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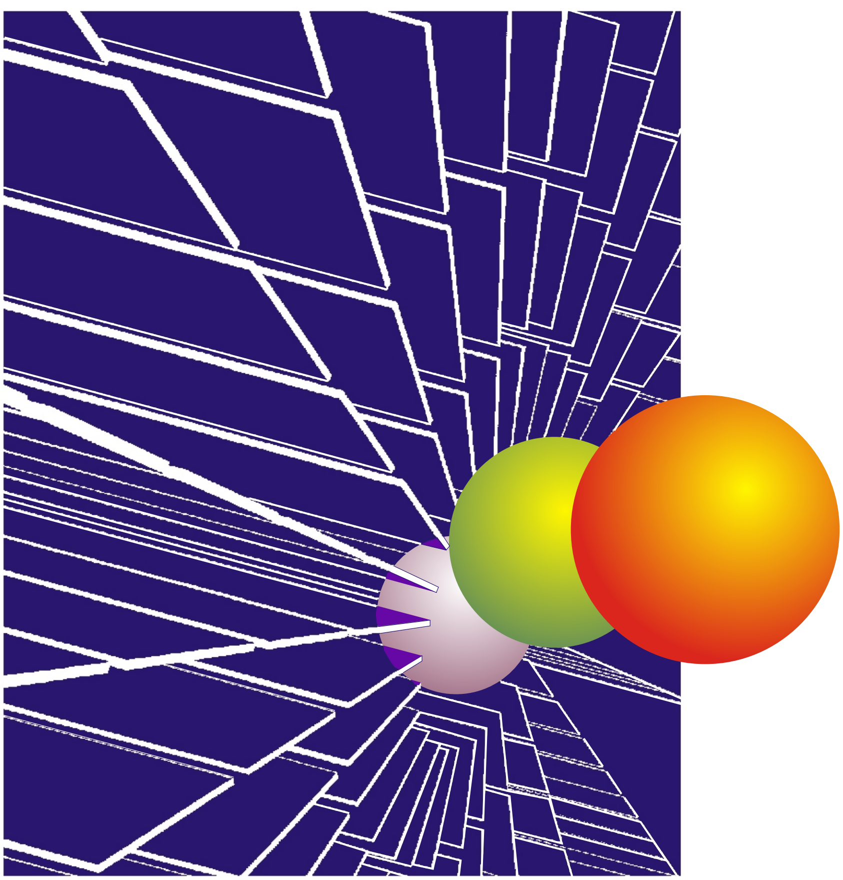
Characterization of Permeability of Pavement Bases in Missouri Department of Transportation's System

RI 01-006



February, 2003

Final Report

RDT 03-005
Research Investigation 01-006

Characterization of Permeability of Pavement Bases in Missouri Department of Transportation's System

PREPARED FOR:

MISSOURI DEPARTMENT OF TRANSPORTATION
RESEARCH, DEVELOPMENT AND TECHNOLOGY
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16. Abstract

The results of field (in-situ) and laboratory permeability testing showed MoDOT's predominant pavement base "Type 5" (and the upper "working" surface of the 2-foot rock fill alternative) has hydraulic conductivities that are several orders of magnitude (10^{-3} to 10^{-5} cm/s) lower than the freely-draining value of 1 cm/sec. Using the measured hydraulic conductivities in the FHWA's DRIP 2.0 analysis for evaluating pavement performance resulted in drainage quality rankings of poor to very poor.

Preliminary strength testing was performed on the Type 5 base in order to quantify the strength behavior. Results of cyclic triaxial tests under saturated, undrained conditions performed on compacted, Type-5 base showed the saturated base loses most of its strength within a few load cycles. Drained cyclic tests resulted in the same behavior – almost complete loss of strength in just a few load cycles. The findings indicate that the Type 5 base does not allow moisture to escape (drain) sufficiently fast enough during traffic loading resulting in build up of excess porewater pressure and loss of strength in the layer.

Given these findings, it is concluded that Missouri pavements are likely to require more frequent maintenance and are not lasting as long as they could be if adequate drainage were provided.

It is recommended that a more durable roadway base be developed. One that provides an adequate working platform during construction and good drainage for extended lifetimes. Several tasks are recommended in order to gather high quality data (evidence) that are prerequisite for the development of an effective specification for roadway base in Missouri. A program of laboratory strength testing, in situ permeability testing, instrumented pavement sections and a geographical information system-based pavement performance database should be undertaken. Such a program will provide the necessary evidence on which to initiate changes to Missouri's pavement base system which will provide higher performance, longer-lasting and more economical pavement systems.

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EXECUTIVE SUMMARY

The pavement structure is the most costly element of the highway system, and its premature failure is of major concern. Inadequate drainage has been identified as the most common cause, among the many reasons cited for pavement failure. There was no quantitative evidence as to whether Missouri Department of Transportations's (MoDOT) Type 5 roadway base provided the necessary effective drainage beneath pavements.

The results of field (in-situ) and laboratory permeability testing showed MoDOT's predominant pavement base "Type 5" (and the upper "working" surface of the 2-foot rock fill alternative) has hydraulic conductivities that are several orders of magnitude (10^{-3} to 10^{-5} cm/s) lower than the freely-draining value of 1 cm/sec. In essence, these materials should be classified as undrained. Using the measured hydraulic conductivities in the FHWA's DRIP 2.0 analysis for evaluating pavement performance resulted in drainage quality rankings of poor to very poor based on the 1986 AASHTO *Guide for Design of Pavement Structures*. Given these findings, it is concluded that Missouri pavements are likely to require more frequent maintenance and are not lasting as long as they could be if adequate drainage were provided.

Preliminary strength testing was performed on the Type 5 base in order to quantify the strength behavior. Results of cyclic triaxial tests under saturated, undrained conditions performed on compacted, Type-5 base showed the saturated base loses most of its strength within a few load cycles. Drained cyclic tests resulted in the same behavior – almost complete loss of strength in just a few load cycles. The findings indicate that the Type 5 base does not allow moisture to escape (drain) sufficiently fast enough during traffic loading resulting in build up of excess porewater pressure and loss of strength in the layer.

The field and laboratory permeability testing showed the Type 5 base to have such low hydraulic conductivities as to be considered undrainable. Preliminary cyclic strength tests showed the base loses most of its strength in just a few loading cycles. This behavior could explain the premature deterioration of some pavements in Missouri. Providing adequately drainable bases will increase the effective performance life, reduce maintenance frequency and reduce replacement costs for Missouri pavements.

It is recommended that a more durable roadway base be developed. One that provides an adequate working platform during construction and good drainage for extended lifetimes. Several tasks are recommended in order to gather high quality data (evidence) that are prerequisite for the development of an effective specification for roadway base in Missouri. A program of laboratory strength testing, in situ permeability testing, instrumented pavement sections and a geographical information system-based pavement performance database should be undertaken. Such a program will provide the necessary evidence on which to initiate changes to Missouri's pavement base system which will provide higher performance, longer-lasting and more economical pavement systems.

TABLE OF CONTENTS

List of Tables

List of Figures

INTRODUCTION

The pavement structure is the most costly element of the highway system, and its premature failure is of major concern. Improper design or application of subsurface drainage can lead to poorer than expected performance of the pavement (NCHRP 1994, NCHRP 1997). Among the many reasons cited for pavement failure, inadequate drainage has been identified as the most common cause. The AASHTO Guide for the Design of Pavement Structures (1986) identifies drainage as a major concern by including drainage as an essential element of pavement design. Also, the FHWA pavement management and design policy encourages performing a drainage analysis for each new and reconstruction pavement design.

Standard designs and specifications exist in the Missouri Department of Transportation's highway engineering practice that should allow for drainage of water beneath the roadway pavement. In all of the designs and most of the applications, a specified base course (Type 5) is used beneath the pavement surface. The specification for Type 5 base allows for up to fifteen percent fines (material passing the Number 200 sieve, 0.074 mm). There was some question as to whether the Type 5 base possessed the necessary hydraulic conductivity to provide effective drainage beneath pavements.

OBJECTIVES

The objective of this project was to characterize the hydraulic conductivity of base course materials used below pavements throughout Missouri. Materials included Type 5 base material from various sources and a rock fill alternate base material. The scope of the work performed to fulfill this objective included:

- Determination of grain size distribution of the base materials,
- Estimation of the hydraulic conductivity of the base materials using empirical equations, some of which are widely used in pavement design,

- Determination of the hydraulic conductivity of Type 5 base material in the laboratory,
- Determination of the in-situ hydraulic conductivity of Type 5 base material using a double-ring infiltrometer (DRI), and
- Determination of the drainage characteristics of the base material using pavement design software.

In addition, preliminary investigations of the strength of the Type 5 base under static and cyclic loading conditions were performed.

PRESENT TECHNICAL CONDITIONS

For the last four decades, with some notable exceptions, most DOT's have been designing and relying on strong but non-drainable bases for their pavements (Figure 1). Infiltrated moisture becomes trapped beneath the pavement and results in decreased strength in the underlying support layers and ultimately premature failure of the overlying pavement. The absence of, or improperly operating, pavement subsurface drainage systems have long been known to greatly decrease the service life of pavements (Cedergren 1996).

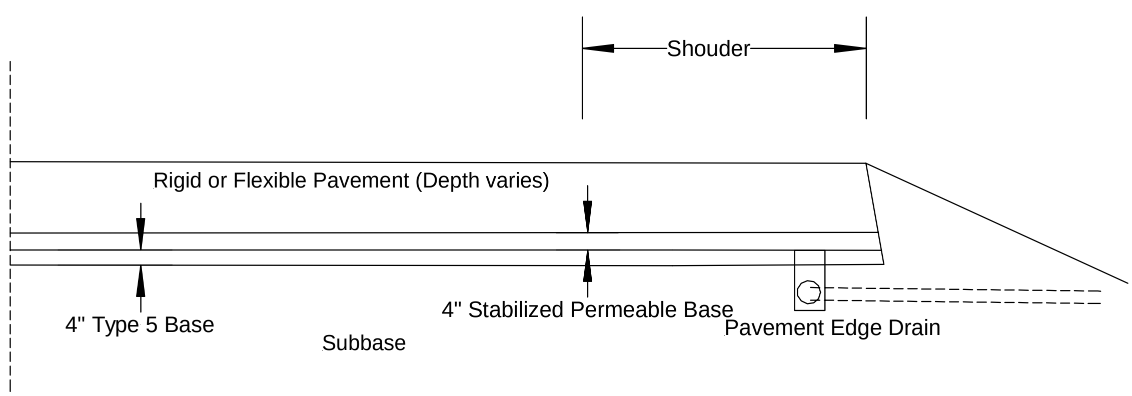


Figure 1 MoDOT's heavy duty pavement section showing Type 5 base application. (Medium and light duty pavements do not include the 4-inch stabilized permeable base.)

The American Road Officials 1955 and 1960 study showed that pavements with water-filled bases failed 10 to 40 times faster than pavements with freely draining substructures and that those in the snow-melt period failed up to 70,000 times faster (Cedergren 1996).

A freely draining material is necessary to remove infiltrating water from base courses in an effective time period. The required characteristics of the base are a function of the amount of water that must be drained; however, clean (no fines) aggregates with a minimum size of ½ inch has been cited as necessary in many cases and provides a hydraulic conductivity on the order of 15 cm/sec. The current MoDOT specification of base aggregate (Type 5 base) allows up to 15 percent fines and this is measured at the stockpile rather than after the base has been placed and compacted (Figure 2). One can expect an increase in the fines content of about ½ percent for each handing operation of the aggregate (Daniel and Koerner 1995). Even at 4 percent fines content, the hydraulic conductivity of the Type 5 base has been shown to be in the $1x10^{-6}$ cm/sec range (Prachantrikal 2001). Thus, the current base course used beneath most Missouri state system roads may not be freely draining. It then follows that the effectiveness of any additional drainage measures such as edge and longitudinal drains is severely limited since infiltrative water in the base cannot reach the drains in a timely manner.

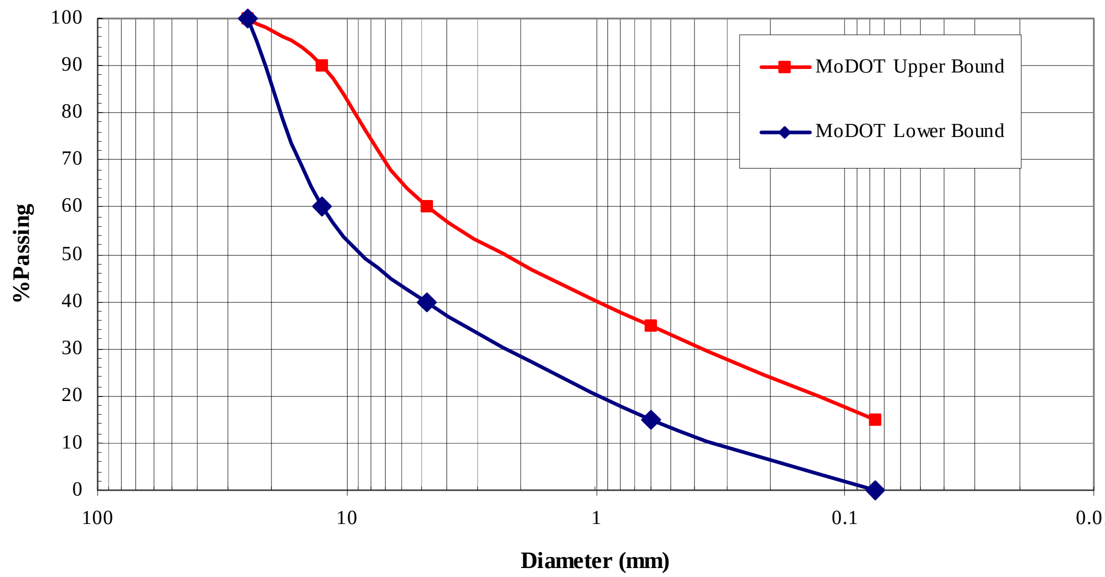


Figure 2 MoDOT specifications for grain size of Type 5 base.

The issue now becomes, what is the hydraulic conductivity of the base system currently being installed beneath the Missouri state roads systems and does it provide adequate drainage of the base?

TECHNICAL APPROACH

Presented in this section are a description of the materials tested, the testing protocols and the methods of analysis.

Materials

Bulk samples of the base materials were obtained from supplier quarries, on-site stockpiles and from compacted, in-place roadway bases around Missouri. Table 1 presents the source and sampling locations of the various materials tested, as well as the tests performed on each. Materials indicating field-sampling locations are materials for which samples of the material were obtained from stockpiles left at the site after compaction was performed.

Samples of the Type 5 base included the complete gradation as used in the field. In the case of the 2-foot rock fill alternatives, samples retrieved for laboratory testing consisted of the overlying working surface that is typically placed on top of the rock fill by the contractor in order to establish a working surface. The make up of the working surface is essentially Type 5 gradation material and ranges in thickness from about 2 inches to 4 inches. The laboratory samples labeled "2-foot rock fill" or "alternative rock base" are actually samples of the material used to establish the working surface and are not the rock fill itself.

Table 1 Location of Base Course Samples and Details of Testing Performed.

Location	Dates	Tests Performed	Source	Sampling location
Rt. 71, McDonald Co.[1]	Sept. 2001	DS, WS, CHP, DRI	Lanagan Quarry	quarry, field
Rt. 13, St. Clair Co.[1]	Sept. 2001	DS, WS, CHP, DRI	Ash Grove Quarry	quarry
Rt. 63, Randolph Co.[1]	Sept. 2001	DS, WS, CHP, DRI	Riggs Quarry	stockpile at site
Rt. 71, Nodaway Co.[1]	Sept. 2001	DS, WS, CHP, DRI	Idecker Quarry	stockpile at site, field
Taney Co.[2]	Dec. 2001	DS, WS, CHP, DRI	Journegan Quarry	stockpile at site
Crawford Co.[2]	Dec. 2001	DS, WS, CHP, DRI	Unknown	stockpile at site

[1] Type 5 Base
[2] Rock Base
DS = dry sieve analysis; WS = wet sieve analysis, CHP = laboratory hydraulic conductivity testing using constant head permeameter; DRI = in-situ hydraulic conductivity testing using double ring infiltrometer

Index Properties

Grain size distributions were performed on samples recovered from all the sampling locations. The samples were brought to the laboratory in 5 gallon buckets, and were retrieved, hand mixed, left to air dry, and quartered to get specimens of approximately 3.3 lb. Dry sieving was performed in two steps: first, the specimen was sieved through 1 in, 0.5 in, No. 4, No. 10, and No. 20 sieves using a mechanical shaker for 5 minutes. Material remaining in the pan was sieved again through the No. 40, No. 60, No. 100, and No. 200 sieves for 5 more minutes. During the testing, it was noticed that even after sieving, fine materials were still adhered to the coarser particles, so wet sieve analyses were performed to determine the complete amount of fines in the specimens. For the wet sieve analyses, samples of approximately 3.3 lb were placed overnight in an oven at 110^o C (230^o F), and washed through a No. 200 sieve. The retained soil was then oven dried to determine the amount of fines lost during washing. Dry sieving was performed as stated above on the washed sample to determine its complete grain size distribution.

Coefficients of uniformity and gradation were determined for all the materials, including materials gathered from the field after compaction. The coefficients were determined using wet sieve analysis information. Since no hydrometer testing was performed, the coefficients were calculated based on extrapolated grain size distribution curves for sizes finer than the No. 200 sieve (0.074 mm).

MoDOT provided compaction curves for most of the materials tested. MU prepared compaction curves for materials where no MoDOT compaction information was available (Ash Grove, Lanagan, Crawford, and Taney). MoDOT provided compaction curves for the Ash Grove and Lanagan materials; but, since strength testing would be performed on these materials, it was determined beneficial to add additional data points to the existing compaction curves. No compaction curves were provided for the Crawford and Taney materials, both of which represent the rock fill alternative. Compaction was performed based on standard Proctor energy, method C, as listed in ASTM 698 (ASTM 2000a). Void ratios were determined, assuming a specific gravity of 2.65.

Hydraulic Conductivity (Tests on Laboratory Compacted Specimens)

The second step of the testing program included the determination of the laboratory hydraulic conductivity of the different materials using a constant head, rigid-wall permeameter (CHP) (Figure 3). Specimens (6-inch diameter, 4.5-inch height) were prepared by compacting the material at maximum dry density and optimum water content using standard Proctor energy, based on method C from ASTM D698 (ASTM, 2000a).

The material labeled “2-foot rock fill” was actually the material used to make up the working surface on top of the 2-foot rock fill. The material was similar to and typically the same gradation as the Type 5 base. It is believed to be the Type 5 material in most cases. The actually “rock fill” was not tested in the laboratory.

Compacted specimens were placed in the permeameter, which provides flow under a constant head. Flows per unit area were measured for different hydraulic gradients. The hydraulic conductivity of the specimen was determined by fitting a best-fit straight line to the flow per unit area vs. hydraulic gradient plot.

During testing, it was noticed that the sensitivity of the constant head hydraulic conductivity device is about 10^{-4} cm/sec, below which accurate hydraulic conductivity measurements were difficult to discern. In cases where no flow was measured using the CHP, the specimens were removed from the CHP, set up and tested in a flexible-wall permeameter (Figure 4) (ASTM D5084 2000b). The flexible wall permeameter allows for the determination of hydraulic conductivity at higher hydraulic gradients which facilitate measurement of low hydraulic conductivity values. The flexible wall device also permits testing the base under various effective confining stresses. An average effective confining stress of 3 psi was used for the specimens tested in the flexible wall permeameter.

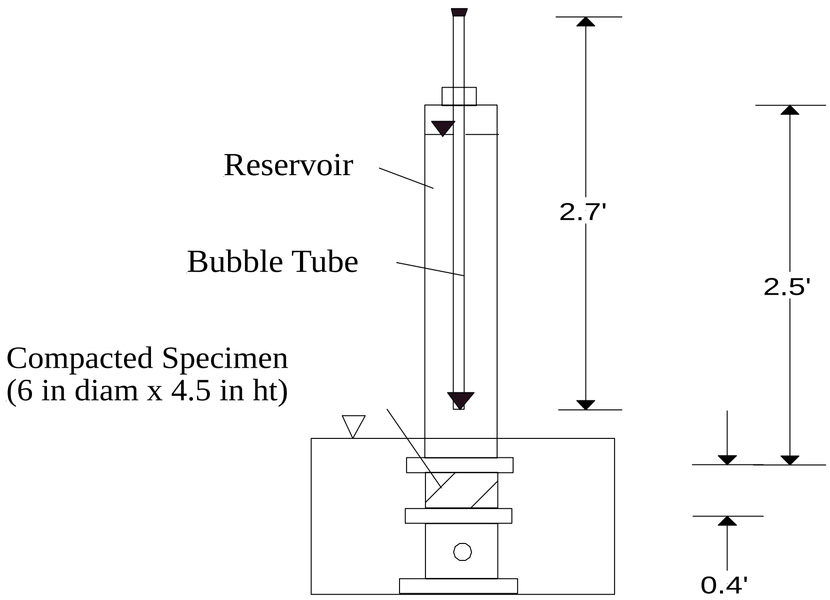


Figure 3 – Constant-head hydraulic conductivity (CHP) apparatus

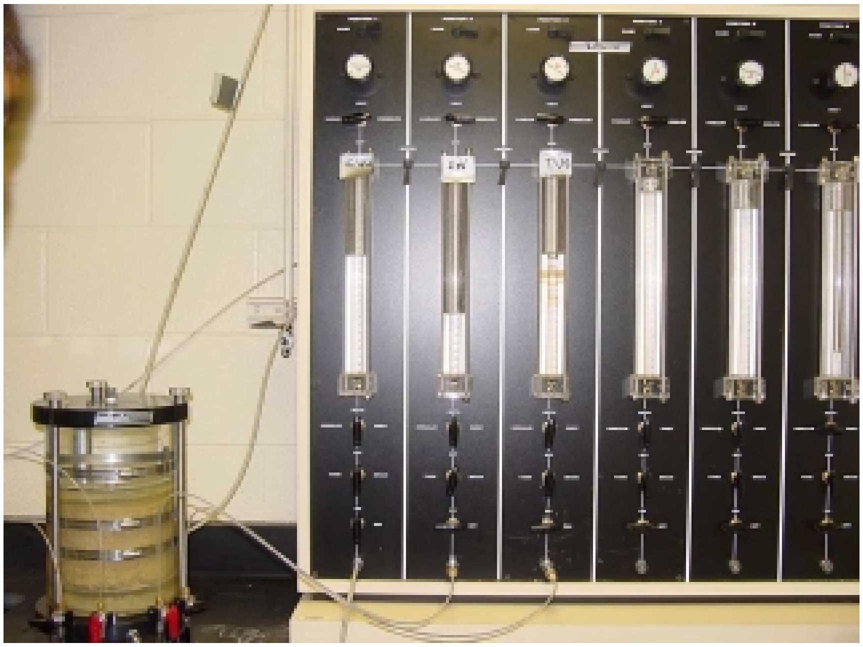
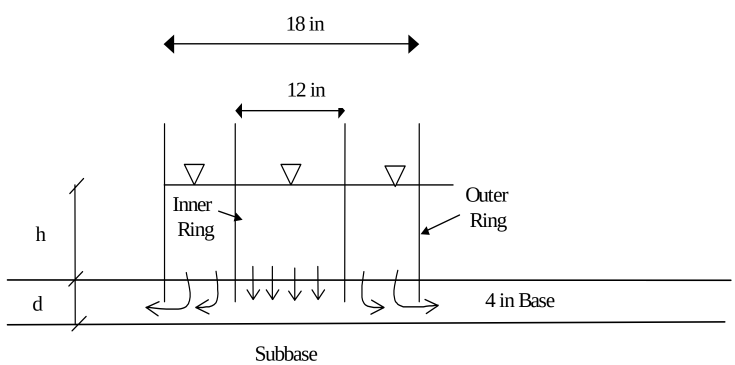


Figure 4 – Flexible-wall permeameter setup and panel board.

Hydraulic Conductivity (Field Tests)

In-situ measurements of hydraulic conductivities were performed using a double-ring infiltrometer (DRI) (Figure 5). The DRI provides a direct determination of the infiltration rate in the material being tested (D3385 ASTM 1994) The hydraulic conductivity can be calculated from the infiltration rate.

(a) Field set up of double-ring infiltrometer.



(b) Elevation schematic of double-ring infiltrometer.

Figure 5 Double Ring Infiltrometer: (a) field setup and (b) elevation schematic of double rings.

The double-ring infiltrometer consists of an inner (12-inch diameter) ring, an outer (18-inch diameter) ring, and two Marriotte bottles. Each ring is embedded approximately two inches into the base by carefully hand-excavating a slot, inserting the ring, and filling the annulus with bentonite paste. The inner ring and annulus space between the two rings are filled with water, and the water level is kept constant during the test by using the Marriotte bottles. These tubes also allow for the measurement of the volume of infiltrated water. The main purpose of the outer ring is to provide a continuous ring of water around the flow from the inner ring so that the water from the inner ring only flows vertically. This provides a one-dimensional flow regime in the base material and facilitates calculation of the hydraulic conductivity. The volume of water infiltrated for a measured time is recorded, and the test is stopped when the infiltration rate becomes steady (saturation).

Infiltration rate is defined as volume of water per time per unit area perpendicular to flow, or

$$I = \frac{Q/t}{A} \tag{1}$$

where I is the infiltration rate [L]/[T], Q is the volume of infiltrated fluid [L^3], t is the time for the volume of water to infiltrate the base [T], and A is the cross-sectional area [L^2]where the water was infiltrated the base.

The hydraulic conductivity can be determined by dividing the infiltration rate by the hydraulic gradient. For this study, full-depth saturation (4 inches) was assumed and the hydraulic gradient was taken as:

$$i = (h+d)/d \tag{2}$$

where h is the height of water ponded above the base [L], and d is the thickness of the base [L] (Figure 2). The hydraulic conductivity is then calculated as

$$k = \frac{I}{i} \tag{3}$$

where k is the hydraulic conductivity [L]/[T], I is the measured infiltration rate and i is the hydraulic gradient.

Hydraulic Conductivity (Prediction Methods)

Hydraulic conductivity values obtained from in-situ and laboratory tests were compared with values calculated using empirical equations. Three different methods were used to empirically estimate the hydraulic conductivity of the materials tested. All of the methods rely on the grain size distribution of the base. The methods included:

A. Hazen Equation: $$k = C\,D_{10}^{2} \qquad (4)$$

where D_{10} represents the size at which 10 percent by dry weight of the sample is smaller, in mm, and the hydraulic conductivity has units of cm/sec, and C is an empirical coefficient ranging from 1 to 1.5 (Hazen 1930).

B. Sherard Equation: $$k = 0.35D_{15}^{2} \qquad (5)$$

where D_{15} represents the size at which 15 percent by dry weight of the sample is smaller, in mm, and the hydraulic conductivity has units of cm/sec(Sherard et al 1984).

C. Moulton Equation: $$k = \frac{6.214x10^{5}\,D_{10}^{1.478}n^{6.654}}{P_{200}^{0.597}} \qquad (6)$$

where n represents the porosity of the material, P_{200} represents the percent of material finer than the No. 200 sieve (0.075 mm and hydraulic conductivity has units of feet/day (Moulton 1980). For this study, information based on the wet sieve analysis was used with these equations to predict the hydraulic conductivity.

Assessment of Drainage Characteristics

The drainage quality of the base materials was studied using a pavement design computer code. *Drainage Requirements in Pavements* 2.0 (DRIP 2.0) (Applied Research Associates, 2002) is a computer program developed by the United States Department of Transportation and the Federal Highway Administration (FHWA) for the design of subsurface drainage. The code is an automated version of the FHWA's Highway Subsurface Drainage Manual prepared by L. K. Moulton (1980). The DRIP 2.0 provides a rating (Excellent to Poor) of a pavement's drainage system based on variables including infiltration, pavement geometry, and drainage characteristics of the pavements' substructure.

For this study, a sensitivity study was performed to determine which design variables would have a larger impact on the drainage characteristics of the tested materials. The width of the road was assumed to be 28 ft and the distance from the edge of the pavement to the edge drain was assumed to be 4 ft. The average unit weight and specific gravity were assumed to be 135 lb/ft^3 and 2.65, respectively. The geometry of the road was kept constant, while the longitudinal slope, transverse slope, and infiltration rates were varied.

After the sensitivity study, the drainage quality of the different materials was determined using a constant infiltration rate, 0.67 units, and a 2-inch rain event. The geometries used included a cross sectional slope ranging from 0.005 to 0.020, and a longitudinal slope ranging from 0.005 to 0.040. Four different values of hydraulic conductivities were used: laboratory and field measured values, and hydraulic conductivities based on dry and wet grain size analyses. The DRIP program allows the user to input the grain size distribution of the material to determine its hydraulic conductivity based on a statistical relationship, or to input values of hydraulic conductivity. The times to drain for the resulting scenarios for the four hydraulic conductivities were determined and rated (by DRIP 2.0) for pavement performance. The code analyzes all the variables based on the 1986 AASHTO *Guide for Design of Pavement Structures* and allows for the determination of the time to reach 50 percent drainage under a constant infiltration rate.

Shear Strength Testing

The effective stress shear strength properties of the Lanagan and Ash Grove Quarries base materials were determined by conducting Consolidated-Undrained type triaxial tests with pore pressure measurement ($\overline{CU}$). This test requires 100 percent saturation of the sample prior to the consolidation and shearing stages. Although base materials are not designed to experience saturation conditions, the intention of the tests was to mimic a worst-case scenario consisting of a poorly-draining material that will eventually reach saturation. Aggregate samples were compacted at their optimum water content using standard Proctor energy to form specimens of 2.9-inch in diameter and 5.6-inch in height. The maximum particle size used for triaxial testing was limited by the size of the compaction mold to an overall particle diameter of 0.5-in. This maximum particle size avoids bridging problems of larger particles in the compaction mold. The material was sieved through 0.5-inch sieve openings to eliminate the larger particles. On average only 2 to 3 particles were removed from the mass used to compact each specimen. Given these conditions, the nature of the materials was deemed not impacted by removal of the largest particles. $\overline{CU}$ tests were performed at strain rates of 1.5 percent per hour on Lanagan samples, and 10%/hr on Ash Grove samples. These rates were determined from time to reach 100 percent consolidation as determined during the consolidations stage of the triaxial tests. Both rates ensure no build up of excess porewater pressure during shearing. Both materials were sheared under effective consolidation stresses of 2, 4, and 6 psi, which are representative of the stress levels close to the ground surface. All samples were sheared to a maximum strain of 30 percent.

