**Abstract**

Crushed aggregate which met the gradation requirements of Arkansas Class 7 base course was sampled from five quarries in the state of Arkansas. The material taken from these quarries had a wide variety of mineral composition (limestone, dolomite, sandstone, syenite, novaculite) which represents a large portion of the mineral types used as Class 7 base material in the State of Arkansas. Three to five-ton samples from each quarry were fractionated and subsequently reblended in smaller batches so that the resulting gradations replicated the average of the historical gradation from each quarry, with the exception that the fines content (minus-#200 material) was varied from 6 to 16 percent in 2 percent increments. Replicate specimens for hydraulic conductivity testing and triaxial strength testing and suction testing were created for each of the six new gradations plus gradations at the upper and lower boundaries of the AHTD specification. Specimens were compacted in 6 inch diameter molds at optimum moisture content to 98 percent of maximum dry density, as determined by AASHTO T-180 method D. Hydraulic conductivity was measured on 6 inch diameter by 4 or 4.625 inch high specimens using both constant and falling head methods at relatively low gradients. Shearing strength was measured on 6 inch diameter by 12 inch high specimens using consolidated-drained triaxial testing at confining pressures of 5, 10 and 20 psi. The results of testing showed that the hydraulic conductivity of all specimens was so low that none of the base materials could be considered drainable at any of the tested fines contents. In addition, the decrease in hydraulic conductivity for fines contents of 10 to 16 percent was essentially negligible. The results of strength testing showed that the strength and modulus of these materials actually increased as the percent of fines increased from 8 percent up to 12 percent. Beyond 14 percent fines there was a slight decrease in both strength and modulus for three of the five quarries tested. As a result of this testing, recommendations were made to the Arkansas Highway and Transportation Department to increase the percentage of allowable non-plastic fines from 10 to 12 percent for Class 7 base course materials.
Investigation of the Affect of Fines on the Performance of Aggregate Base Course

By

Norman D. Dennis, Jr., Ph.D., P.E.
Richard M. Welcher, MSCE, P.E.
John H. Lawrence, MSCE, P.E.

Report prepared by
University Arkansas Department of Civil Engineering

for
Arkansas State Highway and Transportation Department, Little Rock, Arkansas

Contract Number MBTC-2027

August 2006
Disclaimer

This report reflects the views of the authors only, not the Arkansas State Highway and Transportation Department. The content of this report does not constitute a standard, regulation, or specification.
# Table of Contents

List of Figures .................................................................................................................. v
List of Tables ................................................................................................................... viii
Abstract .......................................................................................................................... 1
Chapter 1 - Introduction ................................................................................................. 3
  1.1 Problem Statement ................................................................................................. 3
  1.2 Background ............................................................................................................ 4
  1.3 Objectives .............................................................................................................. 5
Chapter 2 - Literature Review ....................................................................................... 7
  2.1 Base Course Properties ......................................................................................... 7
  2.2 Base Course Requirements .................................................................................... 9
  2.3 Neighboring DOT Aggregate Base Gradation Requirements ......................... 13
  2.4 Pavement Design considerations ......................................................................... 14
  2.5 Aggregate Testing ............................................................................................... 19
    2.5.1 Index Tests ..................................................................................................... 20
    2.5.2 California Bearing Ratio ............................................................................... 21
    2.5.3 R-Value ......................................................................................................... 26
    2.5.4 Texas Triaxial ............................................................................................... 29
    2.5.5 Resilient Modulus ......................................................................................... 31
    2.5.6 Triaxial Testing ............................................................................................. 38
      2.5.6.1 Types of Triaxial Testing ....................................................................... 47
      2.5.6.2 Mohr-Coulomb Failure Criteria ......................................................... 54
      2.5.6.3 Triaxial Test Advantages and Limitations ........................................... 55
  2.6 Selection of Test Method ....................................................................................... 56
  2.7 Hydraulic conductivity and Drainage .................................................................. 57
    2.7.1 Historical Significance ................................................................................. 62
  2.8 Problems Associated with Decreased Hydraulic Conductivity ....................... 64
  2.9 Measurement of Hydraulic Conductivity ............................................................. 66
    2.9.1 In-situ tests ................................................................................................... 69
    2.9.2 Laboratory tests ............................................................................................ 76
      2.3.2.1 Constant Head Test ............................................................................ 76
      2.3.2.2 Falling Head Test ................................................................................ 78
      2.3.2.3 Permeameter Types ......................................................................... 81
  2.10 Correlation of Hydraulic Conductivity to other Soil Parameters ..................... 84
  2.11 Relationship of Hydraulic Conductivity to other Soil Parameters ................. 92
  2.12 Design Philosophy ............................................................................................. 93
  2.13 Soil Suction ......................................................................................................... 94
Chapter 3 - Research Methodology .............................................................................. 102
  3.1 Site Selection ....................................................................................................... 102
  3.2 Sampling ............................................................................................................... 104
  3.3 Index Properties .................................................................................................. 104
  3.4 Model Blending .................................................................................................... 105
  3.5 Selection of Evaluative Methods ........................................................................ 109
  3.6 Triaxial Testing .................................................................................................... 111
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.7 Cyclic Triaxial Testing</td>
<td>114</td>
</tr>
<tr>
<td>3.8 Shear Strength Testing</td>
<td>115</td>
</tr>
<tr>
<td>3.9 Membrane Modulus Testing</td>
<td>117</td>
</tr>
<tr>
<td>3.10 Verification of Consolidated-Drained (CD) Test Conditions</td>
<td>120</td>
</tr>
<tr>
<td>3.11 Hydraulic Conductivity Testing</td>
<td>123</td>
</tr>
<tr>
<td>3.4.1 Equipment for Constant Head Hydraulic Conductivity Test</td>
<td>123</td>
</tr>
<tr>
<td>3.4.2 Constant Head Hydraulic Conductivity Test Procedure</td>
<td>124</td>
</tr>
<tr>
<td>3.4.3 Equipment for Falling Head – Rising Tail Hydraulic Conductivity Test</td>
<td>130</td>
</tr>
<tr>
<td>3.4.4 Falling Head – Rising Tail Hydraulic Conductivity Test Procedure</td>
<td>131</td>
</tr>
<tr>
<td>3.5 Suction Testing</td>
<td>137</td>
</tr>
<tr>
<td>Chapter 4- Results and Discussion</td>
<td>139</td>
</tr>
<tr>
<td>4.1 Material Properties</td>
<td>139</td>
</tr>
<tr>
<td>4.2 Hydraulic Conductivity</td>
<td>143</td>
</tr>
<tr>
<td>4.2.1 Problems Encountered during Hydraulic Conductivity Testing</td>
<td>148</td>
</tr>
<tr>
<td>4.2.2 Predicted Values of Hydraulic Conductivity</td>
<td>149</td>
</tr>
<tr>
<td>4.3 Suction Testing</td>
<td>158</td>
</tr>
<tr>
<td>4.3.1 Problems Encountered during Suction Testing</td>
<td>160</td>
</tr>
<tr>
<td>4.4 Drainage Analysis</td>
<td>167</td>
</tr>
<tr>
<td>4.5 Cyclic Loading Results</td>
<td>178</td>
</tr>
<tr>
<td>4.6 Staged Triaxial Results</td>
<td>185</td>
</tr>
<tr>
<td>4.7 Discussion of Staged Triaxial Results</td>
<td>189</td>
</tr>
<tr>
<td>4.8 Comparison of Elastic and Resilient Modulus Results</td>
<td>206</td>
</tr>
<tr>
<td>Chapter 5 - Conclusions</td>
<td>214</td>
</tr>
<tr>
<td>5.1 Shearing Strength and Modulus</td>
<td>214</td>
</tr>
<tr>
<td>5.2 Hydraulic conductivity and Drainage</td>
<td>217</td>
</tr>
<tr>
<td>5.3 Recommendations for Future Work</td>
<td>220</td>
</tr>
<tr>
<td>References</td>
<td>222</td>
</tr>
<tr>
<td>Appendix A – Material Properties</td>
<td>227</td>
</tr>
<tr>
<td>A.1 Gradation Curves</td>
<td>228</td>
</tr>
<tr>
<td>A.2 Grainsize Distribution Curves</td>
<td>234</td>
</tr>
<tr>
<td>A.3 Compaction Curves</td>
<td>240</td>
</tr>
<tr>
<td>Appendix B</td>
<td>246</td>
</tr>
<tr>
<td>B.1 Step by Step Pictorial Demonstrating Constant Head Test Methodology</td>
<td>246</td>
</tr>
<tr>
<td>B.2 Step by Step Pictorial Demonstrating Falling Head Test Methodology</td>
<td>254</td>
</tr>
<tr>
<td>B.3 Specimen Blending</td>
<td>265</td>
</tr>
<tr>
<td>B.4 Triaxial Sample Preparation</td>
<td>271</td>
</tr>
<tr>
<td>Appendix C Hydraulic Conductivity Test Results</td>
<td>280</td>
</tr>
<tr>
<td>Appendix D Suction Test Results</td>
<td>336</td>
</tr>
<tr>
<td>Appendix E Cyclic and Staged Triaxial Stress-Strain Plots</td>
<td>347</td>
</tr>
<tr>
<td>Appendix F Mohr-Coulomb Failure Envelop Plots</td>
<td>431</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

Figure 2.1 – Three Phases of Increasing Fines Content in Granular Material .............................................. 8  
Figure 2.2 – Nomograph Layer Coefficients, $a_2$, for Treated and Untreated Aggregate Base Course materials ................................................................. 19  
Figure 2.3 – Determining CBR for Water Content Range and Minimum Dry Unit Weight .............................................. 23  
Figure 2.4 – Hveem Stabilometer .......................................................... 26  
Figure 2.5 – Texas Triaxial Specimen .............................................. 31  
Figure 2.6 – Texas Triaxial Classification Chart .............................................. 31  
Figure 2.7 – Concept of Resilient Modulus Testing .............................................. 35  
Figure 2.8 – B Value vs. Degree of Saturation .............................................. 43  
Figure 2.9 – Square-Root-of-Time Method for Determination of C<sub>v</sub> .............................................. 45  
Figure 2.10 – Conventional Triaxial Chamber .............................................. 47  
Figure 2.11 – Undrained-Unconsolidated Triaxial Test Plot .............................................. 49  
Figure 2.12 – Mohr-Coulomb Failure Envelope .............................................. 54  
Figure 2.13 – The Effects of Pumping on Rigid Pavements .............................................. 60  
Figure 2.14 – Flexible Pavement Distress Due to Elevated Porewater Pressure .............................................. 61  
Figure 2.15 – Cross Sectional View of a Typical Pavement Section .............................................. 65  
Figure 2.16 – Definition of Hydraulic Gradient Used in Darcy’s Law .............................................. 67  
Figure 2.17 – Schematic Diagram of a Boutwell Permeameter .............................................. 70  
Figure 2.18 – Chart Showing $k_1/k_2$ v. $m$ Used to solve Horslev’s Equation .............................................. 72  
Figure 2.19 – Elevation Schematic of a Double Ring Infiltrometer .............................................. 74  
Figure 2.20 – Schematic Diagram of Typical Constant Head Test .............................................. 77  
Figure 2.21 – Schematic Diagram of Typical Falling Head Test .............................................. 80  
Figure 2.22 – Compaction Mold Used in the Assembly of a Rigid Wall Permeameter .............................................. 82  
Figure 2.23 – Assembled Flexible Wall Permeameter .............................................. 83  
Figure 2.24 – Values for $k$ versus $D_{10}$ for Uniformly Graded Sands .............................................. 87  
Figure 2.25 – Nomograph Used to Solve Moulton’s Equation .............................................. 90  
Figure 2.26 – Hydraulic Conductivities of Open Graded Bases and Filter Materials .............................................. 91  
Figure 2.27 – Effects of Fines Content on Strength, Density (VMA), Frost Heave Potential, and Hydraulic Conductivity .............................................. 92  
Figure 2.28 – Rise of Water and Pressure in a Capillary Tube .............................................. 96  
Figure 2.29 – Tube Suction Test Set-Up and Typical Results .............................................. 98  
Figure 2.30 – Capillary Wetting Phase of the TXDOT Triaxial Compression Test .............................................. 101  
Figure 3.1 – Geographical Distribution of Selected Quarries for this Research .............................................. 103  
Figure 3.2 – Historical and Model Blend Gradations .............................................. 106  
Figure 3.3 – Triaxial Specimen Split Mold (photo) .............................................. 111  
Figure 3.4 – Compaction of Specimen Using 3” Marshall Hammer (photo) .............................................. 111  
Figure 3.5 – Testing Apparatus (photo) .............................................. 113  
Figure 3.6 – Real-Time Stress vs. Strain Plot Generated During Staged Triaxial Testing .............................................. 116  
Figure 3.7 – Modulus vs. Strain for Single Membrane .............................................. 119
Figure 4.31 – Cohesion Intercept vs. Percentage Fines (Sharps) ........................................ 193
Figure 4.32 – Cohesion Intercept vs. Percentage Fines (Preston) ........................................ 194
Figure 4.33 – Cohesion Intercept vs. Percentage Fines (Black Rock) .................................... 194
Figure 4.34 – Cohesion Intercept vs. Percentage Fines (Glen Rose) ....................................... 195
Figure 4.35 – Cohesion Intercept vs. Percentage Fines (Granite Mountain) ............................... 195
Figure 4.36 – Shear Strength vs. Percentage Fines (Sharps) .................................................. 198
Figure 4.37 – Shear Strength vs. Percentage Fines (Preston) .................................................. 199
Figure 4.38 – Shear Strength vs. Percentage Fines (Black Rock) ............................................ 201
Figure 4.39 – Shear Strength vs. Percentage Fines (Glen Rose) .............................................. 202
Figure 4.40 – Shear Strength vs. Percentage Fines (Granite Mountain) ................................... 202
Figure 4.41 – Comparison of Cyclic and Staged Triaxial and Resilient Modulus Values (Sharps) .......................................................................................................................... 209
Figure 4.42 – Comparison of Cyclic and Staged Triaxial and Resilient Modulus Values (Preston) .......................................................................................................................... 209
Figure 4.43 – Comparison of Cyclic and Staged Triaxial and Resilient Modulus Values (Black Rock) ............................................................................................................... 210
Figure 4.44 – Comparison of Cyclic and Staged Triaxial and Resilient Modulus Values (Glen Rose) .................................................................................................................... 210
Figure 4.45 – Comparison of Cyclic and Staged Triaxial and Resilient Modulus Values (Granite Mountain) .......................................................................................... 211
LIST OF TABLES

Table 2.1 - AHTD Class 7 Material Requirements ........................................................................... 11
Table 2.2 – Comparison of Crushed Stone Aggregate Base Course Gradations ............................................. 15
Table 2.3 – AHTD Established Layer Coefficients for Pavement Design .................................................. 18
Table 2.4 – Testing Sequences for Base/Subbase Materials ..................................................................... 34
Table 2.5 – Thompson and Smith Gradation, Compaction, and PI Data ..................................................... 36
Table 2.6 – Thompson and Smith Resilient Moduli Data ........................................................................ 37
Table 2.7 – Summary of \( \eta \) Parameter Values .................................................................................. 52
Table 2.8 – Summary of \( C_v \) Estimations ......................................................................................... 52

Table 2.9 - Typical Coefficient of Permeability of Various Soils................................................................. 59
Table 2.10 – Rate at Which Water Flows through Uniformly Graded Sands at Various Heads ......................... 85
Table 2.11 – Hydraulic Conductivity (cm/sec) for Uniformly Graded Sands at Various Heads ......................... 86
Table 2.12– Electrical Properties and Field Performance of Granular Base Material .............................. 100
Table 3.1 – Parent Stone of the Selected Quarries ................................................................................ 103
Table 3.2 – Summary of M.B. 10% Fines Gradations ............................................................................ 107
Table 3.3 – Sharps Quarry Blending and Materials Quantities .................................................................. 108
Table 3.4 – Membrane Modulus Determination Raw Data ..................................................................... 118
Table 3.5 – Summary of \( \eta \) Parameter Values .................................................................................. 122
Table 4.1 – Material Properties and Classification of Class 7 base course Used in Study .............................. 142
Table 4.2 – Measured Values of Average Hydraulic Conductivity for Class 7 Base Course Model Gradations Used in Study ........................................................................................................ 145
Table 4.3 – Measured Values of Average Hydraulic Conductivity for Class 7 Base Course Upper and Lower Limit Gradations Used in Study ........................................................................ 145
Table 4.4 - Constant head permeability testing compared to falling head testing for 6 and 8 percent fines, all quarries ............................................................................................................. 149
Table 4.5- Comparison of Predicted Values with Measured Values .......................................................... 157
Table 4.6 – Total % Moisture Gain Versus Fines for Model gradation Used in Study ................................. 160
Table 4.7 – Quality of Pavement Structure Drainage .............................................................................. 171
Table 4.8 – Recommended \( m_i \) Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements ............................................................................ 171
Table 4.9 -Times to Drain Using the Average Hydraulic Conductivity ......................................................... 173
Table 4.10 – Rainfall Intensity Required to Saturate Pavement ................................................................ 177
Table 4.11 – Verification of CD Test Conditions (M.B. 10% Gradations Shown) ........................................ 187
Table 4.12 – Summary of \( \phi, c, \) and \( \tau \) Model Blend Triaxial Testing (Sharps) ......................................... 204
Table 4.13 – Summary of \( \phi, c, \) and \( \tau \) Model Blend Triaxial Testing (Preston) ........................................ 204
Table 4.14 – Summary of \( \phi, c, \) and \( \tau \) Model Blend Triaxial Testing (Black Rock) .............................. 205
Table 4.15 – Summary of \( \phi, c, \) and \( \tau \) Model Blend Triaxial Testing (Glen Rose) .............................. 205
Table 4.16 – Summary of $\phi$, c, and $\tau$ Model Blend Triaxial Testing (Granite Mountain). 206
Table 4.17 – Bulk Stress and Triaxial Secant Modulus Summary Data................................. 208
ABSTRACT

It has been long recognized that pavement service life is highly dependent on the strength characteristics and permeability (hydraulic conductivity) of the underlying base material. Current Arkansas State Highway & Transportation Department (AHTD) specifications limit the maximum fines (material passing a # 200 sieve) content in its Class 7 base aggregate to 10 percent. To decrease costs associated with the production of granular base course, aggregate producers in the State of Arkansas have proposed that the upper limit on fines be increased. The overall objective of this study was to determine if an increase in fines content, above the currently specified 10 percent, would have detrimental effects on base course performance.

In this study samples of Class 7 base course from five (5) different quarries, representing a wide range of geologic materials used in the State of Arkansas, were tested in the laboratory to measure hydraulic conductivity, moisture retention and strength properties at varying fines contents. The focus of this laboratory work was to determine the effect of fines on the strength, hydraulic conductivity and moisture retention of unbound aggregate base course materials. For this study a model gradation blend was developed for each quarry based upon historical gradations and AHTD specifications. Model gradations were developed for 6 percent, 8 percent, 10 percent, 12 percent, 14 percent, and 16 percent fines. The quantity of material retained on the # 40 and larger sieves did not vary for the different gradations from each quarry, only the percentage of fines was varied. A modified proctor of the upper and lower limit gradations from each quarry was performed to establish target dry densities and optimum moisture contents to be used in preparing specimens for testing.
In accordance with AASHTO specifications, replicate 152mm (6 inch) diameter by 117mm (4.265 inch) high samples containing 6 percent and 8 percent fines were tested by the constant head method (T-215). Samples containing 10 percent, 12 percent, 14 percent, and 16 percent fines were tested by the falling head method (ASTM D5084, method C). Two (2) replicate samples were tested for each percentage of fines. Samples were tested for capillary rise (suction) by a procedure developed in this study.

Strength and modulus testing was conducted in accordance with ASTM 2850, using consolidated-drained triaxial testing procedures on replicate 150mm (6 inch) diameter by 305 mm (12 inch) high specimens. Each test specimen was subjected to an initial stress controlled cyclic loading at an effective confining pressure of 5 psi to establish and initial modulus value. This loading was followed by strain controlled staged testing at 5, 10 and 20 psi to establish strength parameters c and φ.

It was determined that historical gradations of Class 7 base course used in Arkansas have hydraulic conductivity values that are from 2 to 6 orders of magnitude lower than what is considered to be “freely draining”. Any increase in the percentage of fines above the current maximum of 10 percent will have only minor effects on the hydraulic conductivity of the granular base course and will not affect its drainability at all. In addition, is was determined that strength and stiffness of Class 7 bases from the selected quarries actually increased as fines contents increased from 8 to 12 percent. For some quarries strength decreased marginally at 14 percent fines, while for others the strength remained essentially constant at fines contents of 14 and 16 percent. Overall the variation in strength for fines contents ranging from 6 to 16 percent was generally less than 10 percent.
CHAPTER 1
INTRODUCTION

1.1 Problem Statement

A typical pavement section consists of a surficial wearing course, which may be flexible asphaltic concrete hot mix or rigid Portland cement concrete. This wearing course is underlain by a base course and finally a subgrade layer. The intermediate layer of material within the pavement structure, the base course, typically consists of crushed stone aggregate, soil-aggregate mixtures, or granular materials treated with either Portland cement or bitumen of varying blends and mixtures. The base course must have significant strength and resistance to deformation in order to adequately support the wearing course, limiting distresses in that layer such as rutting and fatigue cracking. The base course must also protect the subgrade from excessive stresses imposed by traffic loading. Most base courses are also relied upon to drain water from the pavement structure.

The most commonly used unbound aggregate base coarse material within the state of Arkansas is a crushed stone aggregate conforming to the gradation requirements of Class 7 base as defined by the Arkansas Highway and Transportation Department’s Standard Specifications for Highway Construction (AHTD, 1996). Accordingly, Class 7 material was selected as this study’s target material. The Arkansas Highway and Transportation Department (AHTD) currently mandates that the amount of fines (percentage of material passing the U.S. Standard No. 200 sieve) within Class 7 base
range from 3 percent to 10 percent. Class 7 base course is a crusher run material and the ability to control the fines content is a major concern to aggregate producers. Rarely does Class 7 base fail to meet the AHTD gradation requirements because it has less than 3 percent fines; it normally fails because of excessive fines. Aggregate producers incur additional costs if tighter production controls and processing like washing or additional screening of the product are required to create a blend meeting the AHTD upper limit of 10 percent fines.

Many surrounding state departments of transportation allow for a percentage of fines in their respective equivalent of Class 7 crushed stone aggregate base to exceed the AHTD’s imposed maximum value of 10 percent. As a result several aggregate producers have posed the question of why the maximum amount of fines cannot be increased in the state of Arkansas. The purpose of this research is to investigate the effect of fines on the strength and stiffness characteristics of Class 7 crushed stone aggregate base course. From this research, design engineers will be able to evaluate the beneficial or detrimental strength characteristics of base course with fines contents that both fall within the current AHTD acceptance criteria and those with a percentage of fines higher than the currently established upper limit.

1.2 Background

The performance of base course is generally evaluated using two criteria: 1) strength, and 2) permeability. These two criteria are heavily influenced by the amount of fines present within the base material. AHTD Class 7 base must meet particle size tolerances based upon established percentages passing the 37.5 mm (1-1/2 in.), 19.0 mm
The gradation of a base course will place it into one of three fines content conditions: 1) little to no fines, 2) excessive fines, and 3) an intermediate fines content (Yoder and Witczak, 1975). The amount of fines present can affect the shear strength stiffness and permeability of the aggregate base course. The definition of what percentages constitute too few, too many, and appropriate amounts of fines has largely been left to individual state departments of transportation to evaluate and determine. The Arkansas Highway and Transportation Department has determined that the optimal performance for Class 7 base with respect to strength and permeability is achieved through a material that has a fines content ranging from 3 percent to 10 percent of the total weight.

The optimum fines content with respect to strength may differ from the required fines content to promote adequate drainage in the base material. The two conditions are essentially inversely proportional (Yoder and Witczak, 1975). A base material is considered free draining if little to no fines are present. Generally, fines contents of less than 2 percent are required for a material to be considered free draining (Barton, 2004). The permeability of the base material drops substantially as the percentage of fines is increased above the 2 percent threshold. The affect of fines on the drainage characteristics and strength of the Class 7 base material is the focus of this study.

1.3 Objectives

The purpose of this study was to investigate the optimal fines content for AHTD Class 7 aggregate base courses in relation to strength and drainage performance criteria.
This objective was accomplished by evaluating the strength performance for the base as measured by angle of internal friction (\(\phi\)) values obtained with traditional triaxial testing and classical Mohr-Coulomb failure analysis. As well as evaluating the drainage performance by conducting both constant head and falling head hydraulic conductivity testing and suction tests. Specimens of Class 7 base course having varying blends of fines, both inside and outside the current AHTD acceptance tolerances, were examined. Fines contents of these blends are varied from 6 to 16 percent. The information gathered in this study will form the basis for suggesting an optimum fines content (or range of contents) at which the aggregate base strength/stiffness is greatest and drainage performance remains unhindered.
CHAPTER 2
LITERATURE REVIEW

The main objective of this study is to investigate the affect of fines on the strength and drainage characteristics of unbound aggregate base course classified as Class 7 by the AHTD. A full understanding of pavement design concepts is essential to developing the evaluative criteria for strength performance. It is also necessary to discuss the advantages and disadvantages of various field and laboratory tests and procedures used to evaluate strength and drainability.

2.1 Base Course Properties

Base course is defined as the layer of material that lies immediately below the wearing surface of a pavement. The base course is constructed directly on subbase course or on natural subgrade if no subbase course is used. Its major function within the pavement structure is to provide structural support for the wearing course although it is often relied upon to provide drainage for the pavement structure (AASHTO, 1993) as well. When necessary, the base course may also provide protection against frost action.

The requirement for base course to carry load and distribute stress means stability of the aggregate is of utmost importance. The factors contributing to stability of the base course aggregate include particle-size distribution, particle shape, relative density, internal friction, and cohesion (Yoder and Witczak, 1975). High angles of internal friction are required to resist load-induced deformation. Internal friction, in turn, depends largely upon density, particle shape, and grain-size distribution of the
material. Of these properties, Yoder and Witczak state that the proportion of fine to coarse fraction is the most important in relation to overall base course stability.

Figure 2.1 presents three states of base aggregate with differing fines contents. An aggregate with little to no fines relies on coarse aggregate intergranular contact for strength development (Fig. 2.1a). Aggregate in this state has a relatively low density but is permeable and not subject to frost action. Base courses lacking fines are difficult to place properly and compact due to the lack of confinement of the coarse aggregate. Aggregate blends with few fines require some form of confinement before they will exhibit high shearing resistance. The lack of fines makes coarse particle angularity and shape more important to development of aggregate friction.

![Figure 2.1: Three phases of increasing fines content in granular materials (Yoder and Witczak, 1975).](image)

An aggregate blend that has a sufficient amount of fines to fill voids between the coarse particles (Fig. 2.1b) will continue to develop strength from coarse particle contact but with the addition of a fines matrix the coarse particles become effectively “locked” into place because the void spaces are filled with a relatively incompressible material. The density of the aggregate blend is increased but its permeability is reduced. Placement and compaction of these aggregate blends is feasible using conventional
construction methods. Aggregate base with an amount of fines sufficient to fill the voids between the coarse aggregate particles (typically between 8% to 12% by weight depending on the parent geological material) is ideal from the standpoint of stability and will have a relatively high shearing strength in both confined and unconfined conditions (Yoder and Witczak, 1975).

Excessive fines in aggregate matrix will reduce shear strength and stability by pushing the coarse particles away from each other, decreasing point-to-point contact. In essence, the coarse particles “float” in the matrix of fine grained particles. Figure 2.1c illustrates this condition. The density of unbound aggregate bases with excessive fines is low and they should be considered as impervious and highly susceptible to frost action. Aggregate blends with excessive fines are not desirable for pavement base courses.

2.2 Base Course Quality Requirements

The AASHTO design guide does not provide specific quality requirements for base courses. Instead, the Guide relies upon AASHTO’s Manual for Highway Construction or ASTM Specification D-2940, “Graded Aggregate Material for Bases and Subbase for Highways and Airports,” for quality guidance. The authors of the AASHTO Guide encourage the development of quality and acceptance criteria for base course material by individual construction agencies, such as state departments of transportation, or municipalities, based upon their experience with locally available materials and accepted construction methods within their region.

Section 303 of the AHTD Standard Specifications for Highway Construction (AHTD, 1996) establishes acceptance and construction criteria for aggregate base course material. Section 303 specifies that “Class 7 shall be any mechanically crushed natural
rock or stone of igneous, sedimentary, or metamorphic origin produced from a solid geological formation by quarrying methods”. Table 2.1 summarizes the requirements for AHTD Class 7 base, measured in accordance with the following AASHTO test specifications:

- T 11, Materials Finer Than 0.075 mm (No. 200) Sieve in Mineral Aggregates by Washing
- T 27, Sieve Analysis of Fine and Coarse Aggregate
- T 89, Determining the Liquid Limit of Soils
- T 90, Determining the Plastic Limit and Plasticity Index of Soils
- T 96, Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
- AHTD Test Method 304, Test for Crushed Particles in Aggregate.
Table 2.1: AHTD Class 7 material requirements (Section 303, AHTD, 1996).

<table>
<thead>
<tr>
<th>Sieve, mm (U.S.)</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5 (1-1/2&quot;)</td>
<td>100</td>
</tr>
<tr>
<td>25.0 (1&quot;)</td>
<td>60 – 100</td>
</tr>
<tr>
<td>19.0 (3/4&quot;)</td>
<td>50 – 90</td>
</tr>
<tr>
<td>4.75 (No. 4)</td>
<td>25 – 55</td>
</tr>
<tr>
<td>0.425 (No. 40)</td>
<td>10 – 30</td>
</tr>
<tr>
<td>0.075 (No. 200)</td>
<td>3 – 10</td>
</tr>
<tr>
<td>Maximum Liquid Limit (Minus No. 40 Material)</td>
<td>25</td>
</tr>
<tr>
<td>Maximum Plasticity Index (Minus No. 40 Material)</td>
<td>6</td>
</tr>
<tr>
<td>Minimum Percent Crusher-Run Material</td>
<td>90</td>
</tr>
<tr>
<td>Maximum Percent Wear by the Los Angeles Test</td>
<td>45</td>
</tr>
</tbody>
</table>

The AHTD Standard Specifications do not include other material property requirements for the crushed stone material, such as specific gravity or alkali reactivity. A material is deemed acceptable for use as Class 7 base as long as the crusher produced blend meets the requirements established in Table 2.1.

The authors of the AASTHO Guide recommend untreated aggregate base course be compacted to a minimum of 95 percent of maximum laboratory dry density, determined in accordance with AASHTO T 180, Method D, (Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and 457-mm (18-in.) Drop) or the equivalent (AASHTO, 1993). The maximum particle size allowed in the compaction mold according to T 180 Method D procedures is 19.0 mm (3/4 in.). Class 7 base can
have anywhere between 10% to 50% of particles greater than 19.0 mm (3/4 in.) in size; therefore, the presence of oversize particles in placed aggregated base course must be accounted for in Proctor curve development. The AHTD does not accept the premise that the AASHTO T 224 coarse particle correction equation provides an adequate maximum dry density and optimum moisture content representative of the in-place base course. Section 301.04(f) of the AHTD Standard Specifications requires coarse aggregate replacement in moisture density testing in lieu of coarse aggregate correction in accordance with AASHTO T 224, (Correction for Coarse Particles in the Soil Compaction Test) as required by AASHTO T 180. The AHTD specification states that coarse particles retained on the 19.0 mm (3/4 in.) sieve shall be replaced by an equal mass of material passing the 19.0 mm (3/4 in.) sieve and retained on the 4.75 mm (No. 4) sieve. The AHTD does not accept the premise that the AASHTO T 224 coarse particle correction equation provides an adequate maximum dry density and optimum moisture content representative of the in-place base course. The current rock replacement method has been used by the AHTD for a long time with acceptable results (Westerman, 2004). Since AASHTO leaves the decision as to which portion(s) of the Standard Specifications to apply to the individual governing agencies AHTD has elected to apply the provisions of a previous version of the AASHTO testing specifications. The AHTD requires base course aggregate to be compacted to at least 98 percent of maximum laboratory dry density, determined in accordance with the same AASHTO T 180, Method D (AHTD, 1996). The specification further states that the base material should be compacted at or near the optimum moisture content, but makes no reference to the allowable variance from that value. The AHTD originally required aggregate base to
be compacted to 100% of the T 180 maximum dry density when field density was
determined by the sand cone method. According to the AHTD (Westerman, 2004), “The
sand cone method of in-place measurement of field density (AHTD Test Method 345)
yielded higher percentages of maximum dry density than did the nuclear density (AHTD
Test Method 330) method of measurement. The decision was made to specify that field
density measurements be only by the nuclear method. The required minimum
percentage compaction was then lowered to 98% in order to account for differences
between percent compaction readings obtained by the sand cone and nuclear test
methods.”

2.3 Neighboring DOT Aggregate Base Gradation Requirements

ASTM D 2940 “Standard Specification for Graded Material for Bases and Subbases for Highways and Airports” only provides general guidance for establishing suitable gradations for base course materials. The final base gradation is essentially determined by the transportation agencies or other governing bodies (Federal Aviation Administration, U.S. Army Core of Engineers, etc.). This is necessary due to differences in regional geological formations and the availability of different types of quarry produced aggregate. A comparison of the course aggregate gradations recommended by ASTM and the gradation specifications for states surrounding Arkansas for material similar to AHTD Class 7 is presented Table 2.2.

The table reflects the acceptance variability between states with respect to crushed stone aggregate base gradations. Not only are no two of the surveyed state’s base course requirements the same but no two states even use the same sieve sizes for establishing the desired gradation. The state most restrictive on fines content, Kentucky,
allows for a maximum of 8% passing the 0.075 mm (No. 200) sieve. Missouri has the highest maximum fines content of 15%. States such as Tennessee and Texas do not establish gradation tolerances for minus 0.075 mm (No. 200) particles. Tennessee treats material passing the 0.15 mm (No. 100) sieve as fines. In addition to standard Atterberg limits and gradation requirements, Texas requires that the aggregate base comply with either a minimum compressive strength or meet a certain material classification according the Tex-117-E Texas triaxial test procedures. Among state DOT’s, only Kentucky’s maximum allowable fines content is lower than the current Arkansas maximum value of 10%.

2.4 Pavement Design Considerations

The 1993 AASHTO Guide for the Design of Pavement Structures is the basis for flexible pavement design in the State of Arkansas (AHTD, 1996). This design procedure is based on the results of the extensive AASHO Road Test conducted in Ottawa, Illinois, in the late 1950’s and early 1960’s. The data gathered from the Road Test led to the development of a set of empirical performance equations. The initial performance equations resulting from this study were based on climate, soil conditions, and pavement materials unique to the test location. The original equations were later modified to be applicable to other regions of the country. The current design equation for flexible pavement design used in the AASHTO Guide is shown below (AASHTO, 1993).

\[
\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN + 1) - 0.20 + \frac{\log_{10} \left[ \frac{APS}{4.2 - 1.5} \right]}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07
\]

where \( W_{18} = \) predicted number of 18-kip equivalent single axle load applications,
Table 2.2: Comparison of crushed stone aggregate base course gradations.

<table>
<thead>
<tr>
<th>State/Agency</th>
<th>ASTM</th>
<th>Arkansas</th>
<th>Kansas</th>
<th>Kentucky</th>
<th>Louisiana</th>
</tr>
</thead>
<tbody>
<tr>
<td>Designation</td>
<td>D 2940</td>
<td>Class 7</td>
<td>AB-1</td>
<td>Crushed Stone</td>
<td>Stone</td>
</tr>
<tr>
<td>Sieve Size, mm (U.S.)</td>
<td>Percent Passing</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>63.5 mm (2-1/2 in.)</td>
<td>100</td>
<td>100</td>
<td>-</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>50.0 mm (2 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>44.5 mm (1-3/4 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1-1/2 in.)</td>
<td>95 - 100</td>
<td>95 - 100</td>
<td>90 - 100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>25.0 mm (1 in.)</td>
<td>60 - 100</td>
<td>-</td>
<td>-</td>
<td>90 - 100</td>
<td></td>
</tr>
<tr>
<td>22.2 mm (7/8 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>19.0 mm (3/4 in.)</td>
<td>70 - 92</td>
<td>70 - 95</td>
<td>70 - 95</td>
<td>70 - 100</td>
<td></td>
</tr>
<tr>
<td>12.5 mm (1/2 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>9.5 mm (3/8 in.)</td>
<td>70 - 100</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>35 - 55</td>
<td>40 - 65</td>
<td>15 - 55</td>
<td>35 - 65</td>
<td></td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>-</td>
<td>30 - 46</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2.0 mm (No. 10)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.80 mm (No. 30)</td>
<td>12 - 25</td>
<td>-</td>
<td>5 - 20</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>-</td>
<td>10 - 30</td>
<td>10 - 22</td>
<td>12 - 32</td>
<td></td>
</tr>
<tr>
<td>0.150 mm (No. 100)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>0 - 9</td>
<td>2 - 10</td>
<td>0 - 8</td>
<td>5 - 12</td>
<td></td>
</tr>
<tr>
<td>Maximum Liquid Limit</td>
<td>25</td>
<td>25</td>
<td>-</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Maximum Plasticity Index</td>
<td>6</td>
<td>6</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>State/Agency</th>
<th>Mississippi</th>
<th>Missouri</th>
<th>Oklahoma</th>
<th>Tennessee</th>
<th>Texas</th>
</tr>
</thead>
<tbody>
<tr>
<td>Designation</td>
<td>Crushed Stone Base</td>
<td>Type 5</td>
<td>Type A</td>
<td>Class B</td>
<td>Type A- Grade 1</td>
</tr>
<tr>
<td>Sieve Size, mm (U.S.)</td>
<td>Percent Passing</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>63.5 mm (2-1/2 in.)</td>
<td>100</td>
<td>100</td>
<td>-</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>50.0 mm (2 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>44.5 mm (1-3/4 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1-1/2 in.)</td>
<td>-</td>
<td>100</td>
<td>95 - 100</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>25.0 mm (1 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>22.2 mm (7/8 in.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>19.0 mm (3/4 in.)</td>
<td>-</td>
<td>-</td>
<td>65 - 95</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>12.5 mm (1/2 in.)</td>
<td>-</td>
<td>80 - 90</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>9.5 mm (3/8 in.)</td>
<td>50 - 85</td>
<td>30 - 75</td>
<td>-</td>
<td>50 - 70</td>
<td></td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>35 - 65</td>
<td>40 - 60</td>
<td>25 - 60</td>
<td>35 - 55</td>
<td>35 - 55</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2.0 mm (No. 10)</td>
<td>25 - 50</td>
<td>-</td>
<td>20 - 43</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>-</td>
<td>15 - 35</td>
<td>-</td>
<td>15 - 45</td>
<td></td>
</tr>
<tr>
<td>0.80 mm (No. 30)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>15 - 30</td>
<td>8 - 26</td>
<td>-</td>
<td>15 - 30</td>
<td></td>
</tr>
<tr>
<td>0.150 mm (No. 100)</td>
<td>-</td>
<td>-</td>
<td>4 - 15</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>0 - 10</td>
<td>0 - 15</td>
<td>4 - 12</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Maximum Liquid Limit</td>
<td>-</td>
<td>25</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Maximum Plasticity Index</td>
<td>NP</td>
<td>6</td>
<td>3 (by TDOT equation)</td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>
\[ Z_R = \text{standard normal deviate for a given level of reliability}, \]

\[ S_o = \text{combined standard error of the traffic prediction and performance prediction}, \]

\[ \Delta PSI = \text{difference between the initial design serviceability index, } p_0, \text{ and the terminal serviceability index, } p_t, \text{ and} \]

\[ M_R = \text{resilient modulus, psi}. \]

The outcome of the performance equation is the required pavement structural number (SN) which is indicative of the total pavement thickness required to satisfy the design criteria (traffic, life, reliability, etc.).

To account for the differing blends of automobile and truck loading on a given roadway, the concept of an equivalent single axle load (ESAL) was developed. This method reduces traffic streams having different axle loads and axle configurations into a comparable number of passes of an 18-kip single axle load over the selected design period. The reliability input term provides some level of assurance that the pavement will perform as intended over the design period (Huang, 1993). A standard deviation is a convenient way to specify the variability of design factors. The use of a standard deviation covers variances in pavement material properties, design traffic, unexplained variance, and inadequacies of the design procedure or lack of fit of the design equations. A standard deviation representative of local conditions is selected and used in conjunction with reliability concepts. The use of reliability and standard deviation are required to take into account the variability of traffic estimates and the inherent uncertainties of performance predictions.
Design serviceability loss (ΔPSI) considers the level of service for newly constructed roadways (initial service level) and the lowest acceptable level of service below which corrective measures such as rehabilitation, reconstruction, etc. must be completed (terminal service level). The initial service index from the AASHO Road Test was 4.2 for flexible pavements. Terminal service indices of 2.5 for major highways and 2.0 for highways with lower traffic are recommended (Huang, 1993). The design serviceability loss for a new interstate, for example, would be 1.7 (4.2 minus 2.5). The PSI ratings developed during the AASHO Road Test were based on the evaluations of a subject rating personnel group. The panel rated differing pavement areas based upon their assessed need for repair work or corrective measures (Huang, 1993 from Carey and Irick, 1960).

The SN can be expressed in terms of the strength properties of the materials in the pavement system and their thicknesses. Once the required pavement SN has been developed, suitable pavement cross-sections are developed by manipulating the input parameters shown in Eqn. 2.2

\[
SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3
\]

(2.2)

where \(a_i\) = \(i^{th}\) layer coefficient

\(D_i\) = \(i^{th}\) layer thickness (inches), and

\(M_i\) = \(i^{th}\) layer drainage coefficient.

Table 2.3 contains the layer coefficient (\(a_i\)) values established by the AHTD of pavement section courses. The most widely used aggregate base course materials in Arkansas are either Class 7 quarry run crushed stone or Class 5 river run crushed stone.
Selection of either class material for use is largely based on regional availability. Class 7 quarry run crushed stone is accepted as the superior base course material as evidenced by the higher established layer coefficient. The Class 7 layer coefficient of 0.14 is deemed applicable for a particular base course provided its material gradation falls within the established acceptance parameters and it possess the stated aggregate properties. This uniform layer coefficient for all quarry run crushed stone aggregate base material does not recognize any potential strength performance variations due to the actual material gradation within the acceptance bands or to variation in performance due to the mineral composition of the aggregate.

Table 2.3: AHTD established layer coefficients for pavement design (AHTD, 1998).

<table>
<thead>
<tr>
<th>Pavement Section Course/Type</th>
<th>Layer Coefficient, ( a_i )</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACHM Surface Course: 9.5 mm (3/8 in.), 12.5 mm (1/2 in.)</td>
<td>0.44</td>
</tr>
<tr>
<td>ACHM Binder Course: 25.0 mm (1 in.)</td>
<td>0.44</td>
</tr>
<tr>
<td>ACHM Base Course: 37.5 mm (1-1/2 in.)</td>
<td>0.36</td>
</tr>
<tr>
<td>P.C. Stabilized Base Course (Soil Cement)</td>
<td>0.20</td>
</tr>
<tr>
<td>Aggregate Base Course (Class 7)</td>
<td>0.14</td>
</tr>
<tr>
<td>Aggregate Base Course (Class 5)</td>
<td>0.11</td>
</tr>
<tr>
<td>Lime Treated Subgrade</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Various nomographs are available by which the design engineer can correlate material properties to structural layer coefficients. Figure 2.2 shows the relationships of the layer coefficient, \( a_2 \), California bearing ratio (CBR), R-value, Texas triaxial, and modulus. Referring to Table 2.3 and Fig. 2.2, one notes that the layer coefficient for AHTD Class 7 \( a_2 \) of 0.14 equates to a Texas triaxial classification of about 2.0 and a resilient modulus value of approximately 30,000 psi.
Figure 2.2: Nomograph layer coefficients, $a_2$, for untreated aggregate base course materials (AASHTO, 1993).

2.5 Aggregate Testing

Numerous methods of determining the quality and suitability of crushed stone for aggregate base courses have been employed. As stated within Section 2.3, the AASTHO Guide does not include a set of specific criteria which crushed stone aggregate must possess in order to achieve different layer coefficients ($a_2$). This essentially means that the layer coefficient of 0.14 established by the AHTD for Class 7 base may devalue the strength of a high quality material while in some instances it may overvalue the strength for materials of marginal quality.
The AHTD determines the acceptability of crushed stone for base course aggregate largely upon the results of its index properties (Atterberg limits and gradation) testing. Other test methods that may be used for the strength evaluation of base course material include the Texas triaxial, resilient modulus \( (M_R) \), and triaxial tests. California Bearing Ratio (CBR) and R-value testing are generally reserved for testing of subgrade soils. These test methods along with their advantages and disadvantages are further discussed in the following sections of this report.

### 2.5.1 Index Tests

The term "index tests" commonly refers to a test or series of easily performed tests which give a value that can be related to an engineering property of the material such as strength, compressibility or permeability. The index tests required by the AHTD for base material acceptance are Atterberg limits (liquid and plastic limits and plasticity index) and grain size analysis. Completion of index property testing allows for the subject material to be designated according to the desired classification system, typically AASHTO or ASTM. The design engineer may then consult available correlations between a given material designation and the engineering performance properties and characteristics of the material.

Advantages to the use of index testing to evaluate base quality are the low cost ease with which Atterberg limits and gradation testing can be performed by transportation agencies and consulting engineers. The time to complete these tests is short when compared to other more rigorous test methods. It is opined that many states rely on index testing as a manner of affirming material quality during construction due to this time benefit.
The main disadvantages to the use of index testing for the evaluation of base material are that these tests do not provide a direct measure of any engineering property and do not allow for modeling (loading, environment, etc.) of the conditions under which the base material will be placed. While index testing allows for accurate classification of the material, it does not provide any performance based information, such as resilient modulus values, other than that available through generalized correlations.

2.5.2 California Bearing Ratio (CBR)

The California Bearing Ratio (CBR) test (AASHTO T 193) is an empirical test method that measures the penetration resistance of a standardized 49.63 mm (1.954 in.) diameter piston advanced at a uniform rate of 1.27 mm per minute (0.05 in. per minute) into the test specimen. The specimen is compacted into a 152.4 mm (6 in.) diameter mold to a final height of 116.8 mm (4.6 in.). Load readings are to be taken at each 0.64 mm (0.025 in.) of penetration from 0 to 5.1 mm (0 to 0.2 in.) and every 2.54 mm (0.1 in.) thereafter to a maximum penetration of 12.7 mm (0.5 in.). The penetration stress is calculated in megapascals (pounds per square in.) and is plotted versus penetration for each reading. If necessary, methods for correcting the load-penetration curve due to sample surface irregularities or other causes are presented within the AASHTO test method. The bearing ratio of the test specimen is calculated by taking the stress value from the stress penetration curve for 2.54 mm (0.1 in.) and 5.08 mm (0.2 in.) penetrations and dividing the stresses by standard stresses of 6.9 MPa (1000 psi) and 10.3 MPa (1500 psi), respectively then multiplying by 100.
Surcharge weights are applied to the test specimens in order to replicate the field overburden conditions applied by the pavement to the base and/or subgrade courses. Inclusion of this surcharge load during soaking also provides for measurement of the material’s swell under the predicted in-service field conditions. The ability to modify the surcharge weights added to the test specimen to better represent those in-place conditions for the aggregate base course is beneficial. If the overburden weight is unknown during testing a surcharge weight of 4.54 kg (10 lbs) is typically selected. After the specimen is prepared in accordance with one of the following procedures, the mold and weights are immersed in water permitting free access of water to the top and bottom of the specimen and allowed to soak for 96 hours. A constant water level is maintained during this soaking period. The immersion period can be decreased for fine grained or granular soils that absorb moisture readily. Initial and final swell measurements are taken and the swell is calculated as a percentage of the initial height of the specimen.

Bearing ratios can be determined using one of two basic procedures. In the first method, the bearing ratio is determined on materials compacted to optimum moisture content. The optimum moisture content is determined in accordance with either AASHTO T 99 or T 180 Proctor test procedures. Three bearing ratio specimens are compacted using three different compactive efforts (usually 10, 25, and 56 blows per layer) to obtain unit weights both above and below the desired unit weight. After soaking (further described below), each of the three specimens is loaded in accordance with the specified procedure and load vs. penetration curves are developed. A graph
showing the measured CBR vs. the dry unit weight is generated and the design CBR for the desired maximum dry unit weight is selected.

The second test procedure allows for bearing ratios to be determined over a range of water contents. Specimens are prepared similar to the method of moisture-density relations Proctor testing where a range of differing moisture content specimens are compacted at the same compactive effort (number of blows per lift). Typically three moisture-density curves are generated, one for each of three different compactive efforts. Each of the moisture-density specimens in all curves is loaded and the resulting bearing ratio data points are plotted against moisture content. The design CBR value selected for reporting is the lowest CBR within the specified water content range (such as within 2 percent of the optimum moisture content, for example) having a dry unit weight between the specified minimum and the dry unit weight produced by compaction with the water content range. Figure 2.3, taken from AASTHO T 193, displays the moisture-density and dry density-corrected CBR plots developed in this test method for a silty clay soil.
Figure 2.3: Determining CBR for water content range and minimum dry unit weight.

The CBR test has many limitations. It is of critical importance to not vary from the rate of loading, size of piston, and size of the compacted sample during testing in order to reduce testing scatter. The granular nature of crushed stone adds to erratic test results. The maximum particle size permitted within the test specimen is 19.0 mm (3/4 in.) which conflicts with base course gradations where the maximum particle size is greater than 19.0 mm (3/4 in.). Particle size variation within the test specimen, especially within the zone directly beneath the piston, may cause variance in load deformation characteristics, even within samples of the same gradation.

It is known that a difference in CBR values exists between laboratory and field tests under saturated conditions. This difference is attributable to the laboratory specimen being completely confined laterally within a rigid cylindrical mold during
testing. Resistance to piston penetration under laboratory conditions is affected by the confinement provided by the mold that retards displacement along the failure plane and thus inflates the CBR value obtained. Also, granular particles undergo lateral precompression during compaction in the rigid mold, causing increased CBR values.

A modified CBR test for granular materials in which controlled lateral pressure is applied on the specimen walls has been proposed (Livneh, 1967). The modified test specimen is initially prepared, compacted, and saturated in accordance with AASHTO T 193. The specimen is then removed from the rigid mold following saturation and placed in a rubber-sleeved pressure chamber of a type similar to that used in the Texas triaxial shear test (discussed in Section 2.5.4 of this report) and subjected to lateral pressure. The standard 4.54 kg (10 lb) surcharge weight utilized in method T 193 is replaced with a fixed disk to prevent vertical expansion of the specimen and to minimize shear stresses induced on the specimen as a result of the applied lateral pressures.

Studies completed by Livneh and Greenstein (1978) show that CBR values for granular materials vary with changes in lateral confining pressures. Their work showed that the standard test method that utilizes a rigid mold, and thus a single confining pressure, does not adequately reflect confinement conditions of in-place aggregate base course in the field. It is also important to understand that performance of base course aggregate as a structural layer is not directly proportional to its CBR value i.e., base course aggregate having a CBR value of 100 is not necessarily twice as good as one having a value of 50, (Barksdale, 2000). Relation of CBR values to pavement structural layer coefficients, $a_i$, is accomplished through use of accepted nomographs such as those presented in Fig. 2.2.
2.5.3 \textit{R-Value}

The Hveem Stabilometer test (AASHTO T 190) was developed by the California Division of Highways. This test method is used to measure the potential strength of subgrade, subbase, and base course materials for use in road and airfield pavements. The test is not as widely accepted or utilized as the CBR test, and is generally used in the western portion of the United States; however, the AHTD still uses this test for pavement support design. Where used, the material's \textit{R}-value is generally correlated to other strength indices such as the CBR and triaxial tests. The AHTD (1998) directly correlates the \textit{R}-value to resilient modulus, \( M_R \), in their current roadway design guide. The \textit{R}-value is defined as the resistance of a soil to lateral deformation under vertical loading as determined by a stabilometer. Figure 2.4 shows a Hveem stabilometer.

![Figure 2.4: Photo of Hveem stabilometer.](image)
Four 1200-gram samples of soil or base material are prepared with an amount of water estimated to equal half to two-thirds of the water required to produce saturation. This amount of water is described in depth within Sections 4.3 and 4.4 of the AASHTO test method. When 75% or more of the test material passes the 19.0 mm (3/4 in.) sieve, only the portion of material passing that sieve should be used. Even if less than 75% of the sample passes the 25.0 mm (1 in.) sieve only the portion of material passing the 25.0 mm (1 in.) sieve should be utilized to mold the test specimen. A kneading compactor is used to fabricate sample specimens 101.6 mm (4 in.) in diameter by 63 mm (2.5 in.) in height.

Resistance testing begins by placing the specimen into the Hveem stabilometer. A horizontal pressure of 34 kPa (5 psi) is applied to the specimen by means of the displacement pump. Vertical load is applied at a uniform rate of movement of 1.3 mm/minute (0.05 inches/minute). The horizontal pressure is recorded when the vertical load of 8900 N (2000 lbf) is reached and then the vertical load is decreased to 4450 N (1000 lbf). The horizontal pressure is lowered to 27 kPa (4 psi) and then brought back to 35 kPa (5 psi) and the vertical pressure is increased to 8900 N (2000 lbf). This process conditions the sample. The stabilometer pump handle is turned at a rate of approximately two turns per second and the number of turns required (measured using the turns-displacement dial indicator) to raise the horizontal pressure from 34 to 690 kPa (5 to 100 psi) is recorded. The resistance, $R$, value is calculated in Eqn. 2.3:

$$R = 100 - \frac{100}{\left(\frac{2.5}{D}\left[\left(\frac{P_v}{P_k}\right) - 1\right]\right) + 1}$$
where $R =$ resistance value

$P_v =$ applied vertical pressure of 1103 kPa (160 psi)

$P_h =$ transmitted horizontal pressure at $P_v = 1103$ kPa (160 psi)

$D =$ number of turns, displacement dial indicator reading necessary to increase horizontal pressure from 34 to 689 kPa (5 to 100 psi)

The $R$-value calculated using Eqn. 2.3 is for specimens with compacted heights from 62 to 65 mm (2.45 to 2.55 in.). Specimens with dimensions slightly outside of this range are corrected using the correction chart presented in the AASHTO test method. $R$-values vary from 0 for water to approximately 100 for stiff, nearly rigid material. Densely graded, high density crushed stone base course typically has $R$-values in the 80’s and 90’s while less well graded and higher fines content gravelly materials produce $R$-values in the 50’s and 60’s (Barksdale, 2000).

The manner in which a compacted specimen is pushed from the Hveem mold into the stabilometer can cause sample irregularities, especially for granular materials such as crushed stone aggregate base courses. The actual $R$-value of a base course can be unintentionally degraded due to this disturbance. The acceptance limits for AHTD Class 7 aggregate base gradation allow for up to 50 percent of material to be retained on the 19.0 mm (3/4 in.) sieve. The final specimen height of 63 mm (2.5 in.) is too short for material of this nominal diameter. The base course of a pavement section rarely is less than 4 in. for rigid pavement and 6 to 8 in. for flexible pavement. This means that larger
size particles that may be present within the in-place base course are not accounted for in the Hveem mold. The AHTD does not use R-value testing for aggregate bases.

The focus of this investigation is on the effect of fines on base course strength. The small specimens required of this evaluative method limit appropriate blending, mixing, and compaction. Additionally, one would expect to see R-values for crushed stone aggregate base at the upper end of the spectrum. With all testing results, regardless of sample model blend, confined to one end of the R-value spectrum a determination of the effect of fines on base course strength performance could not be adequately determined. As a result, the use of this test to evaluate comparative strength will not be considered for this study.

2.5.4 Texas Triaxial

The Texas triaxial test (formerly AASTHO T 212) is a static test developed by the Texas Department of Transportation to categorize the quality of soils and soil-aggregate combinations into one of six classifications based upon the location of the Mohr-Coulomb failure envelope.

A total of seven cylindrical specimens 152.4 mm (6 in.) in diameter and 203.2 mm (8 in.) in height are molded at optimum moisture content and maximum dry density. Care should be taken to mold the specimens as close to identical as practicable. The specimens are enclosed in lightweight stainless steel cylinders with tubular rubber membranes and top and bottom porous stones in place. The specimen set is placed in an oven air dryer to remove between 1/3 to 1/2 of the molding moisture content at a temperature of 60°C (140°F). The time estimated to remove the moisture should range
from 3 to 6 hours. With a constant lateral pressure of 6.9 kPa (1 psi), the specimens are subjected to capillary absorption for 10 days as shown in Fig. 2.5.

Six of the seven specimens are loaded in axial compression, each at a different confining pressure. The typical lateral pressures used for a series of tests are 0 kPa (0 psi), 20.7 kPa (3 psi), 34.5 kPa (5 psi), 69.0 kPa (10 psi), 103.5 kPa (15 psi), and 138.0 kPa (20 psi). The triaxial specimen is loaded at a rate of 2% strain per minute with (load) readings taken at every 0.5 mm (0.02 in.) of deformation to a maximum of 15.2 mm (0.60 in.) or until specimen failure has occurred. The lateral pressure, $\sigma_3$, is applied only to the sides of the specimen. As a result, the major principal stress, $\sigma_1$, is simply the axial stress applied during loading because confining pressure is not applied to the specimen ends. The principle stresses at failure are calculated and Mohr’s stress circles are plotted for the combined test results of the six specimens. The Mohr-Coulomb failure envelope is developed. The failure envelope is transferred onto the Texas triaxial subgrade and flexible base material classification chart (Fig. 2.6) and the tested material is classified to the nearest 0.1 of a class. The higher the class the lower shear strength the material possesses. Stone base has a class rating of 2.6 or lower while clay is approximately 5.5. The Texas triaxial class values may be correlated to other design properties such as resilient modulus and pavement structural layer coefficient, $a_2$, using nomographs similar to those shown in Fig. 2.2.

Yoder and Witzcak (1975) point out that a disadvantage of the procedure is the amount of friction existing between the rubber membrane and the chamber wall. This friction could lead to inflated axial stress values recorded at failure. The Texas Triaxial test was replaced in the 1986 AASHTO Design Guide with the repeated load triaxial
“resilient modulus” test. TxDOT still utilizes the Texas triaxial test (Tex-117-E) to determine the shearing resistance of base and subgrade materials. However, the Texas triaxial test is a static test that does not offer the same benefits of modeling in-service conditions as the dynamic load and response of the resilient modulus test.

2.5.5 Resilient Modulus

The AASTHO Design Guide (1993) establishes resilient modulus as the standard, or recommended, measure of a material’s strength property for use in the evaluation and design of pavement sections. Resilient modulus is a measure of a material’s elastic behavior under the repeated application of dynamic load to a test specimen. The 1986 AASHTO guide recognized resilient modulus testing over 30 years after the concept was first introduced by Seed et al. (1959) and thus replaced the soil support value called for in previous AASHTO guide editions.

