ABSTRACT

Base course permeability plays an important role in many pavement failures which are subjected to moisture-related problems. The Mack-Blackwell National Rural Transportation Study Center and the Arkansas Highway and Transportation Department (AHTD) sponsored the 'Permeability of Pavement Base Course' research to measure the permeability of some granular base course materials. Limestone, sandstone, igneous rock, and Razorrock chert were the materials tested.

The permeameter, which was obtained from the U.S. Bureau of Reclamation, was used during the permeability test to contain a 19" diameter by 9" thick base course specimen. A laboratory procedure was developed to build the specimen in 3 layers by using a mechanical compactor. The AHTD Class-7 base course gradation was used to construct the base course specimens. Specimens with 3%, 6.5%, and 10% fines were tested to identify the change in permeability due to the variation of fines.

Limestone is the most permeable aggregate tested for all gradations. **The permeability of limestone ranges from 5.52 x 10^{-3} cm/sec at 3% fines to 2.49 x 10^{-3} cm/sec at 10% fines. The least permeable aggregate at 3% fines content is Razorrock chert which has a permeability of 2.91 x 10^{-3} cm/sec. At 10% fines, sandstone has the lowest permeability of 1.86 x 10^{-4} cm/sec.** Samples with 3% fines has an average decrease of 74% in permeability when fines were increased to 10%. For all gradations, permeability results obtained from the DRAINIT spreadsheet program are approximately 100 times less permeable than the results obtained from the permeability tests.

Permeability of Pavement Base Course

Final Report

Permeability of Pavement Base Course

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CHAPTER I

Introduction

Because many pavement failures are subjected to moisture-related problems, the Mack-Blackwell National Rural Transportation Study Center and the Arkansas Highway and Transportation Department funded a research project to measure the permeability of pavement base course materials. The research project, 'Permeability of Pavement Base Course' (AHTD No. TRC-9119 and MBTC No. 1010), was also to compare the measured permeability with an often used empirical formula which is built into the 'DRAINIT' spreadsheet program.

A positive drainage system to remove free water from pavement structures is necessary in order for the pavement to have a long service life. The presence of excessive water in a paving system is known to be responsible for failures of both rigid pavement and flexible pavement. Water can cause premature rutting, cracking, faulting, increased roughness, and a decrease in the level of serviceability (Baldwin, 1987). Without a good drainage system, the pavement may suffer rapid deterioration under the action of pumping caused by dynamic traffic loading, and face the risk of frost damage.

Subsurface drainage design is a part of the pavement structural design procedure (Manual of Pavement Design Principles and Practices, 1987). Most pavement design procedures include some means of adjusting thickness or pavement life based upon the pavement drainage system. If drainage is poor, the base or wearing course thickness must be increased, resulting in a more costly pavement.

One of the best methods to evaluate the internal drainage of pavement is to measure the permeability of the least permeable layer. In a pavement, the least permeable layer is most often the base course.

Base course permeability is dependent on the density, composition, and gradation of the aggregates. However, there is no sufficiently reliable relationship for predicting permeability from the grading characteristics. Therefore, it is necessary to understand the effect of the mixture design and the construction variables on permeability in order to select the best aggregate combination (Forsyth, Wells, and Woodstorm, 1987).

The U.S. Bureau of Reclamation has developed a procedure for preparing coarse aggregate specimen like base course material and measuring the permeability. The USBR permeameter, which is a 19 inches diameter by 16 inches deep steel cylinder, is used to accommodate a specimen with particles up to 3 inches in diameter.

Objectives

The four major objectives of this study are:

- to develop a laboratory procedure for preparing base course specimens for permeability testing,

- to determine the coefficient of permeability for bases made from limestone, sandstone, igneous rock, and Razorrock (chert) aggregates,

- to find the changes in the permeability due to the variation of percent fines used, and

- to compare laboratory results with the estimated values determined from the empirical formula, which was built into the DRAINIT spreadsheet program.

CHAPTER II

Literature Review

Permeability

Permeability is a property of soil that permits the passage of fluid under a gradient of force. The coefficient of permeability is defined as the rate of discharge of water at 20EC under conditions of laminar flow through a unit cross sectional area of a soil medium under a unit hydraulic gradient (Parker and Thornton, 1977). The coefficient of permeability, which is mostly used as hydraulic conductivity, has the same units as velocity. The units of the permeability coefficient are expressed in cm/sec and ft/day in this thesis. The computation of the permeability coefficient is based on Darcy's Law, which in turn, is derived from the velocity and flow rate equations introduced by H. Darcy in 1856.

Darcy's Law

In Darcy's experiments in the 1850's, he found that for laminar flow, the quantity of water flowing through the soil in a given period of time is proportional to the soil area and the difference in piezometer levels, and inversely proportional to the length of soil between piezometers (Darcy, 1856):

$$Q / t \propto (\Delta h)(A) / L$$

where, Q = *flow volume*
t = *time of flow*
Δh = *head difference*
A = *area of soil*
L = *length of flow in soil*

Darcy's equation for velocity (Dunn, Anderson, and Kiefer, 1980) is shown as:

$$v = k\,(h_L / L) = k\,i$$

where, v = *Darcy velocity (superficial velocity)*
k = *coefficient of permeability*
h_L = *head loss*
L = *length of flow*
i = *hydraulic gradient*

In soil flow problems, the total cross-sectional area is most often used as the area of flow. The volume flow rate equation is shown as:

$$q = v\,A$$

where, q = *volume flow rate*
v = *Darcy velocity (superficial velocity)*
A = *area of total cross section*

Darcy's Law is valid for most types of fluid in soils except when the fluid flows at high velocity. Also, Darcy's Law may not be valid when turbulent flow condition exists in coarse sand and gravels. Therefore, with low hydraulic gradient, Darcy's Law is bounded by the following assumptions (Leonards, 1962):

- laminar flow,
- steady-state flow condition,
- homogeneous porous medium,

- no change in voids of porous medium,
- homogeneous and incompressible fluid, and
- continuous (saturated), two dimensional flow.

