An Attempt to Estimate the Resilient Modulus of Construction Materials from Basic Soil Tests

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ABSTRACT

Subgrade soil characterization, expressed in terms of Resilient Modulus (M_R), has become crucial for pavement design. For a new design, resilient modulus is generally obtained by conducting repeated triaxial tests on reconstituted/undisturbed cylindrical specimens. Because of the complexities encountered with the test and unavailability of resources, in terms of manpower and the necessary equipment to develop the M_R test data required for immediate use in design projects, reliable correlations with other laboratory tests are desirable. This research is aimed at establishing an indirect method for subgrade characterization through correlation between California Bearing Ratio (CBR) and basic soil tests, and then providing an approximate relationship for resilient modulus using CBR. The CBR test is relatively quick and simple to operate, and gives an immediate result. It can be carried out on undisturbed or recompacted materials, and can be performed in the field, in a small site laboratory, or in a main laboratory. Soil samples from ten pavement sections reflecting a range of typical subgrade materials in Delaware are selected and tested using standard Proctor compaction and CBR tests. Other soil physical properties are also determined, classifying the soil according to the American Association of State Highway and Transportation Officials (AASHTO) procedure. Regression studies have been performed on the results of the test data to obtain a general

relation between the CBR and soil physical properties. One broad class of soils--granular materials--was identified and created to yield reasonable results, and after several trial combinations a general model equation was developed for the soil class.

Chapter 1

INTRODUCTION

1.1   Background and Motivation

The objective of any pavement design is to ensure that permanent deformation of its structural components are kept within tolerable limits so as not to cause cracking or any undesirable distresses to the surfacing material. Knowledge of pavement layer stiffness is an important factor in pavement design. Unbound aggregates are extensively used in the unstabilized bases and subbases of flexible highway and airport pavements to provide the load distribution through aggregate interlock that is essential to the integrity of the pavement. As the loading and performance requirements of pavements continually increase, a better basic understanding of the response of unbound aggregates to repeated loading is needed.

In the original AASHTO Guide for design of pavement structures, published in 1961 and revised in 1972, the subgrade stiffness was accounted for by assigning a Soil Support Value (SSV) that has a scale ranging from 1 – 10. In 1986, the AASHTO Guide was substantially revised to include replacement of SSV with subgrade Resilient Modulus, (M_R). Resilient modulus values may be estimated directly from laboratory testing, indirectly through correlation with other laboratory/field tests, or backcalculated

from deflection measurements. The empirical nature of traditional pavement design methods have been a topic of discussion in highway engineering for many years. These methods rely on empirical rules developed through long-term experience with certain types of pavement and certain types of pavement construction materials under certain conditions. The main limitation of empirical methods is that they cannot be extrapolated with confidence beyond those conditions on which they are based. The essential need for pavement design procedures that are able to cater for varying design situations has led to widespread research efforts to develop so-called analytical or mechanistic design techniques. In the analytical approach, the road pavement is treated as a structure, and its mechanical behavior is evaluated in terms of load-carrying parameters in a similar manner to that used for concrete and steelwork structures. A conditional prerequisite for the success of the mechanistic approach is that the behavior of the constituent materials is properly understood. Resilient Modulus (M_R) has become a well-known parameter to characterize unbound pavement materials since a large amount of evidence has shown that the elastic (resilient) pavement deflection possesses a better correlation to field performance than total pavement deflection [Witczak, Qi, and Mirza 1995].

The resilient modulus has been identified as a valuable input necessary for modeling the material with the mechanistic pavement design and evaluation. For a new design, resilient modulus values are generally obtained by conducting repeated triaxial tests on reconstituted/undisturbed cylindrical specimens. The laboratory test is tedious, costly and time-consuming procedure. Large numbers of samples need to be collected and tested for reasonably accurate results. Even then, it is difficult to reproduce the in-

situ conditions of the sample [Houston, Mamlouk, and Perera 1992]. Because of the complexity and equipment requirement of repeated load testing, it has become necessary to develop approximate methods for estimation of resilient modulus through correlation with other laboratory/field tests, or backcalculated from deflection measurements. The AASHTO guide for pavement structure has suggested to transportation agencies involved in pavement design to establish correlations based on commonly used standard soil tests such as the California Bearing Ratio (CBR) and basic engineering index properties to obtain design values of M_R. Statistical correlations between resilient modulus and engineering index properties have become useful in practice because the index properties are easy and inexpensive to evaluate.

1.2 Statement of the Problem

Earlier versions of the AASHTO flexible pavement design procedures are based on the empirical data gathered at the AASHO Road Tests. The 2002 AASHTO's Guide for Design of Pavement Structure will be based on a mechanistic-empirical (M-E) approach, and require the engineering properties of the pavement layers to be obtained. The mechanistic-empirical flexible pavement design combines the elements of mechanical modeling and performance observations in determining the required pavement thickness for a set of design conditions. The major components are input (material, traffic, environmental), structural models, transfer functions, and reliability. The advantages of mechanistic design over traditional procedures are as follows:

- Consideration of changing load types,

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- Better utilization and characterization of available materials

- Predict specific failure modes so they can be eliminated,

- Relation of material properties to actual pavement performance,

- Better definition of existing pavement layer properties,

- Accommodation of environmental and aging effects.

The mechanistic characterization of paving materials will permit the application of the principles of engineering mechanics to the pavement analysis problem. Stiffness (moduli) of the various layers relates directly to how those layers respond to applied wheel loads in terms of fatigue cracking and rutting. Similarly, thermal properties relate to how pavement layers behave in response to temperature changes. A system of hierarchical approach to design inputs will allow the devotion of resources to material characterization commensurate with the priority of the pavement design under consideration. The approach is employed with regard to traffic, material, and environmental inputs. The choices of input level range from project-specific testing to the use of regional or national default values. The reliability of the pavement design would be directly related to the level of the inputs. In keeping with the hierarchical approach, materials characterization is comprised of three input levels. Level 1 represents a design approach philosophy of the highest practically achievable reliability, and Levels 2 and 3 have successively lower reliability. A general tabulation of elastic modulus characterization methods is given in Table 1.1.

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Table 1.1: Elastic modulus characterization. From NCHRP Project 1-37A [2002]

MATERIAL	INPUT LEVEL 1	INPUT LEVEL 2	INPUT LEVEL 3
Asphalt Concrete	Measured dynamic modulus	Estimated dynamic modulus	Default dynamic modulus
Portland cement Concrete	Measured elastic modulus	Estimated elastic modulus	Default elastic modulus
Stabilized materials	Measured modulus	Estimated modulus	Default modulus
Granular materials	Measured resilient modulus	Estimated resilient modulus	Default resilient modulus
Subgrades	Measured resilient modulus	Estimated resilient modulus	Default resilient modulus

The modulus of unbound bases, subbases, and subgrades is characterized in terms of the resilient modulus (M_R) first introduced as a design input parameter in 1986 AASHTO Guide. While M_R officially replaced older design parameters such as soil support value, it never gained widespread support or acceptance by the pavement community. This is expected to change at the completion of NCHRP Project 1-28A on the development of a "harmonized" M_R test protocol which makes use of a universal non-linear resilient modulus model applicable for all types of unbound pavement materials, ranging from very plastic clays to clean granular bases. The universality of the model stems from its ability to conceptually represent all types of soils, from pure cohesive to pure cohesionless. In the 2002 Guide, it is expected that a modification to the model will make use of the Integrated Climate Model to adjust for changes in seasonal moisture content. Unbound material inputs are summarized in Table 1.2.

Table 1.2: Unbound Material Inputs (Bases/Subbases, Subgrades). From NCHRP Project 1-37A [2002]

Design Type	Input Level	Description	Tests/Estimation Method Required
New	1	Resilient Modulus (M_R)	Use k_1, -k_3 nonlinear coefficients
	1a	Direct lab test	LTPP P-046 and/or NCHRP 1-28A test protocol
	1b	K_i Parameter prediction models	Estimate k_i values from standard material properties
	1c	Typical k_i values by material group	Tabular summary of k_i parameters versus soil classification group
	2	Correlate M_R from empirical/lab	Prediction equations
	2a	Obtain M_R from CBR or R value equations	M_R=155+555(R) psi or M_R= 2550(CBR)$^{0.64}$ psi
	2b	Obtain M_R from PI/gradation properties	See text for recommended equations
	2c	Convert agency layer coefficient (a_i). Experience for base/subbase M_R	M_R=30,000 (a_i/0.14)^3 psi
	3	Soil classification correlations	AASHTO/USCS correlations
	3a	AASHTO classification	Typical M_R range as default for design/optimum conditions
	3b	USCS classification	Typical M_R range as default for design/optimum conditions
Rehabilitation	1	Backcalculate M_R	FWD tests
	1a	Direct FWD measurement	Use unbound material correction factor: $C_f = E_{lab}/E_{fwd}$
	1b	Typical backcalculated M_R values	Tabular summary of M_R backcalculated values by soil group
	2*	Correlate M_R from empirical/lab	Predictive equations
	2a	Estimate M_R from CBR and DCP test	Determine penetration resistance (PR) from DCP test, then use CBR=(292/PR$^{1.12}$), then M_R=2550(CBR)$^{0.64}$ psi
	2b	Obtain M_R from CBR or R value equation	M_R=155+555(R) psi or M_R= 2550(CBR)$^{0.64}$ psi
	2c	Obtain M_R from PI/gradation properties	See text for recommended equations
	2d	Convert agency layer coefficient (a_i). Experience for base/subbase M_R	M_R=30,000 (a_i/0.14)^3 psi
	3*	Soil classification correlations	AASHTO/USCS correlations
	3a	AASHTO classification	Typical M_R range as default for design/optimum conditions
	3b	USCS classification	Typical M_R range as default for design/optimum conditions

(*) For rehabilitation analysis, M_R values for unbound materials will almost exclusively be associated with "moisture equilibrium" conditions. Thus, "design M_R" correlations should be used and not "optimum M_R" correlations.

For new designs, the Level 1 unbound materials modulus input is M_R from the modified model, while the Level 2 inputs are estimated from tabulated "k" values for typical base and subbase materials. For Level 2 subgrade inputs, it is unlikely that enough historical data could be accumulated to enable useful tabular summaries for the wide range of soils encountered. Therefore, there is no recommended Level 2 approach for subgrades. Level 3a new design inputs for all unbound materials will be estimated

empirically from CBR or R-value. For unbound base and subbase materials, Level 3b and 3c inputs are provided to accommodate historical data where tabular modulus values or those estimated from agency layer coefficient experience can be used. Only Level 3b is provided for subgrades, as legacy layer coefficients are not applicable to those materials.

For rehabilitation design, Level 1 unbound layer moduli are again backcalculated from Falling Weight Deflectometer (FWD) tests while Level 2 moduli are estimated from CBR via dynamic cone penetrometer (DCP) tests. The following represents some correlations that have been established by various authorities for CBR estimates of subgrade soils.

$$\text{Shell: } M_R(\text{MPa}) = 10*\text{CBR}, [M_R(\text{psi}) = 1500*\text{CBR}] \qquad (1.1)$$

$$\text{WES: } M_R(\text{MPa}) = 37.3*\text{CBR}^{0.711}, [M_R(\text{psi}) = 1500*\text{CBR}^{0.711}] \qquad (1.2)$$

$$\text{TRRL: } M_R(\text{MPa}) = 17.6*\text{CBR}^{0.64}, [M_R(\text{psi}) = 2550*\text{CBR}^{0.64}] \qquad (1.3)$$

$$\text{DK: } M_R(\text{MPa}) = 10*\text{CBR}^{0.73}, [M_R(\text{psi}) = 1500*\text{CBR}^{0.73}] \qquad (1.4)$$

Where,

WES - Army Corps or Engineers, Waterways Experimental Station

TRRL - Transportation Road and Research Laboratory (UK)

DK - Danish Road Laboratory.

Generally, these relationships were developed for subgrade CBR values (see Figure 1.1). Level 3a, 3b, and 3c inputs are determined as described above for new pavement design. The moduli generated in level 3b and 3c must be entirely compatible with FWD backcalculated values from the literature and from the project rehabilitation evaluation.
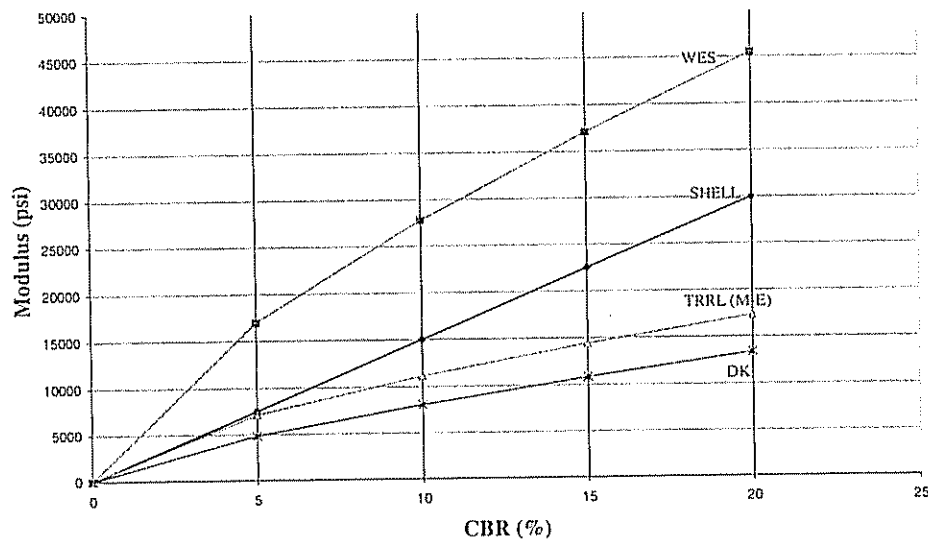
Figure 1.1: CBR-Modulus Relationships

1.3   Objectives

The objectives of this research are:

1. To perform a comprehensive literature review on structural response of granular materials.

2. To perform basic soil tests, CBR and other strength tests on pavement materials from the State of Delaware and to characterize their behavior under traffic loading and environmental conditions.

3. To provide input necessary for modeling the material with the mechanistic pavement design and evaluation by performing computational modeling of CBR response.

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4. To summarize the result and the procedure in a final report format, and to outline further research needs and, if necessary, new possible directions.

1.4   Research Approach

The research approach that was used to attain the objectives involves material testing and analysis. Basic soil tests such as sieve analysis, the Atterberg test, CBR, and standard Proctor tests were conducted on each type of soil. The tests provide the necessary physical parameters for the statistical analysis, and were conducted according to relevant American Society for Testing and Materials (ASTM) and AASHTO standards. The laboratory CBR test measured the shearing resistance of the unbound granular material under controlled moisture and density conditions, which was then used to evaluate the potential strength of subgrade, subbase, and base course material for use in pavement construction.