Cyclic Triaxial Tests:

Cyclic triaxial tests were performed only on aggregate from the Lanagan quarry. Sample dimensions, gradation, test preparation, and saturation process followed the same procedures as for the $\overline{CU}$ type triaxial tests. In order to mimic the loading pattern as close as possible (given existing test equipment limitations), the strain rate was set to 1000 percent per hour. This was an attempt to approximate the loadings which a base might receive as traffic passes by on the pavement above.

Stress-controlled tests were set to stop when reaching either of the following criteria: a maximum strain of 20 percent or one hundred loading cycles. Samples at an effective consolidation stress of 2-psi were sheared to stress levels corresponding to 40 percent, and 60 percent of the maximum principal stress difference as determined from static $\overline{CU}$ tests, respectively; and reloaded at about 5 percent to 10 percent of the maximum principal stress difference.

Strain-controlled tests were set to shear samples to 100 load cycles at low strains in the range of 1% to 4%, that are more representative of field conditions. A strain rate of 1000 percent per hour was again used.

RESULTS AND DISCUSSION

The results of the laboratory hydraulic conductivity testing, field hydraulic conductivity testing, hydraulic conductivity estimation, and drainage time analyses are presented and discussed in this section. The results of the preliminary shear strength of the base are also presented and discussed in this section of the report.

Material Properties

Wet and dry grain size analyses were performed on all the sampled materials. As seen in Figure 6, all materials meet MoDOT specifications for fines content based on dry sieve analyses. Results obtained based on wet sieve analyses (Figure 7) show some materials have more fines content than MoDOT allows by specification. These materials include the Type 5 base material from Riggs Quarry, and the materials gathered

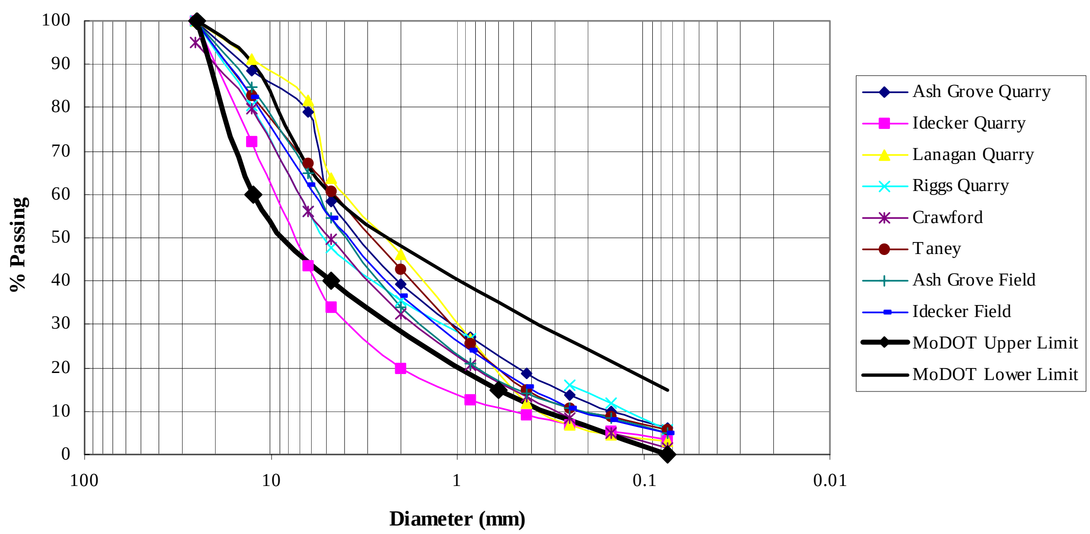


Figure 6 Grain size distribution results based on dry sieving method.

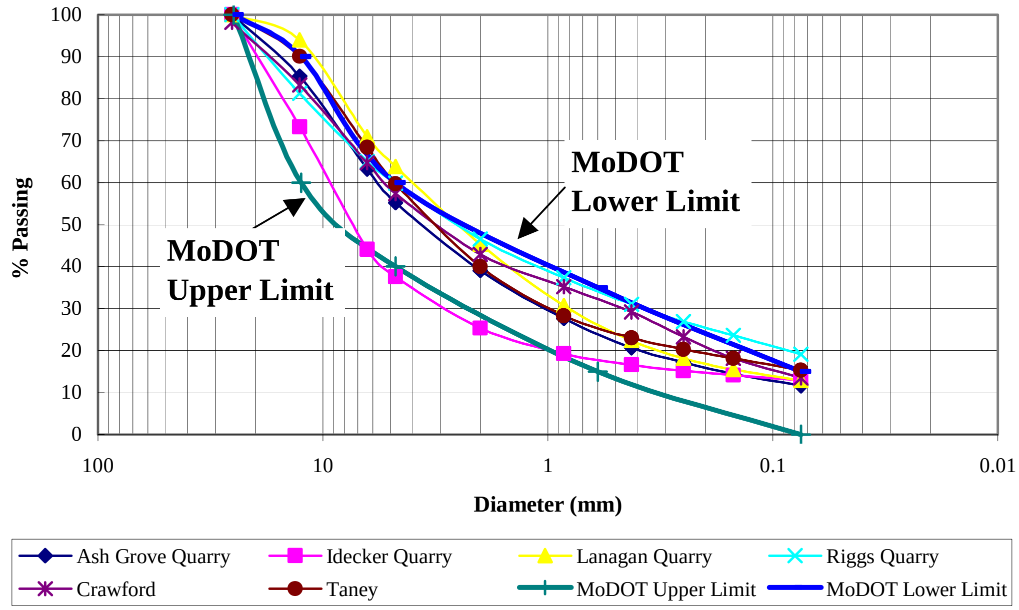


Figure 7 Grain size distribution results based on wet sieving method.

at the field (Ash Grove Field and Idecker Field) (Table 2). Coefficients of uniformity (C_u) and curvature (C_z) are presented in Table 2, as well as the amount of fines determined by dry and wet sieve analyses, as well as the void ratio for each material.

The materials gathered from the field were sampled from stockpiles remaining after compaction of the base material and thus experienced more handling than the samples gathered at the quarries or from stockpiles at the site. Both rock fill alternate materials have high fines content, 14 percent and 15 percent. However, what is refered to as “rockfill alternate” is actually the material used to make up the working surface atop the actually rock fill. The material tested in this investigation meets Type 5 gradation criteria. As shown by these results, it might be expected that after compaction, the materials tested on this study will have higher fines content than what is specified, further compromising the drainage ability of the in-place base layer.

Results of fines content based on wet and dry sieving are compared in Figure 8. Dry sieving results always provided lower amounts of fines than wet sieving. Average values for the percent fines determined by dry sieve analysis and wet sieve analysis were found to be 4 and 15 percent respectively, with standard deviations of 1.6 and 2.6, respectively. The differences in fines content between materials are expected due to the different sources of the materials and differences in the amount of handling. These findings further illustrate the need to use wet sieve analysis when classifying fines content and also to sample *after* the material is on site and when possible after the material has been placed. If fines contents are reported at the quarry, then allowances should be made for generation of additional fines until he material is in place (Daniel and Koerner 1995).

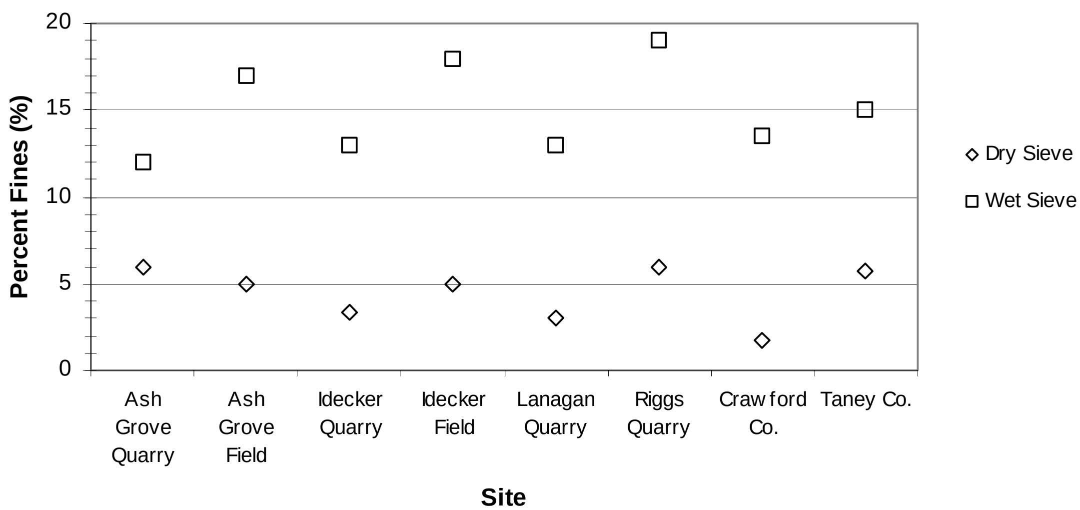


Figure 8 Percent fines by dry sieving and wet sieving, Type 5 base and alternate rockfill, from various sampling locations.

Table 2 Material Properties, Type 5 Base and Material from the Working Surface above the Rock Fill Alternate (Wet Sieve)

Source	D_{60} (mm)	D_{30} (mm)	D_{15} (mm)	D_{10} (mm)	C_u	C_z	% Pass #200	γ_{dmax} (pcf)	OMC (%)	Void Ratio	Soil Classification USCS	AASHTO
Ash Grove Quarry	5.3	1	0.20	**0.05**	106.0	3.8	12	136.5	7.0	0.21	GP-GM	A-1-a
Ash Grove Field	3.8	0.5	**0.10**	**0.02**	190	3.3	17	136.5	7.0	0.21	SM	A-1-b
Idecker Quarry	9.1	2.7	0.25	**0.02**	455.0	40.1	13	125.0	10.0	0.32	GM-GC	A-1-a
Idecker Field	8.0	1.3	**0.03**	**0.01**	800	21.1	18	125.0	10.0	0.32	GM/GM-GC	A-1-b
Lanagan Quarry	4.0	0.8	0.10	**0.04**	100.0	4.0	13	141.0	6.5	0.17	SM	A-1-a
Riggs Quarry	4.8	0.4	**0.04**	**0.02**	237.5	1.7	19	137.0	8.0	0.21	SM	A-1-b
Crawford Co.	5.1	0.5	0.10	**0.05**	102.0	1.0	14	138.7	8.0	0.19	SM	A-1-a
Taney Co.	4.8	1	0.08	**0.02**	237.5	10.5	15	138.9	7.7	0.19	SM	A-1-a

Uniformity Coefficient, $C_u = \dfrac{D_{60}}{D_{10}}$

Curvature Coefficient, $C_z = \dfrac{(D30)^2}{D_{10}D_{60}}$

Compaction curves were prepared for the Ash Grove Quarry material, Lanagan Quarry material, Crawford rock fill alternate and Taney rock fill alternate. The compaction curves are presented in Figure 9. The optimum moisture contents ranged from 6 to 8 percent dry weight basis and the maximum dry density ranged from 136 to 142 pcf. The maximum dry unit weights for the materials from Idecker (quarry and field) were lower (by 10 to 15 pcf) than the other samples. The Idecker materials also exhibited a higher optimum moisture content (Table 2). The fines content was not very different from that of the other samples. No explanation is available for the difference

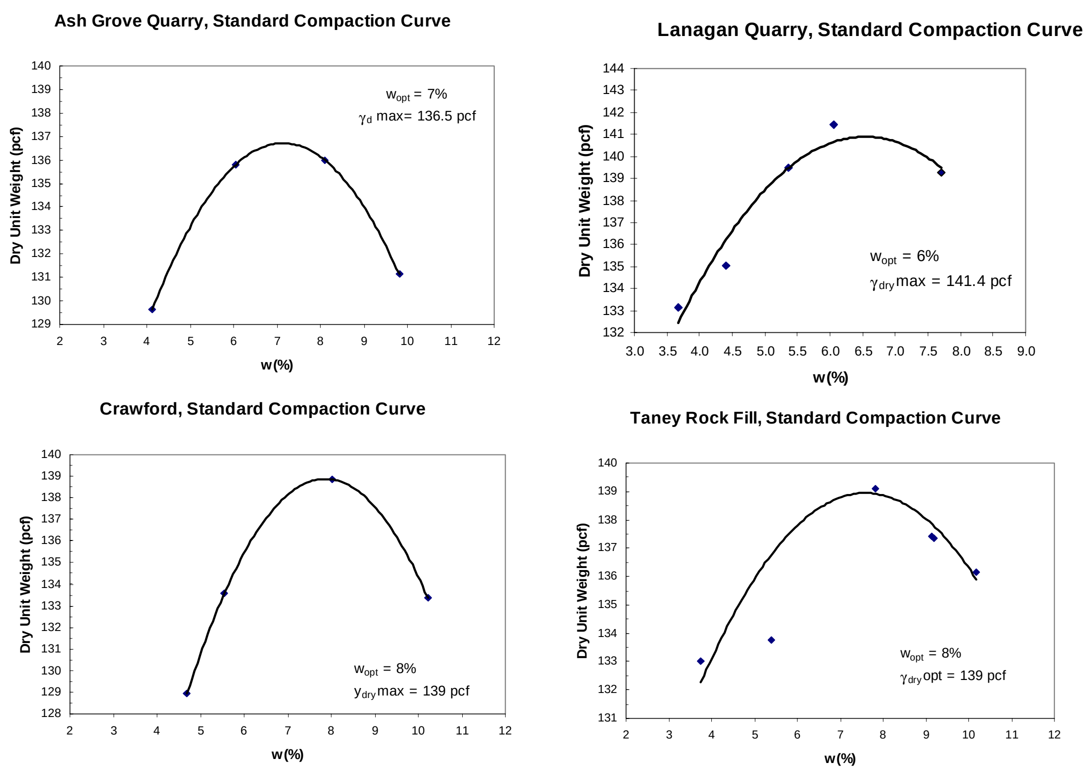


Figure 9 – Compaction Curves, Standard Proctor Energy, ASTM D698, Method C. Note: the Taney rock fill is actually the material from the working surface above the rock fill.

Hydraulic Conductivity

Hydraulic conductivity of the base materials was measured on laboratory compacted samples, in situ, and predicted using empirical equations. The results of the hydraulic conductivity testing and predictions are summarized in Table 3. The hydraulic conductivities determined using each method are discussed individually. A comparison is made between the laboratory and field measured values. The section concludes with a comparison between the predicted and measured hydraulic conductivities.

Table 3 Predicted, Laboratory-Measured and Field-Measured Hydraulic Conductivities of Base Courses

				Predicted k				
Source	**γ_d (kN/m^3)**	**w (%)**	**Void Ratio**	**Hazen k (cm/sec)**	**Sherard k (cm/sec)**	**Moulton k (cm/sec)**	**Lab k (cm/sec)**	**Field k (cm/sec)**
Ash Grove Quarry	20.6	8.0	0.21	2.5×10^{-3}	1.4×10^{-2}	5.4×10^{-6}	2.8×10^{-3}	1.9×10^{-3}
Ash Grove Field	21.2	8.0	0.21	4.0×10^{-4}	3.5×10^{-3}	1.1×10^{-6}	**3.0×10^{-6}**	1.9×10^{-3}
Idecker Quarry	18.8	9.0	0.32	4.0×10^{-4}	2.2×10^{-2}	1.2×10^{-5}	8.8×10^{-2}	4.6×10^{-5}
Idecker Field	21.8	9.0	0.32	1.0×10^{-4}	3.2×10^{-4}	3.6×10^{-6}	**3.0×10^{-7}**	4.6×10^{-5}
Lanagan Quarry	21.0	4.0	0.17	1.6×10^{-3}	3.5×10^{-3}	1.2×10^{-6}	5.4×10^{-3}	9.7×10^{-5}
Riggs Quarry	21.7	9.0	0.21	4.0×10^{-4}	5.6×10^{-4}	9.4×10^{-7}	5.2×10^{-3}	3.7×10^{-5}
Crawford Co.	21.4	8.8	0.19	2.5×10^{-3}	3.5×10^{-3}	2.9×10^{-6}	**4.5×10^{-6}**	9.1×10^{-5}
Taney Co.	22.7	7.9	0.19	4.0×10^{-4}	2.0×10^{-3}	6.8×10^{-7}	3.0×10^{-4}	1.9×10^{-5}

Predicted values are based on wet sieve grainsize analyses.
Values in bold represent flexible wall permeameter tests.
0.157 kN/m^3 = 1 lb/ft^3
1 cm/sec = 2835 ft/day

Hydraulic Conductivity (Tests on Laboratory Compacted Specimens)

Hydraulic conductivity values for most of the materials tested were determined using a rigid-wall, constant-head permability device. Hydraulic conductivites measured using this device ranged from 8.8×10^{-2} cm/sec to 2.8×10^{-4} cm/sec. The sensitivity of the rigid-wall, constant head permeameter is about 10^{-4} cm/sec; the hydraulic conductivity of any material with a lower hydraulic conductivity will not be discernable in this permeameter.

Materials that did not show any flow in the rigid-wall, constant-head permeameter were tested using a flexible-wall permeameter. Hydraulic conductivities in bold in Table 3 were determined with the flexible-wall permeameter. These hydraulic conductivities ranged from $3x10^{-6}$ cm/sec to $3x10^{-7}$ cm/sec. Overall, the laboratory compacted specimens showed hydraulic conductivities from $1x10^{-1}$ cm/s to 1 $x10^{-7}$ cm/s. Some of this variation can be attributed to testing conditions which is discussed in a following section.

Hydraulic Conductivity (Field (*In situ*) Tests)

In-situ hydraulic conductivities were determined using a double-ring infiltrometer which allows for the direct measurement of the infiltration rate for the tested base layer (Table 4). Hydraulic conductivities were calculated by dividing the measured infiltration

Table 4 Measured Infiltration Rates, Hydraulic Gradients and Measured Hydraulic Conductivities Based on an Inner Ring Diameter of 12 inches.

Source	Average Infiltration Rate (cm^3/sec)	Average Gradient	Measured k (cm/s)
Ash Grove Quarry	3.6	2.7	1.9E-03
Ash Grove Field	3.6	2.7	1.9E-03
Idecker Quarry	0.1	2.9	4.6E-05
Idecker Field	0.1	2.9	4.6E-05
Lanagan Quarry	0.15	2.8	9.7E-05
Riggs Quarry	0.083	3.4	3.7E-05
Crawford Co.	0.19	3	9.1E-05
Taney Co.	0.041	3.1	1.9E-05

rate by the hydraulic gradient. The gradient was calculated assuming full-depth saturation of the base. In-situ hydraulic conductivities ranged from $1.9x10^{-3}$ cm/sec to $1.9x10^{-5}$ cm/sec. All of the in situ tests were performed on the Type 5 base or on the working surface applied to the 2-foot rock fill. The working surface was of similar material to the

Type 5. It was not possible to remove all of the working surface from the rock fill in order to test the in situ flow capacity of the rock fill and imbedding the rings into the rock fill would be extremely difficult.

During testing using the double-ring infiltrometer, concerns arose due to the presence of bentonite in the inner-ring area of the double-ring infiltrometer. The bentonite paste was used to ensure a seal between the rings and the surrounding soil; however, the area occupied by the bentonite paste resulted in a decrease in the cross-sectional area available for flow within the inner ring. A sensitivity analysis was performed to determine how the measured hydraulic conductivity varied with decreasing infiltration area by using the measured infiltration rate and varying the infiltration area. The results are summarized in Figure 10. The in situ hydraulic conductivities reported in Table 4 were calculated based on an inner-ring diameter of 12 inches. As shown in Figure 10, even if the effective area of flow were to be decreased by half, to say 6 inches, the calculated hydraulic conductivity would only increase about a factor of 3. Actual effective inner ring diameters were around 10 inches meaning the hydraulic conductivities reported in Table 4 might be about a factor of 2 lower than the actual value. The difference in values still leaves the hydraulic conductivity of the base well below what might be considered freely-draining, e.g., 1 cm/s.

Hydraulic conductivity values were determined in the laboratory and in the field (Table 3, Figure 11). In general, in-situ hydraulic conductivities ranged from 10^{-3} cm/s to 10^{-5} cm/s and averaged about 10^{-4} cm/s. The hydraulic conductivities determined in the laboratory rigid-wall permeameter (CHP) resulted in hydraulic conductivities that were one to three orders of magnitude higher than the in-situ values. The hydraulic conductivities determined in the laboratory using the flexible wall permeameter (FW) were one to three orders of magnitude lower than the in-situ values.

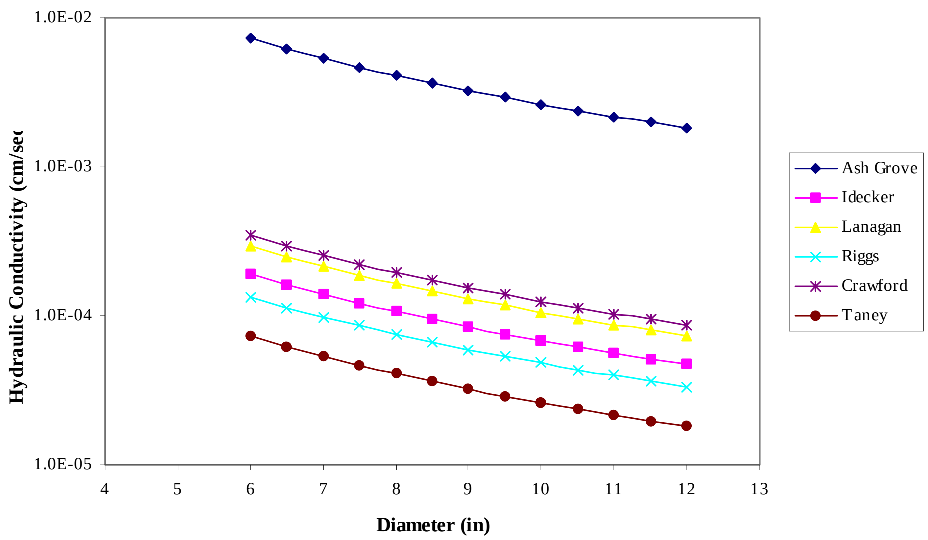


Figure 10 Variation of in-situ hydraulic conductivity with diameter of infiltration area based on measured infiltration rates.

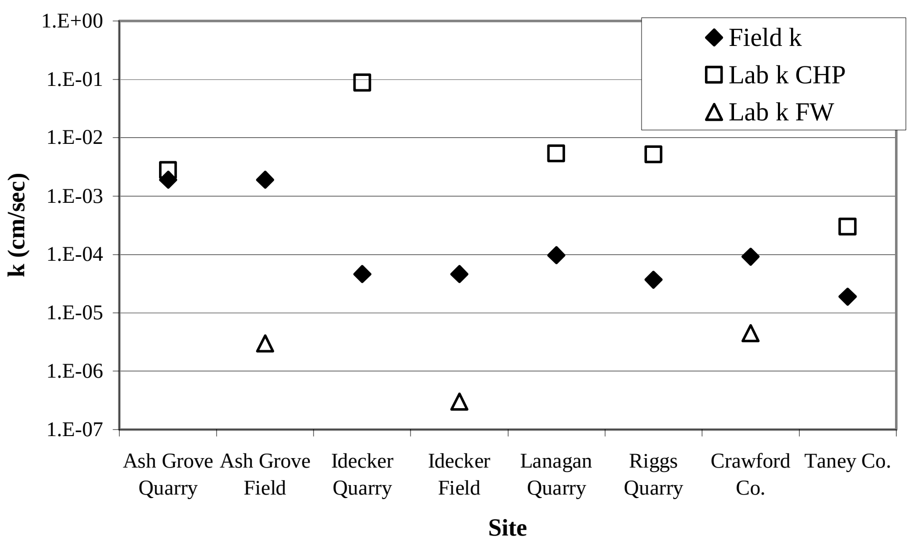


Figure 11 Field and laboratory measured hydraulic conductivities.

In hydraulic conductivity testing, it is typical to give highest weight to the values determined in-situ. Unrepresentative samples and laboratory testing conditions can yield laboratory values that do not represent the in-situ values. In this work highest weight will be given to the in-situ values although it is recognized that that the double-ring infiltrometer is not without fault. Incorrect hydraulic gradient, incomplete saturation, evaporation, limited ability to measure minute quantities of flow and variations in the cross-sectional area of flow all potentially contribute to errors in the measured in-situ hydraulic conductivity. Given the percentage of fine material and the compacted densities, the Type 5 base hydraulic conductivities of 10^{-4} cm/s are realistic. The issue then becomes, why are the hydraulic conductivities measured in the laboratory not representative of those measured in the field?

The laboratory constant head permeameter (CHP) is a rigid-wall permeameter. The sample is compacted in a rigid-steel mold then top and bottom sections are added to the mold to make the permeameter. The effective stress conditions in the specimen are unknown. The only stresses in the specimen are the residual stresses remaining from the compaction process. As the hydraulic gradient is increased in the test, the effective stresses in the specimen are decreased due to the elevated porewater pressures. A decrease in effective stress is typically associated with an increase in the hydraulic conductivity. In addition, the particles in the aggregate base do not make continuous contact with the inner sidewalls of the rigid-mold. This leads to the phenomenon of sidewall leakage, i.e., water travels along the path of least resistance (the interface of the aggregate/rigid mold) and not through the specimen. When sidewall leakage is present, the measured hydraulic conductivity can be significantly greater than the actual hydraulic conductivity of the specimen. These two phenomena, reduced effective stress and sidewall leakage were both likely to be present in the CHP permeameter and contributed to hydraulic conductivities that are higher than the field values.

In a flexible-wall (FW) permeameter the specimen is confined by a flexible membrane. The system allows for application of various effective confining stresses and maintains good contact between the flexible membrane and the sides of the specimen thereby alleviating sidewall leakage. In the FW permeability tests performed in this project, an average effective confining stress of 3 psi was used. This effective stress would be typical of that in a base beneath a paved surface. The increase in effective stress over that in the CHP and also that in the field DRI tests would be expected to produce a slightly lower hydraulic conductivity. However, in the case of the Type 5 base, one would not expect a 3-order-of-magnitude decrease below the in-situ hydraulic conductivities.

Additional reasons for the differences between the laboratory and field-measured hydraulic conductivities include differences in unit weight and the presence of macrostructures. The in-situ tests were made on vibratory compacted base material. The in-situ unit weights were not determined; however, it is believed that the base was compacted at about optimum moisture content. The laboratory specimens were compacted at optimum moisture, using a static compactor and standard Proctor energy (ASTM D698). It is possible that the field specimens achieved a higher unit weight during compaction than the laboratory specimens thus the field permeability would be lower than the laboratory values. However, the laboratory FW tests resulted in the lowest hydraulic conductivity. This may be due to the higher effective stress (3 psi) used in the FW tests that can help to close macrostructures (cracks, voids) which otherwise would tend to yield higher hydraulic conductivities.

As part of the in-situ hydraulic conductivity measurements, one trial was performed using an underdrain lysimeter (Figure 12). The use of the lysimeter enables accurate determination of the hydraulic gradient of the system and subsequently of determination of the hydraulic conductivity. Placement of the lysimeter ensures that the lower boundary of the base is freely draining, i.e., the porewater pressure head is zero. It also allows determination of saturation of the base, i.e., when the outflow volume equals the inflow volume, the base can be assumed saturated in the region of the lysimeter. The lysimeter provides a check on the hydraulic conductivity results from the infiltrometer system. All of the infiltrating water (from both the inner and outer rings) is collected in the lysimeter. The area of infiltration is taken as the area enclosed by the outer ring of the double-ring infiltrometer system. The time for the water to infiltrate is recorded and a hydraulic conductivity is calculated.

Lysimeter installation included the placement of a geomembrane on the subbase, with a geonet-geotextile layer overlying the geomembrane. One of the edges of the geosynthetics was day-lighted on the outer edge of the shoulder area of the road. A half-pipe was installed below the day-lighted edge of the lysimeter to catch all water coming out from the base material through the underlying geonet. After all the materials were in place, the base was placed and compacted, and in-situ hydraulic conductivity testing was performed using the double-ring infiltrometer (Figure 13).

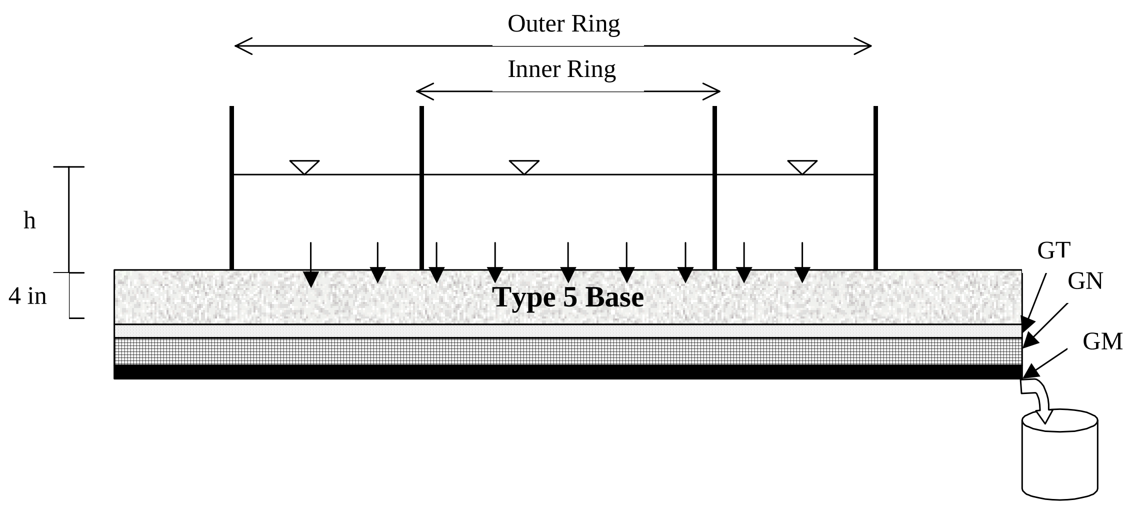


Figure 12 Schematic of underdrain lysimeter and double ring infiltrometer.

