section model. This method can not fully account for the performance of pavement drainage system because the water flow analysis in the pavement is a three dimensional problem.

To find a more suitable analysis method for the pavement subsurface drainage system, the 3-D finite element modeling and statistical models were applied to this research. The 3-D models were developed using the finite element method to simulate the unsaturated drainage process in pavement structures. To simulate the unsaturated flow in pavement, the seepage analysis method for the unsaturated soil was applied to the formulation of the governing equation. The finite element models were verified by using the field data, and the verification results indicated that the model developed in this research can successfully simulate the drainage performance of a pavement. The total outflow, peak flow, layer saturations, and time to drain were selected to evaluate the performance of the PSDS. To investigate the effects of various factors on the performance of PSDS, a parametric analysis was conducted using the newly developed 3-D finite element model. Three categories of influence factors were evaluated in this research. They were pavement geometry, pavement layer materials, and rainfall events. Through a series of parametric analyses, the factors that have significant effect on PSDS were screened out, and recommendations for the improvement of PSDS were provided.

The effects of pavement geometry were studied in this research. The pavement geometry evaluated in this research included: transverse and longitudinal slopes of driving lanes, slope of shoulder and outlet pipe, width and length of pavement, thickness of pavement layers, and diameters and slopes of collection pipes and outlet pipes. In addition, various combinations of
lane slopes and edgdrain positions were evaluated. The analysis results indicate that the edgdrain system can significantly improve the drainage efficiency of a pavement. The drainage performance of a pavement is mainly affected by the geometry that is directly related to the edgdrain system, such as pipe slopes and pipe diameters. The geometry of driving lanes only has limited influence. The transverse slope of driving lanes, which traditionally has been considered to be an important parameter in pavement drainage design, did not show any significant contribution to pavement subsurface drainage. The longitudinal slope of driving lanes showed a significant influence on pavement drainage when the collection pipe was placed at a certain distance below the pavement surface. The thickness of driving lanes will not affect the pavement drainage, provided that the lift of thickness of a layer has no effect on the permeability of that layer. The total drainage outflow is approximately proportional to the width or length of the pavement to be drained. The drainage characteristics of a pavement with two or more lanes are highly related to the combinations of pavement slopes and edgdrain location and characteristics. The assumption that the pavement will be drained by the nearest edgdrain is not a correct one for pavements with an asymmetric cross-section. For these pavements, only the edgdrain on lower side is functional. For pavements with symmetric cross-sections, the drainage potential would be improved if edge drains are constructed at both sides.

Pavement material hydraulic properties play an important role in the performance of PSDS. To conduct an unsaturated water flow analyses, the material hydraulic conductivity must be provided. The hydraulic conductivity of a material varies with the matric suction in the material domain. Since the direct measurement of unsaturated hydraulic conductivity can be time consuming, it is usually estimated from the saturated permeability and the soil water characteristic curve (SWCC). Base upon a physical relationship derived from Poiseuille’s law (Mitchell, 1976) and the permeability data available in literature, a new method was developed in this research for the estimation of the saturated permeability for all types of pavement materials. To estimate the SWCC of HMA, a new method was developed base upon the Cote and Konrad’s method (Cote et al., 2003).
From the test data and the estimated data, it can be demonstrated that the hydraulic conductivity of pavement materials usually has a significant effect on pavement drainage. Field observations have indicated that the matric suction in a pavement usually ranges from 0 to 100 kPa. To increase the drainage capability, pavements must be designed with materials that have high conductivity within the suction range present in pavement layers. However, the materials with high conductivity at a high suction usually have high water retention. The high water retention will increase the degree of saturation within the materials, which can lead to pavement performance problems.

To compare the effect of material properties on the performance of PSDS, a series of models were designed and analyzed using the finite element method. The analysis results of these models indicate that the use of a permeable base is effective in improving the drainage ability of a pavement. A permeable base can significantly increase the drainage efficiency and reduce the degree of saturation of the base and the surface layers. The performance of PSDS is affected by material permeability as well as by the water retention properties (SWCC). The degree of saturation of each layer is mainly affected by the SWCC of that layer. The SWCC of subgrade soil has more influence on drainage than other layers. During the drainage process, the pavement works as an integrated hydraulic system and the infiltrated water usually flows through the layer that is easier to be saturated. Even though the permeable base can increase the drainage efficiency of a pavement, it does not work as a conduit for carrying water to the edgedrain, but it works as a conduit that carries water to the lower layers. The water that is carried to the subbase or subgrade makes the subbase or subgrade saturated, and thus increases the hydraulic conductivity of these layers.

The effects of surface infiltration on pavement drainage were also studied in this research. In the current PSDS design, the 1-hour, 1-year rainfall of the design geographic area is used as the critical input. Field observations have shown that the rainfall quantity as well as the rainfall duration significantly affects the surface infiltration. Therefore, in this research the rainfall events were used as input to check the drainage capability of pavements. A method was developed based upon Poisson’s statistical distribution equation to numerically describe the rainfall events. The developed equation can be used for fitting the recorded rainfall data or it
can be used to generate a rainfall event. A sensitivity analysis was conducted to examine the 
effectiveness of the proposed regression models.

To show the effects of rainfall events on the pavement drainage characteristics, a series of 
experiments were designed and analyzed. The analysis results under various rainfall events 
indicate that rainfall duration is a more important parameter than the rainfall quantity in 
influencing the pavement drainage. The total outflow or infiltration will increase with the 
rainfall quantity until it reaches an upper limit, which usually equals to the effective permeability 
of the pavement surface. The time to drain for a pavement usually depends on the duration of 
the rainfall and the diameters of the collection and outlet pipes. For a given rainfall quantity, 
the total infiltration, peak flow, and time-to-drain will increase with the rainfall duration. Based 
on the analysis results, regression equations were developed for the estimation of pavement 
drainage characteristics. The increase of the permeability of pavement surface will 
significantly increase the total infiltration of the pavement but not increase the drainage time of 
the PSDS. For a positive drainage, the size of the collection pipe and outlet pipe should be 
increased for the pavement with high surface permeability. These factors should be included in 
the design considerations for the pavement subsurface drainage system.

Based upon the findings of this research, the following recommendations are made:

First, an edgedrain system should be included in the pavement. The slopes of collection 
pipe and outlet pipe should be designed as steep as possible. The diameter of the collection 
pipe and the outlet pipe should be designed according to the following: representative rainfall 
conditions, the lanes to be drained, the distance between the outlets, as well as the permeability 
and surface cracking condition of the pavement. When 75-meter (246 ft) outlet spacing is 
designed, the pipe diameters should not be less than 150mm (6 inches) if a 75mm (3 inches), 
6-hour rainfall can occur in that area. The materials used for the trench backfill should be 
selected according to the gradations of the adjacent materials, that is, proper filter criteria must 
be applied. For a pavement with symmetric cross-section, edgedrains should be constructed on 
both sides. For a pavement with asymmetric cross-section, only one edgedrain should be 
constructed at the lower side.
Second, a permeable base with a permeability higher than 0.1 cm/s (283 ft/day) should be included in the pavement. Considering the pavement stability, a permeability ranging from 0.1 to 1 cm/s (283 to 2830 ft/day) is a recommended range for the base layer. A base layer permeability higher than 1 cm/s (2830 ft/day) can increase the drainage capability only slightly.

Third, for the drainage system design, the rainfall event with long duration and high concentration should be selected as the critical design condition. The equations and recommended parameters provided in this research should be used to estimate the total rainfall and peak flow of the drainage system. Some recommendations for PSDS design options are presented in Appendix B.