The soil support value was a performance parameter that was not clearly defined. The soil support, S, value for subgrade at the AASHO road test was taken as 3 while an S value of 100 was established for the crushed limestone aggregate base coarse present in the AASHO road test pavement section (Elliott, 2004). Development of subgrade and base support values was deferred to individual state transportation agencies by the AASHTO guide authors. No specific laboratory test directly provides S values; rather, the support parameter is obtained through correlations to tests such as California Bearing Ratio (CBR), Texas triaxial, R-value, etc. The vagueness and subjective nature of S values made design standardization difficult. Additionally, the correlations between established tests and S values were frequently based on little historical or laboratory testing (Elliott, 2004).
Figure 2.5: Texas Triaxial specimen (Barksdale, 2000).

Figure 2.6: Texas triaxial classification chart (Tex-117-E).
The resilient modulus concept is that of quantifying the “rebound” response of a particular material to repeated cycles of loading and unloading in a regimen that models the wheel loading of passing traffic. The material’s response to this impulse loading is more representative of actual field-performance conditions and provides a better characterization of soil and aggregate behavior than static testing. Resilient modulus testing in accordance with (AASHTO T 307) measures the elastic properties of unbound cohesive soils or unbound aggregate base materials under specified levels of repeated stress and requires the use of highly specialized and expensive testing equipment. AHTD Class 7 crushed stone aggregate base is categorized as Type 2 Material within AASHTO T 307.

Testing for resilient modulus is conducted on a triaxial specimen loaded in axial compression. Cyclic loads are applied to the specimen through use of any device able to control a variable load of fixed cycle length (typically 0.1 second of load followed by 0.9 seconds of relaxation for hydraulic loading devices and between 0.9 and 3 seconds for pneumatic loading devices). The load functions generally take the form of a haversine. The load configuration for resilient modulus testing done at the University of Arkansas includes a modified haversine in which the load is ramped to the target value over 0.02 seconds, is held constant for 0.06 seconds, and then ramped down over 0.02 seconds to a residual value (Dennis, 2004). The 0.06 second load “plateau” helps reduce “overshooting” of the target value during load application due to ringing in the load cell control or improper dither control in the hydraulic actuator.

Resilient modulus samples are tested in an unsaturated condition. Determining the load and response information would be inconclusive for saturated specimens since
the frequency of load applications would not allow for either the measurement or dissipation of excess pore pressures. As a result the determination of the actual effective stresses on the specimen during at a specific stage of loading would not be possible.

The first part of repeated load triaxial testing is to precondition the test specimens with the application of low-level stress repetitions in order to remove specimen end deformities. Once conditioned, the triaxial specimen is subjected to a repeated cyclic axial stress. The resilient deformation equals the total deformation at the highest stress minus the deformation at the lowest stress. The test specimen is first subjected to surrounding confining pressure (designated as $\sigma_3$) and a repeated deviator stress along the specimen’s longitudinal axis (designated as $\sigma_d$ where $\sigma_d = \sigma_1 - \sigma_3$). Table 2.4 is a recreation of the specified testing sequences for base/subbase materials in AASHTO T 307 including the confining pressure, maximum axial stress, cyclic stress, constant stress, and number of load applications. The amount of recoverable axial strain $\varepsilon_a$ is determined during the loading-unloading cycle by measuring the recoverable deformation (rebound) across the sample length. Figure 2.7 further illustrates the concept of resilient strain under a cyclic stress. The data gathered is then used to calculate the specimen’s resilient modulus $M_R$ as shown in Eqn. 2.4.

$$M_R = \frac{\sigma_d}{\varepsilon_r}$$  \hspace{1cm} (2.4)

where $\sigma_d = \text{cyclic deviator stress}$

$\varepsilon_r = \text{resilient strain.}$
Table 2.4: Testing sequences for base/subbase materials (from AASHTO T 307).

<table>
<thead>
<tr>
<th>Sequence No.</th>
<th>Confining Pressure, $\sigma_3$</th>
<th>Max. Axial Stress, $\sigma_{max}$</th>
<th>Cyclic Stress, $\sigma_{cyclic}$</th>
<th>Constant Stress, $0.1\sigma_{max}$</th>
<th>No. of Load Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>103.4 kPa 15 psi</td>
<td>103.4 kPa 15 psi</td>
<td>93.1 kPa 13.5 psi</td>
<td>10.3 kPa 1.5 psi</td>
<td>500 - 1000</td>
</tr>
<tr>
<td>1</td>
<td>20.7 kPa 3 psi</td>
<td>20.7 kPa 3 psi</td>
<td>18.6 kPa 2.7 psi</td>
<td>2.1 kPa 0.3 psi</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>20.7 kPa 3 psi</td>
<td>41.4 kPa 6 psi</td>
<td>37.3 kPa 5.4 psi</td>
<td>4.1 kPa 0.6 psi</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>20.7 kPa 3 psi</td>
<td>62.1 kPa 9 psi</td>
<td>55.9 kPa 8.1 psi</td>
<td>6.2 kPa 0.9 psi</td>
<td>100</td>
</tr>
<tr>
<td>4</td>
<td>34.5 kPa 5 psi</td>
<td>34.5 kPa 5 psi</td>
<td>31.0 kPa 4.5 psi</td>
<td>3.5 kPa 0.5 psi</td>
<td>100</td>
</tr>
<tr>
<td>5</td>
<td>34.5 kPa 5 psi</td>
<td>68.9 kPa 10 psi</td>
<td>62.0 kPa 9.0 psi</td>
<td>6.9 kPa 1.0 psi</td>
<td>100</td>
</tr>
<tr>
<td>6</td>
<td>34.5 kPa 5 psi</td>
<td>103.4 kPa 15 psi</td>
<td>93.1 kPa 13.5 psi</td>
<td>10.3 kPa 1.5 psi</td>
<td>100</td>
</tr>
<tr>
<td>7</td>
<td>68.9 kPa 10 psi</td>
<td>68.9 kPa 10 psi</td>
<td>62.0 kPa 9.0 psi</td>
<td>6.9 kPa 1.0 psi</td>
<td>100</td>
</tr>
<tr>
<td>8</td>
<td>68.9 kPa 10 psi</td>
<td>137.9 kPa 20 psi</td>
<td>124.1 kPa 18.0 psi</td>
<td>13.8 kPa 2.0 psi</td>
<td>100</td>
</tr>
<tr>
<td>9</td>
<td>68.9 kPa 10 psi</td>
<td>206.8 kPa 30 psi</td>
<td>186.1 kPa 27.0 psi</td>
<td>20.7 kPa 3.0 psi</td>
<td>100</td>
</tr>
<tr>
<td>10</td>
<td>103.4 kPa 15 psi</td>
<td>68.9 kPa 10 psi</td>
<td>62.0 kPa 9.0 psi</td>
<td>6.9 kPa 1.0 psi</td>
<td>100</td>
</tr>
<tr>
<td>11</td>
<td>103.4 kPa 15 psi</td>
<td>103.4 kPa 15 psi</td>
<td>93.1 kPa 13.5 psi</td>
<td>10.3 kPa 1.5 psi</td>
<td>100</td>
</tr>
<tr>
<td>12</td>
<td>103.4 kPa 15 psi</td>
<td>206.8 kPa 30 psi</td>
<td>186.1 kPa 27.0 psi</td>
<td>20.7 kPa 3.0 psi</td>
<td>100</td>
</tr>
<tr>
<td>13</td>
<td>137.9 kPa 20 psi</td>
<td>103.4 kPa 15 psi</td>
<td>93.1 kPa 13.5 psi</td>
<td>10.3 kPa 1.5 psi</td>
<td>100</td>
</tr>
<tr>
<td>14</td>
<td>137.9 kPa 20 psi</td>
<td>137.9 kPa 20 psi</td>
<td>124.1 kPa 18.0 psi</td>
<td>13.8 kPa 2.0 psi</td>
<td>100</td>
</tr>
<tr>
<td>15</td>
<td>137.9 kPa 20 psi</td>
<td>275.8 kPa 40 psi</td>
<td>248.2 kPa 36.0 psi</td>
<td>27.6 kPa 4.0 psi</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 2.7: Concept of resilient modulus testing (taken from Foo, 1987).

Simply put, the greatest advantage resilient modulus testing has over other forms of strength/performance testing is that the repeated-load dynamic test method allows for the most accurate modeling of in-service conditions likely to be experienced by a pavement section subjected to traffic loading. The method also provides the design engineer with values of resilient modulus under a wide range of stress conditions. The
benefits of resilient modulus testing, summarized by Claros et al. (1990) are that the resilient modulus is a basic material property that can be used in analysis and design for predicting pavement distresses such as rutting, faulting, cracking, and roughness. The method is widely recognized for pavement materials' characterization, and the resilient moduli of in-place materials can be correlated to results from non-destructive test (NDT) methods.

Many of the drawbacks of resilient modulus testing relate to the complex and expensive testing equipment necessary and the requirement for well-trained technicians to conduct and interpret the results of the test. The recommended minimum specimen diameter size of at least five times the nominal aggregate size and length to diameter ratio of 2:1 requires the use of large triaxial chambers and testing apparatus for testing of crushed stone aggregate base materials. Testing variability is also introduced by the method used to measure axial deformation of the test specimen. According to Claros, et. al., the distortion of data due to poor contact between the loading piston and the test specimen as well as placement of linear variable differential transformers (LVDTs) introduces variability.

Thompson and Smith (1990) characterized granular bases using repeated load triaxial testing. In testing of seven crushed stones, crushed gravels, and gravels meeting Illinois DOT gradation requirements, only limited differences in resilient modulus values between the different materials were observed. Gradation, Proctor values, and plasticity indices for the tested materials are summarized in Table 2.5. This would seem to indicate that gradation and plasticity are effective in predicting resilient modulus. Results of the resilient modulus testing completed on these materials are shown in Table 2.6.
Thompson and Smith concluded that because 6 of the 7 materials tested had resilient moduli ranging from 28.6 to 35.4 ksi (at the representative bulk stress of 20 psi) resilient modulus testing is not a good property for ranking granular base performance potential.

Table 2.5: Thompson and Smith (1990) gradation, compaction, and PI data.

<table>
<thead>
<tr>
<th>Percent Passing</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve</td>
<td>#1</td>
</tr>
<tr>
<td>Gradation</td>
<td>CA-6</td>
</tr>
<tr>
<td>1 in.</td>
<td>100</td>
</tr>
<tr>
<td>3/4 in.</td>
<td>97.5</td>
</tr>
<tr>
<td>1/2 in.</td>
<td>90.2</td>
</tr>
<tr>
<td>#4</td>
<td>53.1</td>
</tr>
<tr>
<td>#16</td>
<td>25.4</td>
</tr>
<tr>
<td>#200</td>
<td>10.5</td>
</tr>
</tbody>
</table>

AASHTO T 99  
Max. Dry Density, pcf 143.6 | 122.5 | 134.1 | 128.4 | 134.4 | 135.0 | 133.4 |
Optimum Moisture, % 6.6 | 4.0 | 7.7 | 6.9 | 7.6 | 8.0 | 9.0 |

AASHTO T 90  
Plasticity Index 4 | NP | NP | NP | 4 | NP | 3 |

Table 2.6: Thompson and Smith (1990) resilient moduli data.

<table>
<thead>
<tr>
<th>Material</th>
<th>Cohesion (psi)</th>
<th>Friction Angle (degrees)</th>
<th>K* (psi)</th>
<th>n</th>
<th>R**</th>
<th>MR*** (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>24.4</td>
<td>45.9</td>
<td>15310</td>
<td>0.28</td>
<td>0.93</td>
<td>35.4</td>
</tr>
<tr>
<td>#2</td>
<td>17.7</td>
<td>44.4</td>
<td>11590</td>
<td>0.33</td>
<td>0.93</td>
<td>31.1</td>
</tr>
<tr>
<td>#5</td>
<td>13.4</td>
<td>45.8</td>
<td>10270</td>
<td>0.35</td>
<td>0.91</td>
<td>29.3</td>
</tr>
<tr>
<td>#6</td>
<td>15.1</td>
<td>46.4</td>
<td>10240</td>
<td>0.35</td>
<td>1.00</td>
<td>29.2</td>
</tr>
<tr>
<td>#7</td>
<td>11.9</td>
<td>43.8</td>
<td>12230</td>
<td>0.31</td>
<td>0.96</td>
<td>31.0</td>
</tr>
<tr>
<td>#8</td>
<td>9.6</td>
<td>42.7</td>
<td>9440</td>
<td>0.37</td>
<td>0.97</td>
<td>28.6</td>
</tr>
<tr>
<td>#8A</td>
<td>11.1</td>
<td>43.5</td>
<td>9720</td>
<td>0.23</td>
<td>0.61</td>
<td>19.4</td>
</tr>
</tbody>
</table>

* Antilog of “a” in Log E_R = a + n log θ  
** Correlation coefficient in above E_R - θ relation  
*** Resilient modulus for θ = 20 psi

In the same study, Thompson and Smith noted that several of the granular materials did not “survive” the established preconditioning regimen of 5000 stress repetitions at a 310 kPa (45 psi) deviator stress under 103 kPa (15 psi) confining pressure.
It is noted, however, that this conditioning regimen was not in accordance with AASHTO T 294 which was in place at the time of this work. The deviator stress during sample conditioning was reduced to 207 kPa (30 psi) in order to avoid failure prior to resilient testing. Thompson and Smith concluded that the failure of specimens during preconditioning indicates that resilient modulus testing of granular materials does not provide a definitive qualification of the material's performance. Despite their conclusion, resilient modulus testing has been established as the "gold standard" for evaluating the load-and-response performance characteristics of pavement, aggregate base, and subgrade materials. Although resilient modulus testing presently provides the most definitive performance data for pavement section components, the high cost and relative complexity of said testing handicaps widespread use.

2.5.6 Triaxial Testing

The conventional triaxial test is one of the most widely utilized tests for evaluation of soil and aggregate shear strength in both research and professional practice settings. The material's shear strength is determined by application of the Mohr-Coulomb failure criteria. The conventional test method (ASTM D 2850) can be conducted on either "undisturbed" samples obtained in-situ or on samples remolded in the laboratory. The type of triaxial test most commonly used in research work and routine analysis and design is the axial compression test performed on cylindrical test specimens.

An understanding of total and effective stress concepts is necessary when conducting triaxial testing. Soil and base materials are multiphase systems consisting of solid particles and void spaces that may be partially or totally filled with water. A load
that is applied to a saturated soil/base specimen will be carried by both the material’s solid particles and water. The portion of the load carried by the water acts equally in all directions. The remaining portion of the total applied load is carried by the soil skeleton consisting of the soil or aggregate solid particles. This concept is represented by Eqn. 2.5:

\[ \sigma = \sigma' + u \]  

where \( \sigma \) = total stress  
\( \sigma' \) = effective stress  
\( u \) = pore water pressure.

Certain field conditions encountered are more suited for analysis by total stress concepts, while others are best evaluated using effective stress parameters. Granular or relatively free draining materials where water will likely drain rapidly should be evaluated using effective stress concepts because any pore pressures developed will be quickly dissipated thus the applied stress will be carried by the soil/aggregate solids at their points of contact. Soils with low hydraulic conductivities, such as clays, should be evaluated using total stress concepts because both the soil solids and pore water present will carry induced stresses since the rate of pore water dissipation will be very low. However, in some instances the previous statement is not always true. Sometimes a drained analysis is more conservative if the in-situ soil is partially saturated and drainage would relieve negative pore pressures, causing swelling and subsequent loss of strength.

It is important during evaluation and design to know whether drained or undrained conditions will be expected in the field. For example, a foundation bearing in clay should be designed using total stress conditions since the low permeability of the
clay causes pore water to dissipate very slowly. The applied load will be supported in combination by the clay soil skeleton and pore water if the soil is in a saturated condition. Conversely, foundations designed to bear in sand where rapid drainage of pore water is expected should utilize effective stress conditions. When foundation loads are applied to the sand, drainage is immediate meaning pore pressures do not build up and the loads will be supported by the sand skeleton. The last condition is for partially saturated soils where both analyses must be performed over the range of stresses expected in the field.

Conventional triaxial testing can be separated into three phases: saturation, consolidation, and loading. If pore water pressures are to be monitored in the specimen it must first be saturated. Prior to discussion of the particular types of triaxial tests, degree of saturation and loading rate concepts will be presented. Determining the level of saturation for specimens subjected to triaxial testing is of critical importance. Skempton proposed the concept of pore-pressure coefficients in 1954.

An isotropic saturated soil element subjected to an isotropic stress increase of magnitude $\Delta \sigma$ will cause a pore water pressure increase of $\Delta u$ if drainage from the soil is not permitted. Equation 2.6 defines the increase in effective stress for a isotropic partially saturated soil:

$$\Delta \sigma' = \Delta \sigma_3 - \Delta u_a$$

where: $\Delta \sigma'$ = effective stress increase in all directions of the element

$\Delta \sigma_3$ = increase in confining stress

$\Delta u_a$ = increase in pore water pressure.
The volume change for a partially saturated specimen subjected to an isotropic loading condition is calculated by Eqn. 2.7:

\[ \Delta V_c = -C_c V(\Delta \sigma_3 - \Delta u_a) \]  

(2.7)

where: \( \Delta V_c = \) soil structure volume change  
\( C_c = \) compressibility of the soil structure  
\( V = \) original volume of the soil sample  
\( \Delta \sigma_3 = \) increase in confining stress  
\( \Delta u_a = \) increase in pore water pressure.

The volume change in the soil structure void space same loading condition is defined by Eqn. 2.8:

\[ \Delta V_v = - C_v n V(\Delta u_a) \]  

(2.8)

where: \( C_v = \) compressibility of the fluid (air and water) in the voids  
\( n = \) the soil porosity  
\( V = \) original volume of the soil sample  
\( \Delta u_a = \) increase in pore water pressure.

Since the change in volume of the void space, \( \Delta V_v \), is equal to the change in the volume of the soil structure, \( \Delta V_c \), Eqns. 2.7 and 2.8 are set to equal each other leaving the following:

\[ \frac{\Delta u_a}{\Delta \sigma_3} = B = \frac{1}{1 + \frac{nC_v}{C_c}} \]  

(2.9)

where: \( B = \) the pore pressure parameter (Skempton, 1954).

For saturated soils the ratio of \( C_v/C_c \) is approximately 0 because the compressibility of water is negligible compared to that of the soil structure. This means that the \( B \) parameter will equal 1 when the degree of saturation also equals 1. The same
$C_v/C_e$ ratio for dry soils will approach infinity since the compressibility of air is much greater than the soil structure’s compressibility. Dry soils will have a B parameter of 0 when the degree of saturation is also 0. Consequently, the B parameter for partially saturated soils will range between 0 and 1.

For saturated soft soils like clays, the B parameter should equal 1 to insure complete saturation. Skempton stated that water is infinitely more incompressible than the soil solids matrix for most soils. However, the degree to which water is more incompressible than the soil structure is seen to be somewhat decreased for more granular materials. As a result, the internal stress response of a dense granular specimen subjected to undrained loading from an all around external stress will not be that predicted by Skempton’s equations. Some of the applied stress will be carried immediately by the soil skeleton and the resulting B-Value will be somewhat less than 1.0 for very dense soils that are 100 percent saturated.

This concept is reinforced by Skempton’s (1954) own findings. A plot of B value vs. degree of saturation (Fig. 2.8) in his paper for a clay gravel soil shows that at approximately 95 percent saturation the B values obtained are approximately 85 percent. It is interesting to note that the clay gravel material tested has a maximum dry density of 136 lbs/ft$^3$ and an optimum water content of 7.8 percent. The liquid limit and plasticity index were 17 and 2, respectively. These values are very similar to those obtained for the crushed stone base course aggregate tested for this report. The B-values obtained in the testing during this study closely matched the clay gravel values presented by Skempton. Once triaxial test specimens are saturated they may be consolidated to a range of effective confining pressures that are expected in the field. This allows the Mohr-
Coulomb failure envelope to be created for this range of confining pressures. Since pore pressures are not measured during the consolidation phase, the volume change, or consolidation of the specimen must be completed under a given confining pressure to assume that all pore pressures have been dissipated and the effective and total confining pressures are the same. The consolidation phase, which may not be applied in all cases, creates a change in volume of a saturated test specimen as a result of the migration of pore fluid under a specified confining pressure. When a saturated test specimen is subjected to an increase in chamber pressure in a triaxial cell, the pore water pressure within the specimen is likewise suddenly increased by the same amount. If the test specimen is allowed to drain the increased pore water pressure is dissipated when the water in the pore spaces migrates to areas of lower pressure. Specimen drainage, and hence consolidation rates, are a function of the size of the pore spaces. In granular materials drainage is rapid and consolidation is completed quickly. The dissipation of the increased pore water pressure takes considerably longer for less permeable materials such as clays. For a saturated test specimen subjected to a confining or consolidating pressure the change in effective stress can be determined using Eqn. 2.10:

\[ \Delta \sigma' = \Delta \sigma - \Delta u \]  \hspace{1cm} (2.10)

where \( \Delta \sigma' \) = change in the effective stress of the test specimen

\( \Delta \sigma \) = change in total stress applied to the test specimen

(chamber pressure)

\( \Delta u \) = change in the specimen pore water pressure.
One may think of the effective stress as that stress carried by the soil skeleton and the pore pressure as the stress carried by the pore fluid in the soil mass. Immediately after increasing the chamber pressure at $t = 0$, the entire change in total stress $\Delta \sigma$, will be carried by the pore fluid in a saturated test specimen. At this point in time none of the increase in total stress will be carried by the soil skeleton. At any time greater than zero and less than infinity, the pore water pressure will decrease from its initial condition as the water in the void spaces is effectively “squeezed out” and allowed to drain from the specimen. During this phase in which partial pore water pressures exist, a portion of the change in total stress, $\Delta \sigma$, is transferred to the soil skeleton and the effective stress, $\Delta \sigma'$ increases. At $t = \infty$ the excess pore water pressure, which was generated by the application of the external load, is completely dissipated by drainage of pore fluid from the specimen, and $\Delta u = 0$. At this point the total change in stress is carried by the soil...
skeleton and \( \Delta \sigma = \Delta \sigma' \). For practical purposes the pore water pressures are dissipated at degrees of consolidation less than 100% which result in consolidation times that do not approach infinity.

For triaxial tests consolidation is monitored by measuring volume change with time instead of changes in pore pressure with time. This volume change can be determined in terms of deformation or by recording changing water levels in a calibrated accumulator attached to the drain line of the triaxial test specimen. The time to achieve a specified percent of ultimate consolidation, for example 90%, can be determined in several ways. The square-root-of time method (Taylor, 1942, from Das, 1997) is discussed below. The whole discussion uses changes in sample height for illustration. The procedure could be applied equally well for changes in volume resulting from the migration of pore fluid from the specimen. Deformation is measured from dial readings at increasing load increments and is plotted vs. the square root of time (Fig. 2.9). A tangent line (labeled PQ) is drawn to the early portion of the curve. An additional line (labeled PR) is drawn such that point OR = (1.15)(OQ). The intersection of PR and the consolidation curve will give the square root of time for 90% consolidation. Time factors and their corresponding degrees of consolidation for a given set of drainage conditions can be obtained from published tables (such as Das, 1997). The time factor \( T_v \) for doubly drained specimens at a 90% average degree of consolidation \( U_{av} \) is 0.848. The coefficient of consolidation, \( C_v \), can then be determined by applying Eqn. 2.11:

\[
c_v = \frac{0.848H^2}{t_{90}}
\]  

(2.11)

where \( c_v = \) coefficient of consolidation

\( H = \) height of the specimen
\[ t_{90} = \text{time to reach 90\% consolidation.} \]

Figure 2.9: Square-root-of-time method for determination of \( c_v \) (Das, 1997).

For most testing, it is practical to assume that excess pore water pressure is completely dissipated when the degree of consolidation reaches 95%.

The loading phase of a triaxial test can either be drained (when drainage is permitted) or undrained (when drainage is not permitted). The total and effective stresses for specimens tested when drainage is permitted should always be equal, since the loading rate is selected such that excess pore water pressure is not allowed to develop. The loading rate is so slow that pore fluid can exit the specimen at the same rate as the volume change induced by the application of load. For undrained conditions, total stress strength parameters are normally measured, but if effective stress parameters are desired the change in pore water pressure during the loading phase must be measured and subtracted from the total stress to obtain the effective stress.

In order to conduct a test in which drainage can be controlled or pore pressures can be measured, a cylindrical specimen is capped with porous media on both specimen
ends and then encased in a rubber membrane. The specimen is placed in the triaxial chamber as shown in Fig. 2.10. A constant all-around confining pressure, denoted as $\sigma_3$, is applied to the test specimen by water (typically) or air that has been placed within the sealed triaxial chamber. The specimen is axially loaded to failure. The change in stress experienced by the specimen under axial loading is called the deviator stress, $\sigma_d$. The stress on the major principal axis is $\sigma_1$, which equals $\sigma_d + \sigma_3$. The specimen’s deformation during loading is measured. Axial loading is halted when the specimen has failed in shear as evidenced by increasing deformation without a change in $\sigma_d$, or at a predefined value of deformation.

![Figure 2.10: Conventional triaxial chamber (After Das, 1994).](image)

### 2.5.6.1 Types of Triaxial Tests

Triaxial compression tests are generally classified according to the consolidation and drainage conditions during each test. The five standard types of triaxial compression tests conducted and their designations are: 1) unconfined compression (UC test), 2)
unconsolidated-undrained test or undrained test (UU test), 3) consolidated-undrained (CU test), 4) consolidated-drained with pore pressure measurement (CU-bar test), and 5) consolidated-drained (CD test). If effective stresses are to be reported these tests must be conducted on saturated specimens.

The unconfined compression (UC) test is a special type of triaxial test in which no confining pressure is applied to the test specimen: \( \sigma_3 = 0 \). This test is completed easily without need for expensive testing apparatus. This test is limited to testing of cohesive soils due to the fact that cohesionless soils (such as sands and the subject crushed stone base) will fall apart without a confining pressure. The macrostructure of cohesive soils can cause erroneous results in UC testing due to planes of weakness in the soil, such as slickensides, that may open up during sampling and testing procedures cannot be closed without the application of a confining pressure. Only total stresses can be measured in the UC test. Since the undrained shear strength for this test is independent of the confining pressure, the cohesion value reported for this test is half of the unconfined compressive strength. Unconfined compression testing should not be used and its use has been discouraged by the geotechnical community for almost 40 years. No important work should be based upon UC testing (Dennis, 2004).

The simplest and quickest triaxial test which produces meaningful results is the unconsolidated-undrained (UU) test. This test is generally considered applicable for low permeability soils (clays) that are expected to be saturated under normal field conditions and where loading is rapid and drainage would be limited. Drainage is not permitted during application of the confining pressure, \( \sigma_3 \), thus consolidation of the specimen does not occur. However, the problem of developing weakened zones along slickensides is
eliminated by the applied confinement. Drainage is also not permitted during the loading phase of the test. With drainage restricted at all stages, UU testing can be completed rapidly.

UU testing is typically conducted on clay specimens. For saturated soils, the added axial stress at failure, \((\Delta u_d)_f\), is basically independent of the confining pressure, \(\sigma_3\), because without drainage the pore pressure equals the confining pressure and there is no increase in effective stress due to increases in confining pressure. The Mohr-Coulomb failure envelope becomes a horizontal line. This testing condition is called a \(\phi = 0\) condition, and the undrained shear strength of the soil, \(c_u\), is equal to the radius of the Mohr stress circles. This concept is illustrated in Fig. 2.11. The main disadvantage of the UU test is that it only provides total stress strength parameters.

![Figure 2.11: Unconsolidated-undrained triaxial test plot (Das, 1997).](image)

Perhaps the most common type of triaxial test is the consolidated-undrained (CU) test. This testing is mainly conducted on cohesive or fine grained soils. The test specimen is first saturated and then consolidated by an all-around confining pressure, \(\sigma_3\), to stress conditions expected in the field. The pore water pressure generated by the
confining pressure is allowed to completely dissipate. During the loading phase of the test drainage of the specimen is not permitted. A major benefit of the CU test is that specimens can be consolidated to replicate existing or anticipated field conditions. More expensive testing equipment and highly skilled laboratory personnel are required to conduct CU testing. Since drainage is not permitted, this too is a total stress test which is why this test is typically reserved for soils with expected low permeability like clays and silts. Without the need to dissipate pore water pressures during deviator stress application, CU tests can be preformed rather quickly.

Certain soils are considered contractive or dilative. The soil solids skeleton of contractive soils compress during load application causing excess pore water to be developed in the specimen (positive pore pressure). Dilative soils exhibit the opposite condition. During load application the soil solids skeleton will expand causing a negative pore water condition in the specimen. In both instances the effective stress is unknown and the effect of the change in pore pressure on the reported strength of the soil is unknown. Therefore, pore pressure measurements are necessary for contractive and dilative soils subjected to triaxial testing if reliable values of effective strength parameters are to be measured.

A variation of the CU test is the CU-bar test. Drainage is still not allowed in the CU-bar test but pore pressures are measured during testing. This means that Mohr’s stress circles for both effective and total stress conditions can be developed. Plotting the Mohr-Coulomb failure envelope for both the total and effective Mohr’s stress circles provides the ability to determine the stress condition under which total or effective stress strength parameters become unconservative. The CU-bar test provides effective stress
parameters for the fine grained and cohesive materials typically assessed with CU testing. Disadvantages of the CU-bar test are mainly related to the decreased loading rates which are required to allow pore water equilibration. For example, loading too quickly would not allow time for pore water to migrate from the center of the specimen where shear occurs to the exterior locations of the specimen where pore pressures are measured.

In the consolidated-drained (CD) test, an all-around confining pressure, $\sigma_3$, is applied to the specimen while keeping the sample drainage lines open. As with the CU test, the excess pore water pressure generated by application of the confining pressure is allowed to completely dissipate thus causing the sample to consolidate. The drainage lines are also left open during the application the deviator stress. No excess pore water pressures are allowed to develop in the specimen and loading rate must be slow enough to allow complete pore pressure dissipation.

Determination of the proper loading rate for specimens tested under drained conditions is important to insure that excess pore water pressures are adequately dissipated and not accidentally built up by a loading rate that is too fast. Gibson and Henkel (1954) developed a theory to estimate the proper shearing time. Since pore pressures generated by loading can never be totally dissipated the selection of an acceptable degree of consolidation at the time of shearing failure, $U_f$, must be made. Equation 2.12 relates the degree of consolidation at shear failure to specimen height, drainage conditions, time to shear failure, and coefficient of consolidation (Gibson and Henkel, 1954 from Olson):

$$U_f = 1 - \frac{H_d^2}{\eta C_v t_f}$$  \hspace{1cm} (2.12)
where $U_f =$ target degree of consolidation

$H_d =$ sample drainage height

$\eta =$ drainage condition parameter

$C_v =$ calculated coefficient of consolidation

$t_f =$ time to shear failure.

Table 2.7 summarizes the $\eta$ parameter values based upon drainage conditions. The first two values for $\eta$ listed in the table are applicable regardless of the specimen height-to-diameter ratio but the remaining values listed are specific for specimens with a height-to-diameter ratio of 2. The coefficient of consolidation, $c_v$, may be estimated using the relations shown in Table 2.8 based upon the drainage condition and the specimen height-to-diameter ratio.

### Table 2.7: Summary of $\eta$ parameter values (from Olson).

<table>
<thead>
<tr>
<th>Drainage condition</th>
<th>$\eta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage from one end only</td>
<td>0.75</td>
</tr>
<tr>
<td>Drainage from both ends</td>
<td>3.0</td>
</tr>
<tr>
<td>Radial drainage only</td>
<td>32.0</td>
</tr>
<tr>
<td>Drainage radially and out one end</td>
<td>35.8</td>
</tr>
<tr>
<td>Drainage radially and out both ends</td>
<td>40.4</td>
</tr>
</tbody>
</table>

### Table 2.8: Summary of $c_v$ Estimations (from Olson).

<table>
<thead>
<tr>
<th>Drainage Condition</th>
<th>$c_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Any $2H/D$</td>
<td>$\frac{\pi H^2}{t_{100}}$</td>
</tr>
<tr>
<td>Drainage from one end only</td>
<td></td>
</tr>
</tbody>
</table>
The estimated time to shearing failure, \( t_f \), can be calculated with the above information by rearranging Eqn. 2.13 and assuming that pore pressures are dissipated at degrees of consolidation above 95%:

\[
 t_f = \frac{H^2}{\eta C_v (1 - U_f)}. 
\]  

(2.13)

The strain rate applied during loading must be sufficiently slow so shearing failure will occur at a time greater than the time calculated in the above equation. If shearing failure occurs at a time less than that predicted by Eqn. 2.13, the target degree of consolidation is not attained and some amount of excess pore water pressure buildup will occur. As a result the specimen will not be tested in the totally drained condition and the effective stresses at failure cannot be determined. With no pore water pressures developed, the total and effective confining and axial stresses are equal.

In commercial practice, the CD type of triaxial test is typically run only on granular materials that drain quickly. Testing of a clay, for instance, would take a long time because the deviator stress would have to be applied at such a slow rate to guarantee full drainage from the specimen.
2.5.6.2 Mohr-Coulomb Failure Criteria

The Mohr-Coulomb failure envelope describes a material’s angle of internal friction, $\phi$, and cohesion, $c$, values. Values of $\phi$ and $c$ are input parameters for a wide variety of geotechnical applications such as bearing capacity and slope stability analysis and design. For triaxial testing, the values of $\sigma_1$ and $\sigma_3$ at failure are plotted on the $x$-axis of a standard $x$-$y$ plot. The distance between the two principal stresses ($\sigma_d$) forms the diameter of Mohr’s Circle of Stress. A sufficient number of triaxial tests are completed over a range of confining and deviator stresses to form several Mohr’s circles. A tangent line, defined as the Mohr-Coulomb failure envelope, is drawn in a fashion so that it is approximately tangent to each of the stress circles. The point at which the Mohr-Coulomb envelope crosses the shear stress ($y$) axis is defined as the cohesion intercept, $c$. Figure 2.12 shows the Mohr-Coulomb failure envelope. The material’s angle of internal friction, denoted as $\phi$, is the angle formed between the failure envelope line and the normal stress $x$-axis. Multiple triaxial tests are needed because an infinite number of tangent lines could be drawn if only a single Mohr’s stress circle were plotted. The exception to this would be the $\phi = 0$ line obtained from UU testing. In this case the tangent is taken as horizontal and the radius of the Mohr’s stress circle is equal to $c_u$. 

54
2.5.6.3 Triaxial Test Advantages and Limitations

Baldi et al. (1988) summarized the following advantages and limitations of the triaxial test on cylindrical specimens. Baldi states: "The main advantages of the conventional triaxial test are

(a) relative simplicity of drainage control and measurement of pore pressure;
(b) ability to apply principal stresses in known directions;
(c) ease of measuring axial and volumetric strains;
(d) use of solid cylindrical specimens which can be conveniently obtained from standard tube samples; and
(e) versatility of the equipment which may be used for a variety of determinations besides triaxial strength and stiffness (consolidation and permeability parameters, wave velocity, and so forth)."
2.6 Selection of Test Method

One of the objectives of this study was to evaluate the effect of fines on the shear strength of unbound crushed stone aggregate base course. Staged triaxial testing was selected as the evaluative method for this study. Excess pore water pressures could be dissipated rapidly during the application of confining pressures and deviator stress for the crushed stone during testing, thus the consolidated-drained (CD) triaxial test option was chosen.

When determining the testing method and regimen, consideration was given to using testing equipment readily available to transportation agencies and engineering consultants. While CBR equipment is cheap and readily available the California Bearing Ratio test was deemed not applicable to this study because the bearing ratios of all crushed stone aggregate having only varied fines contents would be confined to only the upper range of values. Resilient modulus testing to investigate the effect of fines on shear strength was not chosen because the equipment required to complete the testing is owned by very few transportation agencies and engineering consultants.

Triaxial testing was selected because of its modification capabilities and near universal acceptance for shear strength evaluation. The equipment necessary to complete triaxial testing is less expensive than that required for resilient modulus testing and, as such, conventional triaxial testing equipment is more widely owned by government and private engineering agencies. In addition, comparative values of strength over a range of fines contents were required for this study. The CD triaxial test offers a testing protocol which is easily repeatable and subject to perhaps the most limited set of testing variables.
of any of the testing procedures discussed in this section. As a result it was felt that the CD triaxial test is the most appropriate strength test for use in this study.

2.7 Hydraulic Conductivity and Drainage

Proper drainage of water from pavement structures has long been considered an important factor in road design. However, current empirical methods of design rely primarily on material strength and have often resulted in the selection of base courses that do not drain effectively. According to Cedegren (1994), poor drainage and increasing traffic loads have led to the early distress of many of our highways.

“Slow draining pavements have been deteriorating at an alarming rate, costing the traveling public unexpected billions of dollars for repair. It is currently believed by most designers that if they make the pavement strong enough there is no need for rapid drainage of the base course. It is this “undrainage” philosophy pervading the pavement design ranks that is responsible for the premature failure of thousands of miles of pavement”. (Cedegren, 1994)

The primary source of moisture in pavement structures is storm water which enters the pavement from above through cracks and joints saturating the base course. Additional sources of moisture are high groundwater tables, springs, and aquifers which allow water to enter the base course from below. Water from all of these sources can cause damage to the pavement or wearing course in many ways. According to the Federal Highway Administration (FHWA) Highway Subdrainage Design Manual (1980), some of the most
common ways are:

(1) The unbound aggregate base under rigid pavements can pump under traffic load and eject fines which causes a loss of support for the pavement at the joints, as illustrated in Fig. 2.13. This condition can lead to the creation of a void space under the joints which leads to cracking of the pavement.

(2) Elevated pore water pressures that result from moisture and traffic loading will reduce the strength and stiffness of the base course, as illustrated in Fig. 2.14. The reduction in strength of the base course will ultimately cause the surface course to fault and crack.

(3) Moisture can cause expansive subgrade soils under the base course to heave, resulting in distress of the overlying pavement structure. The subgrade soils can also lose bearing strength due to saturation and the accompanying loss of cohesion or frictional resistance.

(4) Freezing can lead to the formation of ice lenses which will promote damage of overlying pavements. During the active freezing period the growth of ice lenses can result in substantial heave of the pavement structure. However, the most potentially damaging effect of frost heave occurs during the spring when the ice lenses thaw. The thawing of the ice lenses can result in voids in the subbase and subgrade materials which can lead to collapse of the overlying pavement structure.

In recent years the number of published accounts of research dealing with the problems of fluid flow through porous media has grown steadily. Consequently, researchers now recognize and understand many of the problems created by excessive
subsurface moisture, and have the means available to provide for the satisfactory control of this moisture (Moulton, 1980).
Figure 2.13 – The effects of pumping on rigid pavements (source FHWA Subdrainage Design manual, 1980).

NOTE: Vertical dimensions of deformations are exaggerated for clarity.
Figure 2.14 – Flexible pavement distress due to elevated porewater pressure (source FHWA Subdrainage Design manual, 1980)
2.7.1 Historical Significance

The need for adequate drainage of subsurface water was recognized by early road builders as soon as formal road building began. More than 2000 years ago the Romans understood the way water clearly damages roads and their engineers designed ways to minimize its effects on a vast military road system (Cedegren, 1994).

By the middle of the 18th century it was understood that appropriate subsurface drainage was absolutely necessary for the satisfactory long term performance of roadways. According to Cedegren, the Scottish road engineer John L. MacAdam reported to the London Board of Agriculture that

"The erroneous opinion that a road may be made sufficiently strong, artificially, to carry carriages though the subsoil be in a wet state has produced most of the defects of the roads of Great Britain".

MacAdam recognized that strengthening a pavement without the creation of a good drainage system for the underlying base course would not work. The introduction of pavement systems designed by MacAdam demonstrated an understanding of the problems associated with poor drainage. His designs were the first documented attempts to incorporate the rapid removal of water from the pavement structure and subgrade into road construction (Cedegren, 1994).

For more than 20 years, Harry R. Cedegren was alone in his crusade to introduce a well-drained base course to pavement design. He had been an advocate for the use of rapidly drained pavements since his first submission to the American Society of Civil Engineers (ASCE) in 1941. His later works *Drainage and Airfield Pavements* (1974) and
Seepage, Drainage, and Flow Nets (1989) have been extensively quoted in the literature. He also wrote controversial articles on the subject, one of them published in CIVIL ENGINEERING (September, 1974), stated that road builders were wasting billions of dollars due to inadequate drainage provisions. He has completed numerous field studies documenting the effects of water on airfield and highway pavements.

In 1973, Cedegren investigated highways in all nine Federal Highway Administration regions to create that agency’s Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Sections. He also investigated airfield pavements for the U.S. Army Corps of Engineers’ Construction Engineering Research Laboratory. The report that resulted from this work, Methodology and Effectiveness of Drainage Systems for Airfield Pavements (1974), recommended good drains for all important airfield pavements.

Past research by Cedegren and others has suggested that the saturated condition which exists when the base course is not well drained leads to a build-up of pore water pressure. This increase in pore water pressure as a result of traffic loading decreases the effective stress on the soil. Under conditions of low or zero effective stress the individual soil granules tend to “float”, resulting in a loss of friction and particle to particle contact causing an overall strength loss. Adequate drainage of the base course will not allow these elevated pore water pressures to develop, resulting in a longer lasting roadway.

Much of Cedegren’s pioneering work on the drainability of base materials demonstrates that the hydraulic conductivity of the soil matrix decreases as the percent of fines increase (Cedegren, 1974). This principle was reiterated by Moulton in his 1980 publication, Highway Subdrainage Design, when he suggested that:
“It is particularly important to note that the amount of fines ($P_{200}$) exerts a marked influence on the hydraulic conductivity for a granular material and a small increase in the percentage of fines can cause a large decrease in the hydraulic conductivity.”

Similarly, the hydraulic conductivity of a granular material is affected to a lesser degree by the grain size distribution, particle shape, relative density, degree of saturation, and mineralogical composition of the soil (Richardson, 1997).

**2.8 Problems Associated with Decreased Hydraulic Conductivity**

Decreased hydraulic conductivity results in a pavement base course that drains slowly. The resulting distress to the pavement and decreased service life is well documented in the literature (Cedegren, 1974; Moulton, 1980;). From a hydraulic perspective, a complete pavement drainage system is typically composed of numerous components, such as the granular base course under the pavement, longitudinal gravel edge drains, and transverse outlet systems to discharge the water. The edge drains and outlet structures are actually sized to meet the anticipated flow regime. However, unless the granular base course has adequate hydraulic conductivity to transmit the infiltrating water freely, the drainage system will not function efficiently.

A cross sectional view of a typical pavement drainage system is illustrated in Fig. 2.15. A positive drainage system should move water from the point of entry into the base to the final exit through materials with sequentially lower resistance to flow.
Longitudinal pipe underdrain which collects water from the base course.

Figure 2.15- Cross sectional view of a typical pavement drainage system (source AHTD standard drawings).
In a pavement drainage system similar to that shown in Fig. 2.15, stormwater infiltrates the pavement structure and enters the granular base course where it flows laterally to the longitudinal pipe underdrains. If the system is “overloaded” due to poor hydraulic conductivity of the base course or insufficient capacity of the mechanical drainage system, the stormwater can also infiltrate the subbase. A saturated subbase will not only lose strength due to elevated pore water pressures it can also swell if plastic clays are present.

### 2.9 Measurement of Hydraulic Conductivity

Henri Darcy attempted to quantify the rate of flow of water through soils in the 19th century (Das, 1998). He identified permeability (hydraulic conductivity) as the property of a soil that quantifies its capacity to transmit a fluid under a specific gradient or head. Whether in-situ testing or laboratory testing was used, Darcy proposed the following equation for calculating the velocity of the flow of water through a soil:

\[ v = k \times i \]  

(2.14)

where:

- \( v \) = flow rate (L/T)
- \( k \) = proportionality constant (L/T)
- \( i \) = hydraulic gradient

The hydraulic gradient is defined as \( i=\Delta h/L \) as shown in Fig. 2.16.

According to Equation 2.14, the proportionality constant, \( k \), directly relates velocity to gradient. Darcy’s work, which was primarily with clean sands, showed that this relationship was indeed linear in the region of laminar flow and that this proportionality could be taken as a constant for a given material. Early literature refers to this proportionality constant as the coefficient of permeability when describing the flow of...
water through soil while later publications have used the term hydraulic conductivity. Unless otherwise specified, the term hydraulic conductivity will be used and $k$ will be reported in cm/sec.

![Figure 2.16 - Definition of hydraulic gradient for flow of water through soil used in Darcy's Law (After Das, 1998).](image)

Hydraulic conductivity is a difficult property to measure since no other engineering property for soils and rock has such a wide range of possible values. Hydraulic conductivity can range from 300 cm/sec (coarse macadam) to $1 \times 10^{-9}$ cm/sec (fat clays), for an overall variation of several billion times (Cedegren, 1974).

This wide range of values for hydraulic conductivity along with recommended testing procedures for various material types is illustrated in Table 2.9.
### Table 2.1 - Typical Coefficient of Permeability of Various Soils (source FHWA Subdrainage Design Manual, 1980)

<table>
<thead>
<tr>
<th>K (Cm/Sec)</th>
<th>K in ft/day</th>
<th>Drainage</th>
<th>Soil Type</th>
<th>Appropriate Test Methods</th>
</tr>
</thead>
<tbody>
<tr>
<td>$10^{-2}$</td>
<td>28,350</td>
<td>GOOD</td>
<td>Clean Gravel</td>
<td>Pumping Tests, Reliable if Properly Conducted</td>
</tr>
<tr>
<td>$10^{-1}$</td>
<td>2,835</td>
<td>GOOD</td>
<td>Clean Gravel</td>
<td>Constant Head Permeameter Reliable</td>
</tr>
<tr>
<td>10</td>
<td>283</td>
<td>GOOD</td>
<td>Clean Coarse Sand</td>
<td>Falling Head Permeameter</td>
</tr>
<tr>
<td>$10^{-1}$</td>
<td>28.3</td>
<td>GOOD</td>
<td>Clean Sand and Gravel Mixtures</td>
<td></td>
</tr>
<tr>
<td>$10^{-2}$</td>
<td>2.8</td>
<td>GOOD</td>
<td>Very Fine Sand; Organic &amp; Inorganic Silts; Mixtures of Sand, Silts &amp; Clay; Glacial Till; Stratified Clay Deposits</td>
<td></td>
</tr>
<tr>
<td>$10^{-3}$</td>
<td>0.28</td>
<td>FAIR</td>
<td>Impervious Soils Such as Homogeneous Clays Below Zone of Weathering</td>
<td></td>
</tr>
<tr>
<td>$10^{-4}$</td>
<td>0.028</td>
<td>FAIR</td>
<td>Impervious Soils Such as Homogeneous Clays Below Zone of Weathering</td>
<td></td>
</tr>
<tr>
<td>$10^{-5}$</td>
<td>0.0028</td>
<td>POOR</td>
<td>Impervious Soils Such as Homogeneous Clays Below Zone of Weathering</td>
<td></td>
</tr>
<tr>
<td>$10^{-6}$</td>
<td>0.00028</td>
<td>POOR</td>
<td>Impervious Soils Such as Homogeneous Clays Below Zone of Weathering</td>
<td></td>
</tr>
<tr>
<td>$10^{-7}$</td>
<td>0.000028</td>
<td>PRACTICALLY IMPERVIOUS</td>
<td>Impervious Soils Such as Homogeneous Clays Below Zone of Weathering</td>
<td></td>
</tr>
<tr>
<td>$10^{-8}$</td>
<td>0.0000028</td>
<td>PRACTICALLY IMPERVIOUS</td>
<td>Impervious Soils Such as Homogeneous Clays Below Zone of Weathering</td>
<td></td>
</tr>
<tr>
<td>$10^{-9}$</td>
<td>0.00000028</td>
<td>PRACTICALLY IMPERVIOUS</td>
<td>Impervious Soils Such as Homogeneous Clays Below Zone of Weathering</td>
<td></td>
</tr>
</tbody>
</table>

(approximately 1 foot per 100 years)
This wide range of values for hydraulic conductivity dictates that different test methods be used to accurately estimate the property. Consequently, the suitability of a test method for a given material type is primarily governed by the range of hydraulic conductivity expected for that material.

2.9.1 In-situ Tests

One of the oldest and most simplistic tests that can used to estimate hydraulic conductivity of a granular base course is the percolation test. The test is simple in concept; pour water in a hole and measure how fast it soaks into the surrounding soil. While the test is empirical and not solely dependent on hydraulic conductivity, it can be useful. During his studies, Cedegren often used the percolation test and the resultant percolation rate to emphasize the need for adequate drainage of pavement base courses. However, in addition to hydraulic conductivity, the percolation rate depends on soil capillarity at the time of the test, test hole radius, depth of the water in the test hole, and soil heterogeneities. As a result, the correlation between the percolation rate and hydraulic conductivity is poor (Elrick and Daniels, 1986).

The original percolation test was improved by Boutwell who developed a two stage borehole hydraulic conductivity test to estimate the in-situ hydraulic conductivities of compacted clays. The test is shown schematically in Fig. 2.17. The test is based on the concept that by varying the geometry of the zone of the wetted front, the relative effect of the vertical and horizontal hydraulic conductivities will also vary in a calculable manner. The device is installed by drilling a hole, placing a casing in the hole, and sealing the annular space between the casing and the borehole with grout. Falling head tests are
performed, and the hydraulic conductivity from Stage I \((k_1)\) is computed from the appropriate Hvorslev equation as follows (Daniel, 1989):

\[
k_1 = \frac{\pi d^2}{11D(t_1 - t_2)} \ln \left( \frac{H_1}{H_2} \right)
\]  

(2.15)

where:

\(d\) = inside diameter of the standpipe
\(D\) = inside diameter of the casing
\(H_1\) = head at time \(t_1\)
\(H_2\) = head at time \(t_2\)

Figure 2.17 – Schematic diagram of a two-stage in situ hydraulic conductivity test with the Boutwell permeameter (after Daniel, 1989).
The values of $k_1$ are plotted versus time until steady state conditions are reached (which typically takes from a few days to two weeks depending on the hydraulic conductivity of the clay), indicating the completion of Stage I.

Next, the hole is deepened by augering or pushing a sampling tube into the soil. Smeared soil is removed from the walls of the hole with a wire brush. The permeameter is reassembled and falling head tests are again performed. The hydraulic conductivity for Stage II is calculated from Horslev’s equation as follows:

$$k_2 = \frac{A}{B} \ln \left( \frac{H_1}{H_2} \right)$$  \hspace{1cm} (2.16)

where:

$$A = d^2 \ln \left[ \frac{L}{D} + \sqrt{1 + \left( \frac{L}{D} \right)^2} \right]$$ \hspace{1cm} (2.17)

$$B = 8D \frac{L}{D} \left( t_2 - t_1 \right) \left\{ 1 - 0.562 \exp \left[ -1.57 \left( \frac{L}{D} \right) \right] \right\}$$ \hspace{1cm} (2.18)

Stage II continues until $k_2$ ceases to change significantly. The anisotropy of the soil with respect to hydraulic conductivity is determined by referring to Figure 2.18 which is a plot of $\frac{k_1}{k_2}$ versus $m$, where $m = \sqrt[3]{\frac{k_h}{k_v}}$ for various values of $\frac{L}{D}$ shown in Figure 2.18.

Experimental values of $\frac{k_1}{k_2}$ and $\frac{L}{D}$ can be used to determine $m$ from the curves.
illustrated in Fig. 2.18. The curves in Fig. 2.18 are developed from the following relationship:

\[
\frac{k_2}{k_1} = \frac{\ln\left[\frac{L/D + \sqrt{1 + (L/D)^2}}{(mL/D) + \sqrt{1 + (mL/D)^2}}\right]^m}{\ln\left[\frac{L/D + \sqrt{1 + (L/D)^2}}{(mL/D) + \sqrt{1 + (mL/D)^2}}\right]}
\]  

(2.19)

Fig. 2.18 – Curves of \(k_2/k_1\) versus \(m\) required to satisfy Eq. 2.7 and Eq. 2.8 for \(L/D = 1.0, 1.5,\) and \(2.0\) (after Daniel, 1989).

Once \(m\) is determined \(k_h\) and \(k_v\) can be calculated using Equations 2.7 and 2.8:

\[
k_h = mk_1
\]  

(2.20)

\[
k_v = \frac{k_1}{m}
\]  

(2.21)
The Boutwell permeameter test takes into account that most natural soils are anisotropic with regard to hydraulic conductivity and that the degree of anisotropy depends upon the type of soil and nature of its deposition. In most cases, the anisotropy is more prevalent in clay type soils than granular soils (Das, 1997). While this test was primarily designed for clayey soils, it could, in theory, be run on granular base course as well.

In reality, the test would normally not be suitable for granular base unless the Stage II auger hole could be drilled with a constant radius. When used for granular base course the larger particles of the base course would create a “ragged” stage II hole introducing errors.

Ring infiltrometers, both single ring (SRI) and double ring (DRI), have been used to obtain in-situ estimates of hydraulic conductivity of a granular base course. The infiltration rate can be hypothetically related to the hydraulic conductivity of the saturated granular base course if the necessary boundary conditions of the test are met. Specifically, an assumption must be made about the flow path length and the point in the soil at which the pressure head in the flowing water returns to atmospheric conditions. Measurements of the infiltration rate are used to provide estimates of the hydraulic conductivity of the saturated material under these assumed conditions.

The main drawback of single ring infiltrometers is that there is a lateral component of flow as the water seeps into the soil which complicates the analysis of the flow problem. The lateral flow causes the volume of soil used in hydraulic conductivity calculations to increase by an unknown amount. This condition results in an apparent infiltration rate that is greater than the actual infiltration rate of the granular base course immediately
below the test ring.

In an attempt to solve this problem, DRI’s, as illustrated in Fig. 2.20, have an outer ring to reduce or eliminate the lateral water movement from the inner ring on which the measurements are made. When one dimensional flow is assumed, the DRI can provide a direct method of estimating hydraulic conductivity.

In a DRI test, two open-ended cylinders are installed into the granular base course being tested. The cylinders are placed in hand excavated slots and the slots are filled with a bentonite paste. The inner ring and annulus space between the two rings are filled with water, and the water level is kept constant by using two (2) Marriotte bottles. The bottles are also used to measure the volume of infiltrated water. The volume of water which infiltrates the test layer is recorded over a series of time intervals and the test is stopped when the infiltration rate becomes steady indicating an assumed state of saturation (Bowders, 2003). Fig 2.20 shows a schematic elevation of a DRI test set-up. Note that the outer ring restricts the lateral flow component of water from the inner ring that is permeating the base course being tested.

Figure 2.19 – Elevation schematic of a double ring infiltrometer test set-up. (source Bowders, 2003).
The infiltration rate can be determined by measuring the amounts of water added to the inner ring with respect to time. The infiltration rate is defined as the volume of water per time per unit area perpendicular to the flow, or

\[
\text{IR} = \frac{Q}{t \times A}
\]  

(2.22)

where:

- \( \text{IR} \) = infiltration rate (L/T)
- \( Q \) = volume of infiltrated fluid (l³)
- \( t \) = elapsed time for water to infiltrate base (sec)
- \( A \) = cross sectional area of inner ring (m²)

Given that:

\[
k = \frac{Q}{i \times A \times t}
\]  

(2.23)

where:

- \( k \) = hydraulic conductivity, L/T
- \( i \) = hydraulic gradient

If full depth saturation with no lateral flow is assumed then,

\[
k_i = \frac{Q}{A \times t} = \text{IR}
\]  

(2.24)

Therefore:

\[
k = \frac{\text{IR}}{i}
\]  

(2.25)
The depth of the base course \( (d) \) through which flow is being measured must be determined or estimated to arrive at the hydraulic gradient used in Eq. (2.25).

While in-situ methods take into account the actual condition of the material being tested, they have several serious drawbacks in that they are very expensive, labor intensive, time consuming, and subject to many testing variables like temperature change and system losses. For most transportation applications it is also preferable that the hydraulic conductivity of the base course be known prior to its placement to provide a basis for design and to avoid costly delays in construction while lengthy field permeability tests are conducted.

**2.9.2 Laboratory Tests**

The three most common methods of determining hydraulic conductivity in the laboratory are the constant head test, the falling head test, and indirect determination from the 1-D consolidation test. The consolidation test is primarily for use on clays and was not applicable for this study. The constant head and falling head tests were the two test methods utilized in this study.

The constant head test is used primarily in geotechnical practice for more permeable granular materials while the falling head test is more suitable for fine grained soils (Das, 1997). In order to limit consolidation influences during the test, ASHTO 215 and ASTM D2434 limit the amount of material passing the # 200 sieve to 10 % for constant head tests.

**2.3.2.1 Constant Head Test**

The constant head test is suited for granular materials when the expected hydraulic conductivity is greater than approximately \( 10^3 \) cm/sec (Richardson, 1997). A typical
arrangement of a constant head permeability test is shown in Fig. 2.21. In this type of test set-up, the water supply at the inlet is adjusted in such a way that the difference of head between the inlet and the outlet remains constant during the test period. After a constant flow rate is established, water is collected in a graduated flask for a known duration of time.

Figure 2.20– Typical constant head permeability test (after Das, 1998).

The total volume of water collected can be expressed as:

\[ Q = A \times v \times t \]  

(2.26)
Since

\[ v = k \times i \]  
(2.14)

Eq. (2.26) can be rewritten as

\[ Q = A \times k \times i \times t \]  
(2.27)

where:

- \( Q \) = volume of water collected
- \( A \) = area of cross section of soil specimen
- \( t \) = duration of water collection

And, because

\[ i = \frac{h}{L} \]  
(2.28)

Eq. (2.28) can be substituted into Eq. (2.27) to yield:

\[ Q = A \left( k \cdot \frac{h}{L} \right) t \]  
(2.29)

or

\[ k = \frac{QL}{Aht} \]  
(2.30)

The values of \( Q \), \( L \), \( A \), \( h \), and \( t \) are determined from the test, and the hydraulic conductivity for the soil is calculated from Eq. (2.30).

2.3.2.2 Falling Head Test

For materials with low hydraulic conductivity, the flow may be too small to be measured accurately, and thus the falling head method can be used (Richardson, 1997). This method is especially suitable for soils with very low hydraulic conductivities, such as clays, where the flow rate is small and precise measurements are required to accurately determine the value for \( k \) (Coduto, 1999).
A typical arrangement of a falling head permeability test, which uses a standpipe on the upstream side, is shown in Fig. 2.22. The water in the standpipe is not replenished as it is in the constant head reservoir. Thus, as the test progresses, the water level in the standpipe falls. The diameter of the standpipe is typically very small in comparison to the diameter of the test specimen, so precise measurements of flow can be made during this test. As compared to the constant head test, the analysis of the falling head test results is more complex because the hydraulic gradient is not constant. This means that the flow rate is also not constant requiring the derivation of a new equation for k.

