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LOCATING THE DRAINAGE LAYER
FOR FLEXIBLE PAVEMENTS

Hossam F. Hassan
Thomas D. White

PURDUE UNIVERSITY

FINAL REPORT

LOCATING THE DRAINAGE LAYER FOR FLEXIBLE PAVEMENTS

FHWA/IN/JHRP-96/14

by

Hossam F. Hassan
Research Assistant

and

Thomas D. White
Research Engineer

Purdue University
Department of Civil Engineering

Joint Highway Research Project

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16. Abstract <p>Pavement subsurface drainage and its effect on pavement performance has been a subject of interest since the 18th and 19th centuries. With no doubt the detrimental effects of heavy wheel loads on pavements with saturated base material is a significant factor. The consequence of subsurface water on pavement performance includes premature rutting, cracking, faulting, and increased roughness, all of which lead to a decrease in serviceability.</p> <p>This research study involves the evaluation of the drainage performance of three section configurations. The sections were built with a difference in the filter as well as the drainage layer. Indiana #5D, and #53 impermeable layers were used as a filter. Indiana #2 and #5C base were used as drainage layer.</p> <p>The study was carried out by field instrumentation, laboratory testing, field data collection, and numerical modeling. The main objective of this study is to evaluate the subdrainage performance of three pavement sections adopted by the Indiana Department of Transportation (INDOT). Instruments were installed to monitor the air and pavement temperature, frost penetration, and pavement moisture conditions, and time and duration of rainfall and pavement outflow volumes. Subgrade and asphalt core samples were obtained from the field. Tests were performed on these samples to determine their hydraulic conductivity characteristics.</p> <p>It was found that the permeability of the #5C drainage base layer material was higher than the #2 base by approximately 10 times. Since most of the water source in the pavement was the surface infiltration, the filter layer plays a key role in controlling the moisture migration from the pavement into the subgrade. The section with the #5D HMA impermeable layer showed the lowest moisture migration into the subgrade. The #5C base had the tendency to retain less water than the #2 base, making the stripping potential less of a problem. Contamination of the trench material from the #53 aggregate fines appears to have occurred, and therefore, section 1 (#5D filter layer). In addition, the outlet pipe inlet capacity was found to be low.</p> <p>Frost penetration was found to be about 1.0 m. This result compared well with empirical methods. From the field temperature measurements, the SHRP coldest surface pavement temperature was evaluated and found in good agreement. A large amount of data was obtained about pavement and subgrade material hydraulic characteristics.</p> <p>The finite element analysis showed good simulation of the actual pavement surface conditions. A simulation of cracked surface pavement showed a full saturation condition of the pavement layers.</p>					
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IMPLEMENTATION REPORT

The study is directed towards evaluating three subdrainage alternatives for flexible pavements. The sections use the Indiana #53 aggregate, and the #5D base. The #2 base and #5C base are used as drainage layers.

In the good surface condition pavement, the filter layer was the main factor affecting the drainage time. Section 1 showed a shorter drainage time.

The following recommendations are made to INDOT to implement the findings of this research.

1. The #5C base is an effective drainage layer. Its effectiveness is based on having a higher permeability and lower moisture retention.
2. A dense HMA such as the #5D HMA is recommended as a filter separation layer. The #5D acted as a barrier to moisture migration into the subgrade and stores and insignificant volume of water itself.
3. The inlet capacity of current collector pipes specified by INDOT is inadequate. Inlet capacity should be increased by at least a factor of four. Choice of pavement section and material should be based on minimizing trench material or pipe inlet contamination.
4. Data indicates frost penetration into pavement sections at Fort Wayne to a depth of one meter. A full frost protection design would involve use of a non frost susceptible material to this depth.

5. Based on only the 1995/1996 winter data, the SHRP predicted coldest pavement surface temperature agreed well with the field measurement temperature. The hottest pavement temperature will be evaluated in the second phase of this study.

CHAPTER 1 - INTRODUCTION

Pavement subsurface drainage has been a subject of interest for many years. Recent Historical concern with water in or adjacent to the pavement structure began with P.M.J. Tresaquet, the originator of the "french drain", in France, and Thomas Telford and John L.McAdam in England during the late 18th and early 19th centuries. These engineers were concerned with the removal of both the surface and subsurface water. McAdam's fundamental principle-"that it is the native soil which really supports the weight of the traffic; that while it is preserved in a dry state it will carry any weight without sinking"-is still applicable today as it was then (Ridgeway, 1982).

Although a number of factors contribute to pavement deterioration and failure. The detrimental effects of heavy wheel loads on pavement systems containing free water is probably the greatest single factor (Cedergren, 1974).

1.1 Problem Statement

The Indiana Department of Transportation (INDOT) adopted a new set of typical pavement sections for new construction and reconstruction in 1993. Most of these sections include a drainage layer which is intended to carry water to edge drains. Bituminous bases 5C, 2, and 5 are being used as drainage layers for these sections. Coarse aggregate No.53 and Bituminous base 5D are being used as filter layers under the drainage layer.

These materials have different maximum aggregate size, gradation and asphalt content. Therefore, their hydraulic characteristics (water retention and hydraulic conductivity) are different. Pavement sections constructed with these materials will have different drainage behavior, and performance. Three particular sections have been identified

by INDOT as sections to be evaluated. These sections are shown in Figure 1.1. A suitable project on I-469 around Fort Wayne, Indiana was selected for constructing these three test sections.

1.2 Objectives and Scope

The present study focuses on the drainage performance of the three candidate sections. The study can be divided into four basic parts: field instrumentation, laboratory testing, analysis of field data, and finite element modeling of pavement drainage.

After selecting a suitable location for the three sections on the project site, the instrumentation was installed in the subgrade and pavement at pre-selected locations. The instrumentation consisted of thermocouples and thermistors for temperature measurement, time domain reflectometry (TDR) probes, Watermark suction blocks, and neutron gauge for moisture content measurements, resistivity probe for frost penetration, and rain gauge and outflow meters. These instruments were monitored by a battery powered data acquisition system.

The main objective of the laboratory part of the study was to obtain the hydraulic characteristics of the materials in the sections. These properties are essential for numerical modeling.

Another objective of the study was to analyze the field data and evaluate the drainage performance, water content changes, and frost penetration in each of the sections. The drainage efficiency and time to drain would be used as factors in the comparisons.

Numerical modeling was included in the study to provide a better understanding of the moisture migration in the sections. Also, experience would be gained about the potential use of such analysis to simulate and analyze different pavement conditions.

The report consists of eight chapters. A review of literature on pavement subdrainage as well as numerical analysis of flow in porous media is presented in chapter two. Chapter three presents a literature review on the instrumentation. Chapter four presents the test site selection and installation scenario. Chapter five presents the laboratory tests performed on the subgrade and the asphalt core samples. Chapter six presents the analysis of the field data.

Figure 1.1 Test Section Configurations.

Chapter seven presents the numerical analysis of the subdrainage. Chapter eight presents the conclusions and recommendations.

CHAPTER 2 - LITERATURE REVIEW

2.1 Historical Background

There are many indications that the drainage of water was an important consideration in road construction by the Romans, Greeks, and Egyptians 2000 years before McAdam, Tresauquet, and Telford. In 1823, John L. McAdam said “The road can never be rendered thus perfectly secure until the following principles be fully understood, admitted, and acted upon, namely, that it is the native soil which really supports the weight of traffic; that while it is preserved in a dry state it will carry any weight without sinking”. For more than a century following McAdam remarks there was an almost unanimous agreement among road builders that pavements should be kept in a well drained state (Ridgeway, 1980).

With the advent of rational methods for designing pavements, there was a tendency to believe that McAdam was wrong, and that as long as soil strengths were determined by making tests on saturated test specimens, environmental factors such as rainfall were of no great importance, and were automatically taken into account by the design methods. Regardless of what method is used in the design, it is a fact that much greater damage occurs to pavements during the periods they contain free water compared to periods when they are without free water (Ridgeway, 1980).

Until the 1940s and 1950s, nearly every text or handbook on pavement design or road building could be counted on to contain a statement to the effect that “There are just three things necessary for good roads, and they are drainage, drainage and more drainage.” (Cedergren, 1974).

2.2 Sources of Water in the Pavement

Water reaches the pavement through the atmosphere from rainfall (usually the largest source), snow, dew, melting ice, capillarity from a free ground water surface, and from melting of frozen pavement layers and/or subgrades. Water from the atmosphere can enter the pavement structural section in several ways (Ridgeway, 1980):

- 1- Cracks in the pavement surface. Although new pavements can be constructed virtually without cracks, but they still have joints and they will develop cracks during the pavement service life.
- 2- Infiltration through the shoulders and the shoulder pavement joint.
- 3- Infiltration through the side ditch.
- 4- Melting of an ice layer.
- 5- Condensation of water vapor, generally small amounts.
- 6- Capillarity.

2.3 Water related Problems in the Pavement

The detrimental effects of heavy wheel loads on pavement systems containing free water is probably the greatest single factor contributing to pavement deterioration and failure. Experts who studied the behavior of important experimental test pavements such as the AASHTO test road at Ottawa, Illinois, and the WASHO test road in Idaho, reported that damage caused by traffic during the short periods of “worst “ environmental conditions were hundreds to thousands of times greater than during the “best” periods (Summer months). Tests by Barenberg and Thompson with a circular test track which was designed to simulate conditions at the AASHTO road test, indicated 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 (Cedergren, 1974).

Deterioration is also significant when there is excess moisture at the interface of successive lifts of AC pavements or at the boundaries between PCC pavements and their subbases. High repair and replacement costs on major highways and airfields are nearly always associated with heavy loads on sections containing free water (Cedergren, 1974).

Some of the most harmful actions of traffic and water occur entirely within the structural section of a pavement and are unrelated to subgrade strength or behavior. Some cases where water caused problems in flexible pavement layers was investigated by Adams (1969). Also, Cedergren (1974) investigated eight interstate highway test sites. There was no evidence that subgrade weakness was a major factor.

Heavy wheel loads can apply a water hammer type action on saturated bases and subbases. Pulsating water pressure not only can cause erosion and ejection of material from beneath pavements, but also can strip asphalt coatings from bituminous-stabilized bases and subbases. In this case, the water acts as if it were in a closed system and the surface tire contact pressure is transmitted to the subgrade through the pore pressure and could reach the same value of the contact pressure without the normally assumed stress distribution due to interparticle contact. Water action can disintegrate cement-treated bases, and weaken base courses by rearranging the internal structure of fine-grained materials in aggregate mixtures. In short, the presence of internal flooding can be blamed for 90% or more of the serious pavement problems (Cedergren, 1974).

The adverse effects of subsurface water causing unsatisfactory pavement performance include premature rutting, cracking, faulting, increased roughness and decrease in serviceability. If the pavement structural section and subgrade become saturated by groundwater and/or infiltration, its ability to transmit dynamic loads imposed by traffic can be greatly impaired. This is due to the induced high pore water pressure from moving wheel loads combined with the loss of strength in unbound bases, subbases, and subgrades. The pore pressure may cause water and fines to be ejected from joints, cracks, and edges of the pavement (pumping). This erosion of material, if continued for a period of time, will lead to loss of support. A solution for this problem lies in sealing the cracks and joints and including good subdrainage to prevent accumulation of water under the pavement (Moulton, 1980)

The frequent or sustained presence of excess moisture in pavement components and intermittent exposure to cycles of freezing and thawing can result in material deterioration. In the case of flexible pavement, this contributes to stripping, and in the case of concrete pavement it appears as "D" cracking. In both cases, excluding excess moisture or providing

for its rapid removal with appropriate drainage can be beneficial in minimizing the damage (Ridgeway, 1980).

In asphalt pavements, frost action can cause bumps and sags in the pavements. Edge cracking also occurs due to frost heave or weakened base or subgrade near the edge of the pavement. These effects can occur with and without traffic. Pavement cracking can be the location for a pothole to form. Accumulation of free water in the pothole accelerates its growth (Shahin et al., 1983).

Results from field studies conducted by Majidzadeh on I-70 and I-77 in Ohio indicated that concrete pavement performance is directly related to drainage conditions. Where adequate drainage was provided for base, subbase, and shoulder, the pavement performed superbly even after 15 years. On the other hand, lack of positive drainage, poor subbase with high fines content resulted in contamination of the base and a plugged collector pipe. The result was joint deterioration and pavement failure (Ring, 1977).

2.4 Performance of Drained Pavements

Dempsey (1979) conducted a study on two jointed concrete pavements, one continuously reinforced pavement and one jointed reinforced pavement. Both sections had subsurface drainage. Water outflow from the sections was measured and precipitation was obtained from the closest weather station. Unsealed, sealed edge and transverse joints were included in the sections. Edge sealing significantly reduced the drainage outflow.

Ahmed et al. (1993) conducted a study on the performance of existing pavement subdrainage systems in Indiana through inspection of collector systems and application of instrumentation to monitor moisture conditions. Observations in the study revealed that outlet pipes were frequently exposed for some length or crushed. This type of damage is shown in Figure 2.1. Outlet markers were not present in the majority of cases, making it difficult to locate the outlets. Rodent screens were damaged or did not cover the pipe outlet, allowing small rodents access. Vegetation growth around the pipes sometimes blocked the flow completely. Internal camera inspection showed intrusion of soil and weeds inside punctured outlet pipes. Figure 2.2 shows an example of such observation. Field results were

Figure 2.1 Crushed Outlet Pipe (Ahmed et al., 1993).

Figure 2.2. Weeds inside an Outlet Pipe (Ahmed et al., 1993).

collected for different types of pavements, and the drainage efficiency was reported for these pavements.

2.5 Need for Drainage

Excess water that enters pavement sections (primarily from rainfall) and other surface infiltration cause serious pavement damage. As a result, pavement life is shortened and annual costs of pavements increased (Cedergren, 1974).

Many designers feel that it is only necessary to keep the surface impermeable and there will be no need for subsurface drainage. Unfortunately, the present state of the art does not guarantee a high level of water tightness, even when pavements are relatively new, significant amounts of water can enter the pavements. Research by the University of Maryland on inflow into surface cracks in PCC pavements revealed that 70 to 95 % of precipitation at a rate of 5 cm/hr entered the pavement. The crack widths ranged from 0.089 to 0.3175 cm. There is a tendency to underestimate the water that can enter into the pavements, and to overestimate the capabilities of the base to remove water (Cedergren, 1974).

Unless high permeability drainage layers are installed the full width of the pavement, the water that enters often remains on the subgrade or within the layers of the structural section for days, weeks, and even months after it stops raining. There is a belief that aggregate base with 2 or even 5 % passing #200 sieve will provide ample drainage for most roadways. This is not the case because experience shows that these materials act as barriers (Cedergren, 1974).

“Beefing up” the design of a road or placing overlays may slow down the rate of damage but it does not eliminate the basic cause, which is free water. Removing water from the pavement section appears to be more desirable. Moreover, if the benefits of good drainage are to be fully realized, design standards need to be changed to provide good subsurface drainage systems for essentially every important pavement to be built (Cedergren, 1974). Figure 2.3 shows the potential life expectancies of pavements with and without good internal drainage (Cedergren, 1989).

Figure 2.3 Life Expectancies of Well Drained Pavements (Cedergren, 1989)

Subsurface drainage systems should be provided for all important highway pavement structural sections unless economic studies indicate that they are not cost-effective; i.e. either precipitation quantities and frequencies are so small or heavy wheel loads per day are limited. INDOT does not use subdrainage layers in special applications where the department cannot be assured that the edgedrain system will be maintained, such as on local projects, where a local agency would be responsible for later maintenance. Examples where subdrainage would not be used also include short sections, such as bridge approaches, where the adjacent, existing pavement does not have a drainage system (Hassan et al., 1995).

Indiana has recently had pavement failures attributed to poor drainage conditions. INDOT has concluded that an unmaintained drainage system can sometimes be worse than an undrained pavement. Therefore, maintenance of the drainage system is as important as providing the system (Hassan et al., 1995).

2.6 Design of Subsurface Drainage Systems

A key element of good drainage systems is a layer of highly permeable drainage material, protected with filters so it cannot become clogged by intrusion of adjacent fine materials (Cedergren, 1974).

Agencies have different practices regarding the drainage layer. For the most part, Iowa, Kentucky, Michigan, Minnesota, New Jersey, Pennsylvania, and Wisconsin use untreated base. Indiana, California, North Carolina, and West Virginia use treated base. An untreated base uses smaller aggregates than treated base to provide stability. The treated base uses a stabilizer (asphalt or cement) to provide stability, which results in more open materials. Thus, untreated bases generally have a lower coefficient of permeability than treated base. Although philosophies differ on the most desirable degree of permeability, agencies agree that rapid base drainage is extremely important. Therefore, a minimum coefficient of permeability of 300 m/day is suggested (USDOT, 1989).

When water flows from a fine grained soil into a coarse grained soil, there is tendency for the fine material to migrate into the coarse soil, and cause clogging and reduction of permeability of the coarse grained soil. This applies to the subgrade soil and the drainage

layer. Therefore, filters have to be placed to protect the drainage layer. Generally a layer of granular soil whose gradation satisfies established filter criteria could be used as a filter layer. Drainage fabrics (Geotextiles) could also be used. However, the choice is based on the history of performance (Moulton, 1980).

Generally the permeable base is placed 30 to 90 cm outside either pavement edge. Daylighting of the permeable base to the ditch slope was not a good idea, as vegetation and roadside debris eventually blind or plug the daylighted layer. Longitudinal edge drains are widely used, with a lateral outlet pipe generally at intervals of 91 to 182 m (USDOT, 1989). If not periodically inspected (and cleaned, if necessary) outlet pipes and subsequently the permeable layer can plug. Permeable bases do not require significant changes compared to conventional bases, and the results of using the permeable layers have been favorable.

The purpose of the design of a drainage layer is to reduce the period of exposure of structural sections to excess water. The design consists of using the subsurface drainage layer as conveyer of the water and analyzes probable inflow rates from all important sources.

2.6.1 Components of Drainage System

Feature of a typical drainage system would include the following (Cedergren, 1974 and Moulton, 1980):

- 1-highly permeable open graded base drainage layer.
- 2-filter layer or impervious membrane.
- 3-collector trench and perforated pipe.
- 4-outlet pipes
- 5-markers

2.6.2 Analysis and Design Criteria

Analysis of subdrain requirements is based on estimating the total inflow into the pavement, being mainly from the infiltration and the ground water (Q). Cedergren (1974) has recommended the estimation of the infiltration rate be based on the design precipitation rate. The design precipitation rate is the 1 hour/1 year frequency precipitation rate. He also

suggests multiplying the precipitation rate by a factor to account for the runoff. The factor was suggested at 0.3 to 0.5, and 0.5 to 0.67 for bituminous and concrete pavements, respectively. Moulton (1980) suggests a more detailed approach, which includes a joint/crack infiltration rate of $0.22 \text{ m}^3/\text{day}/\text{m}$ of joint/crack length in addition to moisture migration through the surface.

After estimating the amount of inflow, the drainage layer design 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 of zero depth at the outlet and of full thickness at the other edge, as shown in Figure 2.4. The layer must have a capacity of at least the inflow, so that the elevation of water does not exceed the thickness of the drainage layer. In the second stage, the rain is assumed to have ceased and the water in the section is freely flowing out of the drainage layer in an unsteady condition. The design criteria applies to the degree of drainage (% drainage), which is defined as the ratio between the volume of the water drained since the end of rainfall and the total storage capacity of the drainage layer (Huang, 1993).

Several criteria for the latter stage have been suggested. Casagrande and Shanon (1952) developed equations and charts for this criteria. They suggested a time no more than 10 days to achieve 50% drainage, as shown in Figure 2.5. AASHTO (1986) uses the time for 50% drainage, and classifies drainage into five categories: excellent, good, fair, poor, and very poor as shown in Table 2.1. Darter et al. (1981) found that the deformation of coarse grained materials increases dramatically when the level of saturation is over 85%. As a result, the recommendation was made that the time to reach 85% saturation should be less than 5 hours. Barksdale and Hicks (1977) suggest that 50% of the water should be removed in a time between 2 to 6 hours.

Figure 2.4 Phreatic Surface Assumption in Drainage Analysis.

Figure 2.5 Design Charts for Drainage (Moulton, 1980).

Table 2.1 AASHTO Drainage Criteria (AASHTO, 1986).

Quality of Drainage	Water Removed Within
Excellent	2 Hours
Good	1 Day
Fair	1 Week
Poor	1 Month
Very Poor	Water will not Drain

2.7 Analysis of Ground Water Flow

Modeling of ground water flow is a tool designed to simulate reality. Several types of models have been used to study groundwater flow systems. They can be divided into three broad categories: sand tank models, analog models, and mathematical models. The first one consists of a tank filled with sand and induced flow. The analog models use the analogy that exists in the differential equations between the electrical current through a resistive medium, or the flow of heat through a solid and the flow of water in porous media. Wired electric boards would be used based on the analogy that exists between Darcy's law for groundwater flow and Ohm's law for the flow of electricity (Wang and Anderson, 1982).