Finally, from this research, one can see that the pavement structure works as an integrated hydraulic system. The performance of this system is related to all of the components within the pavement system. Since the finite element modeling is complex, not every pavement designer can use this method to check the drainage capability of the pavement design. Thus, a user-friendly product is needed for this purpose. Another advantage of this proposed new software would be that it can serve as a verification tool for commercial software results. The next step of this research is to develop a simplified model based on the analysis method presented in this research. To improve the estimation of the material properties used in the finite element model, more laboratory tests should be conducted for the soil water characteristic curves. Furthermore, in this research, laminate flow was assumed for both the flow under the surface of pavement and the flow in the pipes. To advance the simulation of the water flow in pavement, an improved model should consider turbulent flow in the collection and outlet pipes.
APPENDIX A: FLEXPDE SCRIPT OF PAVEMENT MODEL

TITLE"
{    Project: Material Property Effect
     Problem: MP1
     Description: 3D, Transient, Imperial    }

COORDINATES
  Cartesian3

SELECT
  FIXDT
  ITERATE=10000
  NBCMEASURE
  NRMINSTEPS=0.1
  TERRLIM=0.01
  GRIDLIMIT=20
  NODELIMIT=10000

VARIABLES
  h

DEFINITIONS
{Time - Units: hr}
  StartTime = 0
  TimeInc = 1
  EndTime = 48

{General definitions}
  Block
  uww=9.807 \{kN/m^3\}, pw=1000 \{kg/m^3\}
  g=9.81 \{m/s^2\}
  Gs,e,pd,pt,uwt,Sat,gwc,n,vac \{Volume-Mass\}
  s=0
  m2w
  Xoffset=x+(0)
  Yoffset=y+(0)
  u=uvw*(h-z) \{kPa\}
  kx
  ky
  kz
  Gradx= -dx(h)*kx
  Grady= -dy(h)*ky
  Gradz= -dz(h)*kz

{Hydraulic Conductivity Ratios}
  S_1_ky_ratio=1
  S_1_kz_ratio=1
  S_2_ky_ratio=1
  S_2_kz_ratio=1
  S_3_ky_ratio=1
  S_3_kz_ratio=1
  S_4_ky_ratio=1
  S_4_kz_ratio=1
  S_5_ky_ratio=1
  S_5_kz_ratio=1
  S_6_ky_ratio=1

190
{Hydraulic Conductivity}

{Unsaturated Hydraulic Conductivity - Leong and Rahardjo}  !soil
ksat1 = 2.3e-6, p1 = 3, kafx1 = 618.7, knfx1 = 0.202, kmfx1 = 0.955
kmin1 = 1.181102E-08
ksuc1 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
S_1_kx = if stage = 1 then ksat1 else max(kmin1,ksat1*((1/((ln(exp(1)+(ksuc1/kafx1)^knfx1))^kmfx1))^p1))

{Unsaturated Hydraulic Conductivity - Leong and Rahardjo}  !Filter
ksat2 = 0.0155, p2 = 3, kafx2 = 0.712, knfx2 = 1.338, kmfx2 = 0.935
kmin2 = 1.181102E-08
ksuc2 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
S_2_kx = if stage = 1 then ksat2 else max(kmin2,ksat2*((1/((ln(exp(1)+(ksuc2/kafx2)^knfx2))^kmfx2))^p2))

{Unsaturated Hydraulic Conductivity - Leong and Rahardjo}  !Base 1
ksat3 =0.0155, p3 = 3, kafx3 = 0.712, knfx3 = 1.338, kmfx3 = 0.935
kmin3 = 1.181102E-08
ksuc3 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
S_3_kx = if stage = 1 then ksat3 else max(kmin3,ksat3*((1/((ln(exp(1)+(ksuc3/kafx3)^knfx3))^kmfx3))^p3))

{Unsaturated Hydraulic Conductivity - Leong and Rahardjo}  !Base 2
ksat4 = 0.0155, p4 = 3, kafx4 = 0.712, knfx4 = 1.338, kmfx4 = 0.935
kmin4 = 1.181102E-08
ksuc4 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
S_4_kx = if stage = 1 then ksat4 else max(kmin4,ksat4*((1/((ln(exp(1)+(ksuc4/kafx4)^knfx4))^kmfx4))^p4))

{Unsaturated Hydraulic Conductivity - Leong and Rahardjo}  !Surface
ksat5 = 0.003636, p5 = 3, kafx5 = 4.29, knfx5 = 7, kmfx5 = 0.93
kmin5 = 1.181102E-08
ksuc5 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
S_5_kx = if stage = 1 then ksat5 else max(kmin5,ksat5*((1/((ln(exp(1)+(ksuc5/kafx5)^knfx5))^kmfx5))^p5))

{Unsaturated Hydraulic Conductivity - Leong and Rahardjo}  !Trench
ksat6 = 139, p6 = 3, kafx6 = 0.07, knfx6 = 1.0413, kmfx6 = 2.038
kmin6 = 1.181102E-08
ksuc6 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
S_6_kx = if stage = 1 then ksat6 else max(kmin6,ksat6*((1/((ln(exp(1)+(ksuc6/kafx6)^knfx6))^kmfx6))^p6))

{Unsaturated Hydraulic Conductivity - Leong and Rahardjo}  !Pipe
S_7_kx = 500

{SWCC and Storage Functions}

vwc, vwc_min=0.0001

{SWCC - Fredlund and Xing}  !Soil
svwc1=0.42
afx1 = kafx1, nfx1 = knfx1, mfx1 = kmfx1, hrfx1 = 1000000
suc1 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
Sr1 = (1-ln(1+suc1/hrfx1)/(1+1E+06/hrfx1))*(1/ln(exp(1)+(suc1/afx1)^nfx1)^mfx1)
dS1=(1/hrfx1*(1+(suc1/hrfx1)))/(1+1E+06/hrfx1)*ln(exp(1)+(suc1/afx1)^nfx1)^mfx1)-((suc1/hrfx1)*(ln(1+1E+06/hrfx1)))*(mfx1*nfx1^2+((suc1)^((nfx1-1))*(suc1/hrfx1)))/(exp(1)+(suc1/hrfx1)^nfx1)^mfx1)
S_1_swcc = svwc1*Sr1
S_1_m2w = -((svwc1*dS1)