The initial head difference, \( h_1 \), at time \( t = 0 \) is recorded and water is allowed to flow through the soil specimen such that the final head difference at time \( t = t_2 \) is \( h_2 \). The rate of flow through the soil at any time, \( t \), is:

\[
q = k \frac{h}{L} A = -a \frac{dh}{dt}
\]  

(2.31)

Where:
- \( q = \) flow rate
- \( a = \) cross sectional area of the standpipe
- \( A = \) cross sectional area of the soil specimen
- \( t = \) interval of time over which flow occurs, sec
- \( h_1 = \) head difference at start of test, cm
- \( h_2 = \) head difference at end of test, cm

Rearrangement of Eq. (2.31) gives

\[
dt = \frac{aL}{Ak} \left( -\frac{dh}{h} \right)
\]  

(2.32)

From Eq. (2.32)
\[
\int_0^t \, dt = \int_{h_1}^{h_2} aL \left( -\frac{dh}{h} \right) \tag{2.33}
\]

or:

\[
k = \frac{aL}{At} \ln \left( \frac{h_1}{h_2} \right) \tag{2.34}
\]

The values of \( a, L, t, h_1, \) and \( h_2 \) are determined from the test, and the hydraulic conductivity for the soil is calculated from Eq. (2.34).

Figure 2.21—Typical falling head permeability test (after Das, 1998).
2.3.2.3 Permeameter Types

Laboratory permeability tests may be performed with either rigid wall or flexible wall permeameters. Rigid wall permeameters are generally less expensive than flexible wall devices but suffer from the disadvantages of incomplete control over the stresses that act on the soil specimen, difficulty in trimming natural soils into the containing ring, and potential leakage along the interface between the soil and wall of the permeameter. The problem of sidewall leakage is of particular concern because a tiny gap between the specimen and the cell wall could lead to a large error in the calculated hydraulic conductivity. Flexible wall permeameters offer the potential for more complete control over stresses and are better suited for minimizing sidewall leakage (Daniel, 1989).

In this study rigid wall permeameters were used with constant head tests and flexible wall permeameters were used with falling head tests. However, rigid wall permeameters can be used in the falling head test and flexible wall permeameters can be used in the constant head test.

A rigid wall permeameter can be of three (3) types; compaction mold, double ring compaction mold, and consolidation cell (Das, 1997). A compaction mold used as a rigid wall permeameter is shown in Fig. 2.23. In this case, a proctor mold is utilized but any cylindrical mold could be used.

The compaction mold permeameter is a simple and economical device. There are, however, several disadvantages in that leakage is possible along the interface of soil and the sidewall, shrinkage and swelling of the soil specimen are not controlled, and the specimen may be unsaturated if back pressure saturation is not used (Das, 1997).
As compared to rigid wall permeameters, flexible wall permeameters offer the potential for more complete control over stresses and are better suited for minimizing side wall leakage (Daniel, 1984). These advantages stem from the ability to control the pressures within the sample and the ability to apply confining pressure to the sides of the specimen through a flexible rubber membrane.

![Compaction mold being used as a rigid wall permeameter for use in a constant head permeability test.](image)

Figure 2.22 – Compaction mold being used as a rigid wall permeameter for use in a constant head permeability test.

This reduces side wall leakage and allows for the measurement of the effective stresses in the specimen. In order to achieve this control over gradient and effective stress on the specimen, flexible wall permeameters must have independent control over pressures applied to the cell and to the top and bottom of the specimen.
Flexible wall permeability tests, which are performed in triaxial cells or modified triaxial cells, have several disadvantages as well. The membranes used to confine the soil are normally made of latex, butyl, or neoprene rubber which can be attacked and destroyed by certain chemicals. In order to maintain contact between membrane and the soil specimen, the pressure in the cell liquid must be higher than the pore pressure of the specimen. In order to test with an elevated hydraulic gradient, the effective stress at one end of the specimen must be fairly large and the effective confining pressure cannot be less than the pressure drop across the specimen (Das, 1997).

Fig. 2.24 shows an assembled flexible wall permeameter for use in a falling head permeability test. The flexible membrane is used to isolate the sample from the water which provides the confining pressure.

![Flexible Wall Permeameter](image)

Figure 2.23 – Assembled flexible wall permeameter for use in a falling head permeability test.
2.10 Correlation of Hydraulic Conductivity to Other Soil Parameters

While determining the hydraulic conductivity of a granular base course with field or laboratory tests is reliable, these tests are time consuming and expensive. As a result, hydraulic conductivity is often estimated on the basis of correlations with other material properties such as grain size characteristics, dry density, and porosity or void ratio (Crovetti, 1993).

Over the years, a wide variety of theoretical and empirical equations have been presented for estimating the hydraulic conductivity of porous media. One of the earliest correlations was developed by Hazen (1892) for uniform sand. He developed the formula as:

$$k = C \times D_{10}^{1.2}$$

Effective grain size ($D_{10}$) is defined as the particle size, in millimeters, such that 10 percent of the material contained in the sample has a smaller grain size. According to Bowders and Das, the empirical coefficient $C$, proposed by Hazen varies from 1.0 to 1.5.

However, in Hazen’s 1911 paper he states that the coefficient varies from 400 for dirty, angular sands to 1200 for uniform clean sands when units of meters per day are employed in the test. When the values are converted to centimeters per second the coefficient varies from 0.46 to 1.4 which is a significant deviation from the values quoted in the literature. Hazen cautioned that the formula was developed using clean sands of uniform gradation and it was never intended to apply to clays, hardpans, soils, and other materials (Hazen, 1911).

Hazen’s original data is presented in Table 2.10 which shows the rate at which water flows through uniformly graded sands, at various heads, at a temperature of 10º C.
Hazen states that the friction in gravels having an effective size greater than 3 mm varies in such a way as to make the application of a general formula very difficult. While Hazen cautions that his correlation only applies to uniformly graded sands with an effective size smaller than 3 mm, his equation has been used widely to predict hydraulic conductivity for a wide range of effective particle sizes.

Table 2.10 – Rate at which water flows through uniformly graded sands at various heads (after Hazen, 1892).

| h/l | Effective size in mm, 10 % finer than (D<sub>10</sub>): |
|-----|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
|     | 0.1             | 0.2             | 0.3             | 0.4             | 0.5             | 1               | 3               |                 |
|     | V, meters/day   |                 |                 |                 |                 |                 |                 |                 |
| 0.001 | 0.01 | 0.04 | 0.09 | 0.16 | 0.25 | 1 | 9 |                 |
| 0.005 | 0.05 | 0.2 | 0.45 | 0.8 | 1.25 | 5 | 45 |                 |
| 0.01 | 0.1 | 0.4 | 0.9 | 1.6 | 2.5 | 10 | 90 |                 |
| 0.05 | 0.5 | 2 | 4.5 | 8 | 12.5 | 50 | - |                 |
| 0.1 | 1 | 4 | 9 | 16 | 25 | 100 | - |                 |
| 0.5 | 5 | 20 | 45 | 80 | 125 | - | - |                 |
| 1 | 10 | 40 | 90 | 160 | - | - | - |                 |
| 2 | 20 | 80 | 180 | 320 | - | - | - |                 |

A casual inspection of the data presented in Table 2.10 indicates that there is nearly perfect linear agreement between flow and gradient for a given grain size which might suggest that the data was derived, rather than measured. To further investigate this possibility, Hazen’s original data shown in Table 2.10, was used to calculate a value for k in cm/sec for different values of D<sub>10</sub> and various gradients (i) as shown in Table 2.11. When one takes into consideration the extremely rigid controls of Hazen’s experiment (hand measurement of individual sand particles was employed in determining D<sub>10</sub>), the nearly perfect linear agreement between flow and gradient for a given grain size in Table 2.10 can be explained.
Table 2.11 – Hydraulic conductivity in cm/sec for uniformly graded sands at various heads (after Hazen, 1892).

<table>
<thead>
<tr>
<th>$h/l$ (j)</th>
<th>$D_{10}^2$</th>
<th>0.010</th>
<th>0.040</th>
<th>0.090</th>
<th>0.160</th>
<th>0.250</th>
<th>1.000</th>
<th>9.000</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k=v/i$</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>0.289</td>
<td>1.157</td>
<td>10.417</td>
</tr>
<tr>
<td>0.001</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>0.289</td>
<td>1.157</td>
<td>10.417</td>
<td></td>
</tr>
<tr>
<td>0.005</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>0.289</td>
<td>1.157</td>
<td>10.417</td>
<td></td>
</tr>
<tr>
<td>0.010</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>0.289</td>
<td>1.157</td>
<td>10.417</td>
<td></td>
</tr>
<tr>
<td>0.050</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>0.289</td>
<td>1.157</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.100</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>0.289</td>
<td>1.157</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>0.500</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>0.289</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>1.000</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2.000</td>
<td>0.012</td>
<td>0.046</td>
<td>0.104</td>
<td>0.185</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

The data shown in Table 2.11 is illustrated graphically in Fig.2.25 as $k$ versus $D_{10}^2$ to visualize the scatter in Hazen’s data. Additionally, Fig 2.25 can be used to determine the value of $C$ used in the Hazen equation. From the slope of the line drawn in Fig 2.25 one can see the value of $C$ is 1.16 for Hazen’s original data. However, according to Hazen (1911),

“In the flow of water through much more complex passageways between the grains of sand there will obviously variations in the coefficient, depending on the shape and smoothness of the sand particles, the way in which they are packed, and the quantity of finer foreign material also contained in those pores”.

This statement suggests that the value of $C$ will vary with respect to the properties of the soil being tested. This concept is discussed further in 4.2.2.
Sherard (1984) proposed Eq. 2.36 for determining the correlation between grain size and hydraulic conductivity:

\[
k = 0.35 \times D_{15}^2
\]  \hspace{1cm} (2.36)

Where \( D_{15} \) is defined as the size in mm such that 15 percent of the material is of smaller grains.

Drawing from earlier research by Casagrande, Sherard concluded that hydraulic conductivity was a function of the sizes of pore channels. He further hypothesized that the pore channels are determined by the sizes of the finer particles and that these particle sizes are well represented quantitatively by \( D_{15} \). Sherard’s correlation was developed
based on tests of sands and gravels that were compacted to a dense state without appreciable visible segregation and did not have a significant quantity of material finer than the # 200 sieve (Sherard, 1984). Based upon the values of C_u for material used in Sherard’s work, it is likely that P_{200} is less than 5 % which would mean that his correlation is not directly applicable to granular base courses which normally have more than 5% of the material passing the # 200 sieve.

The first attempt to create a relationship between material index properties and the hydraulic conductivity of granular base course might be credited to Moulton (1980). Moulton’s relationship was determined by statistically correlating the measured hydraulic conductivity for a large number of samples in which the properties known to exert an influence on permeability were also measured. Moulton’s algorithm is based upon data from the literature (Chu, et al., 1955; Strohm et al., 1967; Smith, et al., 1964; Barber and Sawyer, 1952; Lane and Washburn, 1946; and Yemington, 1955). For the most part, the hydraulic conductivity tests from these studies were conducted in rigid wall permeameters under low gradients (Richardson, 1997).

In addition to particle size, Moulton incorporated terms which accounted for the percentage of fines and material density in developing Equation 2.37, (Moulton, 1980):

\[
k = \frac{6.214 \times 10^5 \times D_{10}^{1.1478} \times n^{6.654}}{P_{200}^{0.597}}
\]  

(2.37)

Where:

- \( D_{10} \) = the particle size in the particle size distribution curve corresponding to 10% finer
- \( n \) = the porosity of the material
- \( P_{200} = \) percent of material finer than the No. 200 sieve (0.075 mm)
- \( k = \) ft/day.

It is particularly important to note that the amount of fines exerts a marked influence on the coefficient of permeability in Moulton's equation. The FHWA Subdrainage Design Manual (1980) indicates that a small increase in the amount of fines can cause a large decrease in the hydraulic conductivity.

Equation 2.37 is recommended by the Federal Highway Administration in Report No. FHWA-TS-0-80-224 to predict the hydraulic conductivity of granular drainage and filter materials. The nomograph used to solve this equation graphically, as well as a sample solution, is presented in Fig.2.36.

The nomograph assumes saturated soil and a specific gravity of 2.70 to arrive at an estimated value of \( k \). For a granular base course where \( P_{200} = 8\% \), \( D_{10} = .18 \) mm, and \( \gamma_d = 130 \) lb/ft\(^3\) the solution is derived by drawing a line through the appropriate values for \( P_{200} \) and \( D_{10} \) to intersect the vertical pivot line in the center of the table. A line is then drawn from this point of intersection through the appropriate value for \( \gamma_d \) till it intersects the \( k \) value line. In this case, \( k = 0.7 \) ft/dy or \( 2.46 \times 10^{-4} \) cm/sec.

Richardson (1997) reviewed the data on which the FHWA (Moulton) equation is based and concluded the hydraulic conductivity tests suffered from potential problems with air blockage and non-uniform end conditions. In addition, incorrect specific gravity data may have been used to determine specimen porosity. All of these issues would lead to falsely low values. Much of the reported data are from tests in which the specimens were saturated only by submergence. Thus the specimens may have been unsaturated, a condition that can reduce the measured hydraulic conductivity (Richardson, 1997).
Figure 2.25 – Nomograph used to solve Moulton’s equation (source FHWA Subdrainage Design manual, 1980).
Correlations were also developed by Kozeny (1927), Casagrande (1937), Terzaghi (1925), Rose (1950), and others using porosity and effective grain size as the primary property influencing hydraulic conductivity. Though useful, these formulas are more suited to clays and sands, as opposed to well graded aggregate blends.

An additional aid for estimating hydraulic conductivity is provided in Fig 2.27, which shows typical gradations and hydraulic conductivities of open graded bases and filter material.

Figure 2.26 – Typical hydraulic conductivities of open graded bases and filter materials (source FHWA Subdrainage Design manual, 1980).
While open graded base courses are not the subject of this report, the figure emphasizes the basic principle that as fines content increases the hydraulic conductivity decreases.

2.11 Relationship of Hydraulic Conductivity to Other Soil Properties

Figure 2.28 illustrates that the optimum percentage of fines is not the same for different engineering properties of a base course.

![Graph illustrating the relationship between fines content and engineering properties.](image)

Figure 2.27 – Summary of effects of fines content on strength, density (VMA), frost heave potential, and hydraulic conductivity (source NSA handbook, 2000).
This figure suggests that optimizing the percentage of fines with regard to hydraulic conductivity or drainage will also improve frost heave performance, but will result in poor strength and density of the base course.

Not only does the percentage of fines affect hydraulic conductivity it also influences the strength and density of the base material. Up to a certain percentage of fines, the strength increases. However, beyond this point the coarse aggregate tends to float in a matrix of fines and lose grain-to-grain contact which causes a reduction in strength. Similarly, the density peaks at a specific percentage of fines (Persing, 1978).

2.12 Design Philosophy

According to the NSA Handbook (2000), the fact that the optimum percentage of fines for hydraulic conductivity and strength are different has led to the use of two separate philosophies in pavement design practice:

- “One philosophy holds that the gradation must be practically devoid of fines. If necessary to provide stability during construction, the aggregate is treated with just enough asphalt or Portland cement to provide the stability needed to support construction traffic until it can be confined by the final wearing surface. A base so open graded that it must be stabilized with asphalt or cement may cost twice as much as one not quite so open. Also, problems with stability have been observed with these open graded bases that drain well. The second philosophy attempts to improve hydraulic conductivity by removing some of the fines while maintaining the desired stability and minimizing the increase in the cost of production over more conventional dense graded aggregate”.

Both design philosophies have advantages and drawbacks. Use of the open graded base alleviates the problem of drainability, but the pavement systems are harder to construct and are more expensive. The dense graded aggregate is stronger, cheaper and
easier to construct, but problems with drainability is a serious drawback.

When the use of drainable bases is not feasible, either because of cost or construction details, consideration should be given to thickening the structural layers above the base to compensate for the loss of base course strength due to low hydraulic conductivity.

2.13 Soil Suction

While the ability of the base course materials to drain free water from the pavement system is of major concern, the propensity of the base course to draw water from below should also be considered in design. Soil suction is a measure of a material’s affinity for water while the hydraulic conductivity of a granular base course controls the rate of moisture migration within the aggregate layer (Guthrie, et.al, 2001).

The continuous void spaces in the base course serve as bundles of tubes of varying cross sections. The surface tension of water can cause it to rise above the ground water table if the void spaces in the material are less than a given diameter. Since capillary rise into the base course from the underlying subbase soils contributes to moisture ingress and movement of moisture in the base course, this study will attempt to relate capillary rise to the percentage of fines present in a compacted specimen.

Fig. 2.29 illustrates the fundamental concept of the height of rise of water within a capillary tube. The height of the water rise in the capillary tube can be predicted by summing the forces in the vertical direction as follows:

\[
\left( \frac{\pi}{4} d^2 \right) h c \gamma_w = \pi d T \cos \alpha
\]

(2.39)

Where:
\[ T = \text{surface tension (F/L)} \]
\[ \alpha = \text{angle of contact} \]
\[ d = \text{diameter of capillary tube} \]
\[ \gamma_w = \text{unit weight of water} \]

Solving for \( h_c \):

\[ h_c = \frac{4T \cos \alpha}{d\gamma_w} \]  
(2.40)

For pure water and clean glass, \( \alpha = 0 \), thus Eq. (2.40) becomes

\[ h_c = \left( \frac{4T}{d\gamma_w} \right) \]  
(2.41)

Factoring out the constants in Eq. (2.41) yields:

\[ h_c \propto \frac{1}{d} \]  
(2.42)

It is evident that the height of water rise in a capillary tube is inversely related to the radius of the capillary (Scullion, 2000). This principle combined with the fact that the item having the greatest effect on pore size is the grain sizes (Taylor, 1948), suggests that the percentage of fines in a granular base course has a direct bearing on the capillary rise or suction of that base course.

The tube suction test (TST), which was developed in a cooperative effort by the Finnish National Road Association and the Texas Transportation Institute (TTI), is used to assess the moisture susceptibility of granular base material (Guthrie et al., 2001). Moisture susceptibility is the degree to which moisture ingress degrades the engineering
properties of the granular base course. There are no AASHTO or ASTM standards at this time, however the Texas Department of Transportation began implementing the test statewide in 2001.

![Diagram of capillary tube and pressure within the capillary tube](Figure 2.28)

**Figure 2.28 – Rise in water in the capillary tube and pressure within the capillary tube (After Das, 1998).**

In the TST, as shown in Fig. 2.30, a probe is used to measure the dielectric constant at the surface of the sample. The dielectric constant is a measure of the material’s insulating capabilities; it is equal to the ratio of the electrostatic capacity of condenser plates separated by the given material to that of the same condenser with a vacuum between the plates. The dielectric constant of a vacuum is 1.0. The electrical conductivity is the reciprocal of electrical resistivity (1/ohm – cm) and the ratio of the current density to the electric field strength.

The Dielectric Constant and Conductivity Meter is manufactured for engineering applications by Adek, Ltd. of Estonia and costs approximately $3000. Two probes are
supplied with the meter; the first is the surface measurement probe and the second is a tube measurement probe developed for internal dielectric measurements. From experiments conducted at TTI the probe measures the average dielectric properties of the top 20 mm of a typical base course sample (Scullion, et.al., 1997).
The dielectric constant measured with this device is used as an indicator of the free or unbound water within the aggregate sample. It is this unbound water that is thought to
be directly related to the strength of the material and its ability to withstand repeated
freeze-thaw cycling.

The moisture susceptibility ranking established by the TST is based on the mean
surface dielectric value of compacted specimens after a 10-day capillary soak in the
laboratory. On the basis of the test results obtained from both Texas and Finnish
aggregates, Scullion and Saarenketo (1997) have proposed the relationship between
electrical properties and field performance of granular base material presented in Table
2.12. From this table it is concluded that a dielectric value less than 10 would be
representative of a granular material which has good drainage and strength
characteristics, making it entirely suitable as a base material. Values between 10 and 16
would be warning signals that the material has significant levels of free water and may
be susceptible to freeze-thaw damage. Values of 16 or greater represent materials that
are subject to significant loss of shear strength and are extremely susceptible to frost
damage.

The research results show that the suction properties of base aggregates have a very
significant effect on the deformation properties of the base course. Suction properties, in
turn, are primarily dependent on the fines content, but also on the chemical properties of
the aggregate.

A simplified soil suction test, similar to the capillary wetting phase of TxDot
standard triaxial compression test shown in Fig. 2.31, was used in this study to measure
the moisture susceptibility of the granular base courses.
Table 2.12 - Proposed relationship between electrical properties and field performance of granular base material (after Scullion and Saarenketo, 1997)

<table>
<thead>
<tr>
<th>Dielectric value</th>
<th>Electrical conductivity (µS/cm)</th>
<th>Material</th>
<th>Strength and deformation properties</th>
<th>Frost susceptibility and water susceptibility</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;5</td>
<td>&lt;10</td>
<td>dry and open graded base with low water adsorption and large air-solid ratio</td>
<td>low tensile strength, might be sensitive to permanent deformation by compaction</td>
<td>non frost susceptible, non water sensitive</td>
</tr>
<tr>
<td>5-7</td>
<td>&lt;50</td>
<td>Dry base with low water adsorption value and optimum dry density.</td>
<td>optimum strength properties</td>
<td>non frost susceptible, non water sensitive</td>
</tr>
<tr>
<td>7-10</td>
<td>&lt;100</td>
<td>Slightly moist base with high suction value</td>
<td>high shear strength because of suction, hysteresis has great effect on strength</td>
<td>might become water sensitive and frost susceptible if drainage stops working</td>
</tr>
<tr>
<td>10-16</td>
<td>&lt;150</td>
<td>moist base</td>
<td>reduced shear strength because of reduced suction</td>
<td>frost susceptible and water sensitive</td>
</tr>
<tr>
<td>&gt;16</td>
<td>&lt;150</td>
<td>wet or water saturated base</td>
<td>adequate shear strength, no positive pore water pressure under dynamic load</td>
<td>may form ice lenses</td>
</tr>
<tr>
<td>&gt;16</td>
<td>&gt;150</td>
<td>wet or water saturated base</td>
<td>under a dynamic load plastic deformation may occur because of high pore water pressure and low shear strength</td>
<td>extremely frost susceptible</td>
</tr>
</tbody>
</table>

In the TxDot capillary wetting phase for samples with little or no plasticity, the samples are weighed after 10 days to assess the total moisture gain. Since the granular base course used in this study is assumed to be non-plastic, the determination of moisture susceptibility depends only on soil suction and hydraulic conductivity.
Based on this review of the literature it was concluded that hydraulic conductivity would be used as the primary indicator of the drainage quality of the base course materials. Suction testing would only be used as a supplementary indicator of the drainage characteristics of the base materials.

Figure 2.30 – Capillary wetting phase of the TXDOT standard triaxial compression test (source TXDOT specification Tex-117-E).
CHAPTER 3
RESEARCH METHODOLOGY

3.1 Site Selection

The site selection process began with the identification of quarries that produce AHTD Class 7 crushed stone aggregate base course throughout the state of Arkansas. The quarries were divided according to rock type, parent geological formation, and geographic location. Portions of Arkansas, such as the eastern and southeastern portions of the state, do not contain geologic formations conducive to the mining and production of crushed stone aggregate base and were not represented in this study. While not all approved Class 7 base is native to the state, all sites evaluated as part of this research are located within Arkansas.

After consulting with AHTD personnel in the Materials Division along with Resident Engineers from within AHTD Districts 4 and 10, the focus of this study was narrowed to five quarries that represent the predominant rock type and geological formations in Arkansas: Sharps (Benton County), Preston (Crawford County), Black Rock (Lawrence County), Glen Rose (Hot Springs County), and Granite Mountain (Pulaski County). Figure 3.1 illustrates the geographic distribution of the sites throughout the state. The location of each of the selected quarries is represented with a star. The aggregate types and parent geological formations for each quarry are listed in Table 3.1.
Figure 3.1: Geographical distribution of selected quarries for this study.

Table 3.1: Parent stone of the selected quarries.

<table>
<thead>
<tr>
<th>Quarry</th>
<th>Location (County)</th>
<th>Aggregate Type</th>
<th>Geological Formation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharps</td>
<td>Benton</td>
<td>Limestone</td>
<td>Boone</td>
</tr>
<tr>
<td>Preston</td>
<td>Crawford</td>
<td>Sandstone</td>
<td>Hartshorne</td>
</tr>
<tr>
<td>Black Rock</td>
<td>Lawrence</td>
<td>Dolomite</td>
<td>Powell</td>
</tr>
<tr>
<td>Glen Rose</td>
<td>Hot Springs</td>
<td>Noviculite</td>
<td>Arkansas Noviculite</td>
</tr>
<tr>
<td>Granite Mountain</td>
<td>Pulaski</td>
<td>Syenite</td>
<td>Cretaceous</td>
</tr>
</tbody>
</table>
3.2 Sampling

AHTD personnel were largely responsible for sampling of the subject quarries and trucking the material to the University of Arkansas’ Engineering Research Center (ERC) in Fayetteville, Arkansas. It is understood that the material was obtained from the working faces of produced Class 7 stockpiles with heavy-duty front-end loaders. The delivered material was stockpiled at the ERC facility. The close proximity of Sharps quarry to the ERC made it possible for the researchers to obtain material from that quarry without assistance from the AHTD.

Approximately 3000 to 5000 pounds of Class 7 material was sampled from each quarry. The stockpiled material was sampled in accordance with AASHTO T-2. Sieve analyses (AASTHO T-37) were conducted on the materials obtained to determine the “as-received” material gradations. Grainsize distribution curves for each quarry can be found in Appendix A.2. After air-drying, the aggregate was fractioned using the AHTD gradation acceptance criteria particle size breakpoints of 37.5 mm (1-1/2 in.), 19.0 mm (3/4 in.), 4.75 mm (No. 4), 0.425 mm (No. 40), and 0.075 mm (No. 200) sieves.

3.3 Index Properties

Liquid limit (AASHTO T-89), plastic limit and plasticity index (AASHTO T-90) testing was completed on material passing the 0.425 mm (No. 40) sieve. Both wet and dry sieving methods were used to obtain minus 0.425 mm (No. 40) material for use in plasticity testing. The specific gravity and absorption of material retained on the 4.75 mm (No. 4) sieve were determined in accordance with (AASHTO T-85). Two samples of each quarry material were prepared with grain size distributions intended to meet the upper and lower boundaries of the AHTD Class 7 acceptance criteria. This criterion is
given in Section 303 of the AHTD Standard Specifications for Highway Construction (AHTD, 1996). The upper boundary blend followed a grain-size distribution curve that allowed the greatest percentage of material passing a particular gradation breakpoint. The lower boundary blend followed a distribution curve having the least amount of material passing a particular gradation breakpoint. Moisture density relationships were developed for each of these blends in accordance with AASHTO T-180, Method D. In accordance with AHTD requirements, all plus 19.0 mm (3/4 in.) material was replaced with an equivalent weight of plus 4.75 mm (No. 4) material. This process of sample preparation and determination of material index properties was repeated for each quarry.

3.4 Model Blending

The "as-received" material gradation results were compared to historical gradation information furnished by the AHTD Materials Division. Following an evaluation of all the gradation data, a model gradation was established for each quarry. The model gradation was intended to represent the upper boundary of the historical and "as-received" gradation data, under the hypothesis that finer grained blends would represent the worst-case conditions for both strength and hydraulic conductivity. Figure 3.2 illustrates the historical, "as-received", and model gradations for Sharps quarry along with the current AHTD acceptance limits. Historical gradations for all quarries can be found in Appendix A.1. The maximum dry density and optimum moisture content for the model blends was established based upon the results of the density testing for the upper and lower bound blends (Fig. 4.2).
Figure 3.2: Grain size distribution curves for the historical and model blend gradations (Sharps quarry shown).

It was assumed that these values for the extremes in gradation adequately represented the full range of sample blends to be tested. Modified Proctor testing for each of the various sample blends was considered unnecessary since only minor differences in unit weight and optimum moisture content existed between the coarsest and finest blends of each material.

A variety of grain-size distributions were used to create compacted specimens for shear strength testing and evaluation. Boundary blend samples were first established according to the AHTD upper and lower gradation acceptance tolerances. The upper boundary blend had a fines content, minus 0.075 mm (No. 200), of 10 percent. The lower boundary blend had a fines content of 6 percent in lieu of the expected 3 percent
because the historical data from each quarry indicated that 6 percent was the practical minimum percentage of fines produced in the state.

The fines content of the model blends was varied from 6 percent to 16 percent in increments of 2 percent. The amount of material passing the 0.425 mm (No. 40) sieve was adjusted as the fines content increased in order to maintain a constant total sample weight. The purpose of varying the fines content on only the model gradation and not the boundary blends was to replicate likely crusher production scenarios. While Fig. 3.2 graphically illustrates the blending concept for the Sharps quarry, Table 3.2 presents the percentages of material used in the model gradations for each quarry at a fines content of 10 percent. Blending for all quarries and all gradations can be found in Appendix D.

Table 3.2: Summary of model blend gradation percentages for all quarries at a fines content of 10 percent.

<table>
<thead>
<tr>
<th>Sieve, mm (size)</th>
<th>Sharps</th>
<th>Preston</th>
<th>Black Rock</th>
<th>Glen Rose</th>
<th>Granite Mountain</th>
</tr>
</thead>
<tbody>
<tr>
<td>19.0 mm (3/4 inch)</td>
<td>81</td>
<td>75</td>
<td>81</td>
<td>86</td>
<td>81</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>43</td>
<td>35</td>
<td>35</td>
<td>47</td>
<td>45</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>18</td>
<td>20</td>
<td>18</td>
<td>20</td>
<td>18</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

Compacted cylindrical specimens were prepared in a split mold for triaxial shear testing. These specimens measured 15.25 cm (6 in.) in diameter and 30.48 cm (12 in.) in height, resulting in a specimen volume of approximately 5550.1 cubic cm (0.196 cubic ft). The AHTD requires that all base course aggregate be compacted to at least 98 percent of the maximum modified Proctor dry density at a moisture content near the optimum value (AHTD, 1996). Accordingly, the established model blend maximum dry density value was multiplied by 98 percent to obtain a target unit weight in kg per cubic
meter (lbs. per cubic ft). Each of the aggregates were allowed to dry for several days inside the laboratory after which hygroscopic moisture contents on the order of 0.2 percent were measured. Because air-dry weights for these materials were so close to the oven-dry weights, air-dry material weights were used in sample blending. The weight of material to be compacted into each triaxial specimen was determined by multiplying the target unit weight by the specimen volume. Table 3.3 presents the material quantities used for a typical model blend. The material weights use in the blending for all gradations and quarries are presented in Appendix D.

Table 3.3: Summary of material quantities used in preparing test specimens for Sharps quarry.

<table>
<thead>
<tr>
<th>Size</th>
<th>% Passing</th>
<th>Lower bound</th>
<th>Model Gradation</th>
<th>Upper bound</th>
<th>% Passing</th>
<th>Lower bound</th>
<th>Model Gradation</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2&quot;</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>81</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>43</td>
<td>55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#40</td>
<td>10</td>
<td>18</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#200</td>
<td>6</td>
<td>Varied</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PAN</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>6153.1</td>
<td>3/4&quot;</td>
<td>10</td>
<td>1230.6</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>3076.5</td>
<td>#4</td>
<td>35</td>
<td>4307.2</td>
</tr>
<tr>
<td>#40</td>
<td>15</td>
<td>1946.9</td>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>4</td>
<td>492.2</td>
<td>#200</td>
<td>6</td>
<td>2461.2</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>738.4</td>
<td>Pan</td>
<td>10</td>
<td>1230.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4676.3</td>
<td>#4</td>
<td>38</td>
<td>4676.3</td>
</tr>
<tr>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>12</td>
<td>1476.7</td>
<td>#200</td>
<td>6</td>
<td>2461.2</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>738.4</td>
<td>Pan</td>
<td>12</td>
<td>1476.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4676.3</td>
<td>#4</td>
<td>38</td>
<td>4676.3</td>
</tr>
<tr>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1230.6</td>
<td>#200</td>
<td>6</td>
<td>2461.2</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>738.4</td>
<td>Pan</td>
<td>14</td>
<td>1722.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4676.3</td>
<td>#4</td>
<td>36</td>
<td>4676.3</td>
</tr>
<tr>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>984.5</td>
<td>#200</td>
<td>2</td>
<td>2461.2</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1230.6</td>
<td>Pan</td>
<td>16</td>
<td>1969.0</td>
</tr>
</tbody>
</table>

108
3.5 Selection of Evaluative Methods

Consolidated-drained (CD) conventional triaxial testing was selected to evaluate the effect of fines on the shear strength of AHTD Class 7 crushed stone aggregate base course. Index tests such as the CBR or R-Value do not provide direct information on performance capabilities of a material. The values obtained from these index or classification tests must be correlated to the real engineering properties for the material. The CBR and R values obtained for crushed stone aggregate base would all be confined to the very highest range of their respective evaluative scale (perhaps exceeding the scale) and would yield only quantitative information on the relative performance of various base course materials. For example: CBR values for crushed stone base would typically range in the 80’s and 90’s (Barksdale, 2000). While the Texas Triaxial test provides definitive information on engineering properties, it is no longer accepted as a material standard by either AASHTO or ASTM. In addition, the equipment required to conduct this test was not available. Even if the equipment could have been fabricated, the requirement of molding seven test specimens for each single model gradation was considered excessive. Also, the requirement to subject all test specimens to capillary moisture action for 10 days would have greatly extend the testing regimen. The development of a series of Mohr stress circles to define the Mohr-Coulomb failure envelope for Texas Triaxial testing is essentially the same as the methods of CD triaxial testing, except the Texas Triaxial offers less control over applied stresses. The consolidated-drained testing can be accomplished more efficiently than Texas Triaxial testing.
According to the AASHTO design guide, the definitive material property for assessment of subgrade and base material performance in pavement analyses and design is the repeated load triaxial test, or “resilient modulus” test. However, this testing procedure requires elaborate testing equipment and highly trained personnel. Research completed by Thompson and Smith (1990) also revealed that resilient modulus testing may not be an effective method of evaluating the performance potential of crushed stone aggregate base course materials having slightly different gradations.

After evaluating the advantages and disadvantages of each of the test methods, conventional triaxial testing was selected as the method which would offer the most effective comparative evaluation of the effect of fines on strength. Additional considerations are that the test protocol developed from this study would be easily transportable to other testing agencies and the necessary equipment for this testing is relatively inexpensive, when compared to resilient modulus testing equipment and is widely available from many commercial vendors. The drainage characteristics of the granular aggregate base course are such that it may be tested rapidly using the consolidated-drained method.

To evaluate the drainage characteristics of the various gradations both falling head and constant head laboratory hydraulic conductivity tests were selected using both rigid and flexible wall permeameters. Gradients for both test types were kept very low in an attempt to replicate field conditions. While infiltrometers of various configurations could have been used to measure performance of these materials in a field setting the ability to replicate testing procedures and create constant environmental conditions would have been compromised. The selection of commercially available testing
equipment and the standard procedures of the laboratory tests make this testing protocol easily transportable to other testing agencies. In addition to hydraulic conductivity testing, suction, or capillary rise, tests were conducted to determine if variation in fines content had an impact on the materials affinity for water or ability to retain water.

3.6 Triaxial Testing

Triaxial test specimens 152 mm (6 in.) in diameter by 304 mm (12 in.) in height were created in a split mold, Figs. 3.3 and 3.4, using dynamic compaction from a Marshall hammer having a 3” face. A detailed step-by-step description of the preparation of triaxial test specimens is presented in Appendix E.

Figure 3.3: 6 in. by 12 in. cylindrical specimen split mold. Figure 3.4: Compaction of specimen using Marshall 3” hammer.

The equipment used for the testing consisted of a triaxial system having the following components: computer controlled loading frame, triaxial cell, load cell, pressure transducers, pressure panel, variable power supply, communication module, and PC unit. The hardware selected for this research was the Sigma-1 automated load test system manufactured by Trautwein Soil Testing Equipment. The Sigma-1 loading frame is powered by a D.C. servo motor which theoretically provides infinite resolution on the control of the motor’s rate of revolution. In practice however, one revolution of the
motor consists of 2000 counts, or pulses, and the pulse rate can vary from approximately 58 million pulses per second to as slow as 3 seconds per pulse. As a result, the maximum and minimum travel rates are approximately 20 mm/minute (0.787 in./minute) and \(7.52 \times 10^{-3}\) mm/minute \((2.96 \times 10^{-5}\) in./minute\), respectively. The peak and continuous torque provided by the motor is 500 ounce-inches and 250 ounce-inches, respectively. The load frame is capable of providing 62.3 kN (14 kip) of thrust through the use of an efficient ball screw and gear reduction system. The load cell has a 44.5 kN (10 kip) maximum capacity with a linearity of 0.025 percent. The linear displacement sensor has a maximum range of 50 mm (1.97 inches) with a linearity of 0.03 percent. The pore pressure transducers have a maximum pressure 1034 kPa (150 psi) and a linearity of 0.1 percent. Figure 3.5 shows the testing system configuration. The GEOTAC® TestNet component provided automatic data acquisition according to a user developed schedule and control of the load frame motor in accordance with a user defined program.

Staged consolidated-drained (CD) triaxial testing was selected as the method of determining shear strength for the materials tested in this study. Test specimens were backpressure saturated and then sequentially subjected to a confining pressure and loaded axially until yielding occurred. The confining pressure was increased for subsequent stages and an axial load was applied until the specimen yielded. Drained testing was considered appropriate for the aggregate material tested in this study because excess pore water pressure, \(u_c\), was dissipated rapidly during the application of confining pressure and deviator stress. In consolidated-drained (CD) testing drainage is permitted during both stages so that full consolidation occurs under the confining stress \(\sigma_2 = \sigma_3\)
and no excess pressure develops during the application of deviator stress, $\sigma_d$. The test specimen was considered to be fully consolidated when the water levels in the accumulators, which provided pressure to the top and bottom drain lines of the test specimen, ceased to change after application of a specified confining pressure. This stabilization occurred within 10 seconds after application of a confining pressure. The specimens were loaded so slowly that and pore pressures that would tend to develop were dissipated to at least the 95 percent consolidation level at all times during cyclic and staged triaxial testing as described later in this chapter. Each test specimen initially underwent a cyclic test in the elastic region followed by staged load testing at varying

Figure 3.5: Testing Apparatus for Consolidated-Drained Triaxial testing.
effective confining pressures which were carried to deformations somewhat beyond the yield point.

Each sample was backpressure saturated according to the procedure presented in Appendix B.3. Initial testing of the Sharps quarry material indicated that a B-value (Skempton, 1954) of at least 0.85 could be achieved quickly at a backpressure of 414 kPa (60 psi) and a confining pressure of 448 kPa (65 psi). The backpressure was maintained at this level while the cell pressure was increased to 483 kPa (70 psi) and 552 kPa (80 psi) to create effective confining pressures of 34.5, 69, and 138 kPa (5, 10, and 20 psi) for the cyclic and first stage, second stage and third stages of testing, respectively.

Initial cyclic testing consisted of 300 readings at 0.0084 percent strain followed by 100 readings at 0.025 percent strain. The cyclic reading schedule was later refined to include more frequent readings to avoid an overshoot of loading and unloading before the commence and halt commands could be invoked. Ultimately the number of readings was increased to 600 at 0.0042 percent strain intervals. All of the staged testing consisted, in the order shown, of 40 readings at 0.025 percent strain, 40 readings at 0.1 percent strain, and 40 readings at 0.25 percent strain. The following data were collected at each reading: applied load, cell pressure, pore pressure, deformation, and strain, from which the specimen stresses $\sigma_1$, $\sigma_3$, and $\sigma_d$ were calculated.

3.7 Cyclic Triaxial Tests

Cyclic triaxial tests were performed on each specimen under consolidated-drained, CD, conditions. The effective confining pressure of 35 kPa (5 psi) was selected because it represents typical confining stress found in highway base course (Thornton
and Elliott, 1988). The strain rate used for loading the specimen was initially set at 60 percent/hour for a few samples but was later reduced to 30 percent/hour because the computer system could not reliably reverse cycles at the proper level of stress. A review of the testing output data (further discussed within Chapter 4) confirmed that both strain rates were sufficiently slow enough to prevent any buildup of excess pore water pressure during the loading phase.

Testing consisted of 40 stress-controlled loading and unloading cycles. For each cycle, loading of the specimen increased to, and was held at, a 21 kPa (3 psi) deviator stress level for a period of 30 seconds followed by unloading the specimen to a lower limit of 3.5 kPa (0.5 psi) deviator stress. This level was held for 30 seconds until the next loading-unloading cycle began. The low applied axial stress was intentionally selected to avoid a premature failure the specimen prior to the beginning of staged triaxial testing and to determine the modulus values of the test specimen at intermediate levels of strain.

The repeated load testing was conducted at strain rates that were much lower than those for typical resilient modulus, $M_R$, testing. As a result, the modulus values developed were expected to be lower than those from standard resilient modulus testing. The purpose of completing repeated load testing prior to the first stage of triaxial shearing was to develop some correlation between the modulus values from this testing and the more expensive and time consuming resilient modulus testing.

3.8 Shear Strength Testing

At the conclusion of cyclic testing multiple stage triaxial shear tests were performed on each test specimen. The first stage was completed at the conclusion of the
cyclic testing at a confining pressure of 35 kPa (5 psi), using the same strain rates of 60 percent/hour and later 30 percent/hour as discussed in section 3.6. The termination strain for the first stage of testing was set at 3 percent.

The second stage of testing was completed under an effective confining stress of 69 kPa (10 psi). The predetermined stage 2 termination strain was selected as 6 percent. However, as illustrated in Fig 3.6, testing was typically terminated when the real-time stress vs. strain plot indicated the specimen had yielded. Typically loading was terminated at each stage when a clear yield could be established to prevent premature sample destruction prior to completing the final stage testing.

Figure 3.6: Real-time stress vs. strain plot generated during staged triaxial testing.
The third and final stage of testing was completed at an effective confining stress of 138 kPa (20 psi) to a maximum incremental strain level of 4 percent (cumulative total strain for all stages did not exceed 10 percent). The real-time stress vs. strain plots indicated shear failure for a majority of the test specimens occurred prior to a cumulative strain of 10 percent. In these instances testing was halted at the indication of a clear shear failure. The selection of the effective confining pressures in the staged triaxial test was made so that the Mohr-Coulomb failure envelope could be adequately established. Three testing stages were selected to adequately define the Mohr-Coulomb failure envelope over the range of stresses expected under typical highway loading conditions.

3.9 Membrane Modulus Testing

The rubber membranes used to isolate the specimen in the triaxial chamber act as short hollow columns when they are confined on one side by the liquid confining pressure and on the other side by the sample and liquid backpressure. As a result, the modulus of elasticity for the rubber membranes was required to determine what portion of the applied deviator stress was acting on the three (3) confining membranes and what portion was actually acting on the test specimen.

To determine this value a 101.6 mm (4 in.) width of the 0.686 mm (0.027 in.)-thick membrane was cut for testing. The membrane was suspended from a 12.5 mm (No. 4) diameter rebar 406.4 mm (16 in.) in length. A second 12.5 mm (No. 4) rebar was placed inside the test specimen and weights were incrementally suspended from it. An ink line 101.6 mm (4 in.) in length was drawn on the membrane in the direction of loading which established the gage length, L. With each applied load the length between the two endpoints of the gage line was measured. The incremental strain, ΔL/L, was
determined with each addition of weight to the membrane. The stress ($\sigma$) at each increment was found by dividing the total weight by the membrane corrected cross-sectional area. Table 3.4 summarizes the data obtained from the modulus testing. A plot of stress divided by strain (modulus) vs. strain, as illustrated in Fig. 3.7 was generated from which an average membrane modulus was extracted for the range of strains used in the triaxial testing. The membrane modulus was multiplied by the strain at a particular stage of the test to determine the stress in the membranes under the associated load.

Table 3.4: Membrane modulus determination raw data.

<table>
<thead>
<tr>
<th>Reading</th>
<th>Weight, g</th>
<th>Length, mm</th>
<th>Width, mm</th>
<th>Corrected Area, mm$^2$</th>
<th>Thickness, mm</th>
<th>$\Delta$L, mm</th>
<th>$\Delta$L/$\sigma$, g/mm</th>
<th>$\sigma$, g/mm$^2$</th>
<th>$E$, g/mm$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>100</td>
<td>100</td>
<td>0.686</td>
<td>0</td>
<td>100</td>
<td>0.02</td>
<td>3.9</td>
<td>1953.9</td>
<td>-</td>
</tr>
<tr>
<td>1</td>
<td>248</td>
<td>102.0</td>
<td>98.0</td>
<td>0.686</td>
<td>2</td>
<td>0.02</td>
<td>3.7</td>
<td>188.1</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>362.5</td>
<td>103.0</td>
<td>97.0</td>
<td>0.687</td>
<td>3</td>
<td>0.03</td>
<td>5.4</td>
<td>186.9</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>477</td>
<td>104.0</td>
<td>96.5</td>
<td>0.684</td>
<td>4</td>
<td>0.04</td>
<td>7.2</td>
<td>188.0</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>591.5</td>
<td>104.5</td>
<td>96.0</td>
<td>0.684</td>
<td>4.5</td>
<td>0.04</td>
<td>9.0</td>
<td>203.2</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>706</td>
<td>106.0</td>
<td>95.5</td>
<td>0.678</td>
<td>6.0</td>
<td>0.06</td>
<td>10.9</td>
<td>192.7</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>820.5</td>
<td>108.0</td>
<td>95.0</td>
<td>0.669</td>
<td>8.0</td>
<td>0.07</td>
<td>12.9</td>
<td>174.4</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>935</td>
<td>109.0</td>
<td>94.5</td>
<td>0.666</td>
<td>9.0</td>
<td>0.08</td>
<td>14.9</td>
<td>179.9</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>1049.5</td>
<td>111.0</td>
<td>94.0</td>
<td>0.657</td>
<td>11.0</td>
<td>0.10</td>
<td>17.0</td>
<td>171.4</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>1164</td>
<td>112.0</td>
<td>93.5</td>
<td>0.656</td>
<td>12.0</td>
<td>0.11</td>
<td>19.0</td>
<td>177.4</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>1276.5</td>
<td>113.0</td>
<td>93.0</td>
<td>0.653</td>
<td>13.0</td>
<td>0.12</td>
<td>21.1</td>
<td>183.1</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>1393</td>
<td>115.0</td>
<td>92.0</td>
<td>0.648</td>
<td>15.0</td>
<td>0.13</td>
<td>23.4</td>
<td>179.0</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>1507.5</td>
<td>116.0</td>
<td>91.0</td>
<td>0.640</td>
<td>16.0</td>
<td>0.14</td>
<td>25.5</td>
<td>184.8</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>1622</td>
<td>118.0</td>
<td>90.5</td>
<td>0.642</td>
<td>18.0</td>
<td>0.15</td>
<td>27.9</td>
<td>182.9</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>1736.5</td>
<td>120.0</td>
<td>90.0</td>
<td>0.635</td>
<td>20.0</td>
<td>0.17</td>
<td>30.4</td>
<td>182.3</td>
<td>-</td>
</tr>
<tr>
<td>15</td>
<td>1851</td>
<td>121.0</td>
<td>89.0</td>
<td>0.637</td>
<td>21.0</td>
<td>0.17</td>
<td>32.6</td>
<td>186.1</td>
<td>-</td>
</tr>
<tr>
<td>16</td>
<td>1965.5</td>
<td>124.0</td>
<td>88.5</td>
<td>0.625</td>
<td>24.0</td>
<td>0.19</td>
<td>35.5</td>
<td>183.6</td>
<td>-</td>
</tr>
<tr>
<td>17</td>
<td>2080</td>
<td>126.0</td>
<td>88.0</td>
<td>0.619</td>
<td>26.0</td>
<td>0.21</td>
<td>38.2</td>
<td>185.1</td>
<td>-</td>
</tr>
<tr>
<td>18</td>
<td>2194.5</td>
<td>129.0</td>
<td>87.0</td>
<td>0.611</td>
<td>28.0</td>
<td>0.22</td>
<td>41.3</td>
<td>183.6</td>
<td>-</td>
</tr>
<tr>
<td>19</td>
<td>2309</td>
<td>131.0</td>
<td>86.0</td>
<td>0.609</td>
<td>31.0</td>
<td>0.24</td>
<td>44.1</td>
<td>186.3</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>2423.5</td>
<td>134.0</td>
<td>85.0</td>
<td>0.602</td>
<td>34.0</td>
<td>0.25</td>
<td>47.3</td>
<td>186.6</td>
<td>-</td>
</tr>
<tr>
<td>21</td>
<td>2536</td>
<td>134.0</td>
<td>85.0</td>
<td>0.599</td>
<td>36.0</td>
<td>0.26</td>
<td>49.6</td>
<td>189.3</td>
<td>-</td>
</tr>
<tr>
<td>22</td>
<td>2645</td>
<td>138.0</td>
<td>84.0</td>
<td>0.594</td>
<td>39.0</td>
<td>0.32</td>
<td>56.4</td>
<td>196.3</td>
<td>-</td>
</tr>
<tr>
<td>23</td>
<td>2754</td>
<td>140.0</td>
<td>83.0</td>
<td>0.589</td>
<td>42.0</td>
<td>0.37</td>
<td>64.0</td>
<td>199.6</td>
<td>-</td>
</tr>
<tr>
<td>24</td>
<td>2863</td>
<td>142.0</td>
<td>82.0</td>
<td>0.583</td>
<td>45.0</td>
<td>0.42</td>
<td>73.3</td>
<td>233.2</td>
<td>-</td>
</tr>
<tr>
<td>25</td>
<td>2974</td>
<td>144.0</td>
<td>81.0</td>
<td>0.570</td>
<td>48.0</td>
<td>0.49</td>
<td>84.0</td>
<td>265.8</td>
<td>-</td>
</tr>
<tr>
<td>26</td>
<td>3082</td>
<td>146.0</td>
<td>80.0</td>
<td>0.558</td>
<td>51.0</td>
<td>0.56</td>
<td>97.1</td>
<td>179.1</td>
<td>32.5</td>
</tr>
<tr>
<td>27</td>
<td>3190</td>
<td>148.0</td>
<td>79.0</td>
<td>0.546</td>
<td>54.0</td>
<td>0.65</td>
<td>111.0</td>
<td>248.8</td>
<td>38.6</td>
</tr>
<tr>
<td>28</td>
<td>3298</td>
<td>150.0</td>
<td>78.0</td>
<td>0.532</td>
<td>57.0</td>
<td>0.70</td>
<td>128.0</td>
<td>298.5</td>
<td>44.1</td>
</tr>
</tbody>
</table>

118
Figure 3.7 illustrates that the membrane modulus of $E = 185 \text{ g/mm}^2$ (0.41 lb/mm$^2$) remains relatively constant until strains of about 25 percent are achieved. Since the majority of all staged triaxial testing was terminated by a maximum cumulative strain, $\varepsilon$, of under 10 percent, it was reasonable to assume a constant value for modulus over the entire strain range of a triaxial test.

Three membranes were used for each test specimen. The inner membrane closest to the sample material was punctured frequently by the aggregate during compaction process; thus its ability to isolate the sample from the confining fluid was compromised. The middle membrane and outer membranes were added to insure that the specimen would be totally isolated from the confining water. Equation 3.1 was used to determine the total membrane cross-sectional area:
\[ A = \pi nDt \]  \hspace{1cm} (3.1)

where \( A \) = area of all membranes

\( n \) = number of membranes

\( D \) = specimen diameter

\( t \) = individual membrane thickness.

The total area for the three membranes was determined to be 985 mm\(^2\) (1.53 in\(^2\)). As a result the load to be subtracted from the corrected load cell reading is given by Eqn. 3.2.

\[ P_{\text{membrane}} = E\varepsilon A \]  \hspace{1cm} (3.2)

where \( E \) = membrane modulus

\( \varepsilon \) = maximum cumulative strain during testing

\( A \) = total corrected membrane areas.

The load carried by the membranes during deviator stress application varies with strain. Considering the maximum cumulative strain during staged triaxial testing rarely exceeded 8 percent, the maximum load applied to the membranes would be less than 143.7 N (32.3 lbs). When one compares this to the axial load applied to the specimen it can be concluded that the membrane load is low when compared to the total applied loads and as a result would have little affect on the Mohr-Coulomb failure analyses if it were ignored.

3.10 Verification of Consolidated-Drained (CD) Test Conditions

Most reference texts on soil mechanics indicate that no cohesion should exist for granular materials, and they are typically represented in graphical form with a cohesion intercept value of 0. This concept was not verified in this study. While the Mohr-Coulomb failure envelope for cohesionless materials is expected to pass through the
origin, indicative of a cohesion intercept value of 0, each of the Mohr-Coulomb failure envelopes generated from the test data had a finite value for cohesion. Hvorslev (1960) believed that the Mohr-Coulomb failure envelope was actually curved with the envelope being steeper at low stresses and flatter at high stresses. The failure envelope was steeper at low stresses due to the forced intersection with the origin. Testing conducted by Olson (2004) contradicts this concept. Olson’s work with drained tests on kaolinite shows a cohesion intercept even at low stresses. It is thus concluded that the cohesion intercept values observed during testing were valid and the true measurement of shear strength should include cohesion for these granular materials.

Since the findings of this study are contrary to many reported, the possibility that the test specimens were not tested in a totally drained condition was examined. The coefficient of consolidation, \( c_v \), was determined from the initial consolidation stage prior to triaxial shear testing. Consolidation times were taken conservatively as 10 seconds based on the cessation of water level changes within the cell and drain accumulators after application of a confining pressure. A \( c_v \) value of 1695.6 ft\(^2\)/day was calculated for the specimens using Eqn. 3.3. Equation 3.4 was then used to calculate a time to failure at an average degree of consolidation equal to 95 percent considering the specimen drainage conditions, sample height, coefficient of consolidation, and time to shear failure (Gibson and Henkel, 1954). Discussion of this method of analysis has been previously presented in Chapter 2.

\[
c_v = \frac{\pi H^2}{4t_{100}} \tag{3.3}
\]

where \( c_v = \) coefficient of consolidation

\( H = \) specimen height

121
\[ t_{100} = \text{time at which 100 percent consolidation is achieved.} \]

\[ U_f = 1 - \frac{H_d^2}{\eta C_v t_f} \quad (3.4) \]

where \( U_f \) = target degree of consolidation

\[ H_d = \text{sample drainage height} \]

\[ \eta = \text{drainage condition parameter} \]

\[ C_v = \text{calculated coefficient of consolidation} \]

\[ t_f = \text{time to shear failure.} \]

Table 3.5 summarizes the \( \eta \) parameter values based upon drainage conditions. The target degree of consolidation is established as 95 percent for this study and the minimum time to failure, \( t_f \), was calculated to be 1.42 minutes by rearranging Eqn. 3.3.

Table 3.5: Summary of \( \eta \) parameter values for time to failure analysis.

<table>
<thead>
<tr>
<th>Drainage Condition</th>
<th>( \eta )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage from one end only</td>
<td>0.75</td>
</tr>
<tr>
<td>Drainage from both ends</td>
<td>3.0</td>
</tr>
<tr>
<td>Radial drainage only</td>
<td>32.0</td>
</tr>
<tr>
<td>Drainage radially and out one end</td>
<td>35.8</td>
</tr>
<tr>
<td>Drainage radially and out both ends</td>
<td>40.4</td>
</tr>
</tbody>
</table>

All of the tests were conducted at strain rates that gave times to failure in excess of the minimum 1.42 minutes. No excess pore water pressures were developed during the staged or cyclic testing; the CD triaxial testing conditions were present throughout. Results are summarized in Chapter 4.
The testing methods described were repeated for each of the quarry materials. In total 88 independent tests were conducted on samples at varying fines contents from the five quarries under investigation in an attempt to quantify the affect of fines on the shear strength and stiffness of the crushed stone base materials.

3.11 Hydraulic Conductivity Testing

Samples containing 6 percent and 8 percent fines were tested by the constant head method in accordance with AASHTO T-215 (ASTM, D2434). The AASHTO specification used in this study is identical to the ASTM specification, except that the sieve analysis for the test according to AASHTO shall conform to T-88. The ASTM specification requires that the sieve analysis conform to D-422.

Samples containing 10 percent, 12 percent, 14 percent, and 16 percent fines were tested by the falling head –rising tailwater method in accordance with ASTM D5084, method C. The ASTM procedure was followed for falling head tests because no AASHTO specification exists for a falling head test.

3.11.1 Equipment for Constant Head Hydraulic Conductivity Test

In this study a constant head permeameter, shown in Fig. 3.8, was used to determine values of hydraulic conductivity for the lower bound and the model blends at 6 and 8 percent fines.

Item # 1 in Fig. 3.8 denotes the 19 mm O.D. diameter bubble tube used to maintain a constant head on the sample being tested. The bubble tube can be moved vertically by loosening the lock nut at the top of the reservoir to allow adjustment of the gradient used in the test. According to AASHTO 215 (D 2434) the suggested values of the gradient range from 0.3 to 0.5 for densely compacted material.
Item # 2 denotes the 116 mm diameter reservoir that supplies the water used in the test. A graduated scale on the side of the reservoir allows the researcher to record the change in water level and hence the volume of flow during the test.

Item # 3 denotes the inflow line from the water supply tub. This hose is plugged when a hydraulic conductivity test is being run.

Item # 4 denotes the 5 gallon overflow bucket which contains the sample and the permeameter base assembly. A steel proctor mold, which is not visible in the figure, contains the 116 mm tall by 152 mm diameter compacted specimen being tested. Perforated metal plates, # 100 screens, and filter paper are located at the top and bottom of the specimen. The metal plates confine the specimen in the vertical direction while the screens and filter paper prevent the migration of the fines when water flows through the specimen.

Item # 5 denotes the vacuum line that is used when saturating the specimen and filling the water supply reservoir.

3.11.2 Constant Head Hydraulic Conductivity Test Procedure

A schematic of the device shown in Fig. 3.8 is illustrated in Fig. 3.9 which portrays the relative position of the sample in the device and the head supplied to the device through the Marriotte reservoir. The adjustable bubble tube controls the amount of head applied to the test specimen. Prior to starting the test the bubble tube is plugged and water is sucked into the reservoir from the supply tub using a vacuum source. The residual vacuum pressure in the top of the reservoir is equal and opposite to the pressure head created by the column of water suspended above the specimen. When the test is started the bubble tube is unplugged exposing the reservoir to atmospheric pressure at the bubble tube tip. At this point bubbles will enter the reservoir and reduce the vacuum
and cause the pressure head on the test specimen to be equal the distance from the tip of the bubble tube to the water level in the 5-gallon bucket. During the test, water flows from the main.