Mathematical models consist of a set of differential equations that govern the flow of groundwater. The reliability of predictions using a groundwater model depends on how well the model approximates the field situation. Two equations govern the flow of water in porous media; Darcy's law and the continuity equation (Wang and Anderson, 1982).

2.7.1 Hydraulic Properties of Soil

Two relationships are essential for the analysis of flow in unsaturated soils, the soil water characteristic function (suction $\psi(\theta)$ - water content(θ)); also called the soil water retention curve, and the hydraulic conductivity characteristic function ($K(\theta)$ - water content (θ)). Estimating the unsaturated hydraulic conductivity reliably is difficult, time consuming and expensive, therefore, several investigators have used models for calculating the

unsaturated conductivity, from the more easily measured soil-water retention curve (Van Genuchten, 1980).

Closed form analytical expressions for predicting the unsaturated hydraulic conductivity have also been developed. Brooks and Corey (1964), Van Genuchten (1980), Brutsaert (1966), Vauclin et al. (1979) proposed different retention curve equations as follows (Van Genuchten 1980, EL-Kadi 1985).

Brooks and Corey:

$$S_e = \left(\frac{\Psi}{PB} \right)^{-\lambda} \quad \text{for } \Psi \leq PB \quad (2.1)$$

$$S_e = 1.0 \quad \text{for } \Psi > PB \quad (2.2)$$

Brutsaert:

$$S_e = \frac{A}{A + |\Psi|^\beta} \quad \text{for } \Psi \leq 0 \quad (2.3)$$

$$S_e = 1.0 \quad \text{for } \Psi > 0 \quad (2.4)$$

Van Genuchten:

$$S_e = \left[\frac{1}{1 + (\alpha\Psi)^n} \right]^m \quad \text{for } \Psi \leq 0 \quad (2.5)$$

or

$$\theta = \theta_r + \frac{(\theta_s - \theta_r)}{[1 + (\alpha\Psi)^n]^m} \quad (2.6)$$

$$S_e = 1.0 \quad \text{for } \Psi > 0 \quad (2.7)$$

Vauclin et al.:

$$S_e = \frac{b}{b + [\ln \Psi]^c} \quad \text{for } \Psi \leq -1 \text{ cm.} \quad (2.8)$$

$$S_e = 1.0 \quad \text{for } \Psi > -1 \text{ cm.} \quad (2.9)$$

Where,

S_e = effective degree of saturation, defined as

$$S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} \quad (2.10)$$

Ψ = water suction pressure,

θ = volumetric water content,

θ_s = volumetric moisture content at saturation,

θ_r = residual moisture content,

PB = bubbling pressure (air entry pressure),

$\lambda, \beta, \alpha, n, m, a, b$ and c are fitting parameters.

Figure 2.6 shows a typical soil moisture retention curve. A drying and wetting hysteresis effect is shown in the figure.

Brooks and Corey uses a model given by Burdine (1953) to predict the unsaturated hydraulic conductivity. Van Genuchten uses a model proposed by Mualem (1976).

Burdine's Model:

$$K_r(S_e) = S_e^2 \frac{\int_0^{S_e} \frac{1}{\Psi(x)^2} dx}{\int_0^1 \frac{1}{\Psi(x)^2} dx} \quad (2.11)$$

Figure 2.6 Typical Water Retention Curve (ABAQUS, 1995).

Mualem's Model:

$$K_r(S_e) = S_e^{1/2} \frac{\left[\int_0^{S_e} \frac{1}{\Psi(x)} dx \right]^2}{\left[\int_0^1 \frac{1}{\Psi(x)} dx \right]} \quad (2.12)$$

Where K_r is the hydraulic conductivity as a function of S_e . Using the above two models, the Brooks and Corey model for unsaturated hydraulic conductivity is:

$$K_r(S_e) = S_e^{3 + \frac{2}{\lambda}} \quad (2.13)$$

and the Van Genuchten model is:

$$K_r(S_e) = S_e^{1/2} \left[1 - (1 - S_e^{1/m})^m \right]^2 \quad (2.14)$$

where $m = 1 - 1/n$, and n was defined as a fitting parameter in Van Genuchten model for the retention curve.

The solution for the soil moisture characteristic curves can be obtained when θ_r and θ_s are known. Saturated water content is simple to estimate once the soil phase diagram is solved. However, the residual moisture content is difficult to measure. Theoretically, the residual moisture content is where the gradient ($d\theta/d\Psi$) is equal to zero (excluding the region near θ_r). It is most of the time taken to be the moisture at very high suction pressure (-15,000 cm). The parameters (λ , PB) for the Brooks and Corey and (α , n) for Van Genuchten models, respectively, are obtained by fitting the model equation to the soil water retention data. From these, the hydraulic conductivity function is obtained.

El -Kadi (1985) conducted a study on 128 samples of sand, 145 samples of silt, and 175 samples of clay to compare the above described four water retention models. A good fit of the data was obtained with all the models. Some sensitivity was reported for the Brutsaert model, and it was shown that it cannot be used if there are a limited number of

values close to saturation. The Brooks and Corey model was shown to be insensitive to the frequency of the data points and measurements close to saturation.

Some points have to be considered when using the above models. First, θ_r is considered to be at the maximum tested suction pressure. The model equations only represent a least square fit to the measured data, and therefore only "explain" these measured data with a certain degree of accuracy. Although hysteresis exist between the drying and wetting phases, it was shown by Van Genuchten (1980) that hysteresis could also occur in the hydraulic conductivity function, although not necessarily.

ABAQUS (1995) provides a tabular form of input for the suction-degree of saturation relation. Therefore, no regression analysis would be needed to obtain any model parameters. ABAQUS assumes linear segments between each two input points. The unsaturated permeability is assumed by default as:

$$K_u = K_s (S^3) \quad (2.15)$$

Where,

K_u = unsaturated permeability,

K_s = saturated permeability,

S = degree of saturation

2.7.2 Governing Equations (Mathematical Modeling)

The following section presents the governing equations of flow in saturated and/or unsaturated porous media. The equations are based on a constant volume for the porous media.

2.7.2.1 Steady State Versus Transient Flow

The existence of a steady state flow condition means that the head at every point in the medium is independent of time (constant), while a transient flow means that the head at any point can vary with respect to time.

2.7.2.2 Darcy's Law

Darcy's law states that the flow rate is directly proportional to the head drop (difference) and cross section area of flow and is inversely proportional to the length of flow along which the head difference occurs (Wang and Anderson 1982, and Fetter 1989).

$$Q = - K A \frac{dh}{dl} \quad (2.16)$$

$$q = \frac{Q}{A} = - K \frac{dh}{dl} \quad (2.17)$$

$$V = \frac{Q}{n_e} A = \frac{K}{n_e} \frac{dh}{dl} dl \quad (2.18)$$

Where,

- Q = flow rate, cm³/sec,
- q = specific discharge or Darcian velocity, cm/sec,
- V = true velocity or average linear velocity, cm/sec; which is the velocity through the pore spaces,
- K = hydraulic conductivity, cm/sec,
- A = cross section perpendicular to the flow, cm²,
- dh/dl = head gradient.
- n_e = effective porosity.

The minus sign in equations 2.16 and 2.17 indicates that the flow occurs in the direction of head loss. In three dimensions, the general expression for Darcy's law becomes:

$$\begin{bmatrix} q_{x_1} \\ q_{x_2} \\ q_{x_3} \end{bmatrix} = \begin{bmatrix} K_{11} & K_{12} & K_{13} \\ K_{21} & K_{22} & K_{23} \\ K_{31} & K_{32} & K_{33} \end{bmatrix} \begin{bmatrix} \frac{\partial h}{\partial x_1} \\ \frac{\partial h}{\partial x_2} \\ \frac{\partial h}{\partial x_3} \end{bmatrix} \quad (2.19)$$

$$q_i = K_{ij} \frac{\partial h}{\partial x_j} \quad (2.20)$$

$$\mathbf{q} = -\mathbf{K} \nabla h \quad (2.21)$$

Where $\nabla = \partial/\partial x_i$, $\mathbf{q}$ and $\mathbf{K}$ and ∇ are the specific discharge (velocity) vector, the conductivity matrix, and the gradient operator, respectively, i and j are indices having values from 1 to 3. Equations 2.19 through 2.21 are the same, but written in different forms. Equation 2.20 is in index notation. The conductivity tensor could be explained by using an example shown in Figure 2.7. The figure represents a tilted shale bed. If the flow is assumed only parallel to the bedding, a head gradient in the x_2 direction will produce flow in the bedding direction which is decomposed into x_1 and x_2 directions. The same applies to a head gradient in the x_1 direction. Therefore:

$$\begin{aligned} q_{x_1} &= -K_{11} \frac{\partial h}{\partial x_1} - K_{12} \frac{\partial h}{\partial x_2} \\ q_{x_2} &= -K_{21} \frac{\partial h}{\partial x_1} - K_{22} \frac{\partial h}{\partial x_2} \end{aligned} \quad (2.22)$$

If the x_1' and x_2' are as shown in Figure 2.7, then K_{12} and K_{21} are both equal to zero because a gradient in either direction induces only a flow in its own direction. Therefore, the

Figure 2.7 Tilted Shale Bed (Wang and Anderson, 1982).

components of the hydraulic conductivity tensor in an anisotropic medium depends on the choice of the coordinate system (Wang and Anderson, 1982).

The head change in two dimensional flow is:

$$\Delta h = \frac{\partial h}{\partial x_1} \Delta x_1 + \frac{\partial h}{\partial x_2} \Delta x_2 \quad (2.23)$$

the equation of an equipotential line, which is a line of constant head, is:

$$\Delta h = 0 \quad (2.24)$$

Therefore the slope of the tangent ($\Delta x_2 / \Delta x_1$) is equal to $-(\partial h / \partial x_1) / (\partial h / \partial x_2)$ and the slope of the gradient is equal to the ratio of its components $(\partial h / \partial x_2) / (\partial h / \partial x_1)$. Consequently, the slope of the gradient vector times the direction of the contour line is -1. Therefore, the gradient direction is perpendicular to the tangent to the contour of constant head. Referring to Equation 2.20, the flow vector in general, is not in the direction of the gradient vector, because of the K_{ij} tensor, only in the case where K_{ij} reduces to K times the unity matrix (case of an isotropic medium) the flow vector will be in the direction of the gradient; i.e:

$$q_i = K \frac{\partial h}{\partial x_i} \quad (2.25)$$

Stated another way, the flow direction will be perpendicular to the contour of equipotential lines (Wang and Anderson, 1982).

If the medium is considered isotropic (hydraulic conductivity does not change at any point in all 360° directions), then the choice of the coordinate system has no effect, and the conductivity matrix reduces to the unity matrix multiplied by a constant (K).

2.7.2.3 Continuity Equation

The continuity equation is the same as for the conservation of mass. If the water is assumed incompressible (ρ is constant), the conservation principle applies to the volume. Figure 2.8 shows an infinitesimal cube, where the change in flow rate in the x_2 direction is

Figure 2.8 Infinitesimal Element of Flow (Fetter, 1989).

$(\partial q_{x_2} / \partial x_2) \Delta x_2 (\Delta x_1 \Delta y_2)$. For a saturated condition, the sum of the flow rate change in the three directions must be equal to zero:

$$\frac{\partial q_i}{\partial x_i} = 0 \quad (2.26)$$

Where (i) is a dummy index which corresponds to coordinate directions 1,2, and 3.

3.7.2.4 Equation of Flow

Combining Darcy's law with the continuity equation yields (Fetter, 1989):

$$\frac{\partial}{\partial x_1} \left(-K \frac{\partial h}{\partial x_1} \right) + \frac{\partial}{\partial x_2} \left(-K \frac{\partial h}{\partial x_2} \right) + \frac{\partial}{\partial x_3} \left(-K \frac{\partial h}{\partial x_3} \right) = 0 \quad (2.27)$$

In the case K is not dependant on the direction (not a function of x_1 , x_2 and x_3), the equation becomes:

$$\frac{\partial^2 h}{\partial x_1^2} + \frac{\partial^2 h}{\partial x_2^2} + \frac{\partial^2 h}{\partial x_3^2} = 0 \quad (2.28)$$

Which is the Laplace Equation (Fetter, 1989).

For unsaturated flow, the head at any point will be varying with time because of the variation in degree of saturation, and consequently the pressure head. The mass balance comes from the inflow, outflow, and variation in storage. The continuity equation and Darcy's law for saturated-unsaturated porous media, respectively are as follows (Neuman, 1973):

$$-\frac{\partial}{\partial x_i}(\rho v_i) = \frac{\partial}{\partial t}(\rho \phi S_w) \quad (2.29)$$

$$v_i = -K_{ij} K_r \frac{\partial h}{\partial x_i} \quad (2.30)$$

In which ρ = the density of the water; v_i = the Darcy velocity; ϕ = the porosity; S_w = the degree of volumetric saturation ($0 \leq S_w \leq 1$). K_{ij} = is the hydraulic conductivity tensor (which is assumed symmetric), K_r is the relative hydraulic conductivity ($0 \leq K_r(S) \leq 1$). Substituting equation 2.30 into 2.29, the equation of flow is:

$$\frac{\partial}{\partial x_i} \left(\rho K_{ij} K_r \frac{\partial h}{\partial x_j} \right) = \frac{\partial}{\partial t} (\rho \phi S_w) \quad (2.31)$$

Manipulation of the right hand side of this equation is carried out as follows:

$$\frac{\partial}{\partial t} (\rho \phi S_w) = \rho \left(S_w \frac{\partial \phi}{\partial t} + \phi \frac{\partial S_w}{\partial t} \right) \quad (2.32)$$

$$\frac{\partial \phi}{\partial t} = \frac{\partial \phi}{\partial \psi} \frac{\partial \psi}{\partial t} \quad (2.33)$$

$$\frac{\partial S_w}{\partial t} = \frac{\partial \Theta}{\partial t} = \frac{\partial \Theta}{\partial \psi} \frac{\partial \psi}{\partial t} \quad (2.34)$$

noting that $\Theta = V_w/V_T$; $S_w = V_w/V_v$; $\phi = V_v/V_T$; and $\Theta = S_w \phi$, $S_s = \rho g \phi (C_w + C_f)$; where the S_s is the specific storage and the C_w and C_f are the coefficient of compressibility of the water and the porous solid material, respectively. Neglecting the C_w term, and defining C_f as:

$$C_f = \frac{1}{\phi} \frac{\partial \phi}{\partial \psi} = \frac{1}{\rho g \phi} \frac{\partial \phi}{\partial h} \quad (2.35)$$

Equation 2.31 would change to:

$$\frac{\partial}{\partial x_i} \left(K_r K_{ij} \frac{\partial \psi}{\partial x_j} + K_r K_{i3} \right) = \left(C + \frac{\Theta}{\phi} S_s \right) \frac{\partial \psi}{\partial t} \quad (2.36)$$

Where $C = \partial \Theta / \partial \psi$, and is called the specific moisture capacity. It should be noted that the elevation head is only a function of x_3 .

2.7.2.5 Solution of the Equation of Flow in the Analysis Domain

As with any partial differential equation, the solution requires specification of initial conditions and boundary conditions which constrain the problem and make the solution unique. The solution can be carried out by means of finite difference or finite element methods. As the program which will be used is a finite element (FE) program, the following discussion will emphasize the FE method.

2.7.2.5.1 Finite Element Solution

The basic concept of the finite element method (FEM) is that the problem domain is discretized into small elements. The elements intersect at nodes. Each node has specific degrees of freedom according to the element type. In drainage analysis, there is only one degree of freedom, which is either the head or the pore water pressure. For each element the unknown field quantity, head (or pressure), is expressed in terms of the nodal heads (or pressure) by the use of shape functions (Equation 2.37). Figure 2.9 shows a linear triangular element. An approximate solution to the problem at any given instant of time, t , is obtained in the form:

$$\Psi(x, t) = \Psi_n(t) \xi_n \quad (2.37)$$

Where

Ψ = the field pressure head;

Ψ_n = the nodal pressure head;

ξ_n = shape functions; and $n = 1, 2, \dots, N$, where N is the total number of nodes.

The shape function is defined for the element and can be applied piecewise over the whole domain, i.e.:

$$\xi_n(x_i^m) = \delta_{nm}, \text{ for all } x_i^m \quad (2.38)$$

$$\xi_n(x_i) = 0 \text{ for all } x_i \text{ outside the elements surrounding the node.} \quad (2.39)$$

Network of Triangular Elements

Single Triangular Element

Figure 2.9 Triangular Element (Neuman, 1973)

Figure 2.9 shows a node with the surrounding elements. The shape function corresponding to this node would have a value larger than zero or equal to 1. The Galerkin method defines a function $L(\Psi)$ as (Neuman, 1973):

$$L(\psi) = \frac{\partial}{\partial x_i} \left(K_r K^s_{ij} \frac{\partial \psi}{\partial x_j} + K_r K^s_{i3} \right) - \left(C + \frac{\theta}{\phi} S_s \right) \frac{\partial \psi}{\partial t} = 0 \quad (2.40)$$

assuming that Ψ^N is the trial field; the Galerkin method states that:

$$\Omega_n(\Psi^N) = \int_R L(\Psi^N) \xi_n dR = 0 \quad (2.41)$$

for each value of n from 1 to N .

Where,

N = number of nodes;

R = the region or the domain.

By applying the Galerkin principle, a set of first order differential equations (differentiation is with respect to time) is obtained in the following matrix form:

$$A_{nm} \psi + F_{nm} \frac{\partial \psi_m}{\partial t} = Q_n - B_n; \quad n, m = 1, 2, \dots, N \quad (2.42)$$

$$A_{nm} = \sum_e K_{rl} K_{ij} \int_{R^e} \xi_l^e \frac{\partial \xi_n^e}{\partial x_i} \frac{\partial \xi_m^e}{\partial x_i} dR \quad (2.43)$$

$$F_{nn} = \sum_e \int_{R^e} \left(C_l \xi_l^e + \frac{S_s}{\phi} \theta_l \xi_l^e \right) \xi_n^e dR \quad (2.44)$$

The matrix A_{nm} is the global conductance matrix; the summation sign indicates a summation and expansion of the local conductance matrices of the element to the global matrix. The F_{nm} matrix represents the storage term due to transient flow, and is defined as follows: and $F_{nm} = 0$ where $n \neq m$. Q_n is a vector (one dimensional tensor) that represents consistent external flow. This means that the external flow on the face of an element is integrated to nodal flows:

$$Q_n = - \sum_e \int_{\Gamma} V \xi_n^e d\Gamma \quad (2.45)$$

Where,

Γ = the boundary of the domain;

V = is the flow on the boundary.

It should be noted that a numerical integration method is used to evaluate the matrices which form the system of equations. The most widely used numerical integration method is the Gauss Quadrature integration method (Cook et al, 1989).

2.7.2.5.2 Boundary Conditions

For seepage analysis in porous media, two boundary conditions are known; (a) head on the surface bounding the domain (Dirichlet conditions), (b) flow for surfaces bounding the flow region (Neuman conditions) (Wang and Anderson, 1982). The same problem could have a combination of both boundary conditions. It should be noted that only one boundary condition can be given at the same boundary or there would not be a unique solution.

Most software assume that the vector of boundary flow is initially zero unless other input is specified by the user (ABAQUS, 1995). Q_n at the n node will be zero in the following cases:

- All interior nodes, as the integration involves the shape function corresponding to nodes which have a value of zero anywhere outside the surrounding elements.
- All nodes on a no flow boundary.

2.7.2.5.3 Integration Over Time

To integrate the equations over time, the finite difference method is used. The time domain is discretized into a sequence of finite intervals, Δt , and the time derivatives of Ψ_n are replaced by finite differences. As in any time dependant problem, an initial value has to be known. Consequently, Equation 2.42 is written as (Neuman, 1973):

$$A_{nm} \Psi + F_{nm} \frac{\Psi_m^{t+\Delta t} - \Psi_m^t}{\Delta t} = Q_n - B_n; \quad n, m = 1, 2, \dots, N \quad (2.46)$$

The pressure head in the first term can be chosen in three ways and yield an unknown value of head at step $t + \Delta t$, where (wang and Anderson, 1982).

$$\Psi = (1-\alpha) \Psi^t + \alpha \Psi^{t+\Delta t} \quad (2.47)$$

- 1- Fully implicit solution ($\alpha = 1$).
- 2- Fully explicit solution ($\alpha = 0$).
- 3- Time-centered scheme (Crank-Nicolson approximation) ($\alpha = 1/2$).

In the three cases, the coefficient matrices can be evaluated at the beginning of the step or at a half step. It is evaluated at half step to dampen the tendency of (Ψ) to oscillate around its limit (Neuman, 1973). The latter is done by first interpolating the pressure heads at the half step according to:

$$\Psi_n^{k+1/2} = \Psi_n^k + \frac{\Delta t_k}{2\Delta t_{k-1}} (\Psi_n^k - \Psi_n^{k-1}) \quad (2.48)$$

where k represents the time $t = t_k$. It must also be noted that Equation 2.48 is a nonlinear equation; which means that it involves higher order terms of the unknown (Ψ). This will

happen if biquadratic or bicubic elements are used. In this case an iterative procedure will be required to solve the system at each time step. Iteration methods involve Newton or Modified Newton methods (Chen, 1988).