Essentially, completion of the research involved four major efforts:

1. Collection and reviewing of available literature and publications characteristic of both bound and unbound pavement materials.

2. Obtaining soil samples representative of granular materials commonly used in Delaware for highway construction projects.

3. Conducting tests on identified materials, and data analysis and interpretation.

4. Derivation of models to describe test results to be used for implementation of a mechanistic-empirical design for pavements in the State.

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1.5   Thesis Organization

Chapter 1 gives a brief introduction to pavement material characterization using the resilient modulus concept. It touches on the background and motivation for the work, and also outlines the methodology used in achieving the research objectives.

Chapter 2 of the research summarizes the literature and background information on resilient modulus of pavement construction materials. Application of various techniques used in the characterization of pavement construction materials under different traffic loading and environmental conditions, the effects of influencing factors such as stress level, density, grain size distribution, fines content, number of load applications, and moisture content on the resilient response are discussed.

Chapter 3 contains the experimental design, material selection, laboratory methods used in the research, locations and summary of information on the resilient modulus site selection, and information contained in both AASHTO guide for the design of pavement structures and ASTM standards. Specimen preparation, test apparatus and procedures are also discussed.

Chapter 4 deals with the analysis of the experimental test results, the formulation of computational model to describe the relation between CBR and the basic engineering index properties of the pavement materials used in the experimental program. The process will include:

- Organizing CBR test data by material type

- Conducting regression analysis to define selected statistical parameters

- Performing a correlation analysis on the test data, and

- Summarizing the results from the regression and correlation matrix.

Chapter 5 lists the major conclusions of the study as they apply to the resilient modulus as a valuable input for the mechanistic pavement design. The findings and recommendations, when properly applied, will provide an effective indirect procedure for the determination of resilient modulus that is easy, less-time consuming and more cost-effective. The ability to properly characterize the behavior of pavement construction materials under repeated traffic loading and environmental conditions is very promising.

Chapter 2

BACKGROUND

2.1 Introduction

The AASHTO (1993 and the proposed 2002) *Guide for Design of Pavement Structures* have suggested the use of resilient modulus (M_R) to properly characterize pavement construction materials subjected to repeated traffic loading. The resilient modulus is defined as the ratio of the amplitude of a repeated axial deviator stress to the amplitude of the recoverable strain, and is a way to quantify the deformations experienced by a pavement supported on an elastic subgrade under repetitive traffic loads. The values of resilient modulus of the subgrade, subbase and base materials are used in the structural analysis of pavement systems.

The determination of resilient modulus for base, subbase, and subgrade soils from field and laboratory tests has been a continual source of interest to numerous investigators. The laboratory procedures include:

- Repeated load triaxial tests,
- Resonant column tests,
- Hollow cylinder tests.

Current field procedures include:

- Field plate bearing load test

- Cone penetration test (CPT),

- Falling Weight Deflectometer test

- Seismic Pavement Analyzer (SPA).

Of all the test procedures, the resilient modulus from repeated load triaxial test is frequently used because of the repeatability of test results and proper simulation of field stress conditions in the laboratory environment. A typical triaxial setup for resilient modulus test is as shown in Figure 2.1 below.
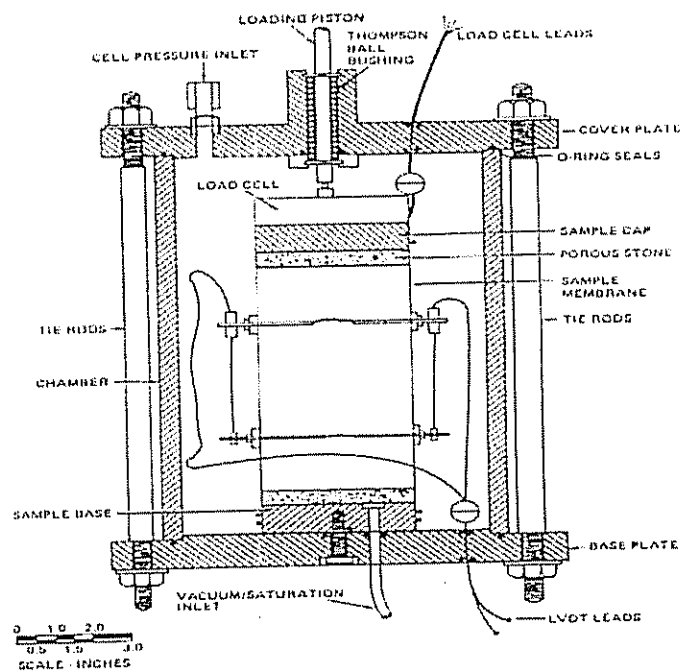
Figure 2.1: Triaxial cell for testing cylindrical specimens
[After FHWA, 1978]

An extensive literature survey has been undertaken to collect findings from previous research. This chapter is organized into two sections. The introductory section presents a summary of the current state of knowledge regarding the determination of M_R of pavement construction materials from various laboratory and in situ techniques. The review covers the following topics:

i)   Resilient Modulus of Subgrade Materials

ii)   Resilient modulus of Granular Base Materials

iii)   Resilient Modulus of Asphaltic Materials

iv)   Resilient Modulus Based Designs

v)   Special Cases.

The information obtained from the literature review is used to develop a better understanding of the behavior as well as various techniques used in the characterization of pavement construction materials under different traffic loadings and environmental conditions. The second section discusses various factors affecting the resilient response.

2.2   Literature Review

2.2.1   Resilient Modulus of Subgrade Materials

The U. S. D. A. Forest Service has been using extensive literature review, detailed regression studies and some laboratory tests to develop models for predicting the resilient modulus on different type of soils. The program was initiated to provide a systematic pavement management tool for the design of Forest Service roads and to serve as

guidance to practicing engineers in the absence of test results. Though the resilient

modulus could be determined from repetitive load triaxial testing of the subgrade soil

samples, such testing did not appear to be readily available to the Forest Service field

engineers. The objective of the overall program was therefore to develop a correlation

for predicting subgrade resilient modulus values from basic soil tests.

In order to achieve the above objectives, the Forest Service undertook an

extensive literature review on the Highway Research Information Service (HRIS)

database where resilient modulus test results were recorded for different soils using basic

parameters such as plasticity index (PI), water content (%w), and amount of material

passing the No. 200 sieve. Regression studies were also made on the individual soil

types according to the unified soil classification (USC) system to establish the various

correlation models. Two broad classes of soils: fine grained (cohesive) and coarse-

grained (granular) soils were created to yield reasonable results and after several trial

combinations a general model equation was developed for each soil class.

To verify the reliability of the models developed, soil samples of a range of

material types were collected at random by the Region 5 laboratory of the Forest Service

and given to researchers for testing using the new AASHTO test method T274-82 for

resilient modulus of subgrade soils. For the limited range of soils tested, the actual

values compared favorably with the predicted values from the two general models

developed. Problems were, however, encountered during the development of the

regression equations such as missing observations, different test procedures, lack of range

in predictor values, collinearity and inconsistent sample sizes. Some deviations were detected in the results but there do not appear to be any major trends in the data that would cause the equations to be rejected. Carmichael and Stuart [1985] however drew the attention of engineers to always use these equations with engineering judgment. It was also recommended that resilient modulus laboratory tests be obtained whenever feasible [Carmichael and Stuart 1985].

The Korean Highway Corporation has undertaken some research work on the characteristics of the resilient moduli of subgrade and subbase soils in order to develop a pavement design guide based on Korean pavement conditions. Due to the difficulties and complexities in the use of the standard testing procedures for determining the resilient modulus (M_R) as adopted by AASHTO, an alternative M_R testing technique has been developed by Kim, Kweon, and Lee [1997] in calibrating the whole testing system using synthetic specimens of known stiffness characteristics. At the onset the characteristics of resilient modulus of the compacted subgrade soils of various plasticity indexes (PIs) using resonance column (RC), torsional shear (TS), and resilient modulus (M_R) tests were determined. In Korea, subgrade soils used in practice are mostly non-plastic, with a PI of less than 5 percent, therefore most of the compacted subgrade soils tested in this study were non-plastic. The authors also discussed the development of a free-free resonant column (FF-RC) test to determine Young's modulus, Poisson's ratio, and shear modulus of soils at small strains below the elastic threshold. There was the need to assess the feasibility of the (FF-RC) test on M_R so as to calibrate the entire M_R testing system and this was done by testing synthetic specimens of known stiffnesses using FF-RC, RC/TS,

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and M_R tests. The advantage of using the synthetic specimens was that they have physical characteristics that remain constant with time and thereby can be tested repeatedly using different testing methods. The effects of strain amplitude and loading frequency on resilient modulus were also investigated by performing both RC and TS tests on the same specimens.

To assess the capability of the alternative procedures in determining the resilient modulus of compacted subgrade soils, several comparisons of M_R values obtained from the proposed method and the direct tests were made. Kim et. al. [1997] discussed the results using graphical illustrations to explain the observations made. It was concluded that moduli obtained from M_R tests marched favorably with M_R values obtained from the FF-RC tests showing that the alternative method can be used to determine M_R values provided that the loading frequency and modulus reduction curve is considered. An added advantage of this method is that resilient modulus could be determined over a wide range of strain amplitudes, whereas M_R tests could not be used to measure a moduli at axial strains smaller than about 0.01 percent [Kim, Kweon, and Lee 1997].

The use of resilient modulus has been accepted widely by highway transportation agencies as an essential tool in the design and analysis of pavement structures. Several laboratory and field nondestructive test methods have been used to determine the resilient modulus of pavement soils and these have been found to be very time consuming, laborious and expensive. The Louisiana Transportation Research Center and the Louisiana Department of Transportation and Development have initiated a program for

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investigating the applicability of the cone penetration test (CPT) in evaluating the resilient characteristics of subgrade soils. This has been necessitated by the fact that the nature of soil deposits in Southern Louisiana have been found to be soft, and as such the use of CPT which is fast and economical has become popular among the present in situ methods.

Two different types of cohesive soils namely, silty clay and heavy clay were selected for the field and laboratory investigations. The penetration tests were conducted with the 2-cm^2 miniature friction cone penetrometer and 15-cm^2 friction cone penetrometer, to obtain continuous measurements of the tip resistance (q_c) as well as the sleeve resistance (fs). Undisturbed soil samples were used for the laboratory resilient modulus testing and a total of twelve soil samples were tested following the AASHTO T-294 procedure. Additionally, the soil samples had to be subjected to different laboratory tests in order to determine their physical properties so as to provide a complete material characterization. Results from both laboratory and field tests have shown that the cone resistance, sleeve friction and resilient modulus are affected by the soil type, unit weight, and moisture content. A statistical model has been developed by Mohammad et al. [1999] for predicting the resilient modulus from the miniature cone tip resistance, sleeve friction, and soil properties. More tests, however, needs to be conducted and consideration be given to a wide range of soil types before proposing a general model based on the cone penetration output [Mohammad, Titi, and Herath 1999].

Ping and Ge [1997] have described efforts by the Florida Department of Transportation (FDOT) to correlate the laboratory resilient modulus (M_R) measurements with field performance data. It could be noted that most recent research lack information on the correlation between the laboratory resilient modulus and the field pavement performance which makes pavement designers feel some sense of uneasiness in using the M_R test in pavement analysis and design. The experimental program was initiated to evaluate the field bearing characteristics of pavement layers on selected types of subgrade soils using the plate bearing load test. After each field test was completed, the reconstituted soil samples simulating the field moisture and density conditions were taken to the laboratory for M_R measurements. Only granular subgrade soils were utilized in this study since they were commonly encountered as roadbed soils in Florida.

The factors that influence the M_R values comprise the bulk stress, confining pressure, and the deviator stress. The effects of these factors on the M_R were evaluated and it was shown that the M_R generally increases as the confining pressure or the bulk stress increases. It is also generally believed that the M_R increases slightly as the deviator stresses increases for granular materials. This is true for confining pressures below 68.95 kPa as indicated from the test results but, however, M_R decreases slightly as the deviator stresses increases under higher confining pressures.

Ping and Ge [1997] also highlighted the difference between the M_R determined from the middle (10.2cm) and full-length (20.3cm) linear variable differential transformers (LVDTs) during the laboratory analysis. In general, the M_R obtained from

the 20.3cm LVDT measurements showed lower values as compared with the 10.2cm LVDT measurements. These differences came about as a result of the presence of air gaps between the specimen and accessories such as porous stones and platens, and errors such as sample alignment and bedding problems in the laboratory set up. Comparing the computed laboratory equivalent M_R values with the field layer modulus for the subgrade soils showed reasonable agreements using the simplified computational procedure. The laboratory M_R values increase with increasing field plate layer moduli for the subgrade soils under identical state of stresses. A procedure has therefore been adopted to compute the equivalent resilient modulus of the subgrade layer from the laboratory resilient modulus test for comparison with the modulus of elasticity of the subgrade layer from the field plate bearing load test [Ping and Ge 1997].

Several studies have been conducted over the past three decades towards the development of engineering models to characterize the resilient behavior of cohesive subgrade soils. In most of these models, it has almost always been necessary to run the repeated triaxial loading test in order to determine the coefficients of the model that depend on soil type. Muhanna, Rahman and Lambe [1998] have described a statistical model for predicting the M_R and the accumulated plastic strain of compacted cohesive subgrade soils using the results of the unconsolidated undrained compression test and the standard compaction test. The testing system comprise of an MTS closed-loop loading system placed in a triaxial cell and a personal computer to collect the data by using an AT-MIO-16 board. Two natural North Carolina soils supplied by the Department of Transportation were tested and used to investigate the performance of cohesive soils

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under repeated loading. These two soils were classified as A-6 and A-5 according to the AASHTO system. Fifteen A-6 soil specimens were prepared at three different moisture contents (± 2.5 percent) and compacted in a mold according to the specifications of the standard proctor test (T-99). Eight A-5 cohesive soil specimens were also prepared on the other hand and compacted at three different moisture contents (± 2.5 percent) and tested using the Strategic Highway Research Program load sequence. The results of both tests were illustrated in diagrams. The findings show that the resilient strains for all three molding water contents under different stress levels vary little in magnitude with the number of load repetitions. Again, for any moisture content, the higher the stress level the higher the resilient strain.