Base Course Permeability

In an article, Harry R. Cedergren (1994) pointed out that the life of a poorly drained pavement is reduced to one third or less of the life of a well-drained pavement. He believes the permeability of the pavement base course should be between 10,000 ft/day (3.53 cm/sec) and 100,000 ft/day (35.28 cm/sec). Cedergren also stated that the permeability increases 40,000 times if the drainage layer material is coarse, open-graded aggregates of 0.5 to 1.0 inch instead of sand.

In another paper, Jones and Jones (1989) suggested that a granular sub-base or capping layer which is to function as a drainage layer must have the following properties:

- stability,
- adequate strength and stiffness,
- adequate permeability,
- the ability to maintain its function throughout its service life, and
- be non-frost susceptible.

Jones and Jones also stated that the method of test, state of compaction, and range of hydraulic gradients should be included in any specification of permeability.

Factors Affecting Permeability

The coefficient of permeability of soil is mostly dependent on the hydraulic gradient, grain size distribution, fluid viscosity, void ratio, and degree of saturation (Das, 1994).

Hydraulic Gradient

In Darcy's equation, the head loss used to calculate the hydraulic gradient, $i = h_L/L$, includes the loss of pressure head, elevation head, and velocity head. The flow rate, q, in a given sand is directly related to the hydraulic gradient when the flow is laminar. When the velocity is high and the flow is turbulent, the flow rate is no longer varying in direct proportion with the hydraulic gradient; the flow rate is increased by about 1.5 times when the hydraulic gradient is doubled (Edward E. Johnson, Inc., 1966).

In 1989, C. J. Baker emphasized that Darcy's Law is only valid at low hydraulic gradient (less than 0.05) for base course material. Baker also pointed out that it is not sufficient to assume that permeability can be specified simply by a characteristic value of particle size, usually D_{10} , (the diameter in the particle size distribution curve corresponding to 10% finer). In reality, permeability is a function of grading, moisture content, compaction and flow direction as well as particle size. The results from Baker's study indicated that Darcy's Law is invalid at high hydraulic gradients, and there was a variability in closely controlled laboratory test results for k due to variations in porosity and degree of saturation.

Grain Size

An increase in grain size increases the coefficient of permeability. However, the relationship between permeability and grain size only exists for fairly coarse soils with rounded grains (Hazen in Das, 1994). Hazen's experiment proposed an empirical relationship for the coefficient of permeability for fairly uniform sand:

$$k\ (cm/sec) = c\ (D_{10})^2$$

where, c = a constant that varies from 1.0 to 1.5
D_{10} = diameter of 10% finer (mm)

Because the formula does not include other factors like grading, moisture content, compaction, and flow direction, it is simplistic and does not accurately predict permeability (Baker, 1989).

Fluid Viscosity

From the Kozeny-Carmen (Parker and Thornton, 1977) equation, the permeability is directly proportional to the unit weight of water, $?_w$, and inversely proportional to the viscosity of soil water, μ:

$$k = D_s^2\ (?_w/\mu)\ (e^3/1+e)\ C$$

where, k = coefficient of permeability
D_s = diameter of spherical grain
$?_w$ = unit weight of water
μ = viscosity of water
e = void ratio of soil
C = a composite shape factor

The above formula, which includes the parameter of spherical grain diameter, is not appropriate for determining the coefficient of permeability for open well-graded base course material.

Temperature

Water viscosity changes primarily with temperature because unit weight and other properties remain constant. Since permeability is inversely proportional to viscosity, permeability increases when the temperature is higher (Parker and Thornton, 1977).

Twenty degrees Celsius is the most convenient temperature for laboratory permeability tests:

$$k_{20^\circ C} = k_T\,(\mu_T / \mu_{20^\circ C})$$

where, $k_{20^\circ C}$ = *permeability at temperature 20°C*
k_T = *permeability at temperature T°C during test*
μ_T = *viscosity of water at temperature T°C during test*
$\mu_{20^\circ C}$ = *viscosity of water at temperature 20°C*

Void Ratio

A decrease in void ratio decreases the permeability of soil. An equation which gives the relationship between void ratio and permeability is suggested by Casagrande as (Terzaghi and Peck, 1968):

$$k = 1.4\,(e^2)\,(k_{0.85})$$

where, k = *coefficient of permeability*
e = *void ratio*
$k_{0.85}$ = *coefficient of permeability at void ratio of 0.85*

Degree of Saturation

A decrease in the degree of saturation, S_r, of soil decreases the permeability. Darcy's Law is valid when the degree of saturation is 85% and higher because much of the air in soil is held in the form of small occluded bubbles (Parker and Thornton, 1976). The permeability is significantly decreased when the degree of saturation is less than 85% because the bubbles block some of the pores and much of the air is continuous through the voids.

McEnroe (1994) emphasized that the best measure of the granular base of a pavement is the minimum degree of saturation that can be achieved through gravity drainage in the field.

Measurements of Permeability

The permeability of soil can be measured in either the laboratory or the field. The laboratory permeability test is much easier to conduct and is most commonly used. However, the field test is necessary for in-situ soil because the laboratory test may be inadequate as a result of differences in soil structure and stress conditions between the laboratory and the field (Fang, 1991).