2.8 Frost Penetration

One of the most important factors affecting pavement performance is frost action. Frost action includes frost heave and loss of subgrade support during the frost melt period. The frost action term refers to all the effects of freezing temperatures upon pavement performance. If a subgrade is kept dry and adverse moisture conditions are kept to a minimum, the problem of frost action will also be minimized (Yoder and Witczak, 1970).

A frost susceptible soil, supply of water, and a slowly depressed air temperature or sustained period of subfreezing temperature of the soil have to exist all together for both frost heave and freezing of the soil to occur. A frost susceptible soil is defined by the Corps of Engineers (COE) as inorganic soils that contains greater than 3 percent by weight of particles finer than 0.02 mm. Soils are rated from F1 to F4 according to their frost susceptibility (Yoder and Witczak, 1970).

Water migrating in a soil with low permeability will have time to freeze. Permeability of soils has been correlated to the different soil characteristics, among them, the effective gradation size (D_{10}), the percent passing #200, and the density. The FHWA established a chart for estimating base course permeability using these parameters (Moulton, 1980). In general, a coarser graded soil, with high density and low percent fines, will have a higher permeability. Soil permeability is greatly reduced with a small increase of P_{200} . Silt is generally a frost susceptible soil. Silts possess a relatively high permeability compared to clays. This permits moisture migration. At the same time the pore sizes are small enough for substantial suction which leads to moisture retention. If penetrated by freezing temperatures, the retained moisture will freeze. On the other hand, granular soils (Coarse sands and gravel) have high permeability and low suction so that they drain readily and do not accumulate moisture that would freeze. Such soils are classified as non frost susceptible.

The importance of determining the frost penetration arises from the fact that this is the zone of the soil that is going to be affected by the frost action. In the COE design method several concepts are considered for the design of pavements that are affected by frost conditions. A full protection design involves replacing the subgrade by non frost susceptible layers for the full depth of possible frost penetration. The pavement can also be designed for reduced subgrade strength for the spring thaw condition. A limited frost penetration design allows limited frost penetration into the subgrade (Yoder and Witczak, 1970).

Soil temperature rather than the air temperature is the significant parameter in frost depth estimation. But generally the air temperature is used to estimate frost penetration. Several methods exist for estimating frost penetration in soils.

2.8.1 Thermal properties of the pavement

Properties that are important for the thermal analysis of pavement and soil are thermal conductivity (K), volumetric heat (C), and latent heat (L). Thermal conductivity (K) is defined as the rate of heat flow through a unit area under a unit thermal gradient. The units are Btu /hr/cm²/°F/cm. Because different materials have different values of K, the soils thermal conductivity depends primarily on dry density and water content. Volumetric heat (C) is defined as the change in thermal energy in a unit volume of soil per unit change in temperature. The units are Btu/cm³/°F. It is referred to as heat content, internal energy, stored heat or enthalpy. The following relationships are used:

$$C_u = 10^{-4} \gamma_d \left(3.7 + 22 \frac{w}{100} \right) \quad (2.49)$$

$$C_f = 10^{-4} \gamma_d \left(3.7 + \frac{11 w}{100} \right) \quad (2.50)$$

Where;

C_u = unfrozen volumetric heat, Btu/cm³/°F,

C_f = frozen volumetric heat, Btu/cm³/°F,

γ_d = dry density of the soil, in gm/cm³,

w = water content (weight basis).

Typical values for K and C are as shown in Table 2.2 (Aldrich, 1956).

Table 2.2 Thermal Properties of Common Materials

Material	Thermal Conductivity, K , Btu/hr/m/deg F.	Volumetric Heat, C , Btu/m ³ /deg F.
Air	0.045	0.67
Water	1.15	2203.6
Ice	4.26	988.8
Bituminous concrete	2.75	988.8
Portland Concrete	1.77	1059.4
Soil (typical)	3.28	1059.4

Latent heat (L) is the change in thermal energy in a unit volume of soil when the soil moisture freezes or thaws at constant temperature. The units are Btu /cm³. Latent heat depends only on the amount of water in a unit volume of soil. One gram of water gives off 0.316 Btu as it freezes.

$$L = 3.16 \times 10^{-3} w \gamma_d \quad (2.51)$$

Where,

L = latent heat, Btu/cm³,

γ_d = dry density, gm/cm³.

2.8.2 Frost Penetration Estimation

Several relationships have been developed to predict the depth of frost penetration. Among these methods are the Stefan equation, the modified Bergren equation, and the Corps

of Engineers empirical curve. The Stefan equation is based on the latent heat of fusion of soil and water.

$$Z = \sqrt{\frac{48 K F}{L}} \quad (2.52)$$

Where,

Z = Depth of penetration in a homogeneous mass, cm,

F = freezing index, degree ($^{\circ}\text{F}$) days.

A degree day (F) represents the difference of the mean daily air temperature from 32°F . Thus a positive value means the mean daily temperature is above air freezing temperature. A typical cumulative plot of degree days versus time is shown in Figure 2.10. The curve can be started at any particular day (horizontal axis). The maximum difference between peaks of the curve is the freezing index (Yoder and Witczak, 1975).

The Corps of Engineers method consists of using the freezing index together with an empirical curve to predict frost penetration. The method provides for a correction of the mean freezing index to an actual freezing index based on severity within a thirty year period. A regional map for the United States provide a distribution of the mean freezing index. Figure 2.11 shows this map.

Aldrich (1956) modified Stefan's equation by incorporating a dimensionless factor λ . The equation, designated as the modified Bergren formula, is:

$$Z = \lambda \sqrt{\frac{48 K F}{L}} \quad (2.53)$$

where λ is a dimensionless coefficient that can be obtained from a chart knowing two parameters α , and μ (Aldrich, 1956):

$$\alpha = \frac{v_o}{v_s} \quad (2.54)$$

Figure 2.10 Typical Degree day - Time Plot (Yoder and Witczak, 1975)

Figure 2.11 Regional Contour of Freezing Index in the United states (Yoder and Witczak, 1975)

$$\mu = \frac{C}{L} v_s \quad (2.55)$$

Where,

α = thermal ratio;

μ = fusion parameter;

v_o = difference between mean annual temperature and freezing point of soil moisture, degree F.

v_s = temperature by which effective surface temperature is less than freezing point of soil moisture during freezing period (F/t), in degree F.

t = duration of freezing period.

Aldrich (1956) presents a detailed procedure to estimate the frost penetration for stratified soil.

CHAPTER 3 - EQUIPMENT LITERATURE REVIEW AND SELECTION

This chapter presents a literature review on different instrumentation used in seasonal monitoring of flexible pavements. Selected instruments for use in this study are also presented. The instrumentation presented here are for monitoring moisture, temperature, frost penetration, rainfall and outflow from edge drains.

3.1 In-situ Measurement of Soil Water Content

Techniques utilized in this research for measuring soil water content include direct and indirect methods. In the direct method, the water content is measured. In one indirect method, the water content is obtained by measuring a property directly related to soil water content. The other indirect method involves inferring the soil water content from the suction measurement and the soil water retention characteristic curve.

Soil water is usually quantified according to one of the following definitions:

$$\theta_g = \frac{w_w}{w_s} \quad (3.1)$$

$$\theta_v = \frac{v_w}{v_T} \quad (3.2)$$

$$\theta_v = \theta_g \frac{\gamma_d}{\gamma_w} \quad (3.3)$$

Where,

- θ_g = weight basis water content;
- θ_v = volume basis water content (volumetric water content),
- w_w = weight of the water;
- w_s = dry soil weight;
- v_w = water volume;
- v_T = bulk sample volume.

These definitions are only for non shrinking or non swelling soil material. In case of shrinking/swelling soils the following definition is used (COE, 1989).

$$v_w = (1 + e) \theta_v \quad (3.4)$$

Where,

$$e = \frac{v_v}{v_s} \quad (3.5)$$

- e = void ratio;
- v_v = volume of voids;
- v_s = volume of solids.

3.1.1 Direct Methods

The direct gravimetric method of determining soil water content is destructive. A sample obtained from the soil mass is weighed wet and after drying. The difference in weight is divided by the dry weight. For a volume basis water content, the dry density of the soil sample has to be determined. The direct method is considered to be the most precise way to obtain water content. But it cannot be used for a long time study, because the same area cannot be sampled (COE, 1989).

3.1.2 Indirect Methods I

In the indirect methods of soil water content measurement based on a related property, soil water content is determined by measuring properties related to soil water

content. These methods include Neutron scattering, Gamma Attenuation, X-ray attenuation, Nuclear magnetic resonance, and Time Domain Reflectometry (TDR) (COE, 1989).

3.1.2.1 Neutron Scattering

Application of a neutron probe to measure water content utilizes the fact that fast neutrons are slowed or scattered if they impact hydrogen atoms. Soil water content measurements are made by inserting a radioactive source that emits neutrons in the soil. Some of the neutrons collide with the water hydrogen atoms. In the collision, a neutron is slowed or thermalized. Fast neutrons are also slowed when colliding with other atoms, but hydrogen is particularly efficient at slowing fast neutrons. The problem becomes one of detecting and counting the number of neutrons that are slowed and relating this number to the amount of water in the soil. The radioactive source is usually americium-241/beryllium. Above the source, there is a tube filled with a boron trifluoride gas. After some of the emitted neutrons collide with the hydrogen atoms in the soil they are thermalized. Thermalized neutrons passing through the gas collide with the boron trifluoride molecules, and alpha particles are released. Alpha particles are attracted to a negative high voltage electrode in the detector, and an electric pulse is counted by the logging instrument. An important note here is that only the thermalized neutron will produce the alpha particle and consequently the electric pulse. The higher the water content in the soil, the higher the number of thermalized neutrons.

Some soils contain hydrogen atoms not associated with the soil water, such as with organic soils. It is for this reason that the neutron probe should be calibrated for each soil in which it is utilized. Calibration involves determining the water content by gravimetric method and the neutron probe and relating the two by the following linear equation:

$$\theta_v = A + B (CT / SC) \quad (3.6)$$

Where CT and SC are the count rate and standard count, respectively, and A and B are calibration constants.

The standard count, SC, is obtained when the neutron probe is used in a standard shield provided with the instrument. Soil water content is measured in a spherical region surrounding the source and detector. The sphere has a radius of about 16 cm and about 70 cm for conditions near saturation and dry conditions, respectively. Average water content is determined within this region. Accuracy is not high when precise water content profiles are needed or when measurements are made near a layer interface. The neutron probe includes a radioactive source, which requires authorization and precautions.

Hydrogen atoms perform this thermalizing function whether the water is frozen or unfrozen. Thus, the neutron probe will provide a measure of the total water; frozen and unfrozen. In this study a permanent access tube was placed vertically in the test sections for use of the neutron probe. A Troxler Model 3332 Moisture depth gauge was used in this study. The gauge was borrowed from the Agronomy Department at Purdue University. Figure 3.1 shows the device and the access tubes.

3.1.2.2 Time Domain Reflectometry

Time Domain Reflectometry (TDR) is a relatively new method for measuring soil water content. The principle is based on the dielectric permittivity of soil components. A TDR unit consists of a pair of parallel steel rods (sometimes three). An electrically generated wave is pulsed through coaxial cables and through the steel rods. The steel rods act as a wave guide so that the wave is transmitted as a plane wave in the soil (the plane is formed by the rods). A zone of influence in the soil is a quasi-elliptical area formed by two 3-4 cm radius circles around the center of each rod. This cross section area of influence extends along the rod length. The zone of influence is affected by the rods length and spacing. The wave propagates to the ends of the rods and is reflected at their ends. For a rod of known length, the problem becomes that of detecting the travel time along the rod length, which is measured by the TDR unit. The material around the rods affect the wave speed of travel, as different materials have different dielectric constant. Dielectric constant of air, dry soil and water are approximately 1, 4, and 80, respectively.

Figure 3.1 Nuclear Gauge access Pipe

The TDR is a time measuring device; it measures the time of propagation of the wave along the coaxial cables as well as along the probe. The velocity of propagation as mentioned before is influenced by the dielectric medium (medium around the transmission line). The Tektronix 1502B has to be adjusted to a propagation velocity (generally 0.99). The propagation velocity is given as a percent of the velocity in vacuum. The wave propagation will transfer the horizontal axis from the time domain to the distance domain.

In Figure 3.2 which shows an idealized TDR signal output trace, the vertical axis represents the reflection coefficient (ρ); which is the ratio of the voltage applied to the cable to the voltage reflected back from the cable due to change in impedance. An initial increase in the reflection coefficient of the signal trace, at point A, is due to an increase in impedance from the TDR unit to the cable. The reflection coefficient, ρ , remains almost constant along the cable. Another increase occurs at the transition from the cable to the probe followed by a drop in the reflection coefficient (marking the first inflection point, point B). This latter drop is due to low impedance of the medium (soil and water). Another inflection point is shown as point C in the Figure which is due an open circuit at the end of the probe. The interest in this wave is in the part between the two inflection points; more precisely in the time or distance between them. The dielectric constant is defined as:

$$K = \left(\frac{C}{V}\right)^2 \quad (3.7)$$

Where,

C = the speed of light in a vacuum; 3×10^8 cm/sec,

V = speed of propagation in the medium.

The time of propagation is:

$$T = \frac{L}{V} = \frac{L_a}{V_a} \quad (3.8)$$

Where,

T = time of propagation,

L = actual length of the probe,

Figure 3.2 TDR Idealized Wave Form (Topp et al., 1980)

- V = Unknown actual velocity of propagation,
 L_a = apparent length of the probe,
 V_a = propagation velocity setting.

Therefore:

$$V = \frac{L V_a}{L_a} \quad (3.9)$$

Substituting back into Equation 3.7,

$$K = \left(\frac{C}{V}\right)^2 = \left(\frac{C L_a}{L V_a}\right)^2 = \left(\frac{L_a}{L}\right)^2 \quad (3.10)$$

as a result, the ratio of apparent length of the probe to the actual length is sufficient to quantify the dielectric constant, considering that V_a is 0.99 (approximately 1).

There are several advantages of the TDR method. One advantage is that it is insensitive to salinity and bulk density (Topp et al., 1980). Furthermore, Topp et al (1980) found that a single empirical calibration equation applies to a variety of soils. Also considering that the dielectric constant of a soil sample is a weighted measure of its phase components, the water content is a measure of the ratio of water volume to the whole sample volume.

The dielectric constant of ice is approximately 4, which is close to dry soil. Therefore, the TDR will not measure frozen water present in a soil. However, in recent work performed on frozen soils, Patterson and Smith (1980) and Stein and Kane (1983) used the TDR in combination with a neutron probe to measure both the frozen and unfrozen moisture content in soils. Figure 3.3 shows that concept. Studies done previously show that the dielectric constant of a dry soil does not vary significantly with density, texture, salt content or temperature between frequencies of 1 MHZ and 1 GHZ (Douglas, 1986).

Figure 3.3 Nuclear Gauge and TDR Readings (COE, 1989)

Dielectric constants of several materials are given in Table 3.1 (Baker, 1990, Topp et al., 1985, Rada et al., 1994). Propagation velocity in various materials expressed as a percentage of the speed of light in a vacuum are given in Table 3.2 (Tektronix, 1988).

The wave propagation velocity is always a number from 0 to 1; where 0.5 indicates that the electrical energy moves through the cable at half the speed of the light. The variables which affect the electrical response in soils, including dielectric constant, are texture, structure, soluble salts, water content, temperature, density, and excitation frequency. The excitation frequency is the most important variable. Topp et al. (1980) conducted tests on four soils of different textures ranging from sandy loam to fat clay with different percentage of organic material. Organic material, vermiculite, and glass beads were also tested. These soils and materials were chosen to cover chemical and physical extremes of pore sizes and particle properties.

Effect of texture and density on dielectric constant were studied together, where the density varied from 1 to 1.6 gm/cm³. The effect of temperature on the dielectric constant was considered first in one series in which the temperature was varied from 10 to 36 °C. Influence of salts was tested by injecting sodium chloride solution instead of water into one of the soils. The tests were cycled through changes in water content to determine if there were hysteresis in measurement of the dielectric constant. It was found that the dielectric constant is independent of soil density, texture, and salt content. Also, there was no significant temperature dependence (Topp et al., 1980).

Topp et al. (1980) found that a single calibration equation applies to a variety of soils. Other researchers established different calibration equations. Figure 3.4 shows some of these relationships. It is to be noted that the Wobschall curve is a theoretical curve. Campbell Scientific (1995) recommends using the calibration equation developed by Topp et al. (1980) or Ledieu et al. (1986). The Topp et al. equation is as follows:

$$\theta_v = -5.3 \times 10^{-2} + 2.9 \times 10^{-2}K - 5.5 \times 10^{-4}K^2 + 4.3 \times 10^{-6}K^3 \quad (3.11)$$

Another calibration developed by Ledieu et al. (1986) for a 15 cm probe, and adjusted by Campbell Scientific (1992) to suit a different probe length is:

Table 3.1 Dielectric Constant of Various Materials (Miller, 1982)

Dielectric Constants at 20° C	
Vacuum	1
Dry air at 1 atm	1.0006
Water	80
Carbon tetrachloride	2.24
Benzene	2.28
Castor oil	4.67
Methyl alcohol	33.1
Glass	4-7
Amber	2.65
Wax	2.25
Mica	2.5-7
Soils	2-5

Table 3.2 Propagation Speed for Some Materials (Tektronix, 1988)

Dielectric	Probable V_p
Jelly Filled	0.64
Polyethylene	0.66
PTFE (Teflon)	0.7
Pulp Insulation	0.72
Foam	0.78
Semi Solid PE	0.84
Air	0.99

Figure 3.4 TDR Calibration Equations (Topp et al., 1980)

$$\theta_v = \frac{0.1138}{V} - 0.1758 \quad (3.12)$$

The TDR system used in this study includes a Tektronix 1502B wave generator and a CR10 Campbell Scientific Datalogger, with interfaces for automatic operation, wave sampling, and analysis. The system includes a multiplexer capable of measuring up to 8 sensors in sequence. Figure 3.5 shows the system.

3.1.3 Indirect Method II

In the second indirect method, the water content is inferred based on the relationship between matric suction and water content. Several models have been developed to represent the relationship between matric suction and water content. Two frequently referenced models are the Brooks & Corey model and the Van Genuchten model (as shown in Chapter 2).

Negative pore-water (u_w) is generally referenced to the air pressure inside the pores (u_a). The matric suction is ($u_w - u_a$). Salts in the pore fluid give rise to a second component of suction; namely, the osmotic suction (ψ_o). The combination of the two components of suction is referred to as total suction. The osmotic component of suction is generally neglected, and since the pore air pressure is assumed atmospheric, the negative pore pressure is equal to the matric suction (COE, 1989).

Devices used for measuring suction include tensiometers, thermal conductivity sensors, and electric resistance blocks.

3.1.3.1 Tensiometer

A tensiometer consists of a porous ceramic cup connected to a pressure measuring device through a capillary tube. When the porous ceramic cup is placed in contact with a soil,

Figure 3.5 TDR system

suction equilibrium with the soil results. Tensiometers have to be serviced or flushed from time to time and kept saturated if not in use (COE, 1989).

3.1.3.2 Thermal Conductivity Sensors

Water suction is measured indirectly with a thermal conductivity sensor. It consists of a ceramic cup with a temperature sensing device and a miniature heater. When the ceramic cup is placed in the soil, the suction inside comes to equilibrium with the soil suction. A controlled amount of heat is generated at the center of the porous block, the temperature is measured at the same point after a fixed amount of time. The rate of heat dissipation is directly proportional to the water content in the cup. The anticipated heat will give rise to the temperature. With calibration, the sensor reading is related to the water content and therefore to water suction (COE, 1989).

3.1.3.5 Electric Resistance Blocks

Electrical resistance blocks consist of a plastic or gypsum porous matrix. When the block is placed in a soil, the matric suction changes to approach equilibrium with that of the soil. The change in matric suction is reflected in a change of resistivity in the block. The blocks have to be soaked for a period of one day before being placed in the soil. They contain a pair of electrodes and the electric resistance is measured between them. Gypsum blocks have no means for offsetting the influence of soil salinity. In other words they measure the total suction including the osmotic suction. Therefore, a new type, such as the Watermark block is made of perforated plastic tubes and contain a chemical buffer to offset the effect of salinity on resistance.

Electric resistance of the block is sensitive to the amount of water. Also, as water freezes its electric resistance rises due to the exclusion of the dissolved ions from the water. Therefore, the electric resistance of ice is very similar to distilled/deionized water (high resistance).