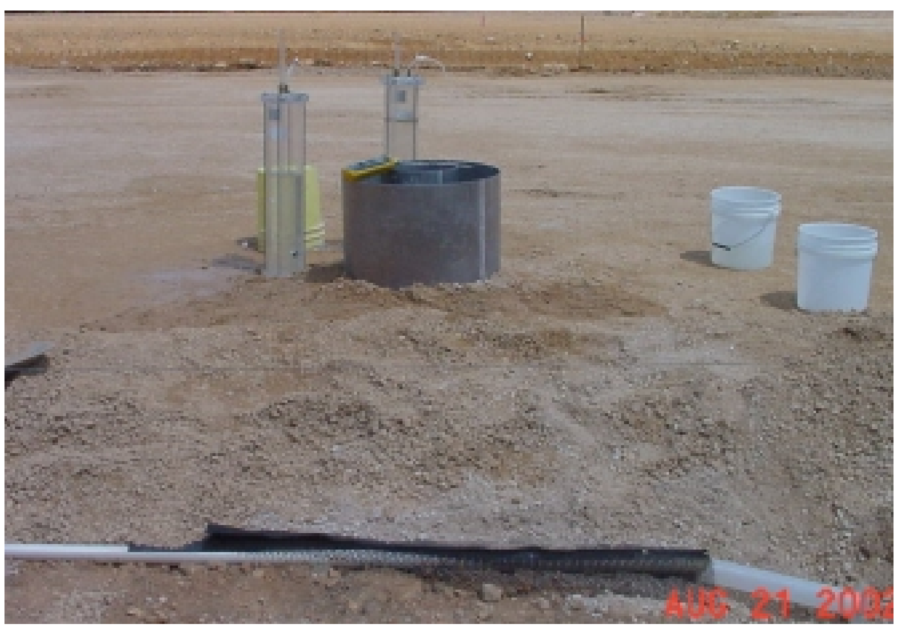


Figure 13 Field setup of underdrain lysimeter and double ring infiltrometer.

Only one lysimeter was installed during the project. A double-ring infiltrometer test was performed on the base overlying the lysimeter. No flow was collected or measured.from the bottom of the base layer. The only water observed in the collection pipe was water due to condensation at the geomembrane interface geonet. Thus, this trial lysimeter test was unable to confirm the results of the infiltrometer studies.

Considering all of the confounding parameters and effects, it is believed that the in-situ hydraulic conductivity of 10^{-4} cm/s is the most representative value for the Type 5 base and the material comprising the working surface above the 2-foot rock fill alternate. Additional infiltration testing including the lysimeter is recommended to both confirm the in situ hydraulic conductivity of the Type 5 and to document the flow capacity of the 2-foot rock alternate.

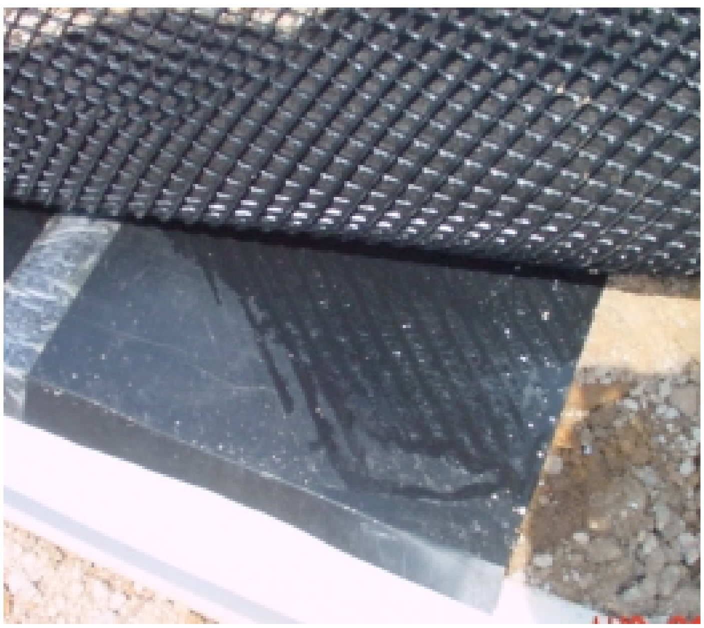


Figure 14 Water resulting from condensation on the lysimeter.

Hydraulic Conductivity (Predicted)

Three different empirical methods were used to predict the hydraulic conductivity of the materials tested. The three methods included the Hazen equation, Sherard equation and Moulton equation. All of these method are based on the grain size distribution of the materials to provide an approximation of the hydraulic conductivity. Measured and predicted hydraulic conductivities are shown in Table 3. The Hazen and Sherard methods result in predictions of hydraulic conductivity ranging from 10^{-2} cm/sec to 10^{-4} cm/sec (29 to 0.3 ft/day), which are in general one to two orders of magnitude higher than the hydraulic conductivities measured in the laboratory using the CHP device (Figure 15). The Hazen correlation was developed for clean sands while the Sherard correlation was developed based on silty soils. Neither provides a good representation for well-graded aggregate with fines. Moulton's expression was the only one specifically developed from correlations with roadway base materials. Estimates of hydraulic conductivities using his expression resulted in values from 10^{-5} cm/sec to 10^{-7} cm/sec (0.03 to $3\text{x}10^{-4}$ ft/day), which is within the range hydraulic conductivities measured in the laboratory using the flexible wall permeameter but several orders of magnitude below the conductivities measured in the CHP device (Figure 15). The predicted and field-measured hydraulic conductivities are shown in Figure 16. The Hazen and Sherard methods tend to over estimate the hydraulic conductivity by one to two orders of magnitude. Moulton's method tends to underestimate the field measured hydraulic conductivity by one to two orders of magnitude.

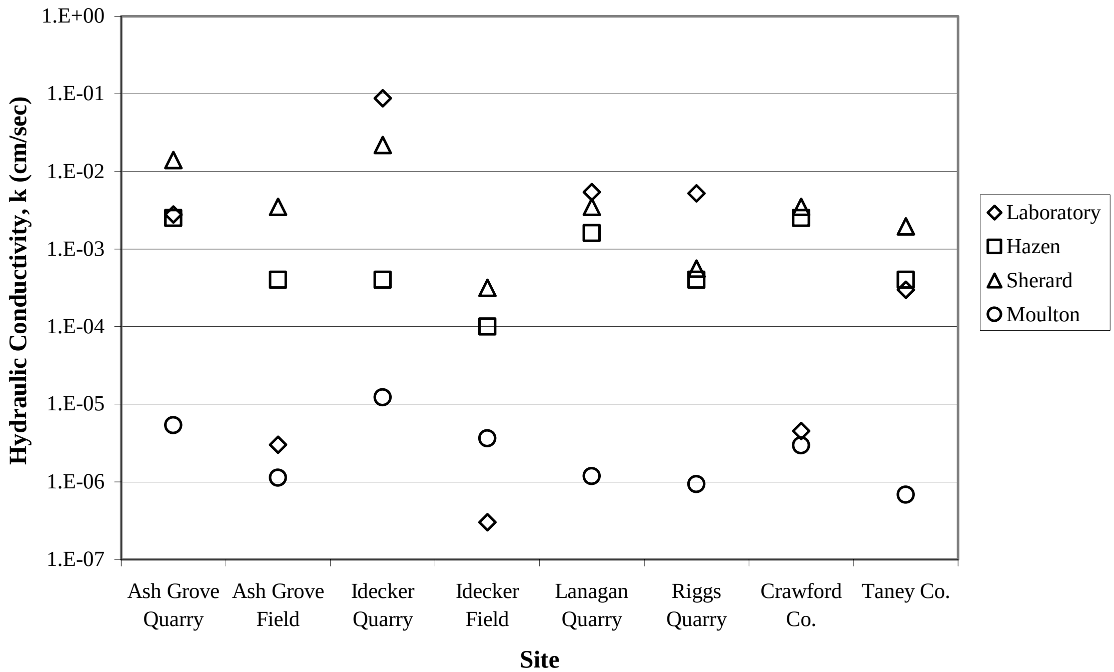


Figure 15 Laboratory measured hydraulic conductivity and predicted hydraulic conductivity (wet sieve) for all sites tested.

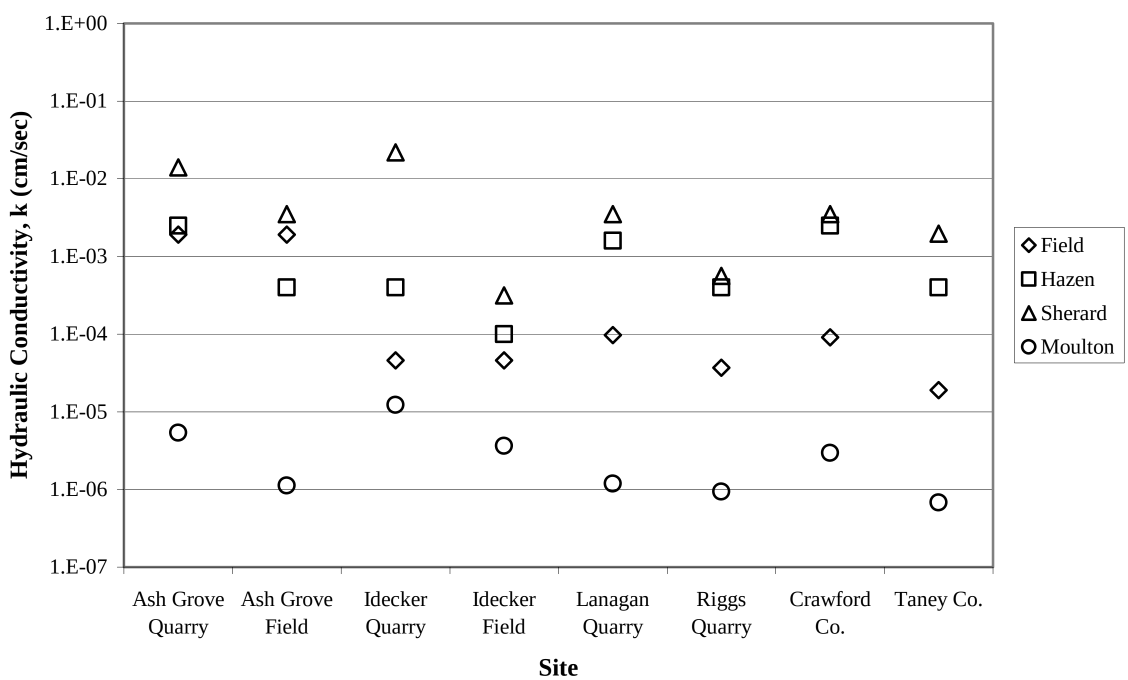


Figure 16 Field measured hydraulic conductivity and predicted hydraulic conductivity (wet sieve) for all sites tested.

Assessment of Drainage Characteristics

The *Drainage Requirements in Pavements Program* (DRIP 2.0) allows for the user to determine the time to drain of a base material by entering information on the geometry of the proposed road, material properties, grain size distribution, and infiltration rate. A sensitivity analysis was performed to determine if changes in the infiltration rate would affect the time to drain of the base material analyzed. It was determined that changing the infiltration rate coefficients did not cause any change in the time to drain for the analyzed material. Results on time to drain based on hydraulic conductivity values determined in the laboratory and in the field are presented in Table 5. These values were determined using a longitudinal slope of 0.01 and a transverse slope of 0.02.

Table 5 DRIP 2.0 Analyses Results

	Measured k values		Time to Drain		Quality of Drainage	
Material	Laboratory k (cm/sec)	Field k (cm/sec)	Laboratory (days)	Field (days)	Laboratory	Field
Ash Grove	$2.8x10^{-3}$	$1.9x10^{-3}$	8.9	13	Poor	Poor
Idecker	$8.8x10^{-2}$	$4.6x10^{-5}$	0.1	253.7	Good	Very Poor
Lanagan	$5.4x10^{-3}$	$9.7x10^{-5}$	2.4	223.3	Fair	Very Poor
Riggs	$5.2x10^{-3}$	$3.7x10^{-5}$	1.1	163.9	Good	Very Poor
Crawford	$4.5x10^{-6}$	$9.1x10^{-5}$	449.5	16.6	Very Poor	Poor
Taney	$3.0x10^{-4}$	$1.9x10^{-5}$	-----	-----	-----	-----

1cm/sec = 2835 ft/day

Time to drain values varied from 0.1 days to 450 days when the laboratory measured hydraulic conductivities were used for the analysis, and from 13 days to 254 days when the in-situ hydraulic conductivities were used for the analysis. As shown in the table, the best quality of drainage achieved by any of the materials was “good”, but most materials presented a "very poor" quality of drainage. Results are not presented for the Taney material due to its high fines content based on wet sieve analysis.

The field measured hydraulic conductivities are more representative of the hydraulic conductivity of the roadway base. Using these hydraulic conductivities, the best drainage quality that could be expected is poor. This means that it could take approximately one month for the base layer to drain 50 percent of the water being infiltrated. It is expected that this material will almost always be wet, increasing the possibility of roadway damage.

Time to drain values were also determined by using the grain size distributions based on both the dry and wet sieve analyses. If these results are taken into consideration as well as the results based on the laboratory and in-situ measured hydraulic conductivities for all the materials and slopes tested, it was found that most of the time to drain values represent very poor drainage quality (Table 6).

Table 6 Quality of Drainage for all materials tested, based on 1986 AASHTO Guide for Pavement Design

	Percentage (%)				
	Very Poor	Poor	Fair	Good	Excellent
Dry Sieve	59.25	40.75	0	0	0
Wet Sieve	100	0	0	0	0
Field k	40.75	53.7	5.55	0	0
Lab k	0	31.11	20	22.22	26.67
Wet Revised	100	0	0	0	0

Static, $\overline{CU}$ Tests

Results of the moisture content before and after testing, and effective stress strength parameters from the static $\overline{CU}$ type triaxial compression tests are presented in Table (7).

Plots of stress difference and change in pore water pressure versus strain, and Mohr-Coulomb failure envelope, for the compacted Type 5 base are presented in Figures 17 through 23. Strength tests performed on the material from Lanagan and Ash Grove Quarries indicated no effective cohesion intercept ($c' = 0$), and an angle of internal friction $\phi' = 42$ and 43 degrees, respectively, that are representative of a highly dense granular soil. The stress difference-strain relationship was non-linear and continuously increasing until about 15 percent strain after which it leveled out. The pore pressures generated during shearing are representative of dense granular soils and help to explain the stress-strain behavior. Positive changes in pore pressures was minimum and was observed at the beginning of the test up to about one percent axial strain, followed by negative changes that steadily continued until about 15 percent to 20 percent strain when they leveled out.

TABLE 7 Static Triaxial Test Results for the Lanagan and Ash Grove Quarry Samples Based on Maximum Stress Difference Criterion

Quarry Location	Sample No.	σ'_c (psi)	γ_d (pcf) compaction	w(%) molding	w(%) during shear	$(\sigma'_1-\sigma'_3)$ peak	(σ'_3) failure	ε (%) peak	(σ'_1/σ'_3) max	ε (%) max	ϕ'	c'
Lanagan	1	2	136.5	6	8.9	97	23.8	16.2	6.9	1.9	42	0
	2	4	137.2	6.5	8.7	115.5	28.8	20.36	8.2	1.45		
	3	6	136.4	6	***	117	28.7	12.7	7.6	1.53		
Ash Grove	1	2	133.9	6.21	13.4	32.7	7.71	23.7	13.8	1.4	42.5	0
	2	4	128	5.25	12	59.44	14.61	15.93	7.76	1.7		
	3	6	130.2	5.99	11.1	76	16.81	27.16	7.65	1.35		

σ'_c is the effective confining stress at the end of the consolidation stage.
γ_d is the effective unit weight of the compacted Type 5 base.
w%molding is the molding water content.
w%during shear is the water content after shearing the specimen.
$\sigma'_1-\sigma'_3$ is the peak stress difference. (deviator stress)
σ'_3 is the effective confining stress at the end of the shearing stage.
ε% peak is the axial strain at peak stress difference.
σ'_1/σ'_3 is the peak stress ratio.
ε%max is the axial strain at peak stress ratio.
ϕ' is the effective internal angle of friction.
c' is the effective cohesion intercept.

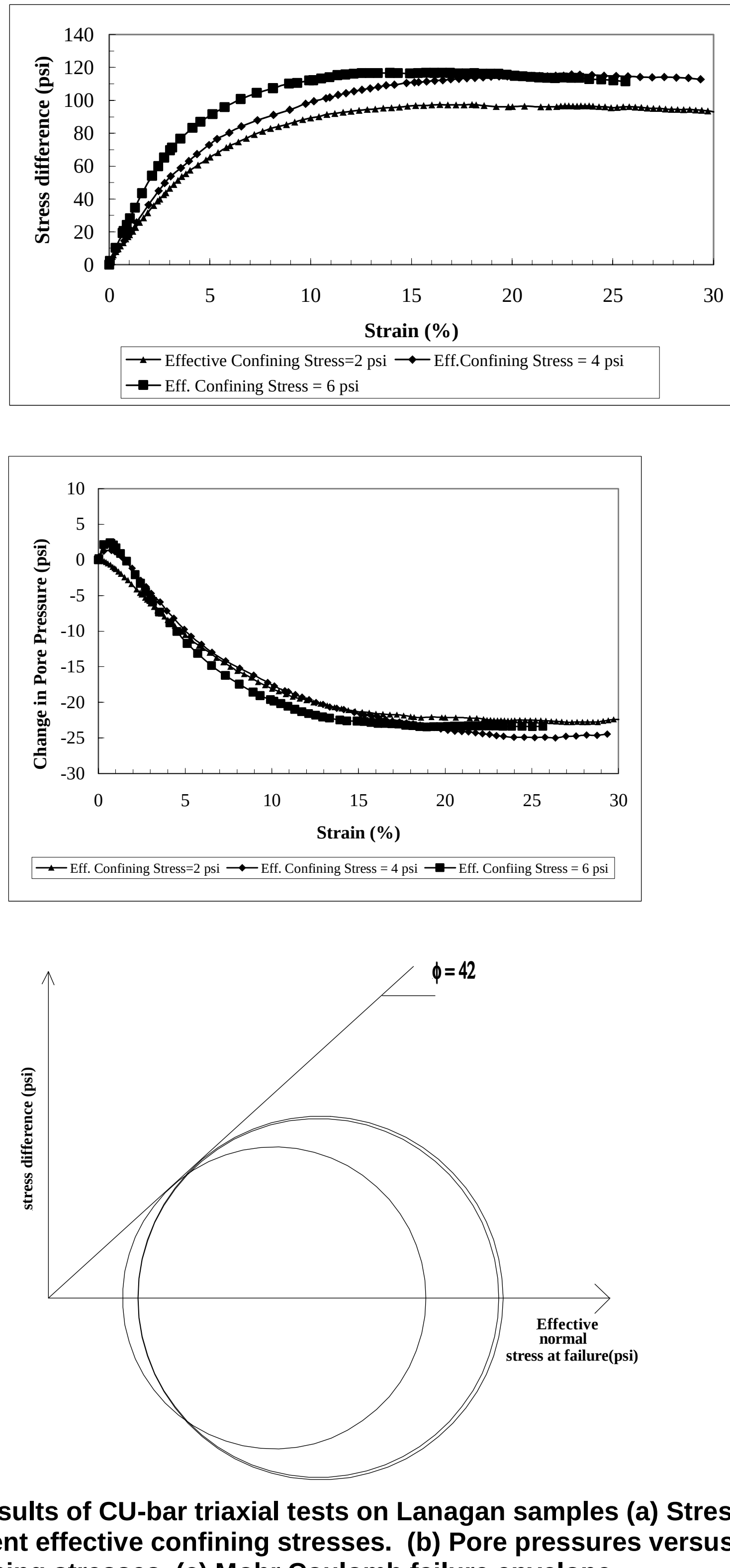


FIGURE 17 Results of CU-bar triaxial tests on Lanagan samples (a) Stress difference versus strain at different effective confining stresses. (b) Pore pressures versus strain at different effective confining stresses. (c) Mohr Coulomb failure envelope

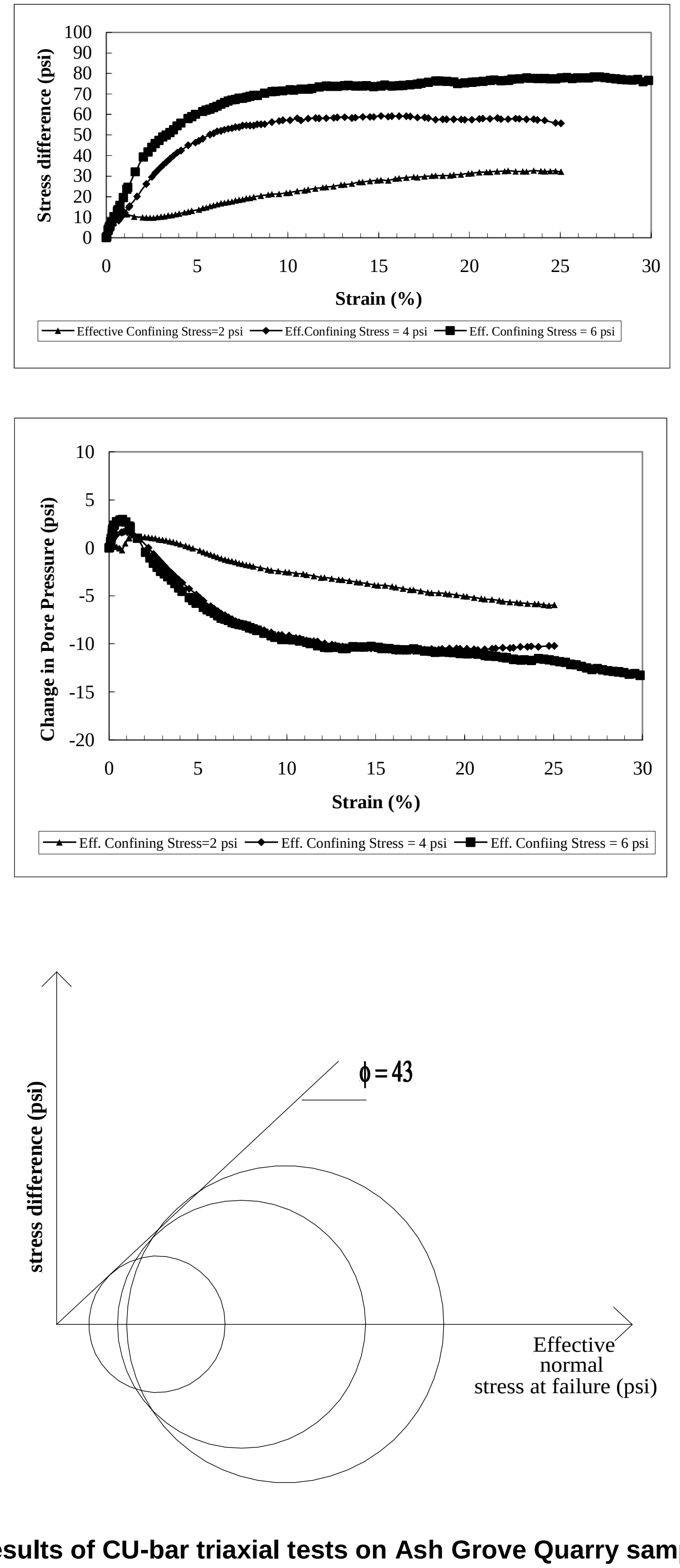
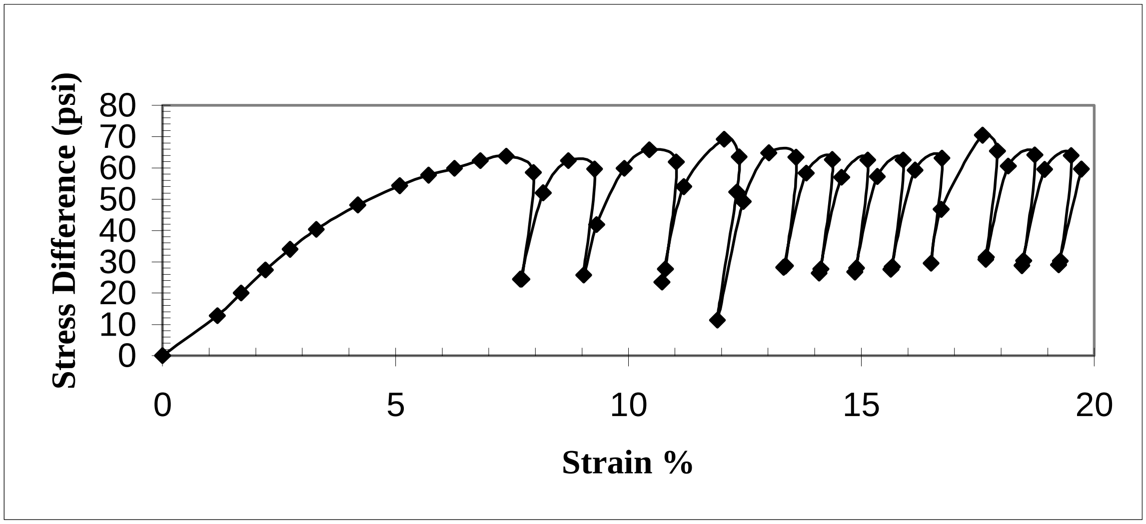


FIGURE 18 Results of CU-bar triaxial tests on Ash Grove Quarry samples (a) Stress difference versus strain at different effective confining stresses. (b) Change in pore pressures versus strain at different effective confining stresses c) Mohr-Coulomb failure envelope



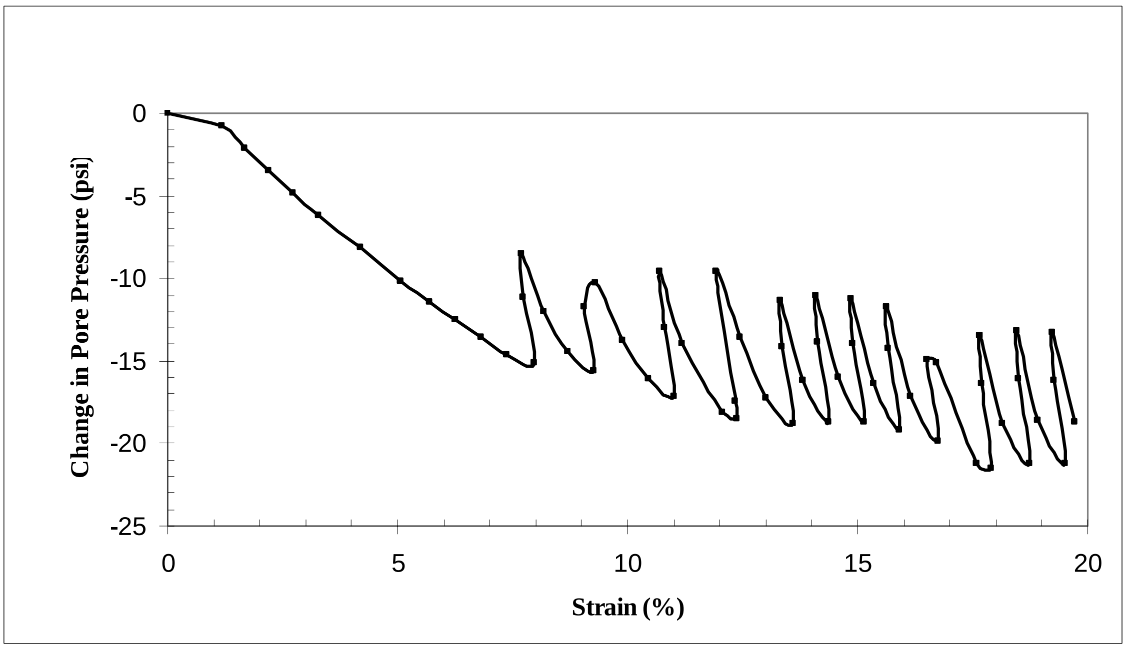
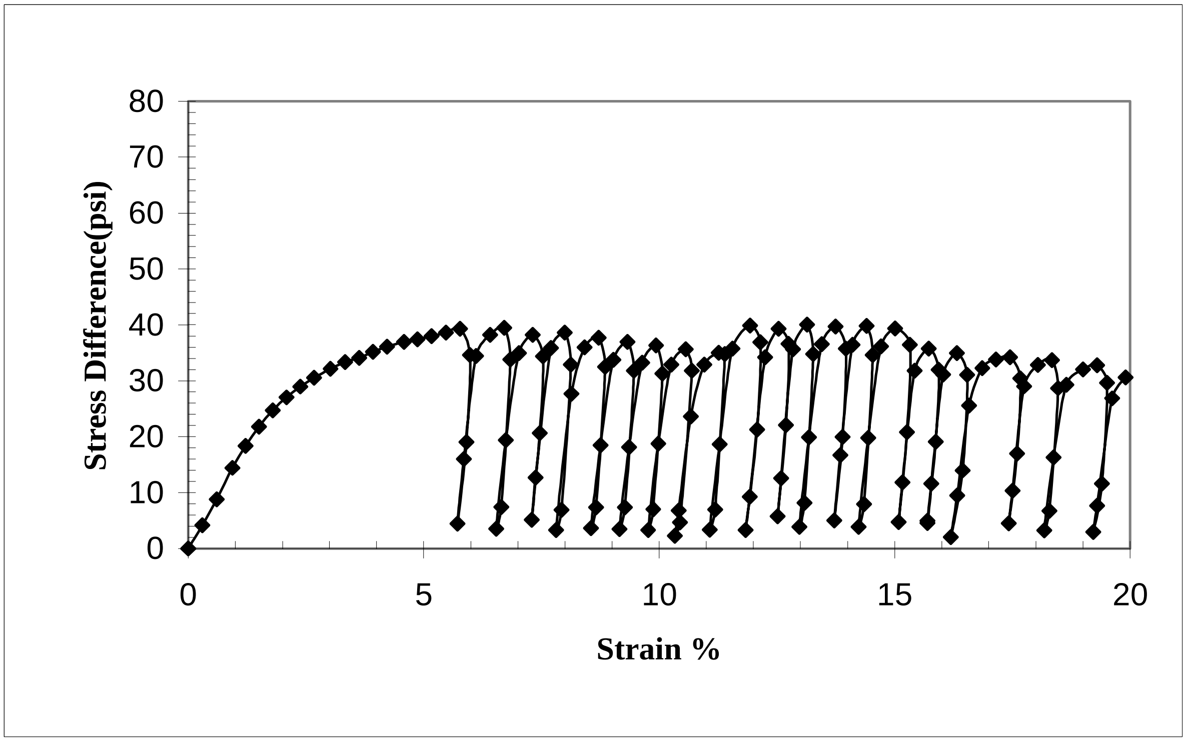