After studying the behavior of the A-6 cohesive subgrade soil in accumulating permanent strain, a model was developed using the effects of variables like stress level and moisture content, and this was found to be useful for pavement analysis and design purposes. The resilient strain of all the specimens tested were also monitored to establish the resilient behavior of the compacted cohesive subgrade loads under repeated loads. Muhanna et. al. [1998] found out that at constant compaction water content of the A-6 cohesive soil, the resilient strain correlates well with the plastic strain. A model has therefore been developed using the resilient strain with respect to the normalized water content which is linearly related to the log of the accumulated plastic strain at the state of apparent shake down. The model thus developed in this literature has the potential of providing a reasonable estimation of the M_R of subgrade soils. Additionally, it could also

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be used to predict the expected accumulated plastic strain for the A-6 soil [Muhanna, et. al. 1998].

Environmental factors such as temperature, water content, and freeze-thaw effects are known to have considerable influence on the performance of flexible pavements. The 1993 AASHTO guide recommends the use of an effective roadbed soil resilient modulus to cater for seasonal changes in the measurement of resilient modulus. Because of the difficulty in quantifying the seasonal variation in resilient modulus, the direct use of the effective roadbed soil resilient modulus has been limited. To reduce the number of required laboratory resilient modulus tests needed for design, it has been suggested that an equivalent season, or moisture be determined, for which the corresponding resilient modulus reflects the equivalent relative damage induced over the whole 12-month year [Guan, Drumm and Jackson 1998]. Many studies have shown that increase in water content generally occurs during one or two months of the year. A procedure for determining the design season condition for subgrade soils under asphaltic concrete pavements has been proposed by Guan et al. [1998]. The proposed procedure is based on a weighting factor (WF_i) that was defined by Gomez-Achecar, to quantify the effect of seasonal changes in pavement temperature. The weighting factor was used to estimate the mean annual air temperature, which represents the location effect and was found to be dependent on the thickness of asphalt concrete layer. The seasonal value of resilient modulus of subgrade soil for the ith month of the year (M_{Ri}), could then be determined by laboratory tests on field samples obtained throughout the year, or from tests on samples compacted in the laboratory to simulate field conditions and subsequently subjecting

them to changes in water content. On the other hand, approximate methods may be used to estimate the change in resilient modulus caused by postcompaction water content changes. The 1993 AASHTO guide also indicates that M_{Ri} can be backcalculated from nondestructive testing (NDT) data.

Guan et al. [1998] presented three examples for the calculation of the monthly weighting factor. These were based on (a) Seasonal Subgrade Water Contents in Oklahoma, (b) Dynaflect tests and, (c) the AASHTO guide. It was further demonstrated from (a) above, on how the design season resilient modulus could be determined. Because the variation in resilient modulus with water content is unknown, the approximate method, as cited earlier could be used in estimating the monthly resilient modulus. This example, however, used an assumed resilient modulus of 106 MPa at an optimum water content of 12.6 percent for the month of April to establish the monthly resilient modulus values and the calculated weighting factors for the site from July 1966 to May 1967 as illustrated in a table. Subsequently, the design resilient modulus corresponding to a weighting factor of 1.0 was then determined. It can therefore be suggested that a pavement design based on this design season could be assumed to reflect the seasonal variations in subgrade modulus and the corresponding relative damage that the pavement would sustain over all seasons of the year [Guan, Drumm, and Jackson 1998].

An experimental program has been initiated by the Florida State Department of Transportation (FDOT) to evaluate the resilient modulus (M_R) of granular materials using

the repetitive rigid plate load test in a test-pit facility, and the laboratory resilient modulus test [Ping and Yang 1998]. The test-pit facility simulates the subgrade and base components of a flexible pavement system and permits full-scale testing of base/subbase sections constructed on a standard subgrade and under different controlled moisture conditions. AASHTO T292-91I resilient modulus test method was used for the laboratory test. Five typical subgrade materials of Florida were used for the study under the same density and moisture conditions (three moisture conditions: optimum, drained and dried, and soaked). The resilient deformations measured in the test pit experiment were then verified by comparisons with the resilient deformations determined from the laboratory resilient modulus tests. The resilient moduli obtained from the plate tests on the subgrade were based on Boussinesq's theory of deflections at the center of a circular plate.

Ping and Yang [1998] observed from the test results that the resilient modulus remained constant or increased slightly with an increase in deviator stress while the confining pressure was kept constant, and this was attributed to the material being silty or clayey sand. The effect of moisture condition on the M_R was also demonstrated. For all the five subgrade soils, an increase in moisture content had a detrimental effect on the resilient modulus, as observed with the test results obtained from the test pit test. Since the M_R values obtained from the triaxial test were stress dependent, whereas the resilient modulus resulting from the test-pit rigid plate test is an equivalent resilient modulus of the whole layer of the testing material, it means that both tests were under different states of stress and cannot be compared directly. To do the comparison, the resilient

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deformation of the testing material in the test-pit test is calculated using the triaxial M_R, and then the calculated deformation is compared with the measured deformation from the test-pit test [Ping and Yang 1998].

Several factors have been found to influence the resilient modulus of fine-grained subgrade soils. Muhanna, Rahman and Lambe [1999] have investigated the effects of specimen end condition, repeated load sequence, rest period, number of load applications, and specimen conditioning have been undertaken as some of the variables. The effects of impact and kneading compaction methods on measured resilient modulus of fine-grained subgrade soils have also been discussed [Muhanna et. al. 1999]. Two types of fine-grained soils, classified as A-6 and A-5 according to the AASHTO system, were supplied by the North Carolina Department of Transportation for the studies. The moisture content, end condition, load sequence, and rest period were each tested at three different levels. The moisture content was either optimum or ±2.5 percent of optimum. End condition was either regular, sloped, or grouted with hydrostone paste as suggested by Pezo et al. [1992]. The load sequence was either increasing, decreasing, or constant at 69 kPa. Finally, the rest period was set to be either short, long, or standard at 0.9s.

The axial deformation in the soil specimen during the repeated load triaxial test was measured using two linear variable differential transformers (LVDT). AASHTO T-274-91I recommends the use of two LVDTs, internally clamped to the middle half of the specimen, while the Strategic Highway Research Program (SHRP) Protocol 46 requires

the use of two externally mounted LVDTs on the vertical load piston. Pezo et al. [1992] have also introduced the use of an LVDT setup which works for specimens whose ends have been grouted to the base and top caps. The M_R values obtained from the above setups were evaluated and compared. Muhanna et al. [1999] argued that the LVDT clamps associated with grouting the ends of the soil specimens registered more consistent results than those clamped to the middle of the specimen as illustrated in the various diagrams. The effect of confining pressure on the M_R of the A-6 cohesive soil was investigated using the impact and kneading compaction methods. The results showed that at any deviator stress the coefficient of variation (COV) varied from 1.5 to 6 percent for the compacted specimen of the A-6 soil. From the results obtained, Muhanna et al. [1999] concluded that the number of load applications, the rest period, and the load sequence do not have any significant effect on the measured M_R. The effect of moisture conditioning was also observed to register low M_R values especially for the specimens that were soaked above the optimum moisture content [Muhanna et al. 1999].

Empirical design procedures for flexible pavement structures use static-strength parameters such as California bearing ratio (CBR), soil support value, and Texas triaxial values. These, however, fail to reflect the dynamic nature of traffic loads. The 1993 AASHTO *Guide for the Design of Pavement Structures* has therefore recommended the use of resilient modulus, a dynamic strength parameter, to characterize flexible pavement materials. Attempts have been made in Louisiana to develop a resilient modulus (M_R) model that can reflect realistic soil behavior and to correlate basic soil properties to that model [Mohammad, Huang, Puppaala, and Allen 1999]. Eight different types of soils

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commonly found in Louisiana were selected to validate the model and to calibrate the model's constants with two basic soil properties, CBR and Unconfined compressive strength (UCS). Basic soil tests such as sieve analysis, the Atterberg limit test, and standard and modified Proctor tests were conducted on each type of soil. Resilient modulus tests were conducted in a triaxial cell using an MTS model 810 system that provides repeated axial load. The AASHTO design procedure recommends the use of bulk and deviatoric stress models to describe granular soils (sands) and cohesive soils (clays), respectively. The equation for the bulk stress model is $M_R = a\theta^b$,

where, θ is the bulk stress i.e., sum of principal stresses, and a and b are model constants. The equation for the deviatoric stress model is $M_R = c(\sigma_1-\sigma_2)^{d_1}$

where, the deviatoric stress, $(\sigma_1-\sigma_2)$, is defined as the difference between the major and minor principal stresses, and c and d_1 are the model constants.

The test results were analyzed using multiple linear regression. Regression analysis on the model constants produced R^2 values ranging from 0.35 to 0.73 in the case of the CBR correlations and 0.28 to 0.73 in the case of the USC correlations. The next step was correlating the model constants with basic soil properties and providing the correlations in the form of empirical relations as functions of density, moisture content, degree of compaction, Atterberg limits, and other soil properties. Three types of correlation were produced: (a) model parameter with soil properties, (b) model parameters with CBR, and (c) model parameters with UCS. The model constant correlations with CBR/USC exhibited lower R^2 values, which may be due to the wide

range of CBR and USC used in developing the correlations. Mohammad et al. [1999] therefore recommended that basic soil properties be used to estimate the resilient modulus. Among the correlations recommended, it appears that the model constants for resilient modulus were mainly governed by moisture content, liquid limit, and plastic limit. The approach presented by Mohammad et al. [1999] has been found to be an improvement over the existing approaches since it accounted for stress effects and also for moisture content, liquid limit, and plastic limit [Mohammad et al. 1999].

Jin, Lee, and Kovacs [1994] have described attempts to evaluate the seasonal variation of the resilient modulus (M_R) of granular soils for flexible-pavement design. Laboratory testing was conducted on reconstituted soil specimens under monitored ranges of temperatures, moisture contents, dry densities, and stress conditions. Two field sites were selected in Rhode Island, and both sites were instrumented with Soiltest MC-310 A soil moisture temperature cells, to measure the in-situ soil moisture content and the in-situ soil temperatures at different depths, as well as the freezing and/or frost depth. The results of the M_R test were given as a function of bulk stress and were plotted on a log-log scale. It was observed from the results that the M_R values decreases as the water content increases up to a certain bulk stress; thereafter, it varied regardless of the water content. The results also suggested the necessity to carefully select the bulk stress so that field conditions are represented as closely as possible to obtain the representative resilient modulus.

Jin et al. [1994] performed multiple regression analysis with the laboratory data in order to predict the M_R under various environmental conditions. Regression equations were therefore developed relating the M_R to four other variables including the bulk stress, percent water content, temperature, and dry density. To identify the seasonal variation of M_R, an effective resilient modulus equivalent to the combined effect of the seasonal moduli values was calculated based on the monthly M_R at the average depth of significant stress (ADSS). This effective M_R reflects the overall capacity of subgrade soils to support the pavement during the year considering its seasonal variation. Jin et al. [1994] also presented a theoretical model that was developed to predict M_R at different environmental conditions by estimating the change of M_R due to the change in temperature and soil suction. A micromechanical approach was used for modeling the granular soil behavior at different temperatures and moisture conditions. In the temperature submodel, the soil particles were considered to be confined in all directions, and that any temperature changes causes a variation of contact pressures between particles. The moisture submodel treats the soil particles as a two-phase system in a solid phase and an air-water phase surrounding it. A summary of the results of the M_R test indicated that the resilient modulus values increases as the moisture content and temperature decreases, and the dry density increases. Comparison of the predicted M_R values obtained by the theoretical model with the laboratory measured ones well found to fit well indicating that the theoretical model can be used to predict the resilient modulus [Jin et al. 1994].

Distresses associated with traffic loading on asphaltic pavements such as cracking, are a result of excessive plastic and repeated elastic deflections. The AASHTO guide for the design of flexible pavements (AASHTO 1993), have suggested the resilient modulus (M_R) test to characterize roadbed soils. Most highway agencies, however, have limited experience with the M_R test because it is complex, expensive and time consuming. This has necessitated the development of empirical correlations for resilient modulus by a number of researchers. The results of a study to develop correlations between resilient modulus and the conventional unconfined compression test has been presented by Lee, Bohra, Altschaeffl, and White [1997]. Extensive research has shown that M_R of cohesive soils is affected by several factors, some of which are briefly outlined below [Lee et al. 1997].

- Keeping all other factors constant, M_R is stress dependant. A simple expression relating M_R to the maximum axial deviator stress (σ_d) is $M_R = k_1(\sigma_d)^{k2}$

where, k_1 and k_2 = regression constants (constants determined experimentally)

- Method of compaction have been found to affect M_R. In general, samples compacted statistically show higher resilient moduli compared to those created by kneading compaction.

- Compaction parameters--moisture content and dry unit weight--control the modulus values.

- Tested specimens compacted at a high degree of saturation exhibited a significant increase in strength when allowed to rest before the test. The conclusions were that the effect of thixotropy on resilient strains varies with the number of stress applications, but a

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marked reduction in total deformation due to thixotropic strength occurred. The effect of thixotropy is more significant for samples compacted on the wet side of optimum than those compacted on the dry side of optimum.

● A linear relationship exists between the M_R and soil moisture suction. The M_R has been proposed to be a function of three stress variables: the net confining stress (σ_3-u_a), the axial stress (σ_1-σ_3), and the matrix suction (u_a-u_w).

Where, u_a = pore air pressure, and

        u_w = pore water pressure.

Three clayey subgrade soils from Indiana were used for the test. The laboratory specimens were compacted in a 73 mm dia., 160 mm high mold to provide a sample with height-diameter-ratio of at least 2. After performing the necessary sample preparations, they were stored in a humidity room for two days. A rubber membrane was then applied after which the conventional unconfined compression test was performed using a strain rate of 1%/min [0.025 mm/s (0.001 in/s)]. The test was terminated at 1.5 mm (0.06 in) axial deformation (about 1% axial strain) and subsequently the M_R test was conducted on the same specimen according to the procedure described in AASHTO T274-82 test method. The M_R test has a conditioning stage, therefore performing the conventional unconfined compression (UC) test is believed to have negligible effect on the M_R results. Lee et al. [1997] illustrated the test results showing the relationship between stress causing 1% strain ($S_{u\ 1.0\%}$) during the UC test, and molding moisture content. It was shown that compaction on the wet side of optimum gave a low value of M_R regardless of

the compactive effort. Due to postcompaction moisture changes and other variations in the field, the M_R estimated from the laboratory-compacted specimen at the same moisture content and dry unit weight may vary significantly from the field M_R, as such a correlation based on variables other than moisture content and dry unit weight was desirable. Regression analysis was therefore conducted to obtain a relationship between M_R and $S_{u\ 1.0\%}$ for laboratory compacted soil. The coefficient of determination (R^2) for the relationship was 0.97. To compare the M_R and $S_{u\ 1.0\%}$ relationship between field and laboratory compacted soils, resilient modulus test was performed on undisturbed samples from the Washington site. The results obtained showed that the relationship between M_R and $S_{u\ 1.0\%}$ for the field compacted soils fits very well, and among the data for the laboratory-compacted soils. Lee et al. [1997] suggested that a similar correlation could be developed and used for predicting M_R for both laboratory and field-compacted conditions by using different types of clayey soils [Lee et al. 1997].