Figure 3.8 – Constant head permeameter used in study.
7) Determination of hydraulic conductivity using Darcy's Law as shown in Equation 2.14

\[ Q = A \times k \times i \times t \] (2.14)

Where gradient is:

\[ i = \frac{H}{L} \] (2.15)

and:

- \( H \) equals distance from bottom of bubble tube to overflow water elevation
- \( L \) = height of the specimen

Equation (2.14) can be rearranged as

\[ Q = A \times k \times \frac{H}{L} \times t \]

And hydraulic conductivity is calculated as follows:

\[ k = \frac{Q}{i \times A_s \times t} \] (3.8)

where:

- \( k \) = hydraulic conductivity, cm/s
- \( Q \) = change in water elevation in reservoir multiplied by net area of reservoir (\( A_r \)), cm³
- \( i \) = hydraulic gradient
- \( A_s \) = cross sectional area of specimen, cm²
- \( A_r \) = inside cross sectional area of reservoir minus the outside cross sectional area of the bubble tube, cm²
- \( t \) = time of test, sec
Figure 3.9 – Typical constant head test set-up using the principle of Marriotte’s bottle to apply head.

**Example Hydraulic Conductivity Calculation**

A sample from the Sharp’s quarry, which is designated as MB8-2, was tested on 7/13/02. The hand calculations as well as the spreadsheet calculation are shown to illustrate the procedure for calculating hydraulic conductivity.

In this case, the reservoir reading went from 52.8 cm to 50.3 cm in 100 minutes. The equation used to calculate hydraulic conductivity is:

\[
k = \frac{Q}{i \times A_s \times t}
\]  

(3.8)

in this case:

\[
A_R = \left[ \pi \times \left( \frac{5.96}{2} \right)^2 - \pi \times \left( \frac{749}{2} \right)^2 \right] \times \left[ 2.54 \text{ cm} \right]^2 = 177.15 \text{ cm}^2
\]

\[
A_s = \left[ \pi \times \left( \frac{6}{2} \right)^2 \right] \times \left[ 2.54 \text{ cm} \right]^2 = 182.41 \text{ cm}^2
\]

\[
Q = 2.5 \text{ cm} \times 177.15 \text{ cm}^2 = 442.87 \text{ cm}^3
\]
\[ i = \frac{1.37 \text{ in}}{4.625 \text{ in}} = .2962 \]

therefore,

\[ k = \frac{442.87 \text{ cm}^3}{.2962 \times 182.41 \text{ cm}^2 \times 100 \text{ min} \times 60 \text{ sec} / \text{min}} \]

\[ k = .00137 \text{ cm/sec} \]

Fig. 3.10 presents a data sheet and the total flow versus time regression line used to determine hydraulic conductivity of the same sample MB8-2 from Sharp's quarry. For values reported in this study, the slope of the regression line for total flow versus time was used to determine hydraulic conductivity based on 10 minute reading intervals rather than the 100 minute interval used in the above example. As a result, the hand calculated value differs slightly from the spreadsheet value.
Sample - MB-8(2)
Sharp's quarry, MB-8
Date-7/13/02

\[ y = 4.75x \]
\[ R^2 = 0.996 \]

\[ A_e = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

\[ L (\text{inches}) = 4.625 \]
\[ H (\text{inches}) = 1.370 \]
\[ I (\text{gradient}) = 0.2962 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>52.80</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.50</td>
<td>0.30</td>
<td>53.14</td>
<td>53.14</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
<tr>
<td>20.0</td>
<td>52.20</td>
<td>0.30</td>
<td>53.14</td>
<td>106.29</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
<tr>
<td>30.0</td>
<td>52.00</td>
<td>0.20</td>
<td>35.43</td>
<td>141.72</td>
<td>0.06</td>
<td>0.0011</td>
</tr>
<tr>
<td>40.0</td>
<td>51.70</td>
<td>0.30</td>
<td>53.14</td>
<td>194.86</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
<tr>
<td>50.0</td>
<td>51.50</td>
<td>0.20</td>
<td>35.43</td>
<td>230.30</td>
<td>0.06</td>
<td>0.0011</td>
</tr>
<tr>
<td>60.0</td>
<td>51.20</td>
<td>0.30</td>
<td>53.14</td>
<td>283.44</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
</tbody>
</table>

\[ *-k \text{ cm/sec based on Total Flow Regression} = 1.43E-03 \text{ cm/sec} \]

Figure 3.10 – Data sheet and total flow v. time regression line used to determine hydraulic conductivity of MB8-2 class 7 base course from Sharp’s quarry.
3.11.3 Equipment for Falling Head – Rising Tail Hydraulic Conductivity Test

In this study the falling head permeameter, shown in Fig. 3.11, was used to determine values of hydraulic conductivity for the upper bound and 10, 12, 14, and 16 percent model blends. The figure shows a falling head – rising tail permeability test set-up for two (2) samples from Sharp’s quarry.

Item # 1 in Figure 3.11 denotes the permeameter water supply which flows to the pressure panel via a siphon. Valves on the pressure panel are used to mechanically route the permeant to the appropriate burette.

Item # 2 in the figure denotes the Wykeham Farrance WF 13100 pressure panel used to measure water flow and to apply an inflow head, back pressure head, and confining pressure to the sample tested. The appropriate valves on the pressure panel are opened to fill the vacuum burette (outflow), backpressure (inflow), and chamber (cell) burette with permeant. Direct measurement of the quantity of water supplied to the specimen and the permeameter cell is accomplished by using the graduated scale on the burettes. The pressure panel is also used to apply the appropriate vacuum, backpressure, and cell pressures via an “air over water” arrangement.

Item # 3 in the figure denotes the standard flexible wall permeability cell used to route permeant through the specimen and apply confining pressure. Use of the flexible wall permeability cell minimizes side wall leakage (a problem with the rigid wall permeameter) and allows measurements of the low flows common with samples of low hydraulic conductivities (less than approximately $10^{-4}$ cm/s).

The equipment also allows for back pressure saturation which is normally required to saturate the specimen. Back pressure saturation is used to dissolve any air bubbles inside
the sample and fill all the pore spaces with permeant. For this study back pressure
saturation with a 450 kPa (65 psi) confining pressure and a 380 kPa (55 psi) pore
pressure was used to produce an effective stress of 70 kPa (10 psi) on the test specimen.
Since hydraulic conductivity decreases as the effective stress increases, the effective
stress should be minimized to replicate field conditions.
3.11.4 Falling Head-Rising Tail Hydraulic Conductivity Test Procedure

A schematic of the falling head permeameter used in the study is illustrated in Fig. 3.11 – Falling head – rising tail tests underway to determine the hydraulic conductivity of two (2) samples from Sharp’s quarry.
3.12 which portrays the relative position of the sample in the device and plumbing from the pressure panel.

Figure 3.12 – Schematic diagram of flexible wall permeameter cell used in study.

The inflow water from the backpressure burette flows into the base plate of the permeameter cell and upwards through the specimen. As the water exits the specimen it collects in grooves in the top cap and exits the permeameter via the two (2) tubes from the top cap. The vacuum burette (outflow) is connected to these tubes from the top cap meaning that permeant flow is through the sample from bottom to top.

The cell is initially filled with water and a 5 psi confining pressure is placed on the sample via the chamber burette. Water is forced through the sample from the back pressure burette under a slight head (1-3 psi) until the amount entering the sample equals the amount exiting the sample. This condition exists when the decrease of water in the back pressure burette is equal to the increase in water in the vacuum burette. When this condition is attained back-pressure saturation can begin to insure saturation of the
Determining the level of saturation for specimens subjected to triaxial testing is of critical importance. Skempton proposed the concept of the pore-pressure coefficients (B value) in his 1954 paper. The equipment used in this test permitted the determination of this B value.

The B value is defined as the ratio of the pore water pressure increase resulting from an increase in confining pressure to the change in confining pressure when no drainage is allowed. For saturated soft soils like clays, the B parameter should equal 1. Skempton stated that in a saturated soil (zero air voids) water is infinitely more incompressible than the soil solids matrix. The degree to which water is more incompressible than the solids structure is seen to be somewhat decreased for more granular materials. In essence, the change in stress applied to a granular specimen will be "carried" to varying amounts by both pore water pressure and stiffness of the solids skeleton.

This concept is reinforced by Skempton's (1954) own findings. A plot of B value vs. degree of saturation for a clay gravel soil showed that at approximately 95 percent saturation the B values obtained are approximately 85 percent. It is interesting to note that the clay gravel material tested has a maximum dry density of 136 lbs/ft$^3$ and an optimum water content of 7.8 percent. The liquid limit and plasticity index were 17 and 2, respectively. Since these values are very similar to those obtained for the crushed stone base course aggregate tested in this study, a target B-value of 0.85 or greater was chosen for this study.

A complete step by step pictorial illustrating the assembly of the device and test
procedure used is presented in Appendix B.2. In general terms, the sequence consisted of:

1) Preparation of the blend in accordance with the percentage of fines being tested
2) Compaction of the material in a flexible membrane using a split mold
3) Assembly of the flexible wall permeability cell
4) Attachment of the lines from the pressure panel which control vacuum pressure, backpressure, and cell pressure
5) Subjecting the sample to backpressure saturation
6) Obtaining an appropriate B value and running the falling head-rising tail hydraulic conductivity test.
7) Determination of hydraulic conductivity using Equation 3.4

\[
k = \left( \frac{a_{in} \times a_{out} \times L}{(a_{in} + a_{out}) \times A_s \times \Delta t} \right) \times \ln \left( \frac{h_1}{h_2} \right)
\]

(3.9)

where:

\[ k = \text{hydraulic conductivity, cm/s} \]
\[ a_{in} = \text{cross sectional area of inflow burette, cm}^2 \]
\[ a_{out} = \text{cross sectional area of outflow burette, cm}^2 \]
\[ L = \text{length of specimen, cm} \]
\[ A_s = \text{cross sectional area of specimen, cm}^2 \]
\[ \Delta t = \text{interval of time over which flow occurs, sec} \]
\[ h_1 = \text{head difference at start of test, cm} \]
\[ h_2 = \text{head difference at end of test, cm} \]

Example Hydraulic Conductivity Calculation
Sharp’s quarry sample MB10-1 was tested on 8/8/02. The recorded data was $h_1 = 100$, $h_2 = 37.6$, $\Delta t = 50$ min. The burettes used in this study were graduated such that 1 cm of the burette’s length contained 1.675 ml of permeant.

\[
k = \left( \frac{a_{in} \times a_{out} \times L}{(a_{in} + a_{out}) \times A_s \times \Delta t} \right) \times \ln \left( \frac{h_1}{h_2} \right)
\]

(3.9)

where:

\[
a_{in} = a_{out} = 1.67 \text{ cm}^2
\]

\[
A_s = \left[ \pi \times \left( \frac{6}{2} \right)^2 \right] \times \left[ \frac{2.54 \text{ cm}}{\text{in}} \right]^2 = 182.41 \text{ cm}^2
\]

\[
L = 4 \text{ in} \times 2.54 \text{ cm/in} = 10.16 \text{ cm}
\]

Solving for $k$:

\[
k = \left( \frac{1.67 \text{ cm}^2 \times 1.67 \text{ cm}^2 \times 10.16 \text{ cm}}{(1.67 \text{ cm} + 1.67 \text{ cm}) \times 182.41 \text{ cm}^2 \times 50 \text{ min} \times 60 \text{ sec/min}} \right) \times \ln \left( \frac{100 \text{ ml}}{37.6 \text{ ml}} \right)
\]

\[= .00001516 \text{ cm/sec (} .0000152 \text{ as per spreadsheet)}
\]

Fig 3.13 presents the data sheet used to determine the hydraulic conductivity of MB10-1 class 7 base course from Sharp’s quarry. A calculator or computer program must be available on site to solve Eq. (3.8) as the test is being run. The specimen should be permeated until at least four (4) values of hydraulic conductivity are obtained over an interval of time in which: the ratio of the inflow to outflow is between 0.75 and 1.25 and the hydraulic conductivity is steady. In this study, the ratio of inflow to outflow was approximately 1.0 for all tests. Steady state flow conditions were assumed when four (4) or more values of hydraulic conductivity were within 50% of each other and a plot of
hydraulic conductivity versus time showed no significant upward or downward trend.

3.12 Suction Testing

Two (2) replicate samples of the model gradations with 6, 8, 10, 12, 14, and 16 percent fines were created. Each sample was compacted at optimum moisture to 98 percent of maximum dry density as determined by the modified proctor using a Marshall compaction hammer. Standard plastic concrete cylinder molds (152.4 mm diameter by 304.8 mm tall) were used to contain the samples used in the test. In addition to the model blends replicate specimens of the lower bound gradation with 6 percent fines and the upper bound gradation with 10 percent fines were created to bring the total number of tube suction test specimens to fourteen (14) per quarry.

<table>
<thead>
<tr>
<th>Sample</th>
<th>MB-10(1)</th>
<th>(a_n=a_{out}=1.67\text{ cm}^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharp's quarry</td>
<td></td>
<td>(A_s = 182.41 \text{ cm}^2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(L = 10.16 \text{ cm})</td>
</tr>
</tbody>
</table>

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/5/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/5/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/8/02</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.5/5 = .90</td>
<td>50.00</td>
<td>68.8</td>
<td>31.2</td>
<td>0.0</td>
<td>37.6</td>
<td>1.52E-05</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.13 – Data sheet used to determine the hydraulic conductivity of MB10-1 class 7 base course from Sharp’s quarry.
To allow water to enter the sample, 12-6.5 mm diameter holes were drilled on approximately 38 mm centers around the base of the mold. The inlet holes were approximately 25 mm from the base of the molds.

The samples were placed in tubs which contained approximately 50 mm of water for soaking. The samples were removed and weighed at intervals of 1 hr, 2 hrs, 4 hrs, 24 hrs, 48 hrs, 96 hrs, 120 hrs, 144 hrs, 168 hrs, 192 hrs, 216 hrs, and 240 hrs. A graph relating the percent of moisture gain versus the square root of time was plotted for each sample blend. Figure 3.14 shows the general setup of the suction test.

![Figure 3.14- Suction test samples from Sharp’s quarry in water bath.](image)

Due to problems with testing which will be described in more detail in Chapter 4, the test procedure was modified throughout the study to assure more uniform conditions.
The test procedure used for Glen Rose and Granite Mountain quarries is described below.

**Test procedure (Glen Rose and Granite Mountain quarries)**

1. The quantity of material to be hand blended was selected based on the unit weight and optimum moisture determined by the modified proctor which would achieve the target dry density for the volume of the test specimen.

2. A 40 mm thick layer of material retained on the # 40 sieve was placed at the base of the mold to serve as a filter to prevent fines from clogging the inlet holes during soaking and to prevent fines from being washed from the sample when it was removed for weighing.

3. The material was compacted into the mold to 98 percent of maximum dry density. For this study a Marshall hammer was used to compact the material in 8 lifts with 25 blows /lift to obtain target density. The samples were oven dried at 100° C. for 48 hours.

4. The samples were then placed into the water soak tub. The water soak tub was plumbed so that a steady flow of water entered the tub with a maximum depth of approximately 50 mm of water.

5. The samples were removed at intervals of 1 hr, 2 hrs, 4 hrs, 24 hrs, 48 hrs, 96 hrs, 120 hrs, 144 hrs, 168 hrs, 192 hrs, 216 hrs, and 240 hrs and the weights recorded.

6. The percentage moisture gain with respect to the square root of time was plotted on an Excel spread sheet.
Chapter 4

RESULTS AND DISCUSSION

4.1 Material Properties

The maximum dry density and optimum moisture content for the model gradation from each quarry was determined by running a modified proctor test (AASHTO T-180) on both the upper and lower gradation limits. To determine the optimum moisture and maximum dry density of the model gradation, an average or “best fit” line was drawn between the two curves generated by the modified proctor tests on the upper and lower gradation limits.

This procedure allowed a target density to be established with only ten (10) modified proctor tests for the entire study. If a unique target density were established for each model gradation over forty (40) modified proctor tests would be required and the variability between tests could introduce additional uncertainty into what variables were actually affecting changes in hydraulic conductivity. By using this averaging procedure only one target density was established for each quarry and all specimens were compacted to the same unit weight.

The compaction curves for the upper and lower gradation limits along with the line of zero air voids for Sharp’s quarry are presented in Fig. 4.1. An optimum moisture content of 5.5 percent and maximum dry unit weight of 141.0 lbs/ft$^3$ were selected as the target values for the model gradation during specimen preparation of the Black Rock material for all hydraulic conductivity and suction testing. The compaction curves for Class 7 base material from the remaining quarries are presented in Appendix A.4.
The effective particle sizes of material passing the .075 mm (# 200) sieve for class 7 base course was determined using a hydrometer analysis in accordance with AASHTO T-88. This data was needed to determine \( D_{10}, D_{30}, \) and \( D_{60} \) which are required to compute the coefficient of uniformity and the coefficient of gradation. In addition, \( D_{10} \) and \( D_{15} \) were used in formulas presented later in this chapter to estimate the hydraulic conductivity of the model gradations for various fines contents.

The grain size distribution curve for the “as received” material from Black Rock quarry is shown in Fig. 4.2, while the grain size distribution curves for the remaining quarries are presented in A.5.
Figure 4.2 – Grain size distribution curve for material from the Black Rock quarry, Lawrence County, Arkansas

Index properties were established for the “as received” gradation of the various class 7 base courses upon their arrival at ERC. A summary of these index properties and the resulting classification of the materials are presented in Table 4.1. To determine the plasticity index of the various materials, AASHTO T87 was followed for dry preparation and AASHTO T146 was followed for wet preparation of the materials. As expected, with the exception of material from the Preston quarry, the plasticity index was greater when using wet preparation techniques as opposed to dry preparation techniques. It was also noted that the material from Granite Mountain quarry went from passing AHTD standards for the dry preparation procedure to failing those standards when the wet preparation procedure was used.
Table 4.1 - Index properties and the resulting classification of “as received” class 7 base course from the various quarries

<table>
<thead>
<tr>
<th>Quarry</th>
<th>Maximum Dry Density (pcf)</th>
<th>Optimum Moisture Content (%)</th>
<th>Apparent Specific Gravity</th>
<th>Bulk Specific Gravity (Historical)</th>
<th>Absorption (%)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharps</td>
<td>141.00</td>
<td>5.5</td>
<td>2.68</td>
<td>2.505</td>
<td>1.80</td>
<td>16</td>
<td>12</td>
<td>4</td>
<td>NP</td>
<td>NP</td>
<td>NA</td>
</tr>
<tr>
<td>Preston</td>
<td>132.00</td>
<td>7.5</td>
<td>2.63</td>
<td>2.509</td>
<td>2.20</td>
<td>NP</td>
<td>NP</td>
<td>NP</td>
<td>NP</td>
<td>NP</td>
<td>NA</td>
</tr>
<tr>
<td>Black Rock</td>
<td>144.00</td>
<td>6.5</td>
<td>2.83</td>
<td>2.750</td>
<td>0.80</td>
<td>16</td>
<td>13</td>
<td>3</td>
<td>NP</td>
<td>NP</td>
<td>NA</td>
</tr>
<tr>
<td>Glen Rose</td>
<td>135.00</td>
<td>6.0</td>
<td>2.61</td>
<td>2.568</td>
<td>0.70</td>
<td>16</td>
<td>11</td>
<td>5</td>
<td>16</td>
<td>11</td>
<td>5</td>
</tr>
<tr>
<td>Granite Mountain</td>
<td>134.20</td>
<td>7.5</td>
<td>2.64</td>
<td>2.619</td>
<td>0.80</td>
<td>28</td>
<td>21</td>
<td>8</td>
<td>26</td>
<td>22</td>
<td>4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Quarry</th>
<th>$D_{50}$ (mm)</th>
<th>$D_{90}$ (mm)</th>
<th>$D_{10}$ (mm)</th>
<th>$C_u$</th>
<th>$C_c$</th>
<th>% passing # 10 sieve</th>
<th>% passing # 40 sieve</th>
<th>% passing # 200 sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharps</td>
<td>0.095</td>
<td>1.70</td>
<td>13.00</td>
<td>136.84</td>
<td>2.34</td>
<td>23.00</td>
<td>15.00</td>
<td>7.50</td>
</tr>
<tr>
<td>Preston</td>
<td>0.095</td>
<td>2.40</td>
<td>11.00</td>
<td>115.79</td>
<td>5.51</td>
<td>27.20</td>
<td>11.00</td>
<td>6.00</td>
</tr>
<tr>
<td>Black Rock</td>
<td>0.110</td>
<td>2.50</td>
<td>13.00</td>
<td>118.18</td>
<td>4.37</td>
<td>24.00</td>
<td>15.00</td>
<td>9.30</td>
</tr>
<tr>
<td>Glen Rose</td>
<td>0.160</td>
<td>1.10</td>
<td>9.20</td>
<td>57.50</td>
<td>0.82</td>
<td>30.00</td>
<td>20.00</td>
<td>8.00</td>
</tr>
<tr>
<td>Granite Mountain</td>
<td>0.100</td>
<td>1.50</td>
<td>9.00</td>
<td>90.00</td>
<td>2.50</td>
<td>35.00</td>
<td>12.50</td>
<td>4.10</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Quarry</th>
<th>Dust Ratio (1)</th>
<th>AASHTO (2)</th>
<th>Unifled (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharps</td>
<td>0.50</td>
<td>A-1-a</td>
<td>GW-GM</td>
</tr>
<tr>
<td>Preston</td>
<td>0.55</td>
<td>A-1-a</td>
<td>GP-GM</td>
</tr>
<tr>
<td>Black Rock</td>
<td>0.62</td>
<td>A-1-a</td>
<td>GP-GM</td>
</tr>
<tr>
<td>Glen Rose</td>
<td>0.40</td>
<td>A-1-a</td>
<td>GP-GM</td>
</tr>
<tr>
<td>Granite Mountain</td>
<td>0.33</td>
<td>A-1-a</td>
<td>GW-GM</td>
</tr>
</tbody>
</table>

1- Dust Ratio- % passing # 200 sieve divided by the % passing # 40 sieve shall not be be greater than 0.66 (AHTD Class 7 base course specification)
2- Classification based on Atterberg Limits from dry preparation
4.2 Hydraulic Conductivity

Samples containing 6 and 8 percent fines were tested by the constant head method in accordance with AASHTO T-215 (ASTM, D2434). Samples containing 10, 12, 14, and 16 percent fines were tested by the falling head method in accordance with ASTM-D5084, method C.

The data in Fig. 4.3 shows the results of measured hydraulic conductivities of the model and boundary blends for the Granite Mountain quarry and illustrates the concept of decreasing hydraulic conductivity with increasing fines content. The agreement in measured hydraulic conductivities for each replicate gradation is exceptionally good. These results suggest that the strongest contributor to reductions in hydraulic conductivity is the percentage of fines. One would expect the upper limit gradations (finer) to have a lower value of k than the model blend and the lower limit gradations (coarser) to have a higher value of k than the model gradations, yet the data suggest that the hydraulic conductivities for the model blends correspond almost exactly to the upper and lower bound blends having the same percent fines.

It is also important to note that the hydraulic conductivity values for the falling head tests conducted on 6 and 8 percent fine gradations were lower than constant head tests for the same gradations. This condition was most likely the result of better control over sidewall leakage and the lower system permeability of the falling head permeameter. The fact that the falling head test result for 8 percent fines is greater than that for 6 percent fines appears to be an anomaly. Based on the data shown in Table 4.4, one would expect that the difference in hydraulic conductivities measured using the two methods to be to be about one order of magnitude due to differences in system losses between the two test
methods. Values of hydraulic conductivity from the constant head test are consistently about 1 order of magnitude more than those from the falling head tests for samples at 6 and 8 percent fines. As a result, the falling head hydraulic conductivity for the 6 percent fines model blend, shown in Fig. 4.3, should more reasonably be placed at around $2 \times 10^{-4}$ cm/sec rather than the $2 \times 10^{-5}$ cm/sec.

Figure 4.3 - Hydraulic conductivity versus percentage of fines for the model, upper, and lower limit gradations from Granite Mountain quarry.

A summary of the average value of hydraulic conductivity for replicate samples tested at each fines content of the model gradation for all quarries are presented in Table 4.2. The test data and calculations supporting the values shown in Table 4.2 are presented in Appendix C.
Table 4.2 – Summary of average hydraulic conductivity in cm/sec for Class 7 base course for the model gradations from all quarries.

<table>
<thead>
<tr>
<th>% FINES</th>
<th>Black Rock</th>
<th>Glen Rose</th>
<th>Granite Mtn</th>
<th>Preston</th>
<th>Sharp's</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>5.80E-03</td>
<td>3.88E-03</td>
<td>2.74E-03</td>
<td>2.91E-03</td>
<td>3.97E-03</td>
</tr>
<tr>
<td>8</td>
<td>7.92E-03</td>
<td>3.83E-03</td>
<td>1.21E-04</td>
<td>1.60E-03</td>
<td>1.89E-03</td>
</tr>
<tr>
<td>10</td>
<td>3.84E-05</td>
<td>5.75E-04</td>
<td>1.72E-06</td>
<td>9.40E-06</td>
<td>2.60E-05</td>
</tr>
<tr>
<td>12</td>
<td>5.48E-05</td>
<td>1.11E-04</td>
<td>6.55E-07</td>
<td>3.55E-06</td>
<td>1.14E-05</td>
</tr>
<tr>
<td>14</td>
<td>8.53E-06</td>
<td>1.25E-04</td>
<td>9.36E-07</td>
<td>2.96E-06</td>
<td>1.06E-05</td>
</tr>
<tr>
<td>16</td>
<td>7.12E-06</td>
<td>5.65E-06</td>
<td>9.78E-07</td>
<td>3.74E-06</td>
<td>5.05E-06</td>
</tr>
</tbody>
</table>

The average values of hydraulic conductivity for replicate samples tested at the upper and lower limit gradations for all quarries are presented in Table 4.3. The test data and calculations supporting the values shown in Table 4.3 are presented in Appendix A.6.

Table 4.3 – Summary of average hydraulic conductivity in cm/sec for Class 7 base course for the upper and lower limit gradations from all quarries.

<table>
<thead>
<tr>
<th>% FINES</th>
<th>Black Rock</th>
<th>Glen Rose</th>
<th>Granite Mtn</th>
<th>Preston</th>
<th>Sharp's</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>5.05E-03</td>
<td>3.03E-01</td>
<td>2.82E-03</td>
<td>2.41E-02</td>
<td>4.35E-02</td>
</tr>
<tr>
<td>10</td>
<td>3.81E-06</td>
<td>8.27E-05</td>
<td>9.04E-07</td>
<td>1.38E-05</td>
<td>2.20E-05</td>
</tr>
</tbody>
</table>

As seen in the tables above, the measured hydraulic conductivities for the material tested in this study were in the range of about $10^{-3}$ to $10^{-6}$ cm/sec. These values are in general agreement with earlier research conducted by others (Richardson, 1997; Hoppe, 1996; Persing, 1978). As anticipated, in most cases the hydraulic conductivities for the upper and lower limit gradations bracketed the hydraulic conductivity for the model gradations having the same percentage of fines.

A graphical representation of the data presented in Table 4.2 and Table 4.3 with a hand drawn trend line is presented in Fig. 4.4 to illustrate the decrease in hydraulic conductivity with respect to increasing fines content. In the figure, the average hydraulic
conductivity of replicate samples are plotted against the percentage of fines for the quarries studied and a hand drawn trend line was inserted. Between 6 and 10 percent fines, the hydraulic conductivity changes approximately 1 order of magnitude for each 1 percent change in fines. However, above fines contents of 10 percent the hydraulic conductivity changes by less than 1 order of magnitude for the entire 6 percent change in fines. This suggests that the decrease in hydraulic conductivity for fine contents greater than about 10 percent is relatively insignificant. In agreement with theory and earlier research, Fig. 4.4 clearly demonstrates that the hydraulic conductivity of granular base course decreases as the percentage of fines increases.

It was noted that material from Granite Mountain had consistently lower hydraulic conductivities than other quarries while material from Glen Rose had consistently higher hydraulic conductivities than the other quarries. In an effort to investigate this trend, the grain size distribution curves for these quarries were compared as shown in Fig 4.5. It should be kept in mind that these grain size distribution curves were developed by the wet sieving procedure and the material was handled multiple times prior to reaching ERC. As a result, it was expected that the percent of fines could be falsely elevated for the “as received” material. When comparing these two quarries it was found that material passing the # 200 sieve was 7.16 percent for Glen Rose quarry and 11.85 percent for Granite Mountain quarry.

An inspection of the data presented in Fig. 4.5, with emphasis placed on material larger than the # 40 sieve, reveals that the material from Granite Mountain quarry is somewhat “finer” than that from Glen Rose quarry. This finer material would result in smaller void space, and hence decreased hydraulic conductivity.
Figure 4.4 - Hydraulic conductivity versus percentage of fines for the model, upper, and lower limit gradations from all quarries
It is also significant to note that the material from the Glen Rose quarry contains river run fines rather than crusher fines. The river run fines are more rounded than the angular crusher fines from Granite Mountain quarry. The more rounded particles would tend to create larger void spaces between individual particles, which would result in an increased hydraulic conductivity of the Glen Rose material.

![Log Particle Size (mm) vs % Finer Graph]

Figure 4.5 Grain size distribution curves for Glen Rose and Granite Mountain quarries.

4.2.1 Problems Encountered During Hydraulic Conductivity Testing

When running the constant head permeability test with a rigid wall permeameter two problems were encountered that could account for some of the variation in measured hydraulic conductivity for replicate specimens. In general, these problems were not encountered when running the falling head permeability tests and as a result measured values were more repeatable with less data scatter for the falling head tests.

Table 4.4 shows the results of constant head permeability testing compared to
falling head permeability testing for 6 and 8 percent fines. The comparison is made to
demonstrate that the values in hydraulic conductivity measured between the two systems
is about one order of magnitude. The difference is attributed to differences in head loss
between the two testing systems. It should be noted that the differences in measured
hydraulic conductivity appear to be diminished as the percentage of fines increase.

Table 4.4 - Constant head permeability testing compared to falling head permeability
testing for 6 and 8 percent fines, all quarries

<table>
<thead>
<tr>
<th>Quarry</th>
<th>K (cm/sec, constant head)</th>
<th>K (cm/sec, falling head)</th>
<th>Difference</th>
<th>K (cm/sec, constant head)</th>
<th>K (cm/sec, falling head)</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black Rock</td>
<td>5.80E-03</td>
<td>2.55E-04</td>
<td>5.54E-03</td>
<td>7.92E-03</td>
<td>1.43E-04</td>
<td>7.77E-03</td>
</tr>
<tr>
<td>Glen Rose</td>
<td>3.88E-03</td>
<td>3.67E-04</td>
<td>3.51E-03</td>
<td>3.83E-03</td>
<td>4.07E-04</td>
<td>3.42E-03</td>
</tr>
<tr>
<td>Granite Mtn</td>
<td>2.74E-03</td>
<td>2.16E-05</td>
<td>2.71E-03</td>
<td>1.21E-04</td>
<td>2.85E-05</td>
<td>9.26E-05</td>
</tr>
<tr>
<td>Preston</td>
<td>2.91E-03</td>
<td>6.73E-05</td>
<td>2.84E-03</td>
<td>1.60E-03</td>
<td>1.87E-05</td>
<td>1.58E-03</td>
</tr>
<tr>
<td>Sharp's</td>
<td>3.97E-03</td>
<td>3.46E-04</td>
<td>3.62E-03</td>
<td>1.89E-03</td>
<td>2.75E-04</td>
<td>1.61E-03</td>
</tr>
</tbody>
</table>

Fig.4.6 presents values of hydraulic conductivity for model gradations from the
Black Rock quarry. The values of hydraulic conductivity for the 6 and 8 percent blends
have been adjusted downward by one order of magnitude to account for the greater
system loss of the falling head tests as compared to the constant head tests and allow a
direct comparison of hydraulic conductivities across the entire range of fines contents.
This figure suggests that the hydraulic conductivities of densely graded crushed stone
base that have fines contents ranging from 6 to 12 percent have less than one order of
magnitude change in hydraulic conductivity.

The data scatter at the 6 percent fines content illustrated in Fig 4.6, as well as the
failure of the constant head tests at 6 and 8 percent fines to conform to the principle of
decreasing hydraulic conductivity with increasing fines content, illustrates the main problems encountered in the constant head permeability tests. It was thought that these problems were a result of “short circuiting” and incomplete saturation of the sample.

Figure 4.6 – Adjusted values of hydraulic conductivity for model gradations from the Black Rock quarry.

The detection of obvious “short circuiting”, which is defined as water flowing in the interconnected void spaces between the sample being tested and the rigid permeameter wall, caused numerous samples to be discarded. This condition was signaled by a bubble rate in the Mariotte tube being faster than approximately 1 bubble every 10 seconds or by leakage of the permeameter at the rubber collars. It should be kept in mind that a minor amount of sidewall leakage (which could not be detected) would also cause k values to be unrealistically high.
To detect minor sidewall leakage the researcher can compare the observed k value to the k value determined in the previous test to see if the value obtained is realistic. As a general rule, it was assumed that the samples were not “short circuiting” if any two values of hydraulic conductivity for a specific fines content were within an order of magnitude of one another. If a divergence greater than one order of magnitude from the previous test was measured, the test was repeated using a new sample and a new k value was measured. If this new value was within an order of magnitude of one of the previously measured values, the two closest values were reported as the measured hydraulic conductivities for the study.

The second issue was the degree of saturation of the tested specimen. The assumption in the constant head test method for this study is that saturation can be obtained by soaking the specimen. Unfortunately there is no guarantee that all of the air has been removed from the sample’s void spaces by soaking. Unlike flexible wall permeameters where a B value can be calculated, no method of checking the degree of saturation was available for the rigid wall permeameter used in this study. Air bubbles remaining in the sample as a result of incomplete saturation will block the flow of water through a portion of the interconnected void spaces and cause hydraulic conductivity values to be unrealistically low.

Based on earlier research (Bowders, 2003; Richardson, 1997; Hoppe, 1996; Persing, 1978), the expected range of hydraulic conductivities for densely graded base course using the constant head method was from $10^{-3}$ to $10^{-4}$ cm/sec. Observed hydraulic conductivities outside this range were suspect and the test was re-run to verify the results. The reported hydraulic conductivities for this study fell within the expected
range after re-testing except for one sample at 8 percent fines content from Granite Mountain quarry. Since the material from the Granite Mountain quarry had the lowest hydraulic conductivities of all quarries tested, its relatively low measured value at the 8 percent fines content was believed to be accurate.

The major problem encountered while running the falling head hydraulic conductivity tests was caused by an inability to apply a constant vacuum to the test panels. As a result, air bubbles in the test panel plumbing would cause an “air lock” by blocking the tubing and inlet ports which, in turn, prevented permeant flow. This condition affected both the saturation and permeation phases and caused numerous samples to be discarded when permeant flow through the specimens did not occur.

A method to apply a continuous vacuum to the testing apparatus was available for the testing of Granite Mountain and Glen Rose quarries. While the measured values were in general agreement with data obtained from previous quarries, there was no need to repeatedly retest samples and the testing procedure was much more efficient.

4.2.2 Predicted Values of Hydraulic Conductivity

The hydraulic conductivity values obtained from laboratory testing which are reported in Table 4.2 and Table 4.3 were compared with values calculated using correlations developed by others. Three (3) different methods were used to empirically estimate the hydraulic conductivity of the materials tested in this study.

Since all of the methods for estimating hydraulic conductivity are based on knowing specific particle sizes in the grain size distribution, a graphical method was developed to determine these values. Fig. 4.7 shows the procedure used on the “as received” grain size distribution curve for Black Rock quarry material to determine $D_{10}$, $D_{15}$, and $D_{30}$ for the
model gradation containing 16 percent fines. The procedure was repeated for all quarries at the various fines contents.

As illustrated in Fig. 4.7, the portion of the graph generated from the hydrometer analysis for the “as received” material passing the # 200 sieve was replicated and placed at a location on the graph which corresponded to the percentage of fines for the model blend.

![Figure 4.7](image)

Figure 4.7 – Hydrometer and sieve analysis for “as received” granular base course from Black Rock quarry that demonstrates the method used to determine $D_{10}$, $D_{15}$, and $D_{30}$ for the model gradation containing 16 % fines.

A straight line was drawn from the point on the replicated line, which represented the percent passing the #200 sieve to the point on the original grain size curve representing the percentage of material passing the # 40 sieve. Where this constructed
curve crossed the grid lines representing 10 percent, 15 percent, and 30 percent finer lines the corresponding values for $D_{10}$, $D_{15}$, and $D_{30}$ were read from the x-axis.

The Hazen equation was the first predictive method used to compare predicted values of hydraulic conductivity to measured values from this study. This method relies on the value of $D_{10}$ of the base material to predict hydraulic conductivity:

$$k = C \times D_{10}^2$$  \hspace{1cm} (2.35)

The graphical method illustrated in Fig. 4.7 was used to determine $D_{10}$ for all quarries. The value of $k$ is reported in cm/sec. The constant $C$ is widely reported in the literature to be approximately equal to 1.0 and was found to be equal to 1.16 for Hazen’s original data when $k$ is reported in cm/sec. For this study, however, the value of $C$ was taken as 0.46 instead of the widely accepted 1.0. According to Hazen (1911),

"The coefficient rarely falls below 400 [for $k$ in ft/day], [0.46 for $k$ in cm/sec] for old and dirty sands, and rarely rises above 1900 [1.38], and, in a majority of ordinary sands, falls between 700 [0.81] and 1000 [1.16]."

For this study, it was thought that the granular base course tested would resemble a dirty sand and using a coefficient of 0.46 would more closely replicate the actual conditions.

The second method used the Sherard equation, which relies on the value of $D_{15}$ of the base course,

$$k = 0.35 \times D_{15}^2$$  \hspace{1cm} (2.36)
The value of k is reported in cm/sec. The tests used to develop this correlation were made on dense sands and gravels commonly used as filters and drains for dams. The graphical method used to determine $D_{10}$ for the Hazen equation was also used to obtain the value of $D_{15}$ used in this correlation.

The third method utilized the Moulton equation which relies on $D_{10}$, the percent passing the #200 sieve and the porosity of the sample:

$$k = \frac{6.214 \times 10^5 \times D_{10}^{1.4178} \times n^{0.654}}{P_{200}^{0.597}}$$  \hspace{1cm} (2.37)

The value of k is reported in ft/day. Porosity was determined using:

$$n = \frac{e}{1+e}$$  \hspace{1cm} (4.1)

$$e = \frac{100wG_s}{S_r}$$  \hspace{1cm} (4.2)

Where:

- $e$ = void ratio
- $G_s$ = Specific gravity of solids
- $w$ = moisture content expressed as a percent
- $S_r$ = Degree of Saturation at the time of sample preparation

The fact that the value of porosity was taken as a constant for all blends introduces error into the correlation since porosity decreases as fines increase. Assuming a constant porosity will produce falsely high estimations of hydraulic conductivity at higher fines contents. For purposes of comparison the value obtained from this equation was multiplied by 0.0003528 to convert to cm/sec from ft/day.
Table 4.5 is a comparison of the predicted values using the Hazen, Sherard, and Moulton correlations to the measured values of hydraulic conductivity for all model gradations from all quarries.

The predictive values of hydraulic conductivity presented in Table 4.5 are also presented graphically in Figure 4.8 for material from Sharp’s quarry. The lines in the figure illustrate the approximate trend for each of the prediction methods. These trend lines allow the prediction methods to be compared to the approximate trend line for the laboratory data. In general terms, the Hazen and Sherard methods tend to overestimate hydraulic conductivity by 1 to 3 orders of magnitude while the Moulton method underestimates the hydraulic conductivity by 1 to 2 orders of magnitude.

The fact that the Hazen and Sherard’s methods were developed using uniformly graded clean sands and gravels explains their over prediction of the measured values for densely graded aggregate base course. The uniformly graded, clean material will have superior pore space continuity and therefore a higher hydraulic conductivity than a denser graded material.

While Hazen reported a range for his coefficient, which accounted for clean versus dirty material, Sherard did not. Also, Hazen’s correlation is more dependent on the smaller D_{10} size, which has a greater effect on pore sizes in crushed stone base than the larger D_{15} size. Given these two facts, it was felt that the Hazen correlation with the appropriate constant, C, would more closely predict hydraulic conductivity of a granular base course than would the Sherard correlation. This concept is clearly illustrated in Fig.4.8. It was noted that an average of Hazens’ predictive values and Moulton’s predictive values would be very close to the hydraulic conductivities measured in this
study.

Table 4.5-Comparison of the Hazen, Sherard, and Moulton predicted values with the laboratory derived values for the hydraulic conductivity of model gradations

<table>
<thead>
<tr>
<th>Quarry</th>
<th>% Fines</th>
<th>( D_{10} ) (mm)</th>
<th>( D_{15} ) (mm)</th>
<th>Predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Hazen</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Sherard</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Moulton</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Lab</td>
</tr>
<tr>
<td>Sharps 6</td>
<td>0.160</td>
<td>0.350</td>
<td>1.18E-02</td>
<td>4.29E-02</td>
</tr>
<tr>
<td>Sharps 8</td>
<td>0.120</td>
<td>0.300</td>
<td>6.62E-03</td>
<td>3.15E-02</td>
</tr>
<tr>
<td>Sharps 10</td>
<td>0.075</td>
<td>0.280</td>
<td>2.59E-03</td>
<td>2.74E-02</td>
</tr>
<tr>
<td>Sharps 12</td>
<td>0.030</td>
<td>0.210</td>
<td>4.14E-04</td>
<td>1.54E-02</td>
</tr>
<tr>
<td>Sharps 14</td>
<td>0.013</td>
<td>0.110</td>
<td>7.77E-05</td>
<td>4.24E-03</td>
</tr>
<tr>
<td>Sharps 16</td>
<td>0.004</td>
<td>0.058</td>
<td>8.91E-06</td>
<td>1.18E-03</td>
</tr>
<tr>
<td>Preston 6</td>
<td>0.150</td>
<td>0.270</td>
<td>1.04E-02</td>
<td>2.55E-02</td>
</tr>
<tr>
<td>Preston 8</td>
<td>0.110</td>
<td>0.250</td>
<td>5.57E-03</td>
<td>2.19E-02</td>
</tr>
<tr>
<td>Preston 10</td>
<td>0.075</td>
<td>0.220</td>
<td>2.59E-03</td>
<td>1.69E-02</td>
</tr>
<tr>
<td>Preston 12</td>
<td>0.047</td>
<td>0.180</td>
<td>1.02E-03</td>
<td>1.13E-02</td>
</tr>
<tr>
<td>Preston 14</td>
<td>0.022</td>
<td>0.110</td>
<td>2.23E-04</td>
<td>4.24E-03</td>
</tr>
<tr>
<td>Preston 16</td>
<td>0.003</td>
<td>0.059</td>
<td>5.01E-06</td>
<td>1.22E-03</td>
</tr>
<tr>
<td>Black Rock 6</td>
<td>0.150</td>
<td>0.260</td>
<td>1.04E-02</td>
<td>2.37E-02</td>
</tr>
<tr>
<td>Black Rock 8</td>
<td>0.110</td>
<td>0.240</td>
<td>5.57E-03</td>
<td>2.02E-02</td>
</tr>
<tr>
<td>Black Rock 10</td>
<td>0.033</td>
<td>0.230</td>
<td>5.01E-04</td>
<td>1.85E-02</td>
</tr>
<tr>
<td>Black Rock 12</td>
<td>0.015</td>
<td>0.190</td>
<td>1.04E-04</td>
<td>1.26E-02</td>
</tr>
<tr>
<td>Black Rock 14</td>
<td>0.002</td>
<td>0.130</td>
<td>1.84E-06</td>
<td>5.92E-03</td>
</tr>
<tr>
<td>Black Rock 16</td>
<td>0.004</td>
<td>0.056</td>
<td>8.11E-06</td>
<td>1.10E-03</td>
</tr>
<tr>
<td>Glen Rose 6</td>
<td>0.130</td>
<td>0.180</td>
<td>7.77E-03</td>
<td>1.13E-02</td>
</tr>
<tr>
<td>Glen Rose 8</td>
<td>0.100</td>
<td>0.160</td>
<td>4.60E-03</td>
<td>8.96E-03</td>
</tr>
<tr>
<td>Glen Rose 10</td>
<td>0.075</td>
<td>0.140</td>
<td>2.59E-03</td>
<td>6.86E-03</td>
</tr>
<tr>
<td>Glen Rose 12</td>
<td>0.007</td>
<td>0.090</td>
<td>2.25E-05</td>
<td>2.84E-03</td>
</tr>
<tr>
<td>Glen Rose 14</td>
<td>0.002</td>
<td>0.019</td>
<td>1.84E-06</td>
<td>1.26E-04</td>
</tr>
<tr>
<td>Glen Rose 16</td>
<td>0.002</td>
<td>0.000</td>
<td>1.04E-06</td>
<td>5.60E-08</td>
</tr>
<tr>
<td>Granite Mtn 6</td>
<td>0.130</td>
<td>0.280</td>
<td>7.77E-03</td>
<td>2.74E-02</td>
</tr>
<tr>
<td>Granite Mtn 8</td>
<td>0.110</td>
<td>0.260</td>
<td>5.57E-03</td>
<td>2.37E-02</td>
</tr>
<tr>
<td>Granite Mtn 10</td>
<td>0.075</td>
<td>0.230</td>
<td>2.59E-03</td>
<td>1.85E-02</td>
</tr>
<tr>
<td>Granite Mtn 12</td>
<td>0.042</td>
<td>0.190</td>
<td>8.11E-04</td>
<td>1.26E-02</td>
</tr>
<tr>
<td>Granite Mtn 14</td>
<td>0.039</td>
<td>0.100</td>
<td>7.00E-04</td>
<td>3.50E-03</td>
</tr>
<tr>
<td>Granite Mtn 16</td>
<td>0.013</td>
<td>0.055</td>
<td>7.77E-05</td>
<td>1.06E-03</td>
</tr>
</tbody>
</table>

Since the Moulton correlation was developed specifically for aggregate base course, its obvious divergence from measured values is of concern. However, according to Richardson (1997), a review of the original work which produced the data used to
develop Moulton’s correlation revealed potential problems with the method. The specimens in the tests were saturated only by submergence which may have lead to air blockages of interconnected void spaces resulting in a reduction in the measured hydraulic conductivity. A smeared end condition would also lead to falsely low values since the flow into and out of the sample would be restricted. Additionally the specific gravity of the mineral aggregate was apparently not measured for any of the tests. Instead estimates of specific gravity were used to determine the porosity of the samples. This discrepancy has an unknown effect on the values predicted by the equation.

![Graph showing comparison of Hazen, Sherard, and Moulton predicted values with measured values of hydraulic conductivity for model gradations from Sharp’s quarry.]

Figure 4.8—Comparison of the Hazen, Sherard, and Moulton predicted values with the measured values of the hydraulic conductivity for model gradations from Sharp’s quarry.

### 4.3 Suction Testing

A simplified suction tested as described in Chapter 3 was run on model gradations having 6, 8, 10, 12, 14, and 16 percent fines content. Two (2) samples of the model
gradation were tested for each percentage of fines. In addition, a sample of the lower bound gradation with 6 percent fines and the upper bound gradation with 10 percent fines was tested. A summary of the results for the suction tests from all quarries is presented in Table 4.6. Test data and calculations supporting the values shown in Table 4.6 are presented in Appendix D.

An analysis of the data presented in Table 4.6 and Fig. 4.9 suggests that the materials from each quarry had a different apparent affinity for water as evidenced by the range in moisture increases from around one percent for the Sharps quarry to around six percent for the Preston and Granite Mountain quarries. However, the percent moisture gain or soil suction versus fines content increased at nearly an identical rate for all quarries. It is believed that the initial moisture condition and variations in testing of the sample had more to do with the total amount of moisture gain than the material properties of the base. The data support the notion that there is a slight increase in suction as the percent fines increases, but the increase is modest, typically less than one percent over the range of fines investigated. Again, the data from Sharps quarry are anomalous, which may point to errors in the testing protocol.

Fig 4.9 presents a graphical representation of the average fines content versus the total moisture gain for each of the model gradations shown in Table 4.6. The average percent moisture gain versus the square root of time for the Granite Mountain quarry is presented in Fig. 4.10. The data shown in Fig. 4.10 is typical of the data for all of the suction tests that were conducted in this study. This data indicates that the percent moisture gain or soil suction increases rapidly during the first 1.5 to 2 days of soaking, after which the moisture gain is minimal. The data also indicates that the rate of moisture
gain is virtually the same for all gradations. Perhaps of most significance is the fact that there is almost no distinction in suction potential between the model blends of 8, 10 and 12 percent fines, while there is a clear difference between the 6 percent gradation and the 14 and 16 percent gradations.

Table 4.6 – Final increase in moisture, expressed as a percent, from suction testing on model gradations of class 7 base course for the target quarries.

<table>
<thead>
<tr>
<th>% FINES</th>
<th>Black Rock</th>
<th>Glen Rose</th>
<th>Granite Mtn</th>
<th>Preston</th>
<th>Sharp's</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>4.11</td>
<td>3.33</td>
<td>5.74</td>
<td>6.00</td>
<td>1.35</td>
</tr>
<tr>
<td>6</td>
<td>3.98</td>
<td>3.09</td>
<td>5.48</td>
<td>5.50</td>
<td>1.05</td>
</tr>
<tr>
<td>8</td>
<td>4.37</td>
<td>3.26</td>
<td>5.91</td>
<td>5.83</td>
<td>1.29</td>
</tr>
<tr>
<td>8</td>
<td>4.23</td>
<td>3.36</td>
<td>5.99</td>
<td>5.73</td>
<td>1.05</td>
</tr>
<tr>
<td>10</td>
<td>4.34</td>
<td>3.54</td>
<td>6.04</td>
<td>5.83</td>
<td>1.50</td>
</tr>
<tr>
<td>10</td>
<td>4.30</td>
<td>3.29</td>
<td>6.04</td>
<td>5.92</td>
<td>1.64</td>
</tr>
<tr>
<td>12</td>
<td>4.55</td>
<td>3.40</td>
<td>6.20</td>
<td>5.80</td>
<td>1.85</td>
</tr>
<tr>
<td>12</td>
<td>4.41</td>
<td>3.25</td>
<td>6.01</td>
<td>6.10</td>
<td>2.12</td>
</tr>
<tr>
<td>14</td>
<td>3.88</td>
<td>3.47</td>
<td>6.27</td>
<td>6.05</td>
<td>1.72</td>
</tr>
<tr>
<td>14</td>
<td>3.62</td>
<td>3.83</td>
<td>6.35</td>
<td>6.03</td>
<td>1.72</td>
</tr>
<tr>
<td>16</td>
<td>4.68</td>
<td>3.60</td>
<td>6.34</td>
<td>6.03</td>
<td>1.99</td>
</tr>
<tr>
<td>16</td>
<td>4.12</td>
<td>3.63</td>
<td>6.40</td>
<td>6.22</td>
<td>2.07</td>
</tr>
<tr>
<td>6 (LB)</td>
<td>3.53</td>
<td>3.10</td>
<td>6.15</td>
<td>5.02</td>
<td>1.91</td>
</tr>
<tr>
<td>10 (UB)</td>
<td>4.65</td>
<td>3.73</td>
<td>6.66</td>
<td>6.48</td>
<td>1.88</td>
</tr>
</tbody>
</table>

4.3.1 Problems Encountered During Suction Tests

The hypothesis of the suction test is that increasing fines will cause increased capillarity in the specimen and will result in a higher increase in moisture content during soaking. Since the item having the greatest effect on pore size is the grain sizes distribution (Taylor, 1948) it follows that the percentage of fines in a base course has a direct bearing on the capillary rise or suction of that base course.
Figure 4.9 - Total moisture gain versus the percent of fines for model gradation blended class 7 base course from the Black Rock, Glen Rose, Granite Mountain, Preston, and Sharp's quarries.

Figure 4.10 - Total moisture gain versus square root of time for the model gradation class 7 base course from the Granite Mountain quarry
For the first quarry tested (Sharp’s), this hypothesis was not completely borne out. The samples were compacted at OMC to 98 percent of maximum dry density in a plastic concrete cylinder mold. The compacted specimens were then air-dried and placed in the soaking tub. Fig. 4.11 illustrates the total moisture gain at the conclusion of soaking versus the percent of fines for model gradations from Sharp’s quarry. An analysis of Fig. 4.11 revealed two (2) anomalies in the results that suggested a problem with the test procedure:

(1) The moisture gain did not increase from 6 to 8 percent fines.

(2) The moisture gain did not increase from 12 to 16 percent fines and actually dropped at 14 percent fines.

At the time of testing for this quarry it was thought that the fines were “plugging” the inlet holes of the plastic concrete mold and not allowing a uniform flow of water to the samples. When the 2nd quarry was tested (Preston) the samples were compacted into the mold then oven dried. At the conclusion of the drying process the inlet holes were reamed with a screwdriver to insure that they were not plugged with fines.

Fig. 4.12 illustrates the total moisture gain at the conclusion of soaking versus the percent of fines for model gradations from Preston quarry. When comparing Figures 4.11 and 4.12 it is significant to note the difference in the total moisture gain between the two. The bulk of this difference is attributed to oven drying the Preston material and only air drying the Sharps material. While the results in Fig. 4.12 demonstrated a more uniform increase in moisture with increasing fines than the Sharp’s test did, it was felt that further modifications to the test procedure were still necessary.
Figure 4.11 - Total moisture gain versus the percent of fines for model gradations from Sharp's quarry.

When the third quarry, Black Rock, was tested a 25 mm layer of material passing the 19 mm (3/4") sieve and retained on the to 4.75 mm (# 4) was placed in the bottom of the mold prior to compaction. Note that the height of water in the soaking tub was greater than 1 inch so it still covered this coarser material during soaking. The intended purpose of this material was to prevent "plugging" of the inlet holes and allow a more uniform flow of water to the entire base of the sample. This procedure solved the problem of the non-uniform flow of water; however during the test fines were "washed out" of the samples as they were removed from the soaking tub for weighing. In the previous test this problem was not observed, presumably because the coarser material to allow uniform water flow was not used. It was noted that a layer of fines covered approximately 25 percent of the bottom of the soaking tub at the conclusion of the test. It
was believed that the loss of this material affected the capillarity of the lower sections of the specimen by increasing the pore sizes and as a result the values obtained were suspect, particularly so at the 14 and 16 percent fines gradations.

![Figure 4.12 - Total moisture gain versus the percent of fines for model gradations from Preston quarry.](image)

Fig. 4.13 shows the total moisture gain versus the percent of fines for the model blends from Black Rock quarry which were oven dried. It was believed that the samples with a higher fines contents would be more susceptible to fines washing out which would produced results that did not conform to those expected.

The fact that there was drop in moisture gain between 12 and 14 percent fines and that the average moisture gain at 12 and 16 percent fines was nearly the same indicated that perhaps even further modifications of the testing procedures were required.
For the Glen Rose and Granite Mountain tests, (fourth and fifth quarries), the 25 mm layer of coarse material was replaced with a 37.5 mm layer of material that passed the 4.75 mm (# 4) and was retained on the 0.425 mm (# 40) sieve. This layer of material allowed for the uniform flow of water across the entire base of the specimen and also served as a filter media to prevent the loss of fines during the weighing operation. The samples were oven dried to completely remove moisture before soaking. The total moisture gain versus the percent of fines for model blends from Glen Rose and Granite Mountain quarries is illustrated in Fig. 4.14.

The results of these tests demonstrated only minor scatter between replicate samples and a smooth trend of moisture increase versus percent fines was observed. These results are clearly what was expected in this test and demonstrate the requirement for a filter.
media in the base of the test specimen to achieve repeatability.

An approximate 2.5 percent higher total moisture gain for material from Granite Mountain quarry as compared to similar material from Glen Rose quarry was noted. It was thought that the higher plasticity of the material from Granite Mountain quarry (7.87 percent) compared to the plasticity of the Glen Rose quarry material (4.59 percent) was the primary factor causing this divergence. Also, the material from the Granite Mountain quarry was finer than that from the Glen Rose quarry, as shown in Figure 4.5. The finer material would result in smaller diameter pore spaces for the Granite Mountain quarry material and cause increased capillarity or suction.

![Figure 4.14 - Total moisture gain versus the percent of fines for model gradation blended class 7 base course from Glen Rose and Granite Mountain quarries.](image)

4.4 Drainage Analysis
Excess moisture in the unbound aggregate base under both rigid and flexible pavements can cause elevated pore water pressures, heaving of subgrade soils, ejection of fines and the formation of ice lenses, all of which will shorten the service life of the pavement structure. In addition, the effectiveness of edge or longitudinal drains in removing water from the pavement system is severely limited without a freely draining base course. Since the gravel edge drain functions only if the aggregate base course has adequate transmissibility to get the water to the drain, the hydraulic conductivity of the base material is critical in the overall performance of the drainage system. To illustrate this point, a calculation of the required hydraulic conductivity to adequately drain the base course for a typical Arkansas highway pavement section and a time to drain analysis were performed.

**Required Hydraulic Conductivity**

The hydraulic conductivity computed in this example is the value required to prevent “ponding” of water on the surface of the base course. Cedergren suggests that the 1 hour/1 year storm be used in drainage design and that a design infiltration rate of 0.67 be used for a PCC pavement. However, research conducted at the University of Arkansas has suggested the infiltration coefficient for flexible pavements is below 0.5 (Dennis, 2004). In this analysis, 0.5 is used as the infiltration coefficient and 4.06 cm/hr (1.6 in/hr) was selected for the 1 hour/1 year storm design precipitation rate (FHWA, 1973; AHTD Drainage manual, 1982).

Sketches of a typical Arkansas highway, which give the dimensions used in the hydraulic conductivity analysis, are shown in Fig. 4.15 and Fig. 4.16. Darcy’s Law was used to analyze the flow conditions for a 30.48 cm (1 ft) wide transverse strip of the
pavement and base course under steady state flow conditions. In this example the length of the flow path is 731.52 cm (24 ft), the base course drainage layer thickness is 22.86 cm (9 in), and the asphalt surface course thickness is 10.16 cm (4 in).

The total change in head, $\Delta h$, was selected as the base course thickness of 22.86 cm plus the cross slope of 7.5 cm. The rationale for this selection was that the asphalt layer should remain completely drained throughout the storm event if the base course is to be considered freely draining.

Figure 4.15 - Plan view of a typical Arkansas highway used in the hydraulic conductivity analysis.
In this example the following parameters are used:

\[ A = (\text{drainage area}) = 30.48 \text{ cm} \times 731.52 \text{ cm} = 22,296 \text{ cm}^2 \]

\[ a = (\text{cross sectional area of base}) = 22.86 \text{ cm} \times 30.48 \text{ cm} = 696.8 \text{ cm}^2 \]

Rainfall intensity = \( \frac{4.06 \text{ cm}}{\text{hr}} \times \frac{1 \text{ hr}}{3600 \text{ sec}} = 0.0011 \frac{\text{cm}}{\text{sec}} \)

Rainfall infiltration rate = \( \frac{4.06 \frac{\text{cm}}{\text{hr}}}{3600 \frac{\text{sec}}{\text{hr}}} \times 0.50 = 0.00055 \frac{\text{cm}}{\text{sec}} \)

\( q_{\text{total}} = 22,296 \text{ cm}^2 \times 0.00055 \frac{\text{cm}}{\text{sec}} = 12.26 \frac{\text{cm}^3}{\text{sec}} \)

\( \Delta h = 22.86 \text{ cm} + 7.5 \text{ cm} = 30.36 \text{ cm} \)

\[ i = \frac{30.36 \text{ cm}}{0.5 \times 731.52 \text{ cm}} = 0.083 \]

From Darcy’s law,

\[ k = \frac{q}{i \times A} \quad (4.3) \]

and
\[ k = \frac{12.26 \text{ cm}^3}{0.083 \times 696.8 \text{ cm}^2} = 0.212 \text{ cm/sec} \]

or

\[ 2.12 \times 10^{-1} \text{ cm/sec required to prevent "ponding"} \]

As an alternative it can be assumed that the asphalt surface course also must be saturated for "ponding" to occur. When this is the case \( \Delta h = 40.52 \text{ cm} \) and \( i = .111 \).