FIGURE 19 Lanagan Samples: Stress-Controlled, Cyclic Triaxial Tests at Effective Consolidation Stress Stress = 4-psi, sheared between 60% and 20% of the maximum stress difference. a) Stress Difference versus Strain, b) Pore Pressure versus strain



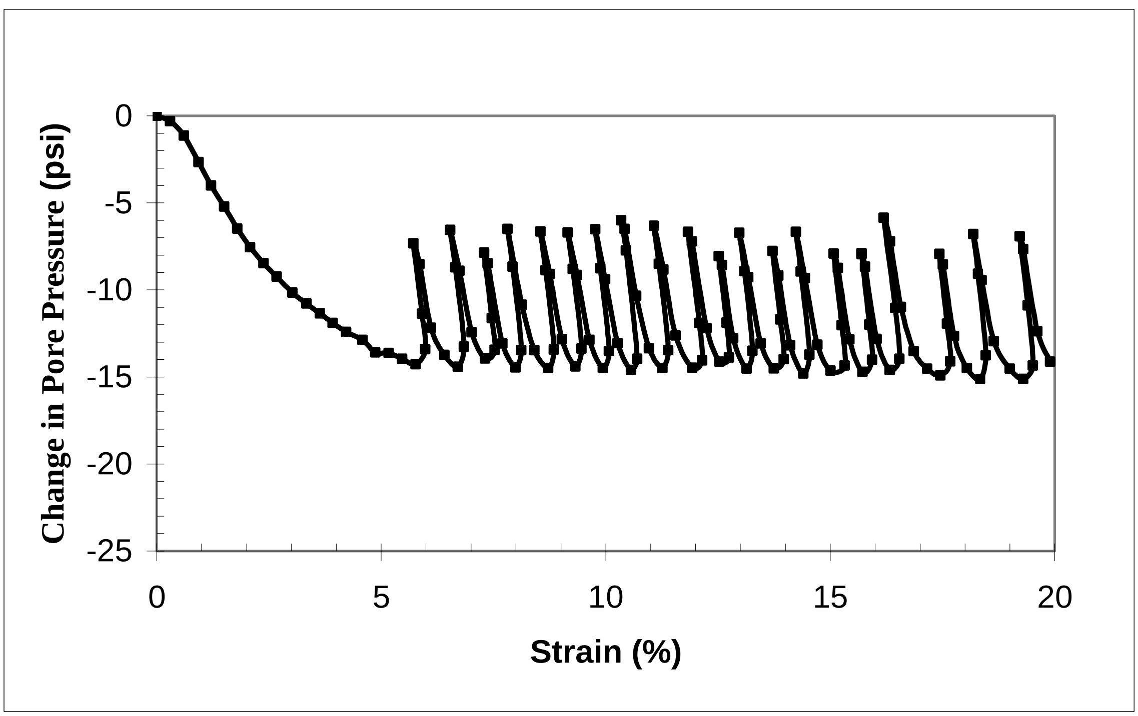
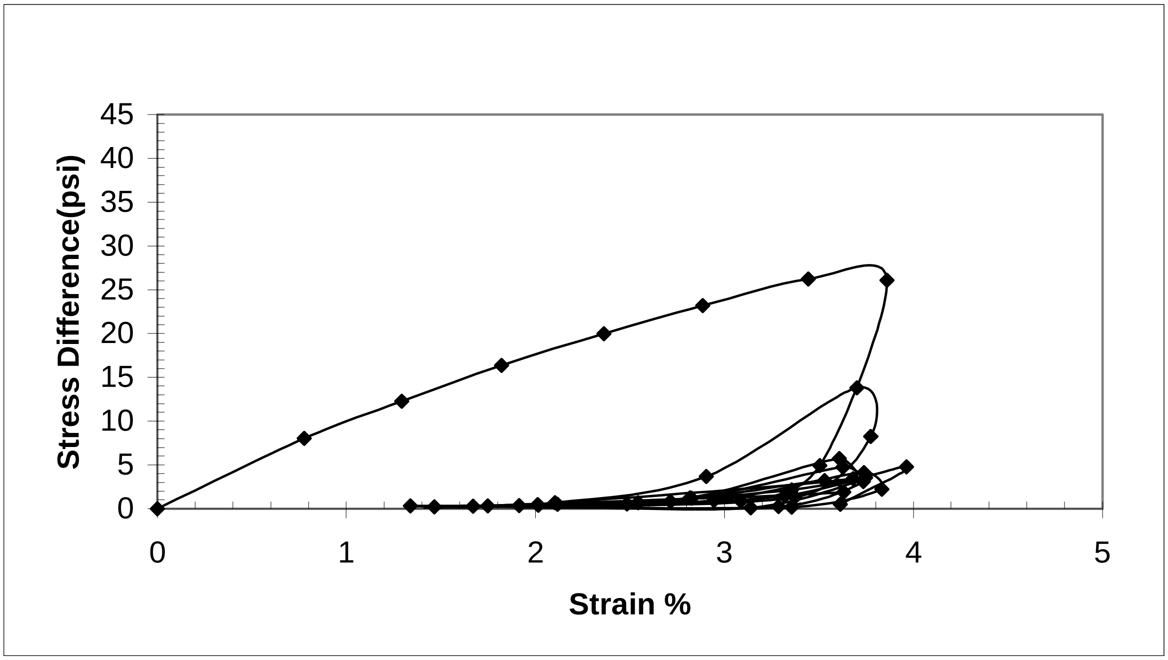


FIGURE 20 Lanagan Samples: Stress-Controlled Cyclic Triaxial Tests at Initial Confining Effective Stress Stress=2 psi, sheared between 40% and 5% of the maximum stress difference. a) Stress Difference versus Strain, b) Change in Pore Pressure versus strain



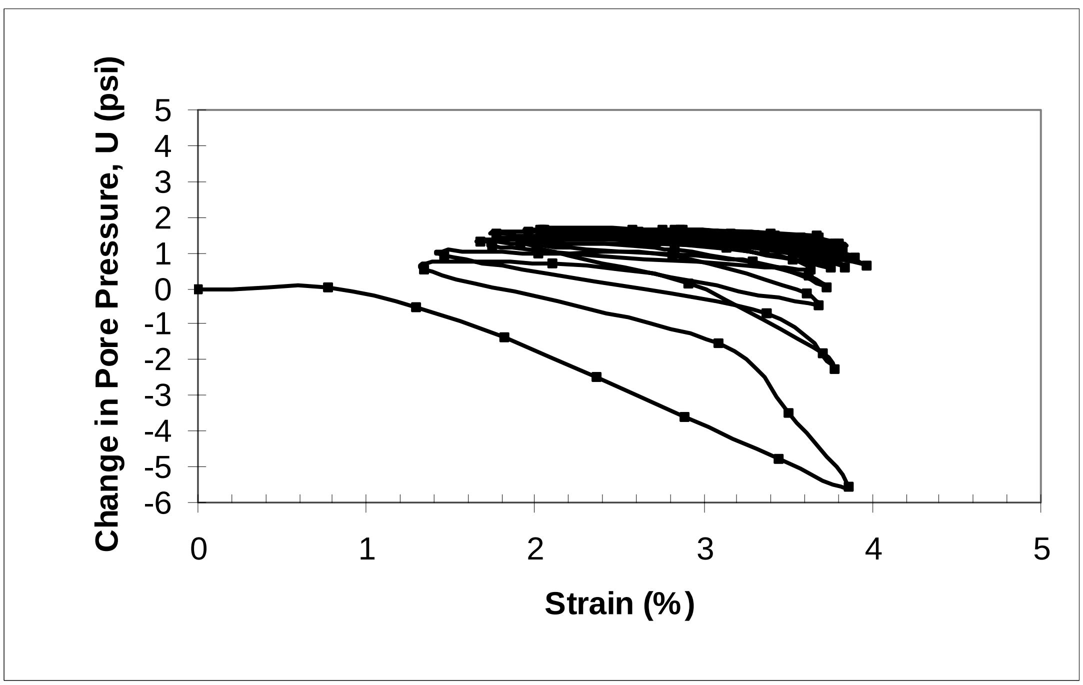
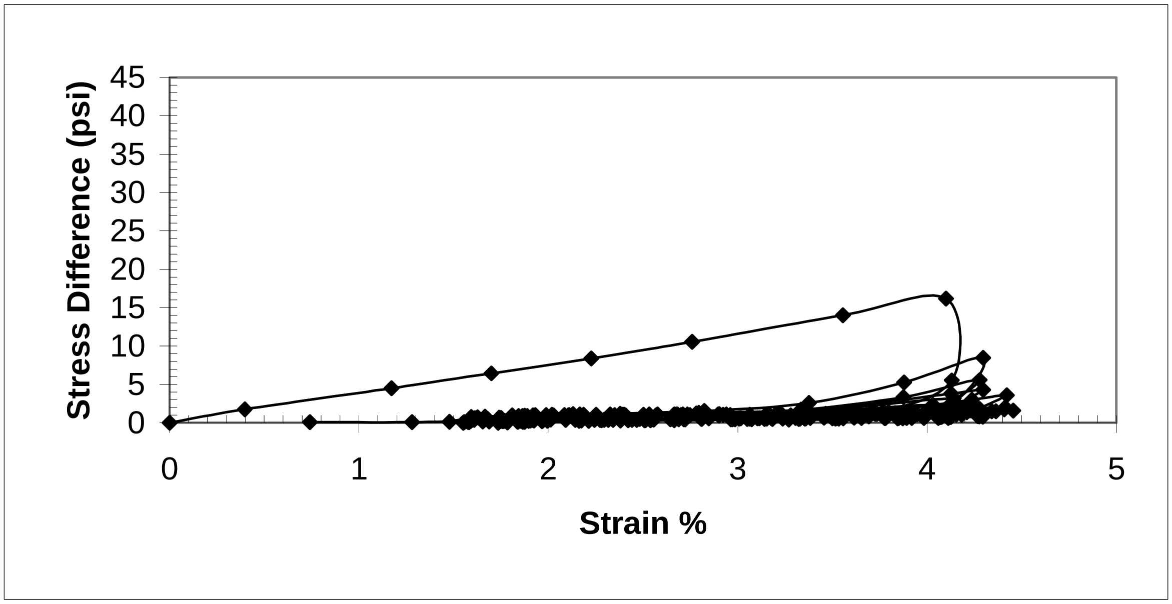


FIGURE 21 Lanagan Samples: Strain-Controlled Cyclic Triaxial Test Results at Initial Effective Confining Stress= 2 psi, a) Stress Difference versus Strain, b) Pore Pressure versus strain



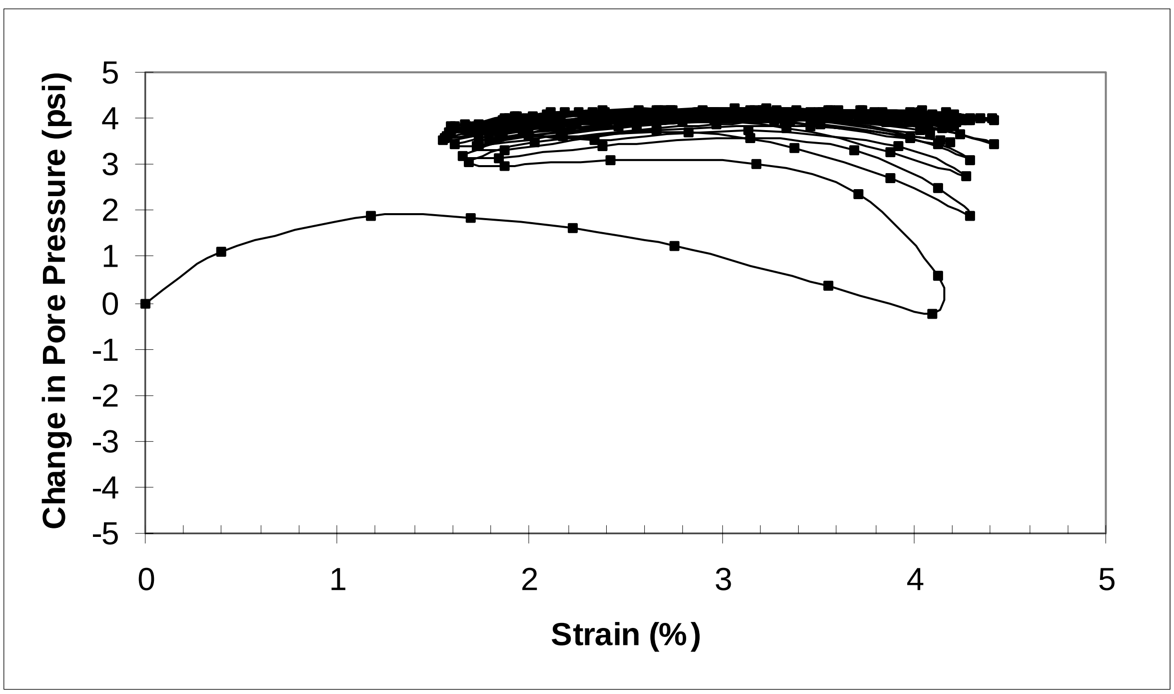


FIGURE 22 Lanagan Samples: Strain-Controlled Cyclic Triaxial Test Results at Initial Effective Confining Stress=4 psi, a) Stress Difference versus Strain, b) Pore Pressure versus strain

Cyclic, Stress-Controlled tests

The stress difference-strain response of the Lanagan quarry aggregate to cyclic loading under stress-controlled conditions was similar to the response observed in the static $\overline{CU}$ triaxial tests. Negative changes in pore pressures with a magnitude as high as 20-psi, occurred almost immediately after beginning the tests and remained negative throughout the test, even during unloading. The stress levels applied to the samples were less than the maximum stress difference; however, failure was defined by excessive bulging of the sample. Tests were stopped at 20 percent strain. Figures 19 and 20 show the shape and slope of the stress difference-strain curves for both specimens. It is clearly observed that the shape of the stress difference-strain curves remained practically unchanged cycle after cycle throughout the test until reaching 20 percent strain. This is attributed to the significant negative pore pressure phenomenon that restrained the samples from dilating.

Cyclic, Strain-Controlled Tests

The stress-strain response of the Lanagan quarry aggregate under strain-controlled conditions showed a totally different behavior compared to the stress-controlled tests. Contrary to the stress-controlled tests, bulging of the samples was not experienced at such low strains; sample appearance remained practically unchanged upon completion of the tests. Figures 21 and 22 show the results of the strain-controlled cyclic test. Samples sheared to strains ranging between 1 percent and 4 percent, and tested at effective consolidation stresses of 2-psi and 4-psi experienced a small drop in pore pressure during the first load cycle, after which changes in pore pressure became positive until the end of the test. Pore pressures increased rapidly in the first two loading cycles until approaching the total confining pressure from the third cycle on, thus reducing the effective stress in the sample to near zero. Consequently, strength degradation caused a flattening of the stress difference-strain curve slope, which progressed with every load cycle. As seen in Figures 21 and 22, after applying twenty load cycles the principal stress difference attained was only about 10 percent of the initial maximum principal stress difference in the first cycle and the stress difference-strain curve flattened out.

Since the base is not necessarily undrained in the field, preliminary drained, cyclic triaxial, strain-controlled tests were performed on saturated samples of compacted, Type 5 from Lanagan. The stress difference (strength) versus axial strain is shown in Figure 23a. Note that, although there was no control on the drainage of pore water from the sample, the strength rapidly decreased within two to three loading cycles. The volume change of the sample during the cyclic test is shown in Figure 23b. These behaviors indicate that the sample is behaving as if the test were being performed as an undrained test; i.e., no allowance for drainage of the pore water pressures developed during shear. In fact, it is the low hydraulic conductivity of the base which restricts the flow of pore water and subsequently the dissipation of pore water pressures developed during the cyclic loading. The effective strength of the sample decreases (Figure 23a) and the sample fails, or in the field the base no longer provides support to the overlying pavement and the pavement must support (as if a bridge) the loads of the traffic.

Drained, cyclic triaxial tests on the Type 5 base yielded results identical to undrained tests; i.e., the strength of the base decreased to almost nil within two to four loading cycles. The behavior indicates that the low hydraulic conductivity of the base renders it slowly draining regardless of the presence of pavement edge drains or an overlying drainable layer. Once saturated, the base quickly loses its strength during the cyclic loading imposed by traffic. The loss of strength in the base translates to unsupported pavements and the greatly increased probability of pavement failure.

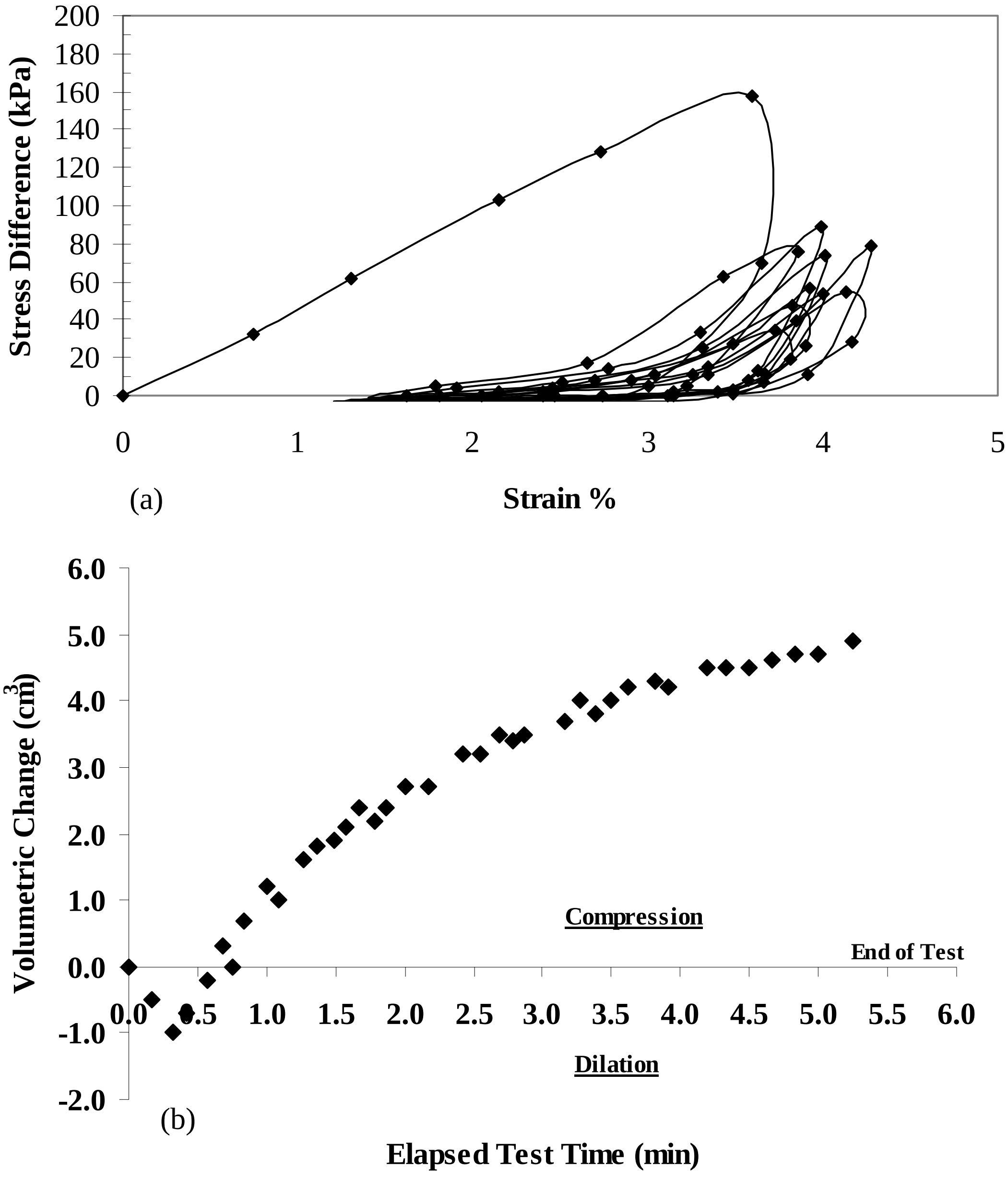


Figure 23 Lanagan Specimen: Strain-Controlled, Drained, Cyclic Triaxial Test with Initial Effective Confining Stress of 2-psi. (a) Stress Difference versus Strain. (b) Volume Change Versus Time of Test.

CONCLUSIONS

Traffic loading on saturated pavement substructures with low hydraulic conductivities result in excessive porewater pressures and subsequent loss of strength and reduced support of the overlying pavement. The performance, especially durability, of a pavement is directly related to the ability of the underlying materials to drain water from their structure. It is well known that pavements placed on well-drained substructures require less frequent maintenance and have greatly increased effective performance lifetimes.

The results of field (in-situ) and laboratory permeability testing showed Missouri DOT's predominant pavement base "Type 5" (and the upper "working" surface of the 2-foot rock fill alternative) to have hydraulic conductivities up to several orders of magnitude (10^{-3} to 10^{-5} cm/s) lower than the freely-draining value of 1 cm/sec. In essence, these materials should be classified as undrained. Using the measured hydraulic conductivities in the FHWA's DRIP 2.0 analysis for evaluating pavement performance resulted in drainage quality rankings of poor to very poor based on the 1986 AASHTO *Guide for Design of Pavement Structures*. Given these findings, it is concluded that Missouri pavements require more frequent maintenance and are not lasting as long as they could be if adequate drainage were provided.

Preliminary strength testing was performed on the Type 5 base in order to quantify the strength behavior. Results of cyclic triaxial tests under saturated, undrained conditions performed on compacted, Type-5 base material from Lanagan and Ash Grove Quarries showed the saturated base loses most of its strength within a few load cycles. Drained cyclic tests resulted in the same behavior – almost complete loss of strength in just a few load cycles. The findings indicate that the Type 5 base does not allow moisture to escape (drain) sufficiently fast enough during traffic loading resulting in build up of excess porewater pressure and loss of strength in the layer.

The field and laboratory permeability testing showed the Type 5 base to have such low hydraulic conductivities as to be considered undrainable. Preliminary cyclic strength tests showed the base loses most of its strength in just a few loading cycles. This behavior could explain the premature deterioration of some pavements in Missouri. Providing adequately drainable bases will increase the effective performance life and reduce replacement costs for Missouri pavements.

RECOMMENDATIONS

One goal of all transportation agencies to provide a specification for economic roadway bases that provide the necessary drainage and support to ensure long-lasting pavements. An effective specification can only be developed when based on sound evidence. The following tasks are recommended to gather high quality evidence that is prerequisite for the development of a highly effective specification for road way base in Missouri.

- *GIS Relational Database of Pavement Performance*: Establish a GIS-based database on pavement performance with relational layers for base type, subbase soil type, site geology, climatic conditions including freeze/thaw, traffic counts, subsurface drainage system, years in-service, etc. Use this database to compliment and extend the results of instrumented field test sections, and to perform lifecycle benefit-cost analysis to develop sound evidence for changes to pavement drainage/support substructure design.

- *In Situ Permeability Tests*: Perform additional in situ (field) permeability tests, using the lysimeter system to ensure one-dimensional flow and known boundary conditions. These should be performed on Type 5 and 2-foot rock alternate base prior to placement of the working surface on the rock. These additional data are necessary to fully and accurately characterize the in situ hydraulic conductivity of the bases.
- *Cyclic Shear Strength Tests*: Perform drained, cyclic triaxial tests on compacted specimens of the Type 5 base to evaluate the strength as a function of the percentage of fines, i.e., the hydraulic conductivity. Preliminary results have shown Type 5 to lose most of its strength within a few load cycles; however, this is directly related to the drainability (hydraulic conductivity) of the base. Field stress, loading and drainage conditions must be carefully represented in the laboratory tests.
- *Instrumented Field Test Sections*: Test sections of base should be instrumented to assess key parameters, including: moisture content, total stresses, pore water pressures and deformations in both the base course and the subbase. The time to become saturated and also to drain should be documented. The data is necessary to document how the Type 5 and 2-foot rock alternative are actually performing in the field.
- *Subsurface Drainage Mechanisms*: Develop and evaluate improved methods for providing subsurface drainage beneath pavements. Consideration should be given to multiple mechanisms for imparting drainage into the subsurface system since no single method is likely to be ideal (or economical for all cases). Several potential mechanisms that should be considered include: change in grain size distribution of the aggregate base, incorporation of geocomposite drainage layers, beneficial reuse of reclaimed

asphalt, light-weight aggregate, combustion bottom ash or synthetic aggregates, among others.

IMPLEMENTATION PLAN

Proceed on two fronts. First working with John Donahue and RDT office, develop a focused plan of testing including documenting the field performance of some instrumented test sections of pavement and continued laboratory testing of the base and pavement section response to cyclic loadings. This information is necessary to isolate the key variables so that appropriate actions can be taken to improve the base/pavement performance. Second, working with the affected pavements-related groups in MoDOT, develop a geographical information system-based database for the purpose of analyzing pavement performance in light of the design, construction process, drainage system, local soils, climate, traffic and maintenance conditions. The effort should focus on a District level and then move forward to include the entire State.

PRINCIPAL INVESTIGATOR AND PROJECT MEMBERS

The principal investigator was John J. Bowders, PE, Associate Professor of Civil & Environmental Engineering, University of Missouri-Columbia (UMC). Ms. Awilda Blanco, geotechnical engineering masters candidate, was the graduate research assistant on this project. She was assisted by Nicholas Roth and Wuttichai Pranchantrikal in the field testing. Ms. Jeannie Sims, undergraduate research honors student, and Robert Hemmelgarn, high school intern, assisted with the laboratory testing. Jorge Parra, geotechnical engineering doctoral candidate at UMC, performed the triaxial shear testing as part of a special problems course. He was assisted by Nicolas Pino an undergraduate honors research scholar. The success of the project was made possible through the efforts of Stowe Johnson and Michael Blackwell of MoDOT Research Development and technology unit. They collected samples of base around the state, located test sites for in situ testing and performed many of the in situ hydraulic conductivity tests.

IMPLEMENTATION OBJECTIVE

The overall objective is to provide long-lasting, low lifecycle cost pavements for the State of Missouri. The focused tasks of the proposed effort are to develop the evidence on which to base changes to the existing pavement base and drainage system.

AFFECTED BUSINESS UNITS AND PRINCIPAL CONTACT

The principal business units impacted by this work is the Research Development and Technology Unit (Contact. Mr. John Donahue) and the pavement design group within the Materials Unit.

IMPLEMENTATION PERIOD

Continuation efforts should begin with the acceptance of a detailed proposal.

Instrumented test sections may require several years of monitoring for collection of sufficient data for analysis. The laboratory cyclic testing program should take about one year duration. The GIS database on pavement performance would require about one year to set up for a single District and likely a second year to expand to include the entire State.

FUNDING

Funding for additional efforts will be discussed with MoDOT RDT and the Districts.

TECHNOLOGY TRANSFER

Developments made through this and any additional projects will be transferred to the Missouri DOT through the final report, published papers (see Appendices), presentations to

MoDOT personnel and at such venues as the National Transportation Board’s Annual meeting.

PROCEDURE

Detailed procedures will be described in subsequent proposals.

BUDGET

A detailed budget will be provided with any subsequent proposals on these topics.

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Appendices

A. "Long Term Flow Behavior of a Base Aggregate," Awilda M. Blanco; John J. Bowders , Grant Krueger; and Wuttichai Prachantrikal, paper submitted to the ASCE *Journal of Geotechnical and Geoenvironmental Engineering* for review and possible publication, under review...\..\Papers\BAse K - Collection of Papers\Long Term Flow Behavior of MoDOT Type 5 Base Aggregate - Journal Format.pdf

B. "Hydraulic and Strength Performance of Missouri's Type -5 Base Material," Jorge R. Parra and Awilda Blanco, paper presented at and published in the *Proceedings of the 2002 Midwest Transportation Consortium Fall Conference*, Iowa State University, November 15, 2002...\..\Papers\BAse K - Collection of Papers\Type5 base performance-final.pdf

C. "Laboratory and In Situ Hydraulic Conductivity of Pavement Bases in Missouri," Awilda M. Blanco, John J. Bowders, and John P. Donahue, submitted to the Transportation Research Board for presentation at the Annual Meeting (January 2003) and possible publication in the *Transportation Research Record*. The paper will be included on the CD from the Annual Meeting...\..\Papers\BAse K - Collection of Papers\BaseK-Final doublespace.pdf

D. "Providing the Evidence for Evidence-Based-Decision Making to Optimize the Performance of a Base Course," John J. Bowders, Jorge Parra, Awilda M. Blanco and John P. Donahue, abstract submitted to: Int'l Conf on Hwy Pavement Data, Analysis & Mechanistic Design Applications, Sept 8-10, 2003, Columbus, Ohio...\..\Papers\BAse K - Collection of Papers\Bowders.etal.BaseCourse.Ohio.pdf

ASCE, *Journal of Geotechnical and Geoenvironmental Engineering*
Submitted: 25 June 2002

Long Term Flow Behavior of a Base Aggregate

Awilda M. Blanco[1]; John J. Bowders[2], member; Grant Krueger[3]; and Wuttichai Prachantrikal[1]

ABSTRACT

One of the most common causes of pavement failure is poor subsurface drainage. Well-drained pavements can outlast poorly drained pavements by 10 to 40 years or longer. The base course beneath pavements largely controls the drainage characteristics of the pavement system. The specified base material for state highways in Missouri was sampled and long-term flow tests were performed at various fines (particles less than 0.075mm in size) contents, but within state specification. Measurements of flow rate and fines migration using uncompacted specimens were performed, and long-term flow conditions were replicated by using a constant head flow test. Fines migration was determined by using a geotextile filter layer to capture fines. Results show that hydraulic conductivity decreased with increasing amount of fines, and that hydraulic conductivity decreased with time (pore volumes of flow**).** Final hydraulic conductivities ranged from 3.5×10^{-4} cm/sec (1 ft/day) at 0 percent fines to 3.5×10^{-5} cm/sec (0.1 ft/day) for 15 percent fines. Some fine particle migration was observed.