{SWCC - Fredlund and Xing}  !Filter
svwc2=0.186
afx2 = kafx2, nfx2 = knfx2, mfx2 = kmfx2, hrfx2 = 1000000
suc2 = if u>=-0.02 then 0.02 else (if u>=-999999 then -u else 999999)
Sr2 = (1-ln(1+suc2/hrfx2)/(1+1E+06/hrfx2))*(1/ln(exp(1)+(suc2/afx2)^nfx2)^mfx2))
\[ dS_2 = \frac{1}{(hrfx^2 + (suc2/hrfx^2)ln(1 + 1E+06/hrfx^2) * ln((exp(1) + (suc2/hrfx^2)^mfx2)) - (1 - ln(1 + (suc2/hrfx^2)ln(1 + 1E+06/hrfx^2) * ln((exp(1) + (suc2/hrfx^2)^mfx2)))) * ((mfx2/nfx2)^((suc2)^2/nfx2)) * ((afx2)^nfx2))} \\
S_{2\_swcc} = svwc2 * Sr2 \\
S_{2\_m2w} = -svwc2 * dS2 \\
\{SWCC - Fredlund and Xing\}  !Base 1 \\
svwc3 = 0.186 \\
afx3 = kafx3, nfx3 = kmfx3, hrfx3 = 1000000 \\
suc3 = if u >= -0.02 then 0.02 else (if u >= -999999 then -u else 999999) \\
Sr3 = (1 - ln(1 + suc3/hrfx3) / ln(1 + 1E+06/hrfx3)) * (1 / (ln(exp(1) + (suc3/hrfx3)^nfx3) + mfx3)) \\
dS3 = \frac{1}{(hrfx^3 + (suc3/hrfx^3) * ln(1 + 1E+06/hrfx^3) * ln((exp(1) + (suc3/hrfx^3)^mfx3)))) - (1 - ln(1 + (suc3/hrfx^3)ln(1 + 1E+06/hrfx^3) * ln((exp(1) + (suc3/hrfx^3)^mfx3)))) * ((mfx3/nfx3)^((suc3)^2/nfx3)) * ((afx3)^nfx3))} \\
S_{3\_swcc} = svwc3 * Sr3 \\
S_{3\_m2w} = -svwc3 * dS3 \\
\{SWCC - Fredlund and Xing\}  !Base 2 \\
svwc4 = 0.186 \\
afx4 = kafx4, nfx4 = kmfx4, hrfx4 = 1000000 \\
suc4 = if u >= -0.02 then 0.02 else (if u >= -999999 then -u else 999999) \\
Sr4 = (1 - ln(1 + suc4/hrfx4) / ln(1 + 1E+06/hrfx4)) * (1 / (ln(exp(1) + (suc4/hrfx4)^nfx4) + mfx4)) \\
dS4 = \frac{1}{(hrfx^4 + (suc4/hrfx^4) * ln(1 + 1E+06/hrfx^4) * ln((exp(1) + (suc4/hrfx^4)^mfx4)))) - (1 - ln(1 + (suc4/hrfx^4)ln(1 + 1E+06/hrfx^4) * ln((exp(1) + (suc4/hrfx^4)^mfx4)))) * ((mfx4/nfx4)^((suc4)^2/nfx4)) * ((afx4)^nfx4))} \\
S_{4\_swcc} = svwc4 * Sr4 \\
S_{4\_m2w} = -svwc4 * dS4 \\
\{SWCC - Fredlund and Xing\}  !Surface \\
svwc5 = 0.016 \\
afx5 = kafx5, nfx5 = kmfx5, hrfx5 = 7.6 \\
suc5 = if u >= -0.02 then 0.02 else (if u >= -999999 then -u else 999999) \\
Sr5 = (1 - ln(1 + suc5/hrfx5) / ln(1 + 1E+06/hrfx5)) * (1 / (ln(exp(1) + (suc5/hrfx5)^nfx5) + mfx5)) \\
dS5 = \frac{1}{(hrfx^5 + (suc5/hrfx^5) * ln(1 + 1E+06/hrfx^5) * ln((exp(1) + (suc5/hrfx5)^mfx5)))) - (1 - ln(1 + (suc5/hrfx^5)ln(1 + 1E+06/hrfx^5) * ln((exp(1) + (suc5/hrfx^5)^mfx5)))) * ((mfx5/nfx5)^((suc5)^2/nfx5)) * ((afx5)^nfx5))} \\
S_{5\_swcc} = svwc5 * Sr5 \\
S_{5\_m2w} = -svwc5 * dS5 \\
\{SWCC - Fredlund and Xing\}  !Trench \\
svwc6 = 0.566 \\
afx6 = kafx6, nfx6 = kmfx6, hrfx6 = 1000000 \\
suc6 = if u >= -0.02 then 0.02 else (if u >= -999999 then -u else 999999) \\
Sr6 = (1 - ln(1 + suc6/hrfx6) / ln(1 + 1E+06/hrfx6)) * (1 / (ln(exp(1) + (suc6/hrfx6)^nfx6) + mfx6)) \\
dS6 = \frac{1}{(hrfx^6 + (suc6/hrfx^6) * ln(1 + 1E+06/hrfx6) * ln((exp(1) + (suc6/hrfx6)^mfx6)))) - (1 - ln(1 + (suc6/hrfx^6)ln(1 + 1E+06/hrfx6) * ln((exp(1) + (suc6/hrfx^6)^mfx6)))) * ((mfx6/nfx6)^((suc6)^2/nfx6)) * ((afx6)^nfx6))} \\
S_{6\_swcc} = svwc6 * Sr6 \\
S_{6\_m2w} = -svwc6 * dS6 \\
\{SWCC - Fredlund and Xing\}  !Pipe \\
svwc7 = 0.9 \\
afx7 = 0.05, nfx7 = 2, hrfx7 = 1000000 \\
suc7 = if u >= -0.02 then 0.02 else (if u >= -999999 then -u else 999999) \\
Sr7 = (1 - ln(1 + suc7/hrfx7) / ln(1 + 1E+06/hrfx7)) * (1 / (ln(exp(1) + (suc7/hrfx7)^nfx7) + mfx7)) \\
dS7 = \frac{1}{(hrfx^7 + (suc7/hrfx^7) * ln(1 + 1E+06/hrfx7) * ln((exp(1) + (suc7/hrfx7)^mfx7)))) - (1 - ln(1 + (suc7/hrfx^7)ln(1 + 1E+06/hrfx7) * ln((exp(1) + (suc7/hrfx^7)^mfx7)))) * ((mfx7/nfx7)^((suc7)^2/nfx7)) * ((afx7)^nfx7))} \\
S_{7\_swcc} = svwc7 * Sr7 \\
S_{7\_m2w} = -svwc7 * dS7

Volume-Mass

! Soil
S_1_Gs=2.75
S_1_pt=1.6
S_1_n=svwc1
S_1_e=S_1_n/(1-S_1_n)
S_1_pd=S_1_Gs/(1+S_1_e)*pw
S_1_Sat=S_1_swcc*(1+S_1_e)/S_1_e
S_1_uwt=S_1_pt*uww
S_1_gwc=S_1_Sat*S_1_e/S_1_Gs
S_1_vac=S_1_n-S_1_swcc

! Filter
S_2_Gs=2.671
S_2_pt=2.1
S_2_n=svwc2
S_2_e=S_2_n/(1-S_2_n)
S_2_pd=S_2_Gs/(1+S_2_e)*pw
S_2_Sat=S_2_swcc*(1+S_2_e)/S_2_e
S_2_uwt=S_2_pt*uww
S_2_gwc=S_2_Sat*S_2_e/S_2_Gs
S_2_vac=S_2_n-S_2_swcc

! Base 1
S_3_Gs=2.671
S_3_pt=2.10
S_3_n=svwc3
S_3_e=S_3_n/(1-S_3_n)
S_3_pd=S_3_Gs/(1+S_3_e)*pw
S_3_Sat=S_3_swcc*(1+S_3_e)/S_3_e
S_3_uwt=S_3_pt*uww
S_3_gwc=S_3_Sat*S_3_e/S_3_Gs
S_3_vac=S_3_n-S_3_swcc

! Base 2
S_4_Gs=2.671
S_4_pt=2.10
S_4_n=svwc4
S_4_e=S_4_n/(1-S_4_n)
S_4_pd=S_4_Gs/(1+S_4_e)*pw
S_4_Sat=S_4_swcc*(1+S_4_e)/S_4_e
S_4_uwt=S_4_pt*uww
S_4_gwc=S_4_Sat*S_4_e/S_4_Gs
S_4_vac=S_4_n-S_4_swcc

! Surface
S_5_Gs=2.539
S_5_pt=1.97
S_5_n=svwc5
S_5_e=S_5_n/(1-S_5_n)
S_5_pd=S_5_Gs/(1+S_5_e)*pw
S_5_Sat=S_5_swcc*(1+S_5_e)/S_5_e
S_5_uwt=S_5_pt*uww
S_5_gwc=S_5_Sat*S_5_e/S_5_Gs
S_5_vac=S_5_n-S_5_swcc

! Trench
S_6_Gs=2.671
S_6_pt=1.26
S_6_n=svwc6
S_6_e=S_6_n/(1-S_6_n)
\[ S_{6\_pd} = \frac{S_{6\_Gs}}{1+S_{6\_e}} \times pw \]
\[ S_{6\_Sat} = \frac{S_{6\_swcc}}{1+S_{6\_e}} \times S_{6\_G} \]
\[ S_{6\_uwt} = S_{6\_pt} \times uww \]
\[ S_{6\_gwc} = \frac{S_{6\_Sat} \times S_{6\_e}}{S_{6\_Gs}} \]
\[ S_{6\_vac} = S_{6\_n} - S_{6\_swcc} \]