2.2.2 Resilient Modulus of Granular Base Materials

Resilient moduli of base and subgrade materials are important tools in the design of pavement systems. In recent years, AASHTO has adopted three methodologies for resilient modulus (M_R) testing of granular materials. The most current procedure (AASHTO T294-92), which basically reflects the procedure suggested by the Strategic Highway Research Program (SHRP) has been found to be more convenient even though some aspects of the of the testing methodology are still being investigated and modified. A testing procedure has been developed by Nazarian, Pezo, Melarkode, and Picornell [1996] for the Texas Department of Transportation. The main focus of the study was to

(a) critically assess the strengths and weaknesses of the AASHTO procedure as it relates to base materials and, (b) to develop a protocol that would avoid the possible weaknesses of that protocol.

Although the AASHTO protocol has been found to be reasonable, several modifications have been suggested relating to (a) the loading sequence, (b) specimen conditioning, (c) the measurement of the axial deformation, and (d) the determination of Poisson's ratio. On account of the above, Nazarian et al. [1996] made several changes which have been incorporated into the proposed testing procedure as follows:

- The loading sequence was altered to avoid high deviatoric stresses at low confining pressures.
- Deformations were measured internally with non-contact probes.
- Grouting of the specimen was incorporated and the conditioning sequence was removed.
- The axial deformations were measured along the middle one-third of the specimen to minimize errors due to shear stresses developed because of grouting.
- Lateral deformations were measured at the midheight of the specimen with non-contact probes, so that Poisson's ratio could be determined

Nine synthetic specimens and nine actual base materials were tested by the procedure suggested earlier, and some of the results obtained have been discussed for clarity. When the deformations were measured along the length of the specimen, Poisson's ratios were more than 0.5 which is impossible for the synthetic materials used in the study. The

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errors in the Poisson's ratio could be attributed to the development of shear stresses in the vicinity of the end platens due to grouting. It was also observed that the resilient modulus based on axial deformations measured over the full length of the specimen were higher than those obtained from the measurements along the middle one-third for all the specimens. Nazarian et al. [1996] concluded by suggesting some improvements to the AASHTO T294-92 procedure for M_R testing of base materials which includes;

- Using non-contact probes to measure deformations more accurately, and more conveniently.

- Grouting the specimen to the top and bottom platens, which may yield more repeatable results instead of using load conditioning.

- A proposal that may minimize the specimen disturbance.

- Measuring the axial deformations at the middle one-third of the specimen if the specimen is grouted instead of the overall length of the specimen [Nazarian et al. 1996].

Khedr [1985] has described attempts to investigate the deformation mechanism of untreated granular material in an experimental program that involved applying stress conditions that simulates those expected in flexible pavements. A crushed limestone aggregate obtained from Franklin County, Ohio, with a maximum grain size of 20mm and limestone fines, were subjected to a dynamic testing program. The tests were carried out in a triaxial cell and it involved applying simultaneous time-variable confining and deviator stresses on the aggregate samples. A linear statistical regression of the results was obtained for the residual deformation. In general, Khedr [1985] observed that

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permanent deformation increase with increasing deviator stress and decreasing confining pressure, resilient modulus, water content, static initial modulus, angle of internal friction, and relative density within the ranges considered in the study. Khedr [1985] noted that the resilient Poisson's ratio had no apparent effect on permanent deformation. Similar trends were also observed for samples tested under zero deviator stress, defined by Khedr [1985] as dynamic consolidation, except that higher dynamic confining pressure resulted in more sample consolidation.

Typical results of the resilient modulus versus number of loading cycles (N) as well as the correlation array of the resilient modulus versus different variables were illustrated. The modulus was most sensitive to changes in stress state. It increased with dynamic deviator stress as well as dynamic confining pressure. In some samples, however, the modulus decreased with increasing deviator stress for low values of deviator stress (<10psi). At high stress levels, the modulus reached a stable maximum value. Khedr [1985] discussed the effect of static stresses on which dynamic stresses were superimposed. The static stresses were necessary to assure proper sample seating and conditioning in the vertical direction and to avoid negative confining pressure from occurring during the test. It was found that changing static confining pressure within 3 psi did not affect the resilient modulus significantly. On the other hand, the deviator static stress had significant effect on the modulus. Also, doubling the value of the static deviator stress from 3 to 6 psi could increase the resilient modulus by 50 percent or more. In conclusion, the resilient modulus of the granular material was found to be sensitive to seating static vertical stress. Khedr [1985] suggested that this should be considered when

standardizing the dynamic testing process for granular materials, by assigning a standard seating static stress in order to assure reproducible and comparable test results. The rate of permanent rate accumulation decreases logarithmically with the number of load repetitions with excellent correlation. The resilient modulus of granular materials was found to be most sensitive to stress state. Further studies, however, need to be undertaken in order to validate the applicability of these conclusions to other types and conditions of base course materials [Khedr 1985].

Nazarian, Rojas, Pezo, Yuan, et al. conducted a study to compare modulus values obtained nondestructively with those obtained from laboratory specimens. The resilient modulus test is the major laboratory test. The state-of-the-art in field testing consist of the Falling Weight Deflectometer (FWD) or the Seismic Pavement Analyzer (SPA) tests. Several investigations have been conducted over the past years in an attempt to develop relationships between laboratory and field moduli. Moduli obtained from SPA and laboratory tests have never been compared in the literature, since the SPA has just been developed, and that most comparisons are typically made between FWD and laboratory moduli, where the main focus has been on the subgrade and not on the base material [Nazarian et al. 1998]. The only relatively comprehensive comparisons between seismic moduli and those from FWD can be found in Nazarian et al. [1987].

Ten sites in ten different districts in the State of Texas were tested. A rational approach was devised to compare moduli from different methods. From the laboratory resilient modulus test, a constitutive model was obtained. In order to compare the results

from SPA and the laboratory tests, the average moduli obtained by the SPA were input into the layered-elastic computer program KENLAYER, where the vertical stresses within the base layer at 25-mm intervals directly under a wheel were determined. The same approach was followed using the FWD moduli. To obtain moduli from constitutive models, representative deviatoric stresses and confining pressures under a standard 44-kN dual tandem load were determined using the KENLAYER program as discussed by Nazarian et al. [1998]. After comparing results from the two field tests as well as the laboratory moduli, it was noticed that large variabilities in the moduli of the base layers were observed. These variabilities were most probably site-related since large coefficients of variation were measured independently by both NDT met6hods. It was also shown that a unique relationship between moduli from laboratory and field tests could not be developed. Nazarian et al. [1998] attributed this to sampling disturbance, nonrepresentative specimens, and long-term time effects. However, both SPA and FWD provided similar trends [Nazarian et al. 1998].

The use of a newly acquired, triaxial testing equipment, referred to as University of Illinois FastCell (UI-FC), for the laboratory determination of anisotropic resilient properties of granular materials had been described by Tutumluer and Seyhan [1999]. Tutumluer and Seyhan [1999] observed that the importance of anisotropic aggregate behavior is currently not considered in material characterization for pavement design because of suitable laboratory equipment and testing capabilities. Resent studies have shown that, unlike the commonly used assumption of isotropy, granular bases and subbases have anisotropic moduli. In the study, critical pavement responses predicted for

a conventional flexible pavement having anisotropic granular base stifffnesses were typically higher than those obtained by the elastic-layered design procedures, which assume homogeneous, isotropic aggregate behavior. Tutumluer and Seyhan [1999] cited a special type of anisotropy, known as cross-anisotropy, which is commonly observed in pavement geomaterials as a result of stratification, compaction, and applied wheel load in the vertical direction. Resilient modulus testing was undertaken on four different aggregates with varying material types and properties using the UI-FC. Two of the aggregates tested were different gradations of a crushed dolomite classified as CA-6 and CA-11 by the Illinois Department of Transportation. The third aggregate CL-3sp, was a class 3 nonplastic uncrushed sandy gravel used in the Minnesota Road Research Project, and the fourth was pea gravel. Repeated load triaxial tests were initially conducted on a synthetic calibration specimen 150 mm in diameter by 150 mm high that was made of a polyester-based urethane elastomer to verify the modulus definition and thereby validate the vertical and horizontal pulsing testing approach employed in this study. Two samples of each aggregate type were tested following two test procedures, A and B, depending on the order of pulsing in each direction. This was done in order to study the directional dependency of granular material stiffnesses in light of previous loading history.

Results of the variations in the vertical and horizontal resilient moduli with increasing deviator stresses for the four granular materials were illustrated in diagrams. In general, Tutumluer and Seyhan [1999] showed that the anisotropic resilient moduli for each granular material increased as the applied hydrostatic stresses increased. The crushed aggregates, CA-6 and CA-11, were found to display higher vertical moduli than

both the pea gravel and the sandy gravel, CL-3sp. Tutumluer and Seyhan [1999] also highlighted the effects of different test procedures on anisotropic moduli for each aggregate type. A suggestion was made to undertake more anisotropic testing of this nature in future studies to investigate the effects of conditioning on the directional dependency of aggregate moduli. The UI-FC device as mentioned has somewhat shown to be ideally suited for simulating dynamic stresses on soil samples and for studying the effects of anisotropic, stress-path-dependent granular material behavior [Tutumluer and Seyhan 1999].

Some studies have reported experimental results in which the main focus has been on the M_R of granular materials having different aggregate sources, different testing procedures, or use different numerical models to simulate the pavement performance such as cracking and rutting. However, most of the test specimens reported previously were prepared with certain gradations or a given moisture content and were tested using different testing procedures which yielded different M_R values and hence differences in pavement design. Tian, Zaman, and Laguros [1998] have evaluated the M_R values due to three different gradations and three different moisture contents for the Richard Spur (RS) aggregate (limestone) and the Sawyer aggregates (sandstone), which are commonly used in Oklahoma as the subbase or base materials of roadway pavements. The Los Angeles abrasion values for both aggregates were determined as 24 percent and 28 percent respectively, which indicates that these two aggregates are of good quality. Three gradations, the median, the finer limit, and the coarser limit, were selected from the Oklahoma Standard Specifications for Highway Construction to investigate the effect of

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gradation on M_R values. To study the effect of moisture content on M_R values, three moisture contents--optimum moisture content (OMC), 2 percent below OMC, and 2 percent above OMC--were selected for the median gradation. In addition, the index properties including the maximum dry density (MDD), OMC, cohesion (C), friction angle (ϕ), and unconfined compressive strength (UC) for the two aggregates were evaluated. Three gradations of the RS and the Sawyer aggregates were tested in the laboratory at the drained condition according to AASHTO T294-94. The mean M_R values of the two aggregates were calculated and the results were presented in terms of bulk stress. It was generally shown that the M_R values for the two aggregates increased with increasing bulk stress. The median gradation of the RS aggregates produced substantially higher M_R values whereas the coarser limit gradation of the Sawyer aggregate produced the highest M_R values on the other hand. The effect of moisture content on M_R values for the median gradation for both aggregates were also presented in terms of bulk stress, and the results show that an increase in moisture content leads to a decrease in M_R values.

Tian et al. [1998] found out that the statistical correlations between M_R and engineering index properties are useful in practice because the basic index properties are easy and inexpensive to evaluate. A multiple linear regression model was therefore developed in this study [Tian et al. 1998] to predict the M_R values using the cohesion (C), friction angle (ϕ), bulk stress (θ), deviator stress (σ_d), unconfined compressive strength (UC) and moisture content (MC). Comparisons of the experimental observations and the

model predictions were illustrated in diagrams. The multiple regression model produced has a wide range of applications because the M_R values of the two selected aggregates at different gradations and moisture contents can be predicted by using the multiple regression model, hence it has some significance in the practical pavement design [Tian et al. 1998].

The Ohio Department of Transportation (ODOT) had initiated a project to investigate several aggregates for permeability and resilient modulus (M_R) in order to specify the gradational characteristics of dense-graded and open-graded aggregates that are unstabilized or stabilized with asphalt or portland cement [Heydinger, Xie, Randolph, and Gupta 1996]. The results of analysis of laboratory resilient modulus testing conducted on both dense- and open-graded aggregates have been presented Heydinger et al. [1996]. The testing program consisted of three different aggregate materials (crushed limestone, natural stone, and slag), five different gradation specifications, and three different moisture conditions. A triaxial pressure chamber and a closed-loop electrohydraulic testing system was used for the M_R tests which was as close as possible to the Strategic Highway Research Program Protocol P-46 (AASHTO T 294-92 I).

Various studies have proposed a two-parameter equation to characterize the resilient modulus of untreated aggregate materials in terms of the bulk stress,

i.e., $M_R = k_1(\theta)^{k2}$ (MPa)   (2.1)

Where, θ = bulk stress, the sum of principal stresses, $\sigma_1 + \sigma_2 + \sigma_3$;

$$\sigma_1 = \text{maximum vertical stresses}$$

$$\sigma_2, \sigma_3 = \text{confining stress, and}$$

$$k_1, k_2 = \text{regression constants.}$$

Heydinger et al. [1996] used a three-parameter equation to characterize M_R in terms of the bulk stress and the octahedral shear stress.

$$M_R = k_3(\theta)^{k4}(\tau_0)^{k5} \qquad \text{(MPa)} \tag{2.2}$$

Where, τ_0 = octahedral shear stress, and

k_3, k_4, k_5 = regression constants.

The results were analyzed using Equations 2.1 and 2.2. Log-linear regression analysis was therefore performed for both the two- and three-parameter expressions using a computer spreadsheet software. On the basis of the results shown, Heydinger et al. [1996] observed that the gravel aggregate consistently had the highest M_R, followed by limestone and then slag. For the limestone aggregate, the open-graded specifications had higher moduli than the dense-graded specifications. The two limestone specimens tested at a saturated condition registered lower moduli, particularly at low stress levels, which illustrates the importance of designs that are effective in preventing saturation in the base and subbase layers. In general, the resilient modulus of aggregates could be said to be dependent on the moisture condition, material type and gradation. Apart from sources of errors which were not highlighted in the investigation, the results of the program could serve as a guide in selecting resilient modulus values [Heydinger et al. 1996].