KEYWORDS

Pavement Subsurface Drainage, Base Course, Long Term Flow, Geotextile Filter, Permeability, Hydraulic Conductivity

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INTRODUCTION

Pavements throughout the world are being damaged by water. Water can flow into the base of the pavement through joints, or cracks, and also from groundwater sources (Moulton, 1980). During recent years, designers have focused on strength design of pavements, while not taking into serious consideration the draining characteristics of their designs. Although edge drains and other types of drainage structures are being specified in the designs, limited consideration for drainage is being placed on the materials that form the base and sub-base of the road. Care should be taken to specify materials that will allow for sufficient drainage characteristics to allow the infiltrated water to reach the edge drains or any other drainage structure (Figure 1).

Due to the inability of base material to drain water rapidly, a bathtub effect occurs, where water becomes trapped in the base (Cedergren, 1994). The presence of water in the base and the cyclic loading due to moving vehicles causes excessively high pore water pressures (Novak et al, 2002). This phenomenon causes a loss in effective stress and strength in the base and eventually pavement failure (Cedergren, 1989). Also, due to the incompressibility of water, the surface contact pressure is transmitted downward to the subbase, which weakens the subbase and reduces support for the base and pavement above.

The Missouri Department of Transportation (MoDOT) has specified five different types of bases for their roads, and the use of the different materials depends on the various circumstances of each project. Type 5 Base can be used for all types of roads (light, medium and heavy traffic) (MoDOT Project Development Manual, 1996). Due to the widespread use of this base material, it is important for it to have adequate drainage behavior.

The specifications for the Type 5 Base allow the base to contain up to 15 percent fines (material passing the No. 200 sieve or ≤ 0.075mm) (MoDOT Materials Specifications, Section 1007). Hydraulic conductivity not only depends on total porosity, but also on the sizes of conducting pores (Hillel, 1998). Larger particles tend to create larger pores, which allow for more water movement. An increase in fines content tends to diminish the amount of larger pores, thus lowering the hydraulic conductivity of the base.

The main objective of this study was to determine the long-term flow characteristics of Type 5 Base having different fines content. The scope of work included the determination of the grain size distribution of the Type 5 base, the determination of the long-term flow characteristics of Type 5 base containing different fines contents, and the determination of the hydraulic conductivity of uncompacted specimens under long term flow conditions and how it changes in time or with pore volumes of flow.

MATERIALS AND METHODS

Type 5 Base aggregate consists of crushed stone or sand and gravel, and should not contain more than 15 percent deleterious rock and shale (MoDOT Materials Specifications, Section 1007.) The gradation limits of this material are presented in Figure 2. Type 5 Base specifications allow for the material to have up to 15 percent fines, or 15 percent of the material (based on dry weight) can pass the No. 200 sieve. The material used for this project was gathered from Boone Quarry, Columbia, Missouri. The material was sampled by shoveling at 0.6 m (2 ft) intervals from the base of the pile, brought to the Geotechnical Laboratories at the University of Missouri – Columbia, and air dried for testing (Prachantrikal, 2002). Grain size analyses were performed (ASTM D422) to determine the gradation of the samples, and then dry-sieved to remove the material passing the No.200 sieve. Fines were then re-introduced into the samples to create mixtures of 0, 3, 6, 9, 12, and 15 percent fines, based on dry weight. These samples were placed in the long-term flow equipment, as shown in Figure 3.

The long-term flow testing apparatus provides a constant head test for determining the flow rate and hydraulic conductivity of the aggregates. Water is continuously supplied to the specimen, and a constant head is maintained by allowing the water to exit the system at a specified height. Six 12 cm (4.75-inch) internal diameter columns of the mixed aggregate at various fines contents (0, 3, 6, 9,12, and 15 percent fines) were placed without compaction at the bottom of the system and separated from the outflow area by a geotextile supported on a fine metal mesh (Figure 4). This allows for the specimen to be kept in place as well as retention of any fines, if migration occurs.

The hydraulic gradient was maintained constant and three flow rate (volume of water in time) measurements were taken at each column at the start of the test, 24 hours, 48 hours, 4 days, and then approximately after 1, 2 and 4 weeks. Plots of the average flow rate versus time were prepared after each reading for each fines content, and tests were stopped after the flow rate stabilized, which in several cases was longer than 4 weeks.

After the flow rates were calculated from the measured volume of water and time, the hydraulic conductivity of each specimen was calculated using Darcy's law (Eq. 1).

$$q = kiA \qquad (1)$$

where q represents the volume of water that flows through the specimen over a given time interval (units of volume/time), k represents the hydraulic conductivity of the specimen, i represents the hydraulic gradient of the system (change in total head/length of specimen) and A represents the cross sectional area of flow.

RESULTS AND DISCUSSION

The main objective for this research was to determine the long-term drainage characteristics of the material used as a base material beneath state roadways in Missouri. The specification allows for this material to have up to 15 percent fines (material $\leq$0.075mm). Long-term flow testing was performed on this material to determine the flow characteristics of the aggregate at various fines contents. Flow rate versus time for all the specimens of various fines contents are shown in Figure 5.

The initial flow for all the specimens was similar, between $6x10^{-3}$ to $2x10^{-2}$ L/sec ($1.6x10^{-3}$ to $5.3x10^{-3}$ gal/sec). All specimens had the same cross-section area (113 cm^2, or 17.5 in^2) and the same length (12.7 cm, or 5 in). The hydraulic gradients were the same for all specimens. The specimen with no fines had the larger initial flow, and as the amount of fines increased, the initial flow rate decreased. The initial flow for the specimen with 15 percent fines was the third highest, indicating that the fines were probably not uniformly distributed throughout the specimen. During visual inspections of the setup for the 15 percent fines content specimen, it was noticed that there was a high amount of fines wedged against the wall of the apparatus, thus allowing for higher initial flow rates than the other specimens with lower but uniformly distributed fines.

This same trend of decreasing flow with increasing fines content was noticed at the end of the tests. The sample with 0 percent fines content had a higher flow rate than the specimen with 15 percent fines (Figure 5).

As stated before, the initial flow rate decreased as the fines content increased, except for the 15 percent fines specimen, which had a higher flow rate. Final flow rate measurements show a similar trend, decreasing flow rates with increasing fines content (Figure 6). Also, in all cases, the initial flow rate for each specimen was higher than the final flow rate (Figure 7). This is due to the fact that the first flow rate readings were made immediately after the specimens were placed in the apparatus. The specimens were placed in a dry state. As time increased, water tended to move the fines in the specimen, decreasing the available pore space for the water to flow. This causes the flow rate to decrease over time. Also, some of the fines that migrated through the specimens were caught in the geotextile filter placed at the bottom of the column, creating a high concentration of fines, which decreases the hydraulic conductivity and resulted in lower flow rates.

The geotextile used in this testing was a non-woven, needle-punched geotextile, with an AOS of 0.212 mm (No. 70 sieve) (Geotechnical Fabrics Report, 1999), and mass per unit area of 150 g/m^2 (0.03 lb/ft^2). The permittivity (ASTM D4491) of an unused portion of geotextile was measured and found to be 1.05 sec^{-1}, which compares favorably with the published value of 1.5 sec^{-1}. The calculated hydraulic conductivity of the unused geotextile was determined to be 0.21 cm/sec (595 ft/day). Permittivity testing was also performed on a piece of used geotextile and was determined to be 1.17 sec^{-1}, and the hydraulic conductivity was determined to be 0.23 cm/sec (663 ft/day). The permittivity of the used geotextile was determined after the geotextile was retrieved from the long-term setup and shaken to check if any fines migration had occurred. It was concluded that the geotextile alone did not control the hydraulic conductivity of the system, although it might have controlled the final hydraulic conductivity of the systems since the fines were collected in it during the test. Furthermore, no clogging due to microorganisms was expected nor found. Previous research on geotextiles with similar characteristics to the one used for this research showed no biological clogging for leachate collection system materials when permeated with water even after 140 days (Koerner et al, 1994).

The preparation of the specimens included the separation of the fines from the soil using the dry sieving method. Tests on this same type of material from other sources showed that dry sieving does not remove all the fines from the sample. Although dry sieving does separate most of the fines from the sample, wet sieving is needed to ensure to have a specimen with no fines. This may the reason why there is a difference between the final and initial flow rates for the specimen labeled as 0 percent fines, it may actually contained some fines.

Table 1 presents the results of wet sieving performed after the samples were tested in the long-term flow apparatus. No data is available for the 3 percent fines specimen. For the remaining specimens, the amount of fines after the test was higher than the amount of fines measured at the beginning of the test. This supports the conclusion that wet sieving is required to remove all fines from the aggregate. Dry sieving was performed to prepare the specimens. The specimens were not handled to the point of being able to create any fines.

The tests were stopped when the flow rate measurements showed little variation. When flow equilibrates, it is assumed that the specimen is saturated, or nearly completely saturated. At the beginning and end of each test, the hydraulic conductivity of the specimen was calculated using Darcy's Law. As seen in Figure 8, the initial hydraulic conductivity is higher than the final hydraulic conductivity. Also, there is little variation in the final hydraulic conductivities among all the specimens. The lack of variation might be due to fines migration towards and collection on the geotextile, thus blinding it and decreasing the hydraulic conductivity of the system, although permittivity tests on the geotextiles post long-term flow tests, showed little or no reduction in flow capacity of the geotextiles.

CONCLUSIONS

The hydraulic behavior of the Type 5 Base material was determined. The initial flow rate and the hydraulic conductivity of the specimens decreased as the amount of fines increased. Also, increased times of flows (pore volumes of flow) caused a decrease in the hydraulic conductivity. The longer the flow period, the lower the hydraulic conductivity. This was due to fines migration, diminishing the effective porosity of the specimen, which decreased the capacity for flow and thus the hydraulic conductivity. The specimens placed on the apparatus were not compacted, thus allowing higher fines migration than one might see in a compacted specimen

It was also noticed that the final hydraulic conductivity of all the specimens was similar among all the fines contents. This fines migration may cause the geotextile to blind or partially clog, thus controlling the hydraulic conductivity of the system. As more fines are attached to the geotextile, the hydraulic conductivity decreases. Permittivity testing showed that the hydraulic conductivity of the used geotextile was very similar to the hydraulic conductivity of an unused geotextile, thus the geotextile alone did not affect the flow.

Although all the hydraulic conductivity values were similar, the higher fines content specimen showed a lower final hydraulic conductivity than the 0 percent fines specimen. Final hydraulic conductivities ranged from 3.5×10^{-4} cm/sec (1 ft/day) at 0 percent fines to 3.5×10^{-5} cm/sec (0.1 ft/day) for 15 percent fines. These values are 10,000 times below the generally accepted minimum hydraulic conductivity for a freely draining base (1 cm/sec, or 3000 ft/day), indicating that the Type 5 base may not provide adequate drainage of water beneath pavements.

ACKNOWLEDGMENTS

The authors would like to express their appreciation to Mr. John Donahue, from the Missouri Department of Transportation, Research Division, for his cooperation and support for this research. We are also appreciative of the efforts of Mr. Stowe Johnson and Mr. Michael Blackwell (MoDOT, RDT) in obtaining aggregate materials and field assistance. The cooperation of the personnel from Boone Quarries, Columbia, MO is greatly appreciated.

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List of Tables

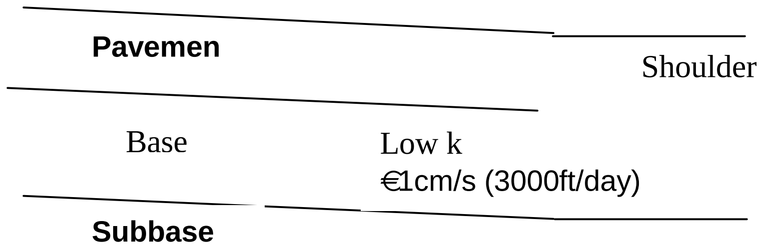
List of Figures

Table 1 – Index Properties of Long Term Flow Specimens

Type 5 Specimen	Desired Initial % Fines	Final % Fines	γ_d (Kn/m^3)	γ_d (lb/ft^3)	n (%)	e	Comments
1	0	5.6	10.84	68.99	62	1.63	Assume Dry
2	3	NA	N/A	N/A	N/A	N/A	
3	6	7.9	10.74	68.36	62	1.63	Assume Dry
4	9	11.0	10.74	68.35	62	1.63	
5	12	14.5	9.73	61.91	66.5	1.96	
6	15	16.0	10.76	68.51	60	1.5	

N/A = Not Available

(a)



(b)

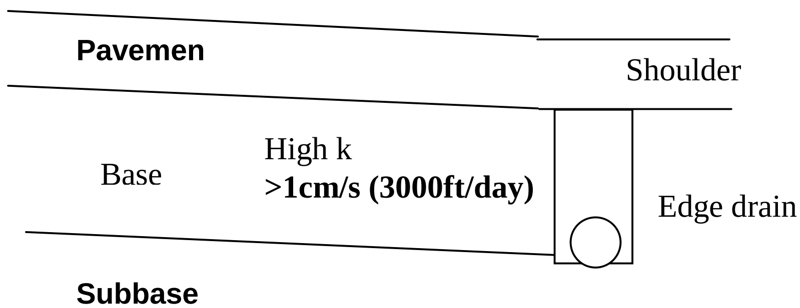


Figure 1 – Typical pavement cross-section (a) poor subsurface drainage and (b) good subsurface drainage.

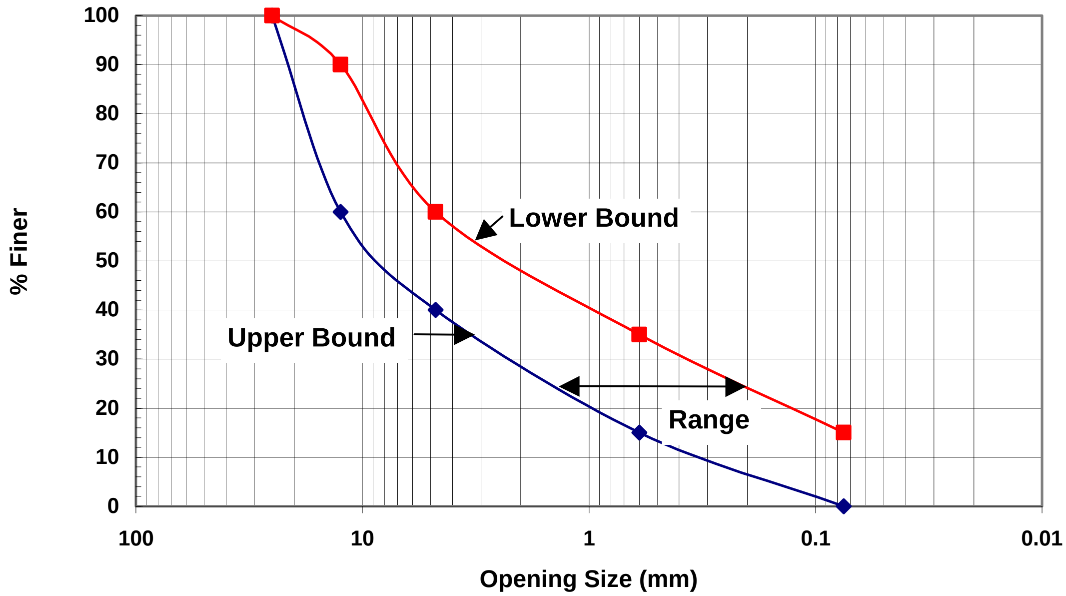


Figure 2 - Gradation requirements for lower and upper limits, Type 5 base. Adapted from Section 1007, Materials Specifications, MoDOT.

Figure 3 – Long-term flow apparatus

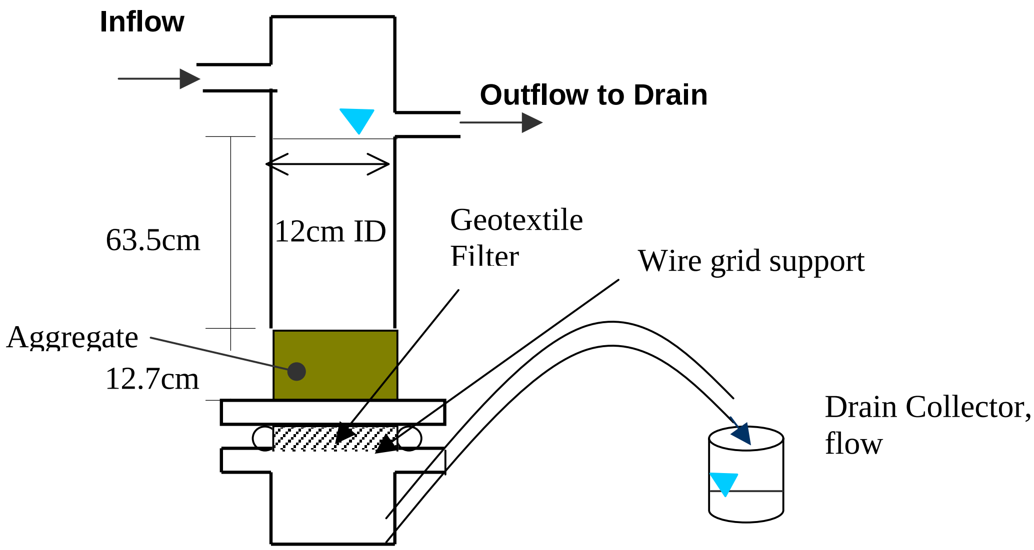


Figure 4 – Schematic of long-term flow apparatus (Not to scale)

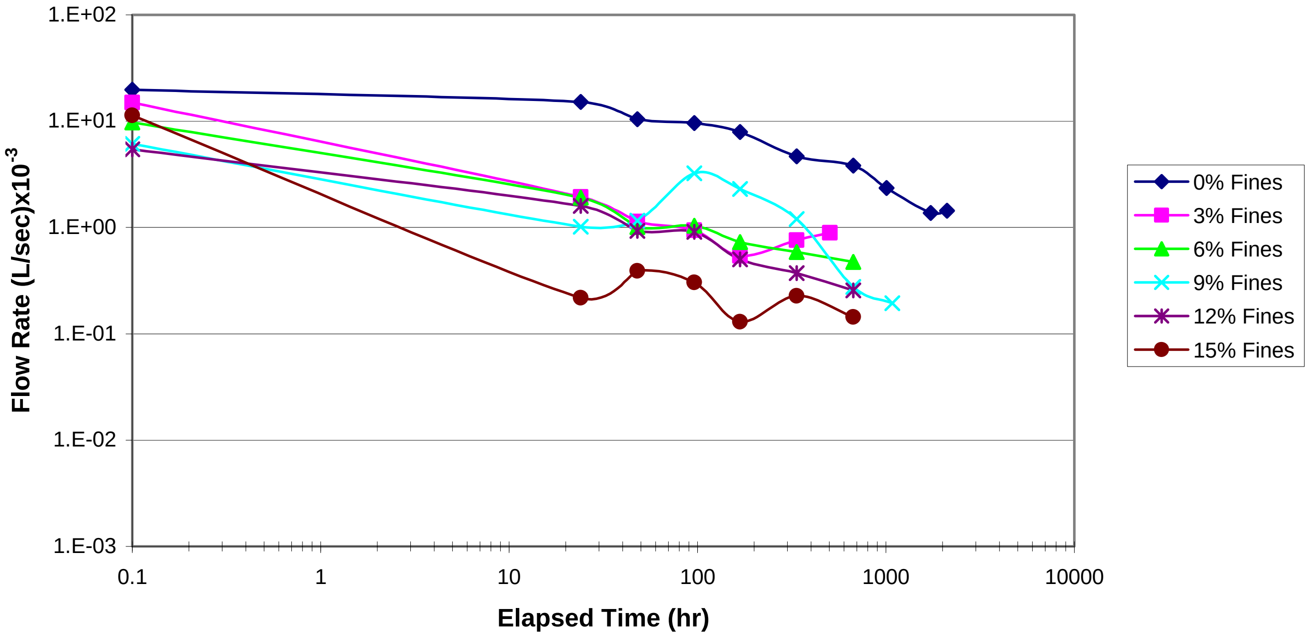


Figure 5 – Flow rate versus time, Type 5 base, long-term flow

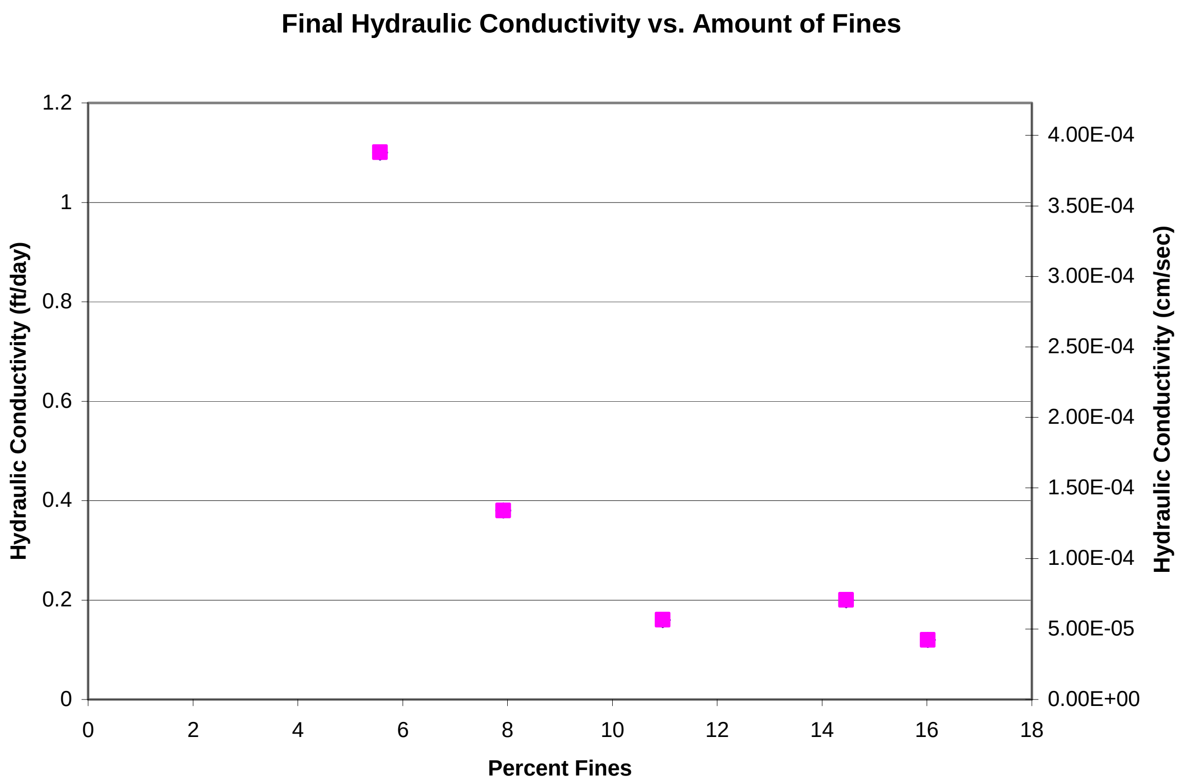


Figure 6 – Hydraulic conductivity versus percent fines after testing

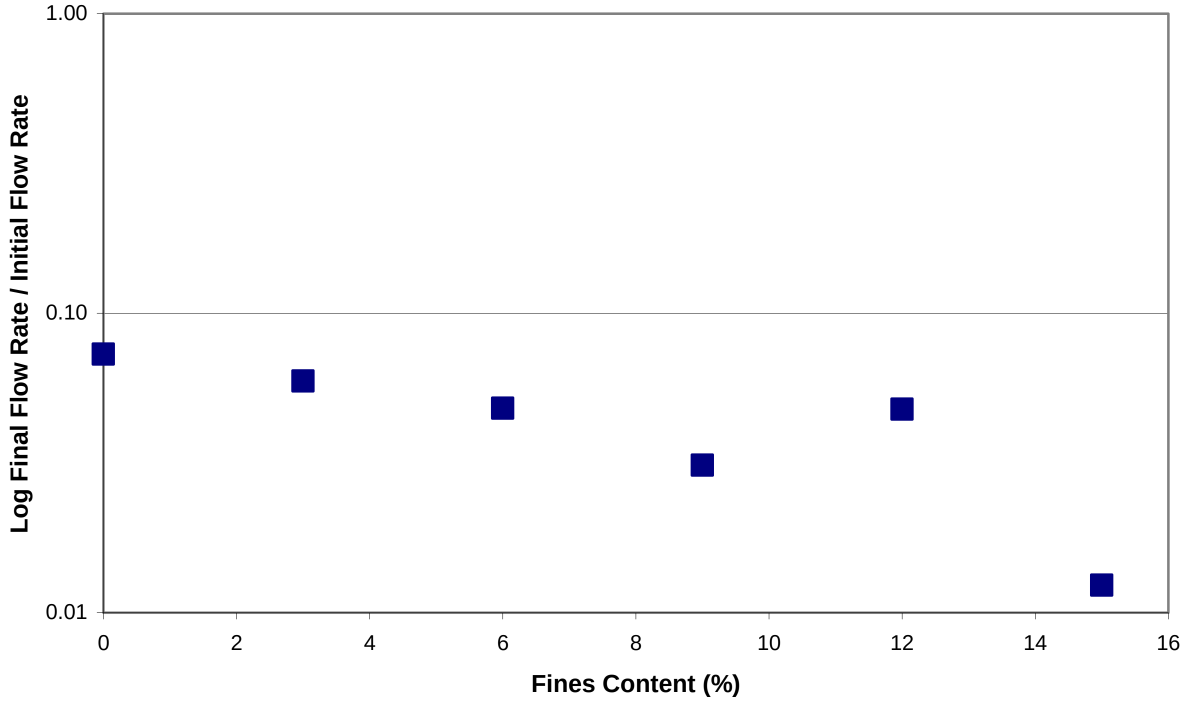


Figure 7 – Ratio of final flow and initial flow versus fines content, Type 5 base, long-term flow

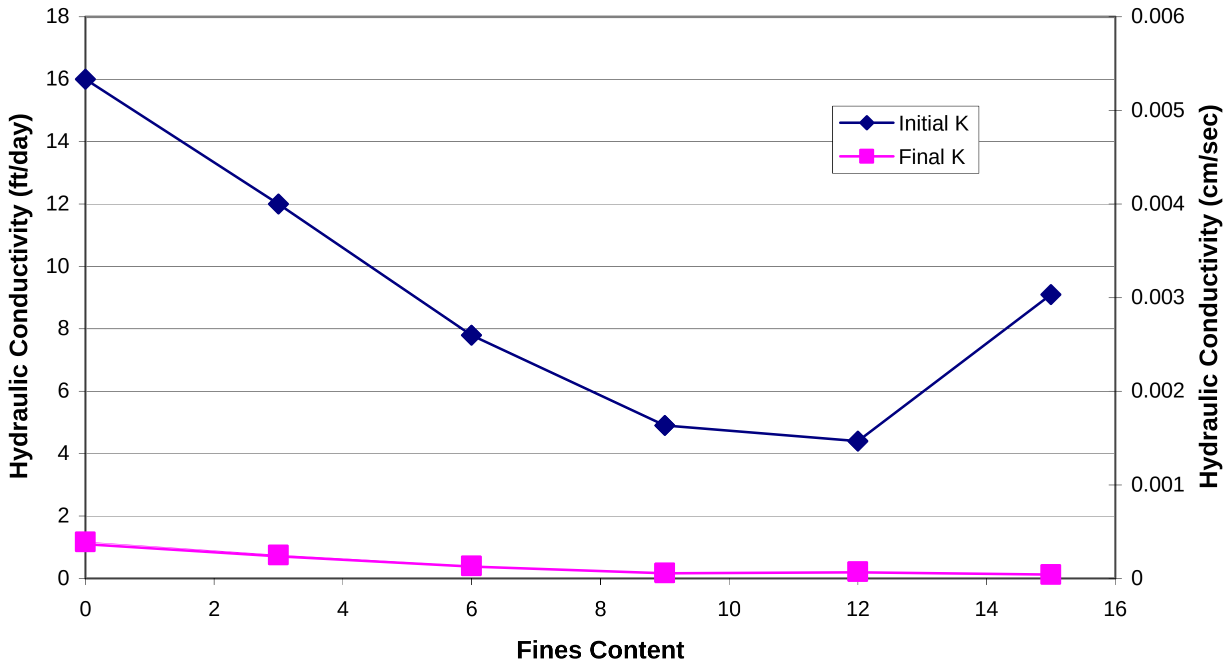


Figure 8 – Hydraulic conductivity versus fines content, Type 5 base, long-term flow

Hydraulic and Strength Performance of Missouri's Type-5 Base Material

by Jorge R. Parra and Awilda Blanco

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ABSTRACT
Across the state of Missouri, it has been noted that paved roads deteriorate at an accelerated rate. The base material has been identified as a possible source of the problem. To investigate this issue, researchers at the University of Missouri-Columbia have conducted in-situ and laboratory hydraulic conductivity testing, as well as cyclic triaxial testing in order to evaluate a relationship between drainage conditions and cyclic behavior of the base material. Laboratory and in-situ hydraulic conductivity testing was performed on aggregates used as base materials for medium and heavy traffic roads. A granular material from one Missouri quarry exhibiting typical engineering properties for road base application was selected for strength evaluation. Test results indicate that the hydraulic conductivity of the base material selected does not comply with requirements to assure long-term effective drainage. The shear strength properties of the compacted base material were tested following strain-controlled and stress-controlled criteria, and for saturated-undrained conditions, which represent a worst drainage scenario that a poorly performing pavement structure might experience. Results show that while the shear strength response to cyclic loads in stress-controlled conditions remains unchanged due to significant negative pore pressure build-up, rapid degradation in strength occurs in strain-controlled conditions when generation of positive pore pressures reduces effective stresses to zero during a small range of strains. Under cyclic loading conditions, the saturated base looses most of its strength within a few load cycles. This behavior could explain the premature deterioration of some pavements.