\!
\text{{Pipe}}

\[ S_{7\_Gs} = 1 \]
\[ S_{7\_pt} = 0.01 \]
\[ S_{7\_n} = svwc7 \]
\[ S_{7\_e} = S_{7\_n} / (1 - S_{7\_n}) \]
\[ S_{7\_pd} = \frac{S_{7\_Gs}}{1+S_{7\_e}} \times pw \]
\[ S_{7\_Sat} = \frac{S_{7\_swcc}}{1+S_{7\_e}} \times S_{7\_G} \]
\[ S_{7\_uwt} = S_{7\_pt} \times uww \]
\[ S_{7\_gwc} = S_{7\_Sat} \times S_{7\_e} / S_{7\_Gs} \]
\[ S_{7\_vac} = S_{7\_n} - S_{7\_swcc} \]

\{Gradients\}
\[ v_x = \frac{\text{Grad}_x}{vwc} \]
\[ v_y = \frac{\text{Grad}_y}{vwc} \]
\[ v_z = \frac{\text{Grad}_z}{vwc} \]

\{Review Boundary Properties\}
\[
\begin{align*}
\text{Big} &= 100 \\
\text{ReviewStatement} &= \text{if } h > z \text{ and } v_x > 0 \text{ then } -\text{BIG} \times (h - z) \text{ else } 0
\end{align*}
\]

\{Flux Sections\}
\[
\begin{align*}
\text{NFluxX} &= \text{Grad}_x \times \text{abs}(\text{normal}(1,0,0)) \\
\text{NFluxY} &= \text{Grad}_y \times \text{abs}(\text{normal}(0,1,0)) \\
\text{NFluxZ} &= \text{Grad}_z \times \text{abs}(\text{normal}(0,0,1))
\end{align*}
\]
\[
\begin{align*}
\text{X\_Flux\_1} &= \text{sintegral}(\text{NFluxX} \times \text{"Flux\_1"}) \\
\text{TotalXFlux\_1} &= \text{tintegral}(\text{X\_Flux\_1}) \\
\text{Y\_Flux\_1} &= \text{sintegral}(\text{NFluxY} \times \text{"Flux\_1"}) \\
\text{TotalYFlux\_1} &= \text{tintegral}(\text{Y\_Flux\_1}) \\
\text{Z\_Flux\_1} &= \text{sintegral}(\text{NFluxZ} \times \text{"Flux\_1"}) \\
\text{TotalZFlux\_1} &= \text{tintegral}(\text{Z\_Flux\_1}) \\
\text{NormalFlux\_1} &= \text{sintegral}(\text{abs}((\text{Grad}_x \times \text{normal}(1,0,0)) + (\text{Grad}_y \times \text{normal}(0,1,0)) + (\text{Grad}_z \times \text{normal}(0,0,1))) \times \text{"Flux\_1"}) \\
\text{TotalNormalFlux\_1} &= \text{tintegral}(\text{NormalFlux\_1})
\end{align*}
\]

\{Flux Data Boundary Conditions\}
\[
\begin{align*}
\text{FBC} &= 0 \\
& \text{If } t < 1 \text{ Then } 0 \text{ Else} \\
& \text{If } t >= 1 \text{ and } t < 2 \text{ Then } 0.0208 \text{ Else} \\
& \text{If } t >= 2 \text{ and } t < 3 \text{ Then } 0.0270 \text{ Else} \\
& \text{If } t >= 3 \text{ and } t < 4 \text{ Then } 0.01175 \text{ Else} \\
& \text{If } t >= 4 \text{ and } t < 5 \text{ Then } 0.0076 \text{ Else} \\
& \text{If } t >= 5 \text{ and } t < 6 \text{ Then } 0.0025 \text{ Else} \\
& \text{If } t >= 6 \text{ and } t < 7 \text{ Then } 0.00064 \text{ Else} \\
& 0
\end{align*}
\]

\{Surfaces\}
\[
\begin{align*}
\text{Surface}_8 &= \text{table}("\text{Material\_MP1Tran\_Surface\_8.tbl}") \\
\text{Surface}_7 &= \text{table}("\text{Material\_MP1Tran\_Surface\_7.tbl}") \\
\text{Surface}_6 &= \text{table}("\text{Material\_MP1Tran\_Surface\_6.tbl}")
\end{align*}
\]
Surface_5 = table("Material_MP1Tran_Surface_5.tbl")
Surface_4 = table("Material_MP1Tran_Surface_4.tbl")
Surface_3 = table("Material_MP1Tran_Surface_3.tbl")
Surface_2 = table("Material_MP1Tran_Surface_2.tbl")
Surface_1 = table("Material_MP1Tran_Surface_1.tbl")

{Initial Head}
IH=0.42
hdiff=h-IH

INITIAL VALUES
h=IH

EQUATIONS
\[
\text{div}(\text{vector}(kx*dx(h),ky*dy(h),kz*dz(h))) + s = dt(h)*uww*m2w \quad \text{[H-Based (Conventional) Formulation]}
\]

EXTRUSION
  Surface "Surface 1"  z = Surface_1
  Layer "Layer 1"
  Surface "Surface 2"  z = Surface_2
  Layer "Layer 2"
  Surface "Surface 3"  z = Surface_3
  Layer "Layer 3"
  Surface "Surface 4"  z = Surface_4
  Layer "Layer 4"
  Surface "Surface 5"  z = Surface_5
  Layer "Layer 5"
  Surface "Surface 6"  z = Surface_6
  Layer "Layer 6"
  Surface "Surface 7"  z = Surface_7
  Layer "Layer 7"
  Surface "Surface 8"  z = Surface_8

BOUNDARIES
Region 1  'Region 1'
Layer 1
  Block=1
  Kx= S_1_Kx
  Ky= Kx*S_1_Ky_ratio
  Kz= Kx*S_1_Kz_ratio
  m2w= S_1_m2w
  vwc= if S_1_swcc > vwc_min then S_1_swcc else vwc_min
  Gs= S_1_Gs
  e= S_1_e
  pd= S_1_pd
  pt= S_1_pt
  uwt= S_1_uwt
  Sat= S_1_Sat
  gwc= S_1_gwc
  n= S_1_n
  vac= S_1_vac

Layer 2
  Block=2
  Kx= S_1_Kx
\[ Ky = Kx \times S_1_{Ky\_ratio} \]
\[ Kz = Kx \times S_1_{Kz\_ratio} \]
\[ m2w = S_1_{m2w} \]
\[ vwc = \text{if } S_1_{swcc} > vwc_{\text{min}} \text{ then } S_1_{swcc} \text{ else } vwc_{\text{min}} \]
\[ Gs = S_1_{Gs} \]
\[ e = S_1_{e} \]
\[ pd = S_1_{pd} \]
\[ pt = S_1_{pt} \]
\[ uwt = S_1_{uwt} \]
\[ Sat = S_1_{Sat} \]
\[ gwc = S_1_{gwc} \]
\[ n = S_1_{n} \]
\[ vac = S_1_{vac} \]

Layer 3
Block=3
\[ Kx = S_1_{Kx} \]
\[ Ky = Kx \times S_1_{Ky\_ratio} \]
\[ Kz = Kx \times S_1_{Kz\_ratio} \]
\[ m2w = S_1_{m2w} \]
\[ vwc = \text{if } S_1_{swcc} > vwc_{\text{min}} \text{ then } S_1_{swcc} \text{ else } vwc_{\text{min}} \]
\[ Gs = S_1_{Gs} \]
\[ e = S_1_{e} \]
\[ pd = S_1_{pd} \]
\[ pt = S_1_{pt} \]
\[ uwt = S_1_{uwt} \]
\[ Sat = S_1_{Sat} \]
\[ gwc = S_1_{gwc} \]
\[ n = S_1_{n} \]
\[ vac = S_1_{vac} \]