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The accuracy and manner in which material properties are evaluated and used in analysis have significant influence on the quality of pavement design. Six most commonly encountered aggregate materials used as subbases/bases in Oklahoma have been selected and tested under dynamic loading to evaluate the resilient modulus (M_R) by using AASHTO designation T292-91I [Zaman, Chen and Laguros 1994]. In addition, the effects on M_R values due to variation of testing procedures, specimen size, compaction method, density, aggregate type, and gradation were investigated and finally, statistical correlations were established between M_R and California bearing ratio (CBR), and between M_R and cohesion and friction angle [Zaman et al. 1994]. The six aggregate types used consisted of three limestones, one sandstone, one granite, and one rhyolite. The engineering properties such as liquid limit (LL), plasticity index (PI), maximum dry density (MDD), optimum moisture content (OMC), specific gravity (SG), cohesion, friction angle, and CBR, were evaluated. The most commonly used specimen sizes for M_R testing 10.16 cm (4 in) and 15.24 cm (6 in) diameter samples. From the test data given, it was observed that for a given bulk stress level, the M_R for 15.24 cm specimens were lower than those for 10.16cm specimens. By using the vibratory compaction method, the 10.16cm specimens always had lower dry densities compared to the 15.24cm specimens at the same level of moisture content. Comparisons were also made on the gradations, aggregate types, and testing procedures.

Following the repeated triaxial testing, Zaman et al. [1994] performed the static triaxial tests to obtain the cohesion and friction angle of the material, which served as a database to establish correlations with M_R values. The possible correlations of CBR,

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cohesion and friction angle with M_R were investigated and illustrated. It was clear that the relationship between M_R and CBR suggested by the AASHTO guide is material dependent and was not applicable to the materials used in the study. A linear model relating cohesion C, and friction angle ϕ with the M_R in terms of the major principal stress σ_1 and bulk stress θ, was formulated. In conclusion, the T294-92I testing procedure gave higher resilient modulus values than those obtained by using T292-91I testing procedure, possibly because the cyclic stress had a stiffening and strengthening effect on the specimen structure as the stress level increased from low to high. Zaman et al. [1994] observed that the gradation influenced the density of the specimens; however, its influence on M_R values was less significant compared to the effects of the stress state. In all cases the M_R values for 10.16 cm specimens were higher than those for the 15.24 cm specimens. The regression analysis demonstrated that it is possible to reliably determine the M_R of construction materials through indirect methods that are easy and inexpensive [Zaman et al. 1994].

Zaman, Zhu and Laguros [1999] have evaluated the ability of chemically stabilized marginal aggregates to resist the deleterious effects of freezing/thawing and wetting/drying cycles in order to determine the durability of the stabilized aggregates. The aggregates used in the study was a crushed Meridian limestone from Marshall County, Oklahoma, meeting the gradation requirements of the Oklahoma Department of Transportation (ODOT). The aggregates were stabilized with 15% cement-kiln dust (CKD) and cured for 7 days. The M_R tests were conducted in accordance with AASHTO

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T294-94 with the procedure specified for Type I (unbound granular) material. The number of freezing/thawing cycles considered in the study was 0, 4, 8, and 12. The mean M_R values at various stresses for four types of specimens were determined. From the results obtained Zaman et al. [1999] observed that M_R decreased markedly when specimens were subjected to four cycles of freezing/thawing but that there was no significant further reduction in M_R after more than eight cycles. This gave the impression that the deleterious effects due to freeze/thaw cycles were more dominant at the initial thawing stage, indicating that roadway pavement would behave poorly at the onset of thaw season.

As in the case of freeze/thaw, the number of wetting/drying cycles considered was also the same with 0 representing the regularly cured specimens. A comparison of M_R values of stabilized specimens subjected to different wetting/drying cycles revealed that the absolute difference in M_R values between the regularly cured specimens and specimens subjected to wetting/drying cycles increased with increasing M_R values. In general, the M_R values of raw Meridian aggregate within the tested stress levels were found to be low and as such may be inadequate for most applications. The data demonstrated that CKD is an effective stabilizing agent for strengthening Meridian aggregate intended for base/subbase construction. Zaman et al. [1999] concluded that freezing/thawing and wetting/drying cycles had considerable adverse effect on durability as measured by the resilient modulus of CKD-stabilized aggregate. There is therefore the need to explore more in depth the durability studies and recommend a protocol for freeze/thaw and wet/dry testing methods [Zaman et al. 1999].

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2.2.3 Resilient Modulus of AC-Layer

Different measures of asphalt mixture stiffness at short loading times have been used widely in the design of pavements as a means to evaluate the relative quality of mixtures and to predict load response of pavements under traffic loading. The resilient modulus measured from indirect tension tests have become more generally accepted for this purpose, unlike the dynamic modulus which require fairly large cylindrical specimens, which are cumbersome and inconsistent with specimens produced during mixture design or obtained from field cores. Roque, Buttler, Ruth, and Dickison [1998] have modified the Superpave indirect tension (IDT) system to determine the short-loading-time stiffness of asphalt mixtures from resilient modulus, creep, and strength tests. The main focus of Roque et al. [1998] was to compare different stiffness measurements to each other, as opposed to comparing the effects of different variables such as asphalt modifiers, aging, and air voids on stiffness, performance, or both. The Superpave IDT system requires the use of three test specimens from which six vertical and six horizontal deformations are obtained. Although, the system and associated software were developed specifically to determine creep compliance and tensile strength, a similar approach was used to determine the resilient modulus except that six resilient horizontal and six resilient vertical deformations were used. The evaluation of different short-loading-time stiffnesses was undertaken by using four modified and unmodified asphalt binders which were blended with a dense-graded limestone aggregate mixture at a fixed asphalt content of 6.5 percent to produce mixtures complying with Florida Department of Transportation S-I specifications for structural mixtures.

In comparing the results, Roque et al. [1998] showed that:

- Short-loading-time stiffnesses determined from different tests are significantly different.

- The resilient modulus was generally greater than the creep stiffness, and the creep stiffness was generally greater than the initial tangent modulus.

- Although the stiffnesses generally followed the same trend, the relationships between them were not very strong.

The differences between the stiffnesses were as a result of differences in loading rates. In conclusion, Roque et al. [1998] noted that short-loading-time stiffness appears to be very sensitive to relatively small differences in loading rates, which emphasizes the need to precisely define load pattern, load level, and data interpretation methods for determining a single stiffness parameter. The testing and data reduction procedures developed in the study appear to provide reasonable values of short-loading-time stiffness of asphalt mixtures from resilient modulus, creep, and strength tests conducted with the Superpave IDT. Finally, resilient modulus is probably the best in prediction of load response, since the wave form used in its determination approximates that of a moving wheel [Roque et al. 1998].

Mallick et al. 2002 have described a study that has been undertaken by the Maine Department of Transportation to develop a mix design system for Full Depth

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Reclamation (FDR) of asphalt pavement sections to produce a stabilized based course. This technique utilizes an in-place recycling method where all of the asphalt pavement section along with part of the underlying unbound layers, are treated with different types of additives such as asphalt emulsions and chemical agents including calcium chloride, Portland cement, fly ash and lime, in order to obtain an improved base. The main steps in this process are pulverization, introduction and blending of additive, shaping of the mixed material, compaction, and application of a surface or wearing course. In order to address the problem of lack of a proper mix design procedure for selection of amount of additive for FDR, and the problem of determining the best additive for specific locations in the state of Maine, resilient modulus and bulk specific gravity tests were conducted to determine the optimum content of each additive. The major benefits of FDR includes;

- Improvement to the structure of the pavement without changing the geometry of the pavement and shoulder reconstruction

- Restoration of old pavement to desired profile.

- Elimination of alligator, transverse, longitudinal, and reflection cracking, with an improvement in ride quality.

- Reduction in production cost, since a thin overlay or chip seal, surfacing is required on many projects.

- Reduced frost susceptibility and lower engineering costs.

- Conservation of materials and energy as well as the elimination of air quality problems resulting from dust, fumes, smoke and fumes.

The test plan comprise the determination of the optimum mix design for FDR mix by first selecting additives for the mix that promised to be suitable for achieving adequate strength and durability. Once the additives were selected, it was then necessary to decide an optimum amount of each additive that would provide the maximum strength for that specific additive by conducting resilient modulus and bulk specific gravity tests. Based on the results obtained in the study, Mallick et al. [2002] suggested that a total fluid content criteria based on peak density and resilient modulus could be used for selection of optimum amounts of emulsion or water for full depth reclamation. Mallick et al. [2002] therefore recommended that density and resilient modulus versus total fluid content criteria be used for selecting optimum additive contents [Mallick, Teto, Kandhal, Brown, et al. 2002].

The stress-strain moduli such as the resilient modulus and the dynamic modulus are sometimes used to characterize the inelastic and time-dependent response pavement materials. The resilient modulus represents the stress-strain relationship after many load repetitions (i.e., current modulus of the material, which is normally different from the initial value). On the other hand, the dynamic modulus is a frequency-dependent parameter obtained from dynamic loading tests on a finite specimen. Mamlouk [1985] has described attempts to use the elasto-dynamic analysis in backcalculating the stiffnesses of various layers of an actual pavement structure. Five test sections at the Pennsylvania Transportation Research Facility, which had similar surface temperature, moisture content, and cumulative equivalent axle loads were considered at the time of testing. Each pavement section consisted of four layers: surface, base, subbase, and

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subgrade. Surface deflection measurements were obtained under the Road Rater device, model 400. Because the Road Rater deflection measurements were obtained at an operating frequency of 25 Hz in the field, that frequency was incorporated in the backcalculation process. A major finding from the results is that, the natural vibration frequencies fall within the common range of the operating frequency of the Road Rater. Typical surface deflections were examined to compare static and dynamic pavement responses. Mamlouk [1985] observed that if a static load is applied to the pavement system, the pavement response would be in phase with the load. However, if a vibrating (harmonic) load is superimposed on an initial static load, the instantaneous pavement load will be generally out of phase with the load because of both geometric and material dampings.

Various conclusions have been drawn based on the above discussions [Mamlouk 1985]:

i)   No "direct" mechanistic solution is currently available to backcalculate material properties from surface deflection data obtained by either static or dynamic loading. Iterative processes are usually used for this purpose.

ii)   The dynamic response of a multilayer pavement system is materially different from its static response. Dynamic analysis incorporates the inertial effect (radiation damping and resonance) of the pavement structure, which cannot be incorporated within static analyses. Simply replacing Young's modulus in a static analysis by the resilient modulus or the dynamic modulus is insufficient to recover the elastodynamic equations.

iii)   If the operating frequency of the deflection measurement device coincides with one of the fundamental frequencies of the pavement system, a resonance condition will occur and a large magnification of the deflection measurements will result. Unless dynamic analysis is used in the interpretation of the dynamic response of the pavement system, misleading results may develop. Further research is needed to study the transient loading of the pavement systems obtained from the path of vehicles or from the use of falling weight deflectometer [Mamlouk 1985].

2.2.4   Resilient Modulus of Sand

Temperature variations within a pavement structure contribute in many different ways to distress and possible failure of that structure. The strength and deformation characteristics of asphalt concrete mixes have been shown to change significantly with temperature, which affects the resilient modulus (M_R) of the asphalt layers [Wahab, Asi and Rezquallah 2001]. Cold temperatures accelerate cracking of the asphalt bound layers due to shrinkage, or cause fracture of these layers due to frost heaving of the underlying soils. High temperatures, on the other hand, can cause distortion of the asphalt-bound layers (rutting) or may produce slippery surfaces due to bleeding of the asphalt. Wahab et al. [2001] have presented the results of a comprehensive study that was carried out to explore trends of temperature variations in an arid environment and their implications on the moduli of flexible pavements. Field samples were collected to build up a database about the material characteristics of asphalt concrete mixes. The collected pavement samples were divided into two groups, each group subjected to different laboratory testing as follows:

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- Group I: Cores were collected from both fast and slow lanes and were subjected to M_R testing at 25°C, followed by 40°C, and then 50°C in addition to specific gravity determination.

- Group II: Cores and slabs were collected from both fast and slow lanes and were subjected to an asphalt mix property analysis such as void analysis, bitumen content and viscosity testing, and aggregate gradation and properties determination.

From the results obtained, Wahab et al. [2001] observed that the mix bulk specific gravity (G_{mb}) and M_R at 25°C of the mix were the only common parameters having a direct effect on rutting for both wearing and base course cores. To predict the resilient modulus of an asphalt mix from the different mix and materials physical properties, stepwise forward-selection regression was run to relate M_R values with the different mix and materials physical properties. The generated M_R at any temperature model was

M_R at Temp. = 5.354-0.212Temp.+0.111sof. pnt.-0.170 surface

Where, sof. pnt. = asphalt softening point (°C)

Surface = surface area of the aggregate (m^2/kg)

A temperature correction factor model was also obtained for Group I samples, which had a coefficient of determination of 89%. Wahab et al. [2001] noted that pavement temperature data was collected over a period of two years. Since pavement temperatures, at different depths in the pavement are difficult to measure, prediction models for pavement temperature estimation from climatological data, like air

temperature was used. This is known to be directly affected by cloud cover and solar radiation. A correlation was therefore established between the recorded air temperature and the pavement temperature at the surface and depths of 4, 8, and 16 cm. The results showed a good correlation between pavement temperature and air temperature with a coefficient of determination value (R^2) of 93%. Further studies in this area is recommended which could lead to development of empirical relations for the estimation of the resilient modulus, temperature correction factor, and pavement temperature at any depth, which are essential for pavement design and evaluation [Wahab et al. 2001].

A significant part of transient deflections of a pavement system results from compression of the subgrade material. This induces cracking of asphaltic pavements as a result of excessive plastic and elastic deflections. Resilient modulus (M_R) tests were performed on dune sand [Lee, et al. 1995], which had been used extensively as subgrade material in northern Indiana, and the results of various influencing factors affecting the resilient response of the granular material had been evaluated. Extensive research has already been undertaken to investigate the most important factors influencing the resilient behavior of granular materials, such as the state of stress, degree of saturation, initial density, and gradation. Lee et al. [1995] examined the effects of compaction method, moisture content, and dry density on the resilient characteristics and plastic deformation of dune sand. In addition, regression equations and a design chart were also developed to estimate the M_R.