There are two direct methods to conduct permeability test in the laboratory: constant head permeability test and falling head permeability test.

Constant Head Permeability Test

Figure 1(a) shows the set-up of a constant head permeability test. The coefficient of permeability may be found by applying Darcy's Law as:

$$k = Q\,L \,/\, t\,h_L\,A$$

where, k = *coefficient of permeability*
Q = *flow volume in time* t
L = *length of flow*
t = *time of flow*
h_L = *head loss*

A = *area of total cross section*

Falling Head Permeability Test

Figure 1(b) shows the set-up of a falling head permeability test. The coefficient of permeability may again be computed from Darcy's equation as:

$$k = 2.303\,(aL/At)\,\log_{10}(h_1/h_2)$$

where,

k = *coefficient of permeability*
a = *area of graduated cylinder*
L = *length of flow*
A = *area of total cross section*
t = *time of flow*
h_1 = *initial head*
h_2 = *final head*

Empirical Formulas

In 1974, Amer and Awad introduced an empirical formula to estimate the coefficient of permeability by including the effective grain size, uniformity coefficient, and the void ratio:

$$k = C_2\, D_{10}^{2.32}\, C_u^{0.6}\, (e^3/1+e)$$

where,

C_2 = *a constant*, 3.5×10^{-4}
D_{10} = *diameter of 10% finer*
C_u = *uniformity coefficient*, D_{60} / D_{10}
D_{60} = *diameter of 60% finer*
e = *void ratio*

Another empirical formula presented in the Highway Subdrainage Design (1980) was developed from data on granular basses and subbases:

$$\textit{Permeability Coefficient, } k\ (ft/day) = [(6.214x10^{5})(D_{10})^{1.478}(1-?_d/62.4xG_s)^{6.654}])\ (P_{200})^{0.597}$$

where,

D_{10} = *diameter of 10% finer (mm)*

$?_d$ = *dry unit weight (lb/ft*^{3}*)*

G_s = *specific gravity*

P_{200} = *percent passing #200 sieve (%)*

This formula was then used in the DRAINIT spreadsheet program to estimate the permeability of base course material.

A report from the Federal Highway Administration (Longitudinal Edge Drains in Rigid Pavement Systems, 1986) illustrated the effect of fines variation on permeability. The permeabilities of limestone, silica, sitlt, and clay were determined to be 0.07, 0.085, 0.001, and 0.0006 ft/day respectively.

FIGURE 1. Constant Head Permeability Test and Falling Head Permeability Test.

CHAPTER III

Permeability Testing

<u>Equipment</u>

The following equipment was used for specimen preparation and permeability testing:

<u>Permeability Apparatus</u>

The permeability apparatus consist of the permeameter, the head tank, and the porous disk. The permeability apparatus was obtained from the Bureau of Reclamation, United States Department of The Interior. The *permeameter* (Figure 2) is a 19 inches diameter by 16 inches deep steel cylinder designed to contain a 9 inches thick base course specimen. The *head tank* (Figure 3) is a 6 inches diameter by 40 inches long plexiglas cylinder which contained water of 425.85 cm^3/in. The *porous disk* was placed at the inside bottom of the permeameter. The disk is made of a 19 inches diameter by 2 inches thick coarse grade carborundum porous material.

<u>Compaction Machine</u>

A vibratory hammer (Figure 4), made by Wacker Corporation in Wisconsin (Model EHB 10/110), was modified and used for compacting the sample aggregates. The compactor was fitted with a 4 inches diameter compaction head and operated at 60 Hz frequency.

FIGURE 2. Permeameter for Containing Base Course Specimen.

FIGURE 3. Head Tank.

FIGURE 4. WACKER Mechanical Compactor.

FIGURE 5. Sponge Rubber Liner Attached to Inside Wall of Permeameter.

Sponge Rubber Sheet and Rubber Cement

A $^1/_2$ inch thick closed cell medium density sponge rubber sheet was used as a liner for the inside wall of the permeability cylinder to prevent flow between the sample and cylinder wall. A rubber cement adhesive was used for attaching the sponge rubber liner to the inside wall of the permeameter (Figure 5).

Geofilter Fabric

A geofilter fabric, made by Phillips Fibers Corporation (Type SUPAC 5-P), was placed between the base course specimen and the porous disk for preventing the fines from clogging the porous disk.

Tubing and Connector

Tubing was needed to connect water from the water supply to the head tank, from the head tank to the permeameter, and from the permeameter to a drain. The clear plastic tubing used was $^5/_8$ inch diameter and approximately 20 feet long. A T-connector was used to inflow water from the head tank to both sides of the bottom of the permeameter.

Mixing Pan

A mixing pan (Figure 6) with a size of approximately 3 feet by 2 feet by 6 inches deep was used to mix the aggregates.

Sieves and Sieve Machine

The sieve machine (Figure 7), made by Gilson Screen Company, Inc. in Ohio (Model TS-1), was used to separate the soil aggregates into the desired gradations. Sieve screens

$1\,^{1}/_{2}$", $^{3}/_{4}$", #4, #40, and #200 were used to meet the AHTD specification for Class-7 base course grading.

FIGURE 6. Mixing Pan for Mixing Aggregates.

FIGURE 7. Sieve Machine for Separating Soil Aggregates into Desired Gradations.

FIGURE 8. Rock Grinder for Grinding Soil Aggregates into Fines.

Rock Grinder

A rock grinder (Figure 8) was used to grind soil particles when the fine material provided was not sufficient. The rock grinder used was Model 103675(RA224) and made by the Denver Fire Clay Company in Colorado. It generated 2 horse power at 1725RPM.