Layer 4
Block=4
\[ Kx = S_2_{Kx} \]
\[ Ky = Kx \times S_2_{Ky\_ratio} \]
\[ Kz = Kx \times S_2_{Kz\_ratio} \]
\[ m2w = S_2_{m2w} \]
\[ vwc = \text{if } S_2_{swcc} > vwc_{\text{min}} \text{ then } S_2_{swcc} \text{ else } vwc_{\text{min}} \]
\[ Gs = S_2_{Gs} \]
\[ e = S_2_{e} \]
\[ pd = S_2_{pd} \]
\[ pt = S_2_{pt} \]
\[ uwt = S_2_{uwt} \]
\[ Sat = S_2_{Sat} \]
\[ gwc = S_2_{gwc} \]
\[ n = S_2_{n} \]
\[ vac = S_2_{vac} \]

Layer 5
Block=5
\[ Kx = S_3_{Kx} \]
\[ Ky = Kx \times S_3_{Ky\_ratio} \]
\[ Kz = Kx \times S_3_{Kz\_ratio} \]
\[ m2w = S_3_{m2w} \]
\[ vwc = \text{if } S_3_{swcc} > vwc_{\text{min}} \text{ then } S_3_{swcc} \text{ else } vwc_{\text{min}} \]
\[ Gs = S_3_{Gs} \]
\[ e = S_3_e \]
\[ pd = S_3_pd \]
\[ pt = S_3_pt \]
\[ uwt = S_3_uwt \]
\[ Sat = S_3_Sat \]
\[ gwc = S_3_gwc \]
\[ n = S_3_n \]
\[ vac = S_3_vac \]

Layer 6
Block=6
\[ Kx = S_4_Kx \]
\[ Ky = Kx \times S_4_Ky \_ratio \]
\[ Kz = Kx \times S_4_Kz \_ratio \]
\[ m2w = S_4_m2w \]
\[ vwc = \begin{cases} \text{if } S_4_swcc > vwc \_min \text{ then } S_4_swcc \text{ else } vwc \_min \end{cases} \]
\[ Gs = S_4_Gs \]
\[ e = S_4_e \]
\[ pd = S_4_pd \]
\[ pt = S_4_pt \]
\[ uwt = S_4_uwt \]
\[ Sat = S_4_Sat \]
\[ gwc = S_4_gwc \]
\[ n = S_4_n \]
\[ vac = S_4_vac \]

Layer 7
Block=7
\[ Kx = S_5_Kx \]
\[ Ky = Kx \times S_5_Ky \_ratio \]
\[ Kz = Kx \times S_5_Kz \_ratio \]
\[ m2w = S_5_m2w \]
\[ vwc = \begin{cases} \text{if } S_5_swcc > vwc \_min \text{ then } S_5_swcc \text{ else } vwc \_min \end{cases} \]
\[ Gs = S_5_Gs \]
\[ e = S_5_e \]
\[ pd = S_5_pd \]
\[ pt = S_5_pt \]
\[ uwt = S_5_uwt \]
\[ Sat = S_5_Sat \]
\[ gwc = S_5_gwc \]
\[ n = S_5_n \]
\[ vac = S_5_vac \]

Surface 1 natural(h)=-S_1_kx
Surface 8 natural(h)= if h<z and FBC<=S_5_Kx then FBC else if h<z and FBC>S_5_Kx then S_5_Kx else 0

start(0,0)
Layer 1 natural(h)=0
Layer 2 natural(h)=0
Layer 3 natural(h)=0
Layer 4 natural(h)=0
Layer 5 natural(h)=0
Layer 6 natural(h)=0
Layer 7 natural(h)=0
Line to (3.66,0)
Layer 1 nobc(h)
Layer 2 nobc(h)
Layer 3 nobc(h)
Layer 4 nobc(h)
Layer 5 nobc(h)
Layer 6 nobc(h)
Layer 7 nobc(h)
Line to (3.66,10)
Layer 1 natural(h)=0
Layer 2 natural(h)=0
Layer 3 natural(h)=0
Layer 4 natural(h)=0
Layer 5 natural(h)=0
Layer 6 natural(h)=0
Layer 7 natural(h)=0
Line to (0,10)
Line to (0,0)

Region 2  
'Region 2'

Layer 1
Block=8
Kx= S._1_Kx
Ky= Kx*S._1_Ky_ratio
Kz= Kx*S._1_Kz_ratio
m2w= S._1_m2w
vwc= if S._1_swcc > vwc_min then S._1_swcc else vwc_min
Gs= S._1_Gs
e= S._1_e
pd= S._1_pd
pt= S._1_pt
uwt= S._1_uwt
Sat= S._1_Sat
gwc= S._1_gwc
n= S._1_n
vac= S._1_vac

Layer 2
Block=9
Kx= S._6_Kx
Ky= Kx*S._6_Ky_ratio
Kz= Kx*S._6_Kz_ratio
m2w= S._6_m2w
vwc= if S._6_swcc > vwc_min then S._6_swcc else vwc_min
Gs= S._6_Gs
e= S._6_e
pd= S._6_pd
pt= S._6_pt
uwt= S._6_uwt
Sat= S._6_Sat
gwc= S._6_gwc
n= S._6_n
vac= S._6_vac

Layer 3
Block=10
Kx= S._6_Kx
Ky= Kx*S._6_Ky_ratio
\[ K_z = K_x \cdot S_{6_Kz}\_ratio \]
\[ m_{2w} = S_{6_m2w} \]
\[ vwc = \text{if } S_{6_swcc} > vwc_{\text{min}} \text{ then } S_{6_swcc} \text{ else } vwc_{\text{min}} \]
\[ G_s = S_{6_Gs} \]
\[ e = S_{6_e} \]
\[ pd = S_{6_pd} \]
\[ pt = S_{6_pt} \]
\[ uwt = S_{6_uwt} \]
\[ Sat = S_{6_Sat} \]
\[ gwc = S_{6_gwc} \]
\[ n = S_{6_n} \]
\[ vac = S_{6_vac} \]

Layer 4

Block=11
\[ K_x = S_{6_Kx} \]
\[ Ky = K_x \cdot S_{6_Qy}\_ratio \]
\[ K_z = K_x \cdot S_{6_Kz}\_ratio \]
\[ m_{2w} = S_{6_m2w} \]
\[ vwc = \text{if } S_{6_swcc} > vwc_{\text{min}} \text{ then } S_{6_swcc} \text{ else } vwc_{\text{min}} \]
\[ G_s = S_{6_Gs} \]
\[ e = S_{6_e} \]
\[ pd = S_{6_pd} \]
\[ pt = S_{6_pt} \]
\[ uwt = S_{6_uwt} \]
\[ Sat = S_{6_Sat} \]
\[ gwc = S_{6_gwc} \]
\[ n = S_{6_n} \]
\[ vac = S_{6_vac} \]

Layer 5

Block=12
\[ K_x = S_{6_Kx} \]
\[ Ky = K_x \cdot S_{6_Qy}\_ratio \]
\[ K_z = K_x \cdot S_{6_Kz}\_ratio \]
\[ m_{2w} = S_{6_m2w} \]
\[ vwc = \text{if } S_{6_swcc} > vwc_{\text{min}} \text{ then } S_{6_swcc} \text{ else } vwc_{\text{min}} \]
\[ G_s = S_{6_Gs} \]
\[ e = S_{6_e} \]
\[ pd = S_{6_pd} \]
\[ pt = S_{6_pt} \]
\[ uwt = S_{6_uwt} \]
\[ Sat = S_{6_Sat} \]
\[ gwc = S_{6_gwc} \]
\[ n = S_{6_n} \]
\[ vac = S_{6_vac} \]