Samples of disturbed sand, which classified as SP in the Unified Soil Classification System (USCS), were prepared by both impact- and vibratory-compaction methods. The M_R test was carried out following the procedure outlined in AASHTO T 274-82 (1982) the test consisted of six conditioning stages and 27 testing stages, which have combinations of different magnitudes of confining stress and repeated deviator stress. From the test results, Lee et al. [1995] observed that the M_R of the vibratory-compacted specimen had higher values than those of the impact-compacted ones, even though the latter had slightly higher density and lower water content. To investigate the effect of compaction water content on the resilient characteristics of dune sand, the parameters, k_1 and k_2 in the M_R equation were plotted with water content. Both parameters were found to be affected insignificantly by water content for both compaction methods, which implies that the resilient behavior is the same for a dune sand compacted at different water content by the same compaction mechanism and the same compactive effort. Since the standard M_R test is a drained type of test and dune sand is a free-draining material, the pore pressure generated during the test dissipated quickly which made the effect of water on the resilient behavior insignificant. This agrees with the proposition by Hicks and Monismith [1971] that the M_R remained approximately unchanged if test results were analyzed on an effective stress basis. The effect of dry density on the M_R were also illustrated, and Lee et al. [1995] showed that specimens of lower density displayed a decrease in M_R at deviator stress less than 13.8 kPa and a slight increase at higher deviator stress. On the other hand, M_R of specimens with larger dry density increases slightly as the magnitude of the repeated deviator stress increases. This observation agrees with Uzan [1985], who stated that the M_R of a dense graded aggregate

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increases with increasing deviator stress. A regression equation and a chart to estimate the M_R directly from water content and dry density were developed in the study. Due to the limited range of dry density examined, the proposed equation and chart were recommended to be used only in the range of relative compaction between 95% and 103% [Lee, Bohra, and Altschaeffl 1995].

2.2.5 Resilient Modulus Based Designs

The various components of a mechanistic design procedure for conventional flexible pavements (asphalt concrete, AC, granular base and subbase) include inputs like materials characterization, traffic and climate. Other components are the structural model, pavement responses, transfer functions and pavement performance. Thompson and Elliott [1985] have conducted an investigation with emphasis on material characterization, structural model, and pavement response components. Concepts for a mechanistic design procedure based on the ILLI-PAVE structural model and design algorithms developed from a comprehensive ILLI-PAVE database were also presented. In ILLI-PAVE the pavement is considered an axisymmetric solid of revolution. Nonlinear, stress-dependent resilient modulus material models and failure criteria for granular materials and fine-grained soils are incorporated into ILLI-PAVE. The principal stresses in the granular and subgrade layers are modified at the end of each iteration so that they do not exceed the strength of the materials as defined by the Mohr-Coulomb theory of failure. Several studies comparing measured and ILLI-PAVE-predicted load deformation responses have yielded favorable results.

Repeated unconfined compression or triaxial testing procedures are often used to evaluate the resilient moduli, which is a function of the applied stress state, of fine-grained soils and granular materials. A relationship was developed between the resilient modulus and the first stress invariant ($\sigma_1+\sigma_2+\sigma_3$). Two stress-dependent behavior model have been proposed [Thompson and Elliott 1985] for describing the stress softening behavior of fine-grained soils. Extensive resilient laboratory testing, nondestructive pavement testing, and pavement analysis and design studies at the University of Illinois have indicated that the arithmetic model is adequate for flexible pavement analysis and design activities. A constant linear resilient modulus is used to represent the AC layer. The AC modulus values selected for this study are consistent with the range of AC moduli and temperatures expected to be encountered in Illinois. Design response algorithms were developed as the basis of a mechanistic design procedure. Algorithms for predicting the following pavement responses were thus established:

1. AC radial strain at the bottom of the AC surface layer,

2. Subgrade deviator stress,

3. Subgrade stress ratio

4. Subgrade vertical strain,

5. Surface deflection, and

6. Subgrade deflection.

These responses are those which are generally used in various transfer functions relating pavement response to pavement performance. Design algorithms were

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developed using the SPSS stepwise regression program. In summary, Thompson and Elliott [1985] illustrated that pertinent algorithm inputs include AC thickness, AC modulus, granular layer thickness, and subgrade resilient modulus. The algorithms should not be extrapolated beyond the range of variables considered in the ILLI-PAVE data-base unless check runs are conducted with ILLI-PAVE to determine the validity of the algorithms in the area of extrapolation. Finally, factors relating to climate, traffic, and transfer functions for local conditions must be appropriately evaluated and included in the development of a complete mechanistic design procedure [Thompson and Elliott 1985].

Several transportation agencies have evaluated the M_R using simple empirical (non-stress-dependent) relationships with the California bearing ratio (CBR). It is however, clear that the response of typical unbound subgrade materials may be nonlinear and as such a function of the state of stress. A study has been conducted by Witczak, Qi and Mirza [1995] to estimate the nonlinear resilient modulus response of fine-grained soils to be used in the flexible design procedure from the AASHTO. Three approaches were presented based on stress dependent Lofti and Moossazadeh predictive equations for estimating nonlinear modulus of subgrade.

The University of Maryland has developed a project-level life-cycle-cost flexible-pavement design model (LCCP-Flex) for the Maryland Department of Transportation (MDOT). The 1986 AASHTO design guide introduced two major changes in the analysis that were not incorporated in version 1.0 of the MDOT LCCP-Flex program. They concerned the design reliability analysis and replacement of the empirical soil

support parameter (s) with the M_R of the subgrade soil. In general, model forms (relating M_R to the deviatoric stress σ_d) can be characterized into one of four analytical approaches that has been developed: bilinear model, semi-log model, hyperbolic model, and log-log model. The authors selected the log-log model form in the development of the nonlinear M_R module for the MDOT LCCP-Flex program solution. The Witczak et al. [1995] relied on two separate historic studies called the Lofti analysis and the Moossazadeh analysis in developing the nonlinear subgrade M_R analysis into the AASHTO flexible pavement design procedure. With the Lofti equation, it was apparent that its implementation required the ability to predict the deviatoric stresses within the subgrade caused by the application of external load on the specific pavement cross-section in question. The prediction of deviatoric stress within the subgrade of the multilayer pavement system can be determined through transforming the multilayer pavement system into an equivalent Boussinesq system using the derived numerical integration equations to determine stress states at the desired computational point. In contrast to the Lofti approach, which was developed from nonlinear M_R laboratory test results, the nonlinear subgrade M_R solution developed by Moossazadeh and Witczak is based on the prediction of an "equivalent subgrade modulus" within a highway pavement cross-section using as an input the k_1-k_2 nonlinear M_R parameters. Because this steady is specifically concerned with the nonlinear subgrade M_R application, the computer framework for this solution under the "constant annual (effective design) resilient modulus" option was illustrated [Witczak et al. 1995]. This module has three optional procedures in which the nonlinear M_R value could be determined. These involve the use of either the Lofti or

Moossazadeh approach coupled with the use of inputting either CBR values or laboratory-derived nonlinear M_R constants (k_1 and k_2). A full-scale comparison of predicted nonlinear M_R values for all three options presented was difficult due to lack of sufficient soil test results (CBR, k_1 and k_2) for a wide range of soils. Witczak et al. [1995] concluded that, the initial analysis of the available M_R solution options within LCCP-Flex appear to be very encouraging. Available options within the program allow the use of either soil CBR value to approximate the nonlinear k_1-k_2 constants or direct laboratory results of the measured k_1-k_2 values. Further investigations are, however, desirable to improve and refine the methodologies developed [Witczak et al. 1995].

Several techniques for estimating M_R from soil index properties such as plasticity index (PI) and percent clay have been developed. Hall and Thompson [1994] have described methodologies for estimating the subgrade resilient modulus for pavement design purposes by using established techniques and readily available soil property information. Hall and Thompson [1994] conducted studies to investigate the repeated load behavior of 50 typical Illinois fine-grained soils. The subgrade modulus was significantly correlated with liquid limit (LL), plasticity index (PI), group index (from the AASHTO soil classification system), silt content, clay content, specific gravity and organic carbon content. A regression equation for conventional flexible pavement design was proposed by Thompson and LaGrow:

$$M_R (OPT) = 4.46 + 0.098C + 0.119PI \qquad (2.3)$$

The regression algorithm is improved if soil organic carbon content is included:

$$M_R (OPT) = 6.90 + 0.0064C + 0.216PI - 1.97OC \qquad (2.4)$$

Where, M_R (OPT) = subgrade resilient modulus,

C = percent clay ($<2\mu m$),

PI = plasticity index

OC = percent organic carbon

The subgrade resilient modulus can also be back-calculated using surface deflections generated from a falling-weight deflectometer (FWD). Studies at the University of Illinois have resulted in algorithms for estimating the subgrade resilient modulus from pavement surface deflections. A database has been generated by the ILLI-PAVE finite-element structural pavement model. The algorithm relates M_R to pavement surface deflection D_3 for each specific pavement structure. Soil property and FWD data from Champaign County, Illinois were also used to develop relationships between soil-property-based and deflection-based estimates of M_R. The relationships developed were then verified with soil property and FWD data from four additional counties in Illinois. Two methods were then developed to relate the soil-property-based M_R estimates and FWD deflection-based M_R estimates. The methods include the use of a moisture adjustment ratio (MAR), which is a function of the optimum moisture content of the soil, and a relationship between the soil's natural drainage class and the required degree of saturation. The analysis presented by Hall and Thompson [1994] suggests that the relationship between soil-property-based M_R and FWD deflection-based M_R depends somewhat on the predominant soil series of an area. The methodologies presented could therefore be easily adapted by pavement agencies to local conditions, potentially

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providing a valuable alternative to doing nothing with regard to subgrade evaluation for pavement design [Hall and Thompson 1994].

The 1986 AASHTO *Guide for Design of Pavement Structures* incorporated the resilient modulus (M_R) in the design process by providing correlation with the more traditional "soil-strength" parameters such as: soil support, California bearing ratio (CBR), Kentucky CBR, Texas triaxial, R-value, and structural coefficient. Several studies have been undertaken on the use of M_R in characterizing pavement materials under traffic loading. A careful examination of the literature have revealed that many of these correlations would not yield the correct predictions when back-correlated through an intermediate parameter. Southgate and Mahboub [1994] have conducted a study to examine several confusing issues surrounding the use and measurement of resilient modulus in the context of the 1986 AASHTO guide. The AASHTO relationship between soil-support value and resilient modulus was presented and the equation could be verified by eliminating like terms in an equation of the 1972 interim AASHO guide. Another figure (HH.2) in the 1986 guide provides a relationship between M_R and modulus of subgrade reaction (k). However, the 1986 ASHHTO guide also defines the following relationship: $M_R = 1,500(CBR) < 10$. Several relationships that are reported in the 1986 AASHTO guide linking soil-support value to other parameters, all of which lead to possible sources of discrepancies were analyzed and discussion of key points from literature have been presented. In the context of the 1986 AASHTO guide, the term *resilient modulus* has been used interchangeably with the following mathematical relationships: (a) 1,500 x CBR; and (b) as a function of soil-support value. The scale

labeled "Kentucky CBR" which is widely reported in the literature has an unknown origin. Also the relationship between soil support value and Kentucky CBR which has been illustrated in a figure was developed from the results of the "round-robin" laboratory tests of soil samples from the AASHO road test in the late 1960's.

Laboratory procedures, generally, allowed for a rather effective control of test variables, however, they are somewhat deficient in addressing many variables introduced under actual field conditions. In order to make up for this deficiency, laboratory tests for M_R of unbound pavement materials are sometimes conducted over a wide range of variables such as degree of saturation, level of compaction and stress level. On the other hand, field-based methods for measurements of M_R have shown to be more representative of the actual field conditions; however, their margin of error can be found to be very high: 30-200% [Southgate and Mahboub 1994]. It has been widely reported that unbound granular materials demonstrate different resilient moduli under different stress levels, which means that the modulus of a granular pavement layer is highly influenced by the stiffness and thickness of the layers just above and underneath. The main issues that remain to be tackled are: "what combination of confining pressure, deviator stress, loading rate and frequency, and other variables should be used for laboratory testing?" Obviously, one expects the field conditions to be best simulated in the laboratory. Unfortunately, the stress-sensitive nature of pavement materials and uncertainties involved in properties of different pavement layers, and large variability in type and magnitude of traffic loads, make approximation of pavement stresses a less precise endeavor. A study is underway for standardization of laboratory M_R testing and

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integration with the AASHTO flexible pavement design. In summary, Southgate and Mahboub noted that both material characterization and methodology for back-calculation need significant improvement, and these goals may be realized through a more mechanistically realistic constitutive modeling, and through three-dimensional dynamic pavement modeling [Southgate and Mahboub 1994].

2.2.6 Special Cases

High-speed railroads have been identified worldwide as the form of transit that will see in the new millennium for medium distance journeys. The dynamic movements of the soils as trains pass over them are typically modeled by loads crossing a homogeneous elastic foundation. To determine the correct stress-strain response (incorporating stiffness, viscoelastic, and damping effects) is not a straightforward task. The piece of equipment more suited for these typical ballast strain levels, and for the calculation of damping, is the resonant column apparatus. In situ measurements involving a continuous surface wave system (CSWS) supplied by GDS Ltd., Surrey, United Kingdom has been used on a railway line to provide estimates of the resilient modulus of the soil. Two tests were performed by Heelis et al. [1999] on a 6m embankment overlying a peaty clay subgrade. The first test was performed with geophones on the ties of a railway line, whereas the second test was performed on the embankment adjacent to the first test but off the railway line. The simplest analytical approach commonly used to model ground response to a traveling train is two-dimensional model of a point load moving on an infinite beam on an elastic foundation comprising a series of discrete springs.

The main problem with the concept of the coefficient of subgrade reaction (the Wrinkler model) is how to relate the coefficient of subgrade reaction to the measurable resilient modulus and Poisson's Ratio for the soils. Heelis et al. [1999] showed that the soil structure models can give values of the subgrade reaction that vary between 10.10 and 16.54 MN/m^2 for the embankment previously considered. The model used can be defined using two nondimensional parameters. The first relates to the relative speed of the train compared to the stiffness of the track foundation. The second relates to the damping of the overall system. The foundation model used is a Wrinkler model; therefore a coefficient of subgrade reaction is required. It is possible to relate the resilient soil modulus to the coefficient of subgrade reaction and to provide design criteria for trains crossing soils with a low resilient modulus [Heelis, Dawson, Collop, Chapman, et al. 1999].

Wolfe, Butalia and Meek [1999] have conducted a study to examine the potential usage of a Pressurized Fluid Bed Combustion (PFBC) material, as a substitute for conventional road construction material in the design of flexible pavement systems. The PFBC materials occur as by-products from coal- fired power plants and have over the years been disposed of as solid waste and landfilled. Studies have shown that the engineering response of compacted industrial by-products can be greatly affected by freeze-thaw cycling and this has been of much concern. The Ohio State University has conducted extensive research into how wet and dry FGD materials (including PFBC) could be used in land reclamation, pavement applications, soil stabilization and

agriculture. The PFBC material used in this study was collected from a pile that had been exposed to Fall weather conditions at the site of the Ohio SR 541 highway embankment repair project near Coshocton in East Central Ohio. Six samples of compacted PFBC materials were used in the test which consisted of the application of cyclic loads at two different moisture contents, and several combinations of freeze-thaw cycling as well as curing times.