Therefore,

\[ k = \frac{12.26 \text{ cm}^3}{0.111 \times 696.8 \text{ cm}^2} = 0.160 \text{ cm/sec} \]

or

\[ 1.60 \times 10^{-1} \text{ cm/sec} \]

This analysis indicates that the granular base course used in this example needs to have a hydraulic conductivity greater than about \( 10^{-1} \text{ cm/sec} \) to effectively drain the pavement structure. Since the hydraulic conductivities of granular base courses in this study ranged from about \( 10^{-3} \) to \( 10^{-6} \text{ cm/sec} \) it can be concluded that the densely graded granular base course used in highway construction in Arkansas does not possess adequate drainage properties to prevent ponding of water during and immediately after rainfall events. The inability of densely graded Class 7 base to effectively drain pavement structures will not only cause accelerated pavement deterioration but will create an increased safety hazard during rainfall events. The inability of water to migrate through the base course could lead to reduced skid resistance and increase the potential for hydroplaning during rainstorms. Saturation of the base course can also lead
to a stripping failure of the bottom layer of the asphalt pavement, which in turn causes rutting of the surface.

**Time to Drain**

Since the above analysis indicates that the pavement system will not be completely drained during a rainfall event, the time required to drain the pavement system becomes a critical factor when selecting layer coefficients in the structural design of pavement systems. The layer coefficients for unbound materials require a modifier to account for drainage conditions. Tables 4.7 and 4.8 are used to determine the value of the layer coefficient modifier used in the design of a pavement system.

Table 4.7 – Quality of pavement structure drainage (after AASHTO, 1993)

<table>
<thead>
<tr>
<th>Quality of Drainage</th>
<th>Water Removed Within</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>2 hours</td>
</tr>
<tr>
<td>Good</td>
<td>1 day</td>
</tr>
<tr>
<td>Fair</td>
<td>1 week</td>
</tr>
<tr>
<td>Poor</td>
<td>1 month</td>
</tr>
<tr>
<td>Very Poor</td>
<td>(water will not drain)</td>
</tr>
</tbody>
</table>

Table 4.8 – Recommended $m_i$ Values for Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements (after AASHTO, 1993)

| Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation |
|-----------------------------------------------|-----------------|-----------------|-----------------|-----------------|
| Quality of Drainage                          | Less than 1%    | 1-5%            | 5-25%           | Greater than 25%|
| Excellent                                     | 1.40-1.35       | 1.35-1.30       | 1.30-1.20       | 1.20            |
| Good                                          | 1.35-1.25       | 1.25-1.15       | 1.15-1.00       | 1.00            |
| Fair                                          | 1.25-1.15       | 1.15-1.05       | 1.00-0.80       | 0.8             |
| Poor                                          | 1.15-1.05       | 1.05-0.80       | 0.80-0.60       | 0.6             |
| Very Poor                                     | 1.05-0.95       | 0.95-0.75       | 0.75-0.40       | 0.4             |

Given the rainfall conditions used in the preceding drainage analysis, the time to
drain for a typical pavement structure after the rainfall event can be computed using a falling head analysis with the following assumptions:

- Void spaces in the asphalt equal 6 percent
- Degree of saturation of the base course at optimum moisture and k of the base course vary as reported in Table 4.9
- Base is considered drained when height of water table is 90 percent of the height of the base and all voids in the asphalt are drained
- \( L = 0.5 \times 731.52 \text{ cm} = 365.8 \text{ cm} \) (average length of drainage path)

Using the assumptions above and dimensions shown in Fig. 4.15 and Fig 4.16, a time to drain analysis for material from Black Rock quarry follows:

Total \( Q \) (water in void spaces)

\[
Q_{\text{asphalt}} = 0.06 \text{ (voids)} \times 10.16 \text{ cm (thickness)} \times 30.48 \text{ cm (width)} \times 731.52 \text{ cm (length)} = 13,592 \text{ cm}^3
\]

\[
Q_{\text{base}} = 0.19 \text{ (81 percent saturation)} \times 0.185 \text{ (voids)} \times 2.28 \text{ cm (10 percent total thickness)} \times 30.48 \text{ cm (width)} \times 731.52 \text{ cm (length)} = 1786.9 \text{ cm}^3
\]

\[
Q_{\text{total}} = Q_{\text{asphalt}} + Q_{\text{base}} = 15378.9 \text{ cm}^3
\]

Since:

\[
k = \frac{aL}{At} \ln \left( \frac{h_1}{h_2} \right) \quad (2.21)
\]

or:

\[
t = \frac{aL}{At} \ln \left( \frac{h_1}{h_2} \right)
\]

and:

\[
q \times \Delta t = Q = a \times dh \quad (4.4)
\]

re-arrangement of (4.2) gives:
\[ a = \frac{Q}{dh} \]  

(4.5)

where:

\[ dh = h_1 - h_2 = 43.18 \text{ cm} - 30.73 \text{ cm} = 12.45 \text{ cm} \]

\[ a = \frac{15378.9 \text{ cm}^3}{12.45 \text{ cm}} = 1235.3 \text{ cm}^2 \]

\[ A = 22.86 \text{ cm} \times 30.48 \text{ cm} = 696.8 \text{ cm}^2 \]

\[ t = \left( \frac{1235.3 \text{ cm}^2 \times 365.7 \text{ cm}}{696.8 \text{ cm}^2 \times 0.000384 \text{ cm/ sec}} \right) \times \ln \left( \frac{43.18 \text{ cm}}{30.73 \text{ cm}} \right) = 5,742,661 \text{ sec} = 66.2 \text{ days} \]

Table 4.9 - Time to drain for model blends @ 10 percent fines for all quarries and the resulting quality of drainage from Table 4.7

<table>
<thead>
<tr>
<th>Quarry</th>
<th>( \delta ) (pcf)</th>
<th>( k ) (cm/sec)</th>
<th>( Q_{\text{total}} ) (cm(^3))</th>
<th>( a ) (cm(^2))</th>
<th>Time to drain (days)</th>
<th>Quality of Drainage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharps (10 % fines)</td>
<td>141.0</td>
<td>2.60E-05</td>
<td>19909.9</td>
<td>1599.2</td>
<td>97.5</td>
<td>VERY POOR</td>
</tr>
<tr>
<td>Preston (10% fines)</td>
<td>132.0</td>
<td>9.40E-06</td>
<td>21657.4</td>
<td>1739.6</td>
<td>273.1</td>
<td>VERY POOR</td>
</tr>
<tr>
<td>Black Rock (10 % fines)</td>
<td>144.0</td>
<td>3.84E-05</td>
<td>21217.5</td>
<td>1704.2</td>
<td>66.2</td>
<td>VERY POOR</td>
</tr>
<tr>
<td>Glen Rose (10% fines)</td>
<td>135.0</td>
<td>5.75E-04</td>
<td>20191.0</td>
<td>1621.8</td>
<td>4.5</td>
<td>FAIR</td>
</tr>
<tr>
<td>Granite Mountain (10% fines)</td>
<td>134.2</td>
<td>1.72E-06</td>
<td>21792</td>
<td>1750.3</td>
<td>1429.5</td>
<td>VERY POOR</td>
</tr>
</tbody>
</table>

A review of the values shown in Table 4.9 indicates that the quality of drainage for a typical base course in Arkansas with 10 percent fines is fair to very poor. An increase in the percentage of fines from 10 to 12 percent will more than double the time required to drain the materials from all quarries except Glen Rose. However, this increase in time does not change the Quality of Drainage category presented in Table 4.9. In the case of the Glen Rose quarry, the time to drain actually went down as a result of increasing the fines from 10 to 12 percent.
To assign a layer modifier as shown in Table 4.8, the return interval for the storm that will cause saturation must be known. The intensity duration and frequency curves taken from the AHTD Drainage Manual (1982) for Area III were used in this analysis. A time of concentration for a typical highway pavement section is conservatively taken as 10 minutes for this analysis.

The object of the analysis is to determine the return interval for a rainfall event that will produce saturation of the base and asphalt layers at the given time of concentration and required intensity. For the purposes of this analysis a conservative assumption is that the antecedent moisture condition in the asphalt and base is at the as installed condition, thus entire void spaces of the asphalt and 19 percent of the void spaces in the base must be filled with infiltrating water from the design storm. This assumption is being made even though studies have shown that insitu moisture contents in base courses are typically higher than OMC a year after installation. The following parameters for material from Black Rock quarry with 10 percent fines are used in this analysis:

\[ V_{\text{voids}}(@OMC) = 15378.9 \text{ cm}^3 \]

\[ A = 22,296 \text{ cm}^2 \text{ (area of pavement subject to rainfall)} \]

\[ a \text{ (cross sectional area of base)} = 22.86 \text{ cm} \times 30.48 \text{ cm} = 696.8 \text{ cm}^2 \]

\[ k = .0000384 \text{ cm/sec} \]

\[ I_c \text{ (infiltration coefficient)} = 0.5 \]

\[ t_c \text{ (time of concentration)} = 10 \text{ min} \]

\[ Q_{\text{outflow}} = k \times i \times a = .0000384 \text{ cm/sec} \times .118 \times 696.8 \text{ cm}^2 = .00316 \text{ cm}^3/\text{sec} \]

\[ Q_{\text{inflow}} = \text{rainfall intensity} \times \text{infiltration coefficient} \times A \]
\[ Q_{\text{inflow}} - Q_{\text{outflow}} = \text{rate of void fill} \]

\[ V_{\text{voids}} \div \text{rate of void fill} = \text{time to fill (10 minutes or 600 sec)} \]

Rate of void fill = 15378.9 cm\(^3\) \div 600 sec = 25.63 cm\(^3\)/sec

\[ Q_{\text{inflow}} = \text{rate of void fill} + Q_{\text{outflow}} \]

\[ Q_{\text{inflow}} = 25.63 \text{ cm}^3/\text{sec} + .00316 \text{ cm}^3/\text{sec} = 25.6346 \text{ cm}^3/\text{sec} \]

\[ \text{Rainfall Intensity} = \frac{Q_{\text{inflow}}}{I_c \times A} = \frac{25.6346 \text{ cm}^3/\text{sec} \times 3600 \text{ sec}}{2.54 \text{ cm/in} \times 0.50 \times 22,296 \text{ cm}^2} = 3.25 \text{ in/hr} \]

According to the IDF curves shown in Fig. 4.17 a 10 minute/.5 year storm will produce a rainfall intensity of about 3.25 in/hr. The black dot in the figure identifies storm frequency for material from this quarry.

Given a 0.5 year return interval for a storm that will produce saturation and a time to drain of 66.2 days for the base material selected in the analysis, the pavement will be exposed to levels approaching saturation approximately 66.2 days \div (365 \times 0.5) days = 36.2 percent of the time it is in service. Given this condition, table 4.7 suggests that a modifier coefficient (m\(_i\)) of 0.4 be used.

Table 4.10 shows the above analysis for each quarry at a 10 percent fines content. The rainfall intensities calculated suggest that a storm with a 1 year return interval can be used for all quarries to determine the time the pavement will be subjected to saturation. The results of the analyses indicate that, except for material from Glen Rose quarry, a significant reduction of the modifier coefficient (m\(_i\)) is warranted.
Figure 4.17 – Rainfall intensity duration and frequency relationship for Area III in Arkansas (source AHTD Drainage Manual, 1982). The dot denotes the rainfall intensity of 3.25 in/hr calculated in the sample problem.

The simplified drainage analyses presented in section 4.4 show that the hydraulic conductivity of a granular base course in a typical Arkansas highway pavement system should be about $10^{-1}$ cm/sec to be considered a free draining material. The hydraulic
conductivities measured for the “as received” gradation of the common base courses used in Arkansas were all 2 to 4 orders of magnitude lower than this criteria. This fact suggests that the densely graded base courses commonly used in Arkansas are ineffective in immediately draining the pavement system. These findings indicate that pavements in Arkansas are susceptible to damage caused by stripping of flexible pavements and a loss of subgrade and base strength. These conditions will lead to accelerated rutting and fatigue cracking which reduces the overall serviceability of the roadway.

Table 4.10 –Rainfall intensity required to saturate pavement with a given time of concentration

<table>
<thead>
<tr>
<th>Quarry</th>
<th>Volume of voids @ OMC</th>
<th>k (cm/sec) @ 10% fines</th>
<th>$Q_{\text{outflow}}$ (cm$^3$)</th>
<th>Rate of void fill (cm$^3$)</th>
<th>$Q_{\text{inflow}}$ (cm$^3$)</th>
<th>Rainfall intensity (in/hr)</th>
<th>Time to drain</th>
<th>Percentage of time pavement exposed to saturation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharps</td>
<td>15248.3</td>
<td>2.60E-05</td>
<td>0.002</td>
<td>25.414</td>
<td>25.416</td>
<td>3.23</td>
<td>97.5</td>
<td>53</td>
</tr>
<tr>
<td>Preston</td>
<td>15473.8</td>
<td>9.40E-06</td>
<td>0.001</td>
<td>25.790</td>
<td>25.791</td>
<td>3.28</td>
<td>273.1</td>
<td>100</td>
</tr>
<tr>
<td>Black Rock</td>
<td>15349.0</td>
<td>3.84E-05</td>
<td>0.003</td>
<td>25.582</td>
<td>25.585</td>
<td>3.25</td>
<td>66.2</td>
<td>36</td>
</tr>
<tr>
<td>Glen Rose</td>
<td>15690.5</td>
<td>5.75E-04</td>
<td>0.047</td>
<td>26.151</td>
<td>26.198</td>
<td>3.33</td>
<td>4.5</td>
<td>2</td>
</tr>
<tr>
<td>Granite Mountain</td>
<td>14815.4</td>
<td>1.72E-06</td>
<td>0.000</td>
<td>24.692</td>
<td>24.692</td>
<td>3.14</td>
<td>1429.5</td>
<td>100</td>
</tr>
</tbody>
</table>

When using the empirical procedure for flexible pavement design contained in the 1993 AASHTO Design Guide, the effect of drainability is accounted for through the use of a layer coefficient modifier, $m_i$. This coefficient represents an estimate of the quality of drainage and the percentage of time the pavement structure is exposed to moisture levels approaching saturation. According to AASHTO design standards, the higher the value of $m_i$ used in pavement design, the thinner the required pavement thickness becomes.
The value of \( m_i \) currently used for pavement design by the AHTD is 1.0 to represent the drainability of the base course used under Arkansas highways. Considering the fact that the drainage analyses performed suggest that this value should be reduced to 0.4-0.75, it appears that conventional pavements have been "under designed" for many years in the State of Arkansas and one would expect premature pavement failures to be common. However, according to research conducted at the University of Arkansas (Dupree, 2000) conventional pavements using class 7 base material are actually performing better than predicted by AASHTO performance equations. If it is accepted that all of the other variables used in the AASHTO performance equation are accurate, then a reduction of the modifier coefficient in pavement design is not warranted. This would indicate that the entire drainage provision in the AASHTO design guide is suspect and in need of revision.

4.5 Cyclic Loading Results

Cyclic triaxial test were performed on each specimen under consolidated-drained, CD, conditions. Forty constant rate of strain stress-controlled loading and unloading cycles were completed at confining and backpressures of 448 kPa and 414 kPa (65 and 60 psi), respectively. The upper and lower stress limits were selected as 21 kPa (3.0 psi) and 3.5 kPa (0.5 psi), respectively. The effective confining stress on each specimen during testing was 35 kPa (5 psi). This effective confining pressure was selected because it represents typical confining stresses found in highway base course (Thornton and Elliott, 1988). The strain rate was initially 60%/hour but was later reduced to 30%/hour because the testing equipment could not reliably control the stress reversals at the higher strain rate.
A plot of deviator stress vs. percent strain for a typical cyclic test is illustrated in Fig. 4.18. Several straight lines were superimposed on these plots to represent the average slope of the stress-strain curve through a complete loading and unloading cycle. The slopes of these superimposed lines were used to obtain the average modulus of the material using Eqn. 4.1.

\[ M = \frac{\Delta \sigma}{\Delta \varepsilon} \quad (4.6) \]

where \( \Delta \sigma = \text{change in deviator stress} \)

\( \Delta \varepsilon = \text{change in strain}. \)

Modulus values were determined for each specimen and plots of modulus value vs. percent fines are presented for each quarry in Figs. 4.19 through 4.23. Samples representing the AHTD upper boundary gradation are denoted by an open triangle while lower boundary samples are symbolized by an open square. The upper boundary blend is the finest gradation permissible and the lower boundary blend the coarsest gradation accepted by current AHTD requirements. Figures 4.19 through 4.23 illustrate that there is a relatively minor amount of scatter between replicate samples. The scatter appears to be somewhat dependent on the fines content of the specimens with tests at higher fines contents exhibiting less scatter. The scatter in data from Sharps quarry may be the result of minor adjustments in the testing protocol rather than actual material property differences.
Figure 4.18: Typical plot of deviator stress vs. percentage fines for a cyclic triaxial test (Sharps M.B. 12% fines shown)

The average elastic secant modulus value from cyclic testing for the Sharps material appears to be about 10,000 psi with a low of about 7,000 psi and a high on the order of 12,500 psi. Modulus values for both the lower and upper boundary blend specimens are within this range. If average values are taken for replicate samples the scatter in the data is reduced with high and low values ranging from approximately 9,000 psi to 10,500 psi.

Test results for Preston quarry material had less scatter between replicate samples than those for the Sharps material. The maximum and minimum elastic secant modulus values were about 10,000 psi and 6,000 psi, respectively, with an average value on the order of 7,000 psi to 7,500 psi. It is noted that the minimum modulus value occurs at
10% fines and the replicate scatter for this blend is the greatest. The upper and lower boundary blend modulus values are contained in the range of modulii values for this model blends.

The elastic secant modulus values for the Black Rock quarry model blends ranged from about 8,500 psi to 14,000 psi with an average of around 10,000 psi. Some of the highest modulus values were obtained for the 16% fines model blend. This is the only quarry in which the greatest percentage fines model blend produces highest modulus values. The upper and lower boundary blend modulus values fall in the range of model blend data.

The replicate specimens for Glen Rose quarry exhibited the tightest agreement of all quarries. The average modulus values for model blends with fines contents between 10% and 16% was essentially the same. When the 6% and 8% specimens are included the average modulus values are in the range of 8,500 psi to 10,500 psi. The average modulus value for all model blends is approximately 9,000 psi. The upper and lower boundary blends modulus values are in the range of those for the model blends.

The elastic secant modulus values for the Granite Mountain model blends ranged from approximately 8,000 psi to 10,500 psi with an average value of about 9,000 psi. The upper boundary blend modulus averaged 11,500 psi. The lower boundary blend modulus values were in the range of model blend values.

When looking at the elastic secant modulus values for all quarries collectively, it is noted that they are relatively unaffected by the percentage of fines present. An approximate average elastic secant modulus value of 9,000 psi reasonably represents all materials and blends tested. The nomograph in Fig. 2.2 indicates that a modulus value of

181
this magnitude correlates to a layer coefficient, $a_2$, of only 0.05. Using the nomograph presented in Fig. 2.2 it is noted that a resilient modulus value of approximately 30,000 psi correlates to a layer coefficient, $a_2$, of 0.14, which is the AASHTO recommended and AHTD accepted value for crushed stone aggregate base course. It is unclear at what stress states the resilient modulus values in the nomograph were determined and that is an important consideration since resilient modulus increases significantly with increases in bulk stress. However, it seems clear that the elastic secant modulii developed in this study cannot be used directly in Fig 2.2 to obtain layer coefficients.

Resilient modulus testing in accordance with AASHTO T-307 was completed by Zhao (2004) for the “as-received” material which had gradations essentially the same as the 10% fines content model blend used in this research. Results from this testing, further discussed in Section 4.6, indicate an average resilient modulus value of 26,000 psi at a 30 psi bulk stress best represents all quarries tested. Resilient modulus values of this magnitude essentially correspond to an $a_2$ coefficient of 0.14 when applying Fig. 2.2.

While results of the elastic secant modulus testing alone are not transferable to layer coefficient values using the AASHTO correlation chart, a correlation between elastic secant modulus and resilient modulus values is presented in Section 4.6 which would allow such a correlation.
Figure 4.19: Elastic secant modulus from cyclic testing vs. percentage fines- Sharps.

Figure 4.20: Elastic secant modulus from cyclic testing vs. percentage fine- Preston.
Figure 4.21: Elastic secant from cyclic testing vs. percentage fines- Black Rock.

Figure 4.22: Elastic secant modulus from cyclic testing vs. percentage fines- Glen Rose.
Figure 4.23: Elastic secant modulus from cyclic testing vs. percentage fines- Granite Mountain.

4.6 Staged Triaxial Results

Staged consolidated-drained (CD) triaxial testing commenced after the final cycle of the repeated load test. Effective confining pressures of 34.5 kPa, 68.9 kPa, and 137.9 kPa (5 psi, 10 psi, and 20 psi) were used for the three stages of shearing. A typical plot of deviator stress vs. axial strain for the three stages is presented in Fig. 4.24. The values of maximum stress difference ($\sigma_1-\sigma_3$) after yielding were extracted from each stage of testing. These values of stress, illustrated in Fig. 4.10 with open stars, were then used to construct Mohr's circle of stress for each confining pressure. The stress-strain points from each of the staged triaxial tests used for defining the elastic secant modulus values reported are illustrated as filled stars in Fig. 4.24. A consolidation analysis was
performed in accordance with the procedure outlined in Chapter 3 to verify that all specimens were tested in a CD condition. The results of this analysis are presented in Table 4.11.

Figure 4.24: Typical plot of deviator stress vs. strain for staged triaxial testing (Black Rock quarry M.B. 10% fines shown)

Figure 4.25 shows a typical Mohr-Coulomb failure envelope for material from the Black Rock quarry. In virtually every test for every quarry a tangent could be constructed to touch the Mohr's circle representing each of the three stages of triaxial testing. The failure envelope was formed by constructing a line which was tangent to each of the Mohr's circles. The slope of the Mohr-Coulomb failure envelope defines the
angle of internal friction, $\phi$, while the y-axis intercept defines the cohesion, $c$, for each sample.

Table 4.11: Verification of CD test conditions (M.B. 10% gradations shown)

<table>
<thead>
<tr>
<th>Calculated $C_{u}$</th>
<th>Target Consolidation, $U_{t}$ (%)</th>
<th>Minimum $t_{m}$ (minutes)</th>
<th>Strain rate (%/hr)</th>
<th>Specimen</th>
<th>Strain at Stage Shear Failure (%)</th>
<th>Calculated Stage $t$ (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Sharps M.B. 10% A- Stage 1</td>
<td>3.02</td>
<td>6.04</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Sharps M.B. 10% A- Stage 2</td>
<td>1.97</td>
<td>3.95</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Sharps M.B. 10% A- Stage 3</td>
<td>1.83</td>
<td>3.65</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Sharps M.B. 10% B- Stage 1</td>
<td>3.00</td>
<td>6.01</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Sharps M.B. 10% B- Stage 2</td>
<td>1.95</td>
<td>3.90</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Sharps M.B. 10% B- Stage 3</td>
<td>4.88</td>
<td>9.73</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Preston M.B. 10% A- Stage 1</td>
<td>2.90</td>
<td>5.01</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Preston M.B. 10% A- Stage 2</td>
<td>2.08</td>
<td>4.17</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Preston M.B. 10% A- Stage 3</td>
<td>1.43</td>
<td>2.85</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Preston M.B. 10% B- Stage 1</td>
<td>2.50</td>
<td>5.00</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Preston M.B. 10% B- Stage 2</td>
<td>1.59</td>
<td>3.18</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Preston M.B. 10% B- Stage 3</td>
<td>2.01</td>
<td>4.02</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Black Rock M.B. 10% A- Stage 1</td>
<td>2.98</td>
<td>5.99</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Black Rock M.B. 10% A- Stage 2</td>
<td>2.08</td>
<td>4.18</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Black Rock M.B. 10% A- Stage 3</td>
<td>2.99</td>
<td>5.79</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Black Rock M.B. 10% B- Stage 1</td>
<td>2.98</td>
<td>5.95</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Black Rock M.B. 10% B- Stage 2</td>
<td>2.47</td>
<td>4.94</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Black Rock M.B. 10% B- Stage 3</td>
<td>1.84</td>
<td>3.69</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Glen Rose M.B. 10% A- Stage 1</td>
<td>3.07</td>
<td>6.15</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Glen Rose M.B. 10% A- Stage 2</td>
<td>2.15</td>
<td>4.30</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Glen Rose M.B. 10% A- Stage 3</td>
<td>3.25</td>
<td>6.50</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Glen Rose M.B. 10% B- Stage 1</td>
<td>3.00</td>
<td>6.00</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Glen Rose M.B. 10% B- Stage 2</td>
<td>1.81</td>
<td>3.61</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Glen Rose M.B. 10% B- Stage 3</td>
<td>1.98</td>
<td>3.79</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Granite Mountain M.B. 10% A- Stage 1</td>
<td>2.80</td>
<td>5.60</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Granite Mountain M.B. 10% A- Stage 2</td>
<td>2.97</td>
<td>5.73</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Granite Mountain M.B. 10% A- Stage 3</td>
<td>1.48</td>
<td>2.98</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Granite Mountain M.B. 10% B- Stage 1</td>
<td>3.22</td>
<td>6.45</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Granite Mountain M.B. 10% B- Stage 2</td>
<td>2.99</td>
<td>5.98</td>
</tr>
<tr>
<td>1695.6 ft²/day</td>
<td>96</td>
<td>1.42</td>
<td>30</td>
<td>Granite Mountain M.B. 10% B- Stage 3</td>
<td>1.70</td>
<td>3.39</td>
</tr>
</tbody>
</table>
Figure 4.25: Typical Mohr-Coulomb failure envelope
(Black Rock M.B. 10% fines shown).
4.7 Discussion of Staged Triaxial Results

The angles of internal friction, $\phi$, and the cohesion intercept, $c$, as defined by the Mohr-Coulomb failure envelope were obtained for each specimen. Comparative plots of $\phi$ vs. percent fines were generated for each quarry and are presented in Figs. 4.26 through 4.30. Specimens representing the AHTD lower boundary acceptance gradation are indicated by a diamond while the upper boundary gradation samples are indicated by a triangle. All model blends are represented by square symbols.

Inspection of the plots of $\phi$ vs. percent fines reveals a general trend for the friction angle. It appears that the friction angle goes up as the percentage of fines increases from 6% to 10% or 12%. As the percent of fines increased beyond 10% or 12% the aggregate friction angle stayed relatively constant or, as in the Preston and Black Rock quarries, continued to increase until the fines content reached 14% before decreasing. While the angle of internal friction tended to be highest in the region between 10% and 12% for all data collected, it should be noted that the absolute difference between friction angles at these fines contents and those at the highest contents of 16% were within 5 degrees of one another and all were above 38 degrees which is considered to be a good value for dense sands (Bowles, 1996).
Figure 4.26: Friction angle vs. percentage fines- Sharps quarry.

Figure 4.27: Friction angle vs. percentage fines- Preston quarry.
Figure 4.28: Friction angle vs. percentage fines- Black Rock quarry.

Figure 4.29: Friction angle vs. percentage fines- Glen Rose quarry.
Figures 4.31 through 4.35 depict the cohesion intercept value $c$ as a function of the fines content. An obvious decrease in cohesion intercept value was observed for each quarry as the amount of fines increased. Review of the Mohr-Coulomb failure envelope plots for each of the blends shows that none of the specimens tested have a cohesion value equal to 0 (tangent line passing through the origin). Discussion has been presented in Chapter 3 of this report regarding why cohesion intercept values for granular materials are expected and must be taken into account in shear strength analysis. Hvorslev (1960) concluded that the Mohr-Coulomb failure envelope is steepest at low confining pressures and approaches a constant slope at higher confining pressures, thus resulting in a cohesion intercept of zero for granular material. However, work done by Olson (2004) at the University of Texas with normally consolidated
kaolinite supports the concept of having a cohesion intercept, even at very low confining stresses. As a result it is believed that the presence of a cohesion intercept for the tests conducted in this study is valid and the cohesion should not be ignored when determining strength.

Figure 4.31: Cohesion intercept vs. percentage fines - Sharps quarry.
Figure 4.32: Cohesion intercept vs. percentage fines- Preston quarry.

Figure 4.33: Cohesion intercept vs. percentage fines- Black Rock quarry.
Figure 4.34: Cohesion intercept vs. percentage fines- Glen Rose quarry.

Figure 4.35: Cohesion intercept vs. percentage fines- Granite Mountain quarry.
Since the cohesion intercept values reported here were not 0 or another recurring value that could be established as the standard for this testing regimen, it was necessary to evaluate the effect of fines on the aggregate shear strength in a manner that combined both the angle of internal friction, $\phi$, and the cohesion intercept, $c$. A typical pavement section consisting of 4” of asphalt concrete hot mix underlain by 8” of AHTD Class 7 crushed stone aggregate base course underlain by a very stiff silty clay subgrade ($M_R = 6,000$ psi) was evaluated using the Kenlayer ® software package to determine a typical bulk stress, $\sigma$, value for the aggregate base course layer. The analysis indicated a bulk stress on the order of 206.8 kPa (30 psi) represents the stress condition developed under a 4,082 kg (9,000 lb) wheel load at a tire pressure of 689.5 kPa (100 psi). Using this bulk stress and Eqn. 4.6 the shear strength was determined for each quarry and plotted as a function of the fines content.

$$\tau = c + \sigma \tan \phi$$  \hspace{1cm} (4.6)

Where
- $\tau$ = shear strength
- $c$ = cohesion intercept
- $\sigma$ = bulk stress (established as 276 kPa (40 psi))
- $\phi$ = angle of internal friction.

The shear strength vs. percent fines plots illustrated in Figs. 4.36 through 4.40 show that the effect of fines on the aggregate specimens differed among quarries and parent geological materials.

The limestone material from Sharps quarry (Fig. 4.36) generally displayed decreasing shear strength as the percentage of fines increased from 6% to 16%. The upper boundary blend specimens have the lowest shear strength of all the Sharps quarry test specimens. The decreasing shear strength was most pronounced from 8% to 16%
fines. Among all the materials tested, Sharps quarry exhibited the greatest variation in shear strength for as a function of fines contents; but this data also exhibit the greatest amount of scatter among all quarries. Sharps quarry, as previously stated, was the first material tested while the testing regimen and protocol were being finalized. For evaluating shear strength performance of Class 7 base over differing fines contents the remaining Preston, Black Rock, Glen Rose, and Granite Mountain materials should be the most scrutinized for performance trends.

The Preston quarry shear strength (Fig. 4.37) was seen to be essentially the same for the 6% and 8% blends. A slight amount of scatter in shear strengths between the replicate 12%, 14%, and 16% specimens was observed for the sandstone material. Using an average value for each of these blends the shear strength is seen to decrease slightly with additional increasing fines. The widest variation in shear strength between replicate samples occurred for the 16% fines blend. Review of the Mohr-Coulomb failure envelope for the 16% A specimen shows that the second Mohr’s stress circle used to define the envelope was slightly out of line with the first and third circles. In lieu of manipulating the failure envelope to match that developed for the B specimen, an average failure path for the three circles was drawn. The lower and upper boundary blend specimens containing 6% and 10% fines, respectively, produced shear strengths similar to those obtained from model blend specimens with the same 6% and 10% fines content.

When averaging strength values for replicate specimens from the Sharps and Preston quarries a trend of increasing shear strength for fines contents up to 8% or 10% was followed by a slight decrease in strength with the addition of fines. It should be
noted that the shear strength for Sharps model blends at 12% fines was equal to that developed at the 10% fines content. The Preston shear strength at 12% fines was similarly close to that obtained by the 8% and 10% specimens. The variation in strength observed for fines contents ranging from 8% to 12% was nearly constant and certainly within the tolerance of the data scatter.

Figure 4.36: Shear strength vs. percentage fines - Sharps quarry.
The Black Rock quarry dolomite had shear strengths (Fig. 4.38) that were generally constant across the range of fines with very little change from 6% to 14% fines. Additionally, little scatter was observed between replicate specimens. The lowest shear strength occurred at 16% fines. The upper and lower boundary blend specimens had shear strengths in line with the model blend specimens of similar fines content. Shear strength for the Black Rock dolomite appears almost indifferent to the varying blends with even the upper and lower boundary blends closely agreeing with the model blends. The Black Rock shear strength remained at or near 345 kPa (50 psi) for every specimen blend tested. The highest shear strengths, although by only a slight amount, were for model blend gradations with 6% and 12% fines.

The shear strength of the Glen Rose aggregate (Fig. 4.39) remained essentially constant from 6% to 12% fines and then decreased at 14% fines. A slight increase in
shear strength was observed at 16% fines. As with the Preston and Black Rock specimens, the upper and lower boundary blend specimens develop shear strengths in line with model blend gradations having the same 6% and 10% fines contents. The pattern of shear strength “dipping” at 14% and then rebounding at 16% fines differs from the trend exhibited by Sharps, Preston, and Black Rock quarry.

The shear strength of the Granite Mountain syenite (Fig. 4.40) followed the same trend as the Glen Rose material. The shear strength remained fairly constant from 6% to 12% fines and then dropped to the lowest values at 14% fines. Unlike the Preston, Black Rock, and Glen Rose aggregates, the upper and lower boundary blend specimens for Granite Mountain had shear strengths that were not in line with the model blend shear strengths.

The Glen Rose and Granite Mountain shear strength trends were analogous to one another over the range of fines investigated in this study as were the Sharps, Preston and Black Rock trends. In an attempt to explain the reasons for the two distinctly different trends the grain size distribution for all quarries was investigated. The percentage of material passing the 0.425 mm (No. 40) sieve for Glen Rose Class 7 was over twice the amount for the other quarries. Glen Rose aggregate had 40% passing the 0.425 mm (No. 40) sieve compared to 15% (Sharps), 11% (Preston), and 15% (Black Rock). Although not as pronounced, Granite Mountain had an increased amount of minus 0.425 mm (No. 40) material (20%) when compared to the Sharps, Preston, and Black Rock quarries. It is possible that the presence of increased fine sands, which exhibited some plasticity, produced the similarities in shear strength patterns over the range of fines contents for Glen Rose and Granite Mountain quarries.
The lower (coarser) AHTD boundary blends exhibited shear strengths above those developed by the model blend gradation samples while the upper (finer) AHTD boundary blends had shear strengths well below the model blend shear strengths. The low shear strengths for the upper boundary blends is attributed to having a decreased amount of plus 4.75 mm (No. 4) aggregate and an increased percentage of fines. These blends lacked coarse particle point-to-point contact and the additional fines further decreased aggregate interlock. This condition is as Yoder and Witczak (1975) described for aggregate blends that contain excessive fines. This boundary blend trend was observed for each quarry.

![Graph](image)

Figure 4.38: Shear strength vs. percentage fines- Black Rock quarry.
Figure 4.39: Shear strength vs. percentage fines- Glen Rose quarry

Figure 4.40: Shear strength vs. percentage fines- Granite Mountain quarry.
The model blend gradation data used to generate Figs. 4.26 through 4.40 along with statistical parameters is presented by quarry in Tables 4.12 through 4.16. The maximum, minimum, average, and deviation from overall mean for each of the parameters analyzed (ϕ, c, and τ) were calculated. The mean angle of internal friction, ϕ, values for each quarry ranged from 40.6 (Preston) to 43.3 (Black Rock). Within the individual quarries the deviation from the overall mean was never more than 4 degrees over the range of blends tested. The average deviation from the quarry mean values was more typically on the order of 1.5 to 2 degrees.
Table 4.12: Summary of $\phi$, $c$, and $\tau$ model blend triaxial testing results- Sharps quarry.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\phi$ (degrees)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$c$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$\tau$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>6% A</td>
<td>38.2</td>
<td>38.9, 38.2</td>
<td>38.6</td>
<td>-3.2</td>
<td>19</td>
<td>20.0, 19.0</td>
<td>19.5</td>
<td>5.2</td>
<td>42.6</td>
<td>52.3, 50.5</td>
<td>43.4</td>
<td>2.1</td>
</tr>
<tr>
<td>6% B</td>
<td>38.9</td>
<td>20</td>
<td>44.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8% A</td>
<td>44.2</td>
<td>50.2, 37.4</td>
<td>43.9</td>
<td>2.2</td>
<td>20</td>
<td>24.4, 11.0</td>
<td>18.5</td>
<td>4.1</td>
<td>49.2</td>
<td>59, 55</td>
<td>47.8</td>
<td>6.5</td>
</tr>
<tr>
<td>8% B</td>
<td>37.4</td>
<td>24.4</td>
<td></td>
<td></td>
<td>11</td>
<td>14.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>47.3</td>
<td></td>
</tr>
<tr>
<td>8% B2</td>
<td>50.2</td>
<td>11</td>
<td>47.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10% A</td>
<td>46.3</td>
<td>46.3, 44.6</td>
<td>45.5</td>
<td>3.7</td>
<td>9.1</td>
<td>10.5, 9.1</td>
<td>9.8</td>
<td>-4.5</td>
<td>40.5</td>
<td>51, 49.9</td>
<td>40.3</td>
<td>-1.0</td>
</tr>
<tr>
<td>10% B</td>
<td>44.6</td>
<td>10.5</td>
<td>40.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12% A</td>
<td>44.7</td>
<td>44.7, 40.9</td>
<td>42.8</td>
<td>1.1</td>
<td>13.6</td>
<td>13.6, 13.0</td>
<td>13.3</td>
<td>-1.0</td>
<td>43.3</td>
<td>53.2, 47.6</td>
<td>41.1</td>
<td>-0.2</td>
</tr>
<tr>
<td>12% B</td>
<td>40.9</td>
<td>13</td>
<td>39.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14% A</td>
<td>38.7</td>
<td>40.9, 38.7</td>
<td>39.8</td>
<td>-1.9</td>
<td>12.7</td>
<td>13.7, 12.7</td>
<td>13.2</td>
<td>-1.1</td>
<td>36.7</td>
<td>48.3, 44.7</td>
<td>38.2</td>
<td>-3.1</td>
</tr>
<tr>
<td>14% B</td>
<td>40.9</td>
<td>13</td>
<td>39.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16% A</td>
<td>39.4</td>
<td>40.5, 39.4</td>
<td>40.0</td>
<td>-1.8</td>
<td>12.7</td>
<td>12.7, 10.9</td>
<td>11.8</td>
<td>-2.5</td>
<td>37.3</td>
<td>45.6, 45.1</td>
<td>36.9</td>
<td>-4.4</td>
</tr>
<tr>
<td>16% B</td>
<td>40.5</td>
<td>10.9</td>
<td>36.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Overall mean</strong></td>
<td><strong>41.7</strong></td>
<td><strong>41.7</strong></td>
<td><strong>41.3</strong></td>
<td></td>
<td><strong>41.3</strong></td>
<td><strong>41.3</strong></td>
<td><strong>41.3</strong></td>
<td></td>
<td><strong>41.3</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.13: Summary of $\phi$, $c$, and $\tau$ model blend triaxial testing results- Preston quarry.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\phi$ (degrees)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$c$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$\tau$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>6% A</td>
<td>40.9</td>
<td>40.9, 34.5</td>
<td>37.7</td>
<td>-2.9</td>
<td>16</td>
<td>19.1, 16.0</td>
<td>17.6</td>
<td>3.5</td>
<td>42.0</td>
<td>50.6, 46.6</td>
<td>40.9</td>
<td>1.0</td>
</tr>
<tr>
<td>6% B</td>
<td>34.5</td>
<td></td>
<td>39.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8% A</td>
<td>41.2</td>
<td>41.2, 36.6</td>
<td>38.9</td>
<td>-1.7</td>
<td>14.7</td>
<td>21.0, 14.7</td>
<td>17.9</td>
<td>3.8</td>
<td>41.0</td>
<td>50.7, 49.7</td>
<td>42.1</td>
<td>2.3</td>
</tr>
<tr>
<td>8% B</td>
<td>36.6</td>
<td>21</td>
<td>43.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10% A</td>
<td>38.8</td>
<td>41.9, 38.8</td>
<td>40.4</td>
<td>-0.2</td>
<td>17</td>
<td>17.0, 13.7</td>
<td>15.4</td>
<td>1.3</td>
<td>41.1</td>
<td>49.6, 49.2</td>
<td>40.9</td>
<td>1.0</td>
</tr>
<tr>
<td>10% B</td>
<td>41.9</td>
<td>13.7</td>
<td>40.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12% A</td>
<td>41.9</td>
<td>41.9, 40.3</td>
<td>41.1</td>
<td>0.5</td>
<td>10.9</td>
<td>16.8, 10.9</td>
<td>13.9</td>
<td>-0.2</td>
<td>37.1</td>
<td>50.7, 48.8</td>
<td>40.0</td>
<td>0.2</td>
</tr>
<tr>
<td>12% B</td>
<td>40.3</td>
<td>16.8</td>
<td>42.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14% A</td>
<td>42.1</td>
<td>43.6, 42.1</td>
<td>42.9</td>
<td>2.3</td>
<td>8.2</td>
<td>11.0, 8.2</td>
<td>9.6</td>
<td>-4.4</td>
<td>35.3</td>
<td>49.1, 44.3</td>
<td>37.4</td>
<td>-2.4</td>
</tr>
<tr>
<td>14% B</td>
<td>43.6</td>
<td>11</td>
<td>39.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16% A</td>
<td>45</td>
<td>45.0, 40.1</td>
<td>42.6</td>
<td>2.0</td>
<td>10</td>
<td>10.0, 10.0</td>
<td>10.0</td>
<td>-4.0</td>
<td>40.0</td>
<td>50.0, 43.7</td>
<td>37.6</td>
<td>-2.2</td>
</tr>
<tr>
<td>16% B</td>
<td>40.1</td>
<td>10</td>
<td>36.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Overall mean</strong></td>
<td><strong>40.6</strong></td>
<td><strong>40.6</strong></td>
<td><strong>39.8</strong></td>
<td></td>
<td><strong>39.8</strong></td>
<td><strong>39.8</strong></td>
<td><strong>39.8</strong></td>
<td></td>
<td><strong>39.8</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4.14: Summary of $\phi$, $c$, and $\tau$ model blend triaxial testing results- Black Rock quarry.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\phi$ (degrees)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$c$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$\tau$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>6% A</td>
<td>43.9</td>
<td>43.9, 34.5</td>
<td>39.2</td>
<td>-4.1</td>
<td>13</td>
<td>23.0, 13.0</td>
<td>18</td>
<td>5.9</td>
<td>41.9</td>
<td>51.5, 50.5</td>
<td>42.7</td>
<td>2.3</td>
</tr>
<tr>
<td>6% B</td>
<td>34.5</td>
<td>23</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8% A</td>
<td>41.6</td>
<td>42.5, 41.6</td>
<td>42.05</td>
<td>-1.2</td>
<td>14</td>
<td>14.0, 13.7</td>
<td>13.85</td>
<td>1.8</td>
<td>40.6</td>
<td>50.4, 49.5</td>
<td>40.9</td>
<td>0.5</td>
</tr>
<tr>
<td>8% B</td>
<td>42.5</td>
<td>13.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10% A</td>
<td>44.3</td>
<td>45.0, 44.3</td>
<td>44.65</td>
<td>1.4</td>
<td>10.5</td>
<td>10.5, 10.5</td>
<td>10.5</td>
<td>-1.6</td>
<td>39.8</td>
<td>50.5, 49.5</td>
<td>40.1</td>
<td>-0.3</td>
</tr>
<tr>
<td>10% B</td>
<td>45</td>
<td>10.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12% A</td>
<td>45.4</td>
<td>46.2, 45.4</td>
<td>45.8</td>
<td>2.5</td>
<td>9.5</td>
<td>9.5, 9.5</td>
<td>9.5</td>
<td>-2.6</td>
<td>39.9</td>
<td>51.2, 50.1</td>
<td>40.4</td>
<td>-0.1</td>
</tr>
<tr>
<td>12% B</td>
<td>46.2</td>
<td>9.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14% A</td>
<td>46.5</td>
<td>46.5, 44.3</td>
<td>45.4</td>
<td>2.1</td>
<td>8.4</td>
<td>10.5, 8.4</td>
<td>9.45</td>
<td>-2.6</td>
<td>40.0</td>
<td>50.6, 49.5</td>
<td>39.9</td>
<td>-0.6</td>
</tr>
<tr>
<td>14% B</td>
<td>44.3</td>
<td>10.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16% A</td>
<td>43.5</td>
<td>43.5, 41.8</td>
<td>42.65</td>
<td>-0.6</td>
<td>10.5</td>
<td>11.6, 10.5</td>
<td>11.05</td>
<td>-1.0</td>
<td>39.0</td>
<td>48.5, 47.4</td>
<td>38.7</td>
<td>-1.8</td>
</tr>
<tr>
<td>16% B</td>
<td>41.8</td>
<td>11.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Overall mean 43.3

Table 4.15: Summary of $\phi$, $c$, and $\tau$ model blend triaxial testing results- Glen Rose quarry.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\phi$ (degrees)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$c$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>$\tau$ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>6% A</td>
<td>42.9</td>
<td>42.9, 41.8</td>
<td>42.4</td>
<td>-0.125</td>
<td>9.5</td>
<td>10.5, 9.5</td>
<td>10.0</td>
<td>2.3</td>
<td>37.4</td>
<td>46.7, 46.3</td>
<td>37.4</td>
<td>2.1</td>
</tr>
<tr>
<td>6% B</td>
<td>41.8</td>
<td>10.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8% A</td>
<td>42.5</td>
<td>42.5, 40.9</td>
<td>41.7</td>
<td>-0.775</td>
<td>7.4</td>
<td>9.5, 7.4</td>
<td>8.5</td>
<td>0.7</td>
<td>34.9</td>
<td>44.1, 44.1</td>
<td>35.2</td>
<td>0.0</td>
</tr>
<tr>
<td>8% B</td>
<td>40.9</td>
<td>9.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10% A</td>
<td>43.6</td>
<td>43.6, 43.2</td>
<td>43.4</td>
<td>0.925</td>
<td>7.4</td>
<td>7.4, 7.4</td>
<td>7.4</td>
<td>-0.3</td>
<td>36.0</td>
<td>45.5, 45.0</td>
<td>35.8</td>
<td>0.6</td>
</tr>
<tr>
<td>10% B</td>
<td>43.2</td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12% A</td>
<td>41.4</td>
<td>43.6, 41.4</td>
<td>42.5</td>
<td>0.025</td>
<td>6.3</td>
<td>8.4, 6.3</td>
<td>7.4</td>
<td>-0.4</td>
<td>32.7</td>
<td>46.5, 41.6</td>
<td>34.9</td>
<td>-0.4</td>
</tr>
<tr>
<td>12% B</td>
<td>43.6</td>
<td>8.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14% A</td>
<td>40.9</td>
<td>41.9, 40.9</td>
<td>41.4</td>
<td>-1.075</td>
<td>6</td>
<td>7.4, 6.0</td>
<td>6.7</td>
<td>-1.0</td>
<td>32.0</td>
<td>43.3, 40.6</td>
<td>33.2</td>
<td>-2.1</td>
</tr>
<tr>
<td>14% B</td>
<td>41.9</td>
<td>7.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16% A</td>
<td>40.5</td>
<td>46.5, 40.5</td>
<td>43.5</td>
<td>1.025</td>
<td>7.4</td>
<td>7.4, 5.3</td>
<td>6.4</td>
<td>-1.4</td>
<td>33.0</td>
<td>47.5, 41.6</td>
<td>35.0</td>
<td>-0.2</td>
</tr>
<tr>
<td>16% B</td>
<td>46.5</td>
<td>5.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Overall mean 42.5

Table 4.14 Summary of $\phi$, $c$, and $\tau$ model blend triaxial testing results- Black Rock quarry.

Overall mean 42.5 Overall mean 7.7 Overall mean 35.2
<table>
<thead>
<tr>
<th>Specimen</th>
<th>φ (degrees)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>c (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
<th>τ (psi)</th>
<th>Max, Min</th>
<th>Mean</th>
<th>Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>6% A</td>
<td>41.6</td>
<td>41.6, 41.6</td>
<td>41.6</td>
<td>-1.5</td>
<td>12.6</td>
<td>12.6, 12.6</td>
<td>12.6</td>
<td>2.0</td>
<td>39.2</td>
<td>48.1, 48.1</td>
<td>39.2</td>
<td>0.5</td>
</tr>
<tr>
<td>6% B</td>
<td>41.6</td>
<td>12.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>39.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8% A</td>
<td>46.9</td>
<td>46.9, 43.5</td>
<td>45.2</td>
<td>2.1</td>
<td>7.4</td>
<td>11.6, 7.4</td>
<td>9.5</td>
<td>-1.1</td>
<td>39.5</td>
<td>50.1, 49.6</td>
<td>39.8</td>
<td>1.1</td>
</tr>
<tr>
<td>8% B</td>
<td>43.5</td>
<td>11.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>40.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10% A</td>
<td>44.3</td>
<td>45.7, 44.3</td>
<td>45</td>
<td>1.9</td>
<td>11.6</td>
<td>11.6, 8.4</td>
<td>10</td>
<td>-0.6</td>
<td>40.9</td>
<td>50.6, 49.4</td>
<td>40.0</td>
<td>1.3</td>
</tr>
<tr>
<td>10% B</td>
<td>45.7</td>
<td>8.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>39.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12% A</td>
<td>45</td>
<td>45.0, 41.6</td>
<td>43.3</td>
<td>0.2</td>
<td>9.5</td>
<td>12.6, 9.5</td>
<td>11.05</td>
<td>0.5</td>
<td>39.5</td>
<td>49.5, 48.1</td>
<td>39.4</td>
<td>0.7</td>
</tr>
<tr>
<td>12% B</td>
<td>41.6</td>
<td>12.6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>39.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14% A</td>
<td>40.6</td>
<td>40.6, 40.3</td>
<td>40.45</td>
<td>-2.65</td>
<td>10.5</td>
<td>10.5, 10.5</td>
<td>10.5</td>
<td>-0.1</td>
<td>36.2</td>
<td>44.8, 44.4</td>
<td>36.1</td>
<td>-2.6</td>
</tr>
<tr>
<td>14% B</td>
<td>40.3</td>
<td>10.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>35.9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16% A</td>
<td>42.9</td>
<td>43.2, 42.9</td>
<td>43.05</td>
<td>-0.05</td>
<td>9.5</td>
<td>10.0, 9.5</td>
<td>9.75</td>
<td>-0.8</td>
<td>37.4</td>
<td>47.6, 46.7</td>
<td>37.8</td>
<td>-0.9</td>
</tr>
<tr>
<td>16% B</td>
<td>43.2</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>38.2</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Overall mean | 43.1 | Overall mean | 10.6 | Overall mean | 38.7 |

Table 4.16: Summary of φ, c, and τ model blend triaxial testing results- Granite Mountain quarry.
4.8 Comparison of Elastic and Resilient Modulus Test Results

Resilient modulus testing in accordance with AASHTO T-307 on compacted specimens from the same quarries was conducted as part of another study and were completed by others (Zhao, 2004). The resilient modulus test specimens were each created using material with the “as-received” gradation. These “as-received” gradations were very similar to the model blends at 10% fines that were tested in this study. Resilient modulus values were compared to the elastic modulus values obtained from the staged load triaxial testing. In order to make a direct comparison, the modulus values had to be compared at the same bulk stresses. Elastic modulus values were calculated from each of the staged triaxial shearing test results for the model blend 10% fines gradations.

Bulk stress, $\theta$, values were determined for each stage of testing using Eqn. 4.7:

$$\theta = 3\sigma_3 + \sigma_{d(failure)}$$

where $\sigma_3 =$ the stage confining stress

$\sigma_d =$ the average secant stress prior to stage yielding.

Equation 4.7 is also used for determining bulk stress in AASHTO T-307 resilient modulus testing. Table 4.17 presents a summary of the modulus values, both cyclic and staged triaxial, and the corresponding bulk stress values for tests conducted on the 10% model blends from each quarry. Plots for the cyclic modulus values obtained from the low-stress cyclic triaxial testing are depicted previously in Figs. 4.19 through 4.23. Plots comparing the secant modulus values from the staged-load triaxial tests to the resilient modulus values obtained by Zhao (2004) are shown for each quarry in Figs. 4.41 through 4.45. Data trendlines along with their equations and coefficients of determination ($R^2$)
are shown on each plot for resilient modulus values (represented by the filled diamond symbol) and staged triaxial modulus values (open triangle symbol).

Table 4.17: Bulk stress and triaxial secant modulus summary data.

<table>
<thead>
<tr>
<th>Quarry</th>
<th>Blend</th>
<th>σ₃ (psi)</th>
<th>σ₃avg (psi)</th>
<th>Bulk stress, $\theta = 3\sigma_3 + \sigma_{3\text{avg}}$ (psi)</th>
<th>Staged Triaxial Modulus (psi)</th>
<th>Cyclical Triaxial Modulus (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sharps M.B. 10% A</td>
<td>5</td>
<td>33.5</td>
<td>48.5</td>
<td>4000</td>
<td>9065</td>
<td></td>
</tr>
<tr>
<td>Sharps M.B. 10% A</td>
<td>10</td>
<td>47</td>
<td>77</td>
<td>9231</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sharps M.B. 10% B</td>
<td>5</td>
<td>34.5</td>
<td>49.5</td>
<td>5000</td>
<td>10205</td>
<td></td>
</tr>
<tr>
<td>Sharps M.B. 10% B</td>
<td>10</td>
<td>44</td>
<td>74</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sharps M.B. 10% B</td>
<td>20</td>
<td>60.5</td>
<td>120.5</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Preston M.B. 10% A</td>
<td>5</td>
<td>34.5</td>
<td>49.5</td>
<td>2759</td>
<td>6254</td>
<td></td>
</tr>
<tr>
<td>Preston M.B. 10% A</td>
<td>10</td>
<td>45</td>
<td>75</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Preston M.B. 10% A</td>
<td>20</td>
<td>67</td>
<td>127</td>
<td>13333</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Preston M.B. 10% B</td>
<td>5</td>
<td>36</td>
<td>51</td>
<td>4211</td>
<td>8891</td>
<td></td>
</tr>
<tr>
<td>Preston M.B. 10% B</td>
<td>10</td>
<td>47.5</td>
<td>77.5</td>
<td>12000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Preston M.B. 10% B</td>
<td>20</td>
<td>64</td>
<td>124</td>
<td>13333</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black Rock M.B. 10% A</td>
<td>5</td>
<td>19</td>
<td>34</td>
<td>4000</td>
<td>11263</td>
<td></td>
</tr>
<tr>
<td>Black Rock M.B. 10% A</td>
<td>10</td>
<td>42</td>
<td>72</td>
<td>10667</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black Rock M.B. 10% A</td>
<td>20</td>
<td>61</td>
<td>121</td>
<td>13333</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black Rock M.B. 10% B</td>
<td>5</td>
<td>20</td>
<td>35</td>
<td>5000</td>
<td>10831</td>
<td></td>
</tr>
<tr>
<td>Black Rock M.B. 10% B</td>
<td>10</td>
<td>43</td>
<td>73</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black Rock M.B. 10% B</td>
<td>20</td>
<td>62.5</td>
<td>122.5</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glen Rose M.B. 10% A</td>
<td>5</td>
<td>23</td>
<td>38</td>
<td>2000</td>
<td>10257</td>
<td></td>
</tr>
<tr>
<td>Glen Rose M.B. 10% A</td>
<td>10</td>
<td>30.5</td>
<td>60.5</td>
<td>8000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glen Rose M.B. 10% A</td>
<td>20</td>
<td>49.5</td>
<td>109.5</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glen Rose M.B. 10% B</td>
<td>5</td>
<td>19.5</td>
<td>34.5</td>
<td>2222</td>
<td>10687</td>
<td></td>
</tr>
<tr>
<td>Glen Rose M.B. 10% B</td>
<td>10</td>
<td>31</td>
<td>61</td>
<td>8000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glen Rose M.B. 10% B</td>
<td>20</td>
<td>49</td>
<td>109</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite Mountain M.B. 10% A</td>
<td>5</td>
<td>36</td>
<td>51</td>
<td>1790</td>
<td>8909</td>
<td></td>
</tr>
<tr>
<td>Granite Mountain M.B. 10% A</td>
<td>10</td>
<td>43.5</td>
<td>73.5</td>
<td>12000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite Mountain M.B. 10% A</td>
<td>20</td>
<td>69.5</td>
<td>129.5</td>
<td>11429</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite Mountain M.B. 10% B</td>
<td>5</td>
<td>33.5</td>
<td>48.5</td>
<td>1860</td>
<td>11270</td>
<td></td>
</tr>
<tr>
<td>Granite Mountain M.B. 10% B</td>
<td>10</td>
<td>41.5</td>
<td>71.5</td>
<td>8571</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite Mountain M.B. 10% B</td>
<td>20</td>
<td>68.5</td>
<td>128.5</td>
<td>10000</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 4.41: Comparison of cyclic, staged, and resilient modulus values- Sharps quarry.

Figure 4.42: Comparison of cyclic, staged, and resilient modulus values- Preston quarry.
Figure 4.43: Comparison of cyclic, staged, and resilient modulus values- Black Rock quarry.

Figure 4.44: Comparison of cyclic, staged, and resilient modulus values- Glen Rose quarry.
The dashed trendline shown on each plot is the result of using a power function to create a correlation between the trendline for the modulus values obtained from staged triaxial testing to the trendline for the resilient modulus values obtained by Zhao. Resilient modulus values may be estimated from staged triaxial modulus values at the same bulk stresses by inserting the bulk stress into the appropriate power function. The equations for the power function trendline for all quarries are of the format shown in Eqn. 4.6:

\[ y = Ax^B \]  

where \( y \) = resilient modulus value, ksi

\( A \) = coefficient value

\( x \) = bulk stress value, psi

\( B \) = coefficient value.
Review of the plots shows the A coefficients varying between 1.8121 and 5.2123 for the five quarry materials and the B coefficients varying from 0.1704 to 0.2328.