INTRODUCTION

Although the importance of well-drained pavement systems has been extensively documented, (1,2,3), the design and construction of pavement systems has not emphasized drainage for the last few decades. During this time, designers have focused on the strength of pavements, while little importance has been given to drainage of the pavement systems.

Water can flow into the base of the pavement through joints, cracks, and from groundwater sources (3). The presence of water and the lack of drainable layers in the pavement cause what can be described as a bathtub effect, owing to the fact that the pavement base remain saturated for long periods of time. The combination of saturated conditions in the road base and the cyclic loading of the pavement system due to moving vehicles cause a loss in strength and eventually pavement failure (4).

In Missouri, two principal base course systems are used beneath the pavement, as shown in Figure 1. The first is a 10-cm (4-in) layer of a well-graded aggregate with particle sizes from 25-mm (1-in) to less than 0.074-mm (0.003-in) (No. 200 sieve), referred to as Type 5 base. The alternative material consists of a 0.6-m (2-ft) layer of rock fill, and the pavement is placed directly on any of these two materials. The main objective of this project was to assess the drainability of the well-graded aggregate by performing a program of in-situ hydraulic conductivity tests and laboratory tests, and to evaluate the strength of the Type 5 base material by a series of strain-controlled and stress-controlled cyclic triaxial tests under saturated-undrained conditions. Saturated-undrained conditions can be expected in pavement layers with insufficient drainage characteristics.

MATERIALS AND METHODS

Bulk samples of the Type 5 base material were obtained from supplier quarries, on-site stockpiles, and from compacted, in-place roadway bases around Missouri. Based on Missouri Department of Transportation (MoDOT) specifications, the base material can have up to fifteen percent by weight passing the No. 200 sieve (5). Dry and wet sieve analyses were performed on all Type 5 base materials and rock fill alternates to determine the amount of fines in each material (Table 1). The last two materials listed in this table are specimens of the rock fill alternate. Individual particles in this material can be as large as 0.3-m (1-ft) and the interstices may be filled with a mixture of coarse aggregates to fines. Specimens used for the cyclic triaxial tests were prepared using the material from Lanagan Quarry because it is the material with one of the lower fines content and it is therefore expected to be the most freely draining material.

LABORATORY AND FIELD HYDRAULIC CONDUCTIVITY TESTING

The laboratory hydraulic conductivity for each material was measured using a constant head permeameter (CHP), as shown in Figure 2. The 15-cm (6-in) diameter specimens were prepared by compacting the material using standard Proctor energy, based on method C from ASTM D698 (6). Specimens were compacted to the respective maximum dry density and optimum water content for each material. MoDOT provided compaction curves for the different Type 5 base materials. The compacted specimens were placed in the permeameter, which provided flow under a constant head, and flows per unit area under different hydraulic gradients were determined. The hydraulic conductivity was then determined by fitting a best-fit straight line to the flow per unit area vs. hydraulic gradient plot. In several cases, no flow was measured using the CHP. In general, all laboratory compacted specimens were tested in the CHP; however, the sensitivity of the device is about 1×10^{-4} -cm/sec (0.3-ft/day) below which, accurate hydraulic conductivity measurements are difficult to discern. Therefore, specimens that exhibited hydraulic conductivities lower than approximately 1×10^{-4}-cm/sec (0.3 ft/day) were removed from the CHP, set up in a flexible wall permeameter and tested according to ASTM D5084 procedures (7).

In-situ measurements of hydraulic conductivities were performed using a double ring infiltrometer (DRI) device (Figure 3). The double ring infiltrometer provides a direct determination of the infiltration rate in the material being tested (8). Infiltration rate is defined as volume per time per unit area perpendicular to flow direction, or

$$I = \frac{Q/t}{A} \tag{1}$$

where I is the infiltration rate [L]/[T], Q is the volume of infiltrated fluid [L^3], t is the time of infiltration [T], and A is the area [L^2] over which the liquid infiltrated.

The hydraulic conductivity can be determined by dividing the infiltration rate by the hydraulic gradient. For the cases in this study, an assumption of full-depth (10-cm or 4-in) saturation of the base was made and the hydraulic gradient was taken as:

$$i = (h+d)/d \tag{2}$$

where h is the height of water ponded above the base [L], and d is the thickness of the base [L] (Figure 3). The hydraulic conductivity is then calculated as

$$k = \frac{I}{i} \tag{3}$$

where k is the hydraulic conductivity [L]/[T], I is the measured infiltration rate and i is the hydraulic gradient.

Hydraulic conductivity values obtained from in-situ and laboratory testing were also compared with the values calculated using empirical equations. Three different methods were used to empirically estimate the hydraulic conductivity of the materials tested. All of the methods rely on the grain size distribution of the aggregate. The methods included:

A. Hazen Equation (11): $k = D_{10}^{\,2}$ (4)

where D_{10} represents the size in mm at which 10 percent of the sample by weight is smaller, and the hydraulic conductivity has units of cm/sec,

B. Sherard Equation (12): $k = 0.35D_{15}^{\,2}$ (5)

where D_{15} represents the size in mm at which 15 percent of the sample by weight is smaller, and the hydraulic conductivity has units of cm/sec,

C. Moulton Equation (3):

$$k = \frac{6.214x10^{5}\,D_{10}^{\,1.478}\,n^{6.654}}{P_{200}^{\,0.597}} \tag{6}$$

where n represents the porosity of the material and P_{200} represents the fraction of material finer than the No. 200 sieve (0.075-mm) in percent.

Shear Strength Testing

The effective stress shear strength properties of the base materials were determined by conducting Consolidated-Undrained type triaxial tests with pore pressure measurement ($\overline{CU}$). This test requires 100 percent saturation of the sample prior to the consolidation and shearing stages. Although base materials are not designed to experience saturation conditions, the intention of the tests was to mimic a worst case scenario consisting of a poorly-draining material that will eventually reach this condition. Aggregate samples were compacted at their optimum water content at standard Proctor energy to form specimens of 7.4-cm (2.9-in) in diameter and 14.2-cm (5.6-in) in height. The maximum particle size used for triaxial testing was limited by the size of the compaction mold to an overall diameter of 1.27-cm (0.5-in). This maximum particle size avoids bridging problems of larger particles in the compaction mold. $\overline{CU}$ Tests were performed at strain rates of 1.5 percent per hour, under effective confining stresses of 14, 28, and 42-kPa (2,4, and 6-psi), which are representative of the stress levels close to the ground surface. Samples were sheared to a maximum strain of 30 percent.

Cyclic Triaxial Tests

Sample dimensions, gradation, test preparation, and saturation process followed the same procedures as for the $\overline{CU}$ type triaxial tests. The strain rate was set to 1000 percent per hour to mimic the cyclic effect produced by traffic loads as closely as possible.

Stress-controlled tests were set to stop when reaching either of the following criteria: a maximum strain of 20 percent or one hundred loading cycles. Samples at an effective stress of 14- kPa were sheared to stresses corresponding to 20 percent, 40 percent, and 60 percent of the maximum principal stress difference as determined from static $\overline{CU}$ tests, respectively; and reloaded at about 5 percent to 10 percent of the maximum principal stress difference.

Strain-controlled tests consolidated under effective confining stresses of 14-kPa and 28-kPa, respectively, were set to shear to 100 load cycles at low strains in the range of 1 percent to 4 percent, that are representative of field conditions. A strain rate of 1000 percent per hour was again used.

RESULTS AND DISCUSSION

Hydraulic Conductivity Testing

Measured values of hydraulic conductivity from the laboratory and field tests, and estimates from empirical equations are presented in Table 2. The estimated values are primarily based on the grain size of the finer fraction as noted in the previous section. The Hazen and Sherard methods result in predictions of hydraulic conductivity ranging from $1x10^{-1}$-cm/sec to $1x10^{-2}$-cm/sec (300 to 30-ft/day). Estimates of hydraulic conductivities using Moulton's expression resulted in values from $1x10^{-3}$-cm/sec to $1x10^{-4}$-cm/sec (3 to 0.3-ft/day).

Laboratory-measured hydraulic conductivities ranged from $1x10^{-1}$-cm/sec to $1x10^{-7}$-cm/sec (300 to $3x10^{-4}$-ft/day) (Table 2). The laboratory and field measured values are plotted for each material in Figure 4. With some exceptions, the in-situ hydraulic conductivities are 1 to 2 orders of magnitude lower than the laboratory-measured values. In general, the in-situ hydraulic conductivities ranged from $1x10^{-4}$-cm/sec to $1x10^{-5}$-cm/sec (0.3 to 0.03-ft/day). Several observations regarding the difference between the laboratory and field values are justified and presented as follows. First, although the laboratory specimens were compacted near optimum water content and maximum dry density, it is likely that the final conditions of the laboratory specimens did not truly represent those of the field-compacted specimens. Laboratory compaction was performed using an impact compactor (6); however, field compaction was performed with self-propelled vibrating equipment. Second, during permeability testing in the CHP, piping of some fine particles, occasionally large amounts, was observed. While such piping will increase the hydraulic conductivity of the laboratory specimens, no such piping was observed in the field tests. Given these two observations, it is understandable that the field hydraulic conductivity would tend to be lower than the laboratory-measured hydraulic conductivity.

The field and laboratory-measured hydraulic conductivities are plotted versus the predicted hydraulic conductivities in Figures 5 and 6, respectively. Predictions using the Hazen and Sherard expressions tend to over- predict both the field and the laboratory-measured hydraulic conductivities, although these expressions yielded slightly better results for the laboratory hydraulic conductivities. The Hazen correlation was developed for clean sands while the Sherard correlation was developed based on silty soils. Neither provides a good representation for well-graded aggregate with fines. Moulton’s expression was the only one specifically developed from correlations with roadway base materials. Results of the Moulton expression better estimated the field measured hydraulic conductivities.

Cedergren (1) has recommended that a drainable base layer on any pavement system be around 1-cm/sec (~3000-ft/day). From the results presented it has been shown that none of the tested materials meet this criterion.

$\overline{CU}$ Tests

Results of the moisture content before and after testing, and effective stress strength parameters from the static $\overline{CU}$ type triaxial compression tests are summarized in Table 3. Figures 7(a) and (b) present plots of stress difference versus strain and pore pressure versus strain, respectively. Strength tests performed on the material from Lanagan Quarry compacted at optimum water content indicated no cohesion intercept and an angle of internal friction that is representative of a highly dense granular soil (42 degrees). The stress difference-strain relationship was non-linear and continuously increasing until about 15 percent strain after which it leveled out. The pore pressures generated during shearing are representative of dense granular soils and helps to explain this behavior. Positive changes in pore pressures was minimum and was observed at the beginning of the test up to about 1percent axial strain, followed by negative changes that steadily continued until about 15 percent to 20 percent strain when they leveled out.

Cyclic, Stress-Controlled Tests

The stress difference-strain response to cyclic loading under stress-controlled conditions was similar to the response observed in the static CU triaxial tests. Negative changes in pore pressures, as high as 140-kPa, occurred almost immediately after beginning the tests and remained negative throughout the test, even during unloading. Figure 8(a) shows the shape and slope of the stress difference-strain curve of a sample tested at an initial effective confining stress of 14-kPa. It is clearly observed that the shape of the curve remained practically unchanged cycle after cycle throughout the test until reaching 20 percent strain. This is attributed to the significant negative pore pressure phenomenon that restrained the sample from dilating, as shown in Figure 8(b). Although the stress levels applied to the samples were less than the maximum stress difference, however, failure was defined by excessive bulging at 20 percent strain (9). This same behavior was also observed in other cyclic tests under stress-controlled conditions performed at lower stress levels under the same effective confining stress of 14-kPa.

Cyclic, Strain-Controlled Tests

The stress-strain response under strain-controlled conditions showed a totally different behavior compared to the stress-controlled type of test. Contrary to the stress-controlled tests, bulging of the samples was not experienced at such low strains; sample appearance remained practically unchanged upon completion of the tests. Figure 9 shows the results of the strain-controlled cyclic triaxial test performed at an effective confining stress 0f 28-kPa. Samples sheared to strains ranging between 1 percent and 4 percent, and tested at effective confining stresses of 14-kPa and 28-kPa experienced a small drop in pore pressure during the first load cycle, after which changes in pore pressure became positive until the end of the test. Figure 9(a) shows that pore pressures increased rapidly in the first two loading cycles until approaching the total confining pressure of 393 kPa from the third cycle on, and reducing the effective stress in the sample to near zero. Consequently, strength degradation caused a flattening of the stress difference-strain curve slope, which progressed with every load cycle. As seen in Figure 9(a), after applying twenty load cycles the principal stress difference attained was only about 10 percent of the initial maximum principal stress difference in the first cycle and the stress difference-strain curve had flattened out.

PRACTICAL IMPLICATIONS

One of the best ways to prevent damages to roads is to allow for complete and rapid drainage of any water that infiltrates into the pavement system. The specifications for the base materials used in roads should provide for the use of more permeable materials, or provide for alternate means to provide sufficient drainage, e.g. geocomposites. Strength and hydraulic conductivity testing should be performed on these new materials or systems to determine if they are capable of removing all the water that might infiltrate the pavement system in a timely manner, and if it will be strong enough to support the loads imparted by the pavement layers and traffic.

Although this paper is focused on the worst-case scenario of a non- draining base, additional work is being performed to evaluate the performance of freely draining granular bases under drained, partly drained, and cyclic loading conditions to compare the behaviors.

CONCLUSIONS

Using a poorly draining aggregate with low hydraulic conductivity as base material that may become saturated in the long term, compromises the performance and strength of road bases when subjected to cyclic loads. Results of cyclic triaxial tests under saturated, undrained conditions performed on compacted, Type-5 base material indicate that the loading nature has a significant impact on the pore pressure generation, and therefore on the strength of the soil. The strength of samples tested under stress-controlled cyclic loading remained unchanged due to the high negative changes in pore pressures in the sample during loading and locked-in during unloading. On the contrary, the strength of samples tested under strain-controlled cyclic loading to no more than 4 percent strain experienced significant degradation and reduction of effective stress to zero after the second load cycle due to build-up of positive pore pressures. Reductions of as much as 90 percent in principal stress difference were attained after 20 cycles. It is concluded that the current base material has too low of a hydraulic conductivity and may saturate and act in undrained behavior during the cyclic loading of traffic. Under cyclic loading conditions the saturated base looses most of its strength within a few load cycles. This behavior could explain the premature deterioration of some pavements.

ACKNOWLEDGMENTS

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LIST OF TABLES

LIST OF FIGURES

Source	D_{60} (mm)	D_{30} (mm)	D_{15} (mm)	D_{10} (mm)	C_u	C_z	% Pass #200 Dry Sieve	% Pass #200 Wet Sieve	g_{dmax} (KN/m^3)	OMC (%)	e	n
Ash Grove Quarry	5.8	1.1	0.3	0.2	38.7	1.4	6	12	21.5	7.0	0.21	0.17
Ash Grove Field	5.5	1.6	0.5	0.2	27.5	2.3	5	17	21.5	7.0	0.21	0.17
Idecker Quarry	9.5	3.7	1.2	0.5	19.0	2.9	3.3	13	19.7	10.0	0.32	0.24
Idecker Field	5.9	1.4	0.4	0.2	28.1	1.6	5	18	19.7	10.0	0.32	0.24
Lanagan Quarry	4.0	1.0	0.5	0.4	11.4	0.7	3	13	22.2	6.0	0.17	0.15
Riggs Quarry	7.0	1.3	0.2	0.1	50.0	1.7	6	19	21.5	8.0	0.21	0.17
Crawford Co.	7.0	1.7	0.5	0.3	23.3	1.4	1.7	14	21.8	8.0	0.19	0.16
Taney Co.	4.8	1.1	0.4	0.2	23.8	1.3	5.7	15	21.8	7.7	0.19	0.16

TABLE 1 Material Properties, Type 5 Base and Alternate Rock Fill

1-in = 25.4-mm
6.361-lb/ft^3 = 1-KN/m^3

TABLE 2 Predicted, Laboratory-Measured and Field-Measured Hydraulic Conductivities of Base Courses

Source	g_d (KN/m^3)	w (%)	Void Ratio	Predicted k Hazen k (cm/sec)	Sherard k (cm/sec)	Moulton k (cm/sec)	Lab k (cm/sec)	Field k (cm/sec)
Ash Grove Quarry	20.6	8	0.21	$2.25x10^{-2}$	$2.94x10^{-2}$	$4.1x10^{-5}$	$2.8x10^{-3}$	$1.9x10^{-3}$
Ash Grove Field	21.2	8	0.21	$4.0x10^{-2}$	$7.09x10^{-2}$	$7.01x10^{-5}$	**$3.0x10^{-6}$**	$1.9x10^{-3}$
Idecker Quarry	18.8	9	0.32	$2.5x10^{-1}$	$5.04x10^{-1}$	$3.24x10^{-3}$	$8.8x10^{-2}$	$4.6x10^{-5}$
Idecker Field	21.8	9	0.32	$4.41x10^{-2}$	$5.60x10^{-2}$	$7.02x10^{-4}$	**$3.0x10^{-7}$**	$4.60x10^{-5}$
Lanagan Quarry	21	4	0.17	$1.23x10^{-1}$	$8.75x10^{-2}$	$7.05x10^{-5}$	$5.4x10^{-3}$	$9.7x10^{-5}$
Riggs Quarry	21.7	9	0.21	$1.96x10^{-2}$	$1.85x10^{-2}$	$3.30x10^{-5}$	$5.20x10^{-3}$	$3.70x10^{-5}$
Crawford Co.	21.4	8.8	0.19	$9.0x10^{-2}$	$8.75x10^{-2}$	$1.43x10^{-4}$	**$4.5x10^{-6}$**	$9.10x10^{-5}$
Taney Co.	22.7	7.9	0.19	$4.00x10^{-2}$	$6.32x10^{-2}$	$3.64x10^{-5}$	$3.00x10^{-4}$	$1.90x10^{-5}$

Values in bold represent flexible wall permeameter tests
6.361-lb/ft^3 = 1-KN/m^3
1-cm/sec = 2835-ft/day

TABLE 3 Triaxial Test Results for the Lanagan Quarry Sample Based on Maximum Stress Difference Criterion

Maximum Dry Unit Weight (kN/m^3)	Optimum Water Content (%)	Sample No.	Effective Consol. Stress (kPa)	w (%) molding	w (%) during shear	Peak Eff. Principal Stress Difference (kPa)	Peak Volumetric Strain (%)	Effective Friction Angle	Effective Cohesion
22.2	6	1	13.8	6	8.9	97.4	16.2	42	0
		2	27.6	6.5	8.7	115	20.4		
		3	41.4	6	9	116.6	12.7		

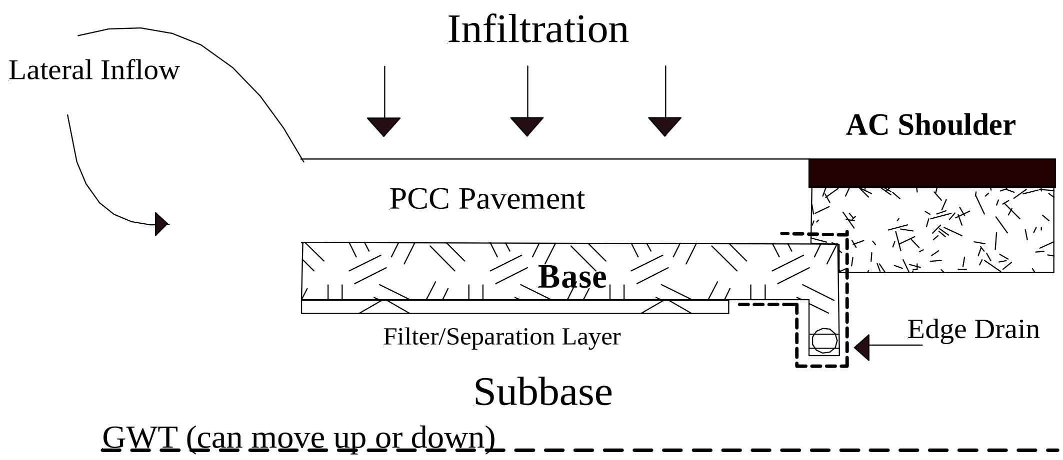


Figure 1 Typical pavement cross-section showing drainage system and possible sources of water.

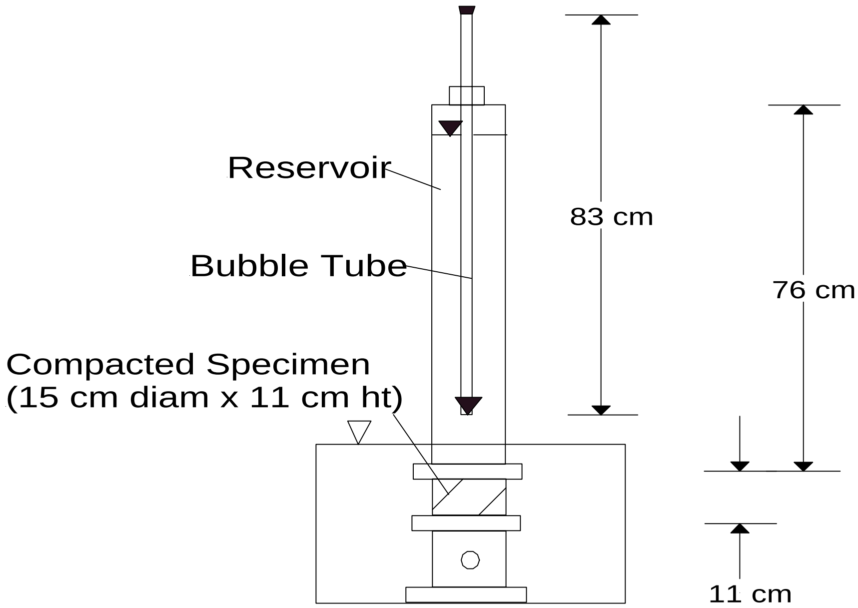


FIGURE 2 Constant head permeability (CHP) apparatus

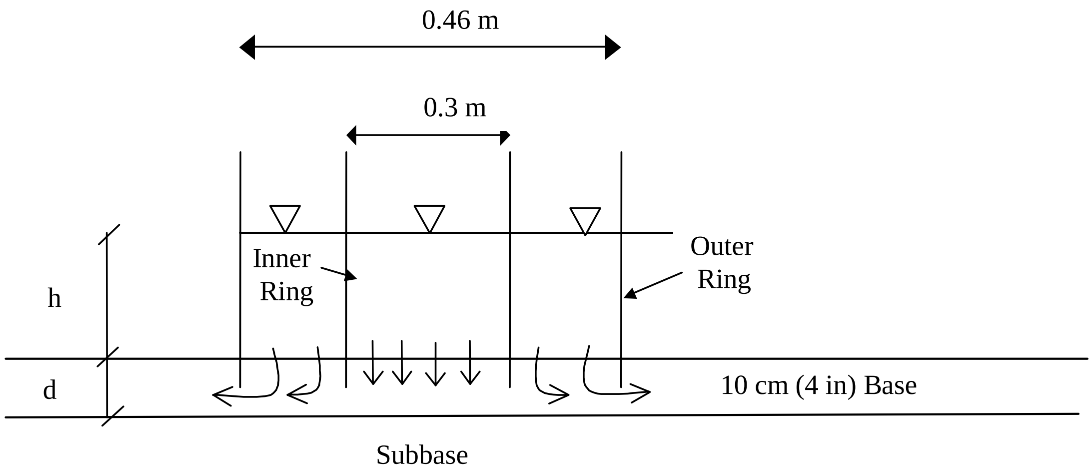


FIGURE 3 Cross section view of double ring infiltrometer (DRI) used for in-situ hydraulic conductivity testing of base course.

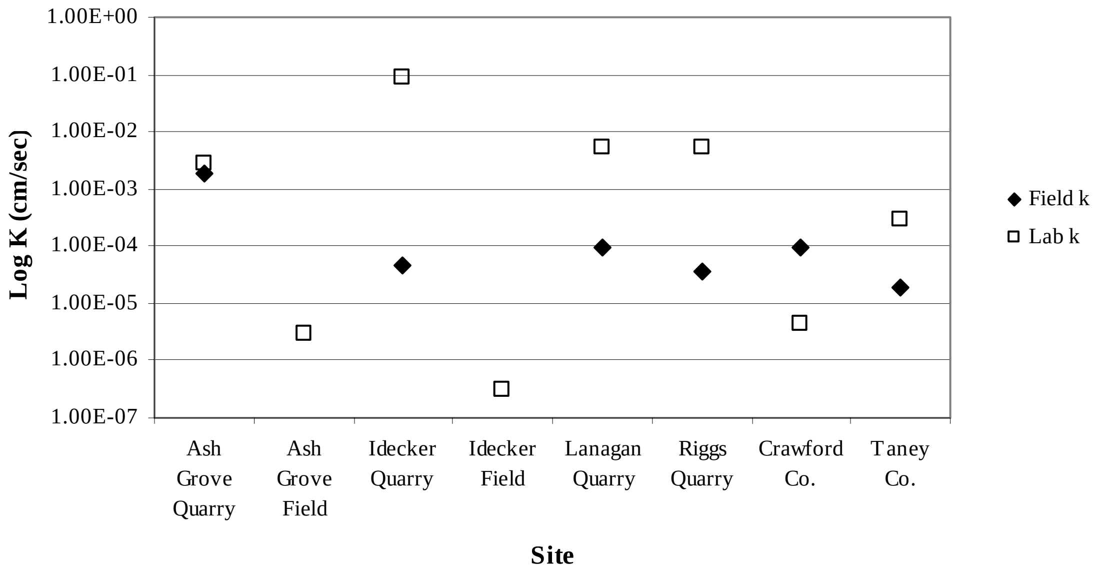


FIGURE 4 Laboratory and field hydraulic conductivities, Type 5 base material and alternate rock fill from various field locations in Missouri.