Oven

A fan circulated oven, made by Blue-M Company in Illinois (Model OV-490A-3), was used to oven-dry the aggregates before they were sieved. The drying temperature was

100°C (212°F). Aggregates were dried at least 12 hours.

Materials

Four cohesionless aggregates were provided by Mr. David W. Lumbert (1994) of the AHTD:

- Limestone - - the limestone aggregates were obtained from McClinton Anchor quarry near West Fork. It is a bedded sedimentary deposit consisting mainly of calcium carbonate ($CaCO_3$).

- Sandstone - - the sandstone aggregates were obtained from the area near Russellville. It is a cemented or otherwise compacted detrital sediment composed predominantly of quartz grains.

- Igneous Rock - - the igneous rock aggregates were obtained from Granite Mountain. It is also called plutonic rock because it was formed by solidification of hot mobile material (magma).

- Razorrock - - the Razorrock aggregates were obtained from Whitehall at a quaternary deposit which composed of crypto-chrystaline chert. These aggregates are crushed gravel having smooth faces and fractured sides.

A hydrometer analysis test was conducted on fines of limestone (selected as an example). Eighty five percent, by weight, of the fines were silt size particles (Appendix C). The fines for all four aggregates by inspection were determined as rock flour.

The aggregate samples were received wet in open/mix gradation. They were oven-dried and sieved in order to construct a sample that meets the AHTD Class-7 specification for base course grading.

Sample Preparation

Sample aggregates were oven-dried for at least 12 hours and then sieved using the sieve sizes of $1^1/_2$", $^3/_4$", #4, #40, and #200 according to the AHTD Class-7 base course grading (Table 1).

TABLE 1. AHTD Class-7 Base Course Grading (Standard Specifications for HIGHWAY CONSTRUCTION, 1993).

SIEVE SIZE	PERCENT PASSING
$1^1/_2$"	100
$^3/_4$"	50 - 90
#4	25 - 55
#40	10 - 30
#200	3 - 10

The base course specimens were built with three gradations in which the content of the fines (-#200) were varied from 3% to 10% by weight in order to find the effect of fines on the permeability. For construction of the 9 inch thick specimen, the sample was divided into three layers (Earth Manual Part II, 1990). Calculation of the specimen weight is shown below:

$$\textit{Specimen Volume} = 3.14159\ (9'' \times 9'')\times 9''$$
$$= 2290.22\ in^3$$
$$= 1.33\ ft^3$$

$$\textit{Target Dry Unit Weight} = 140\ lb/ft^3$$

$$\textit{Total Specimen Weight for Layers} = 140\ lb \times 1.33\ ft^3$$

$$= 186.2\ lb$$

$$Weight\ of\ Each\ Layer = 186.2\ )\ 3$$
$$= 62.07\ lb$$

Table 2 contains the specimen weight, according to sieve sizes, for the three gradations. The plots of the Class-7 gradation limits and the mid-point grain size curve are shown in Figure 9.

TABLE 2. Weight Distribution of Aggregate Gradations for A Base Course Specimen (1 Layer).

SIEVE SIZE	3% FINES		6.5% FINES		10% FINES	
	%Ret'd	Wt.(lb)	%Ret'd	Wt.(lb)	%Ret'd	Wt.(lb)
$1\frac{1}{2}$"	0	0	0	0	0	0
$\frac{3}{4}$"	31.12	19.32	30	18.62	28.9	17.92
#4	31.12	19.32	30	18.62	28.9	17.92
#40	20.75	12.88	20	12.41	19.3	11.95
#200	14.01	8.69	13.5	8.38	12.9	8.07
-#200	3	1.86	6.5	4.03	10	6.21
Total	100	62.07	100	62.07	100	62.07

Each layer of the specimen was prepared separately. The addition of 4% water by weight was necessary in order to prevent segregation during the compaction process. Trial mixes of 2%, 4%, and 6% water were added to the aggregates to develop the sample preparation procedure. At a water content of 2%, the aggregates were too dry, causing segregation. The addition of 6% water caused the fines to flow. No segregation was observed when the aggregates contained 4% water.

Before compaction, the permeameter cylinder was cleaned and dried. Then, a closed cell sponge rubber liner was attached with a rubber cement adhesive to the inside wall of the permeameter to prevent segregation, and the geofilter fabric was placed on the porous disk inside the bottom of the permeameter.

The mixed aggregates for the first layer were placed in the permeameter and carefully leveled in a loose state. Then, the aggregates were compacted by using the mechanical compactor so that the 4 inches diameter compaction head moved along the perimeter from the edge towards the center of the specimen. This process continued until the specimen reached the required density (a 3 inch line marked on the sponge rubber liner). The second and third layers of the specimen were constructed in the same manner.

Testing Procedures

The air cock at the top of the head tank was opened to allow air (atmospheric pressure) to escape while the head tank was filled through the inlet valve from the bottom. After the head tank was filled, the inlet valve was closed and the valve to inflow water to the permeameter was opened. The drain cock at the bottom of the permeameter was opened to let the air to escape while water was let into the permeameter at low hydraulic gradient from both side inlets at the bottom of the permeameter (Figure 10). Water was allowed to permeate slowly upward through the specimen without damaging the specimen. The flow of water was stopped after the specimen was saturated and the water level reached the outlet tube. Before the permeability test was started, the elevation of the head tank was adjusted and recorded to obtain the headwater and tailwater elevations at the start of the test.

To begin the permeability test, the timer was started at the same time the valve at the head tank was opened to allow water into the permeameter. After the water level in the head tank dropped to a predetermined level, the timer was stopped. The elapsed time of this process and the volume of water dispensed from the head tank was used to calculate the permeability coefficient, k. The flow chart of sample preparation and permeability testing procedures is shown in Figure 11.