Based on the ease of use and previous experience (Ahmed et al, 1993), the Watermark 200 (Campbell Scientific 253) was selected for use in this study. The resistance between the

electrodes inside the block is R_s . Thompson and Armstrong (1987) calibrated this sensor and established an equation between the suction (bars) and the resistance ($K\Omega$). They used a pressure plate extractor apparatus and a temperature controlled water bath. The calibration was performed over the 0 to 1 bar range and was recommended for use in the 0 to 2 bar range (Campbell Scientific, 1993). Equation 3.13 gives the relation between suction and resistance inside the sensor. Figure 3.6(a) Shows Equation 3.13 plotted at 20°C. Figure 3.6(b) shows a picture of the sensor.

$$SWP = \frac{R_s}{0.01306 [1.062 (34.21 - T_s + 0.0106 T_s^2) - R_s]} \quad (3.13)$$

Where,

SWP = Soil Water Potential (kpa), 1 kpa = 0.01 bars,

R_s = Sensor Resistance ($K\Omega$),

T_s = Soil Temperature (C).

3.2 Temperature Measurement

High and low temperatures affect the response and performance of pavement layers and subgrade materials. For example, temperature is the major factor determining the stiffness of an asphalt bound layer. If a pavement layer or the subgrade freezes, then it is very stiff. Subsequently, with thaw, moisture stored in a frozen state changes to liquid and the layers, particularly unbound layers, loose substantial stiffness (COE, 1989).

3.2.1 Thermistors

Thermistors, THERmally sensitive resISTORS function on the fact that the resistance is inversely proportional to the temperature. There is roughly a variation of 500 ohms per 1°C

Figure 3.6(a) Watermark Block Calibration Equation (Campbell, 1992)

Figure 3.6(b) Watermark Block 200

at 0°C. Figure 3.7(a) shows such a relation. Figure 3.7(b) shows the air temperature sensor selected for use in this study.

3.2.2 Thermocouple

Thermocouples are simply electrical junctions of two wires made of dissimilar metal. They function based on "Seebeck-effect" voltages generated by the heating or cooling of a two-metal junction. In other words, the junction of the different metals generates a voltage based on the temperature at which the junction exists. The basic thermocouple measurement circuit must consist of a pair of thermocouple junctions, with the measured voltage being characteristic of the wire materials used and indicating the difference in temperature between the two junctions, or the difference in generated voltages at the two junctions. Therefore, one of the two junctions is a reference junction and is kept at a known temperature (0°C), while the other junction provides the temperature measurement of interest. The principal of thermocouple operations is shown in Figure 3.8(a).

3.2.3 Comparison Between Thermocouples and Thermistors

Although thermistors are accurate with reading error of 0.02 °C, sealing the sensor has been a problem. When the seal is broken, water infiltrates into the probe and a drift in the calibration is reported especially over the long term (more than 1-3 years). Drift of 4 °C has been recorded over long term measurements. The error in the reading of the thermocouple is less than 0.01°C (Campbell Scientific, 1994). For long term measurement, the COE, 1989 recommended use of thermocouples. As a result, thermocouples (105T) (Campbell, 1994) were selected for use in this study. Figure 3.8(b) shows this sensor.

3.3 Determination of the Frost Zones in the soil

Around 0°C the change in slope of the temperature gradient with depth in a pavement or subgrade can be small. Consequently, if the temperature is used as an indicator of freezing depth, a small change in temperature can represent a significant difference in depth. The other

Figure 3.7(a) Typical Thermistor Calibration (COE, 1989)

Figure 3.7(b) Air Temperature Thermistor

Figure 3.8(a) Thermocouple Concept of Operation
(COE, 1989)

Figure 3.8(b) Thermocouple Probe

problem is the effect that salts have on depressing the freezing temperature below zero. As a result, a reading of zero degrees does not have to correspond to the soil freezing.

More reliable methods have been developed based on the resistance of the soil, among which is the resistivity probe (RP). The idea is to measure the resistance between coils buried in contact with the soil. Reported values for water resistance are around 20,000 ohms, while ice as well as distilled/deionized water have resistances ranging from 100,000 ohms to 10^6 ohms (COE, 1989). Atkins (1990) reported the starting point for resistance of frozen water as 500,000 Ω . Electric conductivity of the water comes from the ions (the charge carriers). On the other hand, ions do not exist in deionized water, or become restricted in the case of ice, therefore, the resistance is high. Water is not normally deionized in soil. Bulk electrical resistance of soil depends on the soils porosity and degree of saturation and the electrical resistivity of the water. Because the electrical resistance of ice is very high compared to unfrozen pore water, the formation of ice crystals causes a decrease in the effective porosity and an increase in the bulk electrical resistivity. It is suggested that both temperature and resistivity probe measurements be made simultaneously to determine the freezing zone (COE, 1989).

A resistivity probe manufactured by ABF Manufacturing Company was used in this study together with the thermocouples as mentioned before. Figure 3.9 shows the resistivity probe tree. The tree consists of a 5 cm diameter PVC pipe grooved in the middle. Coils are wrapped around the pipe every 5 cm. It is important to maintain a constant spacing between consecutive rings, in order for measurements to be a true indicator of the soil resistance. Thirty- six coils at a spacing of 5 cm were applied to the probe that was a 182 cm long. Frost penetration is determined by making sequential resistance measurements down the length of the resistivity probe. Depth of frost penetration is indicated where the resistance goes from high to low. It is not necessary to report the actual value of the resistance, but instead, the voltage drop referenced to a fixed one megohm resistor located in the readout circuit could be reported. An interface was manufactured by Supple Electronics to automate the resistivity probe reading with the data acquisition system. The original schematics were obtained from the US Army Corps of Engineers Cold Region Research Engineering Laboratory (CRREL).

Figure 3.9(a) Resistivity Probe Tree

Figure 3.9(b) Rings and Connectors of the Resistivity Probe

Additional resistance steps were added to the design of the board (Supple Electronics, 1995), i.e, short circuit, 50 K Ω , 500 K Ω and open circuit. These four checks should determine the thawed (short to 50 K Ω), Transition (50 K Ω to 500 K Ω) and frozen (500 K Ω to open circuit) zones. These values were selected based on the literature (Atkins, 1990). The resistance (R) is defined as:

$$R = \rho L / A \quad (3.14)$$

Where,

ρ = the resistivity of the soil,

A = the surface area of the bare wire,

L = the spacing between the bare wires.

It is important to maintain good contact between the soil and probe. And to keep the probe below the surface (a depth of 6 inches is suggested). This will prevent water from infiltrating between the soil and the probe and giving misleading readings. The probe has to be supplied with an AC current. The reason is that the current is conducted in the soil by ionization and a DC current is one directional which would cause the dissolved minerals to react with one ring and polarize the electrodes, eventually affecting the results (Atkins, 1990). Figure 3.10 shows an example of temperature and resistivity tree readings.

3.4 Drainage Outflow and Rainfall Measurement

Both rainfall and drainage system outflow are measured with a tipping bucket mechanism. Figure 3.11 shows the outflow meter and the rain gauge used in this study. When the accumulated water inside either of two compartments reaches a certain weight (or volume), the bucket rotates due to the moment caused by the weight of the water. Each tip causes a switch to send a pulse to the data acquisition system. The number of switch pulses are counted by the datalogger. Once the bucket tips, the water is funneled out through the

Figure 3.10(a) Resistivity Probe Reading (COE, 1989)

Figure 3.10(b) Resistivity Probe Reading (COE, 1989)

Figure 3.11(a) Tipping Bucket in Outlet Flow Meter

Figure 3.11(b) Assembled Rain Gauge

base of the gauge. Calibration of these devices provide a given volume for each tip. Both the rain gauge and drainage flow meter were calibrated in the laboratory.

3.5 Positive pore pressure Measurement

Existence of positive pore water pressure is important in stress analysis of soils as well as the actual soil stability. Different devices are available to measure the static water pressure.

3.5.1 Pneumatic and Vibrating Wire piezometers

The pneumatic piezometer is a device which consists of a pneumatic system and gas supply, which comes to equilibrium with the water pressure. This type of system is not efficient as the mechanical parts may become clogged, also it is not suitable for automatic data acquisition system. Furthermore, it does not measure dynamic pressure. In the vibrating wire piezometer, the pressure is transmitted from the water to the device through a vibrating wire.

3.5.2 Electrical Piezometers

Pore water pressure in an electrical piezometer causes deflection of a diaphragm to which electrical strain gauges are attached. Resistance in the strain gauge changes as it elongates. There is a corresponding change in the measured output voltage from a wheatstone bridge network. This approach system has a rapid response to change in pressure. As a result, the electrical piezometer is suitable for dynamic measurements. Disadvantages of the gauge are that they are susceptible to freezing, barometric pressure changes and moisture penetration along cables. These probes were used in a previous study (Ahmed et al., 1993) in which there were reported problems. No pore water pressure gauges were used in the current study.

3.6 Main Operating Units

A number of sensors and components were available from the previous study (Ahmed et al., 1993). Equipment available included data acquisition units, four SM192 storage modules, and a CR10KB control keyboard. Three AM416 multiplexers, an SC32A optically

isolated interface and a notebook computer were added as part of this study. In addition to another CR10 unit for the TDR system, three deep cycle marine batteries of about 80 Amp hr capacity were purchased to provide power. Figure 3.12 shows the CR10 and CR10KB, SM192 storage module.

3.7 Instrumentation Case Study

3.7.1 LTPP Instrumentation (Delta Colorado, July 1993)

The Strategic Highway Research Program (SHRP) Long Term Pavement Performance study (LTPP) includes 64 sites instrumented to collect environmental data. These 64 sites were selected from 800 general pavement study (GPS) sites across the country, which covers four environmental zones (wet-freeze, wet-no freeze, dry freeze, dry no freeze). These sites include flexible and rigid pavements with different thicknesses and subgrade type (coarse vs. fine). Objectives of the environmental study are to evaluate and monitor the impact of seasonal moisture and temperature variations on pavement response by measuring the deflection by (FWD) and trying to relate it to the pavement response or stiffness. In other words, to determine the effect of environmental conditions on pavement life. The ultimate goal is to rationally use the acquired deflection data for evaluation, analysis and design. The instrumentation was done as deep as 2.1 m from the pavement surface, and consisted of the following sensors.

- 1- TDR (a 20 cm 3 rod sensor was used).
- 2- Resistivity Probes (36 circular metal at 5 cm spacing).
- 3- Thermistors
- 4- Observation well
- 5- Tipping Bucket Rain Gauge
- 6- Ambient Temperature.

The instrumentation is monitored twice per month during the spring thaw months, and once per month otherwise. As a demonstration, a test site in Colorado was selected on gently-rolling terrain approximately 5 miles south of Delta on westbound US 50. The site has

Figure 3.12 CR10 Datalogger and SM192 Storage Module

a clay subgrade with a high plasticity index ($PI > 20$). The subbase consists of 47.7 cm of river-bed material with big boulders (greater than 15 cm), and the base is 15 cm of crushed stone.

In the instrumentation installation, an 45.7 cm by 45.7 cm section of the pavement in the outside wheel path was sawn through to the top of the base course and removed. Similarly, a 10 cm wide strip of the lane and shoulder was sawn and removed. An auger was used to auger a 25.4 cm hole through the base and subgrade to a depth of 2.13 m below the top of the base. Large boulders in the subbase delayed the operation. Material was removed from the auger at 15 cm intervals and placed in 5-gallon plastic buckets.

Resistivity and thermistor probe trees were installed. Subsequently, TDR sensors were installed at 6 inch intervals. The retained subgrade material, subbase, and base was used as backfill and recompactd once the sensors were placed. Moisture content samples were obtained and tested in the field, at the time of excavation. The cables from all sensors were placed in a 6.3 cm diameter steel conduit and buried in a 10 cm trench leading to the equipment cabinet, which was located 9.1 m away from the instrument hole. Surface temperature probes were installed in the pre-grooved pavement surface and sealed. The asphalt block removed from the section was epoxy coated on all sides and placed back in its original hole and sealed with a self-leveling sealant. Resistivity data was measured only during the winter months. But the data was recorded twice daily during each visit.

The Asphalt pavement layer temperatures are recorded every hour. The subgrade temperatures are measured on a daily basis. Air temperature is measured at 15 min intervals and the hourly and daily mean, maximum, minimum are stored.

A ground water observation well is installed 30 m away from the instrumentation and penetrates 3.65 m below the surface. The well consists of a 1.9 cm galvanized steel pipe, with the bottom slotted and wrapped with a filter cloth to prevent migration of fines into the pipe.

Deflection data is being collected exclusively with the FWD. FWD tests are conducted in the outer wheel path and mid lane in intervals of 7.6 m for the length of the test section. Five additional locations are tested in the vicinity of the test pit. It takes about 1.5 hours to complete one round of deflection testing for flexible pavement. The testing sequence

is repeated a minimum of four times per test day. As a minimum the FWD tests are performed twice a month in the spring thaw period, and once a month otherwise.

Surface elevation, longitudinal and transverse elevation profiles are taken at the time the FWD tests are conducted to determine the subgrade volume change (due to frost heave and/or expansive soil).

3.7.2 Soil Moisture Monitoring under Pavement Structures Using TDR (Douglas, 1986)

Douglas (1986) utilized TDR sensors to demonstrate the TDR sensors, to gain a better understanding of the thaw weakening process and to monitor the unfrozen water content that exists throughout the year under typical roadways. The results of measured moisture contents was used to compare with non-destructive test results and pavement performance data.

In this study, four sites were instrumented with TDR and thermistors. In some sites probes were installed 5 cm apart vertically. Readings were taken approximately weekly during the summer, monthly during the winter and every two or three days during the spring thaw period. Moisture increased as thawing proceeded from the surface downward. If the soil moisture regime is constant, the TDR could be used to indicate freezing or thawing of the moisture by reduction or increase of the unfrozen moisture content. Baker and Davis (1982) found this technique to be useful for the determination of freezing and thawing fronts.

FWD tests were conducted at the three sites approximately weekly. Tests were conducted with a 4.5 ton dynamic load. Deflections were measured at the point of impact and at offsets of 20, 30, 45, 65, 90 and 120 cm. Deflections were corrected to account for variations in air temperature that existed at the time of measurement. The average moisture content in the upper 30 cm versus deflection for 4.5 ton load is shown in Figure 3.13. Unfortunately over the 21 days that FWD data were collected at each site, there were only small changes in the average moisture content in the upper 30 cm. However, among the three sites, the average moisture contents ranged from 10% to 36%. In general, these Figures shows that the structural strength of the roadway decreases as the moisture content increases.

Figure 3.13 Deflection-Moisture Results (Douglas, 1986)

However since the soils vary at each site, a single curve may not be appropriate. It would appear that the moisture content in the upper 30 cm controls the deflection magnitude.

Stubstad and Connor (1982) found that the maximum pavement damage may occur at shallow thaw depths, days or weeks before the total pavement deflection reaches a maximum, which agrees with the finding in this study (Douglas, 1982). Maximum or nearly maximum deflections occurred when the roadway was only partially thawed and this was very early during the spring breakup.

Two types of probes were used in this study, a straight probe and a circular probe. The straight probe was difficult to install due to its length, but gave good results. In contrast, the circular TDR probe was easier to install but exhibited some interferences, possibly caused by its geometry.

3.7.3 Minnesota Road Research Project (Mn Road, November, 1993)

The MnRoad Project is a long term project involving flexible and rigid pavements. The project uses a total of 4572 sensors for monitoring moisture, temperature and strains. The equipment includes TDRs, suction blocks, neutron probes, resistivity trees, temperature sensors, strain gauges, LVDTs and more. Sensors were installed up to 2.4 m into the subgrade from the pavement surface. Data is being collected and analyzed.

CHAPTER 4 EQUIPMENT INSTALLATION AND OPERATION

4.1 Section Configuration

After extended discussions with the Indiana Department of Transportation (INDOT), three candidate drainage section configurations were identified for inclusion in the study. Figure 4.1 shows the layer configuration for each section. Table 4.1 shows INDOT gradation and the asphalt content range for the materials. The same gradation are shown graphically in Figures 4.2, 4.3 and 4.4.

4.2 Site Selection

Several INDOT construction projects were considered as candidate test sites. Finally, a phase of I-469 at Fort Wayne was selected by INDOT for inclusion of the test sections. The pavement is a four-lane divided highway. The project was let in January 1995 and completed in October 1995. Location of the sections within the project was carefully considered. One point of consideration was to minimize changes in the original plans. The sections should be continuous without structures and extend between two outlet pipes and provide a section length of at least 167 m. It was desirable that the location be flat or have at most a moderate slope. Sections should be located to exclude interchanges so that the volume of traffic on each section did not vary.

A location was selected as shown in Figure 4.5 on the Project Map, between Brooks Road and Leo Road (Bridge). The sections lie between Stations 150 + 05 and 173 + 40. Since the same traffic volume was projected for both directions the East bound lane was selected for the test sections as the drainage pipe depth was constant. The section station limits are as shown in Table 4.2.

Figure 4.1 Test Section Configurations.

Table 4.1 INDOT Bituminous Mixture Gradation Specifications (INDOT, 1995)

Sieve Size, in (mm).	#11 Surf	#8 Binder	#9 Binder	#5 Bit. Base	#5C Bit. Base	#5D Bit. Base	#5 BCA	#2 Bit. Base	#53 Coars e agg.
21/2 (63)								100	
2 (50)								45-75	
11/2 (37.5)				100	100	100		30-60	100
1 (25)		100		80-99	70-98	80-99	85-100	20-50	80-100
3/4 (19)		80-89	100	67-90	50-85	68-90	60-90	15-40	70-90
1/2 (12.5)	100	56-80	70-92	42-74	28-62	54-76	30-65	10-35	55-80
3/8 (9.5)	85-98	43-68	50-76	33-60	15-50	45-67	15-50	15+5	--
No.4	62+5	35+5	40+5	30+5	15+5	40+5	0-20	3-20	35-60
No.8	31-62	14-40	18-45	12-34	3-20	20-45	0-15	2-15	25-50
No.16	17-50	8-32	10-36	7-28	2-15	12-36	--	1-10	--
No.30	8-37	5-24	6-26	4-22	1-10	7-28	--	0-7	12-30
No.50	3-25	2-16	2-18	1-16	0-7	3-18	--	0-6	--
No.100	0-14	0-10	0-11	0-10	0-6	1-12	--	0-4	--
No.200	0-4	0-4	0-4	0-4	0-4	0-5	0-5	--	5-10
Min % crushed	--	--	--	--	--	--	95	--	
% Bit.	5.5-7.0	4.1-5.2	4.5-5.4	4.0-5.1	3-4.5	4.0-5.1	2.5-4.0	2.5-3.5	
Max % moist.	0.3	0.5	0.5	0.5	0.5	0.5	--	0.5	

-- No specification given.

Figure 4.2 Indiana Surface Gradation Specification (INDOT, 1995)

Figure 4.3 Indiana Base Gradation Specification (INDOT, 1995)

Figure 4.4 Indiana Base Gradation Specification (INDOT, 1995)

Figure 4.5.a Location of the Test Section Site.

Figure 4.5.b Exact Location of the Test Sections.

Table 4.2 Test Section Stations and Actual Length.

Section #	Section			Drainage pipe (actual section)		
	From	To	Length	From	To	Length
1 (typical)	150 + 05	158 + 05	242m (800 ft)	150 + 15	157 + 95	236 m (780 ft)
2	158 + 05	166 + 05	242 m (800 ft)	158 + 15	165 + 95	236 m (780 ft)
3	166 + 05	173 + 40	223 m (735 ft)	166 + 13	173 + 40	220 m (727 ft)

Only one lane was to be instrumented. The outer lane, which would carry more trucks than the inner lane was selected for instrumentation. The pavement is designed with a centerline crown. A collector pipe is installed on each side of the pavement. The outlet pipe for the inside collector pipe runs under the pavement. Originally, "T" connections would have joined the outlet pipes of both lanes in each direction with one outlet. The "T" connection was changed to "L" type connection. A space of about 3 m was provided between the outside lane outlet pipe and the inner lane outlet pipe. Figure 4.6 shows the two outlets described above.

4.3 Test Pit Location

To minimize the effect on sensor readings of transitions from one section to another, the instrumentation hole was located a distance of 3 m from the lane outlet pipe (Outer Lane). As the TDR sensors measured about 30.48 cm, and the sensor has to be placed horizontally, the holes had to be larger than the sensor length. Consequently, a 40.64 cm hole was considered sufficient to allow for installation of the TDR sensors.

Figure 4.6 Outlet Pipe on One of the Test Sections

4.4 Installation Scenario

Original plans were to install all the probes after finishing the embankment (subgrade). Sensors to be installed in the pavement section were to be protected in a metal casing and excavated after paving. They then would be placed at the desired locations in the pavement. After discussions with all parties involved (Contractors and INDOT), it was agreed that this plan would be more difficult and likely delay project completion. Therefore, it was proposed that the installation would start after placement of the #9 binder layer. There would be a window of two weeks before the #11 surface was started. The plans were to install all the sensors up to the #9 binder. Only a temperature sensor for the surface would have to be excavated and located. The installation followed this plan.