Layer 6

Block=13
\[ K_x = S_{6_Kx} \]
\[ Ky = K_x \cdot S_{6_Qy}\_ratio \]
\[ K_z = K_x \cdot S_{6_Kz}\_ratio \]
\[ m_{2w} = S_{6_m2w} \]
\[ vwc = \text{if } S_{6_swcc} > vwc_{\text{min}} \text{ then } S_{6_swcc} \text{ else } vwc_{\text{min}} \]
\[ G_s = S_{6_Gs} \]
\[ e = S_{6_e} \]
\[ \text{Layer 7} \]
\[ \text{Block} = 14 \]
\[ K_x = S_5.K_x \]
\[ K_y = K_x \times S_5.K_y \text{_ratio} \]
\[ K_z = K_x \times S_5.K_z \text{_ratio} \]
\[ m_{2w} = S_5.m_{2w} \]
\[ vwc = \text{if } S_5.swcc > vwc_{\text{min}} \text{ then } S_5.swcc \text{ else } vwc_{\text{min}} \]
\[ G_s = S_5.G_s \]
\[ e = S_5.e \]
\[ p_d = S_5.p_d \]
\[ p_t = S_5.p_t \]
\[ u_{wt} = S_5.u_{wt} \]
\[ S_a = S_5.S_a \]
\[ g_{wc} = S_5.g_{wc} \]
\[ n = S_5.n \]
\[ v_{ac} = S_5.v_{ac} \]

Surface 1 natural(h)\(=\)\(S_1.k_x\)
Surface 8 natural(h)\(=\)\(\text{if } h<z \text{ and } FBC<=S_5.K_x \text{ then } FBC \text{ else if } h<z \text{ and } FBC>S_5.K_x \text{ then } S_5.K_x \text{ else } 0 \)

\text{start}(3.66,0)
Layer 1 natural(h)\(=0\)
Layer 2 natural(h)\(=0\)
Layer 3 natural(h)\(=0\)
Layer 4 natural(h)\(=0\)
Layer 5 natural(h)\(=0\)
Layer 6 natural(h)\(=0\)
Layer 7 natural(h)\(=0\)
Line to (4.11,0)
Layer 1 nobc(h)
Layer 2 nobc(h)
Layer 3 nobc(h)
Layer 4 nobc(h)
Layer 5 nobc(h)
Layer 6 nobc(h)
Layer 7 nobc(h)
Line to (4.11,10)
Layer 1 nobc(h)
Layer 2 nobc(h)
Layer 3 nobc(h)
Layer 4 nobc(h)
Layer 5 nobc(h)
Layer 6 nobc(h)
Layer 7 nobc(h)
Line to (3.66,0)

Region 3  'Region 3'

Layer 1
Block=15
Kx=S_1_Kx
Ky=Kx*S_1_Ky_ratio
Kz=Kx*S_1_Kz_ratio
m2w=S_1_m2w
vwc=if S_1_swcc > vwc_min then S_1_swcc else vwc_min
Gs=S_1_Gs
e=S_1_e
pd=S_1_pd
pt=S_1_pt
uwt=S_1_uwt
Sat=S_1_Sat
gwc=S_1_gwc
n=S_1_n
vac=S_1_vac

Layer 2
Block=16
Kx=S_6_Kx
Ky=Kx*S_6_Ky_ratio
Kz=Kx*S_6_Kz_ratio
m2w=S_6_m2w
vwc=if S_6_swcc > vwc_min then S_6_swcc else vwc_min
Gs=S_6_Gs
e=S_6_e
pd=S_6_pd
pt=S_6_pt
uwt=S_6_uwt
Sat=S_6_Sat
gwc=S_6_gwc
n=S_6_n
vac=S_6_vac

Layer 3
Block=17
Kx=S_7_Kx
Ky=Kx*S_7_Ky_ratio
Kz=Kx*S_7_Kz_ratio
m2w=S_7_m2w
vwc=if S_7_swcc > vwc_min then S_7_swcc else vwc_min
Gs=S_7_Gs
e=S_7_e
pd=S_7_pd
pt=S_7_pt
uwt=S_7_uwt
Sat=S_7_Sat
gwc=S_7_gwc
n=S_7_n
vac = S_7_vac

Layer 4
Block = 18
Kx = S_6_Kx
Ky = Kx * S_6_Ky_ratio
Kz = Kx * S_6_Kz_ratio
m2w = S_6_m2w
vwc = if S_6_swcc > vwc_min then S_6_swcc else vwc_min
Gs = S_6_Gs
e = S_6_e
pd = S_6_pd
pt = S_6_pt
uwt = S_6_uwt
Sat = S_6_Sat
gwc = S_6_gwc
n = S_6_n
vac = S_6_vac

Layer 5
Block = 19
Kx = S_6_Kx
Ky = Kx * S_6_Ky_ratio
Kz = Kx * S_6_Kz_ratio
m2w = S_6_m2w
vwc = if S_6_swcc > vwc_min then S_6_swcc else vwc_min
Gs = S_6_Gs
e = S_6_e
pd = S_6_pd
pt = S_6_pt
uwt = S_6_uwt
Sat = S_6_Sat
gwc = S_6_gwc
n = S_6_n
vac = S_6_vac

Layer 6
Block = 20
Kx = S_6_Kx
Ky = Kx * S_6_Ky_ratio
Kz = Kx * S_6_Kz_ratio
m2w = S_6_m2w
vwc = if S_6_swcc > vwc_min then S_6_swcc else vwc_min
Gs = S_6_Gs
e = S_6_e
pd = S_6_pd
pt = S_6_pt
uwt = S_6_uwt
Sat = S_6_Sat
gwc = S_6_gwc
n = S_6_n
vac = S_6_vac

Layer 7
Block = 21
Kx = S_5_Kx
\( K_y = K_x \times S_{5\_Ky\_ratio} \)
\( K_z = K_x \times S_{5\_Kz\_ratio} \)
\( m_{2w} = S_{5\_m2w} \)
\( vwc = \text{if} \ S_{5\_swcc} > vwc_{\text{min}} \text{then} \ S_{5\_swcc} \text{else} \ vwc_{\text{min}} \)
\( G_s = S_{5\_Gs} \)
\( e = S_{5\_e} \)
\( pd = S_{5\_pd} \)
\( pt = S_{5\_pt} \)
\( uwt = S_{5\_uwt} \)
\( Sat = S_{5\_Sat} \)
\( gwc = S_{5\_gwc} \)
\( n = S_{5\_n} \)
\( vac = S_{5\_vac} \)

Surface 1 natural(h) = \( -S_{1\_kx} \)
Surface 8 natural(h) = \( \text{if} \ h < z \text{and} FBC \leq S_{5\_Kx} \text{then} FBC \text{else} \text{if} \ h < z \text{and} FBC > S_{5\_Kx} \text{then} S_{5\_Kx} \text{else} 0 \)

start(4.11,0)
Layer 1 natural(h) = 0
Layer 2 natural(h) = 0
Layer 3 natural(h) = 0
Layer 4 natural(h) = 0
Layer 5 natural(h) = 0
Layer 6 natural(h) = 0
Layer 7 natural(h) = 0
Line to (4.26,0)
Line to (4.26,10)
Line to (4.11,10)
Layer 1 nobc(h)
Layer 2 nobc(h)
Layer 3 nobc(h)
Layer 4 nobc(h)
Layer 5 nobc(h)
Layer 6 nobc(h)
Layer 7 nobc(h)
Line to (4.11,0)