FIGURE 9. AHTD Class-7 Gradation and Grain Size Curves for Base Course Aggregates.

FIGURE 10. The Set-up of Permeability Apparatus.

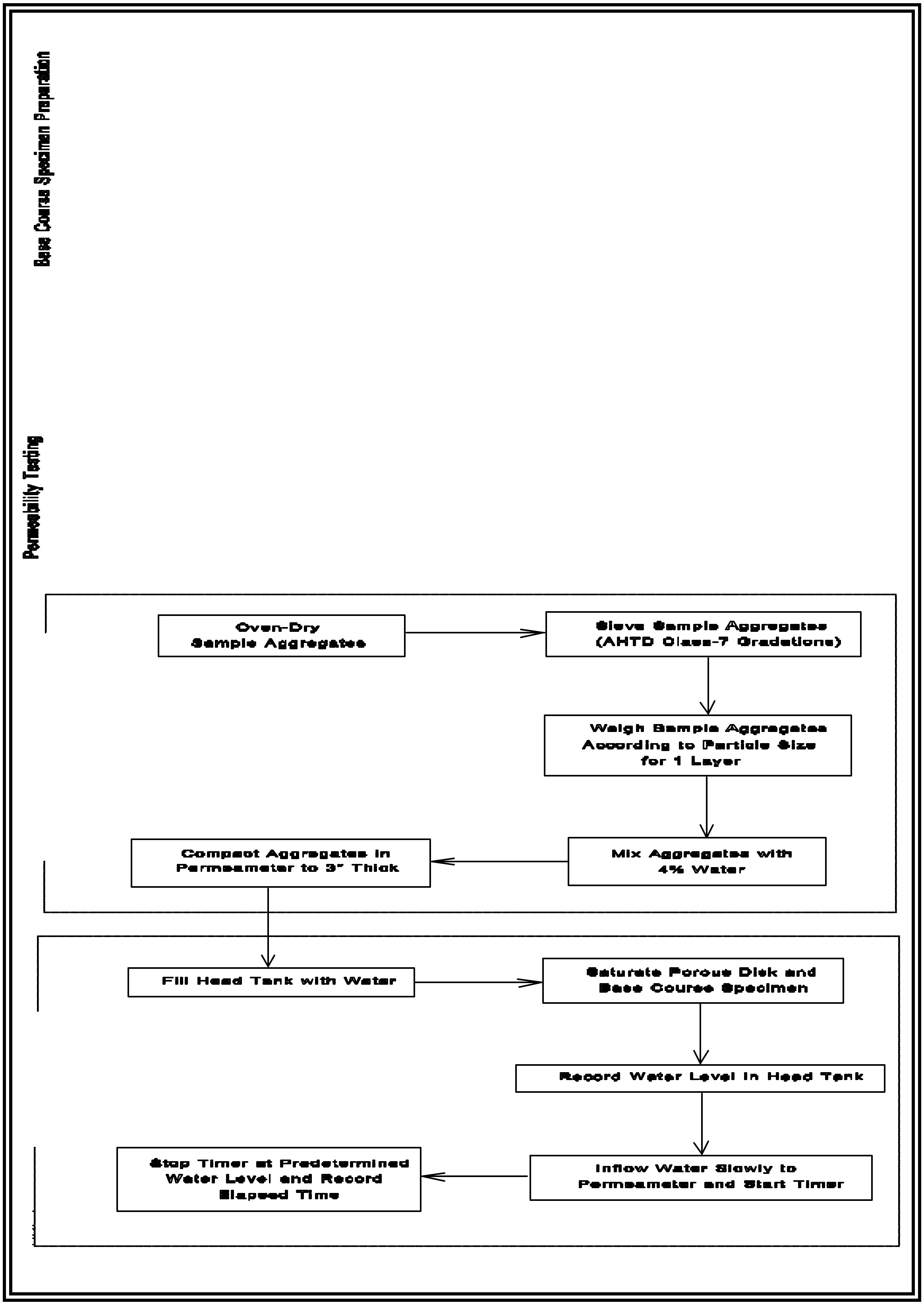
Base Course Specimen Preparation
Oven-Dry Sample Aggregates
Sieve Sample Aggregates (AHTD Class-7 Gradations)
Weigh Sample Aggregates According to Particle Size for 1 Layer
Mix Aggregates with 4% Water
Compact Aggregates in Permeameter to 3" Thick
Permeability Testing
Fill Head Tank with Water
Saturate Porous Disk and Base Course Specimen
Record Water Level in Head Tank
Inflow Water Slowly to Permeameter and Start Timer
Stop Timer at Predetermined Water Level and Record Elapsed Time

FIGURE 11. The Flow Chart of Sample Preparation and Permeability Testing Procedures.