4.4.1 Augering

After marking locations for the holes on the pavement, three 150 mm core samples were obtained for use in laboratory testing. The planned 40.64 cm auger hole allowed three 150 mm diameter cores to be taken from each drainage section. The 150 mm diameter core was the largest core possible for permeability testing. A 40.64 cm auger was provided and operated by TransTech Company. After the augering, the subgrade and asphalt were separated. The subgrade material was covered and left on the side of the hole until the sensors were installed.

4.4.2 Conduit Pipes

Instrumentation cables were routed through a 5 cm diameter steel pipe jacked into the side slope of the embankment until the end of the pipe was daylighted in the auger holes. The pipes ranged from 13.7 m to 16.7 m in length for the three sections. All the cables were stretched out and taped in groups. A snake was attached and taped thoroughly to the bundle of cables. The free end of the snake was inserted into the pipe end in the hole, and pushed through to the side slope. All the cables were labeled at both ends and in the middle. Additional adhesive tape was used to cover the labels, to protect them.

The instrumentation manual for the Watermark blocks states that they should be subjected to periods of drying and soaking in water and then installed in a saturated condition. As a result, the probes were submerged in a bucket of water which was lowered to the bottom of the hole. The probes were left in the water until they were installed. The probes were also subjected to several drying and wetting cycles in the laboratory prior to field installation.

4.4.3 Installation Tools

In planning the installation, several tools were manufactured at the Purdue University Machine Shop. The tools included 1.2 m and 1.8 m threaded steel pipes and a 150 mm round steel plate that could be attached to the pipe. Various lengths of the threaded rod were prepared. These assemblies were used for compaction. Another flat steel sheet with rounded edges was manufactured to lower the probes into the hole. Different lengths of steel pipes were used later with the same end plate for compaction. A jack hammer was also used for compaction. Later a Pogo stick was available and was used for compaction as well.

Five, five gallon buckets of each asphalt mixture was saved from the paving operation. A field oven, in a van and powered by a gasoline powered generator, was used to heat up the mix. An infrared gun was used to check the mixture temperature for compaction. An amount of material to patch the hole for each layer was weighed based on the densities of the cores. The material was weighed on a platform scale and put in buckets for heating in the oven. Several flat pans were also used.

4.4.4 Subgrade Sensor Installation

The resistivity probe consisted of 36 coils at 5 cm intervals with the exception that the top and bottom coils were spaced at 2.54 cm. The total length of the pipe was 182 cm. The neutron gauge access pipe was longer than the resistivity probe by a length equal to the pavement thickness in each section. The resistivity probe and the neutron probe access pipe were installed first and set at the required depths on the side of the hole, as shown in Figure 4.7. The targeted levels for all other sensors were previously determined.

Figure 4.7 Neutron Probe Access Pipe Positioning

A yard stick and a tape were used to check the depth, as the soil was placed in the hole and compacted. At the required depth, the sensors were lowered into the hole until resting on the bottom, as shown in Figures 4.8 and 4.9. The jack hammer with the steel pipe and plate were used for compaction, as shown in Figure 4.10. The manufactured pipes were joined together and inserted into the hammer. Lengths of pipe were changed according to the level attained.

4.4.5 Pavement Sensor Installation

After the subgrade and the #53 aggregate were compacted up to the pavement level in all three sections, the pavement layer thicknesses were determined and marked on the side of the hole. The problem of potential sensor damage and cable damage from the heat of heated asphalt mixture was countered by wrapping the cables with pipe insulation tape. The temperature and suction probes were installed in holes drilled in the side of the auger hole (Figure 4.11(a) and Figure 4.11(b)). This procedure could not be followed for the TDR sensor, as it was impossible to drill two small parallel holes almost for 30.48 cm depth. The TDR cables and connections were covered with insulation tape. Heated asphalt mixture was placed on the probes in the holes. Figure 4.12 shows the field oven setup for heating the asphalt mixture. Figure 4.13(a) and 4.13(b) show the placement and compaction of the hot mixture into the hole.

4.4.6 Sensor Locations

A small change in the targeted sensor location was accepted to facilitate installation. However, the actual location of the sensors was recorded and used in the data analysis. Figure 4.14, 4.15, and 4.16 show the actual sensor locations of the sensors. Sensors installed in each section included 11 thermocouples, 9 Watermark blocks, 8 TDR probes, one resistivity probe, and one neutron probe access pipe.

Figure 4.8 Positioning of Subgrade Probes

Figure 4.9 Installing Subgrade Probes

Figure 4.10 Compaction in The Deep Subgrade Layers

Figure 4.11(a) Drilling on The Side of The Instrumentation Hole

Figure 4.11(b) Positioning the Probes in Asphalt Layers

Figure 4.12 Field Oven Setup

Figure 4.13(a) Compaction of Asphalt Layer Using the Jack Hammer.

Figure 4.13(b) Asphalt Binder Laydown in the Hole

Figure 4.14 Sensor Locations in Section 1 (inches)

Figure 4.15 Sensor Locations in Section 2 (inches)

Figure 4.16 Sensor Locations in Section 3 (inches)

4.5 Cabinet Mounting

The soil surface was graded at the end of the outlet pipes, and concrete pads slightly larger than the data acquisition cabinet dimensions (1 m by 1m) were poured. Anchor bolts were fixed in the concrete base at specific locations. Steel angles were mounted on the sides of the box. After the concrete cured, the boxes were positioned and nuts fitted to the anchor bolts and tightly screwed down to hold the box on the base.

A rubber fitting was placed on the end of the outlet pipe and additional sections of plastic pipes were used to connect to the outflow meter inside the cabinet. The cable bundle was run through a PVC conduit which was slid along the cable and into the steel casing running through the embankment. This PVC conduit was cut off and attached to the cabinet. The excess cable length was wrapped on the cabinet walls.

Subsequently, the rain gauge was installed and the outflow meter was seated on the concrete base and connected to the plastic pipes coming from the outlet pipe. Silicone was applied to seal connections. The air temperature sensor was placed in a radiation shield, manufactured at the Purdue University Machine Shop, and attached on the exterior side of the cabinet. Figure 4.17 shows the instrumentation cabinet.

4.6 Equipment Connections, Wiring, and Operation

The thermocouples and Watermark blocks were connected to the AM416 multiplexer. One thermistor probe was placed inside the multiplexer and used as reference for the thermocouples. The air temperature probe was connected directly to the CR10 unit. The flow meter and the rain gauge used the 2 pulse channels on the CR10 unit. Power was provided by a 12 volt (80 amp hour capacity) deep cycle marine battery. Data acquisition was initiated in October 1995. Wiring and connections are described in detail in the instrument operation manual (Campbell, 1992). Data from all sensors is taken every 15 minutes. A program was written in Quick Basic to translate the data into an hourly basis. However the original format was also saved. For the selected sampling interval, generally the SM192 storage module has capacity for 30 days of data. The field visit interval was originally set at 15 days to allow for manual TDR and resistivity probe readings. This short interval also

Figure 4.17 Instrument Cabinet During Data Downloading

allowed a check on the operation of all equipment. After the malfunctions were solved, especially of the flow meter, the visit interval was increased to 30 days in the summer period.

4.7 Malfunctions and Equipment Problems

All of the sensors were tested after installation and were in proper working condition. Some problems were later encountered with the flow meter switch, which was found not to be waterproof. Water caused a short circuit. The switch was changed in February 1996 to a waterproof (NEMA rated 4 and 12) switch.

The TDR equipment picked up some interference at section 3 after several readings. The source of interference could not be identified. However, a simple filter was manufactured by Supple Electronics in an attempt solve the problem. Unfortunately, the filter was not effective. Use of the TDR in section 3 was discontinued. Moreover, in February, 1996, several TDR sensors in sections 1 and 2 showed an open circuit signal at the transition of the cable to the probe, which was supposed to be protected by a small Epoxy block. This type of malfunction was confirmed by a similar probe failure in the laboratory. The metal rods could be observed to move slightly at the connection point.

CHAPTER 5 - LABORATORY TESTING

For each test section undisturbed samples were obtained from the subgrade, and asphalt cores were obtained for each layer. Laboratory tests performed on these samples included soil classification on subgrade material and hydraulic conductivity on both subgrade and pavement materials. Also, the moisture retention test was performed on all materials in the pavement and subgrade. Unsoaked California bearing ratio (CBR) was performed on subgrade material. A calibration was performed on the subgrade, the filter, and the drainage base materials to obtain TDR calibration equations.

5.1 Subgrade Investigation

Subgrade samples were obtained after the subgrade was graded and compacted. The sampling location for the three test sections is shown in Figure 5.1. Five locations in section (1), Two locations in section (2) and three locations in section (3) were sampled, as shown in the figure. The selected sampling locations were close to the outlet pipe of the section and the instrumentation location. Prior to sampling, approximately 2.5 cm of the subgrade surface was removed. Subsequently, undisturbed and bulk disturbed samples were obtained.

Samples which were to be tested first were transported in plastic bags. Samples for which a delay in testing was expected were coated with wax and stored in wooden boxes (Hvorslev, 1949). Various tests were performed on the subgrade samples. Physical tests conducted included sieve analysis, Atterberg Limits, and specific gravity. Hydraulic characteristic tests performed included permeability tests and soil moisture retention curves. These tests and their results are described below.

Figure 5.1 Subgrade Sample Location

5.2 Asphalt Core Sampling

Cores were obtained after the #9 asphalt binder was completed. Three 150 mm diameter full depth cores were taken in each section. Figure 5.2 shows a core from each section. The different layers were marked on the cores and a concrete saw was used to cut the cores and separate the different layers. The aggregate used in the mix was limestone, and a slurry generally forms as a result of the water used to cool the diamond saw blade. It was observed that the slurry clogged the voids of the samples, especially the base #2 and base #5C mixtures. Therefore, the samples were flushed with a jet of water immediately after the cutting operation. The samples were flushed until the water was clear.

Two of the cores from each section were cut into layers for permeability and suction tests. Layers of #2 base and #5C base of one core from each section was cut in half along the longitudinal axis. Two grooves were cut in one of the two halves to recess the TDR probe. For the #2 base a section of another core was also cut in half and joined with the longer #2 base sample, to produce a sample 30.48 cm long for the TDR calibration.

5.3 Sieve Analysis and Material classification

Sieve analysis was performed on the subgrade samples according to ASTM D422. Atterberg limits tests (liquid limit (LL) and plastic limit (PL)) were performed according to ASTM D 4318. Specific gravity was determined according to ASTM D854. These data are given in Table 5.1. Using these data, the subgrade soils were classified according to the Unified Soil Classification System (USCS) (ASTM D2487). The classifications are also shown in Table 5.1. The gradation curves for the subgrade soil, #53 aggregate, and the #8 aggregate trench backfill material are shown in Figures 5.3 through 5.6. Table 5.2 shows the asphalt layer material properties.

Figure 5.2 Asphalt Field Cores

TABLE 5.1 Subgrade Soils

Section		1					2		3		
Location		1	2	3	4	5	1	2	1	2	3
Gradation, % passing (sieve sizes, in mm)	2.54	100	100	100	100	100	100	100	100	100	100
	1.9	100	100	100	100	100	100	100	100	100	100
	1.27	100	100	100	100	100	100	100	96.6	100	100
	0.95	100	100	100	100	96.7	99.3	99.3	94.9	99.2	99.1
	#4	99.8	99.8	99.9	96.9	95.5	98.7	98.7	92.7	97.8	98.8
	#8	98.8	99.3	99.4	95.4	91.8	97.3	97.3	85.2	97.1	97.8
	#16	97.3	98.4	98.6	93.9	88.6	95.3	95.3	78.9	96.1	97
	#50	92.9	96.5	94.7	86.8	76.8	88.9	88.9	59.7	93	93.9
	#100	88.6	92.2	89.8	77.9	62.6	81.6	91.6	39.1	89.4	90.3
	#200	84.3	87.6	85.6	70.9	47.6	75.3	75.3	25.9	85.8	86.4
Atterberg Limits	LL	39.6	48.6	33	42.1	40.6	33	35	NP	25	22
	PL	22.2	30	20.2	23.9	24.1	20.1	20.6	--	17.8	19.5
	PI	17.4	18.6	12.8	18.2	16.5	12.9	14.4	--	7.2	2.5
Classification		CL	ML	CL	CL	SC	CL	CL	SM	CL	ML
Specific Gravity		2.7	2.7	2.7	2.7	2.6	2.7	2.7	2.7	2.7	2.7
		9	5	7	0	8	6	6	2	7	8

Figure 5.3 Gradation Curves for Subgrade, Section 1

Figure 5.4 Gradation Curves for Subgrade, Section 2

Figure 5.5 Gradation Curves for Subgrade, Section 3

Figure 5.6 Gradation for #8 Trench Backfill

Table 5.2 Asphalt Material Physical Properties (Source: INDOT)

Material		#5C Base	#2 Base	# HMA	#11Surface	#8 Binder
Components		0.92 #5LS	0.6 #2 LS	#5 LS	#11 LS	#8 LS
		0.08	0.2 #5 LS	#24 Ns	#11 Slag	#24 NS
		#24NS	0.1 #11 LS		#24 NS	
			0.1 #24 NS			
Asphalt (%)		3.1%	2.3 %	4.3%	5.5 %	4.1 %
		AC20	MG10-30	AC20	AC20	AC20
Gradation , % passing (sieve sizes in mm	37.5	100	68.9	100	100	100
	25.4	88.5	39.3	95.4	100	100
	19	61.8	28.4	79.8	100	92.5
	12.7	36.7	20.3	67.5	100	68
	9.5	27.5	17.5			
	#4	16.3	11.7	45.0	62.5	35.0
	#8	12.4	9.7			
	#16	10.5	8.6			
	#30	8.9	7.4	17.5	26.5	15.5
	#50	6.4	5.5			
	#100	4.3	3.9			
	#200	3.4	3.1	3.5	4.0	3.0
Unit Weight		-- ^a	-- ^a	2.39	2.28	2.36
Void Ratio		-- ^a	-- ^a	4.1	6.1	6.0

^a Not available

5.4 Hydraulic Conductivity Test

5.4.1 Flexible Wall Permeameter

Based on expected ranges of permeability (ASTM D2487 and ASTM D5084), and the USCS classification performed, all the samples were expected to have permeability lower than 10^{-3} cm/sec, therefore, the permeability test was performed according to ASTM D 5084-90 "Measurement of Hydraulic Conductivity using Flexible Wall Permeameter". A permeameter with a 7.39 cm diameter platen was used for tests in this study. Figure 5.7 shows the permeameter.

The Permeameter consists of a control panel with vacuum lines (plastic tubing) to fill the burettes with water, three pressure lines, two platens (top and base), two porous stones, and a membrane. One pressure line (and regulator) connects to each platen. The third pressure line is used to apply pressure to the cell.

In preparation for testing, a soaked porous stone and a filter paper are placed on the base platen. The sample is then placed on the filter paper. Subsequently, in order, the remaining filter paper, porous stone and top platen are placed on top of the sample. Vacuum is applied to the membrane expander with the membrane installed. The membrane expander with the membrane is slipped around the sample in place, and the vacuum is released so that the membrane collapses on the sample. A layer of high vacuum grease is applied to the platens. O rings are placed over the membrane on the platens to seal and isolate the flow inside the sample from the cell water. The cell is then closed and filled with water to cover the sample. The two pressure lines connected to the platens inside the cell are connected to valves outside the cell. A vacuum is used to fill the burettes with water.

The cell pressure has to be higher than the top pressure (U_i) to ensure a positive effective vertical stress. Otherwise the membrane would be under tension and water from the cell would tend to leak by the membrane into the sample. Therefore, the pressure regulator for the sample is generally connected to the cell regulator so that any increase in the sample water pressure would automatically increase the cell pressure by the same amount.

Figure 5.7 Flexible Wall Permeameter

The cell pressure (confining pressure) is selected according to the effective stress state of the sample in the field. After the confining pressure is selected, the first stage of the test is back saturation, or the application of pressure on the top and base platens simultaneously to saturate the sample. One way of performing the test is to apply increments of 35,000 N/m² back pressure until a small change in the inflow, less than 0.1 ml, is observed. With this condition, the sample could be considered close to 100% saturation (Geotest, 1992). The second stage is the permeation stage, which proceeds by applying a difference in head on top and bottom of the sample.

The top pressure can be estimated from the following equation (Bowles, 1992).

$$U_i = \Delta h \gamma_w + U_o - \gamma_w L \quad (5.1)$$

Where,

- U_i = top pressure,
- U_o = base pressure,
- Δh = head difference,
- γ_w = unit weight of the water,
- L = flow path length (sample height).

This is only considered constant if the weight of the water in the burette is neglected. If the weight of the water is taken into account, the head should be considered a falling/rising head. For constant head, the permeability is estimated from the following equation (Bowles, 1992).

$$K = \frac{q L}{A \Delta h} \quad (5.2)$$

Where,

- K = permeability,
- q = flow,
- A = sample area,

Δh = head loss across the sample.

For the falling/rising head, the following equation (obtained after manipulation), with the same burette diameter for the bottom and the top, applies (Lambe, 1951).

$$K = \frac{a}{2} \frac{L}{A} (t_2 - t_1) \ln \frac{(\Delta p + H_1)}{(\Delta p + H_2)} \quad (5.3)$$

Where,

a = burette cross section area,

L = sample height,

A = sample cross section area,

t_1, t_2 = time increments,

H_1, H_2 = height of water in the burettes from a common datum,

Δp = pressure difference $U_i - U_o$.

5.4.2 Constant Head Permeameter

The constant head permeameter is simply a cell with an inlet connected to a reservoir and a free draining outlet. The constant head cell used is 20.32 cm in diameter. Figure 5.8 shows the cell connected to an inlet reservoir. Several points in the flow path are connected to manometers mounted on a panel. This cell is used for materials with large aggregate sizes.

5.4.3 Subgrade Hydraulic Conductivity

Based on soil classification, the subgrade permeability was expected to be less than 10^{-3} cm/sec. Therefore, the flexible wall permeameter was used for all subgrade tests. It is recommended (ASTM D5084, 1990) that a gradient of 30 not be exceeded for materials with a permeability of less than 10^{-7} cm/sec. The reason is that a high gradient will create a high effective stress at the bottom of the sample. Consequently, higher consolidation will occur at the bottom of the sample resulting in lower void ratio and lower permeability.

Figure 5.8 Constant Head Permeability Cell

A cell pressure of 35,000 N/m² and a gradient of 30 were used throughout the tests. In the permeation stage the inflow and outflow are measured at successive time periods. A steady state is achieved when the ratio of both flows is within 20% of each other (ASTM D 5084, 1990). The time to reach this stage depends on the degree of saturation at the end of the backsaturation stage. If a high degree of saturation is achieved at the backsaturation stage, then the time to terminate the test will be less. The permeability test results of the subgrade samples are shown in Figure 5.9 to 5.15. These figures clearly show when the inflow is close to the outflow. Permeability results for these tests based on constant head and rising/falling head are given in Table 5.3. It is shown that the difference is negligible between using Equation 5.2 And 5.3. The equation for constant head, Equation 5.2, is recommended for ease of computation. The test results compare well with the Bureau of Reclamation (Moulton, 1980) average values, except for sample 3.1 (SM). The reason could be that this sample had 49% passing the #200 sieve and could be easily (with a slight change in the sample) classified as an ML soil. Soils having permeability from 10^{-4} to 10^{-6} cm/sec are considered to drain poorly. Soils such as those in the test section subgrade with permeability below 10^{-6} cm/sec are considered practically impervious (Holtz et al., 1981).

5.4.4 Pavement Hydraulic Conductivity

The pavement layers were separated into two groups, the first group includes the #11 surface, #8 and #9 binder, and #5D base. As this group was expected to have a permeability less than 10^{-3} cm/sec, the samples were tested in the flexible wall permeability apparatus. The second group includes the #2, #5C, #53 and #8 coarse aggregate. This group was expected to have a permeability higher than 10^{-3} cm/sec, and therefore, samples were tested with the 20.32 cm constant head cell.

5.4.4.1 Flexible Wall Permeability

As the maximum size aggregate in the binder mixes and in the #5D base is 3.81 cm, 15 cm diameter triaxial cell was used to test these materials. As no permeameter panels were available, a triaxial control panel was modified for use in the tests. All tube connections,

Figure 5.9 Permeability Test for Sample 1.2

Figure 5.10 Permeability Test of Sample 1.3

Figure 5. 11 Permeability Test for Sample 1.4

Figure 5.12 Permeability Test for Sample 1.5

Figure 5.13. Permeability Test for Sample 1.2

Figure 5.14 Permeability Test for Sample 2.2

Figure 5.15 Permeability Test of Sample 3.1

TABLE 5.3 Hydraulic Conductivity of Subgrade Samples

Section	Location	USCS Soil Classification	Permeability (constant head), cm/sec	Permeability (Rising Falling), cm/sec	Bureau of Reclamation (FHWA, 1982) cm/sec
1	1	CL	---	---	7.73E-08
	2	ML	2.4E-08	3.7E-08	5.7E-07
	3	CL	3.9E-08	2.4E-08	7.73E-08
	4	CL	5.8E-08	7.4E-08	7.73E-08
	5	SC	3.9E-07	3.8E-07	2.89E-07
2	1	CL	2.4E-08	6.4E-08	7.73E-08
	2	CL	6.2E-08	7.5E-08	7.73E-08
3	1	SM	8.5E-08	7.1E-08	7.25E-06
	2	CL	---	---	7.73E-08
	3	ML	---	---	5.7E-07

* Intact samples could not be obtained.

control valves, pressure regulators and vacuum lines were changed to be able to conduct the permeability test. Figure 5.16 shows the cell and the modified panel for asphalt core testing. In thick layers (#5C, and #2 Base), the samples were identified as upper (U), medium (M), or lower (L). According to the location of the sample in the core.