The B coefficients for each of the five quarries were in close agreement; therefore, an average value on the order of 0.2 is deemed sufficient for defining this parameter for any of the materials tested. The A coefficient for Preston, Glen Rose, and Granite Mountain was fairly constant between 1.8121 and 2.3557 while the Sharps (3.6231) and Black Rock (5.2123) values were higher. The Sharps and Black Rock aggregates are from limestone and dolomite formations, respectively. It is important to note that the resilient modulus scatter was also greatest for these two quarries. Additional resilient modulus testing completed on the Sharps and Black Rock materials could reveal outlier points in the existing data. This would allow for increasing the coefficients of determination and tightening the developed relationship equations among the five quarries. Both are carbonate minerals and the materials from these quarries had the highest maximum dry density values of the materials tested. Two A coefficients are suggested: 2.0 and 4.4. Selection of which A value to use can be based upon maximum dry density (AASHTO T-180, Method D) values. An A value of 2.0 should be used for materials having maximum dry density values less than or equal to approximately 137 lbs/cubic ft and the A value of 4.4 could be used when the maximum dry density value exceeds 137 lbs/cubic ft. These suggested coefficients represent average values that are based upon a very limited number of comparative test results. Obviously, additional testing is required to more completely define the secant modulus trendlines over a wider range of bulk stresses.
Figures 4.41 through 4.45 also portray the low-stress cyclic triaxial modulus values (denoted as shaded circles) at the stage 1 bulk stress values. These cyclic triaxial modulus values plot either with the resilient modulus values or with the staged triaxial elastic secant modulus values. Again, the Sharps and Black Rock quarries have similar trends and the other quarries have similar trends which are different from the Sharps and Black Rock trends. For the Sharps and Black Rock quarries the cyclic modulus tends to group with the staged load secant modulii while the cyclic modulus from the remaining quarries tend to group with the resilient modulus values. The Glen Rose elastic secant modulus values actually plotted on top of the resilient modulus trendline. It was expected that the secant modulus would be lower than the resilient modulus values since the cyclic triaxial loading was conducted at rates of strain that were much lower than those used for resilient modulus testing.

In summary, it appears that the shearing strength of Class 7 material from all of the quarries tested in this study is not materially affected until the percentage of fines reaches 14 percent. Resilient modulus values measured for all quarries would suggest that a layer coefficient ($a_2$) of 0.14 for Class 7 crushed stone aggregate base course material produced in Arkansas is appropriate. Based on the static modulus values measured over a range of bulk stresses and fines contents, it appears that the percentage of fines does not materially affect modulus values in the range of 8 to 14 percent.
Chapter 5

CONCLUSIONS

5.1 Shearing Strength and Modulus

The crushed stone base course layer within a pavement section is designed to provide adequate support for the surficial wearing course by adequately resisting deformation, thus limiting distresses such as rutting and fatigue cracking. The base course must also adequately distribute stresses from anticipated traffic loading to protect the underlying subgrade from load induced damage or failure. Significant strength and stiffness must be developed within the aggregate base layer to function in the required manner. In addition, most pavement designs require the aggregate base to perform a drainage function as well. The most commonly used aggregates used for base course sections in Arkansas meet either AHTD Class 5 or Class 7 requirements.

This study focused on the effect of fines on shear strength and hydraulic conductivity of AHTD Class 7 crushed stone base aggregate. Five quarries were selected to represent the main geologic materials for crushed stone aggregate produced in Arkansas. Index testing was performed on each of the materials to establish the initial specimen preparation parameters. To assess the effect that varying the fines content had on the shear strength of Class 7 base course, consolidated-drained (CD) staged triaxial tests were conducted on at least 16 prepared specimens from each of the sampled quarries. In addition, constant rate of strain cyclic load tests were conducted at low stress levels on each of the test specimens to obtain a modulus value at intermediate strains prior to initiating staged triaxial loading tests to failure. The results from the CD triaxial tests for each of the selected quarries were compared to resilient modulus test data for
each quarry which was determined by others (Zhao, 2004). All comparative test specimens were prepared from the same material stockpile and had similar gradations and preparation procedures.

The results of the laboratory staged triaxial test regimen indicated that:

- The shear strength, $\sigma$, measured by staged triaxial testing of Class 7 base courses that are commonly used in Arkansas remains essentially constant at fines contents ranging from 8% to 12%;
- Maximum shear strengths for all of the aggregates tested typically occurred between fines contents of 8% and 12% with little reduction in shear strength until the 14% fines content;
- A decrease in strength and stiffness was observed for the 14% and 16% fines contents, although shear strengths over the entire range of fines contents studied were within an average of approximately 5% of the maximum values;
- Variance in strength measurements between the replicate test specimens was on the order of 3 to 4%;
- The assumption that granular material tested in triaxial apparatus will develop a Mohr-Coulomb failure envelope plotting through the origin (cohesion intercept, $c$, equal to 0) does not hold true for the aggregate base tested;
- Cyclic modulus values determined from CRS triaxial testing average about 7 ksi lower than those at similar bulk stress, $\sigma$, values developed during
resilient modulus testing due to the difference in testing strains. However, the difference in values was not constant, ranging between 0 and 10 ksi;

- For a given bulk stress, $\sigma$, a relationship can be developed between staged triaxial elastic modulus values and resilient modulus values; and

- The current AHTD specified base course layer coefficient ($a_2$) of 0.14 appears conservative for Class 7 crushed stone aggregate for fines contents ranging between 8% and 12%.

The discussion presented in the literature review (Chapter 2) indicates that an optimal amount of fines exists at which maximum shear strength will be developed through coarse particle point-to-point contact as well as a matrix of fine, relatively incompressible, materials filling void spaces within the material effectively "locking" the coarse particles into place. The testing regimen completed affirms this concept. According to this laboratory study the decrease in shear strength does not occur until a 14% fines content is reached for the majority of the materials tested. This finding suggests that an increase in the allowable percentage of fines at the upper gradation limit to a maximum content of 12% will have little, if any, effect on the shear strength of Class 7 material which is in compliance with the other gradation and plasticity requirements specified in the AHTD Construction Specifications. In fact the shear strength for Black Rock quarry material was essentially indifferent to the percentage of fines over the entire range of contents examined.

The results of this study indicate that a relationship exists between elastic modulus values developed from staged triaxial shear strength testing and resilient modulus values obtained in accordance with AASHTO T-307 guidelines. As expected,
the resilient modulus values obtained via T-307 testing were higher than the elastic modulus values obtained from controlled rate of strain cyclic tests or strain controlled staged triaxial testing. On a quarry-by-quarry basis, linear trendlines fitted through the test data on log–log plots of modulus versus bulk stress were essentially parallel to one another for each testing method. This is a significant finding because it is possible for one to estimate resilient modulus values from staged triaxial testing results at similar bulk stresses. Based upon a limited number of tests, Eqn. 5.1 has been developed for estimating resilient modulus values:

\[ y = Ax^{0.21} \]  

(5.1)

where \( y \) = estimated resilient modulus value, ksi

\( A \) = material dependent coefficient value

\( x \) = bulk stress value, psi.

The A coefficient value is based upon the type of material from which the aggregate base is derivative. Determining the maximum dry density according to AASHTO T-180, Method D will provide further guidance about which A value to select. This limited study indicates that stronger aggregate such as limestone and dolomite which have maximum dry densities greater than 137 lbs/cubic ft should have an A coefficient of 4.4. An A coefficient of 2.0 should be used for aggregates with maximum dry density values less than or equal to 137 lbs/cubic ft.

5.2 Hydraulic Conductivity and Drainage

According to Cedegren, pavements that are built without effective drainage deteriorate at much faster rates than if they were well drained (Cedergren, 1974). The
primary cause of this premature failure is the elevated pore water pressures that result from traffic loading and the resulting loss of strength and stiffness of the base course. This loss of strength in the base, and even in subgrade, results in premature pavement rutting and cracking in flexible pavements and faulting in rigid pavements. The literature documents that pavement structures supported by well draining base courses require less maintenance and have an extended service life (Bowders, 2003; Cedergren, 1994).

This study focused on the material property that is central to the concept of drainage, which is hydraulic conductivity. Index testing was performed on the “as received” gradations to establish the initial specimen preparation parameters for materials from each of the five (5) target quarries. To assess the effect that varying the fines content had on the hydraulic conductivity of Class 7 base course, constant head or falling head permeability tests were conducted on fourteen (14) prepared specimens from each of the sampled quarries. In addition, a simplified suction test was run on fourteen (14) specimens from each of the five quarries with specimen parameters that paralleled those of the hydraulic conductivity testing. A total of one hundred and forty (140) tests were conducted in this study.

The results of the laboratory hydraulic conductivity testing indicated that:

- Class 7 base course that is commonly used in Arkansas has a range of hydraulic conductivity of about $10^{-3}$ to $10^{-7}$ cm/sec.

- Class 7 base course used in Arkansas has hydraulic conductivity values that are from 2 to-6 orders of magnitude lower than that of “freely draining” material.
Studies cited in the literature review as well as laboratory testing from the current study shows that as the percentage of fines increases the hydraulic conductivity of the base course decreases. However, this relationship was found not to be linear throughout the range of fines contents examined in this study. According to the current laboratory study, hydraulic conductivity drops quickly with increases in fines until about 8 to 10 percent fines. However, for fines contents of 10 percent and greater the decrease in hydraulic conductivity in relation to the increase in fines is only slight. This finding suggests that any increase in the percentage of fines from 10 to as high as 14 percent will have little, if any, effect on the hydraulic conductivity of the granular base course.

The current study, in agreement with those cited in the literature review, determined that the suction of a granular base course increases as the percentage of fines increases with almost a linear relationship. It was significant to note the gain in total moisture for 8, 10, and 12 percent fines was virtually the same, while 6 percent and 16 percent fines have a very noticeable difference in moisture gain. However, the total moisture gain due to a 10 percent increase in the percentage of fines from 6 to 16 percent was only about 1 percent which suggests that suction testing is not sensitive to increases in fines content over the range expected in Arkansas bases. As such it is not considered to be a useful parameter when setting an upper limit on the amount of fines to be allowed.

The simplified drainage analysis, which presented a best-case scenario for pavement drainage suggests that the layer coefficient modifiers for Arkansas crushed stone bases, should be less than one. However, work by Dupree (2000) suggested that crushed stone bases in Arkansas are performing slightly better than design equations predict. The
conclusion to be drawn here is that the procedures in the AASHTO design guide (1993) to incorporate drainage conditions are flawed.

The results of this study indicate that a modest increase in the percentage of fines allowed in Class 7 granular base course will not significantly affect the performance of the base course when the aspect of drainability of the pavement system is considered. It is recommended that strength, and not drainability, be considered when determining the upper limits on fines in granular base course used in the state of Arkansas.

5.3 Recommendations for future Work

Shearing Strength

All testing in this study was conducted on saturated test specimens. Future work on the subject matter should include specimens at varying degrees of saturation in order to better model base course section in-service conditions. Work completed by Lawrence (2004) on the permeability of base course aggregates with varying amounts of fines could be incorporated to determine the likely field saturation level as related to fines percentage. Strength testing could then be conducted at the likely in-service degrees of saturation.

While the effect of increased fines on shear strength is well documented by this study, the relationship(s) between elastic modulus values obtained from staged triaxial and standard resilient modulus values obtained following the AASHTO T-307 procedure requires additional investigation. The current relationship is based upon a very limited number of cyclic triaxial specimens. A pilot study which could utilize numerous replicate samples for conventional resilient modulus and staged triaxial testing and subsequent elastic modulus calculations is necessary to further define any relationship
between the two. Auxiliary studies also could be extended to include materials other than crushed stone aggregate base course.

**Hydraulic Conductivity**

While the effect of increased fines on hydraulic conductivity is well documented by this study and those studies cited in the literature, the effect of the drainability of the base course on the performance of the pavement system is not as clear. Further laboratory studies are not recommended to resolve this issue.

However, a well controlled pilot study which could create precisely defined drainage conditions for a well used roadway would yield valuable insight on the affects of drainability on pavement performance. If instrumented field test sections could be built with varying percentages of fines in the granular base course ranging from 0-16 percent definitive information regarding key parameters such as time to drain, moisture content, porewater pressures, and deformations in both the base and subbase could be obtained. The effect of stabilizing open graded bases and the use of a geocomposite drainage layer could also be investigated. The data from these test sections could also quantify the effect of drainability on stripping, rutting, cracking and other mechanisms of pavement distress.
REFERENCES


Dennis, N.D., 2004. Personnel Communications, Department of Civil Engineering, University of Arkansas.


Guthrie, W. S., P.M.. Ellis, T. Scullion, (2001) Repeatability and Reliability of the Tube Suction Test, Transportation Research Record 1772, Paper No. 01-2486

Hazen, A., (1892) Some Physical Properties of Sands and Gravels, Annual Report of the State Board of Health of Massachusetts

Hazen, A., (1911), Transactions of the American Society of Civil Engineers


Olson, R.E., 2004. Personnel Communications, Department of Civil Engineering, University of Texas- Austin.


225
Scullion, Tom, I. Syed, R. Randolph, (2000), Tube Suction Test for Evaluating Aggregate Base Materials in Frost and Moisture-Susceptible Environments, Transportation Research Record # 1709

Scullion, T., T. Saarenketo, (1997), Using Suction and Dielectric Measurements as Performance Indicators for Aggregate Base Materials, Transportation Research Record # 1577


APPENDIX A

MATERIAL PROPERTIES
APPENDIX A.1

Historical gradations, upper and lower AHTD gradation limits, and model blend for Class 7 base material coming from the Preston, Sharp’s, Black Rock, Glen Rose and Granite Mountain quarries
Figure A.1.1 – Historical gradations, upper and lower AHTD gradation limits and model blend for Class 7 base material coming from the Sharp's Quarry in Benton County, Arkansas (Limestone)
PRESTON QUARRY GRADATIONS

Figure A.1.2 – Historical gradations, upper and lower AHTD gradation limits and model blend for Class 7 base material coming from the Preston Quarry in Crawford County, Arkansas (Sandstone)
Figure A.1.3 – Historical gradations, upper and lower AHTD gradation limits and model blend for Class 7 base material coming from the Glen Rose Quarry in Hot Springs County, Arkansas (Novaculite)
Figure A.1.4 – Historical gradations, upper and lower AHTD gradation limits and model blend for Class 7 base material coming from the Black Rock Quarry in Lawrence County, Arkansas (Dolomite)
Figure A.1.5 – Historical gradations, upper and lower AHTD gradation limits and model blend for Class 7 base material coming from the Granite Mountain Quarry in Pulaski County, Arkansas (Cyanite)
APPENDIX A.2

Grain size distribution curve for class 7 base material coming from the Glen Rose, Preston, Sharp’s, and Granite Mountain quarries
Figure A.2.1 – Grain size distribution curve for material from the Black Rock quarry, Lawrence County, Arkansas
Figure A.2.2 – Grain size distribution curve for material from the Glen Rose quarry, Hot Springs County, Arkansas
Figure A.2.3 – Grain size distribution curve for material from the Preston quarry, Crawford County, Arkansas
Figure A.2.4 – Grain size distribution curve for material from the Sharp’s quarry, Benton County, Arkansas
Granite Mountain quarry

Figure A.2.5 – Grain size distribution curve for material from the Granite Mountain quarry, Pulaski County, Arkansas
APPENDIX A.3

Compaction curves for upper and lower AHTD gradation limits, line of zero air voids, and model blend for Class 7 base material coming from the Preston, Glen Rose, Black Rock, Granite Mountain, and Sharp’s quarries
Figure A.3.1 – Compaction curves and line of zero air voids for Class 7 base material coming from the Preston quarry in Benton County, Arkansas (Sandstone)
Glen Rose quarry

![Compaction curves and line of zero air voids for Class 7 base material coming from the Glen Rose quarry in Hot Springs County, Arkansas (Novaculite)](image)

Figure A.3.2 – Compaction curves and line of zero air voids for Class 7 base material coming from the Glen Rose quarry in Hot Springs County, Arkansas (Novaculite)
Black Rock quarry

Figure A.3.3 – Compaction curves and line of zero air voids for Class 7 base material coming from the Black Rock quarry in Lawrence County, Arkansas (Dolomite)
Figure A.3.4 – Compaction curves and line of zero air voids for Class 7 base material coming from the Granite Mountain quarry in Pulaski County, Arkansas (Cyanite)
Figure A.3.5 – Compaction curves and line of zero air voids for Class 7 base material coming from the Sharp’s quarry in Benton County, Arkansas (Limestone)
Step by step procedure to determine Hydraulic Conductivity by the constant head method
Test procedure for base course containing 6%, 8% fines

Sample construction and permeameter assembly:

Steps 1-8 are shown in Fig B1.1 which is a pictorial sequence demonstrating assembly of constant head permeameter.

1. As described in section 3.3, the weight of material to be compacted into each specimen was found by multiplying the target unit weight by the compaction mold volume. The material was then hand blended at optimum moisture.

2. Compact the blended material into the mold to 98% of maximum dry density. For this study, it was found that a Marshall hammer used to compact 3 lifts at 25 blows/lift would obtain the target density in a standard proctor mold.

3. A small amount of fine material and a steel straightedge is used to dress and level the top of sample.

4. Place a #100 screen, filter paper, and perforated metal protection plate on the dressed face of the sample.

5. Attach the base of the permeameter to the proctor mold using a rubber collar and two (2) hose clamps. Insure that the sides of the mold and the base assembly are in line. NOTE: THE PERMEAMETER BASE ASSEMBLY WILL BE INVERTED AT THIS TIME. This inversion is necessary to insure that the filter assembly for the bottom of specimen is properly positioned.

6. Invert the assembly and remove the base plate from the compaction mold. This face of the sample (bottom) should not require dressing or leveling. Place a #100 screen, filter paper, and perforated metal protection plate on the face of the sample.

7. Attach the upper portion of the permeameter (reservoir) to the proctor mold using a rubber collar and two (2) hose clamps. Insure that assembled permeameter is plumb and the permeameter base, sample, and permeameter reservoir form a straight line.

8. To minimize trapped air in the permeameter base assembly, a soft plastic tube approximately 300 mm long and 10 mm in diameter should be threaded through a drain hole in the permeameter base until it comes in contact with the bottom of
the sample.
9. The tube should then be folded up against the side of the permeameter and the assembly placed in an empty 5 gallon bucket.
10. As the bucket is filled air trapped inside the permeameter base will “bleed” out the plastic tube inserted in steps 7 and 8. **Steps 7-9 will minimize air blockage that may occur when saturating the sample.**
11. Fill the bucket till it overflows and water spills into the sink. The plastic tubing used for bleeding the air from the permeameter base can now be removed.
Figure B.1.1 – Pictorial sequence demonstrating assembly of constant head permeameter
Fig. B.1.2 shows the assembled permeameter and 5 gallon bucket which is placed in a sink to catch overflow water. The sample is ready to be saturated and the permeameter reservoir filled as described in steps 12-23.

**Figure B.1.2 - Assembled permeameter and 5 gallon bucket used for study which is placed in a sink to catch overflow water. The electric vacuum pump can be seen on the left-hand side of the photo while the water supply tub is on the right-hand side of the photo.**

**Sample saturation and reservoir fill:**

Fig 3.5 from chapter 3 is shown to allow identification of the permeameter components described in steps 12-23.

12. Plug the bubble tube with a rubber stopper and close the water inflow line used to fill the reservoir with a pinch clamp
13. Apply 25-50 mm (1-2 in) Hg of vacuum to the empty reservoir via the vacuum port while keeping the 5 gallon bucket full of water. The water in the bucket will slowly recede as it is drawn up through the sample and into the reservoir. Since the use of the vacuum to draw water through the sample creates a pressure head, the smallest amount of vacuum that can be used to achieve saturation is desired. The use of a large pressure head to saturate the sample can cause migration of the fines which will require the test to be abandoned.

14. Assume that the sample is saturated when ½” to 1” of water can be seen over top of sample and no air bubbles are present in the water exiting the sample. This condition represents from 2 to 3 pore volumes of water passing through the specimen. Research by Richardson (1997) and others suggest that incomplete
sample saturation and air blockage of the interconnected void spaces are serious concerns that will cause unrealistically low values of hydraulic conductivity

15. Once vacuum saturation is complete release any residual vacuum in the reservoir by venting the vacuum port.

16. Reach down into 5 gallon bucket and place three (3) rubber stoppers into base assembly drain holes to prevent water from entering the bottom of the chamber while the reservoir is filled. NOTE: This step is necessary to prevent water from being drawn up through the sample when a vacuum is applied to the reservoir. The movement of water under excessive gradients through the sample while the reservoir fills can cause significant migration of the fines.

17. Insure that the bubble tube is still plugged with a rubber stopper and un-crimp the water inflow line. (see step 12)

18. Fill a 3+ gallon bucket with tap water and place it at approximately the same level as the top of the 5 gallon bucket containing the permeameter. This tub will serve as the water supply to fill the reservoir. The inflow line from the permeameter should be submerged in the tub.

19. Loosen the slip nut on top of the reservoir and set the bubble tub at $H$ for the desired hydraulic gradient. Tighten the slip nut and insure that the rubber stopper is firmly fixed in the bubble tube.

20. Attach a vacuum pump to reservoir via a vacuum tube attached the vacuum vent and apply approximately 75-125 mm Hg of vacuum. Water will be rapidly drawn into the permeameter reservoir from the supply reservoir via the inflow line. Make sure that the end of the inflow line stays submerged in the water supply tub.

21. When the water level approaches the top of the reservoir or from 53-58 cm on the graduated scale attached to the side of the permeameter shut off the vacuum line using the plastic pinch clamp.

22. Turn off the vacuum pump. Close the inflow line with the pinch clamp.

23. Remove the three plugs from base assembly which were inserted in step 16 and re-fill the 5-gallon bucket to overflowing.

**Determination of hydraulic conductivity:**

24. Record the initial water elevation on the graduated scale of the reservoir and
remove the stopper in the bubble tube. Watch closely as the air-water interface in the bubble tube begins to migrate towards the bottom of the tube.

25. The time at which the first air bubble exits the bubble tube and starts to rise to the surface of the reservoir marks the start of the test. A bubble formation rate of 5-30 seconds should be expected for densely graded base course materials. A rapid formation of air bubbles normally signals "short circuiting" or sidewall leakage while extremely slow bubble formation signals air blockage in the sample. In either case, the sample will probably have to be discarded.

26. The reservoir readings, taken at time intervals of 1, 5, or 10 minutes (based on the permeability of the sample being tested) should be recorded. The test is terminated when a steady state flow condition is reached as evidenced by 3-4 comparable incremental flow readings. The incremental flow readings can be accurately recorded during tests on specimens with a hydraulic conductivity greater than about $10^{-3}$ cm. For more permeable materials the test time is relatively short (less than about 2 minutes) and it will probably be necessary to re-run the test to get accurate incremental readings.

27. Equation 3.2 is used to determine hydraulic conductivity.
APPENDIX B.2

Step by step procedure to determine Hydraulic Conductivity by the falling head – rising tail method
Test procedure for base course containing 10%, 12%, 14% and 16% fines

Sample construction and permeameter assembly:

Steps 1-13 are shown in Fig B.2.1 and Fig. B.2.2 which is a pictorial sequence demonstrating the assembly of a falling head permeameter.

1. As described in 3.3, the weight of material to be compacted into each specimen was found by multiplying the target unit weight by the compaction mold volume. The material was then hand blended at optimum moisture.

2. Place filter paper, # 100 screen, and perforated metal protection plate on top of the permeability cell base plate. Add a small amount of vacuum grease to the side of the base plate. Note: The specimen will be compacted on the base plate.

3. Use a membrane expander and place an 8” tall inner or compaction membrane on the base plate. Place an O ring over the membrane to secure the membrane to the base plate. This O-ring will prevent fines from migrating down the membrane to the base plate to side wall interface during the compaction process. If not contained, these fines can act as a wick between the sample and the chamber allowing water to move in both directions or “short-circuit” along the sides of the base plate.

4. Place an aluminum split mold around the base plate and attach a hose clamp around the base of the split mold. Place a 2” metal collar on top of split mold to securely fasten the mold together.

5. Stretch the inner membrane over top of the mold collar and place an O-ring around it to secure it to the collar.

6. Compact the blended material into the mold to 98% of maximum dry density using the Marshall hammer and 3 lifts with 25 blows /lift.

7. Remove the collar, split mold and hose clamp from the sample.

8. A small amount of fine material and a steel straightedge is used to dress the top of sample. Similar to step 2, place a # 100 screen, filter paper, and perforated metal protection plate on dressed face of the sample.

9. Place the top cap on the sample and add a small amount of vacuum grease to the
sides of the top cap. Pull the inner membrane over the top cap and place an O-ring over the membrane to secure it to the top cap. For the reasons defined in step 3, care should be taken to insure that fines are not allowed to migrate between the top cap and membrane.

10. If the inner membrane was damaged in the compaction process any holes or perforations would allow water from the chamber to enter the sample and give unrealistically high values of hydraulic conductivity. To avoid this problem a second membrane is placed around the sample. Use a membrane expander to place the 2nd or outer membrane over the inner membrane. Place O-rings over the second membrane to attach it to the base plate and top cap.

11. Insert the top drain lines shown in Fig 3.6 that run from the permeability cell base plate into the holes in top cap. Care must be taken to insure that the o-rings are in the seat of the holes in the top cap place and tubes are inserted full depth into the top cap. Unless these lines are firmly seated water from the chamber, which is subjected to a higher pressure than the pore fluid in the sample, will enter the sample and compromise the test.

12. Care should be taken to insure that the O-ring located in the chamber wall seat of the base plate is seated in the O-ring groove and that the groove and seat are clean. Place a small amount of vacuum grease on the O-ring and seat. Place the cylindrical Plexiglas chamber wall over the sample and seat the cylinder firmly on the O-ring of the base plate seat. Attach the permeability cell top plate in an identical manner insuring that the seat and O-ring in the top plate are clean and have a light film of vacuum grease applied.

13. Place the three (3) stainless steel triaxial cell bolts in their slots and lightly snug the nuts “finger tight”. The nuts should then be sequentially tightened approximately 1/2 turn until the nuts can no longer be turned by hand pressure. Fill the assembled permeability cell with water and inspect closely for leaks. If any leaks are detected repeat steps 12-13.
Figure B.2.1 – Pictorial sequence demonstrating assembly of falling head permeameter
Figure B.2.2 – Pictorial sequence demonstrating assembly of falling head permeameter
Sample saturation:

Fig. B.2..3 shows the permeability cell attached to the pressure panel. Fig. B.2..4 is a schematic diagram of the permeability cell and the pressure panel showing the line attachments and valve locations.

14. Before connecting the vacuum and backpressure lines to the permeability cell run water through the pressure panel lines to remove as much air as possible from the panel.

15. Close the upper and lower drain lines to the permeability cell and open the appropriate valves on the pressure panel to fill the chamber burette. At this time fill the vacuum burette and back pressure burette approximately ¼ full of water.

16. Attach the lower drain line, upper drain line, and chamber line to the permeability cell with no air pressure on any burette. Insure that the lower drain line and upper drain line valves are closed.

17. Apply 5 psi air pressure to the chamber burette and open the valve at the pressure panel to place the confining pressure on the sample. There is no valve at the permeability cell for the chamber pressure. Keep confining pressure on sample at all times during test.

18. Open the valves to the upper and lower drain lines at the pressure panel to allow flow from the sample.

19. Slowly open the upper and lower drain lines at the permeability cell. As the specimen begins to consolidate water and air will be forced out of the sample and slowly fill the upper and lower drain burettes at the workstation panel.

20. As the sample consolidates observe the water level in the chamber burette to insure that it does not become empty which will force air into the sample. It may be necessary to close the appropriate valves and re-fill the chamber burette.

21. Allow water levels in the upper and lower drain burettes at the workstation panel to stabilize as the sample consolidates.

22. Close the valves at the permeability cell and fill the lower drain burette and empty the upper drain burette at the workstation panel.
Figure B.2..3 - Assembled permeability cell attached to the pressure panel.
23. Using the appropriate valves at the workstation panel apply 1-3 psi on the lower drain burette with no pressure on the upper drain burette. When the upper and lower drain line valves at the permeability cell are opened water will be forced
under head through the specimen. This flow will force air out of the sample as evidenced by bubbles seen in the upper drain tubing. For samples with low hydraulic conductivity it may be necessary to increase the pressure head on the lower drain burette. Excessive head pressure can cause migration of fines which will invalidate the test.

24. When using air pressure to force water through sample always insure that confining pressure is approximately 5 psi greater than the air pressure applied to the lower drain line.

25. When the volume of water entering the sample (inflow) equals the volume of water exiting the sample (outflow) the degree of saturation can be determined by calculating a B-value.

26. As a general rule, a B-value of .85 or greater was achieved before testing began. However, some of the finer blends did not achieve this value due to stiffness of the solids skeleton; though no testing began with a B-value lower than .7. Acceptable B-values will not normally be attained without back pressure saturation.

_Back pressure saturation:_

27. Close the upper and lower drain line valves at the permeability cell. Use the appropriate valves at the pressure panel to fill the upper, lower, and chamber burettes approximately 3/4 full.

28. Use the appropriate valves at the pressure panel to apply 10 psi of air pressure to the chamber burette and 5 psi of air pressure to upper and lower burettes. Open the upper and lower drain line valves at the permeability cell.

29. Slowly increase both the chamber pressure and the lower and upper drain line pressures at the pressure panel. Insure that the difference the chamber pressure is never allowed to be 10 psi greater than the pressure on the drain lines.

30. When the chamber pressure is 65 psi and drain line pressures are 55 psi allow 30-60 minutes for the air in the sample to dissolve into solution and water to permeate the pores.

_B-value determination:_

31. Attach an electric pore pressure transducer to the permeability cell at the valve for
one of the lower drain lines. The observed pore pressure should be around 55 psi at this time.

32. Insure that the valves for both the upper and lower drain lines are closed at the permeability cell.

33. Increase the chamber pressure 5 psi at the workstation panel and record the increase in pore water pressure on the transducer.

34. The B value is defined as the ratio of the pore water pressure increase in the specimen to the increase in confining pressure

\[ B = \frac{\Delta \text{pore water pressure (value)}}{\Delta \text{confining pressure (5 psi)}} \]  

(3.4)

35. When the B value is greater than or equal to 0.85 testing can begin. If this value is not attained steps 14-35 should be repeated.

**Determination of hydraulic conductivity:**

36. The pressure required to achieve saturation should be maintained on the specimen during the test.

37. Shut the valve at permeability cell to the upper and lower drain line and remove the pore pressure transducer.

38. Vent or release the pressure on the upper and lower drain burettes. Fill the lower drain burette at the pressure panel to the 100 cm level and empty the upper burette at the workstation panel to the 0 cm level. Re-apply the original pressures to the upper and lower drain line burettes and open valves at the pressure panel.

39. Remove the chamber pressure line from the permeability cell and vent the pressure from the chamber burette. Fill the chamber burette at the pressure panel to the 100 cm level. Re-apply the original pressure to the chamber burette and connect to the permeability cell.

40. During testing the water level in the chamber burette should be monitored. Any decrease of the water level in the chamber burette means that water is entering the specimen from a leak in the membranes, through the upper drainline connections in the top cap, or at the cell wall and end plate seats.
41. Open the lower and upper drain line valves at permeability cell and record the readings of the inflow and outflow burettes at time intervals of 5, 10, or 60 minutes (based on the permeability of the sample being tested).

42. A calculator or computer program must be available on site to solve Eq. (3.3) as the test is being run. The specimen should be permeated until at least four (4) values of hydraulic conductivity are obtained over an interval of time in which: the ratio of the inflow to outflow is between 0.75 and 1.25 and the hydraulic conductivity is steady. In this study, the ratio of inflow to outflow was approximately 1.0 for all tests. Steady state flow conditions were assumed when four (4) or more values of hydraulic conductivity were within 50% of each other and a plot of hydraulic conductivity versus time showed no significant upward or downward trend.
Appendix B.3
Specimen Blending
Table B.3.1: Sharps quarry triaxial specimen blending.

<table>
<thead>
<tr>
<th>Size</th>
<th>Lower bound</th>
<th>Model Gradation</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2&quot;</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>81</td>
<td>90</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>43</td>
<td>55</td>
</tr>
<tr>
<td>#40</td>
<td>10</td>
<td>18</td>
<td>30</td>
</tr>
<tr>
<td>#200</td>
<td>6</td>
<td>Varied</td>
<td>10</td>
</tr>
<tr>
<td>PAN</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>% Passing</th>
<th>( \gamma_{\text{d}, \text{max}} ) (pcf)</th>
<th>141.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>98% ( \gamma_{\text{d}, \text{max}} ) (pcf)</td>
<td>138.2</td>
<td></td>
</tr>
<tr>
<td>Mold Dia (in)</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Mold Ht.(in)</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Mold Vol (cf)</td>
<td>0.1963</td>
<td></td>
</tr>
<tr>
<td>Material/sample (lb)</td>
<td>27.13</td>
<td></td>
</tr>
<tr>
<td>OMC (%)</td>
<td>5.5</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>6153.1</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#40</td>
<td>15</td>
<td>1845.9</td>
</tr>
<tr>
<td>#200</td>
<td>4</td>
<td>492.2</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>738.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
</tr>
<tr>
<td>#4</td>
<td>38</td>
<td>4876.3</td>
</tr>
<tr>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>12</td>
<td>1476.7</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>738.4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
</tr>
<tr>
<td>#4</td>
<td>38</td>
<td>4876.3</td>
</tr>
<tr>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1230.6</td>
</tr>
<tr>
<td>Pan</td>
<td>8</td>
<td>984.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4876.3</td>
</tr>
<tr>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>984.5</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1230.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2338.2</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4876.3</td>
</tr>
<tr>
<td>#40</td>
<td>25</td>
<td>3076.5</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>984.5</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1230.6</td>
</tr>
</tbody>
</table>
Table B.3.2: Preston quarry triaxial specimen blending.

<table>
<thead>
<tr>
<th>Size</th>
<th>Lower bound</th>
<th>Model Gradation</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2&quot;</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>75</td>
<td>90</td>
<td>90</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>35</td>
<td>55</td>
</tr>
<tr>
<td>#40</td>
<td>10</td>
<td>18</td>
<td>30</td>
</tr>
<tr>
<td>#200</td>
<td>6</td>
<td>Varied</td>
<td>10</td>
</tr>
<tr>
<td>PAN</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>% Passing</th>
<th>( \gamma_{d, \text{max}} ) (pcf)</th>
<th>98% ( \gamma_{d, \text{max}} ) (pcf)</th>
<th>Mold Dia (in)</th>
<th>Mold Ht.(in)</th>
<th>Mold Voi (cf)</th>
<th>Material/sample (lb)</th>
<th>OMC (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>132.0</td>
<td>129.4</td>
<td>6</td>
<td>12</td>
<td>0.1963</td>
<td>25.40</td>
<td>7.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>5780.7</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>2880.4</td>
</tr>
<tr>
<td>#40</td>
<td>15</td>
<td>1728.2</td>
</tr>
<tr>
<td>#200</td>
<td>4</td>
<td>460.9</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>691.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>25</td>
<td>2880.4</td>
</tr>
<tr>
<td>#4</td>
<td>40</td>
<td>4608.6</td>
</tr>
<tr>
<td>#40</td>
<td>18</td>
<td>2073.9</td>
</tr>
<tr>
<td>#200</td>
<td>11</td>
<td>1267.4</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>691.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>25</td>
<td>2880.4</td>
</tr>
<tr>
<td>#4</td>
<td>40</td>
<td>4608.6</td>
</tr>
<tr>
<td>#40</td>
<td>17</td>
<td>1958.6</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1152.1</td>
</tr>
<tr>
<td>Pan</td>
<td>8</td>
<td>921.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>25</td>
<td>2880.4</td>
</tr>
<tr>
<td>#4</td>
<td>40</td>
<td>4608.6</td>
</tr>
<tr>
<td>#40</td>
<td>15</td>
<td>1728.2</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1152.1</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1152.1</td>
</tr>
</tbody>
</table>
Table B.3.3: Black Rock quarry triaxial specimen blending.

<table>
<thead>
<tr>
<th>Size</th>
<th>Lower bound</th>
<th>Model Gradation</th>
<th>Upper bound</th>
<th>% Passing</th>
<th>Y_d, max (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2&quot;</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td></td>
<td>144.0</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>81</td>
<td>90</td>
<td></td>
<td>98% Y_d, max (pcf) 141.1</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>35</td>
<td>55</td>
<td></td>
<td>Mold Dia (in) 6</td>
</tr>
<tr>
<td>#40</td>
<td>10</td>
<td>18</td>
<td>30</td>
<td></td>
<td>Mold Ht.(in) 12</td>
</tr>
<tr>
<td>#200</td>
<td>6</td>
<td>Varied</td>
<td>10</td>
<td></td>
<td>Mold Vol (cf) 0.1963</td>
</tr>
<tr>
<td>PAN</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td>Material/sample (lb) 27.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>OMC (%) 6.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>6284.6</td>
<td>3/4&quot;</td>
<td>10</td>
<td>1256.9</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>3142.3</td>
<td>#4</td>
<td>35</td>
<td>4399.2</td>
</tr>
<tr>
<td>#40</td>
<td>15</td>
<td>1885.4</td>
<td>#40</td>
<td>25</td>
<td>3142.3</td>
</tr>
<tr>
<td>#200</td>
<td>4</td>
<td>502.8</td>
<td>#200</td>
<td>20</td>
<td>2513.9</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>754.2</td>
<td>Pan</td>
<td>10</td>
<td>1256.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2388.2</td>
<td>3/4&quot;</td>
<td>19</td>
<td>2388.2</td>
</tr>
<tr>
<td>#4</td>
<td>46</td>
<td>5781.9</td>
<td>#4</td>
<td>46</td>
<td>5781.9</td>
</tr>
<tr>
<td>#40</td>
<td>17</td>
<td>2136.8</td>
<td>#40</td>
<td>17</td>
<td>2136.8</td>
</tr>
<tr>
<td>#200</td>
<td>12</td>
<td>1508.3</td>
<td>#200</td>
<td>6</td>
<td>754.2</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>754.2</td>
<td>Pan</td>
<td>12</td>
<td>1508.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2388.2</td>
<td>3/4&quot;</td>
<td>19</td>
<td>2388.2</td>
</tr>
<tr>
<td>#4</td>
<td>46</td>
<td>5781.9</td>
<td>#4</td>
<td>46</td>
<td>5781.9</td>
</tr>
<tr>
<td>#40</td>
<td>17</td>
<td>2136.8</td>
<td>#40</td>
<td>17</td>
<td>2136.8</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1256.9</td>
<td>#200</td>
<td>4</td>
<td>502.8</td>
</tr>
<tr>
<td>Pan</td>
<td>8</td>
<td>1005.5</td>
<td>Pan</td>
<td>14</td>
<td>1759.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2388.2</td>
<td>3/4&quot;</td>
<td>19</td>
<td>2388.2</td>
</tr>
<tr>
<td>#4</td>
<td>46</td>
<td>5781.9</td>
<td>#4</td>
<td>46</td>
<td>5781.9</td>
</tr>
<tr>
<td>#40</td>
<td>17</td>
<td>2136.8</td>
<td>#40</td>
<td>17</td>
<td>2136.8</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>1005.5</td>
<td>#200</td>
<td>2</td>
<td>251.4</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1256.9</td>
<td>Pan</td>
<td>16</td>
<td>2011.1</td>
</tr>
</tbody>
</table>
Table B.3.4: Glen Rose quarry triaxial specimen blending.

<table>
<thead>
<tr>
<th>Size</th>
<th>Lower bound</th>
<th>Model Gradation</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2&quot;</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>86</td>
<td>90</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>47</td>
<td>55</td>
</tr>
<tr>
<td>#40</td>
<td>10</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>#200</td>
<td>6</td>
<td>Varied</td>
<td>10</td>
</tr>
<tr>
<td>PAN</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>% Passing</th>
<th>$\gamma_d$, max (pcf)</th>
<th>98% $\gamma_d$, max (pcf)</th>
<th>Mold Dia (in)</th>
<th>Mold Hgt.(in)</th>
<th>Mold Vol (cf)</th>
<th>Material/sample (lb)</th>
<th>OMC (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>135.0</td>
<td>132.3</td>
<td>6</td>
<td>12</td>
<td>0.1963</td>
<td>25.98</td>
<td>6.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>5892.3</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>2946.1</td>
</tr>
<tr>
<td>#40</td>
<td>15</td>
<td>1767.7</td>
</tr>
<tr>
<td>#200</td>
<td>4</td>
<td>471.4</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>707.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>14</td>
<td>1649.8</td>
</tr>
<tr>
<td>#4</td>
<td>39</td>
<td>4596.0</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3181.8</td>
</tr>
<tr>
<td>#200</td>
<td>14</td>
<td>1649.8</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>707.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>14</td>
<td>1649.8</td>
</tr>
<tr>
<td>#4</td>
<td>39</td>
<td>4596.0</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3181.8</td>
</tr>
<tr>
<td>#200</td>
<td>12</td>
<td>1414.1</td>
</tr>
<tr>
<td>Pan</td>
<td>8</td>
<td>942.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>14</td>
<td>1649.8</td>
</tr>
<tr>
<td>#4</td>
<td>39</td>
<td>4596.0</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3181.8</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1178.5</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1178.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>14</td>
<td>1649.8</td>
</tr>
<tr>
<td>#4</td>
<td>39</td>
<td>4596.0</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3181.8</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1178.5</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1178.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>14</td>
<td>1649.8</td>
</tr>
<tr>
<td>#4</td>
<td>39</td>
<td>4596.0</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3181.8</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1178.5</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1178.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>14</td>
<td>1649.8</td>
</tr>
<tr>
<td>#4</td>
<td>39</td>
<td>4596.0</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3181.8</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1178.5</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1178.5</td>
</tr>
</tbody>
</table>
Table B.3.5: Granite Mountain quarry triaxial specimen blending.

<table>
<thead>
<tr>
<th>Size</th>
<th>Lower bound</th>
<th>Model Gradation</th>
<th>Upper bound</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1/2&quot;</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>50</td>
<td>81</td>
<td>90</td>
</tr>
<tr>
<td>#4</td>
<td>25</td>
<td>45</td>
<td>55</td>
</tr>
<tr>
<td>#40</td>
<td>10</td>
<td>18</td>
<td>30</td>
</tr>
<tr>
<td>#200</td>
<td>6</td>
<td>Varied</td>
<td>10</td>
</tr>
<tr>
<td>PAN</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>% Passing</th>
<th>Y_d,max (pcf)</th>
<th>134.2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>98% Y_d,max (pcf)</td>
<td>131.5</td>
</tr>
</tbody>
</table>

| Mold Dia (in) | 6 |
| Mold Ht.(in)  | 12 |
| Mold Voi (cf) | 0.1963 |

| Material/sample (lb) | 25.82 |
| OMC (%)              | 7.5 |

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2225.3</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4218.3</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3162.2</td>
</tr>
<tr>
<td>#200</td>
<td>12</td>
<td>1405.4</td>
</tr>
<tr>
<td>Pan</td>
<td>6</td>
<td>702.7</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2225.3</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4218.3</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3162.2</td>
</tr>
<tr>
<td>#200</td>
<td>10</td>
<td>1171.2</td>
</tr>
<tr>
<td>Pan</td>
<td>8</td>
<td>937.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2225.3</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4218.3</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3162.2</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>937.0</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1171.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2225.3</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4218.3</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3162.2</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>937.0</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1171.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2225.3</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4218.3</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3162.2</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>937.0</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1171.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Size</th>
<th>% Retained per sieve</th>
<th>Weight Material Ret. (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/4&quot;</td>
<td>19</td>
<td>2225.3</td>
</tr>
<tr>
<td>#4</td>
<td>36</td>
<td>4218.3</td>
</tr>
<tr>
<td>#40</td>
<td>27</td>
<td>3162.2</td>
</tr>
<tr>
<td>#200</td>
<td>8</td>
<td>937.0</td>
</tr>
<tr>
<td>Pan</td>
<td>10</td>
<td>1171.2</td>
</tr>
</tbody>
</table>
Appendix B.4

Sample Preparation
Initial Specimen Mold Preparation

1. Place a thin coating of vacuum grease around the bottom loading followed by positioning of one end of a single 0.686 mm (0.027 in.) thick rubber membrane and two rubber O-rings as shown in Figure B.2.1.

2. Place a 15.24 cm (6 in.) diameter by 30.48 cm (12 in.) height steel cylindrical split mold over the bottom platen and membrane (Figure B.4.2) then secure the split mold with two hose clamps and extension collar (Figure B.4.3).

3. Pull the free end of the rubber membrane over the mold extension collar and secure in place using two rubber O-rings, Figure B.4.4.

4. Place inside the specimen mold in the following sequence one each of 15.24 cm (6 in.) diameter filter paper, No. 100 filter screen, and perforated metal protection plate and review the completed assembly for proper split mold and rubber membrane fit.

Sample Preparation and Compaction

1. Spread the selected pre-blended specimen within an adequately large mixing pan.

2. Add the required volume of water to the dry specimen needed to achieve the optimum moisture content as determined by respective Modified Proctor testing and adequately hand-blend.

3. Compact the sample into the split mold apparatus in lifts of equal mass with a 3”-diameter Marshall hammer as shown in Figures B.4.5 through B.4.7. Vary the number of blows from the Marshall hammer per lift as needed to insure that the entire blended specimen fits into the 30.48 cm (12 in.) height mold. Blows per lift typically range from 20 for finer blends to 30 for more coarse blends with trial-and-error
experience providing the most guidance for material mass and required compactive effort per lift.

Final Specimen Mold Preparation

1. Cleanse the exposed inward facing portion of the membrane that covers the mold extension collar with a moist paper towel to remove material remaining from specimen compaction.

2. Place inside the top portion of the specimen mold in the following sequence (Figure B.4.8): one 15.24 cm (6 in.) diameter perforated metal protection plate, one 15.24 cm (6 in.) diameter No. 100 filter screen, and one 15.24 cm (6 in.) diameter filter paper.

3. Place a thin covering of vacuum grease on the top load platen then place said platen on top of the specimen, using the extension collar as a guide (B.4.9).

4. Remove the O-rings and release the rubber membrane from the extension collar.

5. Remove the extension collar from the split mold.

6. Remove the hose clamps and split mold from the bottom platen (Figure B.4.10).

7. Using the membrane expander shown in Figure B.4.11, place an additional rubber membrane around the specimen.

8. Add one O-ring each to the top and bottom platens around the second membrane, placing the additional O-rings on the outer sides of those already placed (Figure B.4.12).

9. Repeat Steps 8 and 9 for a third membrane.

10. Remove, thoroughly clean, cover with a thin coat of vacuum grease, and replace the triaxial cell base O-ring as shown in Figure B.4.13.
11. Repeat the process for the triaxial cell top O-ring.

12. Fold down the excessive rubber membranes on the top end of the specimen to be flush with the top platen.

13. Connect the two drain lines to the top platen (Figure B.4.14).

14. Place the clear confining chamber onto the triaxial cell base (Figure B.4.15) and then the triaxial cell top onto the chamber (Figure B.4.16). Apply hand pressure and gently turn the triaxial cell top and confining chamber to assure proper seating.

15. Insert and tighten tie-rods into the four notched spaces to clamp the triaxial cylinder between the cell top and bottom as shown in Figure B.4.17.

16. Within the assembled triaxial cell, lower and tighten the triaxial loading piston (Figure B.4.18) into the recessed center of the top platen.

Sample Saturation

1. Connect the top drain line and bottom fill line from the triaxial cell base to the pressure panel.

2. Connect the pressure panel fill valve to the triaxial specimen cell pressure valve and connect an open ended outlet line to the triaxial cell top cap.

3. Fill the triaxial cell with water until fill water runs through the outlet line.

4. Transition the cell pressure line from the pressure panel fill valve to the cell pressure accumulator valve.

5. Insert the specimen pore pressure transducer to the triaxial cell base and the cell pressure transducer to the triaxial cell top (Figures B.4.19 and B.4.20).
6. Open the fill and drain accumulator and pore pressure transducer valves on the triaxial cell base.

7. Vent the cell, fill, and drain accumulators to atmospheric pressure and zero the cell and pore pressure transducers.

8. Apply approximately 10.34 kPa (1.5 psi) of confining pressure to the specimen while keeping the fill and drain accumulator valves closed.

9. Fill the bottom fill line accumulator with de-aired water.

10. Drain the top drain line accumulator of all water.

11. Apply approximately 186.16 kPa (27 psi) of vacuum pressure to the top drain line accumulator and open the panel valve, thus applying a vacuum to the specimen. Take care to avoid unintentionally overconsolidating the specimen by maintaining less than 34.47 kPa (5 psi) in absolute difference between the pore and cell pressure readings at all times. Adjust the valve positions if necessary.

12. Once the vacuum applied to the specimen has stabilized (as evidenced by cessation of the vast majority of air bubbling within the drain accumulator), open the fill accumulator valve thus allowing de-aired water to enter the specimen under only atmospheric and the created vacuum pressures.

13. Refill the fill accumulator with de-aired water as it is emptied until the rate of water flowing into the specimen (observed in the fill accumulator) matches the rate of water leaving the specimen (observed in the drain accumulator). A constant specimen filling and draining rate indicates that the specimen is essentially saturated.

14. Once the sample is deemed essentially saturated, fill both the bottom fill and top drain line accumulators with de-aired water. Turn the drain accumulator operation valve to
off and turn the fill accumulator operation valve to pressurB.4. Open the bridge valve between the two accumulators, thus exposing both the fill and drain chambers to the same pressurB.4. This force is applied to the specimen as pore pressure.

15. Initially set the cell confining pressure to approximately 68.95 kPa (10 psi) and the specimen pore pressure to 34.48 kPa (5 psi). Increase both the cell and pore pressures incrementally so as to maintain a net effective confining stress of 34.48 kPa (5 psi) on the sample at all times until the cell and pore pressures are 413.69 and 379.21 kPa (60 and 55 psi), respectively.

16. Check Skempton’s B-value by closing both the specimen fill and drain accumulator lines at the triaxial chamber base (to avoid compressing of any possible air bubbles located between the triaxial chamber and pressure panel) and increase the cell confining pressure 34.48 kPa (5 psi). The Class 7 base specimen was deemed saturated when the B value was approximately 0.85 to 0.90.

17. Begin cyclical and staged triaxial testing regimen.
Triaxial Specimen Preparation Figures - continued
APPENDIX C

Hydraulic conductivity results using constant head and falling head tests for the upper/lower AHTD gradation limits and model blend for Class 7 base material coming from the Black Rock, Glen Rose, Granite Mountain, Preston and Sharp’s quarries
Sample - CB-6(1)
Black Rock quarry, CB-6
Date - 5/11/03
Time-1:30 p.m.

\[ R^2 = 0.9994 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>55.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>2.0</td>
<td>53.00</td>
<td>2.00</td>
<td>354.30</td>
<td>354.30</td>
<td>2.95</td>
<td>0.0500</td>
</tr>
<tr>
<td>4.0</td>
<td>51.20</td>
<td>1.80</td>
<td>318.87</td>
<td>673.17</td>
<td>2.66</td>
<td>0.0450</td>
</tr>
<tr>
<td>6.0</td>
<td>49.10</td>
<td>2.10</td>
<td>372.02</td>
<td>1045.19</td>
<td>3.10</td>
<td>0.0525</td>
</tr>
<tr>
<td>8.0</td>
<td>47.40</td>
<td>1.70</td>
<td>301.16</td>
<td>1346.34</td>
<td>2.51</td>
<td>0.0425</td>
</tr>
<tr>
<td>10.0</td>
<td>45.30</td>
<td>2.10</td>
<td>372.02</td>
<td>1718.36</td>
<td>3.10</td>
<td>0.0525</td>
</tr>
</tbody>
</table>

\[ y = 170.57x + 3.3743 \]
\[ R^2 = 0.9994 \]

* - k cm/sec based on Total Flow Regression = 4.81E-02

Figure C.1 - Total flow versus time for material from Black Rock quarry containing 6 % fines
Sample - FB-10

Black Rock quarry, FB-10

\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]

\[ A_g = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6/21/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6/25/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>6/27/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>pp transducer broken</td>
<td>50.0</td>
<td>89.1</td>
<td>10.9</td>
<td>0.0</td>
<td></td>
<td>78.2</td>
<td>3.81E-06</td>
</tr>
</tbody>
</table>

282
Sample - MB-6(1)
Black Rock quarry, MB-6
Date - 4/30/03
Time-1:30 p.m.

\[ y = 33.001x - 10.123 \]
\[ R^2 = 0.9988 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_g = 182.41 \text{ cm}^2 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.40</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.80</td>
<td>1.60</td>
<td>283.44</td>
<td>283.44</td>
<td>0.47</td>
<td>0.0080</td>
</tr>
<tr>
<td>20.0</td>
<td>50.60</td>
<td>2.20</td>
<td>389.73</td>
<td>673.17</td>
<td>0.65</td>
<td>0.0110</td>
</tr>
<tr>
<td>30.0</td>
<td>48.80</td>
<td>1.80</td>
<td>318.87</td>
<td>992.04</td>
<td>0.53</td>
<td>0.0090</td>
</tr>
<tr>
<td>40.0</td>
<td>47.00</td>
<td>1.80</td>
<td>318.87</td>
<td>1310.91</td>
<td>0.53</td>
<td>0.0090</td>
</tr>
<tr>
<td>50.0</td>
<td>45.20</td>
<td>1.80</td>
<td>318.87</td>
<td>1629.78</td>
<td>0.53</td>
<td>0.0090</td>
</tr>
<tr>
<td>60.0</td>
<td>43.60</td>
<td>1.60</td>
<td>283.44</td>
<td>1913.22</td>
<td>0.47</td>
<td>0.0080</td>
</tr>
</tbody>
</table>

*\( k \) cm/sec based on Total Flow Regression = 9.31E-03

Figure C.2- Total flow versus time for material from Black Rock quarry containing 6 % fines
Sample - MB-6(2)
Black Rock quarry, MB-6
Date - 4/30/03
Time-2:30 p.m.

\[ y = 8.2501x - 20.246 \]
\[ R^2 = 0.9897 \]

L (inches) = 4.6250
H (inches) = 1.5000
I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.60</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>54.40</td>
<td>0.20</td>
<td>35.43</td>
<td>35.43</td>
<td>0.06</td>
<td>0.0010</td>
</tr>
<tr>
<td>20.0</td>
<td>53.80</td>
<td>0.60</td>
<td>106.29</td>
<td>141.72</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>30.0</td>
<td>53.30</td>
<td>0.50</td>
<td>88.58</td>
<td>230.30</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
<tr>
<td>40.0</td>
<td>52.80</td>
<td>0.50</td>
<td>88.58</td>
<td>318.87</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
<tr>
<td>50.0</td>
<td>52.40</td>
<td>0.40</td>
<td>70.86</td>
<td>389.73</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>60.0</td>
<td>52.00</td>
<td>0.40</td>
<td>70.86</td>
<td>460.59</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
</tbody>
</table>

\*-*k cm/sec based on Total Flow Regression = 2.33E-03

Figure C.3 - Total flow versus time for material from Black Rock quarry containing 6 % fines

284
Sample - MB-6 (falling head)  
Black Rock quarry  

<table>
<thead>
<tr>
<th>SATURATION</th>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6/13/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>6/15/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Intial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>6/27/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.65/.72</td>
<td>1.00</td>
<td>86.0</td>
<td>14.0</td>
<td>0.0</td>
<td>72.0</td>
<td></td>
<td>2.55E-04</td>
</tr>
</tbody>
</table>

Sample - MB-8 (falling head)  
Black Rock quarry  

<table>
<thead>
<tr>
<th>SATURATION</th>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>6/13/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>6/15/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Intial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>6/27/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.65/.72</td>
<td>1.00</td>
<td>91.6</td>
<td>8.4</td>
<td>0.0</td>
<td>83.2</td>
<td></td>
<td>1.43E-04</td>
</tr>
</tbody>
</table>
Sample - MB-8(1)
Black Rock quarry, MB-8

Date - 5/4/03

Time-3:30 p.m.

\[
y = 34.266x + 20.246 \\
R^2 = 0.9994
\]

\[
A_4 = 177.15 \text{ cm}^2 \\
A_r = 182.41 \text{ cm}^2
\]

\[
L \text{ (inches)} = 4.6250 \\
H \text{ (inches)} = 1.5000 \\
I \text{ (gradient)} = 0.3243
\]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.90</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.80</td>
<td>2.10</td>
<td>372.02</td>
<td>372.02</td>
<td>0.62</td>
<td>0.0105</td>
</tr>
<tr>
<td>20.0</td>
<td>50.80</td>
<td>2.00</td>
<td>354.30</td>
<td>726.32</td>
<td>0.59</td>
<td>0.0100</td>
</tr>
<tr>
<td>30.0</td>
<td>49.00</td>
<td>1.80</td>
<td>318.87</td>
<td>1045.19</td>
<td>0.53</td>
<td>0.0090</td>
</tr>
<tr>
<td>40.0</td>
<td>47.00</td>
<td>2.00</td>
<td>354.30</td>
<td>1399.49</td>
<td>0.59</td>
<td>0.0100</td>
</tr>
<tr>
<td>50.0</td>
<td>45.20</td>
<td>1.80</td>
<td>318.87</td>
<td>1718.36</td>
<td>0.53</td>
<td>0.0090</td>
</tr>
<tr>
<td>60.0</td>
<td>43.30</td>
<td>1.90</td>
<td>336.59</td>
<td>2054.94</td>
<td>0.56</td>
<td>0.0095</td>
</tr>
</tbody>
</table>

\[* \text{-k cm/sec based on Total Flow Regression} = 9.66E-03\]

Figure C.4- Total flow versus time for material from Black Rock quarry containing 8 % fines
Sample - MB-8(2)
Black Rock quarry, MB-8
Date - 5/20/03

\[ \text{Area} = 177.15 \text{ cm}^2 \]

\[ \text{Area} = 182.41 \text{ cm}^2 \]

Time-2:30 p.m.