Region 4
Layer 1
VOID
Layer 2
VOID
Layer 3
Block=22
\( K_x = S_{7\_Kx} \)
\( K_y = K_x \times S_{7\_Ky\_ratio} \)
\( K_z = K_x \times S_{7\_Kz\_ratio} \)
\( m_{2w} = S_{7\_m2w} \)
\( vwc = \text{if} \ S_{7\_swcc} > vwc_{\text{min}} \text{then} \ S_{7\_swcc} \text{else} \ vwc_{\text{min}} \)
\( G_s = S_{7\_Gs} \)
\( e = S_{7\_e} \)
\( pd = S_{7\_pd} \)
\( pt = S_{7\_pt} \)
\( uwt = S_{7\_uwt} \)
\( Sat = S_{7\_Sat} \)
\( gwc = S_{7\_gwc} \)
\texttt{n= S\_7\_n} \\
\texttt{vac= S\_7\_vac} \\

Layer 4 \\
\texttt{VOID} \\
Layer 5 \\
\texttt{VOID} \\
Layer 6 \\
\texttt{VOID} \\
Layer 7 \\
\texttt{VOID} \\
\texttt{Surface 3 natural(h)=0} \\
\texttt{Surface 4 natural(h)=0} \\

\texttt{start(4.26,0)} \\
\texttt{Layer 1 nobc(h)} \\
\texttt{Layer 2 nobc(h)} \\
\texttt{Layer 3 natural(h)=0} \\
\texttt{Layer 4 natural(h)=0} \\
\texttt{Layer 5 nobc(h)} \\
\texttt{Layer 6 nobc(h)} \\
\texttt{Layer 7 nobc(h)} \\
\texttt{Line to (7.31,0)} \\
\texttt{Layer 3 natural(h)= ReviewStatement} \\
\texttt{Layer 4 natural(h)= ReviewStatement} \\
\texttt{Line to (7.31,0.102)} \\
\texttt{Layer 3 natural(h)=0} \\
\texttt{Layer 4 natural(h)=0} \\
\texttt{Line to (4.26,0.102)} \\
\texttt{Layer 3 nobc(h)} \\
\texttt{Layer 4 nobc(h)} \\
\texttt{Line to (4.26,0)} \\

\{Flux Sections\} \\
\texttt{feature "Flux\_1" Start (7.3,-2) Line to (7.3,3)} \\

\{Review Boundary Paths\} \\
\texttt{feature start "Review Boundary 1: Region 4, Surface 3, Point 1" (7.31,0) Line to (7.31,0.10)} \\
\texttt{feature start "Review Boundary 2: Region 4, Surface 4, Point 1" (7.31,0) Line to (7.31,0.10)} \\

\texttt{TIME} \\
\texttt{From StartTime To EndTime By TimeInc} \\

\texttt{PLOTS} \\
\texttt{For T=0 by 1 to 48 Contour(h) on Y=0.1 as "Head"} \\

\{Output Files\} \\
\texttt{For T = 0 by 1 to 48} \\
\texttt{Summary as "Flux Section Report: Flux 1"} \\
\texttt{report " "} \\
\texttt{report"Instantaneous Flow Rate"} \\
\texttt{report(X\_Flux\_1) as "X Component of Flow in (m^3/hr) "} \\
\texttt{report(Y\_Flux\_1) as "Y Component of Flow in (m^3/hr) "} \\
\texttt{report(Z\_Flux\_1) as "Z Component of Flow in (m^3/hr) "} \\
\texttt{report(NormalFlux\_1) as "Normal Flow in (m^3/hr) "} \\

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report "" ""report""Total Flow"
    report(TotalXFlux_1) as "Total X Flow in (m^3)",
    report(TotalYFlux_1) as "Total Y Flow in (m^3)",
    report(TotalZFlux_1) as "Total Z Flow in (m^3)",
    report(TotalNormalFlux_1) as "Total Normal Flow in (m^3)"
report "" ""

history(X_Flux_1) Export format"#t,#" Print file "X_Flux_1.tbl" as "Flux Section: Flux 1 - X Component of Flow - Rate (m^3/hr)"
history(NormalFlux_1) Export format"#t,#" Print file "NormalFlux_1.tbl" as "Flux Section: Flux 1 - Normal Flow - Rate (m^3/hr)"

{Review Boundary Plots}
HISTORIES
history(Sat) at(0,1,0.52)(0,10,0.7)(1.8,1,0.484)(1.8,10,0.664)(3.6,1,0.448)(3.6,5,0.528)(3.6,10,0.628) as "Sat of Subgrade" EXPORT FORMAT "##ttt,###I"
history(u) at(0,1,0.52)(0,10,0.7)(1.8,1,0.484)(1.8,10,0.664)(3.6,1,0.448)(3.6,5,0.528)(3.6,10,0.628) as "PWP of Subgrade" EXPORT FORMAT "##ttt,###I"
history(Sat) at(0,1,0.72)(0,10,0.9)(1.8,1,0.684)(1.8,5,0.764)(1.8,10,0.864)(3.6,1,0.648)(3.6,5,0.728)(3.6,10,0.828) as "Sat of Filter" EXPORT FORMAT "##ttt,###I"
history(u) at(0,1,0.72)(0,10,0.9)(1.8,1,0.684)(1.8,5,0.764)(1.8,10,0.864)(3.6,1,0.648)(3.6,5,0.728)(3.6,10,0.828) as "PWP of Filter" EXPORT FORMAT "##ttt,###I"
history(Sat) at(0,1,0.82)(0,10,1.1)(1.8,1,0.784)(1.8,5,0.864)(1.8,10,0.964)(3.6,1,0.748)(3.6,5,0.828)(3.6,10,0.928) as "Sat of Base 1" EXPORT FORMAT "##ttt,###I"
history(u) at(0,1,0.82)(0,10,1.1)(1.8,1,0.784)(1.8,5,0.864)(1.8,10,0.964)(3.6,1,0.748)(3.6,5,0.828)(3.6,10,0.928) as "PWP of Base 1" EXPORT FORMAT "##ttt,###I"
history(Sat) at(0,1,0.92)(0,10,1.1)(1.8,1,0.884)(1.8,5,0.964)(1.8,10,1.064)(3.6,1,0.848)(3.6,5,0.928)(3.6,10,1.028) as "Sat of Base 2" EXPORT FORMAT "##ttt,###I"
history(u) at(0,1,0.92)(0,10,1.1)(1.8,1,0.884)(1.8,5,0.964)(1.8,10,1.064)(3.6,1,0.848)(3.6,5,0.928)(3.6,10,1.028) as "PWP of Base 2" EXPORT FORMAT "##ttt,###I"
history(Sat) at(0,1,1.2)(0,10,1.2)(1.8,1,0.984)(1.8,5,1.064)(1.8,10,1.164)(3.6,1,0.948)(3.6,5,1.028)(3.6,10,1.128) as "Sat of Surface" EXPORT FORMAT "##ttt,###I"
history(u) at(0,1,1.2)(0,10,1.2)(1.8,1,0.984)(1.8,5,1.064)(1.8,10,1.164)(3.6,1,0.948)(3.6,5,1.028)(3.6,10,1.128) as "PWP of Surface" EXPORT FORMAT "##ttt,###I"