5.4.4.2 Constant Head permeability

The cell used for determining constant head permeability was 20.32 cm, and all the cores are 15.24 cm. The gap between the sample and the cell was filled glazing putty to ensure that there was no leakage between the sample and the cell. The benefit of using this putty over other materials is that it is flexible and therefore easy to form. It also adhered well to both the sample and the cell.

Samples of the #53 coarse aggregate was compacted to target field density, using a Marshall compaction hammer. The #8 trench backfill was poured into the cell in a loose state similar to its field application.

5.4.4.3 Permeability Results for Pavement Layers

The tests performed on the pavement layers and the results are shown in Tables 5.4(a) and 5.4(b), respectively. It may be noted that for low permeability such as 10^{-4} cm/sec, comparison should be made on the order of magnitude and not the ratio. Based on this fact, the permeability for the #8 Binder, #9 Binder, and #2 Base are in agreement for the three test sections. The permeability of the #5C Base in section 2 is low compared with sections 1 and 3. Of all the materials, the #9 Binder has the lowest permeability ($9.5\text{E-}05$ cm/sec). Permeability of the #11 surface and #8 Binder agree reasonably well.

Permeability of the #5C is significantly higher than #2 (0.11, and 0.015 cm/sec, respectively). The #5D base has a much lower permeability than the #53 coarse aggregate ($1.4\text{E-}04$, 0.035 cm/sec, respectively). No trend is shown for change of density or permeability variation versus the location of the sample (upper, medium, or lower section in the core).

Figure 5.16 Modified Triaxial Panel and Cell

TABLE 5.4(a) Permeability Testing for Asphalt Cores

Layer	Section 1	Section 2	Section 3
#11 srface	2 Samples mixed in the laboratory, BSG based on SSD =97% of field density (142.4 pcf).		
#9binder	1	1	1
#8 binder	1	1	1
#5C base	1	1	3 (U, M, L)
#2 base	2 (U, L)	2 (U, L)	N/A
#5D base	2	N/A	N/A

U = upper portion of the core, M = medium portion of the core, L = lower portion of the core.

TABLE 5.4(b) Permeability Results for Pavement Layer

Layer	Section 1			Section 2			Section 3			INDOT Results		
	Density	Permeability	Density	Permeability	Density	Permeability	Density	Permeability	Average	Density	AC%	K
#11 surface		$\gamma = 2.21 \text{ gm/cm}^3 \text{ (SSD)}$		$K = 1.01\text{E-}04 \text{ cm/sec}$								
#9binder	1.789	8E-05	2.136	9.5E-05	1.995	1.11E-04			1.01E-04	N/A	N/A	N/A
#8 binder	2.11	1E-04	2.15	9.7E-05	2.21	1.34E-04			9.53E-05	N/A	N/A	N/A
#5C base	1.96	0.1297	2.16	0.027289	1.96	0.197154			1.103E-04	N/A	N/A	N/A
					1.95	0.127529			0.11889	2.25	3.3	0.025
					2.00	0.112797						7
#2 base	2.25	0.007213	2.28	0.012822	N/A	N/A			0.01597	N/A	N/A	N/A
	2.24	0.019319	2.24	0.00997								
	2.25 ^a	0.030511 ^a										
#5D base	2.33	1.28E-04	2.14	N/A	N/A	N/A			1.425E-04	2.32	4.2	1.93
		1.57E-04										E-04
#53 Coarse Agg.			$\gamma = 2.30 \text{ gm/cm}^3, K = 0.035573 \text{ cm/sec}$							2.02	N/A	0.023
#8 Trench Backfill			$\gamma = 1.26 \text{ gm/cm}^3, K = 1.179627 \text{ cm/sec}$							1.702	N/A	0.679

Density for asphalt samples, except #11 surface are based on a 15.24 cm diameter core physical volume measurement,

^aDifferent samples, N/A = not available.

Comparison of current test results with available INDOT results shows close agreement for #5D base and #53 coarse aggregate. There is a difference in permeability for the #8 aggregate, but the difference may be associated with the difference in density. The #5C base permeability is also different. The difference may be a result of a difference in density and the job mix formula.

5.5 Retention Curve

There are two ASTM standards, D 2325 and D 3152, for measuring the soil retention curve, for coarse and fine textured soils, respectively. As the suction pressure reaches very high value in testing fine textured soils (small particle and small pore size) compared to coarse grained soils (large particles and large pore size), the major difference between the two standards is the attainable maximum suction pressure. The test is conducted by placing an undisturbed sample of soil, 5 cm in diameter by 1 cm high on a ceramic plate. The plate and the sample are saturated. The porous plate maintains atmospheric pressure at the bottom of the sample while the desired pressure is applied at the top. This creates a gradient which forces water from the sample to flow downward. Pressure equilibrium is achieved at some point in time. The water content at that time is determined. This water content and pressure would represent one point on the retention curve. A series of tests are conducted with different pressures to provide the retention curve. Figure 5.17 shows the device and the samples tested.

Klute (1982) suggested that from 0 to 1.0 bar, the water is retained in the voids and therefore, the particle arrangement and density would affect the test results. It is important that samples to be tested in this range be carefully prepared so as not to affect particle arrangement. Therefore, in this range field samples are cut with the help of special tools. The tools consist of a cutting cylinder, rings in which to place the sample, and a small hammer. Three rings are placed inside the cylinder, and the cylinder is driven through the specimen by gentle tamping. Once the cylinder is fully driven into the specimen, it is retracted. Subsequently, the three rings are pushed out of the cylinder. Each ring is then cut flush at the surface, and the sample is then ready for testing.

Figure 5.17 Soil Moisture Suction apparatus

For the pressure range of 1.0 to 15 bar range loose disturbed samples were used. Figure 5.18 to 5.20 show the test results for the subgrade at locations 1.2, 2.1, and 3.1 (Figure 5.1), and for the asphalt core samples. Figure 5.21 shows the results of the test performed on the #53 aggregate and the #8 aggregate.

5.5.1 Generalized Moisture Retention curves

To facilitate future numerical analysis of pavement subdrainage, the hydraulic characteristics for the materials involved have to be known. In the case where the laboratory data is not available, reasonable assumptions have to be made. Assumption for the hydraulic conductivity can be made with a certain degree of confidence. However, the assumptions of the moisture suction characteristic is more difficult to make. A rational approach was followed to use the available laboratory data to help make the assumptions on the soil moisture characteristic curves. The source of the data is from the previous laboratory tests in this study as well as those developed by Ahmed et al. (1993).

The laboratory moisture suction data were regressed to obtain the Brooks and Corey Model with two parameters (PB and v). These two parameters were then correlated with soil characteristics. The soil characteristics considered were dry density, percent passing the #200 sieve, and gradation. In the latter characteristic the coefficient of uniformity (C_u), and coefficient of curvature (C_v) of the gradation curve were considered for coarse soils. The data was separated into sets for coarse and fine material conforming with the Unified Soil Classification System (USCS). Table 5.5 shows the results of the regression for the Brooks & Corey model obtained for the coarse soil. Table 5.6 shows the results for fine soils.

Several stepwise regression analyses were performed to determine the best model. As a starting point, a regression was performed on each individual variable and also with a second order polynomial regression. The regression with the lowest P-value was retained. The second step was to investigate whether an additional parameter should be added to the model to explain more of the variability. An F-test was made to determine if an additional parameter should be retained. P-values were compared at a level of significance (α) of 0.05 throughout the tests. The following equations were obtained for the coarse soil:

Figure 5.18 Subgrade Moisture Retention Curves

Figure 5.19 Moisture Retention Curves for Surface and Binder Layers

Figure 5.20 Moisture Retention Curves for Base Layers

Figure 5.21 Moisture Retention Curves for Filter and Trench Materials

Table 5.5 Brooks and Corey Regressed Parameters for Coarse Soil.

Soil Type	PB	ν	R^2
SM-SC*	52	3.1	0.929
SC*	68.5	3.18	0.724
SP-SM*	87	2.6	0.908
SP*	78	2.34	0.846
SW*	82	3.2	0.750
GW*	80	2.31	0.927
SM	95	2.9	0.72

*Source: Ahmed et al., 1993.

Table 5.6 Brooks and Corey Regressed Parameters for Fine Soil.

Soil Type	PB	ν	R^2
CH ^a	67.5	2.8	0.815
CL*	60	3	0.729
CL*	61.5	3	0.89
CL*	72	2.78	0.87
ML	50	3.3	0.86
CL	58	3.0	0.86

*Source: Ahmed et al., 1993.

^a P_{200} not available.

$$PB = 51.7 + 23.1 C_u - 3.6 C_u^2 \quad (5.4)$$

and,

$$v = 2.38 + 0.0185 P_{200} \quad (5.5)$$

Where P_{200} is the percentage passing the #200 sieve .

The same procedure was repeated for the fine soil, and the equations and results were as follows:

$$PB = 104 - 0.619 P_{200} \quad (5.6)$$

$$v = 1.95 + 0.0152 P_{200} \quad (5.7)$$

Figure 5.22 to 5.25 show graphically the above equations. The regression analysis is presented in Appendix A.

5.5.2 Moisture Retention Test Problems

In the soil moisture retention test, a soil sample is placed in a metal cell (pot) and subjected to an all around pressure. The sample is placed on a porous plate that will remain saturated under the applied pressure up to specific pressures for different plates (i.e. 1, 3, 5, and 15 bars). The plate is sealed all around and at the bottom with a membrane leaving only the upper surface exposed to the sample. The plate has a hole on the bottom with a tube extending outside the pressure pot. Consequently, when pressure is applied inside the pot, the bottom of the sample is exposed to the atmosphere. As a result, a gradient is established through the sample. Moisture flow will occur until equilibrium pressure between the applied pressure and the soil water suction inside the sample is attained. A certain amount of water will be retained in the voids and around the particles.

Figure 5.22 Bubbling Pressure for Coarse Soils

Figure 5.23 Pore Size Distribution Index for Coarse Soils

Figure 5.24 Bubbling Pressure for Fine Soils

Figure 5.25 Pore Size Distribution Index for Fine Soils

The main concern in this test is to establish and maintain contact between the specimen and porous plate. Otherwise an air gap will form. The air in this gap will develop a pressure equal to the applied pressure. This means that the desired pressure gradient will not develop at this gap location. The test results could be affected by the amount of entrapped air between the specimen and the porous plate. This may be more of a problem when testing asphalt samples because it is difficult to achieve a flat, sawn surface.

5.6 California Bearing Ratio (CBR) for the Subgrade Soil

The California Bearing Ratio (CBR) test was performed on subgrade samples closest to the instrumentation locations; samples at location 1.2, 2.1, 1.3, as shown in Figure 5.1. The test was performed according to ASTM D 1883, and the compaction of the samples was performed according to ASTM D 1557. Subgrade samples were compacted over a range of moisture content using a modified proctor hammer with 20 and 30 blows per layer on 5 layers. Therefore, two moisture density curves were obtained along with the corresponding CBR-moisture relations. The CBR was obtained only on Unsoaked samples. The reason for performing the test was to establish a relation between the subgrade strength and the moisture content. This relation may be of potential use for future work on this test site.

Figure 5.26 to 5.28 show the results of the CBR test at different moisture contents for three samples at 20 and 30 blows per layer. Compaction energy ($\text{kN}\cdot\text{m}/\text{m}^3$) for preparing the samples is shown in Table 5.7 and compared with standard and modified proctor compaction effort.

Table 5.7 CBR Test Compaction Effort

Current Tests		Standard Proctor	Modified Proctor
20 Blows	30 Blows		
978	1467	592	2693

Figure 5.26 Unit Weight for Sample 1.2

Figure 5.27 Unit Weight for Sample 2.1

Figure 5.28 Unit Weight for Sample 3.1

5.7 TDR Calibration

Reading with the TDR equipment were computed through various calibration equations. Check readings were also made with the TDR equipment in water, air, and methanol and were compared with the dielectric constants for these materials. Table 5.8 presents the results which show close agreement at slightly different temperatures.

Table 5.8 Comparison of TDR Reading with Literature

Material	Actual Reading (Temperature °C)	Miller, 1982 (Temperature: 20°C)
Water	81.4 (19)	80
Air	1.0016 (23.5)	1.006
Methanol	31.7 (21.2)	33.1

Various calibration equations have been published for calibrating the Time Domain Reflectometry (TDR) systems, as mentioned in Chapter 3. Two recommended equations (Campbell Scientific, 1992) are by Topp et al., and Ledieu et al. Figure 3.4 shows these two equations. The equations are based on regression analysis of results for different soils. In the current study, calibration was conducted on project field samples. TDR probes were also installed in asphalt layers (#2, #5C, and #5D). A review of the literature did not reveal applications or calibration for asphalt mixtures. Consequently, the probe was calibrated using samples of the asphalt layers in which probes were placed.

In the calibration process, different amounts of water are introduced to the specimen and the dielectric constant is measured. By conducting tests over a range of moisture content, a calibration curve can be obtained.

5.7.1 Subgrade TDR Calibration

The TDR probes used in this study were 30.48 cm long. A mold 45 cm by 10 cm by 10 cm was considered suitable for soil calibration tests. The soil was dried in an oven at 110°C overnight. After cooling to room temperature, water in a small amount was added in

a pan and mixed, as shown in Figure 5.29. The soil was compacted to the field dry density by compacting a predetermined weight of soil to a desired volume. A mark was made on the side of mold for the desired volume. Field nuclear gauge density measurements (average of 3 readings at each location for each value) are given in Table 5.9. Figure 5.30 shows the nuclear gauge being used in the field. Figure 5.31(a) and 5.31(b) show the calibration process.

Table 5.9 Nuclear Gauge Moisture Density Field Reading

Section	Station	γ_{dry} (gm/cc)*	γ_{wet} (gm/cc)	w/c, %
1	150 + 85	1.67	2.02	20.5
2	158 + 67	1.91	2.18	14.3
3	166 + 60	2.07	2.28	10.3

* targeted dry density

In the compaction process, one-half of the soil was placed in the mold and compacted with a Marshall hammer until the mark for one-half the volume was achieved. The probe was then placed, and the other one-half of the soil was placed on the probe, and compacted to the final volume. Measurements were made immediately. Seven readings with 128 wave form at each reading was taken. The use of 128 wave form would guarantee a minimum noise. The seven readings were made to check repeatability. After all the readings are taken, five water content samples were taken along the probe length. The water content was determined. Water content was consistent, suggesting that a uniform distribution of moisture was achieved during mixing. Figure 5.32 shows the calibration for the subgrade soil of the three sections. Figure 5.33 shows the calibration for the #53 aggregate. Most of the curves tend to be parallel to the Topp and Ledieu equations.

5.7.2 Asphalt TDR Calibration

A different procedure was used to calibrate the TDR for the asphalt mixture. Asphalt mixtures can not be handled in the same way as soils. Field cores of the asphalt layers were

Figure 5.29 Adding and Mixing Moisture for Subgrade TDR Calibration

Figure 5.30 Nuclear Density Gauge

Figure 5.31(a) TDR Calibration for Subgrade Soil

Figure 5.31(b) TDR Signal Trace During Calibration of Subgrade

Figure 5.32 TDR Calibration of Subgrade Soil

Figure 5.33 TDR Calibration of #53 Coarse Aggregate

either not long enough (smaller than 30.48) or only marginally long enough. For example, field cores of the #5D base were not long. Consequently, a laboratory sample was prepared by compacting a given weight of the mix from the field bag sample with a Marshall hammer to form a sample at least 30.48 cm long, and 15.2 cm in diameter. The sample was compacted in layers. Density achieved was 87 % of the field density.

The #2 and #5C bases could not be effectively compacted with the Marshall hammer because of the large maximum size aggregate in these mixtures. Field cores of the #5C base layers were slightly longer than the probe. Two sections of cores from the #2 base (22.59 and 7.62 cm) were taped together with duct tape to obtain the necessary total length. In all the asphalt samples, the sample was cut in two halves along the longitudinal axis, and two grooves were formed in one of the halves to place the probe in. The two halves of the sample were then taped together with the probe inside. Plastic bags were used to envelop the sample, and were water sealed with duct tape. The probe was inserted through the plastic bag into the formed grooves in the sample.

For testing, the samples were laid down on their side. Water was added through three holes in the plastic bags, on the top of the samples. Three readings instead of seven were taken at each water content level. The average of the three readings was used.

Calibrations for the #5D, #5C, and #2 bases are shown in Figure 5.34. For comparison, Topp (1985) and Ledieu (1986) are also shown. The curves fall below the Topp and Ledieu equations.

5.7.3 Regression Analysis

Regression analysis was performed on the calibration test data. The regression was performed by using both a linear function and a sixth order series function (i.e., the same form as the Topp and Ledieu equations). Figure 5.35 to 5.41 show plots of the regression equations as well as the R^2 value. Table 5.10 shows the obtained equations and the R^2 value.

Figure 5.34 TDR Calibration for Asphalt Samples

Figure 5.35 TDR Regression Equation for Subgrdae, Section 1

Figure 5.36 TDR Regression Equation for Subgrdae, Section 2

Figure 5.37 TDR Regression Equation for Subgrdae, Section 3

Figure 5.38 TDR Regression Equations for #53 Coarse Aggregate

Figure 5.39 TDR Regression Equations for #5D Base

Figure 5.40 TDR Regression Equations for #5C Base

Figure 5.41 TDR Regression Equations for #2 Base

Table 5.10 TDR Calibration Equations

Material	Model	Equation	R ²	R ² adj
Subgrade Section 1	Linear	$W_v = -0.128 + 0.087 \, 1/v_p$	0.98	0.98
	Power	$W_v = -0.9193 + 0.128 (1/v_p)^2 - 0.0045 (1/v_p)^4 + 5.6 \times 10^{-5} (1/v_p)^6$	0.99	0.97
Subgrade Section 2	Linear	$W_v = -0.0436 + 0.0839 \, 1/v_p$	0.98	0.98
	Power	$W_v = -0.026 + 0.038 (1/v_p)^2 - 0.0016 (1/v_p)^4 + 2.8 \times 10^{-5} (1/v_p)^6$	0.99	0.99
Subgrade Section 3	Linear	$W_v = -0.121 + 0.087 \, 1/v_p$	0.97	0.97
	Power	$W_v = 0.143 - 0.03 (1/v_p)^2 + 0.00446 (1/v_p)^4 - 0.00013 (1/v_p)^6$	0.98	0.97
#53 Coarse Aggregate	Linear	$W_v = -0.126 + 0.076 \, 1/v_p$	0.94	0.93
	Power	$W_v = -0.34 + 0.108 (1/v_p)^2 - 0.0082 (1/v_p)^4 + 0.000214 (1/v_p)^6$	0.98	0.97
#5D Base	Linear	$W_v = -0.257 + 0.1234 \, 1/v_p$	0.96	0.95
	Power	$W_v = -0.059 + 0.0074 (1/v_p)^2 - 0.00189 (1/v_p)^4 + 7.4 \times 10^{-5} (1/v_p)^6$	0.96	0.86
#5C Base	Linear	$W_v = -0.078 + 0.052 \, 1/v_p$	0.90	0.89
	Power	$W_v = -0.203 + 0.075 (1/v_p)^2 - 0.00633 (1/v_p)^4 + 0.000183 (1/v_p)^6$	0.98	0.98
#2 Base	Linear	$W_v = -0.0519 + 0.027 \, 1/v_p$	0.88	0.85
	Power	$W_v = -0.2404 + 0.09 (1/v_p)^2 - 0.01 (1/v_p)^4 + 0.000381 (1/v_p)^6$	0.98	0.96

CHAPTER 6. FIELD DATA RESULTS

Data collected from the instrumented test sections included moisture content, suction, resistance, air temperature, pavement and subgrade temperature, rainfall amount and duration events, and outflow volumes. The data collected from October 1995 until July 1996 are presented below.

6.1. Water Content

6.1.1. Field Results

Limited TDR moisture data was collected for section 3. An unidentified wave interference source prevented detection of the wave form. In addition, after February, 1996 the TDR probes showed a wave form indicating an open circuit. Therefore data is only presented until February, 1996.

Variation in volumetric moisture content (W_v) is shown in Figure 6.1 to 6.8. These figures show the moisture variation at a specific depth in the subgrade or in the specific layers for the time period from October 1995 to February, 1996. Figure 6.1 is for a depth of 58 cm from the top of the subgrade for the three sections. It may be noted that this represents a different depth from the surface for section 1 and sections 2 and 3. The moisture contents at this depth were low, approximately equal and show little variations for the period observed. Section 3 did have a lower moisture content on October 20. No freezing occurred at this depth.