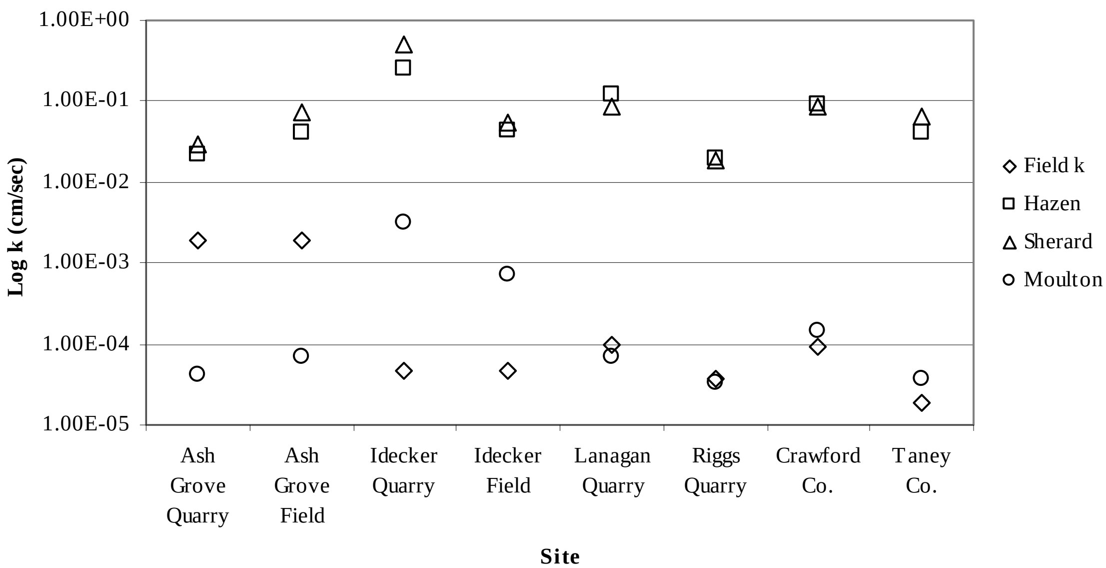


FIGURE 5 Field hydraulic conductivity and predicted hydraulic conductivity (dry sieve) for all sites tested

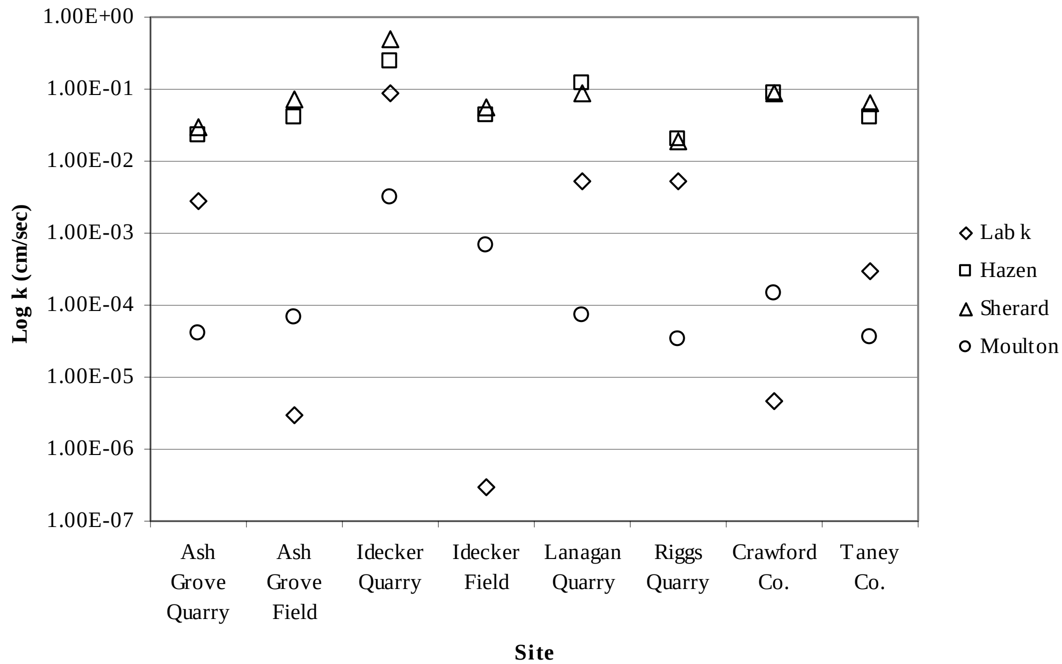


FIGURE 6 Laboratory hydraulic conductivity and predicted hydraulic conductivity (dry sieve) for all sites tested

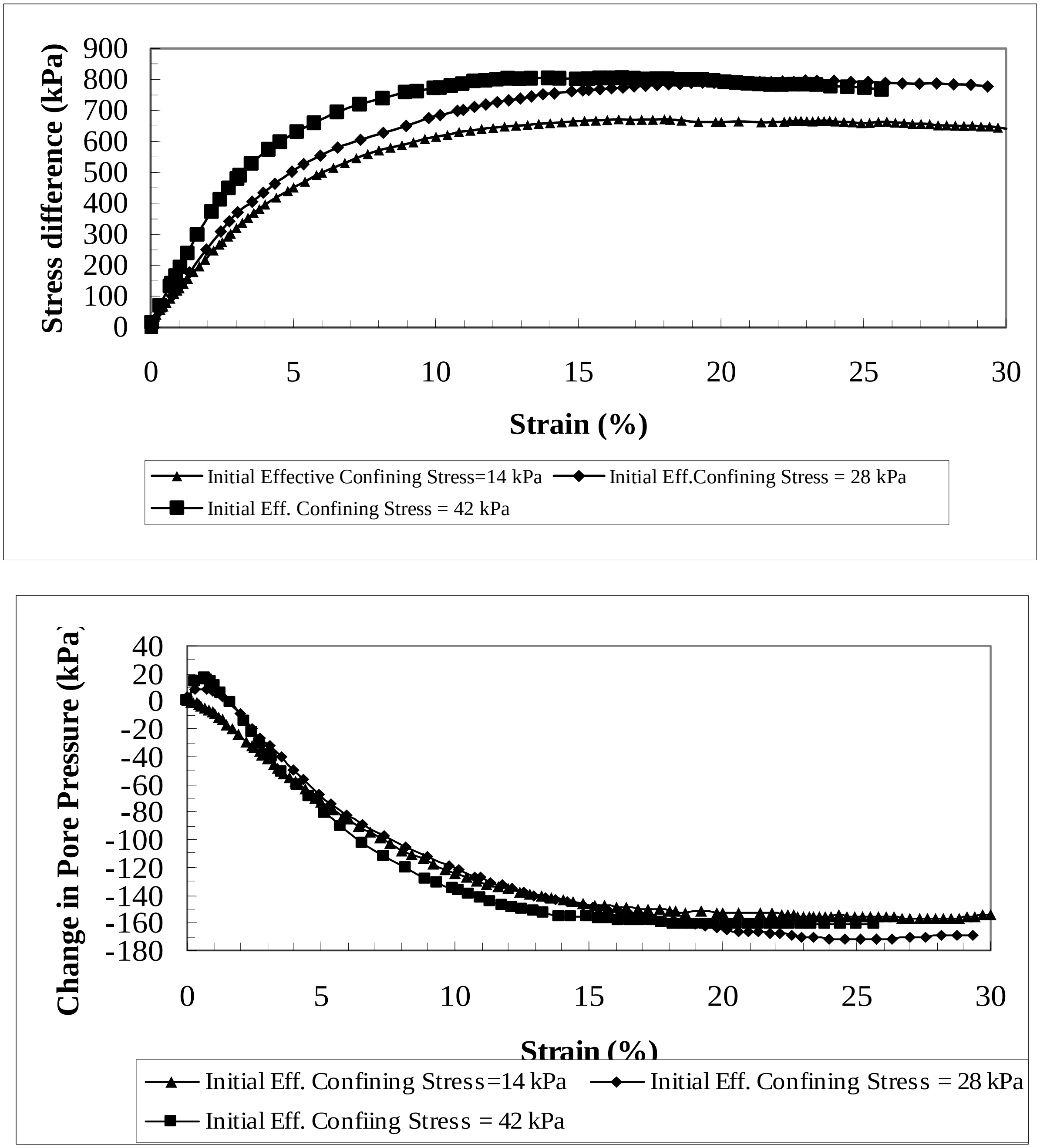
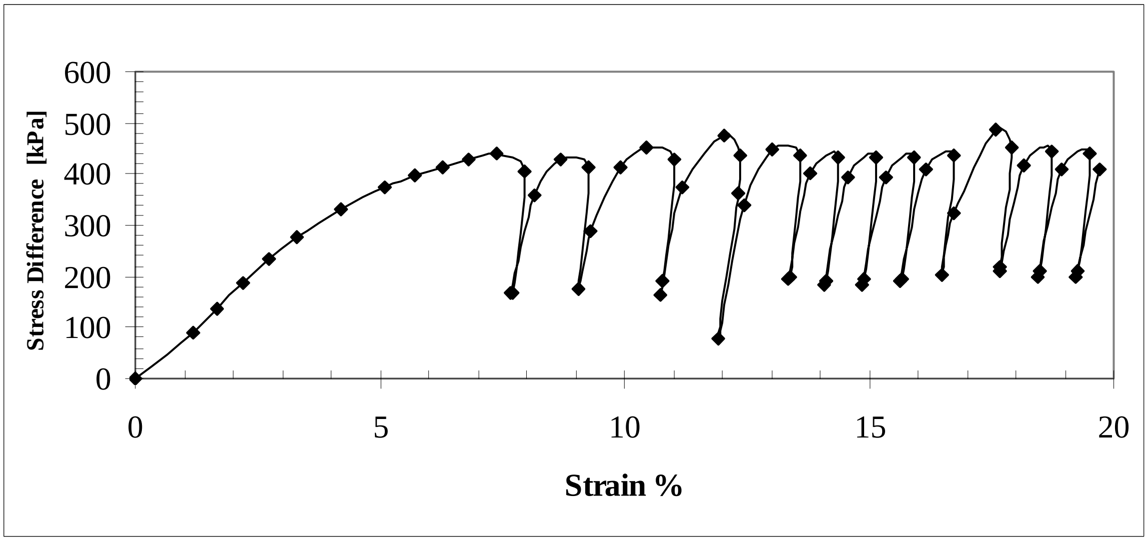


FIGURE 7 Results of CU-bar triaxial tests on Lanagan samples (a) Stress difference versus strain at different effective confining stresses. (b) Change in pore pressures versus strain at different effective stresses



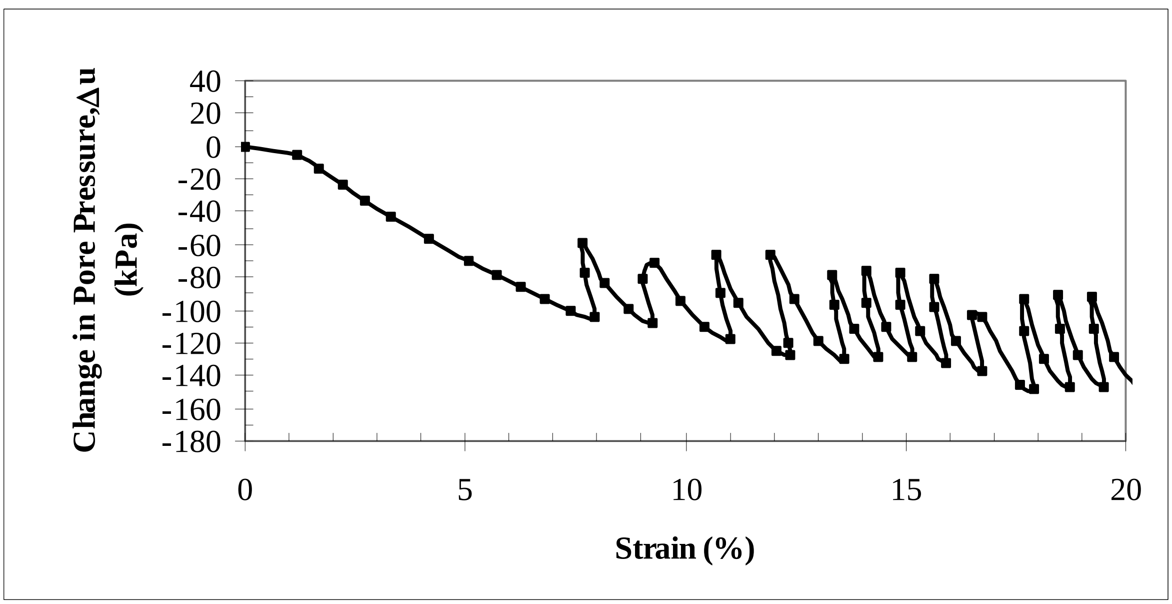


FIGURE 8 Stress-Controlled cyclic triaxial tests at initial effective confining stress =14-kPa, (a) Stress difference versus strain, (b) Change in pore pressure versus strain

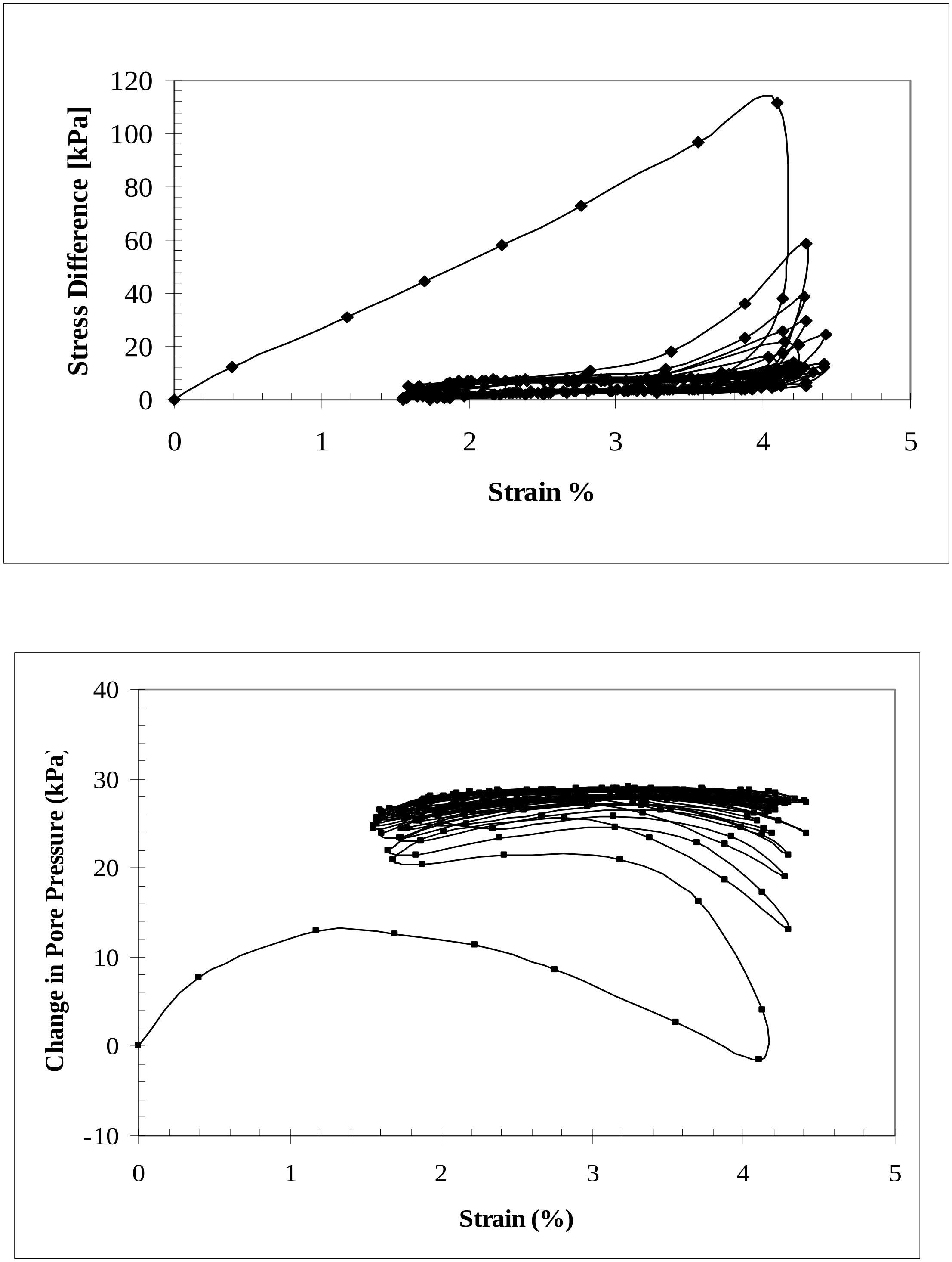


FIGURE 9 Strain-controlled cyclic triaxial test results at initial effective confining stress= 28-kPa, (a) Stress difference versus Strain, (b) Change in pore pressure versus strain.

LABORATORY AND IN-SITU HYDRAULIC CONDUCTIVITY OF PAVEMENT BASES IN MISSOURI

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ABSTRACT

For the last few decades, the design of pavement systems has not emphasized drainage. A major part of the damage found in pavement systems is caused by poor subsurface drainage, often the result of base materials that are not providing adequate drainage. In Missouri, two principal base course systems are used beneath the pavement. One is a 100 mm (4 in) layer of a well-graded aggregate with up to 15 percent fines. The alternate system includes a 600 mm (24 in) layer of rock fill. A program of laboratory and in-situ testing was performed on the base materials in order to assess their drainability. Grain size distributions were determined for all the materials, and in-situ and laboratory hydraulic conductivity testing was performed on 6 different materials. Drainage quality was also determined using pavement design software. In-situ hydraulic conductivities ranged from $2x10^{-3}$ to $4x10^{-5}$cm/sec. Laboratory hydraulic conductivities were in the range of $9x10^{-2}$ to $3x10^{-7}$cm/sec. The bases tested in this program averaged 1000 times lower hydraulic conductivities than typically necessary for good drainage of the pavement subsurface.

INTRODUCTION

Although the importance of well-drained pavement systems has been extensively documented (1, 2, 3), the design and construction of pavement systems has not emphasized drainage for the last few decades. During this time, designers have focused on the strength of the structural pavement layers, while little importance has been given to the drainage of the underlying pavement layers.

Water can flow into the base of the pavement through joints, cracks, and from groundwater sources (3). The presence of water, lack of drainage, and the cyclic loading due to vehicles result in high pore water pressures, which in turn cause a loss in effective strength in the base and eventually pavement failure (4). Also, fine materials from the subbase and base can be pushed out of the pavement through cracks and joints, creating voids underneath the surface layer. If the surface layer is not adequately designed, it can show signs of distress soon after construction.

In order to promote the rapid removal of water from the different layers of the pavement system, the current trend is to reinstate subsurface drainage into the systems. This is being accomplished by various means, such as using open-graded bases, stabilized bases, transverse piping, edge drains, and geosynthetics. Although analytical procedures exist for the design of pavement drainage systems (1, 3, 5), the procedures are either not being applied correctly or the construction process is incorrectly performed since adequate drainage is often not provided in existing pavements (6).

In Missouri, two principal base course systems are used beneath the paving. The first is a well-graded aggregate known as Type 5 Base material. This material is compacted into a 100 mm (4 in) lift, and the pavement placed directly on this material. The alternative base material consists of a 600 mm (24 in) layer of rock fill, over which the pavement is placed. The main objective of this project was to assess the drainability of the two types of base layers by performing a program of laboratory and in-situ hydraulic conductivity testing.

MATERIALS AND METHODS

Bulk samples of the base materials were obtained from supplier quarries, on-site stockpiles and from compacted, in-place roadway bases around Missouri (Table 1). Two different materials are used as base material in Missouri roadways. The first is a well-graded aggregate with particle sizes from 25 mm (1 in) to 0.075 mm (0.003 in), and is known as Type 5 base material. Based on Missouri Department of Transportation's specifications (7), the base material can have up to fifteen percent by dry weight passing the No. 200 sieve. The alternate base material consists of rock fill, with individual particles as large as 300 mm (12 in) and its interstices may be filled with a mixture of coarse aggregates to fines.

The first step of the testing program was to determine the grain size distributions of the samples recovered from the quarries and the field. The samples were brought to the laboratory and dry grain size distribution analyses were performed on them. After sieving, fine materials still adhered to the coarser particles of the specimens, so wet sieve analyses were performed on the samples to determine the total amount of fines of each material.

The second step of the testing program included the determination of the laboratory hydraulic conductivity of the different materials using a constant head, rigid wall permeameter (CHP). In this work the CHP-measured-hydraulic conductivities represent the upper bound (highest conductivity) for this property. One hundred and fifty millimeter (6 in) diameter specimens were prepared by compacting the material at maximum dry density and optimum water content using standard Proctor energy, based on method C from ASTM D698 (8). Compaction curves for various materials were provided by The Missouri Department of Transportation (MoDOT). Compacted specimens were placed in the permeameter, which provides flow under a constant head (Figure 1), and flows per unit area were determined under different hydraulic gradients. The hydraulic conductivity of the specimens was determined by fitting a best-fit straight line to the linear portion of the flow per unit area vs. hydraulic gradient plot. In cases where no measurable flow occurred using the CHP, the specimens were removed from the CHP, set up and tested in a flexible wall permeameter (9).

In-situ measurements of hydraulic conductivities were performed using a double ring infiltrometer (DRI) (Figure 2). Each ring was embedded approximately 50 mm (2 in) into the base by carefully hand-excavating a slot, inserting the ring, and filling the annulus with bentonite paste. The DRI provides a direct determination of the infiltration rate in the material being tested (10). Infiltration rate is defined as volume per time per unit area perpendicular to flow, or

$$I = \frac{Q/t}{A} \qquad (1)$$

where I is the infiltration rate [L]/[T], Q is the volume of infiltrated fluid [L^3], t is the time of infiltration [T], and A is the area [L^2] where the fluid was infiltrated.

The hydraulic conductivity is determined by dividing the infiltration rate by the hydraulic gradient. For this study, full-depth saturation (100 mm or 4 in) was assumed and the hydraulic gradient was taken as:

$$i = (h+d)/d \qquad (2)$$

where h is the height of water ponded above the base [L], and d is the thickness of the base [L] (Figure 2). The hydraulic conductivity is then calculated as:

$$k = \frac{I}{i} \qquad (3)$$

where k is the hydraulic conductivity [L]/[T], I is the measured infiltration rate and i is the hydraulic gradient.

Hydraulic conductivity values obtained from in-situ and laboratory tests were compared with values calculated using empirical equations. Three different methods were used to empirically estimate the hydraulic conductivity of the materials tested, all relying on the grain size

distribution of the media. The methods included:

A. Hazen Equation (11): $k = C\,D_{10}^{\;2}$ (4)

where D_{10} represents the particle size at which 10 percent by dry weight of the sample is smaller, in mm, and the hydraulic conductivity has units of cm/sec, and C is an empirical coefficient ranging from 1 to 1.5.

B. Sherard Equation (12): $k = 0.35 D_{15}^{\;2}$ (5)

where D_{15} represents the particle size at which 15 percent by dry weight of the sample is smaller, in mm, and the hydraulic conductivity has units of cm/sec.

C. Moulton Equation (3): $k = \dfrac{6.214\mathrm{x}10^{5}\,D_{10}^{\;1.478}\,n^{6.654}}{P_{200}^{\;0.597}}$ (6)

where n represents the porosity of the material, P_{200} represents the percent of material finer than the No. 200 sieve (0.075 mm), and the hydraulic conductivity has units of ft/day.

Drainage Requirements in Pavements 2.0 (DRIP 2.0) (5) is a computer program developed by the United States Department of Transportation and the Federal Highway Administration (FHWA) for the design of subsurface drainage. DRIP 2.0 provides a rating (Excellent to Poor) of a pavement's drainage system based on variables including infiltration, pavement geometry, and the drainage characteristics of the pavements' substructure. For this study, an infiltration based on a 50 mm (2 in) rain event, and infiltration coefficient of 0.67 were held constant. The geometries used included a cross sectional slope ranging from 0.005 to 0.020, and a longitudinal slope ranging from 0.005 to 0.040. The width of the road was assumed to be 8.5 m (28 ft) and the distance from the edge of the pavement to the edge drain was assumed to be 1.2 m (4 ft). The average unit weight of the base was assumed to be 21.2 KN/m^3 (135 lb/ft^3) and specific gravity of solids of 2.65. Laboratory measured and field measured hydraulic conductivity values were used for the analysis, although the DRIP program also allows the user to input the grain size distribution of the material to determine its hydraulic conductivity based on Moulton's equation (3,5), or to directly input values of hydraulic conductivity. The code analyzes all the variables based on the AASHTO Guide for Design of Pavement Structures and determines the time required for 50 percent drainage under a constant infiltration rate. The times to drain for the resulting scenarios for the laboratory and field measured hydraulic conductivities were determined and rated (by DRIP 2.0) for pavement performance.

RESULTS AND DISCUSSION

Dry Versus Wet Sieving

During the first phase of this project, it was noticed that after dry sieving was performed, the specimens still had fines attached to the coarser particles. Thus, it was deemed necessary to perform wet sieve analyses on the specimens. Most of the grain size distributions based on wet sieving for the materials tested are within the range specified by MoDOT, as seen in Figure 3. As shown in Figure 4, wet sieve analyses always resulted in higher fines contents than dry sieving. Average values for the percent fines determined by the dry sieve method and by the wet sieve method were found to be 4 and 15 percent respectively, with standard deviations of 1.6 and 2.6, respectively. The differences in fines content between materials were expected due to the different sources of the materials and the differing amounts of handling each material received prior to sampling.

Values for the coefficients of uniformity and gradation are presented in Table 2, as well as the USCS and AASHTO soil classifications. The coefficients and soil classifications were determined using information based on the wet sieve analysis, although for some materials D_{15} and D_{10} values had to be extrapolated from the grain size distribution plots. Both coefficients of uniformity and gradation show the materials to be well-graded sands with considerable silt-sized particles.

Hydraulic Conductivity

Measured and predicted hydraulic conductivities are shown in Table 3. The predicted values are based on the grain size of the finer fraction as previously noted. The Hazen and Sherard methods result in predictions of hydraulic conductivity ranging from 10^{-2} cm/sec to 10^{-4} cm/sec (29 to 0.3 ft/day). The Hazen correlation was developed for clean sands while the Sherard correlation was developed based on silty soils. Neither provides a good representation for well-graded aggregate with fines. Moulton's expression was the only one specifically developed from correlations with roadway base materials. Estimates of hydraulic conductivities using his expression resulted in values from 10^{-5} cm/sec to 10^{-7} cm/sec (0.03 to $3x10^{-4}$ ft/day).

In general, all laboratory compacted specimens were tested in the CHP; however, the sensitivity of the device is about 10^{-4} cm/sec (0.3 ft/day) below which, accurate hydraulic conductivity measurements are difficult to discern. Therefore, specimens that exhibited hydraulic conductivities lower than approximately 10^{-4} cm/sec (0.3 ft/day) were removed from the CHP, set up in a flexible wall permeameter and tested according to ASTM D5084 procedures (9). Laboratory-measured hydraulic conductivities ranged from 10^{-1} cm/sec to 10^{-7} cm/sec (285 to $3x10^{-4}$ ft/day) (Table 3). The laboratory and field measured values are shown in Figure 5. With three exceptions, the in-situ (field) hydraulic conductivities are 1 to 2 orders of magnitude lower than the laboratory-measured values. In general, the in-situ hydraulic conductivities ranged from 10^{-4} cm/sec to 10^{-5} cm/sec (0.3 to 0.03 ft/day). Several observations regarding the difference between the laboratory and field values are presented as follows. First, although the laboratory specimens were compacted near optimum Proctor conditions, it is possible that the final conditions of the laboratory specimens did not adequately represent those of the field-compacted specimens. The laboratory compaction was performed using a static compactor (8); however, the field compaction was performed with self-propelled vibrating equipment. Second, during permeability testing in the CHP, the piping of some fine particles (occasionally large amounts) was observed. While such piping will increase the hydraulic conductivity of the laboratory specimens, no such piping was observed in the field tests. Given these two observations, it is understandable that the field hydraulic conductivity would tend to be lower than the laboratory-measured hydraulic conductivity. Only the hydraulic conductivities measured in the flexible wall permeameter (3 cases) were lower than the field values. This is probably due to the increase of testing area during the field procedure. The in-situ test tested an area of 70000 mm^2 (110 in^2) while the lab test specimen was about one half that size. Thus, the field test was more likely to include macro-discontinuities than the lab specimen, resulting in higher measured hydraulic conductivities.

Prachantrikal (13) showed a reduction in hydraulic conductivity as the percentage of fines in the aggregate increased for a similar base material. The work was based on dry sieving of the aggregate and the percentages of fines were reported from 0 to 15 percent (dry mass basis) (Figure 6). The results of the laboratory-measured hydraulic conductivity and the corresponding dry sieve analyses for specimens from the project reported herein are also shown in Figure 6. The hydraulic conductivities measured in this project are over a smaller range in fines content and do not show any trend. Prachantrikal specimens were all from the same source aggregate, and the gradations were prepared in the laboratory. In the current work, every specimen is from a different source quarry and the gradations were variable (Table 2, Figure 3).

Hydraulic conductivities versus percentage fines (wet sieve) for all field and laboratory tests in this project are shown in Figure 7. The fines contents range from 12 to 19 percent (wet sieve, dry mass basis). A slight trend of decreasing hydraulic conductivity with increasing fines exists; however, at fines contents greater than approximately 15 percent, the hydraulic conductivities are in the 10^{-5} cm/sec (0.03 ft/day) range and changes in the fines content, e.g., 2 to 5 percent by dry mass, have limited influence on the hydraulic conductivity. At these fines contents, compaction density, particle segregation and the resulting macro-structure dominate the flow behavior.

As shown in Figure 7, the highest hydraulic conductivities measured in the laboratory correspond to values measured using the constant head permeameter, where no confining pressure is applied and it is prone to sidewall leakage. The laboratory hydraulic conductivity values measured using the flexible wall permeameter represent the lowest hydraulic conductivities measured in the laboratory. This is may be due to the higher confining pressures used during flexible wall testing, which can decrease or eliminate any sidewall leakage and close cracks or macro-structures. The hydraulic conductivities measured in the field tend to represent the intermediate hydraulic conductivities. The effective confining stress during testing was very low, and even with the use of a double ring infiltrometer, it is not guaranteed that the flow measured during testing is entirely vertical flow.

The field and laboratory-measured hydraulic conductivities are plotted versus the predicted hydraulic conductivities in Figures 8 and 9, respectively. Predictions using the Hazen and Sherard expressions tend to over predict both the field and the laboratory-measured hydraulic conductivities, although they provide better results when compared to the laboratory measured (CHP) hydraulic conductivities. Neither of these empirical correlations was developed for base materials so their lack of agreement with the measured values is not unexpected. It was expected that the results from the Moulton expression, developed for base materials, would better estimate the measured hydraulic conductivities, but due to the high amount of fines in the materials, the expression tends to under predict the measured laboratory and field hydraulic conductivity values. Only the values determined using the flexible wall permeameter were similar to the values determined using Moulton's equation.

Sensitivity Analysis

The results of the DRIP 2.0 analyses showed that pavement slope had negligible effect on the drainage performance, at least for drainage systems having hydraulic conductivities in the range of the systems tested in this project (10^{-2} to 10^{-7} cm/sec) (28 to $3x10^{-4}$ ft/day). Times to drain for the different materials are presented in Table 4. Time to drain values based on lab-measured hydraulic conductivities ranged between 0.1 days and 450 days. Those based on field-measured hydraulic conductivity ranged between 13 days and 254 days. Although values for hydraulic conductivity were entered in the analysis, the porosity of the materials was determined using wet size distribution data. It can be noticed that small variations in hydraulic conductivity can double the time to drain, and that most of the times to drain based on the field hydraulic conductivity tend to be higher than the times based on laboratory values, the only exception being the material from Crawford, which was tested using the flexible wall permeameter. No results are provided for the Taney material due to its high amount of fines, for which the software was unable to determine a porosity. As seen in Table 4, the best rating of drainage found for both laboratory and field hydraulic conductivity values was good, which corresponds to pavement systems that take longer than one day for achieving 50 percent drainage.