CHAPTER IV

Results and Analysis

Permeability Test Results

The permeability test was conducted with three different gradations on each base course material type. A minimum of nine permeability tests were conducted on each specimen, three each at head differences of 3, 6, and 10 inches. Additional tests were conducted when the results were not consistent. The three best results for head differences of 3, 6, and 10 inches were averaged for all tests. The falling head permeability formula (Das, 1994) was used to calculate the coefficient of permeability:

$$\text{Permeability Coefficient, } k \text{ (cm/sec)} = 2.303 \times [\,(a)(L)/(A)(t)\,] \times \log(h_1/h_2)$$

where,

a = *cross-sectional area of head tank, 167.5* cm^2
L = *height of base course specimen, "22.86 cm*
A = *cross-sectional area of permeameter, 1641* cm^2
t = *time recorded (sec)*
h_1 = *initial head (in)*
h_2 = *final head (in)*

The calculation of *k* value for the limestone specimen with 3% fines is an example:

Obtained from permeability test:
$t = 118$ *sec*
$h_1 = 13.25$ *in*
$h_2 = 10.25$ *in*

Permeability Coefficient, $k = 2.303 \times [(167.5)(22.23)/(1641)(118)] \times \log(13.25/10.25) = 0.00552$ *cm/sec (15.7 ft/day)*

The results for all permeability tests are shown in Table 3(a) for SI units and 3(b) for US units. The permeability for limestone is 5.52×10^{-3} cm/sec for 3% fines and 2.49×10^{-3} cm/sec for 10% fines. The decrease in permeability due to the increase in fines is 54.9%. For sandstone, the permeability decreased from 4.34×10^{-3} cm/sec for 3% fines to 1.86×10^{-4} cm/sec for 10% fines, a 95.7% decrease. For igneous rock, the decrease is 81.5% due to the change in permeability from 4.53×10^{-3} cm/sec for 3% fines to 8.36×10^{-4} cm/sec for 10% fines. Also, for Razorrock chert, the permeability has dropped from 2.91×10^{-3} cm/sec for 3% fines to 1.05×10^{-4} cm/sec for 10% fines, a 63.9% decrease.

Table 3(a). Permeability Test Results (cm/sec).

TYPE OF AGGREGATES	3% FINES	6.5% FINES	10% FINES
	k (cm/sec)	*k* (cm/sec)	*k* (cm/sec)
Limestone	5.52×10^{-3}	3.48×10^{-3}	2.49×10^{-3}
Sandstone	4.34×10^{-3}	1.66×10^{-3}	1.86×10^{-4}
Igneous Rock	4.53×10^{-3}	1.57×10^{-3}	8.36×10^{-4}
Razorrock	2.91×10^{-3}	1.76×10^{-3}	1.05×10^{-3}
AVERAGE	4.33×10^{-3}	2.12×10^{-3}	1.14×10^{-3}

Table 3(b). Permeability Test Results (ft/day).

TYPE OF AGGREGATES	3% FINES	6.5% FINES	10% FINES
	k (ft/day)	*k* (ft/day)	*k* (ft/day)
Limestone	15.70	9.87	7.05

Sandstone	12.30	4.70	0.53
Igneous Rock	12.80	4.45	2.37
Razorrock	8.26	5.00	2.96
AVERAGE	12.30	6.01	3.23

Permeability results and best fit straight lines, which were generated by using the least square best fit method, were plotted on a semi-log graph (Figure 12(a) and 12(b)). The form of the semi-log equation of a best fit line is shown as:

$$log_{10}\, k = m\, P_{200} + C$$

where, k = *permeability coefficient*
P_{200} = *percent fines used*
m = *slope*
C = *constant (k-intercept)*

Best fit equations are contained in Table 5.

Limestone is the most permeable aggregate tested for all gradations. Razorrock chert is the least permeable aggregate when the specimen contained 3% fines; igneous rock is least permeable when the specimen contained 6.5% fines, and sandstone is least permeable when the specimen contained 10% fines.

When samples contained 10% fines, there was more variation in permeability than when they contained 3% fines. For 3% fines, the permeability ranges from 2.91×10^{-3} cm/sec to 5.52×10^{-3} cm/sec, which yields a 60.3% change. For 6.5% fines, the difference is 90.1% due to the change in permeability from 1.57×10^{-3} cm/sec to 3.48×10^{-3} cm/sec. Also, the percent changes of permeability for 10% fines is 145.1% due to the change in permeability from 1.86 x

10^{-4} cm/sec to 2.49 x 10^{-3} cm/sec.

According to Darcy's law, the flow rate (for a laminar flow) in a saturated sand varies directly with the hydraulic gradient (Edward E. Johnson, Inc., 1966). The existence of laminar flow was confirmed by comparing the ratio of selected flow rates to the ratio of selected hydraulic gradients obtained from the permeability test results:

$$q_1 / q_2 \ \% \ {}^{a}h_1 / {}^{a}h_2$$

Darcy's Law, $q_1 = (k_1)\,(A)\,({}^{a}h_1 / L)$
$q_2 = (k_2)\,(A)\,({}^{a}h_2 / L)$

since, $A = constant$ *and* $L = constant$

thus, $q_1 = (k_1)({}^{a}h_1)$
$q_2 = (k_2)({}^{a}h_2)$

where, q = *flow rate*
k = *coefficient of permeability*
A = *area of total cross section*
${}^{a}h$ = *average head,* $(h_1 + h_2 / 2)$
L = *length of flow*

The permeability result of sandstone with 3% fines is used as an example:

$$q_1 = (11.9)\,(29.85) = 355.16$$
$$q_2 = (11.9)\,(18.42) = 219.14$$

thus, $q_1 / q_2 = 1.62$
${}^{a}h_1 / {}^{a}h_2 = 1.61$ Y *laminar flow*

In order to eliminate the random testing errors and the inaccuracies of the average heads in a falling head permeability test, the ratios of q and ${}^{a}h$ of 3%, 6.5%, and 10% fines for all tested aggregates were summed:

Total ratio of q = 19.56
Total ratio of ªh = 19.98 Y *laminar flow*

DRAINIT Results

The DRAINIT spreadsheet program, developed by Carpenter (1986) from the University of Illinois at Urbana-Champaign, estimates the drainage time of a given pavement base course. Only the first portion of the program, 'PAVEMENT SECTION PROPERTIES', was used in this research to estimate the permeability for the tested base course aggregates (Appendix B). The empirical formula used in DRAINIT, which was developed from data on granular bases and subbases (Highway Subdrainage Design, 1980), is as follows:

$$\text{Permeability Coefficient, } k\ (ft/day) = [(6.214x10^{5})(D_{10})^{1.478}(1-?_{d}/62.4xG_{s})^{6.654}]\ )\ (P_{200})^{0.597}$$

where,
D_{10} = *diameter of 10% finer (mm)*
$?_{d}$ = *dry unit weight* (lb/ft^3)
G_s = *specific gravity*
P_{200} = *percent passing #200 sieve (%)*

The material properties, which were entered in DRAINIT, are shown below:

Diameter of 10% Finer, D_{10} = *0.18 mm (3% fines)*
= *0.14 mm (6.5% fines)*
= *0.075 mm (10% fines)*

Dry Unit Weight, $?_{d}$ = $140\ lb/ft^3$ *(assumed for all samples)*

Specific Gravity, G_s = *2.65 (assumed for all samples)*

Percent Passing #200 Sieve, P_{200} = *3 %*
= *6.5 %*
= *10 %*

Table 4 contains the permeability results obtained from DRAINIT.

TABLE 4. DRAINIT Results.

PERCENT FINES	PERMEABILITY COEFFICIENTS	
	k (cm/sec)	*k* (ft/day)
3	3.44×10^{-5}	9.76×10^{-2}
6.5	1.50×10^{-5}	4.24×10^{-2}
10	4.60×10^{-6}	1.30×10^{-2}

The DRAINIT results and a straight line, which was generated by using the least square best fit method, are shown in Figure 13(a) and 13(b). The DRAINIT results show a decreased of 86.6% due to the change of fines from 3% to 10%.

Figure 14(a) and 14(b) compare the results from the permeability tests to the results from DRAINIT. Figure 15(a) and 15(b) compare the resulting slopes of the averaged permeability test results and the DRAINIT results. The linear equations of the best fit lines are listed in Table 5(a) and 5(b).

TABLE 5(a). Best Fit Equations for Permeability Coefficients, *k* (cm/sec).

TYPE OF AGGREGATES	UNITS OF PERMEABILITY COEFFICIENT, *k* (cm/sec)
Limestone	$\log_{10} k = -(4.86\text{x}10^{-2})\ P_{200} - 2.12$
Sandstone	$\log_{10} k = -(1.96\text{x}10^{-1})\ P_{200} - 1.69$
Igneous Rock	$\log_{10} k = -(1.06\text{x}10^{-1})\ P_{200} - 2.05$
Razorrock	$\log_{10} k = -(6.29\text{x}10^{-2})\ P_{200} - 2.35$
AVERAGE	$\log_{10} k = -(8.28\text{x}10^{-2})\ P_{200} - 2.12$
DRAINIT	$\log_{10} k = -(1.25\text{x}10^{-1})\ P_{200} - 4.06$

TABLE 5(b). Best Fit Equations for Permeability Coefficients, *k* (ft/day).

TYPE OF AGGREGATES	UNITS OF PERMEABILITY COEFFICIENT, *k* (ft/day)
Limestone	$\log_{10} k = -(5.03 \times 10^{-2})\ P_{200} + 1.34$
Sandstone	$\log_{10} k = -(1.95 \times 10^{-1})\ P_{200} + 1.77$
Igneous Rock	$\log_{10} k = -(1.05 \times 10^{-1})\ P_{200} + 1.39$
Razorrock	$\log_{10} k = -(6.37 \times 10^{-2})\ P_{200} + 1.11$
AVERAGE	$\log_{10} k = -(8.30 \times 10^{-2})\ P_{200} + 1.33$
DRAINIT	$\log_{10} k = -(1.25 \times 10^{-1})\ P_{200} - 0.61$

DRAINIT predicts significantly lower permeabilities than the test results. For all 3%, 6.5%, and 10% fines, the DRAINIT results are approximately 100 times less permeable compared to the average permeability obtained from the permeability tests. These results are not so suprising because the empirical formula used in DRAINIT may consider clay as fines instead of rock flour. The comparison of permeability between limestone and clay shows an approximately 117 times difference (Longitudinal Edge Drains in Rigid Pavement Systems, 1986). The comparison between the slopes of the DRAINIT results and the averaged permeability test results has not shown a significant difference.

Coefficients of permeability were also calculated by using the empirical formula introduced by Amer and Awad (1974) in Chapter II. The permeability obtained for 3%, 6.5%, and 10% fines are 4.68×10^{-7} cm/sec, 2.92×10^{-7} cm/sec, and 9.53×10^{-8} cm/sec respectively. Further consideration is not given to these results because they are much lower compared to the results obtained from the permeability tests.

FIGURE 12(a). Semi-log Plot of Permeability Coefficient, *k* (cm/sec) vs Percent Fines Obtained from Permeability Tests.

FIGURE 12(b). Semi-log Plot of Permeability Coefficient, *k* (ft/day) vs Percent Fines Obtained from Permeability Tests.

FIGURE 13(a). Semi-log Plot of Permeability Coefficient, *k* (cm/sec) vs Percent Fines Obtained from DRAINIT.

FIGURE 13(b). Semi-log Plot of Permeability Coefficient, *k* (ft/day) vs Percent Fines Obtained from DRAINIT.

FIGURE 14(a). Semi-log Plot of Permeability Coefficient, *k* (cm/sec) vs Percent Fines.

FIGURE 14(b). Semi-log Plot of Permeability Coefficient, *k* (ft/day) vs Percent Fines.