Variation in moisture at a depth of 30.48 cm into the subgrade is shown in Figure 6.2. The variation can also be described as low, approximately equal and shows little variation.

Figure 6.1 Field Moisture at 58.4 cm in the Subgrade

Figure 6.2 Field Moisture at 30.48 cm in the Subgrade

Figure 6.3 Field Moisture at 15 cm in the Subgrade

Figure 6.4 Field Moisture at 7.6 cm in the Subgrade

Figure 6.5 Field Moisture at 2.5 cm in the Subgrade

Figure 6.6 Field Moisture in the Filter Layer

Figure 6.7 Field Moisture in the Base Layer

Figure 6.8 Field Moisture in the Top Base Layer

At a depth of 16.5 cm, as shown in Figure 6.3, the moisture content continued to be low and approximately equal. There was similar variation and a reduction starting in late January and in February. Freezing occurred in this period at this location. The moisture content decrease can be associated with freezing, as the TDR can only detect the unfrozen portion of the moisture content.

Similar observations can be made for depths of 7.6 cm and 2.5 cm into the subgrade, shown in Figure 6.4 and 6.5, respectively. However, in addition to the decrease in January and February, the moisture contents at these depths tend to be higher.

Moisture variation in the filter layer (#5D HMA and #53 coarse aggregate) is shown in Figure 6.6. Moisture in section 2 and 3 are low, approximately equal and vary in the same way. However, the filter for section 1 is almost dry for the full period.

In the respective base layers (#2 HMA in section 1 and 2 and #5C in section 3) the moisture shown in Figure 6.7 increased initially and then remained relatively constant. Overall, the moisture content was low and approximately equal and showed little variation. In the top base layer (#5C for all three sections) the moisture content was also low, with section 1 slightly higher than section 2, and section 3 closer to section 2.

The same field data is shown in Figure 6.9 to 6.16 in terms of degree of saturation (S). At 58 cm section 1 and 2 have close values, while section 3 is lower. At 30.48 cm depth, the saturation at the three sections is close and almost constant throughout the period. At 16.5, 7.6 and 2.5 cm depths, section 2, in general, showed the highest saturation and section 1 the lowest saturation. The moisture content of section 3 was between sections 1 and 2. In the filter material, as shown in Figure 6.14, the degrees of saturation in sections 2 and 3 are in close agreement with the same variations, while section 1 is almost dry. In the base layers of section 1 and 2, the saturation started at a low value, and increased to almost a constant level. The values of degree of saturation were close in the two sections. In the top base, section 1 had higher degree of saturation compared with section 2. The degree of saturation of section 3 appears to be close to section 2.

Figure 6.9 Field Degree of Saturation at 58.4 cm in the Subgrade

Figure 6.10 Field Degree of Saturation at 30.48 cm in the Subgrade

Figure 6.11 Field Degree of Saturation at 15 cm in the Subgrade

Figure 6.12 Field Degree of Saturation at 7.6 cm in the Subgrade

Figure 6.13 Field Degree of Saturation at 2.54 cm in the Subgrade

Figure 6.14 Field Degree of Saturation in the Filter Layer

Figure 6.15 Field Degree of Saturation in the Base Layer

Figure 6.16 Field Degree of Saturation in the Top Base Layer

6.2. Temperature

6.2.1. Field Results

Figure 6.17 to 6.19 shows for the three sections, the measured temperatures on the hottest days during the monitoring period from October, 1995 to July, 1996. The highest temperatures during this period occurred on October 18 for section 1 and 3 and on October 19 for section 2. The figures show that the subgrade temperatures are approximately equal up to 58 cm from the top of the subgrade. Above this depth the temperature varies with greater variation near the pavement surface. The variation is similar to the air temperature variation. The figures also show that the pavement temperature, as expected, reaches a higher temperature than the air temperature. The highest temperature in the pavement at 2 cm below the surface was 86.3°F, 87.9°F, and 83.9°F, in section 1, 2, and 3, respectively. The maximum air temperature was 76.1°F, 79.7°F, 76.2°F, in section 1, 2, and 3, respectively.

The coldest temperatures during the monitoring period occurred on February 3-4, 1996. Figure 6.20 to 6.22 show that event for the three sections. As expected, the pavement layers closer to the surface reach the lowest temperature, and follow the air temperature variation. In the subgrade, the difference is more than observed in the hottest temperature event. The rate of change of temperature in the pavement increases the closer the layer is to the surface. In contrast to the hot temperatures, the cold air temperature was always lower than the pavement surface temperature. The lowest air temperatures recorded were -10.45°F, -10.3°F, -10.35°F, in sections 1, 2, and 3, respectively. The lowest temperatures at 2 cm below the pavement surface were -2.43 °F, -1.89 °F, -3.36 °F, in section 1, 2, and 3, respectively.

6.2.2. Comparison With SHRP Equations

The highest and lowest pavement temperatures are critical in the Superpave binder grade selection method (Asphalt Institute, 1995). Three primary pavement distress types are considered when selecting the asphalt grade. These distress types are low temperature cracking, rutting (permanent deformation), and fatigue cracking. The asphalt grade is defined

Figure 6.17(a) Hottest Temperature in October, 1995, Section 1

Figure 6.17(b) Hottest Temperature in October, 1995, Section 1

Figure 6.18(a) Hottest Temperature in October, 1995, Section 2

Figure 6.18(b) Hottest Temperature in October, 1995, Section 2

Figure 6.19(a) Hottest Temperature in October, 1995, Section 3

OCT 17-18
Section 3

Figure 6.19(b) Hottest Temperature in October, 1995, Section 3

Feb 3-4, 96
Section 1

Figure 6.20(a) Coldest Temperature Event in February, 1996, Section 1

Feb 3-4, 96
Section I

Figure 6.20(b) Coldest Temperature Event in February, 1996, Section I

Feb 3-4, 96
Section 2

Figure 6.21 (a) Coldest Temperature Event in February, 1996, Section 2

Feb 3-4, 96
Section 2

Figure 6.21(b) Coldest Temperature Event in February, 1996, Section 2

Feb 3-4, 96
Section 3

Figure 6.22(a) Coldest Temperature Event in February, 1996, Section 3

Feb 3-4, 96
Section 3

Figure 6.22(b) Coldest Temperature Event in February, 1996, Section 3

by expected low and high temperatures. If the asphalt satisfies the specified criteria, it means that the asphalt is expected to exhibit acceptable performance.

The lowest temperature is the historical one day lowest pavement surface temperature. The highest temperature is taken at 20 mm below the surface and is the result of the historical highest seven-day average temperature. In the case of only the air temperature being available, empirical equations have been developed to estimate the highest and lowest temperatures to be considered in a certain region. The Asphalt Institute presents an equation to predict the high temperature at a 20 mm depth. The equation to predict the coldest pavement surface temperature was referenced by the Asphalt Institute as being developed by Canadian SHRP researchers. The equations are as follows:

$$T_{20mm} = (T_{air} - 0.00618Lat^2 + 0.2289Lat + 42.2)0.9545 - 17.78 \quad (6.1)$$

$$T_{surf} = 0.859 T_{air} + 1.7 \quad (6.2)$$

Where

- T_{20mm} = high pavement temperature design temperature at a depth of 20 mm,
- T_{surf} = historical minimum pavement surface temperature,
- T_{air} = T_{air} historical seven-day average high air temperature, °C, in Equation 6.1 and 1-day, minimum air temp, °C, in Equation 6.2,
- Lat = geographic latitude of the project in degrees.

Figure 6.23 and 6.24 shows a comparison of the minimum pavement, air and SHRP low temperature equation. The lowest monthly recorded events in the monitoring period are shown in the graphs. The SHRP equation should be expected to give temperatures between the 2 cm sensor and the air temperature. As is shown, this was achieved in all events, except in February where the temperature was higher than at the 2 cm sensor. The highest

Figure 6.23 Lowest Temperature of the Month (Nov, 1995 - Feb, 1996)

Figure 6.24 Comparison of Lowest Temperatures with SHRP Equation

difference between the estimated pavement surface temperature and the measured pavement temperature at the 2 cm sensor was 3.1°C , and occurred in January.

6.3. Frost Penetration

The COE method and modified Berggren formula to estimate the frost penetration are presented below. These methods were used to estimate the frost penetration. These methods provide a tool for comparison with the field results. Field data from the Watermark block, and the resistivity probes are also presented. The Watermark block data is shown here and evaluated for detecting frost protection.

6.3.1. COE Method

Frost penetration for the test sections was estimated using the Corps of Engineers method. Table 6.1 shows the estimated frost penetration using the COE method.

Table 6.1 Summary of COE Frost Penetration Estimation

Period	Freezing index F, deg. °F days.	Frost penetration (cm)
Mean of 30 years	346	68.5
Avg. 5 coldest in 30 years	691	82.6
Avg 3 coldest in 30 years	746	95.9
Coldest in 30 years	850	106.7

6.3.2. Modified Berggren Equation

As presented before, the modified Berggren equation takes into account the different layers in a pavement system and their thermal properties. Table 6.2 shows the assumed data to estimate the frost penetration. Table 6.3 shows the assumed thermal properties.

An iteration procedure was carried out and the frost penetration was found to be 86.3 cm, based on the an air freezing index of 850 degree (F) days.

6.3.3. Watermark Blocks

As Watermark blocks measure the resistance between two electrodes, the resistance is expected to increase when the water inside the block freezes. A test was conducted in the laboratory to observe the resistance changes with temperature and freezing.

Table 6.2 Assumption for Pavement Materials.

Material	Kg/m ²	Thickness (cm)	dry density (gm/cc)	Moisture content W%
Surface	66	2.5	2.36	0
Binder	200	8.9	2.13	0
Base 5C	200	7.5	2.13	5
Base 2	390	24.1	2.13	5
Separation 5D	242	10	2.13	5
Subgrade	66	varies	1.76	20

Table 6.3 Assumed Thermal Properties for the Use of the Modified Berggren Formula.

Material	K _u	K _f	K	C _u	C _f	C	L
Surface	2.6	2.6	2.6	1006	1006	1006	0
Binder	2.6	2.6	2.6	1006	1006	1006	0
Base 5C	3.3	3.3	3.3	934	934	934	32343
Base 2	3.3	3.3	3.3	934	934	934	32343
Separation 5D	3.3	3.3	3.3	934	934	934	32343
Subgrade	3.3	4.3	3.8	1463	1067	1269	113381

6.3.3.1. Watermark Block Laboratory Testing

The block was subjected to cycles of drying and wetting to fully saturate the porous matrix. After treatment, two blocks were placed in a water filled container with a thermocouple. A reference junction was established on the data logger, and another thermocouple was used to measure the air temperature. The resistance readings were carried through a freezing cycle followed by a thawing cycle. Figure 6.25 shows the freezing cycle, and Figure 6.26 shows the thawing cycle.

Data in Figure 6.25 shows the temperature dropping to 0°C. The temperature remained around 0°C for several hours, during which the water was partially frozen. The temperature then dropped below 0°C. The resistance started at 0.6 KΩ and gradually increased until it reached 500 KΩ, which is 830 times the value at the beginning of the test. Approximately the same values were obtained in the reverse, thawing cycle.

6.3.3.2 Field Measurements

Figure 6.27 to 6.29 show the measured resistance and temperature for the month of February at three different depths in the three sections. A dramatic increase in resistance is shown in each of these figures. It has to be noted that the resistance is inversely proportional to the temperature, but the maximum range in resistance change is 11 KΩ for a 32°C range at 1 bar of suction, while it can be less than 1 KΩ of resistance change for the same temperature range at 0.1 bar of suction. Therefore, as shown before in the laboratory, the large increase in resistance is due to freezing.

These figure also shows that the water temperature could drop significantly below 32°F and still not be frozen. This is because the time for a layer to lose latent heat will vary according to the thermal characteristics of individual layer and system layered characteristics. Therefore, the use of depressed temperature as an indication of the freezing front could be misleading.

As initial resistance before freezing will vary with depth, the maximum and minimum resistances at each location throughout the month were determined. The ratio of maximum to minimum resistance at all sensor locations is also determined. Two zones are identified

Figure 6.25 Freezing Cycle of Watermark Block

Figure 6.26 thawing cycle of Watermark Block

Figure 6.27 Resistance and Temperature in February, 1996, Section 1

Figure 6.28 Resistance and Temperature in February, 1996, Section 2

Figure 6.29 Resistance and Temperature in February 1996, Section 3

as shown in Figure 6.30, high and low ratio zones. The transition between the two zones (transition zone) is where the frost front is forming. A slope trend close to the transition zone on both the high and low sides is established and the intersection of these two slopes would be approximately the frost penetration depth. Figure 6.30 shows the transition zone for section 1. Following the same procedure, the maximum frost penetration for sections 1, 2, and 3, was found to be 94.7, 96.26, and 97.3 cm from the pavement surface, respectively.

6.3.4. Resistivity Probe Tree

Figure 6.31 shows the recorded resistivity values from the resistivity probe tree on February 3, 1996. The voltage measurement range was -2.5 to + 2.5 volts. Four different resistors were added to the circuit to obtain voltage values corresponding to a short circuit, 50 K Ω , 500 K Ω , and an open circuit reading. Thawed, transition, and frozen zones are represented in Figure 6.31 by four vertical lines. The recorded values for this date did not show the expected results. The resistivity tree was indicating no freezing. After investigation, it was concluded that there was a problem with the manufactured interface. Appropriate modifications are being taken to correct this problem in the second phase of this study.

6.3.5 Discussion

Figure 6.32 shows the degree days plot from October 1995 until June 30, 1996. The freezing index (FI) was found to be 804.6 degree (°F) days. Using the COE frost penetration chart (Yoder and Witczak, 1975), the frost penetration was estimated to be 101.6 cm. From this same chart, estimated frost penetrations are shown in Table 6.1 for the freezing index for the mean of 30 years, the average five and three coldest years, and the coldest year in 30 years. The measured frost penetration was 94 to 96.5 cm. Based on these results, the winter of 1995/1996 would lie among the three coldest years in thirty years.

Air temperature data for the Fort Wayne airport for a twenty nine year period (1966-1995) was obtained. The freezing index was estimated based on a monthly as well as a daily cumulative value. Yoder and Witczak (1975) explained the procedure by using a daily

Figure 6.30 Frost Penetration (February, 1996).

Figure 6.31 Frost Penetration by Using the Resistivity Probe

Figure 6.32 Freezing Index for 1995/1996 at the Test Site

accumulation of the difference between the air temperature and 32°F, the monthly method was recommended by Huang (1993). In the monthly method, the cumulative degree days for the month, which represent one point on the curve, is used, while for the daily method, a day by day summation is used to plot the curve. The values are shown in Table 6.4 and include the 1995/1996 measured freezing index. There is a difference in the results of the two methods. The daily method is more specific in identifying the inflection point. Figure 6.33 shows the frequency distribution for the last thirty years freezing index. Figure 6.34 shows the percentile distribution for the same data. According to this data, the 1995/1996 year would be the 79.3 % percentile value.

6.4. Rain and Outflow

Three rainfall events are presented. Data was lost on section one on the first event due to a continuous malfunction of the flowmeter at that section. The flow meters were modified by fixing 1.9 cm Teflon pad on the base where the ends of the tipping bucket strike. This pad establishes a higher starting position for the tipping bucket, which increases the moment arm of the water mass, and therefore allows the container to rotate at a lower water mass. The latter change caused an increase in sensitivity of the tipping bucket rotation. A recalibration was performed, and the flow meters functioned well after this modification.

The volume of water falling on the test sections was estimated by calculating an area with a length equal to the section length and width equal to the lane width (365 cm) plus a distance in the shoulder up to the trench side (60 cm). The intensity of rainfall was multiplied by the test section area to yield the rainfall volume, or the total amount of water falling on the pavement.

6.4.1. First Rainfall Event

The first rainfall event presented occurred on March 12, 1996. Figure 6.35 shows the event which consisted of two periods of rain during the day. A total of 11 cm of rain fell on the pavement. Measured outflow for sections 2, and 3 were 10,281.6 liter and 10495.7 liter, respectively. The ratio of outflow to rainfall volume was 0.092 and 0.10 for sections 2 and

Table 6.4 Freezing Index (Degree (°F) Day) for 1966-1996 (* From this project test site)

Year	Monthly	Daily
1966/1967	439	470
1967/1968	486	771
1968/1969	482	566
1969/1970	809	813
1970/1971	476	621
1971/1972	441	476
1972/1973	259	278
1973/1974	493	579
1974/1975	328	535
1975/1976	1183	1276
1976/1977	1239	1429
1977/1978	947	969
1978/1979	568	691
1979/1980	509	712
1980/1981	967	1072
1981/1982	23	158
1982/1983	757	853
1983/1984	591	675
1984/1985	697	741
1985/1986	253	305
1986/1987	489	524
1987/1988	284	352
1988/1989	465	473
1989/1990	217	337
1991/1992	97	185
1992/1993	320	371
1993/1994	596	718
1994/1995	327	357
1995/1996*	---	804

Figure 6.33 Freezing Index Distribution from 1966 to 1996 at Fort Wayne

Figure 6.34 Percentile Distribution for Freezing Index at Fort Wayne

3, respectively. This ratio is referred to in the literature as drainage efficiency. However, where seepage into the subgrade would be expected to be minimal, this ratio would more appropriately represent the pavement condition in combination with the drainage system efficiency.

Although the outflow volume of water from both sections was very close, section 3 is responding quicker to the inflow. This is indicated by the higher peaks of the outflow curve. Also, the outflow ends sooner in section 3, while it continues at a very small rate in section 2.

The difference in total time of flow between the two sections is small. The flow from section 3 continued for 37 hours after the second main rainfall event on the first rain period, and 18 hours after the last minor rainfall event of the same rain period. For the same section, on the second rain period, the flow continued for 39 hours after the rain ended. For section 2 the flow was similar to section 3, with slightly lower peaks.

6.4.3. Second Rainfall Event

Figure 6.36 shows the rainfall event which occurred on May 12 to 14, 1996. This event is a double rainfall event. The percentages of outflow to rainfall volume were 0.18, 0.15, 0.155, for sections 1, 2, and 3, respectively. The time to drain is only determined after the second main rainfall volume, which was on May 13 at 16:30 pm. The time to drain was 13.5, 47, and 47.25 hours for sections, 1, 2, and 3, respectively. Section 1 responded quicker and had the higher flow rate change compared with sections 2 and 3.

6.4.2. Third Rainfall Event

The second rainfall event shown occurred on May 15 and 16, 1996. Figure 6.37 shows the rain and outflow at all three sections. The total rain fall recorded was 1.61 cm. The percentage outflow to rain volume was 0.17, 0.189, and 0.19 for sections 1, 2 and 3, respectively. The time to drain was 10.5, 25.5, 25.5 hours after the end of rainfall, for section 1, 2, and 3, respectively. It is clear that section 1 had a quicker response time to the rainfall

Figure 6.35 Rainfall and Outflow Volume on March 12, 1996

Figure 6.36 Rainfall and Outflow Volume on May12, 1996

Figure 6.37 Rainfall and Outflow Volume on May 15 and 16, 1996

event. The rates of changes throughout the inflow and outflow stages are also higher in section 1 compared with section 2 and 3. The peak flow is also higher.

6.4.4. Summary of Rainfall and Outflow Results

Table 6.5 summarizes the three rainfall events. The time to drain presented in Table 6.5 corresponds to the time from the end of the rainfall until the outflow stops or becomes asymptotic.

Table 6.5 Summary of Rainfall Events

Event	Section	Section 1	Section 2	Section 3
1	Percentage Outflow volume to Rainfall volume	--- ^a	0.092	0.10
	Time to Drain	--- ^a	37/39 ^c	37/39 ^c
2	Percentage Outflow volume to Rainfall volume	0.17	0.189	0.19
	Time to Drain	10.5	25.5 ^b	25.5 ^b
3	Percentage Outflow volume to Rainfall volume	0.18	0.15	0.155
	Time to Drain	13.5	47	47.25

^aData not recorded due to flow meter malfunction.

^bFlow continues with another rainfall at that point.

Numbers correspond to time to drain at the first and second flow portions (1st/2nd).

CHAPTER 7 FINITE ELEMENT ANALYSIS

7.1 Model Geometry

Often migration of moisture in pavements has two gravity components; one in the cross slope direction and one in the longitudinal slope direction. As a result, internal pavement moisture migration is other than transverse to the centerline. In this analysis, however, a simplifying assumption was made to analyze the pavement in two dimensions, i.e. transverse to the centerline. Figure 7.1 shows the model used in the analysis. The model consists of a 3.65 m pavement lane with 60 cm of the shoulder. This shoulder width is the area covering the collector pipe trench. It was necessary to model only one-half of the pavement because of the crowned section.