L (inches) = 4.6250
H (inches) = 1.5000
I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)$^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.60</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>53.40</td>
<td>1.20</td>
<td>212.58</td>
<td>212.58</td>
<td>0.35</td>
<td>0.0060</td>
</tr>
<tr>
<td>20.0</td>
<td>52.20</td>
<td>1.20</td>
<td>212.58</td>
<td>425.16</td>
<td>0.35</td>
<td>0.0060</td>
</tr>
<tr>
<td>30.0</td>
<td>51.00</td>
<td>1.20</td>
<td>212.58</td>
<td>637.74</td>
<td>0.35</td>
<td>0.0060</td>
</tr>
<tr>
<td>40.0</td>
<td>49.60</td>
<td>1.40</td>
<td>248.01</td>
<td>885.75</td>
<td>0.41</td>
<td>0.0070</td>
</tr>
<tr>
<td>50.0</td>
<td>48.50</td>
<td>1.10</td>
<td>194.87</td>
<td>1080.62</td>
<td>0.32</td>
<td>0.0055</td>
</tr>
<tr>
<td>60.0</td>
<td>47.40</td>
<td>1.10</td>
<td>194.87</td>
<td>1275.48</td>
<td>0.32</td>
<td>0.0055</td>
</tr>
</tbody>
</table>

*\(-k \text{ cm/sec based on Total Flow Regression} = 6.15E-03\)

Figure C.5- Total flow versus time for material from Black Rock quarry containing 8% fines
Sample - MB-10(1)  
Black Rock quarry

$A_s = 182.41 \text{ cm}^2$
$L = 10.16 \text{ cm}$

SATURATION

<table>
<thead>
<tr>
<th>Start Date</th>
<th>Chamber Pressure (psi)</th>
<th>Pore Burette Pressure (psi)</th>
<th>Vacuum Burette Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/17/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/18/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>Start Date</th>
<th>Elapsed Time (minutes)</th>
<th>Outflow (ml, Vac Bur)</th>
<th>Inflow (ml, B-P Bur)</th>
<th>Inflow (ml, Cham Bur)</th>
<th>$H_1$ (Initial head diff)</th>
<th>$H_2$ (Final head diff)</th>
<th>$k$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/24/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$B=4.3/5 = .86$</td>
<td>25.00</td>
<td>62.6</td>
<td>37.4</td>
<td>0.0</td>
<td>25.2</td>
<td></td>
<td>$4.27E-05$</td>
</tr>
</tbody>
</table>

Sample - MB-10(2)  
Black Rock quarry

$A_s = 182.41 \text{ cm}^2$
$L = 10.16 \text{ cm}$

SATURATION

<table>
<thead>
<tr>
<th>Start Date</th>
<th>Chamber Pressure (psi)</th>
<th>Pore Burette Pressure (psi)</th>
<th>Vacuum Burette Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/17/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/18/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>Start Date</th>
<th>Elapsed Time (minutes)</th>
<th>Outflow (ml, Vac Bur)</th>
<th>Inflow (ml, B-P Bur)</th>
<th>Inflow (ml, Cham Bur)</th>
<th>$H_1$ (Initial head diff)</th>
<th>$H_2$ (Final head diff)</th>
<th>$k$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/24/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$B=4.4/5 = .88$</td>
<td>25.00</td>
<td>66.4</td>
<td>33.2</td>
<td>0.0</td>
<td>33.2</td>
<td></td>
<td>$3.42E-05$</td>
</tr>
</tbody>
</table>
**Sample - MB-12(1)**

**Black Rock quarry**

\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/1/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/4/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/8/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.9/5 = .78</td>
<td>10.00</td>
<td>78.0</td>
<td>21.8</td>
<td>0.0</td>
<td>56.2</td>
<td></td>
<td>4.47E-05</td>
</tr>
</tbody>
</table>

**Sample - MB-12(2)**

**Black Rock quarry**

\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/1/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/4/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/8/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5 = .72</td>
<td>30.00</td>
<td>54.0</td>
<td>45.9</td>
<td>0.0</td>
<td>8.1</td>
<td></td>
<td>6.50E-05</td>
</tr>
</tbody>
</table>
Sample - MB-14(1)  \( a_{\text{in}}=a_{\text{out}}=1.67\text{cm}^2 \)

Black Rock quarry  \( A_s = 182.41 \text{ cm}^2 \)
\( L_s = 10.16 \text{ cm} \)

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/29/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/30/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>( H_1 ) (Initial head diff)</th>
<th>( H_2 ) (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>6/6/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td>64.0</td>
<td>6.92E-06</td>
</tr>
<tr>
<td>( B=3.7/5 =.74 )</td>
<td>50.0</td>
<td>82.0</td>
<td>18.0</td>
<td>0.0</td>
<td>64.0</td>
<td>64.0</td>
<td>6.92E-06</td>
</tr>
</tbody>
</table>

Sample - MB-14(2)  \( a_{\text{in}}=a_{\text{out}}=1.67\text{cm}^2 \)

Black Rock quarry  \( A_s = 182.41 \text{ cm}^2 \)
\( L_s = 10.16 \text{ cm} \)

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/29/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/30/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>( H_1 ) (Initial head diff)</th>
<th>( H_2 ) (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>6/6/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td>64.0</td>
<td>6.92E-06</td>
</tr>
<tr>
<td>( B=3.6/5 =.72 )</td>
<td>50.0</td>
<td>76.0</td>
<td>24.0</td>
<td>0.0</td>
<td>52.0</td>
<td>52.0</td>
<td>1.01E-05</td>
</tr>
</tbody>
</table>

290
**Sample - MB-16(1)**

**Black Rock quarry**

\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/25/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/26/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/27/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.9/5 = .78</td>
<td>50.0</td>
<td>80.0</td>
<td>20.0</td>
<td>0.0</td>
<td>60.0</td>
<td>7.92E-06</td>
<td></td>
</tr>
</tbody>
</table>

**Sample - MB-16(2)**

**Black Rock quarry**

\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/1/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5/4/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/8/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5 = .72</td>
<td>50.0</td>
<td>83.3</td>
<td>16.8</td>
<td>0.0</td>
<td>66.5</td>
<td>6.33E-06</td>
<td></td>
</tr>
</tbody>
</table>
Sample - CB-6(1)
Glen Rose quarry, CB-6

Date - 7/16/03
Time-7:30 p.m.

\[ y = 1073.5x - 11.81 \]
\[ R^2 = 0.9999 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

\[ L (\text{inches}) = 4.6250 \]
\[ H (\text{inches}) = 1.5000 \]
\[ I (\text{gradient}) = 0.3243 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>53.50</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>1.0</td>
<td>47.70</td>
<td>5.80</td>
<td>1027.47</td>
<td>1027.47</td>
<td>17.12</td>
<td>0.2898</td>
</tr>
<tr>
<td>2.0</td>
<td>41.40</td>
<td>6.30</td>
<td>1116.05</td>
<td>2143.52</td>
<td>18.60</td>
<td>0.3147</td>
</tr>
<tr>
<td>3.0</td>
<td>35.20</td>
<td>6.20</td>
<td>1098.33</td>
<td>3241.85</td>
<td>18.31</td>
<td>0.3097</td>
</tr>
<tr>
<td>4.0</td>
<td>29.40</td>
<td>5.80</td>
<td>1027.47</td>
<td>4269.32</td>
<td>17.12</td>
<td>0.2898</td>
</tr>
<tr>
<td>5.0</td>
<td>23.30</td>
<td>6.10</td>
<td>1080.62</td>
<td>5349.93</td>
<td>18.01</td>
<td>0.3047</td>
</tr>
</tbody>
</table>

\[ *-k \text{ cm/sec based on Total Flow Regression} = 3.03E-01 \]

Figure C.6- Total flow versus time for material from Glen Rose quarry containing 6% fines
Sample - FB-10
Glen Rose quarry, FB-10

\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]
\[ A_a = 182.41 \text{ cm}^2 \]
\[ L = 10.16 \text{ cm} \]

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/26/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/26/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/27/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.9/5 = .78</td>
<td>10.0</td>
<td>67.2</td>
<td>32.8</td>
<td>0.0</td>
<td>34.4</td>
<td>8.27E-05</td>
<td></td>
</tr>
</tbody>
</table>
### Sample - MB-6(1)
Glen Rose quarry, MB-6

Date - 7/14/03
Time-3:30 p.m.

- L (inches) = 4.6250
- H (inches) = 1.5000
- I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Flow Rate (cm$^3$/sec)$^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>53.5</td>
<td>0.50</td>
<td>88.58</td>
<td>88.58</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
<tr>
<td>20.0</td>
<td>52.8</td>
<td>0.70</td>
<td>124.01</td>
<td>212.58</td>
<td>0.21</td>
<td>0.0035</td>
</tr>
<tr>
<td>30.0</td>
<td>52.3</td>
<td>0.50</td>
<td>88.58</td>
<td>301.16</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
<tr>
<td>40.0</td>
<td>51.9</td>
<td>0.40</td>
<td>70.86</td>
<td>372.02</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>50.0</td>
<td>51.4</td>
<td>0.50</td>
<td>88.58</td>
<td>460.59</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
<tr>
<td>60.0</td>
<td>50.9</td>
<td>0.50</td>
<td>88.58</td>
<td>549.17</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
</tbody>
</table>

*-$k$ cm/sec based on Total Flow Regression = 2.61E-03

Figure C.7- Total flow versus time for material from Glen Rose quarry containing 6% fines.

\[
y = 9.2624x + 7.5921 \\
R^2 = 0.9936
\]
Sample - MB-6(2)
Glen Rose quarry, MB-6
Date - 7/15/03
Time-2:30 p.m.

\[ y = 18.373x + 27.838 \]

\[ R^2 = 0.9952 \]

\[ A_r = 177.15 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

L (inches) = 4.6250
H (inches) = 1.5000
I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>53.7</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.5</td>
<td>1.20</td>
<td>212.58</td>
<td>212.58</td>
<td>0.35</td>
<td>0.0060</td>
</tr>
<tr>
<td>20.0</td>
<td>51.3</td>
<td>1.20</td>
<td>212.58</td>
<td>425.16</td>
<td>0.35</td>
<td>0.0060</td>
</tr>
<tr>
<td>30.0</td>
<td>50.3</td>
<td>1.00</td>
<td>177.15</td>
<td>602.31</td>
<td>0.30</td>
<td>0.0050</td>
</tr>
<tr>
<td>40.0</td>
<td>49.4</td>
<td>0.90</td>
<td>159.44</td>
<td>761.75</td>
<td>0.27</td>
<td>0.0045</td>
</tr>
<tr>
<td>50.0</td>
<td>48.5</td>
<td>0.90</td>
<td>159.44</td>
<td>921.18</td>
<td>0.27</td>
<td>0.0045</td>
</tr>
<tr>
<td>60.0</td>
<td>47.7</td>
<td>0.80</td>
<td>141.72</td>
<td>1062.90</td>
<td>0.24</td>
<td>0.0040</td>
</tr>
</tbody>
</table>

\[ y = 18.373x + 27.838 \]

\[ R^2 = 0.9952 \]

\[ k \text{ cm/sec based on Total Flow Regression} = 5.18 \times 10^{-3} \]

Figure C.8- Total flow versus time for material from Glen Rose quarry containing 6% fines.

* - k cm/sec based on Total Flow Regression = 5.18E-03
Sample - MB-6 (falling head)  \( a_{ep} = a_{oc} = 1.67 \text{cm}^2 \)
Glen Rose quarry  \( A_s = 182.41 \text{ cm}^2 \)
\( L = 10.16 \text{ cm} \)

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/20/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.1/5</td>
<td>.82</td>
<td>2.00</td>
<td>69.8</td>
<td>31.0</td>
<td>38.8</td>
<td>3.67E-04</td>
<td></td>
</tr>
</tbody>
</table>

Sample - MB-8 (falling head)  \( a_{ep} = a_{oc} = 1.67 \text{cm}^2 \)
Glen Rose quarry  \( A_s = 182.41 \text{ cm}^2 \)
\( L = 10.16 \text{ cm} \)

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/20/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5</td>
<td>.72</td>
<td>2.00</td>
<td>67.4</td>
<td>32.4</td>
<td>35.0</td>
<td>4.07E-04</td>
<td></td>
</tr>
</tbody>
</table>
Sample - MB-8(1)
Glen Rose quarry, MB-8
Date - 7/16/03
Time-5:30 p.m.

\[ y = 19.183x + 1.6871 \]
\[ R^2 = 0.9977 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_e = 182.41 \text{ cm}^2 \]

\[ L \text{ (inches)} = 4.6250 \]
\[ H \text{ (inches)} = 1.5000 \]
\[ I \text{ (gradient)} = 0.3243 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total Flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>53.7</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.7</td>
<td>1.00</td>
<td>177.15</td>
<td>177.15</td>
<td>0.30</td>
<td>0.0050</td>
</tr>
<tr>
<td>20.0</td>
<td>51.5</td>
<td>1.20</td>
<td>212.58</td>
<td>389.73</td>
<td>0.35</td>
<td>0.0060</td>
</tr>
<tr>
<td>30.0</td>
<td>50.3</td>
<td>1.20</td>
<td>212.58</td>
<td>602.31</td>
<td>0.35</td>
<td>0.0060</td>
</tr>
<tr>
<td>40.0</td>
<td>49.3</td>
<td>1.00</td>
<td>177.15</td>
<td>779.46</td>
<td>0.30</td>
<td>0.0050</td>
</tr>
<tr>
<td>50.0</td>
<td>48.4</td>
<td>0.90</td>
<td>159.44</td>
<td>938.90</td>
<td>0.27</td>
<td>0.0045</td>
</tr>
</tbody>
</table>

\[ y = 19.183x + 1.6871 \]
\[ R^2 = 0.9977 \]

*-k cm/sec based on Total Flow Regression = 5.41E-03

Figure C.9- Total flow versus time for material from Glen Rose quarry containing 8% fines.
Sample - MB-8(2)
Glen Rose quarry, MB-8
Date - 7/16/03
Time-6:30 p.m.

\[ y = 6.1749x - 6.7486 \]
\[ R^2 = 0.9967 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

L (inches) = 4.6250
H (inches) = 1.5000
I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>53.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.7</td>
<td>0.30</td>
<td>53.14</td>
<td>53.14</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
<tr>
<td>20.0</td>
<td>52.4</td>
<td>0.30</td>
<td>53.15</td>
<td>106.29</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
<tr>
<td>30.0</td>
<td>52.0</td>
<td>0.40</td>
<td>70.86</td>
<td>177.15</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>40.0</td>
<td>51.6</td>
<td>0.40</td>
<td>70.86</td>
<td>248.01</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>50.0</td>
<td>51.3</td>
<td>0.30</td>
<td>53.15</td>
<td>301.16</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
</tbody>
</table>

* - k cm/sec based on Total Flow Regression = 1.74E-03

Figure C.10- Total flow versus time for material from Glen Rose quarry containing 8% fines.
Sample - MB-10(1) \[a_{in}=a_{out}=1.67 \text{cm}^2\]
Glen Rose quarry \[A_s = 182.41 \text{ cm}^2\] \[L = 10.16 \text{ cm}\]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/20/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.3/5 =.86</td>
<td>2.00</td>
<td>60.2</td>
<td>39.6</td>
<td>0.0</td>
<td>20.6</td>
<td></td>
<td>6.13E-04</td>
</tr>
</tbody>
</table>

Sample - MB-10(2) \[a_{in}=a_{out}=1.67 \text{cm}^2\]
Glen Rose quarry \[A_s = 182.41 \text{ cm}^2\] \[L = 10.16 \text{ cm}\]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/20/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.5/5 =.90</td>
<td>2.00</td>
<td>62.5</td>
<td>37.4</td>
<td>0.0</td>
<td>25.1</td>
<td></td>
<td>5.36E-04</td>
</tr>
</tbody>
</table>
Sample - MB-12(1) \[a_{in}=a_{out}=1.67\text{cm}^2\]
Glen Rose quarry
\[A_s = 182.41\text{ cm}^2\]
\[L = 10.16\text{ cm}\]

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/23/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/23/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/23/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.0/5 = .80</td>
<td>10.00</td>
<td>59.5</td>
<td>40.6</td>
<td>0.0</td>
<td>18.9</td>
<td></td>
<td>1.29E-04</td>
</tr>
</tbody>
</table>

Sample - MB-12(2) \[76.0000 \ a_{in}=a_{out}=1.67\text{cm}^2\]
Glen Rose quarry
\[A_s = 182.41\text{ cm}^2\]
\[L = 10.16\text{ cm}\]

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/23/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/23/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/23/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.9/5 = .78</td>
<td>10.00</td>
<td>65.0</td>
<td>35.0</td>
<td>0.0</td>
<td>30.0</td>
<td></td>
<td>9.34E-05</td>
</tr>
</tbody>
</table>
**Sample - MB-14(1)**

Glen Rose quarry

\[ a_{in}=a_{out}=1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/28/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/28/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/29/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( B = 4.0/5 = .80 )</td>
<td>5.00</td>
<td>81.2</td>
<td>18.8</td>
<td>0.0</td>
<td>62.4</td>
<td>7.31E-05</td>
<td></td>
</tr>
</tbody>
</table>

**Sample - MB-14(2)**

Glen Rose quarry

\[ a_{in}=a_{out}=1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/28/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/28/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/29/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( B = 3.9/5 = .78 )</td>
<td>5.00</td>
<td>66.0</td>
<td>34.0</td>
<td>0.0</td>
<td>32.0</td>
<td>1.77E-04</td>
<td></td>
</tr>
</tbody>
</table>
Sample - MB-16(1)
Glen Rose quarry

a_in = a_out = 1.67 cm²
A_s = 182.41 cm²
L = 10.16 cm

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/31/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7/31/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/1/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B = 3.6/5</td>
<td>=.72</td>
<td>50.00</td>
<td>79.8</td>
<td>20.2</td>
<td>0.0</td>
<td>59.6</td>
<td>8.03E-06</td>
</tr>
</tbody>
</table>

Sample - MB-16(2)
Glen Rose quarry

76.0000

a_in = a_out = 1.67 cm²
A_s = 182.41 cm²
L = 10.16 cm

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/1/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/1/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/20/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B = 3.8/5</td>
<td>=.76</td>
<td>50.00</td>
<td>90.5</td>
<td>9.5</td>
<td>0.0</td>
<td>81.0</td>
<td>3.27E-06</td>
</tr>
</tbody>
</table>
Sample - CB-6(1)
Granite Mountain quarry, CB-6
Date 8/14/03

\[ y = 9.4649x + 20.246 \]
\[ R^2 = 0.9922 \]

\( A_r = 177.15 \text{ cm}^2 \)
\( A_s = 182.41 \text{ cm}^2 \)


\begin{array}{cccccc}
\text{Time (min)} & \text{Reservoir (cm)} & \text{Difference (cm)} & \text{Interval Flow (cm}^3\text{)} & \text{Total Flow (cm}^3\text{)} & \text{Flow Rate (cm}^3/\text{sec)} & \text{Hydraulic Conductivity (cm/sec)*} \\
0.0 & 54.80 & 0.00 & 0.00 & 0.00 & 0.00 & 0.0000 \\
10.0 & 54.10 & 0.70 & 124.00 & 124.00 & 0.21 & 0.0035 \\
20.0 & 53.50 & 0.60 & 106.29 & 230.30 & 0.18 & 0.0030 \\
30.0 & 53.10 & 0.40 & 70.86 & 301.15 & 0.12 & 0.0020 \\
40.0 & 52.50 & 0.60 & 106.29 & 407.44 & 0.18 & 0.0030 \\
50.0 & 52.10 & 0.40 & 70.86 & 478.30 & 0.12 & 0.0020 \\
\end{array}

\[ *-k \text{ cm/sec based on Total Flow Regression} = 2.67E-03 \]

Figure C.11- Total flow versus time for material from Granite Mountain quarry containing 6% fines.
Sample - FB-10
Granite Mountain quarry, FB-10

\[ a_{tot} = a_{orr} = 1.67 \text{cm}^2 \]
\[ A_p = 182.41 \text{ cm}^2 \]
\[ L = 10.16 \text{ cm} \]

## SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/12/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/13/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.7/5 = .84</td>
<td>223.0</td>
<td>88.5</td>
<td>11.4</td>
<td>0.0</td>
<td>77.1</td>
<td>9.04E-07</td>
<td></td>
</tr>
</tbody>
</table>
Sample - MB-6(1)
Granite Mountain quarry, MB-6
Date - 8/12/03
Time - 11:00 a.m.

\[ A_e = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

\[ L \text{ (inches)} = 4.6250 \]
\[ H \text{ (inches)} = 1.5000 \]
\[ I \text{ (gradient)} = 0.3243 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>53.9</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>53.2</td>
<td>0.70</td>
<td>124.00</td>
<td>124.00</td>
<td>0.21</td>
<td>0.0035</td>
</tr>
<tr>
<td>20.0</td>
<td>52.6</td>
<td>0.60</td>
<td>106.29</td>
<td>230.30</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>30.0</td>
<td>51.9</td>
<td>0.70</td>
<td>124.01</td>
<td>354.30</td>
<td>0.21</td>
<td>0.0035</td>
</tr>
<tr>
<td>40.0</td>
<td>51.2</td>
<td>0.70</td>
<td>124.00</td>
<td>478.30</td>
<td>0.21</td>
<td>0.0035</td>
</tr>
<tr>
<td>50.0</td>
<td>50.6</td>
<td>0.60</td>
<td>106.29</td>
<td>584.59</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>60.0</td>
<td>49.9</td>
<td>0.70</td>
<td>124.01</td>
<td>708.60</td>
<td>0.21</td>
<td>0.0035</td>
</tr>
</tbody>
</table>

*-k cm/sec based on Total Flow Regression = 3.32E-03

Figure C.12- Total flow versus time for material from Granite Mountain quarry containing 6 % fines.
Sample - MB-6(2)
Granite Mountain quarry, MB-6
Date - 8/15/03
Time-1:15 p.m.

\[ y = 7.6554x + 8.2248 \]

\[ R^2 = 0.9984 \]

\[ A_r = 177.15 \text{ cm}^2 \]

\[ A_g = 182.41 \text{ cm}^2 \]

\[
\begin{array}{cccccccc}
\text{Time (min)} & \text{Reservoir (cm)} & \text{Difference (cm)} & \text{Interval Flow (cm}^3\text{)} & \text{Total flow (cm}^3\text{)} & \text{Flow Rate (cm}^3\text{/sec)} & \text{Hydraulic Conductivity (cm/sec)*} \\
0.0 & 50.5 & 0.00 & 0.00 & 0.00 & 0.00 & 0.0000 \\
10.0 & 50.0 & 0.50 & 88.58 & 88.58 & 0.15 & 0.0025 \\
20.0 & 49.6 & 0.40 & 70.86 & 159.44 & 0.12 & 0.0020 \\
30.0 & 49.1 & 0.50 & 88.58 & 248.01 & 0.12 & 0.0025 \\
40.0 & 48.7 & 0.40 & 70.86 & 318.87 & 0.12 & 0.0020 \\
50.0 & 48.3 & 0.40 & 70.86 & 389.73 & 0.12 & 0.0020 \\
60.0 & 47.9 & 0.40 & 70.86 & 460.59 & 0.12 & 0.0020 \\
\end{array}
\]

\[ y = 2.16 \times 10^{-3} \text{ cm/sec} \]

Figure C.13- Total flow versus time for material from Granite Mountain quarry containing 6 % fines.
Sample - MB-6 (falling head)  \( \text{area} = 1.67 \text{cm}^2 \)
Granite Mountain quarry  \( A_a = 182.41 \text{ cm}^2 \)
\( L = 10.16 \text{ cm} \)

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/3/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/3/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/3/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.4/5 = .88</td>
<td>40.00</td>
<td>66.4</td>
<td>33.6</td>
<td>0.0</td>
<td>32.8</td>
<td></td>
<td>2.16E-05</td>
</tr>
</tbody>
</table>

Sample - MB-8 (falling head)  \( \text{area} = 1.67 \text{cm}^2 \)
Granite Mountain quarry  \( A_a = 182.41 \text{ cm}^2 \)
\( L = 10.16 \text{ cm} \)

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/3/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/3/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/3/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.7/5 = .74</td>
<td>40.00</td>
<td>61.5</td>
<td>38.5</td>
<td>0.0</td>
<td>23.0</td>
<td></td>
<td>2.85E-05</td>
</tr>
</tbody>
</table>
Sample - MB-8(1)
Granite Mountain quarry, MB-8
Date - 8/14/03

\[ y = 0.5831x - 4.7319 \]
\[ R^2 = 0.9977 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

L (inches) = 4.6250
H (inches) = 1.5000
I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.5</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>240.0</td>
<td>53.8</td>
<td>0.70</td>
<td>124.01</td>
<td>124.01</td>
<td>0.01</td>
<td>0.0001</td>
</tr>
<tr>
<td>480.0</td>
<td>52.9</td>
<td>0.90</td>
<td>159.44</td>
<td>283.44</td>
<td>0.01</td>
<td>0.0002</td>
</tr>
<tr>
<td>710.0</td>
<td>52.2</td>
<td>0.70</td>
<td>124.00</td>
<td>407.44</td>
<td>0.01</td>
<td>0.0002</td>
</tr>
</tbody>
</table>

* - k cm/sec based on Total Flow Regression = 1.64E-04

Figure C.14- Total flow versus time for material from Granite Mountain quarry containing 8 % fines.
Sample - MB-8(2)
Granite Mountain quarry, MB-8
Date - 8/14/03

\[ y = 0.277x + 11.678 \]
\[ R^2 = 0.9709 \]

\( A_c = 177.15 \text{ cm}^2 \)
\( A_s = 182.41 \text{ cm}^2 \)

\[ A_c = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

\[ L (\text{inches}) = 4.6250 \]
\[ H (\text{inches}) = 1.5000 \]
\[ I (\text{gradient}) = 0.3243 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.6</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>240.0</td>
<td>54.1</td>
<td>0.50</td>
<td>88.58</td>
<td>88.58</td>
<td>0.01</td>
<td>0.0001</td>
</tr>
<tr>
<td>480.0</td>
<td>53.7</td>
<td>0.40</td>
<td>70.86</td>
<td>159.44</td>
<td>0.00</td>
<td>0.0001</td>
</tr>
<tr>
<td>710.0</td>
<td>53.5</td>
<td>0.20</td>
<td>35.43</td>
<td>194.87</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

\*\(-k \text{ cm/sec based on Total Flow Regression} = 7.81E-05\)

Figure C.15- Total flow versus time for material from Granite Mountain quarry containing 8% fines.
### Sample - MB-10(1)

Granite Mountain quarry

\[ a_{in}=a_{out}=1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

#### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/13/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/14/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.7/5 = .94</td>
<td>260.00</td>
<td>86.0</td>
<td>15.0</td>
<td>0.0</td>
<td>71.0</td>
<td>1.02E-06</td>
<td></td>
</tr>
</tbody>
</table>

### Sample - MB-10(2)

Granite Mountain quarry

\[ a_{in}=a_{out}=1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

#### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/13/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/14/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5 = .72</td>
<td>260.00</td>
<td>72.2</td>
<td>27.8</td>
<td>0.0</td>
<td>44.4</td>
<td>2.42E-06</td>
<td></td>
</tr>
</tbody>
</table>
Sample - **MB-12(1)**

Granite Mountain quarry

\[ a_{in} = a_{out} = 1.67 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/23/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/23/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

**PERMEATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/24/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.5/5 = .7</td>
<td>150.00</td>
<td>94.0</td>
<td>6.2</td>
<td>0.0</td>
<td>87.8</td>
<td></td>
<td>6.73E-07</td>
</tr>
</tbody>
</table>

Sample - **MB-12(2)**

Granite Mountain quarry

\[ a_{in} = a_{out} = 1.67 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/23/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/23/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

**PERMEATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/24/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5 = .7</td>
<td>150.00</td>
<td>94.2</td>
<td>5.8</td>
<td>0.0</td>
<td>88.4</td>
<td></td>
<td>6.37E-07</td>
</tr>
</tbody>
</table>
### Sample - MB-14(1)  
Granite Mountain quarry  
\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]  
\[ A_s = 182.41 \text{ cm}^2 \]  
\[ L = 10.16 \text{ cm} \]

#### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/7/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10/7/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/8/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.5/5 = .7</td>
<td>224.00</td>
<td>85.4</td>
<td>15.0</td>
<td>0.0</td>
<td>70.4</td>
<td>1.21E-06</td>
<td></td>
</tr>
</tbody>
</table>

### Sample - MB-14(2)  
Granite Mountain quarry  
\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]  
\[ A_s = 182.41 \text{ cm}^2 \]  
\[ L = 10.16 \text{ cm} \]

#### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/7/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10/7/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/8/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.4/5 = .88</td>
<td>224.00</td>
<td>94.0</td>
<td>6.0</td>
<td>0.0</td>
<td>88.0</td>
<td>6.61E-07</td>
<td></td>
</tr>
</tbody>
</table>
Sample - MB-16(1)
Granite Mountain quarry
\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]
\[ L = 10.16 \text{ cm} \]

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/5/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/5/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/7/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9/9/03</td>
<td>216.00</td>
<td>83.4</td>
<td>17.0</td>
<td>0.0</td>
<td>66.4</td>
<td></td>
<td>1.47E-06</td>
</tr>
</tbody>
</table>

Sample - MB-16(2)
Granite Mountain quarry
\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]
\[ L = 10.16 \text{ cm} \]

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/8/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/9/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/9/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.6/5 =.92</td>
<td>140.00</td>
<td>95.8</td>
<td>4.2</td>
<td>0.0</td>
<td>91.6</td>
<td></td>
<td>4.86E-07</td>
</tr>
</tbody>
</table>
Sample - CB-6(1)
Preston quarry, CB-6
Date 12/28/02

\[ y = 84.323x + 38.973 \]
\[ R^2 = 0.9994 \]

\[ A_e = 177.15 \text{ cm}^2 \]
\[ A_g = 182.41 \text{ cm}^2 \]

L (inches) = 4.6250
H (inches) = 1.5000
I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)$^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.80</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>49.60</td>
<td>5.20</td>
<td>921.18</td>
<td>921.18</td>
<td>1.54</td>
<td>0.0260</td>
</tr>
<tr>
<td>20.0</td>
<td>44.90</td>
<td>4.70</td>
<td>832.61</td>
<td>1753.79</td>
<td>1.39</td>
<td>0.0235</td>
</tr>
<tr>
<td>30.0</td>
<td>40.40</td>
<td>4.50</td>
<td>797.18</td>
<td>2550.96</td>
<td>1.33</td>
<td>0.0225</td>
</tr>
<tr>
<td>40.0</td>
<td>35.60</td>
<td>4.80</td>
<td>850.32</td>
<td>3401.28</td>
<td>1.42</td>
<td>0.0240</td>
</tr>
</tbody>
</table>

*-k cm/sec based on Total Flow Regression = \(2.38E-02\)

Figure C.16 - Total flow versus time for material from Preston quarry containing 6% fines.
Sample - FB-10
Preston quarry, FB-10

$a_{re}=a_{ce}=1.67 \text{cm}^2$
$A_s = 182.41 \text{ cm}^2$
$L = 10.16 \text{ cm}$

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/26/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/28/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/2/03</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.8/5</td>
<td>50.0</td>
<td>71.4</td>
<td>28.5</td>
<td>0.0</td>
<td>42.9</td>
<td>1.31E-05</td>
<td></td>
</tr>
</tbody>
</table>
Sample - MB-6(1)  
**Preston quarry, MB-6**  
Date - 12/25/02  
Time-1:00 p.m.

$$y = 11.705x - 14.552$$  
$$R^2 = 0.997$$

$$A_r = 177.15 \text{ cm}^2$$  
$$A_s = 182.41 \text{ cm}^2$$

| L (inches) | 4.6250 |
| H (inches) | 1.5000 |
| I (gradient) | 0.3243 |

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total Flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>55.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>54.4</td>
<td>0.60</td>
<td>106.29</td>
<td>106.29</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>20.0</td>
<td>53.8</td>
<td>0.60</td>
<td>106.29</td>
<td>212.58</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>30.0</td>
<td>53.2</td>
<td>0.60</td>
<td>106.29</td>
<td>318.87</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>40.0</td>
<td>52.5</td>
<td>0.70</td>
<td>124.01</td>
<td>442.88</td>
<td>0.21</td>
<td>0.0035</td>
</tr>
<tr>
<td>50.0</td>
<td>51.8</td>
<td>0.70</td>
<td>124.01</td>
<td>566.88</td>
<td>0.21</td>
<td>0.0035</td>
</tr>
<tr>
<td>60.0</td>
<td>51.0</td>
<td>0.80</td>
<td>141.72</td>
<td>708.60</td>
<td>0.24</td>
<td>0.0040</td>
</tr>
</tbody>
</table>

**Figure C.17** - Total flow versus time for material from Preston quarry containing 6% fines.

*-k cm/sec based on Total Flow Regression = 3.30E-03*
Sample - MB-6(2)

Preston quarry, MB-6

Date - 12/29/02

Time-1:30 p.m.

\[ y = 9.3636x + 2.5307 \]

\[ R^2 = 0.9978 \]

\[ A_r = 177.15 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

L (inches) = 4.6250

H (inches) = 1.5000

I (gradient) = 0.3243

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>52.8</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.2</td>
<td>0.60</td>
<td>106.29</td>
<td>106.29</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>20.0</td>
<td>51.7</td>
<td>0.50</td>
<td>88.58</td>
<td>194.86</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
<tr>
<td>30.0</td>
<td>51.3</td>
<td>0.40</td>
<td>70.86</td>
<td>265.73</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>40.0</td>
<td>50.7</td>
<td>0.60</td>
<td>106.29</td>
<td>372.01</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>50.0</td>
<td>50.1</td>
<td>0.60</td>
<td>106.29</td>
<td>478.30</td>
<td>0.18</td>
<td>0.0030</td>
</tr>
<tr>
<td>60.0</td>
<td>49.6</td>
<td>0.50</td>
<td>88.58</td>
<td>566.88</td>
<td>0.15</td>
<td>0.0025</td>
</tr>
</tbody>
</table>

\* - k cm/sec based on Total Flow Regression = 2.64E-03

Figure C.18 - Total flow versus time for material from Preston quarry containing 6% fines.
### SATURATION

#### Sample - MB-6 (falling head)

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/1/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/2/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### Preston quarry

\[ A_s = 182.41\ cm^2 \]

\[ L = 10.16\ cm \]

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/3/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td>6.73E-05</td>
</tr>
<tr>
<td>B=4.1/5</td>
<td>10.00</td>
<td>71.4</td>
<td>29.4</td>
<td>0.0</td>
<td>42.0</td>
<td></td>
<td>6.73E-05</td>
</tr>
</tbody>
</table>

#### Sample - MB-8 (falling head)

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/3/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### Preston quarry

\[ A_s = 182.41\ cm^2 \]

\[ L = 10.16\ cm \]

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/3/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td>1.87E-05</td>
</tr>
<tr>
<td>B=4.2/5</td>
<td>50.00</td>
<td>65.4</td>
<td>35.5</td>
<td>0.0</td>
<td>29.9</td>
<td></td>
<td>1.87E-05</td>
</tr>
</tbody>
</table>
Sample - MB-8(1)

Preston quarry, MB-8

Date - 12/29/02

Time-12:30 p.m.

\[ y = 6.5799x - 5.0614 \]

\[ R^2 = 0.9985 \]

\[ A_r = 177.15 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L (\text{inches}) = 4.6250 \]

\[ H (\text{inches}) = 1.5000 \]

\[ I (\text{gradient}) = 0.3243 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total Flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>54.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>53.7</td>
<td>0.30</td>
<td>53.14</td>
<td>53.14</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
<tr>
<td>20.0</td>
<td>53.3</td>
<td>0.40</td>
<td>70.86</td>
<td>124.01</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>30.0</td>
<td>52.9</td>
<td>0.40</td>
<td>70.86</td>
<td>194.87</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>40.0</td>
<td>52.5</td>
<td>0.40</td>
<td>70.86</td>
<td>265.73</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
<tr>
<td>50.0</td>
<td>52.2</td>
<td>0.30</td>
<td>53.14</td>
<td>318.87</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
<tr>
<td>60.0</td>
<td>51.8</td>
<td>0.40</td>
<td>70.86</td>
<td>389.73</td>
<td>0.12</td>
<td>0.0020</td>
</tr>
</tbody>
</table>

*-k cm/sec based on Total Flow Regression = 1.86E-03

Figure C.19 - Total flow versus time for material from Preston quarry containing 8% fines.

319
Sample - MB-8(2)

Preston quarry, MB-8

Date - 12/30/02

Time-2:30 p.m.

\[ y = 4.6818x + 1.2654 \]

\[ R^2 = 0.9978 \]

\[ A_r = 177.15 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L (\text{inches}) = 4.6250 \]

\[ H (\text{inches}) = 1.5000 \]

\[ I (\text{gradient}) = 0.3243 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>43.6</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>43.3</td>
<td>0.30</td>
<td>53.15</td>
<td>53.15</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
<tr>
<td>20.0</td>
<td>43.1</td>
<td>0.20</td>
<td>35.43</td>
<td>88.58</td>
<td>0.06</td>
<td>0.0010</td>
</tr>
<tr>
<td>30.0</td>
<td>42.8</td>
<td>0.30</td>
<td>53.15</td>
<td>141.72</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
<tr>
<td>40.0</td>
<td>42.5</td>
<td>0.30</td>
<td>53.14</td>
<td>194.87</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
<tr>
<td>50.0</td>
<td>42.3</td>
<td>0.20</td>
<td>35.43</td>
<td>230.30</td>
<td>0.06</td>
<td>0.0010</td>
</tr>
<tr>
<td>60.0</td>
<td>42.0</td>
<td>0.30</td>
<td>53.14</td>
<td>283.44</td>
<td>0.09</td>
<td>0.0015</td>
</tr>
</tbody>
</table>

Time (min)\[ k \text{ cm/sec based on Total Flow Regression} = 1.32E-03 \]

Figure C.20 - Total flow versus time for material from Preston quarry containing 8 % fines.

320
Sample - MB-10(1)  
*Preston quarry*  
$a_{in}=a_{out}=1.67 cm^2$  
$A_s = 182.41 cm^2$  
$L = 10.16 cm$

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/4/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

**PERMEATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>$k$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/7/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.7/5 =.84</td>
<td>60.00</td>
<td>72.4</td>
<td>27.6</td>
<td>0.0</td>
<td>44.8</td>
<td></td>
<td>1.04E-05</td>
</tr>
</tbody>
</table>

Sample - MB-10(2)  
*Preston quarry*  
$a_{in}=a_{out}=1.67 cm^2$  
$A_s = 182.41 cm^2$  
$L = 10.16 cm$

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/4/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

**PERMEATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>$k$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/7/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5 =.72</td>
<td>60.00</td>
<td>75.9</td>
<td>23.8</td>
<td>0.0</td>
<td>52.1</td>
<td></td>
<td>8.43E-06</td>
</tr>
</tbody>
</table>
Sample - MB-12(1) a_in=a_out=1.67cm²  
**Preston quarry** A_s = 182.41 cm²  
L = 10.16 cm

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/5/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/12/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5 =.72</td>
<td>50.00</td>
<td>92.2</td>
<td>8.7</td>
<td>0.0</td>
<td>83.5</td>
<td>2.33E-06</td>
<td></td>
</tr>
</tbody>
</table>

Sample - MB-12(2) a_in=a_out=1.67cm²  
**Preston quarry** A_s = 182.41 cm²  
L = 10.16 cm

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/4/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/7/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.6/5 =.72</td>
<td>50.00</td>
<td>92.2</td>
<td>8.7</td>
<td>0.0</td>
<td>83.5</td>
<td>2.33E-06</td>
<td></td>
</tr>
</tbody>
</table>
Sample - MB-14(1)

Preston quarry

\[ a_{in} = a_{out} = 1.67 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/12/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/14/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

**PERMEATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/18/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.5/5 = .90</td>
<td>60.00</td>
<td>90.0</td>
<td>9.8</td>
<td>0.0</td>
<td>80.2</td>
<td>2.85E-06</td>
<td></td>
</tr>
</tbody>
</table>

Sample - MB-14(2)

Preston quarry

\[ a_{in} = a_{out} = 1.67 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

**SATURATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/12/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/14/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

**PERMEATION**

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/18/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.5/5 = .90</td>
<td>60.00</td>
<td>89.0</td>
<td>10.2</td>
<td>0.0</td>
<td>78.8</td>
<td>3.08E-06</td>
<td></td>
</tr>
</tbody>
</table>
**Sample - MB-16(1)**

Preston quarry

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/19/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/20/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/26/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td>85.9</td>
<td>2.36E-06</td>
</tr>
<tr>
<td>B=4.2/5 =.84</td>
<td>50.00</td>
<td>92.9</td>
<td>7.0</td>
<td>0.0</td>
<td>85.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Sample - MB-16(2)**

Preston quarry

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/19/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1/20/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/26/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.2/5 =.84</td>
<td>40.00</td>
<td>88.2</td>
<td>11.4</td>
<td>0.0</td>
<td>76.8</td>
<td></td>
<td>5.12E-06</td>
</tr>
</tbody>
</table>
Sample - CB-6(1)
Sharp's quarry, CB-6
Date 6/31/02

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

\[ L \text{ (inches)} = 4.6250 \]
\[ H \text{ (inches)} = 1.3700 \]
\[ I \text{ (gradient)} = 0.2962 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm$^3$)</th>
<th>Total Flow (cm$^3$)</th>
<th>Flow Rate (cm$^3$/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>51.60</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>42.80</td>
<td>8.80</td>
<td>1558.92</td>
<td>1558.92</td>
<td>2.60</td>
<td>0.0481</td>
</tr>
<tr>
<td>20.0</td>
<td>36.30</td>
<td>6.50</td>
<td>1151.48</td>
<td>2710.40</td>
<td>1.92</td>
<td>0.0355</td>
</tr>
<tr>
<td>30.0</td>
<td>27.50</td>
<td>8.80</td>
<td>1558.92</td>
<td>4269.32</td>
<td>2.60</td>
<td>0.0481</td>
</tr>
<tr>
<td>40.0</td>
<td>19.50</td>
<td>8.00</td>
<td>1417.20</td>
<td>5686.52</td>
<td>2.36</td>
<td>0.0437</td>
</tr>
</tbody>
</table>

\[ y = 140.83x + 28.344 \]
\[ R^2 = 0.9983 \]

\* - k cm/sec based on Total Flow Regression = 4.35E-02

Figure C.21 - Total flow versus time for material from Sharp's quarry containing 6% fines.
Sample - FB-10

Sharp's quarry, FB-10

\[ a_{\text{in}} = a_{\text{out}} = 1.67 \text{cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

\[ L = 10.16 \text{ cm} \]

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/7/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/7/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/9/02</td>
<td>0.0</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
</tr>
<tr>
<td>B=3.8/5 = .76</td>
<td>50.0</td>
<td>62.2</td>
<td>38.0</td>
<td>0.0</td>
<td>24.2</td>
<td>2.20E-05</td>
<td></td>
</tr>
</tbody>
</table>
Sample - MB-6(1)
Sharp's quarry, MB-6
Date-6/29/02

\[ y = 12.578x + 3.543 \]
\[ R^2 = 0.9978 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_g = 182.41 \text{ cm}^2 \]

L (inches) = 4.6250
H (inches) = 1.3700
I (gradient) = 0.2962

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir Difference</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>51.90</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>51.20</td>
<td>0.70</td>
<td>124.00</td>
<td>124.00</td>
<td>0.21</td>
</tr>
<tr>
<td>20.0</td>
<td>50.40</td>
<td>0.80</td>
<td>141.72</td>
<td>265.73</td>
<td>0.24</td>
</tr>
<tr>
<td>30.0</td>
<td>49.70</td>
<td>0.70</td>
<td>124.00</td>
<td>389.73</td>
<td>0.21</td>
</tr>
<tr>
<td>40.0</td>
<td>49.10</td>
<td>0.60</td>
<td>106.29</td>
<td>496.02</td>
<td>0.18</td>
</tr>
</tbody>
</table>

\[ *-k \text{ cm/sec based on Total Flow Regression} = 3.88E-03 \]

Figure C.22 - Total flow versus time for material from Sharp’s quarry containing 6 % fines.
Sample - MB-6(2)  
Sharp's quarry, MB-6  
Date-6/29/02  

\[ y = 13.109x + 14.172 \]

\[ R^2 = 0.9971 \]

\[ A_e = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>53.50</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.60</td>
<td>0.90</td>
<td>159.44</td>
<td>159.44</td>
<td>0.27</td>
<td>0.0049</td>
</tr>
<tr>
<td>20.0</td>
<td>51.90</td>
<td>0.70</td>
<td>124.01</td>
<td>283.44</td>
<td>0.21</td>
<td>0.0038</td>
</tr>
<tr>
<td>30.0</td>
<td>51.20</td>
<td>0.70</td>
<td>124.00</td>
<td>407.44</td>
<td>0.21</td>
<td>0.0038</td>
</tr>
<tr>
<td>40.0</td>
<td>50.50</td>
<td>0.70</td>
<td>124.01</td>
<td>531.45</td>
<td>0.21</td>
<td>0.0038</td>
</tr>
</tbody>
</table>

\[ y = 13.109x + 14.172 \]
\[ R^2 = 0.9971 \]

* -k cm/sec based on Total Flow Regression = \[ 4.05E-03 \]

Figure C.23 - Total flow versus time for material from Sharp’s quarry containing 6% fines.
Sample - MB-6 (falling head)  
\[ a_{eq}=a_{ce}=1.67 \text{cm}^2 \]
Sharp's quarry  
\[ A_s = 182.41 \text{ cm}^2 \]
\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/2/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2/5/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/9/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.1/5 =.82</td>
<td>2.00</td>
<td>70.4</td>
<td>29.4</td>
<td>0.0</td>
<td>41.0</td>
<td></td>
<td>3.46E-04</td>
</tr>
</tbody>
</table>

Sample - MB-8 (falling head)  
\[ a_{eq}=a_{ce}=1.67 \text{cm}^2 \]
Sharp's quarry  
\[ A_s = 182.41 \text{ cm}^2 \]
\[ L = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/2/03</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2/5/03</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>2/9/03</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.5/5 =.70</td>
<td>5.00</td>
<td>58.0</td>
<td>41.0</td>
<td>0.0</td>
<td>17.0</td>
<td></td>
<td>2.75E-04</td>
</tr>
</tbody>
</table>
Sample - MB-8(1)

Sharp's quarry, MB-8

Date - 7/3/02

\[ y = 7.744x + 21.933 \]

\[ R^2 = 0.987 \]

\[ A_c = 177.15 \text{ cm}^2 \]

\[ A_s = 182.41 \text{ cm}^2 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>51.70</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>51.10</td>
<td>0.60</td>
<td>106.29</td>
<td>106.29</td>
<td>0.18</td>
<td>0.0033</td>
</tr>
<tr>
<td>20.0</td>
<td>50.60</td>
<td>0.50</td>
<td>88.58</td>
<td>194.87</td>
<td>0.15</td>
<td>0.0027</td>
</tr>
<tr>
<td>30.0</td>
<td>50.20</td>
<td>0.40</td>
<td>70.86</td>
<td>265.73</td>
<td>0.12</td>
<td>0.0022</td>
</tr>
<tr>
<td>40.0</td>
<td>49.80</td>
<td>0.40</td>
<td>70.86</td>
<td>336.59</td>
<td>0.12</td>
<td>0.0022</td>
</tr>
<tr>
<td>50.0</td>
<td>49.50</td>
<td>0.30</td>
<td>53.14</td>
<td>389.73</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
<tr>
<td>60.0</td>
<td>49.10</td>
<td>0.40</td>
<td>70.86</td>
<td>460.59</td>
<td>0.12</td>
<td>0.0022</td>
</tr>
</tbody>
</table>

\[ y = 7.744x + 21.933 \]

\[ R^2 = 0.987 \]

*-k cm/sec based on Total Flow Regression = 2.39E-03

Figure C.24 - Total flow versus time for material from Sharp’s quarry containing 8 % fines.
Sample - MB-8(2)
Sharp's quarry, MB-8
Date-7/13/02

\[ y = 4.6186x + 5.6941 \]
\[ R^2 = 0.9972 \]

\[ A_r = 177.15 \text{ cm}^2 \]
\[ A_s = 182.41 \text{ cm}^2 \]

\[ L \text{ (inches)} = 4.6250 \]
\[ H \text{ (inches)} = 1.3700 \]
\[ I \text{ (gradient)} = 0.2962 \]

<table>
<thead>
<tr>
<th>Time (min)</th>
<th>Reservoir (cm)</th>
<th>Difference (cm)</th>
<th>Interval Flow (cm³)</th>
<th>Total Flow (cm³)</th>
<th>Flow Rate (cm³/sec)</th>
<th>Hydraulic Conductivity (cm/sec)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>52.80</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.0000</td>
</tr>
<tr>
<td>10.0</td>
<td>52.50</td>
<td>0.30</td>
<td>53.14</td>
<td>53.14</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
<tr>
<td>20.0</td>
<td>52.20</td>
<td>0.30</td>
<td>53.14</td>
<td>106.29</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
<tr>
<td>30.0</td>
<td>52.00</td>
<td>0.20</td>
<td>35.43</td>
<td>141.72</td>
<td>0.06</td>
<td>0.0011</td>
</tr>
<tr>
<td>40.0</td>
<td>51.70</td>
<td>0.30</td>
<td>53.14</td>
<td>194.86</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
<tr>
<td>50.0</td>
<td>51.50</td>
<td>0.20</td>
<td>35.43</td>
<td>230.30</td>
<td>0.06</td>
<td>0.0011</td>
</tr>
<tr>
<td>60.0</td>
<td>51.20</td>
<td>0.30</td>
<td>53.14</td>
<td>283.44</td>
<td>0.09</td>
<td>0.0016</td>
</tr>
</tbody>
</table>

\[ y = 4.6186x + 5.6941 \]
\[ R^2 = 0.9972 \]

*-k cm/sec based on Total Flow Regression = 1.43E-03

Figure C.25 - Total flow versus time for material from Sharp’s quarry containing 8% fines.
Sample - MB-10(1)  \( a_{in} = a_{out} = 1.67 \text{cm}^2 \)

Sharp's quarry  \( A_s = 182.41 \text{ cm}^2 \)
\( L = 10.16 \text{ cm} \)

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/5/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/5/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/8/02</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.5/5 = .90</td>
<td>50.00</td>
<td>68.8</td>
<td>31.2</td>
<td>0.0</td>
<td>37.6</td>
<td></td>
<td>1.52E-05</td>
</tr>
</tbody>
</table>

Sample - MB-10(2)  \( a_{in} = a_{out} = 1.67 \text{cm}^2 \)

Preston quarry  \( A_s = 182.41 \text{ cm}^2 \)
\( L = 10.16 \text{ cm} \)

SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/10/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/17/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/17/02</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4/5 = .8</td>
<td>30.00</td>
<td>62.0</td>
<td>37.8</td>
<td>0.0</td>
<td>24.2</td>
<td></td>
<td>3.67E-05</td>
</tr>
</tbody>
</table>
### Sample - MB-12(1)

a_{in}=a_{out}=1.67 \text{cm}^2

**Sharp's quarry**

A_s = 182.41 \text{ cm}^2  
L_s = 10.16 \text{ cm}

#### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/5/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/5/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/5/02</td>
<td></td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.5/5 = .90</td>
<td></td>
<td>50.00</td>
<td>75.0</td>
<td>24.8</td>
<td>0.0</td>
<td>50.2</td>
<td>8.91E-06</td>
</tr>
</tbody>
</table>

### Sample - MB-12(2)

a_{in}=a_{out}=1.67 \text{cm}^2

**Preston quarry**

A_s = 182.41 \text{ cm}^2  
L_s = 10.16 \text{ cm}

#### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/10/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/17/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

#### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1 (Initial head diff)</th>
<th>H2 (Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/17/02</td>
<td></td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4/5 = .8</td>
<td>60.00</td>
<td>67.0</td>
<td>33.0</td>
<td>0.0</td>
<td>34.0</td>
<td></td>
<td>1.39E-05</td>
</tr>
</tbody>
</table>
Sample - MB-14(1)  \(a_{in} = a_{out} = 1.67 \text{cm}^2\)

**Sharp's quarry**  

\(A_s = 182.41 \text{ cm}^2\)

\(L = 10.16 \text{ cm}\)

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/25/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8/27/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/1/02</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=4.5/5 = .90</td>
<td>60.00</td>
<td>69.0</td>
<td>31.0</td>
<td>0.0</td>
<td>38.0</td>
<td></td>
<td>1.25E-05</td>
</tr>
</tbody>
</table>

Sample - MB-14(2)  \(a_{in} = a_{out} = 1.67 \text{cm}^2\)

**Preston quarry**  

\(A_s = 182.41 \text{ cm}^2\)

\(L = 10.16 \text{ cm}\)

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/1/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/4/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>H1(Initial head diff)</th>
<th>H2(Final head diff)</th>
<th>k</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/02</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B=3.9/5 = .88</td>
<td>60.00</td>
<td>75.5</td>
<td>24.3</td>
<td>0.0</td>
<td>51.2</td>
<td></td>
<td>8.65E-06</td>
</tr>
</tbody>
</table>
Sample - MB-16(1)  
\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]
Sharp's quarry  
\[ A_s = 182.41 \text{ cm}^2 \]
\[ L_s = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/15/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>( H_1 ) (Initial head diff)</th>
<th>( H_2 ) (Final head diff)</th>
<th>( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/17/02</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( B = 3.9/5 = .78 )</td>
<td>60.00</td>
<td>85.6</td>
<td>14.3</td>
<td>0.0</td>
<td>71.3</td>
<td>4.37E-06</td>
<td></td>
</tr>
</tbody>
</table>

Sample - MB-16(2)  
\[ a_{in} = a_{out} = 1.67 \text{cm}^2 \]
Preston quarry  
\[ A_s = 182.41 \text{ cm}^2 \]
\[ L_s = 10.16 \text{ cm} \]

### SATURATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>CHAMBER PRESSURE (psi)</th>
<th>PORE BURRETTE PRESSURE (psi)</th>
<th>VACUUM BURRETTE PRESSURE (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/02</td>
<td>10</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>9/15/02</td>
<td>65</td>
<td>55</td>
<td>55</td>
</tr>
</tbody>
</table>

### PERMEATION

<table>
<thead>
<tr>
<th>START DATE</th>
<th>ELAPSED TIME (minutes)</th>
<th>OUTFLOW (ml, Vac Bur)</th>
<th>INFLOW (ml, B-P Bur)</th>
<th>INFLOW (ml, Cham Bur)</th>
<th>( H_1 ) (Initial head diff)</th>
<th>( H_2 ) (Final head diff)</th>
<th>( k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/16/02</td>
<td>0.00</td>
<td>100.0</td>
<td>0.0</td>
<td>0.0</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( B = 3.8/5 = .76 )</td>
<td>60.00</td>
<td>82.0</td>
<td>17.8</td>
<td>0.0</td>
<td>64.2</td>
<td>5.73E-06</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX D

Suction test result of the class 7 base material coming from the Black Rock Glen Rose, Preston, Sharp’s, and Granite Mountain quarries
SUCTION TEST - BLACK ROCK
Start date: 4/11/2003