END
## APPENDIX B: DRAINAGE RECOMMENDATIONS FOR PAVEMENT DESIGN OPTIONS

Table Appendix B.1 Summary of Effects on Pavement Drainage

<table>
<thead>
<tr>
<th>Effect Factors</th>
<th>Level</th>
<th>Peak Flow Effect</th>
<th>Time-to-drain Effect</th>
<th>Total Flow Effect</th>
<th>Power</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade Soil Permeability</td>
<td>L</td>
<td>↑</td>
<td>↑</td>
<td>↑</td>
<td>+1</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>↓</td>
<td>↓</td>
<td>↓</td>
<td>-1</td>
</tr>
<tr>
<td>Base Permeability</td>
<td>L</td>
<td>↓</td>
<td>↑</td>
<td>↓</td>
<td>+1</td>
</tr>
<tr>
<td></td>
<td>M</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>↑</td>
<td>↓</td>
<td>↑</td>
<td>+1</td>
</tr>
<tr>
<td>Filter Water Retention</td>
<td>L</td>
<td>Base Perm. L</td>
<td>↑</td>
<td>↑</td>
<td>+1</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>Base Perm. H</td>
<td>↓</td>
<td>↓</td>
<td>+1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Base Perm. L</td>
<td>↓</td>
<td>↓</td>
<td>-1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Base Perm. H</td>
<td>↑</td>
<td>↓</td>
<td>-1</td>
</tr>
<tr>
<td>Surface Permeability</td>
<td>L</td>
<td>↓</td>
<td>--</td>
<td>↓</td>
<td>-1</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>↑</td>
<td>--</td>
<td>↑</td>
<td>+2</td>
</tr>
<tr>
<td>Rainfall Quantity</td>
<td>L</td>
<td>Rain Duration L</td>
<td>--</td>
<td>↓</td>
<td>-2</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>Rain Duration H</td>
<td>↓</td>
<td>↑</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rain Duration L</td>
<td>↑</td>
<td>↓</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rain Duration H</td>
<td>↑</td>
<td>↑</td>
<td>+2</td>
</tr>
</tbody>
</table>

Note: 1. L=Low, M=Medium, H=High, --=No Effect, ↑=Increase, ↓=Decrease.

2. These combinations include an edgedrain system.

3. The effect powers were evaluated based on the analyses in Chapters 5.0 and 6.0.
### Table Appendix B.2 Pavement Material Parameter Range

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Level</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade Soil Permeability</td>
<td>L</td>
<td>$&lt; 10^{-6}$ cm/s (0.00283 ft/day)</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$\geq 10^{-6}$ cm/s (0.00283 ft/day)</td>
</tr>
<tr>
<td>Base and Subbase Permeability</td>
<td>L</td>
<td>$&lt; 0.01$ cm/s (28.3 ft/day)</td>
</tr>
<tr>
<td></td>
<td>M</td>
<td>$0.01 \sim 1$ (28.3 \sim 2830 ft/day)</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$&gt;1$ cm/s (2830 ft/day)</td>
</tr>
<tr>
<td>Filter Water Retention</td>
<td>L</td>
<td>$\frac{\theta_r}{\theta_s} &gt; \frac{1}{4}$</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$\frac{\theta_r}{\theta_s} \leq \frac{1}{4}$</td>
</tr>
<tr>
<td>Surface Permeability</td>
<td>L</td>
<td>$&lt; 10^{-3}$ cm/s (2.83 ft/day)</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$\geq 10^{-3}$ cm/s (2.83 ft/day)</td>
</tr>
<tr>
<td>Rainfall Quantity</td>
<td>L</td>
<td>$&lt;50.8$ mm (2 inches)</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$&gt;50.8$ mm (2 inches)</td>
</tr>
</tbody>
</table>

Note: $\theta_r$ = residual volumetric water content, $\theta_s$ = saturated volumetric water content.
Table Appendix B.3 Drainage Design Recommendations for Pavements at Dry Area

<table>
<thead>
<tr>
<th>Pavement Design Options</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade Permeability</td>
<td>Base Permeability</td>
</tr>
<tr>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>L</td>
<td>L</td>
</tr>
<tr>
<td>L</td>
<td>M</td>
</tr>
<tr>
<td>L</td>
<td>M</td>
</tr>
<tr>
<td>L</td>
<td>M</td>
</tr>
<tr>
<td>L</td>
<td>M</td>
</tr>
<tr>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>L</td>
<td>H</td>
</tr>
<tr>
<td>H</td>
<td>L</td>
</tr>
<tr>
<td>H</td>
<td>L</td>
</tr>
<tr>
<td>H</td>
<td>L</td>
</tr>
<tr>
<td>H</td>
<td>L</td>
</tr>
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Note: L=Low, M=Medium, H=High, Y=Yes, N=No, C=Coarse, -=No Requirement.
Table Appendix B.4 Drainage Design Recommendations for Pavements at Medium Wet Area

<table>
<thead>
<tr>
<th>Pavement Design Options</th>
<th>Recommendations</th>
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<td><strong>Subgrade Permeability</strong></td>
<td><strong>Base Permeability</strong></td>
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Note: L=Low, M=Medium, H=High, Y=Yes, N=No, C=Coarse, -=No Requirement.
Table Appendix B.5 Drainage Design Recommendations for Pavements at Wet Area

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<tr>
<th>Subgrade Permeability</th>
<th>Base Permeability</th>
<th>Filter Water Retention</th>
<th>Surface Permeability</th>
<th>Edgedrain Required</th>
<th>Minimum Collection Pipe Slope (%)</th>
<th>Minimum Outlet Pipe Slope (%)</th>
<th>Minimum Pipe Diameters (mm (in.))</th>
<th>Trench Backfill Gradation</th>
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REFERENCES


PERSONAL INFORMATION

Date of Birth: December 27, 1970                      Place of Birth: Heilongjiang, China
Citizenship: P. R. China                                      Race: Asian

EXPERTIES & SKILLS

- Pavement performance evaluation and mechanical-experimental design
- Application of pavement design and management packages, including AASHTO 2002 Design Guide Software, KENLAYER, KENSLAB, DRIP, DATAPAVE, etc
- Pavement surface and subsurface drainage design and analysis
- Finite element modeling and analysis using FEA software, such as ANSYS, FlexPDE, etc
- Statistical modeling and analysis using SAS, SPSS, EXCEL and MAPLE.
- Saturated and unsaturated groundwater modeling and analysis
- Application of Various geotechnical analysis packages, such as GEO-SLOPE and SOILVISION.
- Pavement material property characterization and prediction
- Pavement material quality assessment and control
- GIS application and analysis using ARCVIEW GIS

EDUCATION

- University of Kentucky, Lexington, KY, MSCE Pavement and Materials, December 2001
- Harbin Institute of Technology, Harbin, China, MSCE Materials, July 1996
- Harbin Institute of Technology, Harbin, China, BSCE Materials (ABET ECCI accredited), July 1993

PUBLICATIONS


WORK EXPERIENCE

- 2000-date Research Assistant at University of Kentucky, Lexington, KY

Projects Conducted
- Pavement Subsurface Drainage System Modeling and Analyses--Jan. 2003-date
- Pavement Concrete Quality Control and Assurance Software Development--Dec. 2002-May 2003

**Academic Work**
- Research on engineering student creativity promotion. The paper “Creativity in design: A Cross-discipline approach” has been awarded by American Society of Engineering Education.

- **1996-2000 Faculty Member of Civil Engineering Department at Beijing Polytechnic University, Beijing, China**

**Classes Taught**
- Building Materials--a class imparting knowledge about the production, properties (chemical, physical and mechanical), and application of various materials used in building construction, such as cement, concrete, timber, steel, polymer materials, etc..
- Highway Materials--a class imparting knowledge about the production, properties and application of materials used in highway construction, such as asphalt, and asphalt concrete, soil, aggregate, Portland cement and concrete, etc..

**Researches Involved**
- Study of Sulpho-alumious High Performance Concrete
- Application of Sulpho-alumious Material in Soil Consolidation

**Horns & Awards**
- ASEE Civil Engineering Division’s Glen L. Martin Best Paper Award for 2003
- Beijing Polytechnic University Young Teacher Best Technical Paper Award for 1998
- Beijing Polytechnic University Best Instructor Award for 1998
- Harbin Institute of Architecture and Civil Engineering Excellent Student Award 1990-1993

**Licensure and Certification**
- EIT of Kentucky