7.2 Finite Element Mesh

ABAQUS (1995) offers only a quadrilateral element with eight nodes for the analysis that includes pore pressure as a degree of freedom. The eight node element used in the analysis features biquadratic displacement, bilinear pressure, and reduced integration. The reduced integration means that only four Gauss quadrature integration points are used. The element and mesh of the model are shown in Figure 7.2. General rules followed in the meshing process include using straight sided elements, increased mesh refinement where high flow gradients are expected, and preserving the smallest possible aspect ratio and skewness for the element.

Figure 7.1 Modeled Area of the Pavement

Figure 7.2 Finite Element Mesh

7.3 Boundary Conditions

Several boundary conditions were applied in the model. First, a no flow boundary was applied on all the outer edges of the analysis domain. Second, a constant zero pore pressure was used to simulate the outflow pipe. Third, the rainfall event was simulated by a constant pressure of zero on the top surface of the domain.

7.4 Convergence Criteria

Convergence of the solution is the decrease of the solution error through iterations. A solution is obtained when the error term is smaller than the convergence criteria. There are two main factors related to convergence problems. The first factor is how far from an equilibrium condition the analysis is started. If the analysis is started far from an equilibrium condition, the solution may encounter difficulty in converging. The second factor is the element size versus the time increment. The latter problem is encountered mostly at the beginning of an analysis step, where a sudden boundary condition is applied. Suggested criteria for the initial time step (ABAQUS, 1995):

$$\Delta t > \frac{\gamma}{6k} \frac{n}{k_s} \frac{ds}{du} (\Delta l)^2 \quad (7.1)$$

Where,

- Δt = initial time step,
- γ = specific weight of the wetting fluid,
- n = porosity,
- k = fully saturated permeability,
- k_s = permeability - saturation function ($k_s = 0$ to 1),
- ds/du = rate of change of saturation with respect to pore pressure,
- Δl = typical element dimension.

7.5 Analysis

Two Cases of analysis were performed. Case I and Case II considered the pavement as uncracked and cracked, respectively. With these two cases of analysis, a better understanding of the section potential performance would be obtained. Analysis of Case 1 includes two rainfall events, event one and three (Chapter 6). Case 2 is analyzed on rainfall event one.

The material hydraulic characteristics for defining the layer properties in the model were obtained from laboratory testing (Chapter 5). Initial moisture conditions were used as measured in the field, except for the subgrade soil, where the degree of saturation was increased to 0.9 to facilitate convergence. A period of about 28 hours was analyzed without the application of rain to achieve a steady state condition before including the rainfall event. Table 7.1 shows the equilibrium conditions in terms of degree of saturation after the equilibrium analysis period.

Table 7.1 Equilibrium Condition

Layer	Degree of Saturation		
	Section 1	Section 2	Section 3
Subgrade	0.9	0.9	0.9
Filter	0.9	0.4	0.4
Base	0.7	0.7	0.8
Top Base	0.8	0.8	0.8

7.5.1 Case 1 Analysis

As stated, Case 1 considers the pavement as uncracked. Rainfall event 1 and 3 (Chapter 6) are analyzed.

7.5.1.1 Rainfall Event One

7.5.1.1.1 Analysis

Rainfall event 1 (Chapter 6) was analyzed for section 2 and 3. The rainfall was modeled with the consideration that any intensity lower than 0.2 cm/hour is neglected (impervious boundary was assumed on the surface). Table 7.2 shows the rainfall intensity for the first rainfall event, and the time the constant head was applied on the pavement surface in the analysis, where I refers to impervious and 0 refers to the zero pore pressure on the surface.

Table 7.2 Rainfall Modeling

Time, hours	Rainfall, cm/hr	Modeled Pavement Surface Condition
1	0.2	I
2	0.48	I
3	0.37	0
4	0.19	0
5	0.15	0
6	0.37	0
7	0.95	0
8	0.87	0
9-18	< 0.2	I
19	0.9	0
20	2.47	0
21	1.23	I
22-72	< 0.2	I

Rainfall event 1 occurred on March 12, 1996. Section outflow and moisture changes in the trench, filter and base layer were predicted in the analysis. Moisture changes were monitored at six nodal points were in the analysis. These nodal points are shown in Figure 7.2.

Figure 7.3 shows the predicted and measured outflow for sections 2, and 3. Data for section 1 was not available because the outflow tipping bucket malfunctioned. Table 7.3 summarizes the total predicted outflow and measured outflow. Predicted results for sections 2 and 3 are shown to be close to the measured outflow..

Table 7.3 Comparison of Field and FEM Results (Case 1, Event 1).

	Field Outflow Volume (Field), liter	Finite Element Outflow Volume (FEM), liter	Ratio of FEM to Field
Section 2	6344	6505	1.025
Section 3	7198	6500	0.90

Figure 7.4 shows the pore pressure at the bottom of the trench. The pore pressure at this point indicates the water elevation in the trench. The water elevation in the trench follows the patterns of rainfall inflow and outflow. The two peaks for the rainfall event are clearly shown. The water reaches an elevation of 14 and 8 cm in the trench (above the pipe) at the two peaks of the curve, respectively.

Changes in the base layer saturation at location 4 (Figure 7.2) are shown in Figure 7.5. Even the small changes in saturation occur at the same time and have a similar shape to the rainfall event and the outflow curves.

7.5.1.1.2 Outlet Pipe Capacity

The area of the slots in the outlet pipe was measured from a sample pipe length collected during construction. The area was found to be $0.5 \text{ cm}^2/\text{cm}$ length of the pipe. The

Figure 7.3 Comparison of FEM Results and Field Measurements (Case 1)

Figure 7.4 Pore Water Pressure at the Bottom of the Trench (Case 1)

Figure 7.5 Change in Degree of Saturation at Point 4 (Case 1)

pipe was modeled by a tapered element that reduces to the slot area. A permeability had to be assigned to that element. The permeability was varied until the predicted and measured outflow were in agreement. This occurred with a permeability of 4×10^{-4} cm/sec. The area and permeability were used in the analysis of rainfall event 1, presented above.

7.5.1.2 Rainfall Event Three

Rainfall event 3 (Chapter 6) was analyzed to further validate the analysis and to compare the measured and predicted outflow for section 1. Table 7.4 shows the rainfall event and the corresponding pavement surface condition. The predicted outflow curve did not show the quicker measured response nor the lower drainage time for section 1. This result suggested that the different materials in the sections were not affecting the outflow response. After careful consideration, limited inlet collector pipe capacity was identified as a reason for the slower response and higher drainage time in sections 2 and 3. One difference between section 1 and sections 2 and 3 is the #5D HMA impermeable layer in section 1 versus the #53 limestone aggregate impermeable layer in sections 2 and 3. The #53 aggregate contains 5 to 10 % fines (passing the #200 sieve). These fines could migrate as a result of the layer drainage. If the fines migrate then they could contaminate the trench backfill material around the collector pipe. Another possibility is that the fines could constrict and therefore reduce the collector pipes inlet capacity. In either case, the result would be slowed drainage.

Pipe inlet capacity in the analysis of rainfall event 1 for section 2 and 3 was adjusted through the value of permeability for the tapered element at the bottom of the trench. A value was assigned to obtain good agreement between the predicted and measured outflow. As a result the pipe inlet capacity (tapered element permeability) for section 1 was increased two and four times that of sections 2 and 3. The results are shown in Figure 7.6. A four time increase of inlet capacity appears to provide the best agreement between predicted and measured outflow. Table 7.5 shows a comparison of the measured and predicted total volumes for this rainfall event. The predicted and measured outflow volumes are in good agreement. Figure 7.3, including the predicted outflow curve for section 1 is shown as Figure 7.7. The predicted total volume for section 1, event 1 was 6572 liters.

Table 7.4 Rainfall Modeling for Event Three.

Time, hours	Rainfall, cm/hr	Modeled Pavement Surface Condition
1.75	0.114	I
2.75	0.2794	0
3.75	0.6096	0
4.75	0.2921	0
5.75	0.3048	0
6.75	0.01	I
7	0	I
8 to 33	0	I

I = impervious.

0 = constant zero pore pressure.

Figure 7.6 Comparison of FEM Results and Field Measurements
(Event 3, Case 1)

Figure 7.7 Comparison of FEM Results and Field Measurements
(Event 1, Case 1)

Table 7.5 Comparison of Field and FEM Results (Case One, Event Three)

	Field Outflow Volume (Field), liter	Finite Element Outflow Volume (FEM), liter	Ratio of FEM to Field
Section 1	2722	3290	1.20
Section 2	3094	3260	1.05
Section 3	3521	3250	0.92

7.5.2 Case 2 Analysis

In this analysis, five cracks were assumed to exist in the pavement surface. The cracks were assumed to have propagated through the surface and binder layer. Rainfall event 1 was modeled. The cracks were assumed to extend to the bottom of the binder layer and were implemented by setting the pore pressure at the bottom of the binder layer equal to zero. This condition was imposed for the same time period as rainfall event on the pavement surface. The nodes at which cracks were superimposed are shown in Figure 7.8.

7.5.2.1. Analysis

Figures 7.9 and 7.10 shows the section 1 analysis. Results for section 2 are shown in Figure 7.11 to 7.14. In these figures, the dark area indicates full saturation and the light areas indicate partial saturation. It is obvious that the cracks are the major source of water infiltration. Figure 7.9 and 7.10 show water accumulation in section 1 inside the #5D and above the subgrade. Water accumulating in section 2 above the subgrade and inside the #53 aggregate is shown in Figure 7.13. Void ratios in the #5D mix and the #53 aggregate were estimated as 2 % and 38%, respectively. Therefore, although full saturation is reached in both layers, the #53 aggregate layer stores significant water compared to the #5D mix layer. The actual amount of water in these sections must be considered in terms of air voids.

Outflow for rainfall event 1, section 1, 2 and 3 are shown in Figure 7.15. This rainfall event totaled 9.8 cm. With the assumed cracks, predicted outflow from section 1 increased

Figure 7.8 Location of Simulated Cracks in the Pavement

Figure 7.9 Water Movement in Section 1 (Rainfall Event 1, Case 2)

Figure 7.10 Water Movement in Section 1 (Rainfall Event 1, Case 2)

7.12 Water Movement in Section 2 (Rainfall Event 1, Case 2)

Figure 7.13 Water Movement in Section 2 (Rainfall Event 1, Case 2)

Figure 7.15 FEM Outflow Results (Event 1, Case 2)

to 48308 liters. This amount of outflow is 7.35 times the outflow for rainfall event 1. The outflow was 0.44 of the total rainfall on the pavement. The predicted outflow for section 2 increased to 85778 liters or 13 times that of the original analysis for case 1. The outflow was 0.78 the total rainfall on the pavement. For section 3, the predicted outflow was 89763 liters, or 13.7 times that of the original analysis for case 1. The outflow to rainfall volume ratio was 0.78.

Table 7.6 presents the percentage outflow to rainfall measured for different pavement sections in Indiana (Ahmed, et al., 1993). As may be noted, in some cases, this percentage reached 70%. In one case the pavement had a PCI of 94.6% with an edge crack, and the ratio was about 30%.

Table 7.6 Outflow to Rainfall Volumes at Several Sites in Indiana (Ahmed et al, 1993).

Pavement Type	PCI	% outflow to rainfall	Section Location/County
Asphalt	36.8 Major Distress	50.64	SR-63 Vermillion
Asphalt	94.6 Edge Crack	29.31	SR-9 Noble
Concrete	96.6 Edge Crack	69.82	US-36 Hendricks

Figure 7.16 shows the pore water pressure at the bottom of the trench. The water elevation rises until the pavement becomes fully saturated. After the first half of the dual rainfall event, the water level drops and desaturation occurs until the second half of the rainfall event starts. Water level in the trench rises again with resaturation until the rainfall event is complete. Subsequently, desaturation occurs as the trench drains. The difference in maximum attainable head in sections 1 and 2 is due to the fact that the water accumulates on the subgrade surface on both sections, and the subgrade elevation is different.

Figure 7.16 Pore Pressure at the Bottom of the Trench
(Event 1, Case 2)

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

8.1. Conclusions

Based on laboratory determined material hydraulic characteristic, measured test section data, and finite element analysis, the following conclusions are made:

8.1.1. Subdrainage

- 1 - Surface infiltration was the main source of moisture entering the test sections. Therefore, the filter layer plays a key role in controlling moisture migration from the pavement into the subgrade. Field data indicated that the moisture content in the top 30 cm of the subgrade is higher in sections 2 and 3. These two sections have a #53 aggregate layer as a filter. Section 1 has a dense #5D HMA base. The average of the measured values of volumetric moisture content for section 2 was 0.036, 0.033, 0.089, and 0.077 higher than section 1 at the 30, 16.5, 7.6, 2.5 cm depth, respectively. In terms of degree of saturation, the degree of saturation for section 2 was 0.09, 0.13, 0.2, and 0.17 higher than section 1, at the 30, 16.5, 7.6, 2.5 cm, respectively. The limited data for section 3 indicates values of moisture content in between those of section 1 and 2.
- 2 - Void ratio of the #5D HMA filter layer was estimated at 2% based on interconnected voids only. As a result, it has limited storage capacity, and the measured moisture content was close to zero percent. Moisture content of the #53 filter in sections 2 and 3 was measured to be approximately 10%. Based on the limited data for section 3, there is less moisture in the impermeable layer in section 3 compared to section 2. This indicates limited moisture migration in section 3 probably because of the higher

permeabilities of the base and top base layers. This would explain its lower subgrade moisture content.

- 3 - Moisture content in the drainable base layers was lowest for the #5C HMA full depth base (section 3). On a relative basis, the #2 HMA base retained higher amounts of moisture than the #5C. The #5C HMA on top of the #2 HMA had a higher moisture content compared to #5C HMA full depth base.
- 4 - Section 1 showed a quicker response and less drainage time compared with section 2 and 3. This is due to the filter layer effect. Contamination of the trench material and/or the outlet pipe appears to have occurred from the fines in the #53 aggregate filter layer in sections 2 and 3. In section 1, the #5D is a dense HMA, and does not contain loose fines.
- 5 - The outflow pipe has limited capacity, even without the contamination. This trend was clearly shown in the finite element analysis of Case II. The reason is that the openings in the pipe are not large enough for the volume of water that can reach the pipe. The area was measured at $0.5 \text{ cm}^2/\text{cm}$ length of the pipe.
- 6 - In the current, new condition of the pavement, there is limited infiltration. Also, there is limited moisture storage in the pavement and subgrade. This is confirmed by the relative constant moisture and the finite element analysis of Case 1.
- 7 - When severe surface cracking was simulated, the pavement became saturated for the entire period of the rainfall. This further highlights the low inlet capacity of the pipe openings. Extended periods of full saturation will affect pavement performance.
- 8 - The ratio of outflow to the volume of rainfall has been referred to as efficiency. This is an incorrect definition. The ratio should be defined as index representing the

pavement surface condition (i.e. pavement surface condition index (PSCI)). On the other hand, the drainage system efficiency would be indicated by the time to drain, as defined by AASHTO as well as other agencies criteria.

- 9- The flexible wall permeability test was found to be very effective for dense asphalt cores. For high permeability drainage layer material, the rigid wall test apparatus is effective. In this test, the void around the perimeter between the sample and wall of the test cylinder is sealed with glazing putty.
- 10 - There are minor differences in the predicted low surface temperatures with the SHRP equation compared to measured test section temperatures.
- 11 - Frost penetration at the Fort Wayne test sections for the 1995-1996 winter was found to be 94 to 96.5 cm from the pavement surface. This results in a frost penetration into the subgrade of 46.4 cm for section 1, 33.6 cm for section 2, and 34.3 cm for section 3.
- 12 - The measured freezing index for the 1995-1996 winter was 804. An analysis of temperatures over the last twenty nine years at the Fort Wayne airport indicated that the severity of 1995-1996 winter was the 79th percent for the period. By comparing the measured freezing index with the predicted freezing index from the COE, the 1995-1996 winter would be among the three coldest winters in thirty years. Using the measured freezing index (804 degree (°F) days and the COE method, the predicted frost penetration would be 101.6 cm for the Fort Wayne area. Therefore, the measured frost penetration and freezing index compare well with the predicted values.

8.1.2 Instrumentation

- 1 - When functioning, the TDR is effective for water content determination. However, early failure occurred when the cable connections became loose. This appears to be a result of freezing.
- 2 - The suction blocks have a very low resolution for measuring water content. However, they were effective in detecting frost penetration.
- 3 - The thermocouples are effective and dependable.
- 4 - The battery used for a power supply has lasted more than 10 months without recharging.
- 5 - Initially the outflow meter performed poorly. After adding a waterproof switch and a 1.9 cm Teflon strike pad for the end of the bucket, the outflow meter performance improved substantially. The Teflon pad reduced the height of the bucket movement, reducing its momentum and increasing its measurement sensitivity.

8. 2. Recommendations

Based on the above conclusions, the following recommendations are made.

8.2.1 Subdrainage

- 1- For the early, good pavement condition, the major factor in the drainage of these sections was the filter layer type, not the drainage layer. This is because both the #2 HMA and the #5C have permeabilities high enough so that the controlling factor is the surface layer permeability. However, this may change as surface cracking develops.

- 2 - The use of a dense HMA filter is recommended as it reduces moisture migration into the subgrade.
- 3 - The use of full depth #5C HMA is recommended, as it retains less moisture than #2 HMA. Stripping would be less of a problem.
- 4 - If a policy is adopted to provide full subgrade frost protection, a depth of subgrade should be replaced by a non frost susceptible material equal to the frost penetration. If #5D HMA filter is used, the replaced material depth will be greater than if the #53 filter is used.
- 5 - For full frost protection, a different section configuration would then be recommended consisting of #5D HMA filter, #5C HMA full depth base, with a replacement of 46.5 cm of subgrade with a non frost susceptible soil.
- 6 - The collector pipe inlet capacity needs to be increased. Also, the trench and pipe contamination from materials along the flow path of the drainage needs to be addressed.
- 6 - The techniques used here for permeability testing are recommended for use in future testing of asphalt materials. The flexible wall permeameter is a good test apparatus for dense HMA.
- 7 - The analysis performed on the moisture retention laboratory data provide the necessary data to perform the numerical analysis, in the case where materials and/or laboratory testing is not available. The equations could be used to predict the Brooks and Corey model parameters. Either the model parameters or generated data from the model could be used as input for the numerical modeling depending on the software being used.

- 8 - Pavement Surface Condition index (PSCI) could be traced from The history of The outflow and rainfall measurements. More work needs to be done towards establishing a PSCI measurement system for drainage analysis.
- 9 - The structural performance of the base layers still remains to be evaluated. This will be done in the second phase of this study.

8.2.2 Instrumentation

- 1 - Thermocouples should be used for pavement temperature measurements.
- 2 - Light weight material such as aluminum or even plastic should be used to fabricate the outflow meter. Stainless steel is too heavy.
- 3 - Watermark blocks are effective for frost penetration detection. However they have poor resolution and are unreliable for moisture content determination.
- 4 - The TDR probe is not effective.
- 5 - Because of interface problems, the resistivity probes have not been evaluated.

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APPENDIX

APPENDIX A

MOISTURE RETENTION REGRESSION ANALYSIS

Table A.1(a) Regression Results for Coarse Soils

PB Equation	Coefficient	STDev	t-ratio	P
Constant	51.69	11.16	4.63	0.019
C_u	23.074	5.951	3.88	0.03
C_u^2	-3.595	0.72	-4.99	0.015

Table A.1(a) Regression Results for Coarse Soils

PB Equation	DF	SS	MS	F	P	R ²	R ² adj
Regression	2	1001	500.5	25.49	0.013	94.4	90.7
Error	3	58.91	19.64				
Total	5	1060					

Table A.1(c) Regression Results for Coarse Soils

v Equation	Coefficient	STDev	t-ratio	P
constant	2.38	0.084	28.18	0
P_{200}	0.0185	0.0032	5.69	0.005

Table A.1(d) Regression Results for Coarse Soils

v Equation	DF	SS	MS	F	P	R ²	R ² adj
Regression	1	0.634	0.634	32.43	0.005	0.89	0.863
Error	4	0.078	0.019				
Total	5	0.713					

Table A.2(a) Regression Results for Fine Soils

PB Equation	Coefficient	STDev	t-ratio	P
constant	104	12.51	8.28	0.004
P ₂₀₀	-0.619	0.1772	-3.50	0.04

Table A.2(b) Regression Results for Fine Soils

PB Equation	DF	SS	MS	F	P	R ²	R ² adj
Regression	1	200.5	200.5	12.22	0.04	0.803	0.737
Error	3	49.25	16.42				
Total	4	249.8					

Table A.2(c) Regression Results for Fine Soils

v Equation	Coefficient	STDev	t-ratio	P
constant	1.95	0.227	8.61	0.003
P ₂₀₀	0.0152	0.0032	4.72	0.018

Table A.2 (d) Regression Results for Fine Soils

v Equation	DF	SS	MS	F	P	R ²	R ² adj
Regression	1	0.120	0.120	22.38	0.018	0.882	0.842
Error	3	0.016	0.005				
Total	4	0.137					

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