From the results obtained Wolfe et al. [1999] showed that the observed values of M_R for compacted PFBC materials subjected to freeze-thaw cycling compare well with those of conventional base and subgrade materials. Although the M_R values observed were much higher than the minimum values recommended by AASHTO, Wolfe et al. [1999] have suggested its use as potential subgrade material in pavement construction to be limited to low traffic volume roads. The effect of moisture content revealed that the sample with higher moisture content exhibited lower M_R values for the range of bulk stresses under consideration. The effect of repeated thermal cycling on resilient modulus also show lower M_R values for samples with high water content [Wolfe et al. 1999].

Resilient modulus can be determined by laboratory testing or by an analytical procedure known as backcalculation involving the interpretation of nondestructive testing. Several investigations have shown that the resilient modulus determined from laboratory testing differs from that determined from backcalculation. Mikhail, Seeds, Alavi, and Ott [1999] have conducted a study to compare and evaluate the differences between resilient moduli from laboratory testing and those determined through

backcalculation analysis of falling weight deflectometer (FWD) data from the West Track experiment with the intention of setting the stage for further research. Mikhail et al. [1999] further sought to identify many of the strengths and weaknesses of both methods, offer some recommendations for addressing the limitations, and also identify which methodology is better suited for pavement evaluation and design. A wide range of testing was carried out on West Track materials in order to characterize their in situ properties. Of relevance to this study are (a) the nondestructive test data collected using a FWD seismic pavement analyzer (SPA), (b) the laboratory resilient modulus testing carried out on the natural soil and each layer of the pavement structure, and (c) the quality-assurance testing carried out during testing.

Some known and potential limitations and weakness of laboratory resilient modulus testing are as follows [Mikhail et al. 1999];

1. The laboratory resilient modulus may not be representative of the condition in situ because of factors such as sample disturbance, aggregate orientation, uniformity of moisture content in the sample and level of compaction of the specimen.

2. The in-situ state of stress may not be accurately simulated in laboratory specimens under axial or triaxial loading.

3. There is lack of uniform equipment calibration and verification procedures for resilient modulus testing.

4. Laboratory specimens represent properties of a localized material because of the limited geographical area from which they were taken.

The materials used for the tests were natural soil, engineered fill, base course and hot-mix asphalt (HMA). Inspection of the laboratory test results showed that the HMA surface layer was dependent upon temperature and that the unbound layers depended upon the state of stress. Backcalculation refers to a process of estimating in-situ pavement and soil layer properties through the use of nondestructive pavement surface deflection measurements obtained at a given site. NDT measurements obtained using the Dynatest FWD were collected during the West Track experiment. An outcome of the backcalculation process was an apparent similarity in stiffness between the engineered fill and the natural soil directly beneath. Comparison between the laboratory and NDT-based resilient modulus values were made on the various materials. The following observations were made about the results of the study.

● Significant differences were found between resilient modulus determined from backcalculation and those determined through laboratory testing for certain layer materials. For the unbound granular base course and especially for the HMA layer, the differences are meaningful. For the fine-grained engineered fill and natural soil, the limited test results and comparisons did not indicate a significant difference.

● The resilient modulus determined from laboratory testing represent homogeneous specimens, while the backcalculated moduli represent effective or equivalent moduli.

● NDT-based backcalculation does not necessarily produce a unique solution. In fact, there are (in the case of West Track) a number of solutions-some reasonable, some not-that can have a low root-mean-square error.

In conclusion, Mikhail et al. [1999] noted that further improvements are required in backcalculating procedures to address the problem with multiple solutions and the sensitivity of the results to structural and environmentally related factors. Mikhail et al. [1999] also suggested that further research be conducted to close the gap between laboratory based resilient modulus and backcalculated resilient modulus or to derive an appropriate correction factor [Mikhail et al. 1999].

To address the issue of environmental impacts on pavements at a national level, the Federal Highway Administration (FHWA) launched the Seasonal Monitoring Program (SMP) as a major component of the Long-Term Pavement Performance (LTPP) program. The performance of a pavement is affected by its structural conditions, which can be represented by the elastic moduli of the pavement layers. On the other hand, the elastic modulus of a flexible layer is a main property that depends on the material type and condition. For example, the asphalt bound layer is sensitive to temperature variation, while untreated layers such as base and subgrade are more sensitive to moisture variation. In pavement design and performance evaluation process, it is essential to assess the variation of the material properties with varied environmental conditions. A research program is being undertaken [Abo-Hashema et al. 2002] to develop a pavement climatic database that may encompass the varied climates and soil conditions in the State of Idaho. The rational adopted in incorporating the environmental effects in the design system was to ensure a more engineering-base approach to enable the design engineer to adjust the subgrade resilient modulus from one season to another for a given soil at a given climatic region. To achieve the above objective, five sites were installed in the

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State of Idaho to measure temperature and moisture variations throughout the pavement system. The data collected were analyzed, in addition to some data from the LTPP-SMP database, to develop regression equations between the volumetric moisture content and the subgrade resilient modulus for the various soil types and climatic regions. Abo-Hashema et al. [2002] observed that knowing the average moisture content, the developed equations can be used to adjust the subgrade resilient modulus from one season to another [Abo-Hashema, Bayomy, Smith, and Salem 2002].

For a new design of pavement structures, the resilient modulus values as recommended by AASHTO could be generally obtained by conducting repeated triaxial tests on reconstituted/undisturbed cylindrical specimens. The laboratory test is a tedious, costly and time-consuming procedure. Moreover, large numbers of samples need to be collected and tested for reasonably accurate results. Even then, it is difficult to reproduce the in-situ sample conditions. Considering the complexity of the repeated load triaxial test, field testing procedures have been proposed to estimate subgrade moduli. Rahim and George [2002] have conducted a study in Mississippi to investigate the viability of using Automated Dynamic Cone Penetrometer (ADCP, abbreviated as DCP) for subgrade characterization through correlation between DCP index (penetration per blow) and resilient modulus. Twelve as-built subgrade sections reflecting a range of typical subgrade materials of Mississippi were selected and tested with DCP. Undisturbed samples were retrieved, using a Shelby tube, and tested in a repeated triaxial equipment for resilient modulus. Since the DCP test is destructive in nature, it was not realistic to expect a direct relation between the resilient modulus and the DCP Index (DCPI). It was

therefore prudent to include soil state properties such as dry density, moisture content, liquid limit, plasticity index, percent passing #200 sieve, and uniformity coefficient, as independent variables in the regression models. The models were verified by repeating the tests in another site and comparing the measured and predicted moduli. Rahim and George [2002] therefore illustrated that for the range of soils tested, the developed resilient modulus–DCPI models provided useful predictions of resilient modulus. This suggests that the automated DCP is a simple and expedient device for field testing of soils and particulate material. [Rahim and George 2002]

2.3   Factors Affecting Resilient Response

The development of permanent strains in pavements when subjected to repeated traffic loads is of major practical significance since it could influence the performance and serviceability of the pavement structure. To establish a more rational pavement design and construction criteria, it is essential that the response of granular layers under traffic loading be thoroughly understood and taken into consideration. The stress pattern induced in a pavement structure as a result of traffic loading is complex. Figure 2.2 illustrates that an element in a pavement is subjected to three stress pulses, each consisting of vertical, horizontal, and shear stress components [Lekarp, Isacsson, and Dawson 2000]. These stresses are transient and change with time as the wheel passes. The vertical and horizontal stresses are positive in unbound layers since unbound granular materials do not carry significant tensile stresses. The shear stress is reversed as the load passes, thus causing a rotation of the axes of principal stress.

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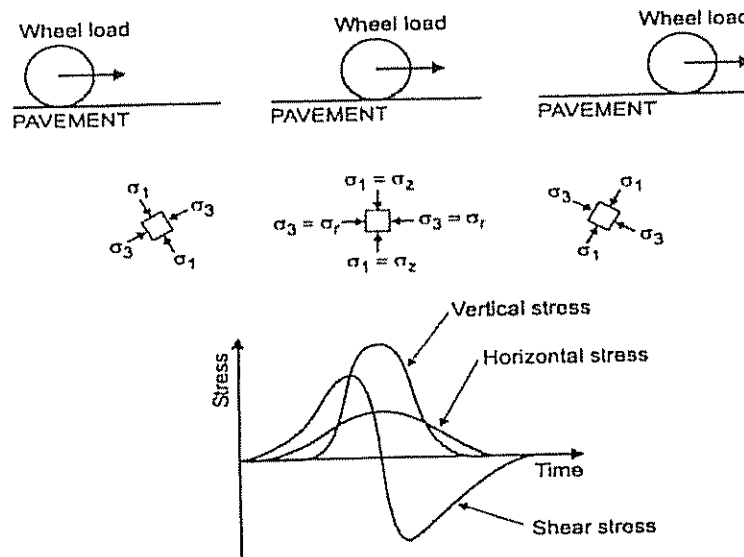
Figure 2.2: Stresses under a wheel load. From Lekarp et al. [2000].

Two types of deformation are produced in a pavement subjected to traffic loading:

(a) resilient (recoverable) deformation, possibly leading to fatigue cracking of the asphalt surface, and

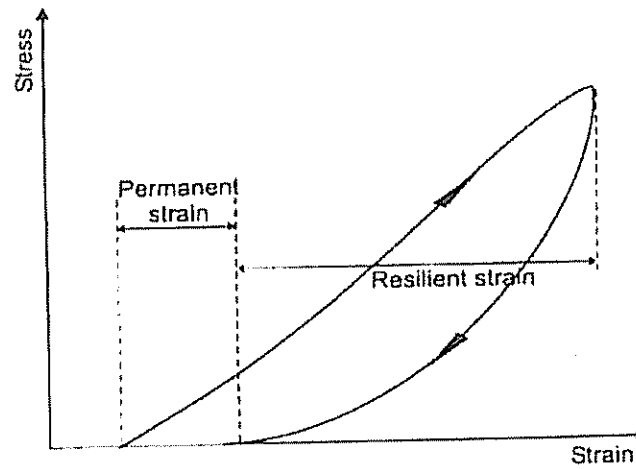(b) accumulated plastic (irrecoverable) deformation, which may lead to excessive rutting as illustrated in Figure 2.3.

Figure 2.3: Deformations in a pavement structure. From Lekarp et al. [2000].

It has been postulated [Luong 1982] that the deformation of granular soils under loading is the result of three main mechanisms: consolidation, distortion, and attrition. The consolidation mechanism (densification/dilation) is the change in shape and compressibility of particle assemblies, whereas the distortion mechanism is characterized by bending, sliding, and rolling of individual particles. Particle bending is important in the case of flat particles whereas sliding and rolling are usually associated with rounded grains. The resistance to particle sliding and rolling depends on the interparticle friction in the grain assembly. The attrition mechanism is the crushing and breakage that occurs when the applied load exceeds the strength of the particles. Crushing is a progressive process that can start at relatively low stresses, and results in gradual changes in the soil fabric and packing. Particle crushing is governed by grain size and shape, magnitude of applied stresses, mineralogy and strength of individual grains. When the behavior of

granular materials is analyzed at the macroscopic level, the observed deformation may be volumetric, shear, or a combination of the two. These volume and shear strains result from various combinations of the three mechanisms mentioned. It therefore seems probable that distortional particle movements contribute mainly to shear strain, whereas consolidation and attrition contribute mainly to volumetric strain. However, this separation is not exact, as shear strain in granular assemblies is usually associated with volumetric strain (e.g., dilation of a dense material).

In order to study the response of pavement materials, it is necessary to create testing conditions as close as possible to that occurring in the field. The stress condition is one of the most important factors affecting granular materials in pavements. Most studies into this problem use devices which can simulate the stresses occurring beneath a loaded wheel. Laboratory and field procedures are currently either used or evaluated for determining soil properties. Several laboratory procedures such as repeated load triaxial tests, resonant column tests, and hollow cylinder tests have been used for determining and investigating the influence of these variables on the dynamic properties of the aggregates. Among the testing procedures, the resilient modulus from the repeated load triaxial test is frequently used because of the repeatability of the test results and its proper representation of simulation of field stress in laboratory environments. The resilient response of aggregates is affected by several factors with varying degrees of importance. These include the influence of soil and instrumental variables such as stress level, density, grading, fines content, number of load applications, and moisture content. Aggregate characteristics, including shape, angularity, surface roughness, and roundness

have an important influence on the resilient and permanent response of an unbound aggregate. The following represents different views on the impact of each individual factor.

2.3.1 Effect of Stress

Previous investigations, from early studies reported by Williams [1963] to the recent studies by Kolisoja [1997], have shown without exception that stress level is the factor that has the most significant impact on resilient properties of granular materials. Many studies have shown a very high degree of dependence on confining pressure and sum of principal stresses for the resilient modulus of untreated granular materials. The resilient modulus is said to increase considerably with an increase in confining pressure and sum of principal stresses. Compared to confining pressure, deviator or shear stress is said to be much less influential on material stiffness. In a study conducted by Morgan [1966], the M_R was shown to decrease slightly with increasing repeated deviator stress (or stress difference) under constant confinement. Other have, however, suggested that the M_R is practically unaffected by the magnitude of the deviator stress applied, provided excessive plastic deformation is not generated. Slight softening of the material has also been reported at low deviator stress levels and slight stiffening at higher stress levels. In laboratory triaxial testing, both constant confining pressure (CCP) and variable confining pressure (VCP) are used. Comparison of the results obtained from these two types of tests as reported by Allen and Thompson [1974], generally, indicated higher values of resilient modulus computed from CCP test data.

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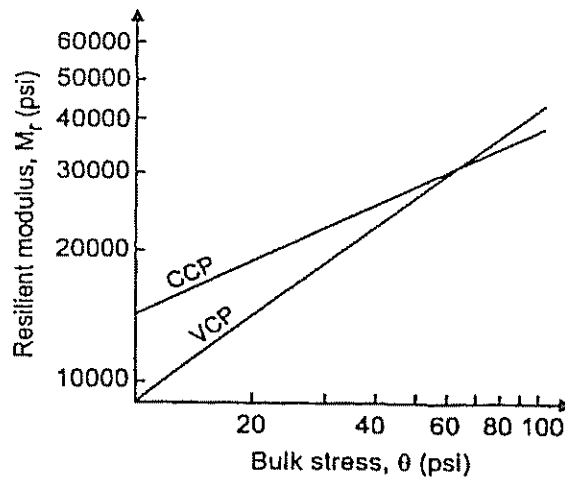
Figure 2.4: Example of triaxial test result with CCP and VCP
From Allen and Thompson [1974].

The magnitude of the difference was itself non-constant and varied with stress level. The result also showed that the CCP tests resulted in larger lateral deformations.