PRACTICAL IMPLICATIONS

The main objective of this research was to assess the drainability of materials being used as roadway base in Missouri. It was re-confirmed that wet sieving is necessary to accurately determine the fines contents of the base materials instead of the dry sieve analysis. None of the predicted hydraulic conductivities accurately represented the laboratory and field-measured hydraulic conductivities. In some cases, they over-estimated the field hydraulic conductivities and often by several orders of magnitude. Values obtained using Moulton's equation were the only values that correlated well with the hydraulic conductivities measured using the flexible wall permeameter. The fines contents should be checked after in-place compaction, since the amount of fines in a material typically increases as handling increases. It is recommended that in-situ hydraulic conductivity tests be performed on the in-place compacted materials to determine the actual drainability. The sensitivity study results indicated that, in general, a hydraulic conductivity of the base needs to be 1 cm/sec (2835 ft/day) or greater in order to obtain fair or better performance of the pavement drainage system; however, pavement system geometry and infiltration rate also influence the required values. In addition, the need for adequate drainage must be balanced by the requirement for stability of the base.

CONCLUSIONS

Field and laboratory permeability tests were performed on roadway base materials used in Missouri. The laboratory-measured hydraulic conductivities ranged from 10^{-1} cm/sec to 10^{-7} cm/sec (284 to $3x10^{-4}$ ft/day). The hydraulic conductivities measured in the rigid wall permeameter were 1 to 2 orders of magnitude higher than the field measured hydraulic conductivities (10^{-4} to 10^{-5} cm/sec) (0.3 to 0.03 ft/day). Those measured in the flexible wall device were 1 to 2 orders lower than the field values. Hydraulic conductivities estimated based on grain size distribution tended to both overestimate and underestimate the measured hydraulic conductivities and were poor representations of the field-measured values. A sensitivity analysis using the DRIP 2.0 computer code indicated that a high percentage of the base materials tested would provide only fair to very poor drainage performance. Good subsurface drainage is key to long term effective pavement performance and estimations and laboratory measurements of the hydraulic conductivities of base courses are currently the best guidance for design, however, these can result in unrepresentative values. Therefore, it is recommended that in-situ permeability testing, as well as flexible wall permeability testing of roadway bases be incorporated into quality control and assurance practices, especially if base materials with some fines contents are used for the design of pavement systems.

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LIST OF TABLES

LIST OF FIGURES

TABLE 1 Location of Base Course Samples and Details of Testing Performed

Location	Date	Tests Performed	Source	Sampling Location	Type
Rt. 71, McDonald Co.	Sept. 2001	DS, WS, LP, DRI	Lanagan Quarry	Quarry	Type 5
Rt. 13, St. Clair Co.	Sept. 2001	DS, WS, LP, DRI	Ash Grove Quarry	Quarry, field	Type 5
Rt. 63, Randolf Co.	Sept. 2001	DS, WS, LP, DRI	Riggs Quarry	Stockpile at site	Type 5
Rt. 71, Nodaway Co.	Sept, 2001	DS, WS, LP, DRI	Idecker Quarry	Stockpile at site, field	Type 5
Taney Co.	Dec. 2001	DS, WS, LP, DRI	Journegan Quarry	Stockpile at site	Rock Fill
Crawford Co.	Dec. 2001	DS, WS, LP, DRI	Unknown	Stockpile at site	Rock Fill

DS = dry sieve analysis; WS = wet sieve analysis; LP = laboratory permeability testing; DRI = on-site hydraulic conductivity testing using double ring infiltrometer.

TABLE 2 Material Properties, Type 5 Base and Alternate Rock Fill

Source	D_{60} (mm)	D_{30} (mm)	D_{15} (mm)	D_{10} (mm)	C_u	C_z	% Pass #200	γ_{dmax} (pcf)	OMC (%)	Void Ratio	Soil Classification USCS	AASHTO
Ash Grove Quarry	5.3	1	0.20	**0.05**	106.0	3.8	12	136.5	7.0	0.21	GP-GM	A-1-a
Ash Grove Field	3.8	0.5	**0.10**	**0.02**	190	3.3	17	136.5	7.0	0.21	SM	A-1-b
Idecker Quarry	9.1	2.7	0.25	**0.02**	455.0	40.1	13	125.0	10.0	0.32	GM-GC	A-1-a
Idecker Field	8.0	1.3	**0.03**	**0.01**	800	21.1	18	125.0	10.0	0.32	GM/GM-GC	A-1-b
Lanagan Quarry	4.0	0.8	0.10	**0.04**	100.0	4.0	13	141.0	6.5	0.17	SM	A-1-a
Riggs Quarry	4.8	0.4	**0.04**	**0.02**	237.5	1.7	19	137.0	8.0	0.21	SM	A-1-b
Crawford Co.	5.1	0.5	0.10	**0.05**	102.0	1.0	14	138.7	8.0	0.19	SM	A-1-a
Taney Co.	4.8	1	0.08	**0.02**	237.5	10.5	15	138.9	7.7	0.19	SM	A-1-a

Values in bold represent grain sizes extrapolated from grain size distributions based on wet sieving
25.4 mm = 1 in
0.157 kN/m^3 = 1 lb/ft^3

TABLE 3 Predicted, Laboratory-Measured and Field-Measured Hydraulic Conductivities of Base Courses

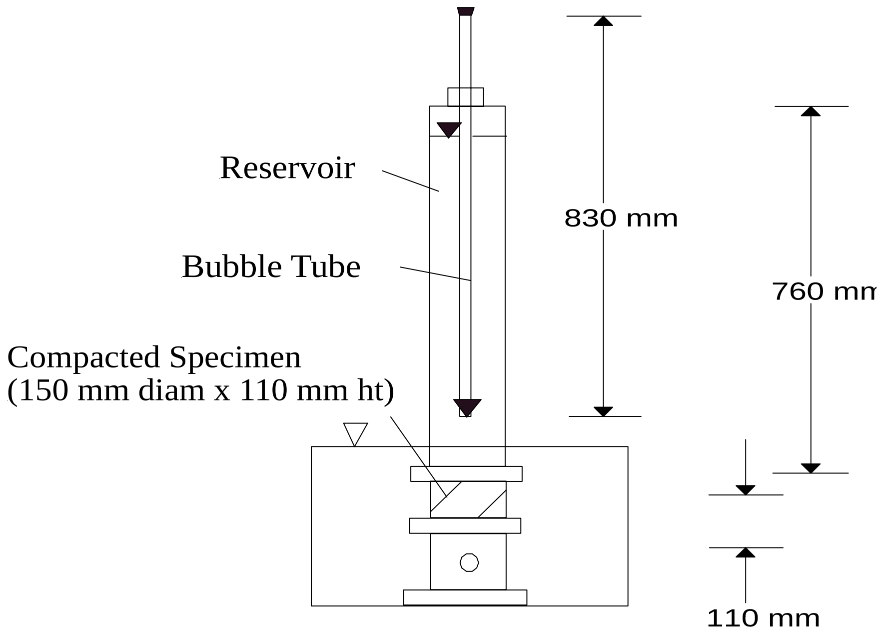
				Predicted k				
Source	**γ_d (kN/m^3)**	**w (%)**	**Void Ratio**	**Hazen k (cm/sec)**	**Sherard k (cm/sec)**	**Moulton k (cm/sec)**	**Lab k (cm/sec)**	**Field k (cm/sec)**
Ash Grove Quarry	20.6	8.0	0.21	$2.5x10^{-3}$	$1.4x10^{-2}$	$5.4x10^{-6}$	$2.8x10^{-3}$	$1.9x10^{-3}$
Ash Grove Field	21.2	8.0	0.21	$4.0x10^{-4}$	$3.5x10^{-3}$	$1.1x10^{-6}$	**$3.0x10^{-6}$**	$1.9x10^{-3}$
Idecker Quarry	18.8	9.0	0.32	$4.0x10^{-4}$	$2.2x10^{-2}$	$1.2x10^{-5}$	$8.8x10^{-2}$	$4.6x10^{-5}$
Idecker Field	21.8	9.0	0.32	$1.0x10^{-4}$	$3.2x10^{-4}$	$3.6x10^{-6}$	**$3.0x10^{-7}$**	$4.6x10^{-5}$
Lanagan Quarry	21.0	4.0	0.17	$1.6x10^{-3}$	$3.5x10^{-3}$	$1.2x10^{-6}$	$5.4x10^{-3}$	$9.7x10^{-5}$
Riggs Quarry	21.7	9.0	0.21	$4.0x10^{-4}$	$5.6x10^{-4}$	$9.4x10^{-7}$	$5.2x10^{-3}$	$3.7x10^{-5}$
Crawford Co.	21.4	8.8	0.19	$2.5x10^{-3}$	$3.5x10^{-3}$	$2.9x10^{-6}$	**$4.5x10^{-6}$**	$9.1x10^{-5}$
Taney Co.	22.7	7.9	0.19	$4.0x10^{-4}$	$2.0x10^{-3}$	$6.8x10^{-7}$	$3.0x10^{-4}$	$1.9x10^{-5}$

Values in bold represent flexible wall permeameter tests
0.157 kN/m^3 = 1 lb/ft^3
1 cm/sec = 2835 ft/day

TABLE 4 Laboratory and Field Hydraulic Conductivity and Time to Drain Values Based on DRIP2 analysis for Quarry Samples, Type 5 Base and Alternate Rock Fill.

	Measured k values		Time to Drain		Quality of Drainage	
Material	Laboratory k (cm/sec)	Field k (cm/sec)	Laboratory (days)	Field (days)	Laboratory	Field
Ash Grove	$2.8x10^{-3}$	$1.9x10^{-3}$	8.9	13	Poor	Poor
Idecker	$8.8x10^{-2}$	$4.6x10^{-5}$	0.1	253.7	Good	Very Poor
Lanagan	$5.4x10^{-3}$	$9.7x10^{-5}$	2.4	223.3	Fair	Very Poor
Riggs	$5.2x10^{-3}$	$3.7x10^{-5}$	1.1	163.9	Good	Very Poor
Crawford	$4.5x10^{-6}$	$9.1x10^{-5}$	449.5	16.6	Very Poor	Poor
Taney	$3.0x10^{-4}$	$1.9x10^{-5}$	-----	-----	-----	-----

1 cm/sec = 2835 ft/day



10 mm = 0.03 ft

FIGURE 1 Constant head permeability (CHP) apparatus.

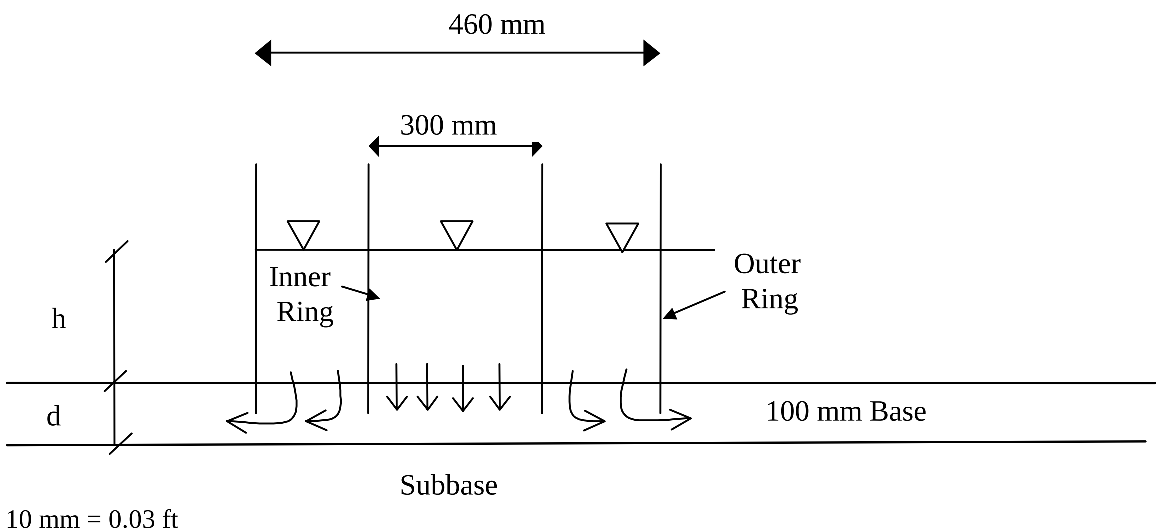


FIGURE 2 Cross section view of double ring infiltrometer (DRI) used for in-situ hydraulic conductivity testing of base course.

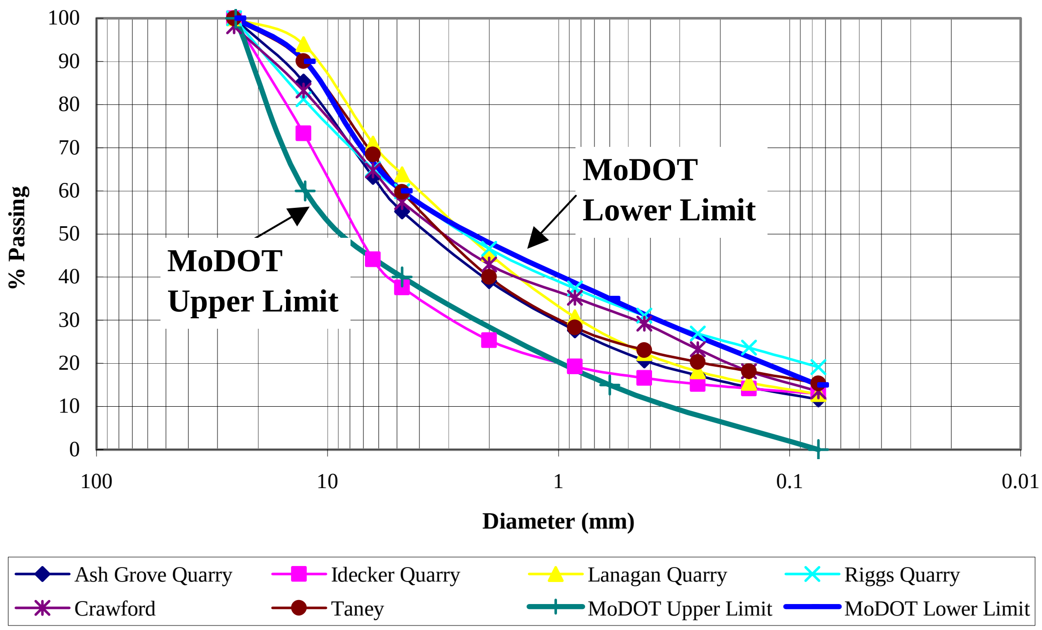


FIGURE 3 Grain size distribution based on wet sieve analysis and specification limits for base materials.

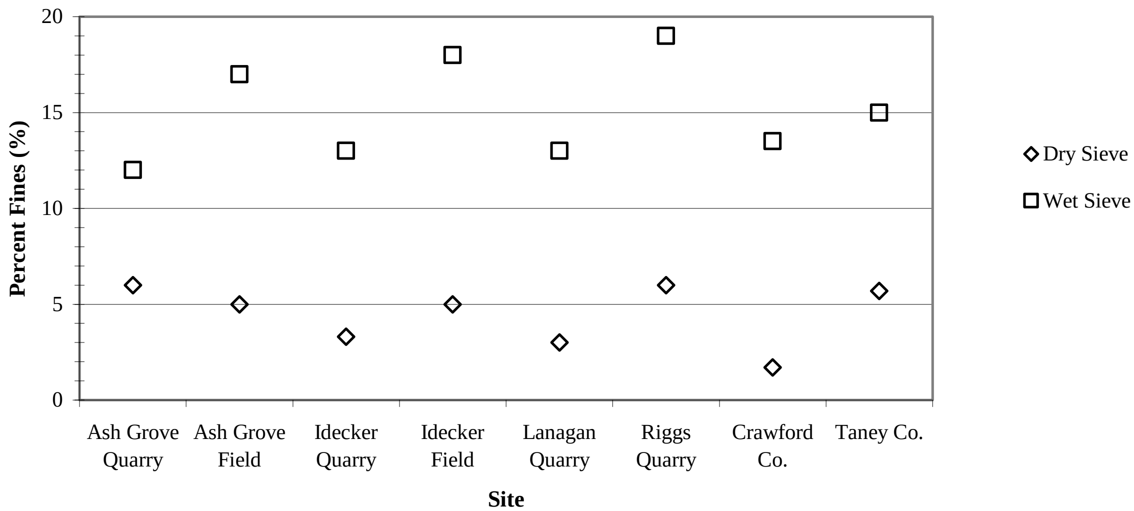
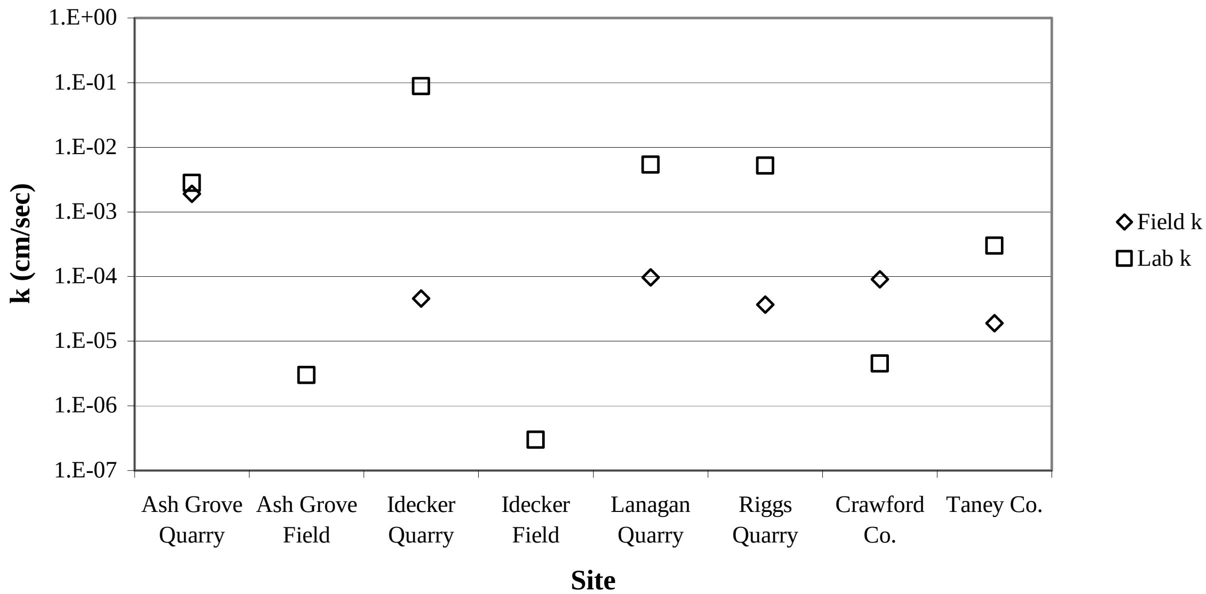
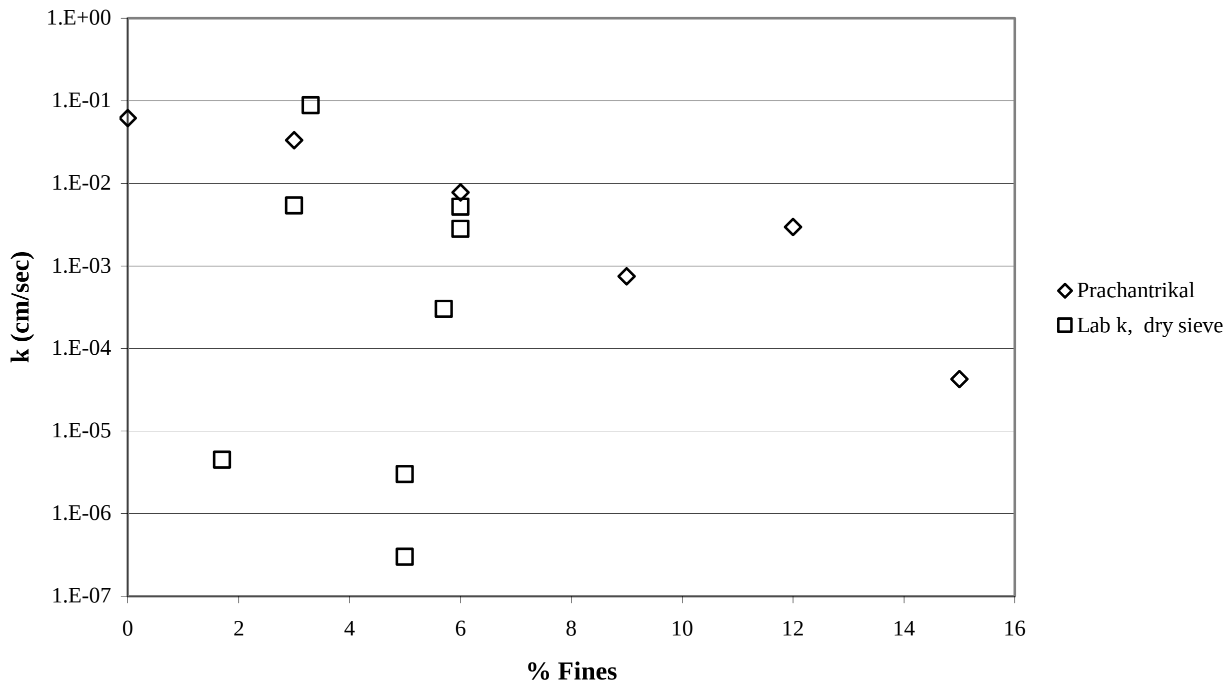


FIGURE 4 Percent fines by dry sieving and wet sieving, Type 5 base and alternate rock fill, from various sampling locations in Missouri.



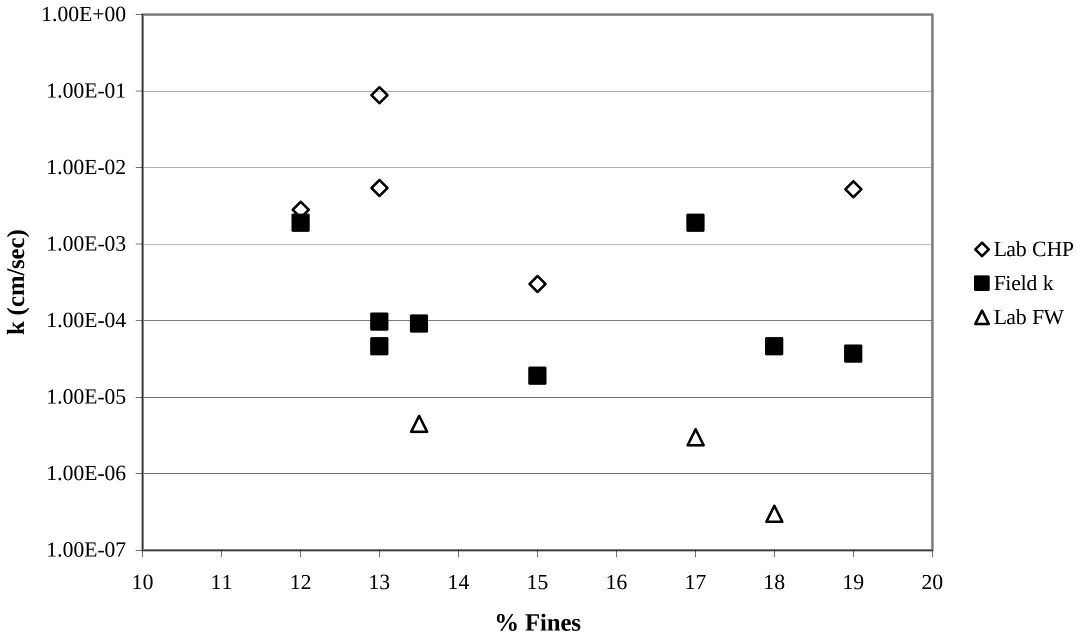
1cm/sec = 2835 ft/day

FIGURE 5 Laboratory and field hydraulic conductivities, Type 5 base and alternate rock fill from various field locations in Missouri.



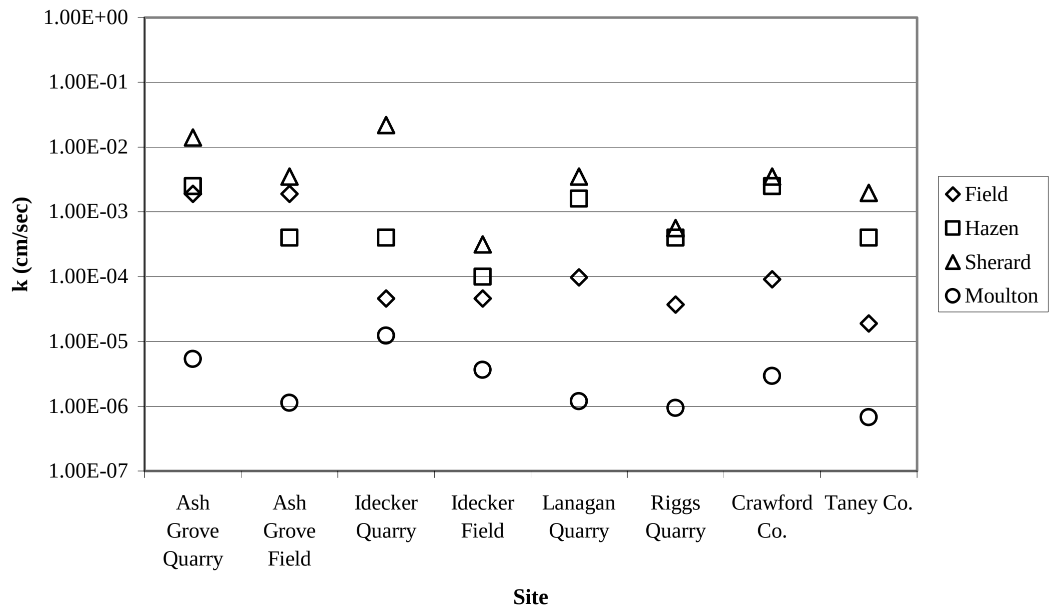
1 cm/sec = 2835 ft/day

FIGURE 6 Laboratory hydraulic conductivity versus percent fines (13).



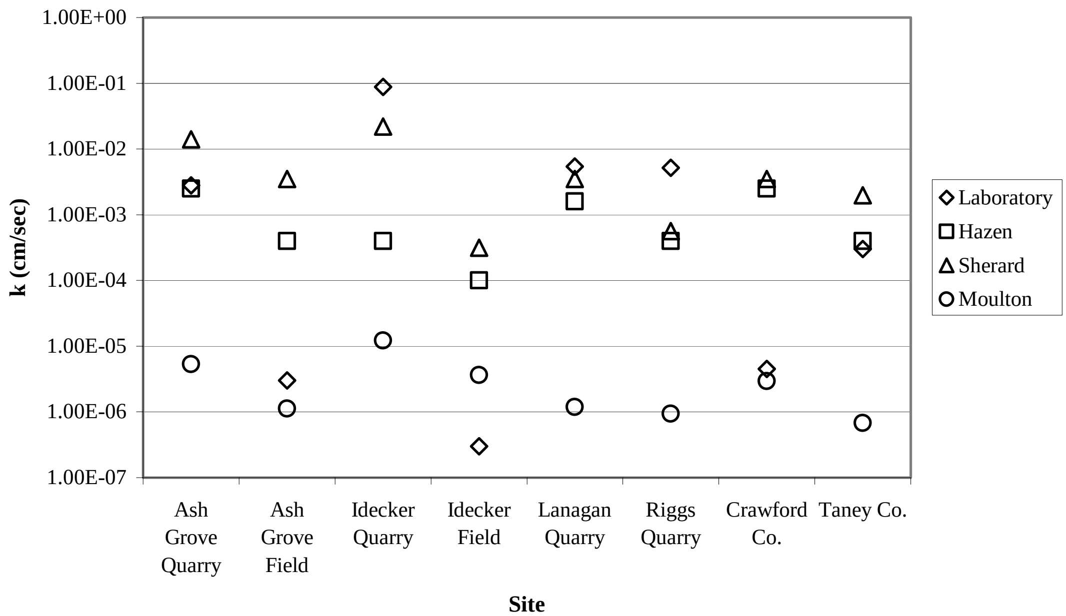
1 cm/sec = 2835 ft/day

FIGURE 7 Laboratory and field hydraulic conductivity measurements and percent fines (wet sieve), Type 5 base and alternate rock fill.



1 cm/sec = 2835 ft/day

FIGURE 8 Field hydraulic conductivity and predicted hydraulic conductivity (wet sieve) for all sites tested.



1 cm/sec = 2835 ft/day

FIGURE 9 Laboratory hydraulic conductivity and predicted hydraulic conductivity (wet sieve) for all sites tested.

Providing the Evidence for Evidence-Based Decision Making to Optimize the Performance of a Base Course

John J. Bowders[4], Jorge Parra[5], Awilda M. Blanco[2] and John P. Donahue[6]

Although the impact and role of water in the substructure of a pavement have long been known, these basic tenets have largely been ignored during recent times. The Missouri Department of Transportation specifies for most pavements to be constructed using a base course (base) that in some instances provides adequate, long-term support of the pavement, but in many cases does not. Effective changes should be made to the present design specification for the base in order to account for the factors that produce poor performance and serviceability. Although the present design is based on the resilient modulus values, it adds little to our understanding of the behavior of the base, i.e., no information on the pore pressure response. An understanding of the performance and/or shortcomings of the existing base are required. "Evidence-Based-Decisions" can then be made regarding changes to the base. This research program is designed to develop the "evidence" on which to make decisions regarding changes to the base. Such a process has a much higher success rate for any subsequently derived specification.

An extensive program has been underway to document the hydraulic conductivity and strength behavior of the base used in Missouri. In situ and laboratory hydraulic conductivity testing, as well as cyclic triaxial testing have been conducted in order to evaluate a relationship between drainage conditions, cyclic loading and the strength behavior of the base. Using cyclic triaxial tests we are able to document pore pressure, stress and deformation behavior of the base and to evaluate the impact of varying the drainage (hydraulic conductivity) of the base on the strength behavior. Test results indicate that the hydraulic conductivity of the base does not comply with requirements to assure long-term effective drainage. The shear strength properties of the compacted base were tested using strain-controlled criteria, for saturated-undrained conditions

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under cyclic loading. The results show that the saturated base looses most of its strength within a few load cycles due to generation of positive pore pressures which reduce the effective stress to zero during a small range of strains. Ongoing testing is aimed at developing a drainage-strength correlation for the base. Such a correlation will allow for the development of a specification for the base that optimizes strength by considering drainage performance.