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>dry wt. (g)</th>
<th>1HR weight (g)</th>
<th>% H₂O</th>
<th>2HR weight (g)</th>
<th>% H₂O</th>
<th>4HR weight (g)</th>
<th>% H₂O</th>
<th>24HR weight (g)</th>
<th>% H₂O</th>
<th>2 DAYS weight (g)</th>
<th>% H₂O</th>
<th>3 DAYS weight (g)</th>
<th>% H₂O</th>
<th>4 DAYS weight (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MB-6a</td>
<td>12652</td>
<td>12718</td>
<td>0.522</td>
<td>12736</td>
<td>0.664</td>
<td>12759</td>
<td>0.846</td>
<td>12857</td>
<td>1.616</td>
<td>12954</td>
<td>2.387</td>
<td>13019</td>
<td>2.901</td>
<td>13082</td>
</tr>
<tr>
<td>MB-6b</td>
<td>12597.00</td>
<td>12664.00</td>
<td>0.53</td>
<td>12681.00</td>
<td>0.67</td>
<td>12706.00</td>
<td>0.87</td>
<td>12816.50</td>
<td>1.74</td>
<td>12927.00</td>
<td>2.62</td>
<td>12998.00</td>
<td>3.18</td>
<td>13062.00</td>
</tr>
<tr>
<td>MB-8a</td>
<td>12545.00</td>
<td>12649.00</td>
<td>0.83</td>
<td>12671.00</td>
<td>1.00</td>
<td>12700.00</td>
<td>1.24</td>
<td>12815.50</td>
<td>2.16</td>
<td>12931.00</td>
<td>3.08</td>
<td>13003.00</td>
<td>3.65</td>
<td>13062.00</td>
</tr>
<tr>
<td>MB-8b</td>
<td>12630.00</td>
<td>12706.00</td>
<td>0.60</td>
<td>12726.00</td>
<td>0.76</td>
<td>12751.00</td>
<td>0.96</td>
<td>12852.50</td>
<td>1.76</td>
<td>12954.00</td>
<td>2.57</td>
<td>13023.00</td>
<td>3.11</td>
<td>13093.00</td>
</tr>
<tr>
<td>MB-10a</td>
<td>12551.00</td>
<td>12677.00</td>
<td>1.00</td>
<td>12698.00</td>
<td>1.17</td>
<td>12724.00</td>
<td>1.38</td>
<td>12837.00</td>
<td>2.28</td>
<td>12950.00</td>
<td>3.18</td>
<td>13026.00</td>
<td>3.78</td>
<td>13073.00</td>
</tr>
<tr>
<td>MB-10b</td>
<td>12572.00</td>
<td>12669.00</td>
<td>0.77</td>
<td>12685.00</td>
<td>0.90</td>
<td>12712.00</td>
<td>1.11</td>
<td>12826.50</td>
<td>2.02</td>
<td>12941.00</td>
<td>2.94</td>
<td>13010.00</td>
<td>3.48</td>
<td>13067.00</td>
</tr>
<tr>
<td>MB-12a</td>
<td>12550.00</td>
<td>12630.00</td>
<td>0.64</td>
<td>12653.00</td>
<td>0.82</td>
<td>12683.00</td>
<td>1.06</td>
<td>12794.00</td>
<td>1.94</td>
<td>12905.00</td>
<td>2.83</td>
<td>12976.00</td>
<td>3.39</td>
<td>13041.00</td>
</tr>
<tr>
<td>MB-12b</td>
<td>12523.00</td>
<td>12613.00</td>
<td>0.72</td>
<td>12637.00</td>
<td>0.91</td>
<td>12667.00</td>
<td>1.15</td>
<td>12775.50</td>
<td>2.02</td>
<td>12884.00</td>
<td>2.88</td>
<td>12953.00</td>
<td>3.43</td>
<td>13014.00</td>
</tr>
<tr>
<td>MB-14a</td>
<td>12642.00</td>
<td>12694.00</td>
<td>0.41</td>
<td>12711.00</td>
<td>0.55</td>
<td>12733.00</td>
<td>0.72</td>
<td>12814.50</td>
<td>1.36</td>
<td>12896.00</td>
<td>2.01</td>
<td>12950.00</td>
<td>2.44</td>
<td>13006.00</td>
</tr>
<tr>
<td>MB-14b</td>
<td>12767.00</td>
<td>12824.00</td>
<td>0.45</td>
<td>12838.00</td>
<td>0.56</td>
<td>12856.00</td>
<td>0.70</td>
<td>12928.00</td>
<td>1.26</td>
<td>13000.00</td>
<td>1.83</td>
<td>13049.00</td>
<td>2.21</td>
<td>13099.00</td>
</tr>
<tr>
<td>MB-16a</td>
<td>12532.00</td>
<td>12634.00</td>
<td>0.65</td>
<td>12657.00</td>
<td>0.84</td>
<td>12682.00</td>
<td>1.04</td>
<td>12789.00</td>
<td>1.89</td>
<td>12896.00</td>
<td>2.74</td>
<td>12963.00</td>
<td>3.27</td>
<td>13028.00</td>
</tr>
<tr>
<td>MB-16b</td>
<td>12613.00</td>
<td>12700.00</td>
<td>0.69</td>
<td>12715.00</td>
<td>0.81</td>
<td>12733.00</td>
<td>0.95</td>
<td>12816.50</td>
<td>1.61</td>
<td>12900.00</td>
<td>2.28</td>
<td>12952.00</td>
<td>2.69</td>
<td>13006.00</td>
</tr>
<tr>
<td>LB-6a</td>
<td>11396.00</td>
<td>11577.00</td>
<td>1.59</td>
<td>11578.00</td>
<td>1.60</td>
<td>11587.00</td>
<td>1.68</td>
<td>11645.50</td>
<td>2.19</td>
<td>11704.00</td>
<td>2.70</td>
<td>11750.00</td>
<td>3.11</td>
<td>11776.00</td>
</tr>
<tr>
<td>UB-10a</td>
<td>11990.00</td>
<td>12088.00</td>
<td>0.82</td>
<td>12103.00</td>
<td>0.94</td>
<td>12127.00</td>
<td>1.14</td>
<td>12229.50</td>
<td>2.00</td>
<td>12332.00</td>
<td>2.85</td>
<td>12401.00</td>
<td>3.43</td>
<td>12472.00</td>
</tr>
<tr>
<td>SAMPLE</td>
<td>5 DAYS weight(g)</td>
<td>% H2O</td>
<td>6 DAYS weight(g)</td>
<td>% H2O</td>
<td>7 DAYS weight(g)</td>
<td>% H2O</td>
<td>8 DAYS weight(g)</td>
<td>% H2O</td>
<td>9 DAYS weight(g)</td>
<td>% H2O</td>
<td>10 DAYS weight(g)</td>
<td>% H2O</td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>-----------------</td>
<td>-------</td>
<td>-----------------</td>
<td>-------</td>
<td>-----------------</td>
<td>-------</td>
<td>-----------------</td>
<td>-------</td>
<td>-----------------</td>
<td>-------</td>
<td>-----------------</td>
<td>-------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-6a</td>
<td>13120.00</td>
<td>3.70</td>
<td>13148.00</td>
<td>3.92</td>
<td>13154.00</td>
<td>3.97</td>
<td>13161.00</td>
<td>4.02</td>
<td>13168.00</td>
<td>4.08</td>
<td>13172.00</td>
<td>4.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-6b</td>
<td>13071.00</td>
<td>3.76</td>
<td>13083.00</td>
<td>3.86</td>
<td>13085.00</td>
<td>3.87</td>
<td>13089.00</td>
<td>3.91</td>
<td>13095.00</td>
<td>3.95</td>
<td>13098.00</td>
<td>3.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-8a</td>
<td>13069.00</td>
<td>4.18</td>
<td>13078.00</td>
<td>4.25</td>
<td>13081.00</td>
<td>4.27</td>
<td>13083.00</td>
<td>4.29</td>
<td>13090.00</td>
<td>4.34</td>
<td>13093.00</td>
<td>4.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-8b</td>
<td>13125.00</td>
<td>3.92</td>
<td>13146.00</td>
<td>4.09</td>
<td>13153.00</td>
<td>4.14</td>
<td>13155.00</td>
<td>4.16</td>
<td>13161.00</td>
<td>4.20</td>
<td>13165.00</td>
<td>4.24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-10a</td>
<td>13075.00</td>
<td>4.17</td>
<td>13084.00</td>
<td>4.25</td>
<td>13086.00</td>
<td>4.26</td>
<td>13087.00</td>
<td>4.27</td>
<td>13094.00</td>
<td>4.33</td>
<td>13096.00</td>
<td>4.34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-10b</td>
<td>13084.00</td>
<td>4.07</td>
<td>13096.00</td>
<td>4.17</td>
<td>13100.00</td>
<td>4.20</td>
<td>13103.00</td>
<td>4.22</td>
<td>13109.00</td>
<td>4.27</td>
<td>13112.00</td>
<td>4.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-12a</td>
<td>13078.00</td>
<td>4.21</td>
<td>13099.00</td>
<td>4.37</td>
<td>13107.00</td>
<td>4.44</td>
<td>13112.00</td>
<td>4.48</td>
<td>13118.00</td>
<td>4.53</td>
<td>13121.00</td>
<td>4.55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-12b</td>
<td>13041.00</td>
<td>4.14</td>
<td>13053.00</td>
<td>4.23</td>
<td>13059.00</td>
<td>4.28</td>
<td>13065.00</td>
<td>4.33</td>
<td>13072.00</td>
<td>4.38</td>
<td>13075.00</td>
<td>4.41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-14a</td>
<td>13041.00</td>
<td>3.16</td>
<td>13085.00</td>
<td>3.50</td>
<td>13107.00</td>
<td>3.68</td>
<td>13121.00</td>
<td>3.79</td>
<td>13129.00</td>
<td>3.85</td>
<td>13133.00</td>
<td>3.88</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-14b</td>
<td>13133.00</td>
<td>2.87</td>
<td>13176.00</td>
<td>3.20</td>
<td>13201.00</td>
<td>3.40</td>
<td>13216.00</td>
<td>3.52</td>
<td>13225.00</td>
<td>3.59</td>
<td>13229.00</td>
<td>3.62</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-16a</td>
<td>13069.00</td>
<td>4.12</td>
<td>13109.00</td>
<td>4.44</td>
<td>13117.00</td>
<td>4.50</td>
<td>13129.00</td>
<td>4.60</td>
<td>13135.00</td>
<td>4.64</td>
<td>13139.00</td>
<td>4.68</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-16b</td>
<td>13042.00</td>
<td>3.40</td>
<td>13085.00</td>
<td>3.74</td>
<td>13107.00</td>
<td>3.92</td>
<td>13121.00</td>
<td>4.03</td>
<td>13129.00</td>
<td>4.09</td>
<td>13133.00</td>
<td>4.12</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LB-6a</td>
<td>11785.00</td>
<td>3.41</td>
<td>11789.00</td>
<td>3.45</td>
<td>11795.00</td>
<td>3.50</td>
<td>11797.00</td>
<td>3.52</td>
<td>11797.00</td>
<td>3.52</td>
<td>11798.00</td>
<td>3.53</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UB-10a</td>
<td>12509.00</td>
<td>4.33</td>
<td>12535.00</td>
<td>4.55</td>
<td>12541.00</td>
<td>4.60</td>
<td>12543.00</td>
<td>4.61</td>
<td>12545.00</td>
<td>4.63</td>
<td>12547.00</td>
<td>4.65</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## SUCTION TEST - GLEN ROSE

**Start date:** 8/7/2003

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>dry wt. (µ)</th>
<th>1HR % H2O</th>
<th>2HR % H2O</th>
<th>4HR % H2O</th>
<th>24HR % H2O</th>
<th>2 DAYS % H2O</th>
<th>3 DAYS % H2O</th>
<th>4 DAYS % H2O</th>
</tr>
</thead>
<tbody>
<tr>
<td>MB-6a</td>
<td>1154.00</td>
<td>11619.00</td>
<td>11638.00</td>
<td>11647.00</td>
<td>11723.00</td>
<td>11785.00</td>
<td>11829.00</td>
<td>11873.00</td>
</tr>
<tr>
<td>MB-6b</td>
<td>11189.00</td>
<td>11262.00</td>
<td>11272.00</td>
<td>11283.00</td>
<td>11355.00</td>
<td>11417.00</td>
<td>11458.50</td>
<td>11500.00</td>
</tr>
<tr>
<td>MB-8a</td>
<td>11083.00</td>
<td>11160.00</td>
<td>11175.00</td>
<td>11192.00</td>
<td>11275.00</td>
<td>11345.00</td>
<td>11381.00</td>
<td>11417.00</td>
</tr>
<tr>
<td>MB-8b</td>
<td>11187.00</td>
<td>11267.00</td>
<td>11280.00</td>
<td>11292.00</td>
<td>11363.00</td>
<td>11430.00</td>
<td>11474.00</td>
<td>11518.00</td>
</tr>
<tr>
<td>MB-10a</td>
<td>11097.00</td>
<td>11183.00</td>
<td>11204.00</td>
<td>11214.00</td>
<td>11291.00</td>
<td>11355.00</td>
<td>11401.50</td>
<td>11448.00</td>
</tr>
<tr>
<td>MB-10b</td>
<td>11202.00</td>
<td>11280.00</td>
<td>11302.00</td>
<td>11315.00</td>
<td>11390.00</td>
<td>11449.00</td>
<td>11491.00</td>
<td>11533.00</td>
</tr>
<tr>
<td>MB-12a</td>
<td>1144.00</td>
<td>11530.00</td>
<td>11548.00</td>
<td>11569.00</td>
<td>11653.00</td>
<td>11711.00</td>
<td>11750.00</td>
<td>11789.00</td>
</tr>
<tr>
<td>MB-12b</td>
<td>11371.00</td>
<td>11438.00</td>
<td>11452.00</td>
<td>11463.00</td>
<td>11541.00</td>
<td>11604.00</td>
<td>11645.00</td>
<td>11686.00</td>
</tr>
<tr>
<td>MB-14a</td>
<td>10865.00</td>
<td>10953.00</td>
<td>10984.00</td>
<td>11009.00</td>
<td>11118.00</td>
<td>11188.00</td>
<td>11201.00</td>
<td>11214.00</td>
</tr>
<tr>
<td>MB-14b</td>
<td>11249.00</td>
<td>11368.00</td>
<td>11384.00</td>
<td>11463.00</td>
<td>11541.00</td>
<td>11604.00</td>
<td>11645.00</td>
<td>11686.00</td>
</tr>
<tr>
<td>MB-16a</td>
<td>11023.00</td>
<td>11104.00</td>
<td>11124.00</td>
<td>11158.00</td>
<td>11264.00</td>
<td>11327.00</td>
<td>11352.00</td>
<td>11377.00</td>
</tr>
<tr>
<td>MB-16b</td>
<td>10663.00</td>
<td>10759.00</td>
<td>10775.00</td>
<td>10804.00</td>
<td>10891.00</td>
<td>10959.00</td>
<td>10989.00</td>
<td>11019.00</td>
</tr>
<tr>
<td>LB-6a</td>
<td>10372.00</td>
<td>10482.00</td>
<td>10511.00</td>
<td>10530.00</td>
<td>10611.00</td>
<td>10651.00</td>
<td>10658.00</td>
<td>10665.00</td>
</tr>
<tr>
<td>UB-10a</td>
<td>10859.00</td>
<td>10928.00</td>
<td>10947.00</td>
<td>10960.00</td>
<td>10990.00</td>
<td>11020.00</td>
<td>11070.00</td>
<td>11105.50</td>
</tr>
<tr>
<td>SAMPLE</td>
<td>5 DAYS</td>
<td>6 DAYS</td>
<td>7 DAYS</td>
<td>8 DAYS</td>
<td>9 DAYS</td>
<td>10 DAYS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>---------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-6a</td>
<td>11910.00</td>
<td>11905.00</td>
<td>11931.00</td>
<td>11928.00</td>
<td>11934.00</td>
<td>11938.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-6b</td>
<td>11539.00</td>
<td>11529.00</td>
<td>11555.00</td>
<td>11558.00</td>
<td>11545.00</td>
<td>11534.60</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-8a</td>
<td>11432.00</td>
<td>11429.00</td>
<td>11434.00</td>
<td>11434.00</td>
<td>11440.00</td>
<td>11444.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-8b</td>
<td>11543.00</td>
<td>11546.00</td>
<td>11554.00</td>
<td>11556.00</td>
<td>11560.00</td>
<td>11563.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-10a</td>
<td>11460.00</td>
<td>11464.00</td>
<td>11476.00</td>
<td>11475.00</td>
<td>11483.00</td>
<td>11489.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-10b</td>
<td>11550.00</td>
<td>11554.00</td>
<td>11561.00</td>
<td>11562.00</td>
<td>11567.00</td>
<td>11571.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-12a</td>
<td>11805.00</td>
<td>11812.00</td>
<td>11817.00</td>
<td>11820.00</td>
<td>11827.00</td>
<td>11832.60</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-12b</td>
<td>11706.00</td>
<td>11713.00</td>
<td>11722.00</td>
<td>11726.00</td>
<td>11734.00</td>
<td>11740.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-14a</td>
<td>11223.00</td>
<td>11230.00</td>
<td>11232.00</td>
<td>11235.00</td>
<td>11239.00</td>
<td>11242.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-14b</td>
<td>11656.00</td>
<td>11662.00</td>
<td>11667.00</td>
<td>11671.00</td>
<td>11676.00</td>
<td>11680.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-16a</td>
<td>11383.00</td>
<td>11391.00</td>
<td>11395.00</td>
<td>11395.00</td>
<td>11409.00</td>
<td>11420.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-16b</td>
<td>11026.00</td>
<td>11030.00</td>
<td>11035.00</td>
<td>11039.00</td>
<td>11045.00</td>
<td>11049.80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LB-6a</td>
<td>10672.00</td>
<td>10675.00</td>
<td>10681.00</td>
<td>10681.00</td>
<td>10688.00</td>
<td>10693.60</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UB-10a</td>
<td>11181.00</td>
<td>11211.00</td>
<td>11225.00</td>
<td>11233.00</td>
<td>11250.00</td>
<td>11263.60</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### SUCTION TEST - GMQ

Start date: 1/2/2004

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>dry wt.(g)</th>
<th>1HR % H₂O</th>
<th>2HR % H₂O</th>
<th>4HR % H₂O</th>
<th>24HR % H₂O</th>
<th>2 DAYS % H₂O</th>
<th>3 DAYS % H₂O</th>
<th>4 DAYS % H₂O</th>
</tr>
</thead>
<tbody>
<tr>
<td>MB-6a</td>
<td>11118.00</td>
<td>11310.00</td>
<td>1.73</td>
<td>11342.00</td>
<td>2.01</td>
<td>11376.00</td>
<td>2.32</td>
<td>11590.00</td>
</tr>
<tr>
<td>MB-6b</td>
<td>11214.00</td>
<td>11392.00</td>
<td>1.59</td>
<td>11433.00</td>
<td>1.95</td>
<td>11479.00</td>
<td>2.36</td>
<td>11703.00</td>
</tr>
<tr>
<td>MB-8a</td>
<td>11310.00</td>
<td>11509.00</td>
<td>1.76</td>
<td>11551.00</td>
<td>2.13</td>
<td>11602.00</td>
<td>2.58</td>
<td>11851.00</td>
</tr>
<tr>
<td>MB-8b</td>
<td>11168.00</td>
<td>11360.00</td>
<td>1.72</td>
<td>11400.00</td>
<td>2.08</td>
<td>11446.00</td>
<td>2.49</td>
<td>11711.00</td>
</tr>
<tr>
<td>MB-10a</td>
<td>11077.00</td>
<td>11261.00</td>
<td>1.66</td>
<td>11310.00</td>
<td>2.10</td>
<td>11363.00</td>
<td>2.58</td>
<td>11557.00</td>
</tr>
<tr>
<td>MB-10b</td>
<td>11123.00</td>
<td>11447.00</td>
<td>1.72</td>
<td>11560.00</td>
<td>2.73</td>
<td>11596.00</td>
<td>3.05</td>
<td>11763.00</td>
</tr>
<tr>
<td>MB-12a</td>
<td>11137.00</td>
<td>11558.00</td>
<td>1.63</td>
<td>11623.00</td>
<td>2.20</td>
<td>11670.00</td>
<td>2.61</td>
<td>11898.00</td>
</tr>
<tr>
<td>MB-12b</td>
<td>11181.00</td>
<td>11365.00</td>
<td>1.65</td>
<td>11411.00</td>
<td>2.06</td>
<td>11454.00</td>
<td>2.44</td>
<td>11616.00</td>
</tr>
<tr>
<td>MB-14a</td>
<td>11194.00</td>
<td>11506.00</td>
<td>1.88</td>
<td>11573.00</td>
<td>2.47</td>
<td>11605.00</td>
<td>2.75</td>
<td>11789.00</td>
</tr>
<tr>
<td>MB-14b</td>
<td>11130.00</td>
<td>11312.00</td>
<td>1.64</td>
<td>11353.00</td>
<td>2.00</td>
<td>11402.00</td>
<td>2.44</td>
<td>11599.00</td>
</tr>
<tr>
<td>MB-16a</td>
<td>10836.00</td>
<td>11043.00</td>
<td>1.89</td>
<td>11059.00</td>
<td>2.04</td>
<td>11104.00</td>
<td>2.45</td>
<td>11301.00</td>
</tr>
<tr>
<td>MB-16b</td>
<td>11191.00</td>
<td>11407.00</td>
<td>1.93</td>
<td>11423.00</td>
<td>2.07</td>
<td>11475.00</td>
<td>2.54</td>
<td>11668.00</td>
</tr>
<tr>
<td>LB-6a</td>
<td>10311.00</td>
<td>10540.00</td>
<td>2.22</td>
<td>10609.00</td>
<td>2.89</td>
<td>10655.00</td>
<td>3.34</td>
<td>10908.00</td>
</tr>
<tr>
<td>UB-10a</td>
<td>11275.00</td>
<td>11451.00</td>
<td>1.56</td>
<td>11494.00</td>
<td>1.94</td>
<td>11535.00</td>
<td>2.31</td>
<td>11830.00</td>
</tr>
<tr>
<td>SAMPLE</td>
<td>5 DAYS</td>
<td>% H20</td>
<td>6 DAYS</td>
<td>% H20</td>
<td>7 DAYS</td>
<td>% H20</td>
<td>8 DAYS</td>
<td>% H20</td>
</tr>
<tr>
<td>---------</td>
<td>--------</td>
<td>-------</td>
<td>--------</td>
<td>-------</td>
<td>--------</td>
<td>-------</td>
<td>--------</td>
<td>-------</td>
</tr>
<tr>
<td>MB-6a</td>
<td>11740.00</td>
<td>5.59</td>
<td>11743.00</td>
<td>5.62</td>
<td>11745.00</td>
<td>5.64</td>
<td>11749.00</td>
<td>5.68</td>
</tr>
<tr>
<td>MB-6b</td>
<td>11814.00</td>
<td>5.35</td>
<td>11817.00</td>
<td>5.38</td>
<td>11819.00</td>
<td>5.40</td>
<td>11823.00</td>
<td>5.43</td>
</tr>
<tr>
<td>MB-8a</td>
<td>11962.00</td>
<td>5.76</td>
<td>11963.00</td>
<td>5.77</td>
<td>11965.00</td>
<td>5.79</td>
<td>11969.00</td>
<td>5.83</td>
</tr>
<tr>
<td>MB-8b</td>
<td>11825.00</td>
<td>5.88</td>
<td>11825.00</td>
<td>5.88</td>
<td>11824.00</td>
<td>5.87</td>
<td>11830.00</td>
<td>5.93</td>
</tr>
<tr>
<td>MB-10a</td>
<td>11735.00</td>
<td>5.94</td>
<td>11738.00</td>
<td>5.97</td>
<td>11740.00</td>
<td>5.99</td>
<td>11743.00</td>
<td>6.01</td>
</tr>
<tr>
<td>MB-10b</td>
<td>11922.00</td>
<td>5.95</td>
<td>11925.00</td>
<td>5.97</td>
<td>11927.00</td>
<td>5.99</td>
<td>11930.00</td>
<td>6.02</td>
</tr>
<tr>
<td>MB-12a</td>
<td>12061.00</td>
<td>6.05</td>
<td>12066.00</td>
<td>6.09</td>
<td>12069.00</td>
<td>6.12</td>
<td>12071.00</td>
<td>6.14</td>
</tr>
<tr>
<td>MB-12b</td>
<td>11839.00</td>
<td>5.88</td>
<td>11843.00</td>
<td>5.92</td>
<td>11845.00</td>
<td>5.94</td>
<td>11848.00</td>
<td>5.97</td>
</tr>
<tr>
<td>MB-14a</td>
<td>11989.00</td>
<td>6.15</td>
<td>11992.00</td>
<td>6.18</td>
<td>11994.00</td>
<td>6.20</td>
<td>11998.00</td>
<td>6.23</td>
</tr>
<tr>
<td>MB-14b</td>
<td>11822.00</td>
<td>6.22</td>
<td>11825.00</td>
<td>6.24</td>
<td>11828.00</td>
<td>6.27</td>
<td>11831.00</td>
<td>6.30</td>
</tr>
<tr>
<td>MB-16a</td>
<td>11512.00</td>
<td>6.22</td>
<td>11513.00</td>
<td>6.23</td>
<td>11515.00</td>
<td>6.25</td>
<td>11520.00</td>
<td>6.29</td>
</tr>
<tr>
<td>MB-16b</td>
<td>11892.00</td>
<td>6.26</td>
<td>11896.00</td>
<td>6.30</td>
<td>11900.00</td>
<td>6.34</td>
<td>11902.00</td>
<td>6.35</td>
</tr>
<tr>
<td>LB-6a</td>
<td>10935.00</td>
<td>6.05</td>
<td>10938.00</td>
<td>6.08</td>
<td>10940.00</td>
<td>6.10</td>
<td>10942.00</td>
<td>6.12</td>
</tr>
<tr>
<td>UB-10a</td>
<td>12008.00</td>
<td>6.50</td>
<td>12013.00</td>
<td>6.55</td>
<td>12020.00</td>
<td>6.61</td>
<td>12023.00</td>
<td>6.63</td>
</tr>
</tbody>
</table>
## SUCTION TEST - PRESTON

Start date: 12/7/2002

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>dry wt.(g)</th>
<th>1HR</th>
<th>2HR</th>
<th>4HR</th>
<th>24HR</th>
<th>2 DAYS</th>
<th>3 DAYS</th>
<th>4 DAYS</th>
</tr>
</thead>
<tbody>
<tr>
<td>MB-6a</td>
<td>11731.00</td>
<td>11850.00</td>
<td>1.01</td>
<td>11881.00</td>
<td>1.28</td>
<td>11919.00</td>
<td>1.60</td>
<td>12095.00</td>
</tr>
<tr>
<td>MB-6b</td>
<td>11627.00</td>
<td>11746.00</td>
<td>1.02</td>
<td>11772.00</td>
<td>1.25</td>
<td>11805.00</td>
<td>1.53</td>
<td>11945.00</td>
</tr>
<tr>
<td>MB-8a</td>
<td>11101.00</td>
<td>11222.00</td>
<td>1.09</td>
<td>11250.00</td>
<td>1.34</td>
<td>11282.00</td>
<td>1.63</td>
<td>11443.00</td>
</tr>
<tr>
<td>MB-8b</td>
<td>11574.00</td>
<td>11689.00</td>
<td>0.99</td>
<td>11715.00</td>
<td>1.22</td>
<td>11748.00</td>
<td>1.50</td>
<td>11907.00</td>
</tr>
<tr>
<td>MB-10a</td>
<td>11572.00</td>
<td>11697.00</td>
<td>1.08</td>
<td>11725.00</td>
<td>1.32</td>
<td>11760.00</td>
<td>1.62</td>
<td>11925.00</td>
</tr>
<tr>
<td>MB-10b</td>
<td>11541.00</td>
<td>11628.00</td>
<td>0.75</td>
<td>11660.00</td>
<td>1.03</td>
<td>11695.00</td>
<td>1.33</td>
<td>11894.00</td>
</tr>
<tr>
<td>MB-12a</td>
<td>11493.00</td>
<td>11586.00</td>
<td>0.81</td>
<td>11613.00</td>
<td>1.04</td>
<td>11648.00</td>
<td>1.35</td>
<td>11822.00</td>
</tr>
<tr>
<td>MB-12b</td>
<td>11507.00</td>
<td>11620.00</td>
<td>0.98</td>
<td>11653.00</td>
<td>1.27</td>
<td>11694.00</td>
<td>1.63</td>
<td>11901.00</td>
</tr>
<tr>
<td>MB-14a</td>
<td>11627.00</td>
<td>11724.00</td>
<td>0.83</td>
<td>11752.00</td>
<td>1.08</td>
<td>11787.00</td>
<td>1.38</td>
<td>11973.00</td>
</tr>
<tr>
<td>MB-14b</td>
<td>11638.00</td>
<td>11730.00</td>
<td>0.79</td>
<td>11761.00</td>
<td>1.06</td>
<td>11799.00</td>
<td>1.38</td>
<td>12006.00</td>
</tr>
<tr>
<td>MB-16a</td>
<td>11546.00</td>
<td>11636.00</td>
<td>0.78</td>
<td>11664.00</td>
<td>1.02</td>
<td>11699.00</td>
<td>1.33</td>
<td>11880.00</td>
</tr>
<tr>
<td>MB-16b</td>
<td>11565.00</td>
<td>11665.00</td>
<td>0.86</td>
<td>11699.00</td>
<td>1.16</td>
<td>11740.00</td>
<td>1.51</td>
<td>11950.00</td>
</tr>
<tr>
<td>LB-6a</td>
<td>11506.00</td>
<td>11635.00</td>
<td>1.29</td>
<td>11674.00</td>
<td>1.46</td>
<td>11705.00</td>
<td>1.73</td>
<td>11839.00</td>
</tr>
<tr>
<td>UB-10a</td>
<td>11532.00</td>
<td>11647.00</td>
<td>1.00</td>
<td>11681.00</td>
<td>1.29</td>
<td>11722.00</td>
<td>1.65</td>
<td>11933.00</td>
</tr>
<tr>
<td>SAMPLE</td>
<td>5 DAYS</td>
<td>6 DAYS</td>
<td>7 DAYS</td>
<td>8 DAYS</td>
<td>9 DAYS</td>
<td>10 DAYS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>--------</td>
<td>---------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-6a</td>
<td>12396.00</td>
<td>5.67</td>
<td>12410.00</td>
<td>5.79</td>
<td>12420.00</td>
<td>5.87</td>
<td>12427.00</td>
<td>5.93</td>
</tr>
<tr>
<td>MB-6b</td>
<td>12202.00</td>
<td>4.95</td>
<td>12225.00</td>
<td>5.14</td>
<td>12244.00</td>
<td>5.31</td>
<td>12255.00</td>
<td>5.40</td>
</tr>
<tr>
<td>MB-8a</td>
<td>11714.00</td>
<td>5.52</td>
<td>11726.00</td>
<td>5.63</td>
<td>11734.00</td>
<td>5.70</td>
<td>11741.00</td>
<td>5.77</td>
</tr>
<tr>
<td>MB-8b</td>
<td>12190.00</td>
<td>5.32</td>
<td>12208.00</td>
<td>5.48</td>
<td>12221.00</td>
<td>5.59</td>
<td>12229.00</td>
<td>5.66</td>
</tr>
<tr>
<td>MB-10a</td>
<td>12205.00</td>
<td>5.47</td>
<td>12222.00</td>
<td>5.62</td>
<td>12234.00</td>
<td>5.72</td>
<td>12240.00</td>
<td>5.77</td>
</tr>
<tr>
<td>MB-10b</td>
<td>12195.00</td>
<td>5.67</td>
<td>12204.00</td>
<td>5.74</td>
<td>12212.00</td>
<td>5.81</td>
<td>12217.00</td>
<td>5.86</td>
</tr>
<tr>
<td>MB-12a</td>
<td>12119.00</td>
<td>5.45</td>
<td>12138.00</td>
<td>5.61</td>
<td>12149.00</td>
<td>5.71</td>
<td>12154.00</td>
<td>5.75</td>
</tr>
<tr>
<td>MB-12b</td>
<td>12185.00</td>
<td>5.89</td>
<td>12193.00</td>
<td>5.96</td>
<td>12199.00</td>
<td>6.01</td>
<td>12202.00</td>
<td>6.04</td>
</tr>
<tr>
<td>MB-14a</td>
<td>12291.00</td>
<td>5.71</td>
<td>12307.00</td>
<td>5.85</td>
<td>12317.00</td>
<td>5.93</td>
<td>12322.00</td>
<td>5.98</td>
</tr>
<tr>
<td>MB-14b</td>
<td>12314.00</td>
<td>5.81</td>
<td>12322.00</td>
<td>5.88</td>
<td>12329.00</td>
<td>5.94</td>
<td>12333.00</td>
<td>5.97</td>
</tr>
<tr>
<td>MB-16a</td>
<td>12196.00</td>
<td>5.63</td>
<td>12218.00</td>
<td>5.82</td>
<td>12231.00</td>
<td>5.93</td>
<td>12237.00</td>
<td>5.98</td>
</tr>
<tr>
<td>MB-16b</td>
<td>12259.00</td>
<td>6.00</td>
<td>12265.00</td>
<td>6.05</td>
<td>12273.00</td>
<td>6.12</td>
<td>12278.00</td>
<td>6.17</td>
</tr>
<tr>
<td>LB-6a</td>
<td>12048.00</td>
<td>4.71</td>
<td>12061.00</td>
<td>4.82</td>
<td>12070.00</td>
<td>4.90</td>
<td>12076.00</td>
<td>4.95</td>
</tr>
<tr>
<td>UB-10a</td>
<td>12257.00</td>
<td>6.29</td>
<td>12262.00</td>
<td>6.33</td>
<td>12269.00</td>
<td>6.39</td>
<td>12273.00</td>
<td>6.43</td>
</tr>
</tbody>
</table>
## SUCTION TEST - SHARP'S

Start date: 5/11/02

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>dry wt.(g)</th>
<th>1HR</th>
<th>% H2O</th>
<th>2HR</th>
<th>% H2O</th>
<th>4HR</th>
<th>% H2O</th>
<th>24HR</th>
<th>% H2O</th>
<th>2 DAYS</th>
<th>% H2O</th>
<th>3 DAYS</th>
<th>% H2O</th>
<th>4 DAYS</th>
<th>% H2O</th>
</tr>
</thead>
<tbody>
<tr>
<td>MB-6a</td>
<td>12711.00</td>
<td>12738.00</td>
<td>0.21</td>
<td>12743.00</td>
<td>0.25</td>
<td>12747.00</td>
<td>0.28</td>
<td>12777.00</td>
<td>0.52</td>
<td>12801.00</td>
<td>0.71</td>
<td>12820.00</td>
<td>0.86</td>
<td>12836.00</td>
<td>0.98</td>
</tr>
<tr>
<td>MB-6b</td>
<td>12788.00</td>
<td>12814.00</td>
<td>0.20</td>
<td>12818.00</td>
<td>0.23</td>
<td>12822.00</td>
<td>0.27</td>
<td>12845.00</td>
<td>0.45</td>
<td>12866.00</td>
<td>0.61</td>
<td>12880.00</td>
<td>0.72</td>
<td>12890.00</td>
<td>0.80</td>
</tr>
<tr>
<td>MB-8a</td>
<td>12764.00</td>
<td>12797.00</td>
<td>0.26</td>
<td>12800.00</td>
<td>0.28</td>
<td>12802.00</td>
<td>0.30</td>
<td>12829.00</td>
<td>0.51</td>
<td>12851.00</td>
<td>0.68</td>
<td>12869.00</td>
<td>0.82</td>
<td>12884.00</td>
<td>0.94</td>
</tr>
<tr>
<td>MB-8b</td>
<td>12886.00</td>
<td>12912.00</td>
<td>0.20</td>
<td>12915.00</td>
<td>0.23</td>
<td>12918.00</td>
<td>0.25</td>
<td>12941.00</td>
<td>0.43</td>
<td>12964.00</td>
<td>0.61</td>
<td>12978.00</td>
<td>0.71</td>
<td>12989.00</td>
<td>0.80</td>
</tr>
<tr>
<td>MB-10a</td>
<td>12804.00</td>
<td>12824.00</td>
<td>0.16</td>
<td>12828.00</td>
<td>0.19</td>
<td>12836.00</td>
<td>0.25</td>
<td>12878.00</td>
<td>0.58</td>
<td>12913.00</td>
<td>0.85</td>
<td>12937.00</td>
<td>1.04</td>
<td>12954.00</td>
<td>1.17</td>
</tr>
<tr>
<td>MB-10b</td>
<td>12656.00</td>
<td>12681.00</td>
<td>0.20</td>
<td>12687.00</td>
<td>0.24</td>
<td>12694.00</td>
<td>0.30</td>
<td>12739.00</td>
<td>0.66</td>
<td>12777.00</td>
<td>0.96</td>
<td>12806.00</td>
<td>1.19</td>
<td>12826.00</td>
<td>1.34</td>
</tr>
<tr>
<td>MB-12a</td>
<td>12898.00</td>
<td>12935.00</td>
<td>0.29</td>
<td>12942.00</td>
<td>0.34</td>
<td>12950.00</td>
<td>0.40</td>
<td>13004.00</td>
<td>0.82</td>
<td>13045.00</td>
<td>1.14</td>
<td>13076.00</td>
<td>1.38</td>
<td>13096.00</td>
<td>1.54</td>
</tr>
<tr>
<td>MB-12b</td>
<td>12748.00</td>
<td>12822.00</td>
<td>0.58</td>
<td>12828.00</td>
<td>0.63</td>
<td>12836.00</td>
<td>0.69</td>
<td>12893.00</td>
<td>1.14</td>
<td>12940.00</td>
<td>1.51</td>
<td>12972.00</td>
<td>1.76</td>
<td>12989.00</td>
<td>1.89</td>
</tr>
<tr>
<td>MB-14a</td>
<td>12675.00</td>
<td>12721.00</td>
<td>0.36</td>
<td>12726.00</td>
<td>0.40</td>
<td>12734.00</td>
<td>0.47</td>
<td>12783.00</td>
<td>0.85</td>
<td>12822.00</td>
<td>1.16</td>
<td>12848.00</td>
<td>1.36</td>
<td>12866.00</td>
<td>1.51</td>
</tr>
<tr>
<td>MB-14b</td>
<td>12762.00</td>
<td>12795.00</td>
<td>0.26</td>
<td>12801.00</td>
<td>0.31</td>
<td>12808.00</td>
<td>0.36</td>
<td>12856.00</td>
<td>0.74</td>
<td>12893.00</td>
<td>1.03</td>
<td>12922.00</td>
<td>1.25</td>
<td>12941.00</td>
<td>1.40</td>
</tr>
<tr>
<td>MB-16a</td>
<td>12795.00</td>
<td>12846.00</td>
<td>0.40</td>
<td>12853.00</td>
<td>0.45</td>
<td>12862.00</td>
<td>0.52</td>
<td>12917.00</td>
<td>0.95</td>
<td>12959.00</td>
<td>1.28</td>
<td>12991.00</td>
<td>1.53</td>
<td>13010.00</td>
<td>1.68</td>
</tr>
<tr>
<td>MB-16b</td>
<td>12765.00</td>
<td>12809.00</td>
<td>0.34</td>
<td>12818.00</td>
<td>0.42</td>
<td>12828.00</td>
<td>0.49</td>
<td>12887.00</td>
<td>0.96</td>
<td>12937.00</td>
<td>1.35</td>
<td>12971.00</td>
<td>1.61</td>
<td>12992.00</td>
<td>1.78</td>
</tr>
<tr>
<td>LB-6a</td>
<td>12372.00</td>
<td>12428.00</td>
<td>0.45</td>
<td>12428.00</td>
<td>0.45</td>
<td>12434.00</td>
<td>0.50</td>
<td>12494.00</td>
<td>0.99</td>
<td>12534.00</td>
<td>1.31</td>
<td>12563.00</td>
<td>1.54</td>
<td>12581.00</td>
<td>1.69</td>
</tr>
<tr>
<td>UB-10a</td>
<td>12843.00</td>
<td>12876.00</td>
<td>0.26</td>
<td>12885.00</td>
<td>0.33</td>
<td>12895.00</td>
<td>0.40</td>
<td>12949.00</td>
<td>0.83</td>
<td>12991.00</td>
<td>1.15</td>
<td>13020.00</td>
<td>1.38</td>
<td>13037.00</td>
<td>1.51</td>
</tr>
<tr>
<td>TIME</td>
<td>5 DAYS % H₂O</td>
<td>6 DAYS % H₂O</td>
<td>7 DAYS % H₂O</td>
<td>8 DAYS % H₂O</td>
<td>9 DAYS % H₂O</td>
<td>10 DAYS % H₂O</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>------</td>
<td>--------------</td>
<td>--------------</td>
<td>--------------</td>
<td>--------------</td>
<td>--------------</td>
<td>--------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-6a</td>
<td>12849.00 1.09</td>
<td>12862.00 1.19</td>
<td>12871.00 1.26</td>
<td>12874.00 1.28</td>
<td>12879.00 1.32</td>
<td>12883.00 1.35</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-6b</td>
<td>12898.00 0.86</td>
<td>12908.00 0.94</td>
<td>12913.00 0.98</td>
<td>12915.00 0.99</td>
<td>12919.00 1.02</td>
<td>12922.00 1.05</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-8a</td>
<td>12896.00 1.03</td>
<td>12908.00 1.13</td>
<td>12915.00 1.18</td>
<td>12919.00 1.21</td>
<td>12925.00 1.26</td>
<td>12929.00 1.29</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-8b</td>
<td>12996.00 0.85</td>
<td>13006.00 0.93</td>
<td>13011.00 0.97</td>
<td>13013.00 0.99</td>
<td>13018.00 1.02</td>
<td>13021.00 1.05</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-10a</td>
<td>12965.00 1.26</td>
<td>12977.00 1.35</td>
<td>12982.00 1.39</td>
<td>12985.00 1.41</td>
<td>12991.00 1.46</td>
<td>12996.00 1.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-10b</td>
<td>12838.00 1.44</td>
<td>12849.00 1.52</td>
<td>12855.00 1.57</td>
<td>12857.00 1.59</td>
<td>12860.00 1.61</td>
<td>12864.00 1.64</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-12a</td>
<td>13109.00 1.64</td>
<td>13121.00 1.73</td>
<td>13126.00 1.77</td>
<td>13128.00 1.78</td>
<td>13133.00 1.82</td>
<td>13136.00 1.85</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-12b</td>
<td>12998.00 1.96</td>
<td>13008.00 2.04</td>
<td>13011.00 2.06</td>
<td>13013.00 2.08</td>
<td>13016.00 2.10</td>
<td>13018.00 2.12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-14a</td>
<td>12875.00 1.58</td>
<td>12881.00 1.63</td>
<td>12884.00 1.65</td>
<td>12886.00 1.66</td>
<td>12889.00 1.69</td>
<td>12893.00 1.72</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-14b</td>
<td>12953.00 1.50</td>
<td>12965.00 1.59</td>
<td>12970.00 1.63</td>
<td>12972.00 1.65</td>
<td>12977.00 1.68</td>
<td>12982.00 1.72</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-16a</td>
<td>13021.00 1.77</td>
<td>13032.00 1.85</td>
<td>13037.00 1.89</td>
<td>13040.00 1.91</td>
<td>13045.00 1.95</td>
<td>13050.00 1.99</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MB-16b</td>
<td>13003.00 1.86</td>
<td>13014.00 1.95</td>
<td>13018.00 1.98</td>
<td>13021.00 2.01</td>
<td>13025.00 2.04</td>
<td>13029.00 2.07</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LB-6a</td>
<td>12588.00 1.75</td>
<td>12600.00 1.84</td>
<td>12603.00 1.87</td>
<td>12604.00 1.88</td>
<td>12606.00 1.89</td>
<td>12608.00 1.91</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UB-10a</td>
<td>13052.00 1.63</td>
<td>13064.00 1.72</td>
<td>13070.00 1.77</td>
<td>13074.00 1.80</td>
<td>13079.00 1.84</td>
<td>13084.00 1.88</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix E
Cyclic and Staged Triaxial Test Stress vs. Strain Plots
Figure E.1: Sharps quarry cyclic triaxial- L.B. 6% B

Figure E.2: Sharps quarry staged triaxial- L.B. 6% B
Figure E.3: Sharps quarry cyclic triaxial- L.B. 6% 2B

Figure E.4: Sharps quarry staged triaxial- L.B. 6% 2B
Figure E.5: Sharps quarry cyclic triaxial - L.B. 6% C

Figure E.6: Sharps quarry staged triaxial - L.B. 6% C
Figure E.7: Sharps quarry cyclic triaxial- M.B. 6% A

Figure E.8: Sharps quarry staged triaxial- M.B. 6% A
Figure E.9: Sharps quarry cyclic triaxial- M.B. 6% B

Figure E.10: Sharps quarry staged triaxial- M.B. 6% B
Figure E.11: Sharps quarry cyclic triaxial- M.B. 8% A

Figure E.12: Sharps quarry staged triaxial- M.B. 8% A
Figure E.13: Sharps quarry cyclic triaxial- M.B. 8% B

Figure E.14: Sharps quarry staged triaxial- M.B. 8% B
Figure E.15: Sharps quarry cyclic triaxial- M.B. 8% B2

Figure E.16: Sharps quarry staged triaxial- M.B. 8% B2

355
Figure E.17: Sharps quarry cyclic triaxial- M.B. 10% A

Figure E.18: Sharps quarry staged triaxial- M.B. 10% A
Figure E.19: Sharps quarry cyclic triaxial- M.B. 10% B

Figure E.20: Sharps quarry staged triaxial- M.B. 10% B
Figure E.21: Sharps quarry cyclic triaxial- M.B. 12% A

Figure E.22: Sharps quarry staged triaxial- M.B. 12% A
Figure E.23: Sharps quarry cyclic triaxial- M.B. 12% B

Figure E.24: Sharps quarry staged triaxial- M.B. 12% B
Figure E.25: Sharps quarry cyclic triaxial- M.B. 14% A

Figure E.26: Sharps quarry staged triaxial- M.B. 14% A
Figure E.27: Sharps quarry cyclic triaxial- M.B. 14% B

Figure E.28: Sharps quarry staged triaxial- M.B. 14% B
Figure E.29: Sharps quarry cyclic triaxial- M.B. 16% A

Figure E.30: Sharps quarry cyclic triaxial- M.B. 16% A
Figure E.31: Sharps quarry cyclic triaxial- M.B. 16% B

Figure E.32: Sharps quarry staged triaxial- M.B. 16% B
Figure E.33: Sharps quarry cyclic triaxial- U.B. 10% A

Figure E.34: Sharps quarry staged triaxial- U.B. 10% A
Figure E.35: Sharps quarry cyclic triaxial - U.B. 10% C

Figure E.36: Sharps quarry staged triaxial - U.B. 10% C
Figure E.37: Preston quarry cyclic triaxial- L.B. 6% A

Figure E.38: Preston quarry staged triaxial- L.B. 6% A
Figure E.39: Preston quarry cyclic triaxial- L.B. 6% B

Figure E.40: Preston quarry staged triaxial- L.B. 6% B
Figure E.41: Preston quarry cyclic triaxial- M.B. 6% A

Figure E.42: Preston quarry staged triaxial- M.B. 6% A
Figure E.43: Preston quarry cyclic triaxial- M.B. 6% B

Figure E.44: Preston quarry staged triaxial- M.B. 6% B
Figure E.45: Preston quarry cyclic triaxial- M.B. 8% A

Figure E.46: Preston quarry staged triaxial- M.B. 8% A
Figure E.47: Preston quarry cyclic triaxial- M.B. 8% B

Figure E.48: Preston quarry staged triaxial- M.B. 8% B
Figure E.49: Preston quarry cyclic triaxial- M.B. 10% A

Figure E.50: Preston quarry staged triaxial- M.B. 10% A
Figure E.51: Preston quarry cyclic triaxial- M.B. 10% B

Figure E.52: Preston quarry staged triaxial- M.B. 10% B
Figure E.53: Preston quarry cyclic triaxial- M.B. 12% A

Figure E.54: Preston quarry cyclic triaxial- M.B. 12% A
Figure E.55: Preston quarry cyclic triaxial- M.B. 12% B

Figure E.56: Preston quarry staged triaxial- M.B. 12% B
Figure E.57: Preston quarry cyclic triaxial - M.B. 14% A

Figure E.58: Preston quarry staged triaxial - M.B. 14% A
Figure E.59: Preston quarry cyclic triaxial - M.B. 14% B

Figure E.60: Preston quarry staged triaxial - M.B. 14% B
Figure E.61: Preston quarry cyclic triaxial- M.B. 16% A

Figure E.62: Preston quarry staged triaxial- M.B. 16% A
Figure E.63: Preston quarry cyclic triaxial- M.B. 16% B

Figure E.64: Preston quarry cyclic triaxial- M.B. 16% B
Figure E.65: Preston quarry cyclic triaxial- U.B. 10% A

Figure E.66: Preston quarry staged triaxial- U.B. 10% A
Figure E.67: Preston quarry cyclic triaxial- U.B. 10% B

Figure E.68: Preston quarry staged triaxial- U.B. 10% B
Figure E.69: Black Rock quarry cyclic triaxial- L.B. 6% A.

Figure E.70: Black Rock quarry staged triaxial- L.B. 6% A.
Figure E.71: Black Rock quarry cyclic triaxial- L.B. 6% B.

Figure E.72: Black Rock quarry staged triaxial- L.B. 6% B.
Figure E.73: Black Rock quarry cyclic triaxial - M.B. 6% A.

Figure E.74: Black Rock quarry staged triaxial - M.B. 6% A.
Figure E.75: Black Rock quarry cyclic triaxial- M.B. 6% B.

Figure E.76: Black Rock quarry staged triaxial- M.B. 6% B.
Figure E.77: Black Rock quarry cyclic triaxial- M.B. 8% A.

Figure E.78: Black Rock quarry staged triaxial- M.B. 8% A.
Figure E.79: Black Rock quarry cyclic triaxial- M.B. 8% B.

Figure E.80: Black Rock quarry staged triaxial- M.B. 8% B.
Figure E.81: Black Rock quarry cyclic triaxial- M.B. 10% A.

Figure E.82: Black Rock quarry staged triaxial- M.B. 10% A.
Figure E.83: Black Rock quarry cyclic triaxial- M.B. 10% B.

Figure E.84: Black Rock quarry staged triaxial- M.B. 10% B.
Figure E.85: Black Rock quarry cyclic triaxial- M.B. 12% A.

Figure E.86: Black Rock quarry staged triaxial- M.B. 12% A.
Figure E.87: Black Rock quarry cyclic triaxial- M.B. 12% B.

Figure E.88: Black Rock quarry staged triaxial- M.B. 12% B.
Figure E.89: Black Rock quarry cyclic triaxial- M.B. 14% A.

Figure E.90: Black Rock quarry staged triaxial- M.B. 14% A.
Figure E.91: Black Rock quarry cyclic triaxial- M.B. 14% B.

Figure E.92: Black Rock quarry staged triaxial- M.B. 14% B.
Figure E.93: Black Rock quarry cyclic triaxial- M.B. 16% A.

Figure E.94: Black Rock quarry staged triaxial- M.B. 16% A.
Figure E.95: Black Rock quarry cyclic triaxial- M.B. 16% B.

Figure E.96: Black Rock quarry staged triaxial- M.B. 16% B.
Figure E.97: Black Rock quarry cyclic triaxial- U.B. 10% A.

Figure E.98: Black Rock quarry staged triaxial- U.B. 10% A.
Figure E.99: Black Rock quarry cyclic triaxial- U.B. 10% B.

Figure E.100: Black Rock quarry staged triaxial- U.B. 10% B.

397
Figure E.101: Black Rock quarry cyclic triaxial- U.B. 10% C.

Figure E.102: Black Rock quarry staged triaxial- U.B. 10% C.
Figure E.103: Glen Rose quarry cyclic triaxial- L.B. 6% A.

Figure E.104: Glen Rose quarry staged triaxial- L.B. 6% A.
Figure E.105: Glen Rose quarry cyclic triaxial- L.B. 6% B.

Figure E.106: Glen Rose quarry staged triaxial- L.B. 6% B.
Figure E.107: Glen Rose quarry cyclic triaxial- M.B. 6% A.

Figure E.108: Glen Rose quarry staged triaxial- M.B. 6% A.
Figure E.109: Glen Rose quarry cyclic triaxial- M.B. 6% B.

Figure E.110: Glen Rose quarry staged triaxial- M.B. 6% B.
Figure E.111: Glen Rose quarry cyclic triaxial- M.B. 8% A.

Figure E.112: Glen Rose quarry staged triaxial- M.B. 8% A.
Figure E.113: Glen Rose quarry cyclic triaxial- M.B. 8% B.

Figure E.114: Glen Rose quarry staged triaxial- M.B. 8% B.
Figure E.115: Glen Rose quarry cyclic triaxial- M.B. 10% A.

Figure E.116: Glen Rose quarry staged triaxial- M.B. 10% A.
Figure E.117: Glen Rose quarry cyclic triaxial- M.B. 10% B.

Figure E.118: Glen Rose quarry staged triaxial- M.B. 10% B.

406
Figure E.119: Glen Rose quarry cyclic triaxial- M.B. 12% A.

Figure E.120: Glen Rose quarry staged triaxial- M.B. 12% A.
Figure E.121: Glen Rose quarry cyclic triaxial- M.B. 12% B.

Figure E.122: Glen Rose quarry staged triaxial- M.B. 12% B.
Figure E.123: Glen Rose quarry cyclic triaxial- M.B. 14% A.

Figure E.124: Glen Rose quarry staged triaxial- M.B. 14% A.
Figure E.125: Glen Rose quarry cyclic triaxial - M.B. 14% B.

Figure E.126: Glen Rose quarry staged triaxial - M.B. 14% B.
Figure E.127: Glen Rose quarry cyclic triaxial- M.B. 16% A.

Figure E.128: Glen Rose quarry staged triaxial- M.B. 16% A.
Figure E.129: Glen Rose quarry cyclic triaxial- M.B. 16% B.

Figure E.130: Glen Rose quarry staged triaxial- M.B. 16% B.
Figure E.131 Glen Rose quarry cyclic triaxial- U.B. 10% A.

Figure E.132: Glen Rose quarry staged triaxial- U.B. 10% A.
Figure E.133: Glen Rose quarry cyclic triaxial- U.B. 10% B.

Figure E.134: Glen Rose quarry staged triaxial- U.B. 10% B.
Figure E.135: Granite Mountain quarry cyclic triaxial- L.B. 6% A.

Figure E.136: Granite Mountain quarry staged triaxial- L.B. 6% A.
Figure E.137: Granite Mountain quarry cyclic triaxial- L.B. 6% B.

Figure E.138: Granite Mountain quarry staged triaxial- L.B. 6% B.
Figure E.139: Granite Mountain quarry cyclic triaxial- M.B. 6% A.

Figure E.140: Granite Mountain quarry staged triaxial- M.B. 6% A.
Figure E.141: Granite Mountain quarry cyclic triaxial- M.B. 6% B.

Figure E.142: Granite Mountain quarry staged triaxial- M.B. 6% B.
Figure E.143: Granite Mountain quarry cyclic triaxial- M.B. 8% A.

Figure E.144: Granite Mountain quarry staged triaxial- M.B. 8% A.
Figure E.145: Granite Mountain quarry cyclic triaxial- M.B. 8% B.

Figure E.146: Granite Mountain quarry staged triaxial- M.B. 8% B.
Figure E.147: Granite Mountain quarry cyclic triaxial- M.B. 10% A.

Figure E.148: Granite Mountain quarry staged triaxial- M.B. 10% A.
Figure E.149: Granite Mountain quarry cyclic triaxial- M.B. 10% B.

Figure E.150: Granite Mountain quarry staged triaxial- M.B. 10% B.
Figure E.151: Granite Mountain quarry cyclic triaxial- M.B. 12% A.

Figure E.152: Granite Mountain quarry staged triaxial- M.B. 12% A.
Figure E.153: Granite Mountain quarry cyclic triaxial- M.B. 12% B.

Figure E.154: Granite Mountain quarry staged triaxial- M.B. 12% B.
Figure E.155: Granite Mountain quarry cyclic triaxial- M.B. 14% A.

Figure E.156: Granite Mountain quarry staged triaxial- M.B. 14% A.
Figure E.157: Granite Mountain quarry cyclic triaxial- M.B. 14% B.

Figure E.158: Granite Mountain quarry staged triaxial- M.B. 14% B.
Figure E.159: Granite Mountain quarry cyclic triaxial- M.B. 16% A.

Figure E.160: Granite Mountain quarry staged triaxial- M.B. 16% A.
Figure E.161: Granite Mountain quarry cyclic triaxial - M.B. 16% B.

Figure E.162: Granite Mountain quarry staged triaxial - M.B. 16% B.
Figure E.163: Granite Mountain quarry cyclic triaxial- U.B. 10% A.

Figure E.164: Granite Mountain quarry staged triaxial- U.B. 10% A.
Figure E.165: Granite Mountain quarry cyclic triaxial- U.B. 10% B.

Figure F.166: Granite Mountain quarry staged triaxial- U.B. 10% B.
Appendix F
Mohr-Coulomb Failure Envelope Plots
Figure F.1: Sharps quarry L.B. 6% 2B consolidated drained strength.

Figure F.2: Sharps quarry L.B. 6% B consolidated drained strength.
Figure F.3: Sharps quarry L.B. 6% C consolidated drained strength.

Figure F.4: Sharps quarry M.B. 6% A consolidated drained strength.
Figure F.5: Sharps quarry M.B. 6% B consolidated drained strength.

Figure F.6: Sharps quarry M.B. 8% A consolidated drained strength.
Figure F.7: Sharps quarry M.B. 8% B consolidated drained strength.

Figure F.8: Sharps quarry M.B. 8% B2 consolidated drained strength.
Figure F.9: Sharps quarry M.B. 10% A consolidated drained strength.

Figure F.10: Sharps quarry M.B. 10% B consolidated drained strength.
Figure F.11: Sharps quarry M.B. 12% A consolidated drained strength.

Figure F.12: Sharps quarry M.B. 12% B consolidated drained strength.
Figure F.13: Sharps quarry M.B. 14% A consolidated drained strength.

Figure F.14: Sharps quarry M.B. 14% B consolidated drained strength.
Figure F.15: Sharps quarry M.B. 16% A consolidated drained strength.

Figure F.16: Sharps quarry M.B. 16% B consolidated drained strength.
Figure F.17: Sharps quarry U.B. 10% A consolidated drained strength.

Figure F.18: Sharps quarry U.B. 10% C consolidated drained strength.
Figure F.19: Preston quarry L.B. 6% A consolidated drained strength.

Figure F.20: Preston quarry L.B. 6% B consolidated drained strength.
Figure F.21: Preston quarry M.B. 6% A consolidated drained strength.

Figure F.22: Preston quarry M.B. 6% B consolidated drained strength.
Figure F.23: Preston quarry M.B. 8% A consolidated drained strength.

Figure F.24: Preston quarry M.B. 8% B consolidated drained strength.
Figure F.25: Preston quarry M.B. 10% A consolidated drained strength.

Figure F.26: Preston quarry M.B. 10% B consolidated drained strength.
Figure F.27: Preston quarry M.B. 12% A consolidated drained strength.

Figure F.28: Preston quarry M.B. 12% B consolidated drained strength.
Figure F.29: Preston quarry M.B. 14% A consolidated drained strength.

Figure F.30: Preston quarry M.B. 14% B consolidated drained strength.
Figure F.31: Preston quarry M.B. 16% A consolidated drained strength.

Figure F.32: Preston quarry M.B. 16% B consolidated drained strength.
Figure F.33: Preston quarry U.B. 10% A consolidated drained strength.

Figure F.34: Preston quarry U.B. 10% B consolidated drained strength.
Figure F.35: Black Rock quarry L.B. 6% A consolidated drained strength.

Figure F.36: Black Rock quarry L.B. 6% B consolidated drained strength.
Figure F.37: Black Rock quarry M.B. 6% A consolidated drained strength.

Figure F.38: Black Rock quarry M.B. 6% B consolidated drained strength.
Figure F.39: Black Rock quarry M.B. 8% A consolidated drained strength.

Figure F.40: Black Rock quarry M.B. 8% B consolidated drained strength.
Figure F.41: Black Rock quarry M.B. 10% A consolidated drained strength.

Figure F.42: Black Rock quarry M.B. 10% B consolidated drained strength.
Figure F.43: Black Rock quarry M.B. 12% A consolidated drained strength.

Figure F.44: Black Rock quarry M.B. 12% B consolidated drained strength.
Figure F.45: Black Rock quarry M.B. 14% A consolidated drained strength.

Figure F.46: Black Rock quarry M.B. 14% B consolidated drained strength.
Figure F.47: Black Rock quarry M.B. 16% A consolidated drained strength.

Figure F.48: Black Rock quarry M.B. 16% B consolidated drained strength.
Figure F.49: Black Rock quarry U.B. 10% A consolidated drained strength.

Figure F.50: Black Rock quarry U.B. 10% B consolidated drained strength.
Figure F.51: Black Rock quarry U.B. 10% C consolidated drained strength.

Figure F.52: Glen Rose quarry L.B. 6% consolidated drained strength.
Figure F.53: Glen Rose quarry L.B. 6% B consolidated drained strength.

Figure F.54: Glen Rose quarry M.B. 6% A consolidated drained strength.
Figure F.55: Glen Rose quarry M.B. 6% B consolidated drained strength.

Figure F.56: Glen Rose quarry M.B. 8% A consolidated drained strength.
Figure F.57: Glen Rose quarry M.B. 8% B consolidated drained strength.

Figure F.58: Glen Rose quarry M.B. 10% A consolidated drained strength.
Figure F.59: Glen Rose quarry M.B. 10% B consolidated drained strength.

Figure F.60: Glen Rose quarry M.B. 12% A consolidated drained strength.
Figure F.61: Glen Rose quarry M.B. 12% B consolidated drained strength.

Figure F.62: Glen Rose quarry M.B. 14% A consolidated drained strength.
Figure F.63: Glen Rose quarry M.B. 14% B consolidated drained strength.

Figure F.64: Glen Rose quarry M.B. 16% A consolidated drained strength.
\[ \phi = 46.5 \, \text{deg.} \]

\[ c = 5.3 \, \text{psi} \]

Figure F.65: Glen Rose quarry M.B. 16% B consolidated drained strength.

\[ \phi = 42.5 \, \text{deg.} \]

\[ c = 7.4 \, \text{psi} \]

Figure F.66: Glen Rose quarry U.B. 10% A consolidated drained strength.
Figure F.67: Glen Rose quarry U.B. 10% B consolidated drained strength.

Figure F.68: Granite Mountain quarry L.B. 6% A consolidated drained strength.
Figure F.69: Granite Mountain quarry L.B. 6% B consolidated drained strength.

Figure F.70: Granite Mountain quarry M.B. 6% A consolidated drained strength.
Figure F.71: Granite Mountain quarry M.B. 6% B consolidated drained strength.

Figure F.72: Granite Mountain quarry M.B. 8% A consolidated drained strength.
Figure F.73: Granite Mountain quarry M.B. 8% B consolidated drained strength.

Figure F.74: Granite Mountain quarry M.B. 10% A consolidated drained strength.
Figure F.75: Granite Mountain quarry M.B. 10% B consolidated drained strength.

Figure F.76: Granite Mountain quarry M.B. 12% A consolidated drained strength.
Figure F.77: Granite Mountain quarry M.B. 12% B consolidated drained strength.

Figure F.78: Granite Mountain quarry M.B. 14% A consolidated drained strength.
Figure F.79: Granite Mountain quarry M.B. 14% B consolidated drained strength.

Figure F.80: Granite Mountain quarry M.B. 16% A consolidated drained strength.
Figure F.81: Granite Mountain quarry M.B. 16% B consolidated drained strength.

Figure F.82: Granite Mountain quarry U.B. 10% A consolidated drained strength.
$c = 9.5 \text{ psi}$

Figure F.83: Granite Mountain quarry U.B. 10% B consolidated drained strength.