FIGURE 15(a). The Comparison of Averaged Permeability Test Results and DRAINIT Results (cm/sec).

FIGURE 15(b). The Comparison of Averaged Permeability Test Results and DRAINIT Results (ft/day).

CHAPTER V

Conclusions

A laboratory procedure was developed for testing the base course permeability of a simulated field sample. The permeability test using the 19" permeameter was experimentally proven operational. Several minor considerations are necessary to avoid segregation and to achieve the required density.

Limestone is the most permeable aggregate tested for all gradations. The permeability of limestone ranges from 5.52 x 10^{-3} cm/sec at 3% fines to 2.49 x 10^{-3} cm/sec at 10% fines. The least permeable aggregate at 3%fines content is Razorrock chert which has a permeability of 2.91 x 10^{-3} cm/sec, at 10% fines. At 10% fines, sandstone has the lowest permeability of 1.86 x 10^{-4} cm/sec. Samples with 3% fines has an average decrease of 74% in permeability when fines were increased to 10%.

The DRAINIT spreadsheet program does not accurately predict the permeability of base course materials. The permeability predicted by DRAINIT is 99.4% lower than the averaged permeability test results. The empirical formula used in DRAINIT depends only on effective grain size, dry unit weight, specific gravity, and percent fines. As a result, DRAINIT predicts the same permeability for all aggregate types. In order to modify the empirical formula (DRAINIT) to predict permeability for

each type of aggregate, soil classification (involving plasticity), uniformity coefficient, and coefficient of gradation may need to be considered.

REFERENCES

Amer, Abdelwahab M. and Amin A. Awad, 1974, "Permeability of Cohesionless Soils", *Journal of the Geotechnical Engineering Division,* ASCE, New York, v 100, n GT12, pp 1309-1316.

Baker, C. J., 1989, "General Report: Drainage and Permeability", ***Proceedings of The 3rd International Symposium on Unbound Aggregate in Roads,*** **University of Nottingham, England, pp 7-8.**

Baldwin, John S., 1897, "Use of Open-Graded, Free-Draining Layers in Pavement Systems: A National Synthesis Report", *Transportation Research Record 1121,* TRB, National Research Council, Washington D. C.

Carpenter, S. H., 1986, *'DRAINIT', Drainability Prediction Spreadsheet Program,* Department of Civil Engineering Department, University of Illinois of Urbana-Champaign, Illinois.

Cedergren, Harry R., Sep 1994, "America's Pavements: World's Longest Bathtubs", *Civil Engineering,* ASCE, New York, v 64, n 9, pp 56-58.

Darcy, H., 1856, ***Les Fontaines Publiques de La Ville de Dijon (The Public Water Supply of The City of Dijon),*** **Dalmont, Paris, France, pp 674.**

Das, Braja M., 1994, ***Principal of Geotechnical Engineering,*** **3rd Edition, PWS Publishing Company, Boston, Massachusetts, pp 134-148.**

Dunn, I. S., L. R. Anderson, and F. W. Kiefer, 1980, ***Fundamentals of Geotechnical Analysis,*** **John Wiley & Sons, New York, pp 55-56.**

Edward E. Johnson, Inc., 1966, ***Ground Water and Wells,*** **Saint Paul, Minnesota, pp 37-46.**

Fang, Hsai-Yang, 1991, ***Foundation Engineering Handbook,*** **2nd Edition, Van Nostrand Reinhold, New York, pp 752-755.**

"Earth Manual Part 2: Procedure for Determining Permeability and Settlement of Soils Containing Gravel", 1990, *A Water Resources Technical Publication,* 3rd Edition, U.S. Department of Interior, Bureau of Reclamation, Denver Office, Colorado, pp 538-557.

Forsyth, Raymond A., Gordon K. Wells, and James H. Woodstorm, 1987, "Economic Impact of Pavement Subsurface Drainage", *Transportation Research Record 1121*, TRB, National Research Council, Washington D. C.

Highway Subdrainage Design, Aug 1980, U.S. Department of Transportation, Federal Highway Administration Report FHWA-TS-80-224, Washington D. C., pp 51.

Jones, R. H. and H. A. Jones, 1989, "Keynote Paper: Granular Drainage Layers in Pavement Foundations", *Proceedings of The 3rd International Symposium on Unbound Aggregate in Roads,* University of Nottingham, England, pp 55-69.

Leonard, G. A., 1962, *Foundation Engineering,* McGraw-Hill, New York, pp 107-110.

Longitudinal Edge Drains in Rigid Pavement Systems, July 1986, Final Report of Federal Highway Administration (No. FHWA-TS-86-208), Washington D. C., pp 9.

Lumbert, David W, 1994, "Personal Communication", Arkansas Highway and Transportation Department, Box 2261, Little Rock, AR 72203.

Manual of Pavement Design Principles and Practices (A Training Course Participant Notebook), 1987, Federal Highway Administration, ERES Consultants, Inc., Washington D. C., pp 149-203.

McEnroe, Bruce M., 1994, "Drainability of Granular Bases for Highway Pavements", *Transportation Research Record 1434,* TRB, National Research Council, Washington D. C.

Parker, David G. and Sam I. Thornton,1977, "Permeability of Fly Ash and Fly Ash Stabilized Soils", *Final Report of HRC 47*, Arkansas Highway and Transportation Department, Arkansas, pp 2-17.

Standard Specifications for HIGHWAY CONSTRUCTION, 1993, Arkansas Highway and Transportation Department, Edition of 1993, Arkansas, pp 161.

Terzaghi, K, and R. B. Peck, 1948, *Soil Mechanics in Engineering Practice,* John Wiley & Sons, New York, pp 51.