2.3.2 Effect of Density

It has been known for many years that increasing density of a granular material significantly alters its response to static loading, causing it to become both stiffer and stronger. However, the effect on resilient stiffness has been less thoroughly studied. Several studies have suggested that the resilient modulus generally increases with increasing density. A slow repeated load test on a uniform sand [Trollope et al. 1962] was reported to show that the resilient modulus increased up to 50% between loose and dense specimens. It could be argued that the number of particle contacts per particle increases greatly with increased density resulting from additional compaction of the particulate system. This, in turn, decreases the average contact stress corresponding to a

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certain external load. Hence, the deformation in the particle contacts decreases and the resilient modulus increases [Kolisoja 1997]. Other studies also found the effect of density to be greater for partially crushed than for fully crushed aggregates. The resilient modulus was reported to increase with relative density for the partially crushed aggregate tested, whereas it remained almost unchanged when the aggregate was fully crushed.

2.3.3 Effect of Moisture Content

Water has always been a matter of serious concern in road engineering. The degree of saturation or moisture content of most untreated granular materials has been found to affect the resilient response characteristics of the material in both laboratory and in situ conditions. It is generally agreed that the resilient response of dry and most partially saturated granular materials is similar, but as complete saturation is approached, the resilient behavior may be affected significantly. Saturated granular materials develop excess pore-water pressure under repeated loading. As pore water develops, the effective stress in the material decreases with a subsequent decrease in both strength and stiffness of the material. It can be argued that it is not the degree of saturation per se that influences the material behavior, but rather the pore pressure response controls deformation behavior. Studies conducted on the behavior of granular materials at high degrees of saturation, have all reported a notable dependence of resilient modulus on moisture content, with the modulus decreasing with growing saturation level, especially as the moisture content increases above its optimum value. Figure 2.5 shows a study on the effect of moisture content variation on Sawyer aggregates which are typically used in Oklahoma as subbase and base materials for roadway pavements.

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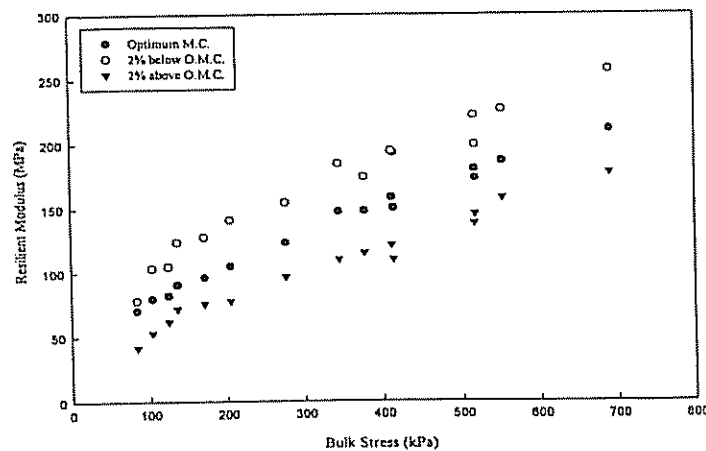
Figure 2.5: Mean M_R values of Sawyer aggregate at different moisture contents. From Ping Tian et al. [1998]

Some researchers have argued that the presence of water in an aggregate assembly has some lubricating effect on the particles, and that this would increase the deformation in the aggregate assembly with a consequent reduction of the resilient modulus, even without generation of any pore-water pressure. This hypothesis was confirmed by Thom and Brown [1987], with a series of repeated load triaxial tests on a crushed rock, where the moisture content was one of the parameters changed. Using drained tests and loading frequencies of 0.1-3 Hz, no noticeable pore pressures were developed for degrees of saturation up to 85%. Despite the lack of pore pressure, the test results showed a reduction to the resilient modulus with increasing moisture content, and this was attributed to the lubricating effect of water. However, another way of interpreting these observations would be that localized pore suctions decrease with higher water content, leading to lower interparticle contact forces.

2.3.4 Effect of Stress History and Number of Load Cycles

Studies have indicated that stress may have some impact on the resilient behavior of granular materials. The stress history effects appear as a result of progressive densification and particle rearrangement under repeated application of stress. The results of a repeated load triaxial test on samples of a well-graded crushed limestone, all compacted to the same density in the dry state , showed that the material was subjected to stress history effects. These effects could be reduced by preloading with a few cycles of the current loading regime and avoiding high stress ratios in tests for resilient response.

In a study conducted by Lufti Raad et al. on a saturated granular base G3 (classified as GW-well-graded) as illustrated in Figures 2.6 and 2.7 below, it was observed that the resilient modulus decreases as the number of load repetition increases.
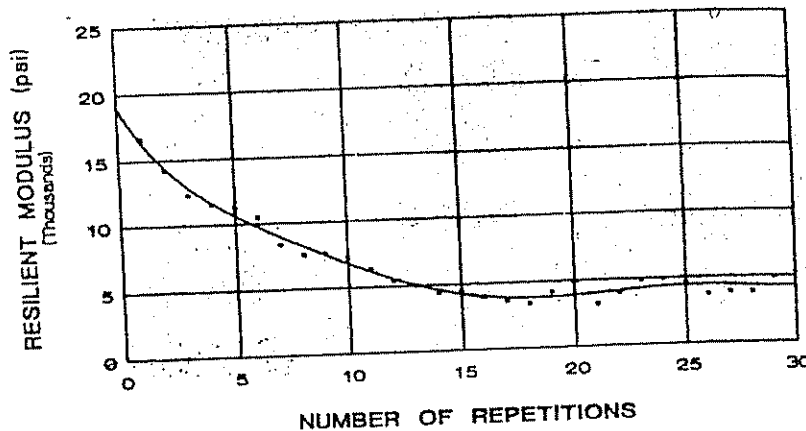
Figure 2.6: Variation of resilient modulus with number of repetitions for Aggregate G3 (saturated conditions). From Lufti Raad et al. [1992]
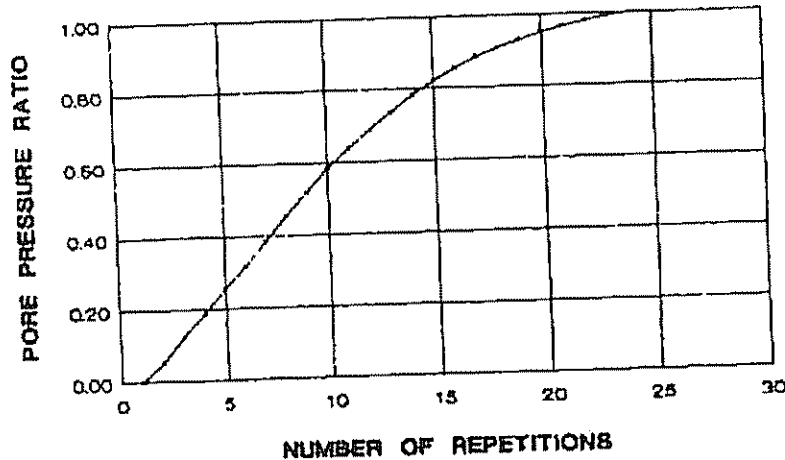
Figure 2.7: Variation of pore pressure ratio with number of repetitions for Aggregate G3 (saturated condition). From Lufti Raad et al. [1992]

This was caused by increase in pore pressure ratio (i.e., ratio of excess pore water pressure in specimen to initial applied effective pressure), thereby resulting in decrease in effective stresses and gradual softening and collapse of the stress-strain curve. In case pore pressure ratio increases and reaches unity, liquefaction will occur and result in the possible pumping of underlying soil through pavement cracks and joints.

2.3.5 Effect of Load Duration, Frequency, and Load Sequence

The general view regarding the impact of load duration and frequency on the resilient behavior of granular materials is that these parameters are of little or no significance [e.g., Seed et al.(1965), Morgan (1966), Hicks (1970)]. For instance, a study was reported [e.g., Seed et al. 1965], in which the resilient modulus of sands increased only slightly (from 160 to 190 MPa) as the duration of load decreased from 20 min. to 0.3s. Other tests at stress durations of 0.1, 0.15, and 0.25 seconds have indicated no change in the resilient or Poisson's ratio [Hicks 1970]. It has been suggested that there is

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the likelihood that the resilient modulus will show a reduction with increasing loading frequency when the moisture content approaches saturation as transient pore pressures may then develop, causing a reduction in effective stress [Lekarp et al. 2000]

2.3.6 Effect of Downscaling on Triaxial Test Results

One of the issues that should be taken into consideration in triaxial testing is the limitation that exist due to specimen size, i.e., the size of specimen must have a relation to the size of aggregate to be tested. In this regard, it is recommended that the diameter of the specimen should be at least 5 times the maximum particle size and the length of the specimen at least two times its diameter. Most repeated load triaxial testing facilities presently available have specimen diameters of 300 mm, and 150 mm or even less. Considering the relationship between specimen size and maximum particle size in the aggregate to be tested, it is common practice that for coarse aggregates the tests are carried out on so-called scaled-down gradings. When scaling down the material, the large particles are replaced by finer particles, keeping the shape of the grading curve intact. The effect of scaling down is not yet understood. It has been suggested that scaling down the triaxial specimen would exclude a large portion of the original aggregate, implying that, the remaining of the aggregate will be almost like a different material behaving differently under loading which would result in inconsistent response.

2.3.7 Effect of Grading, Fines Content, and Maximum Grain Size

Granular materials consist of a large number of particles, normally of different sizes. Previous studies have shown that the stiffness of such material is, to some extent,

dependent on particle size and its distribution. The literature is not quite clear regarding the impact of fines content on material stiffness, however, some researchers have reported that the resilient modulus generally decreases when the amount of fines increases. Hicks and Monismith [1971] observed some reduction in resilient modulus with increasing fines content for the partially crushed aggregates tested, whereas the effect was reported to be the opposite when the aggregates were fully crushed. Other studies showed an initially increasing stiffness and then a considerable reduction as clayey fines were added to a crushed aggregate as illustrated in Figure 2.8 below.

AGGREGATE INFLUENCE FACTOR (AIF)
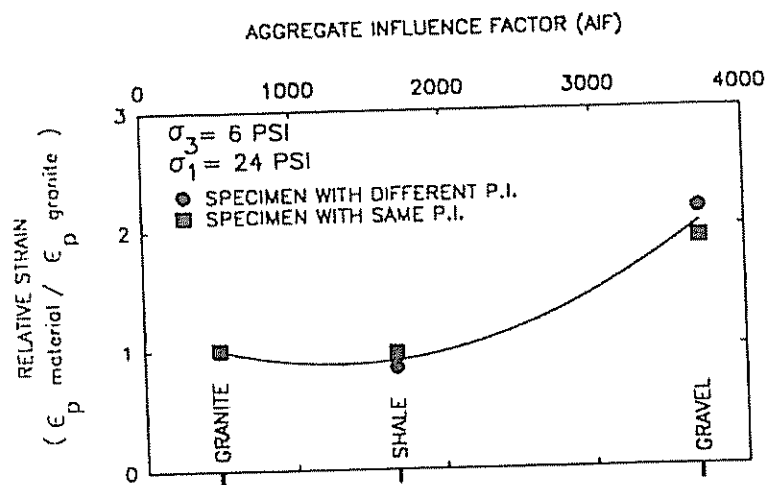
Figure 2.8: Influence of plasticity of fines on relative strain for the medium gradation.
From Barksdale and Itani [1989]

The initial improvement in stiffness is attributed to increased contacts as pore space is filled. Gradually, excess fines displace the coarse particles so that the mechanical performance relies only on the fines, and stiffness decreases. The particle size

distribution, or grading, of granular materials seems to have some influence on material stiffness, though it is generally considered to be of minor significance. Plaistow [1994] argued that when moisture is introduced to well-graded materials, the effect of grading can be significantly increased, because these materials can hold water in the pores, and they can also achieve higher densities than uniformly graded materials. The smaller grains fill the voids between the larger particles and therefore able to form a minisci. Deformation in particle contacts decreases, resulting in overall increase in material stiffness. He therefore concluded that grading has an indirect effect on the resilient behavior of unbound aggregates by controlling the impact of moisture and density of the system. Figure 2.9 shows test on three gradations, finer limit, median, and coarser limit as specified by Oklahoma Department of Transportation for type A aggregate.
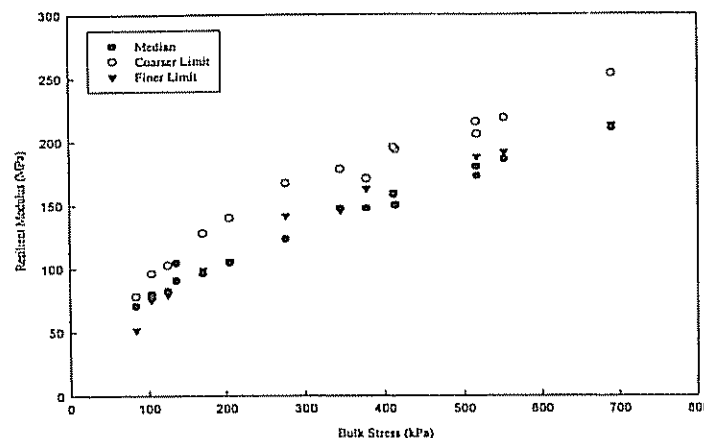
Figure 2.9: Mean M_R values of Sawyer aggregate at different gradations.
From Ping Tian et al. [1998]

The coarser limit gradation produced the highest M_R values (nearly 10 to 36 percent higher than the finer limit and the median gradations), and the M_R values of the median and the finer limit gradations are nearly in the same range. Ping Tian et al.

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[1998] argued that the lower M_R values obtained for the Sawyer aggregates may be because the drainage rate of the finer limit aggregates is slower than that of the coarser limit aggregates, or because the finer limit aggregates lack larger irregular particles to provide a strong interlock between particles.

2.3.8 Effect of Aggregate Type and Particle Shape

A recent study conducted by Heydinger et al. [1996], suggested that gravel have a higher resilient modulus than crushed limestone. Several researchers have, however, reported that crushed aggregate, having angular to subangular shaped particles, provides better load spreading properties and a higher resilient modulus than uncrushed gravel with subrounded or rounded particles. Increasing particle angularity and surface roughness could also result in a higher resilient modulus. Barksdale and Itani [1989] investigated several types of aggregate and observed that the resilient modulus of the rough, angular crushed materials was higher than that of the rounded gravel by a factor of about 50% at low mean normal stress and about 25% at high mean normal stress.

Table 2.1: Shape and Surface characteristics of aggregates. From Barksdale and Itani [1989]

BASE TYPE	GRANITE GNEISS	GRAVEL	SHALES	QUARTZITE	LIME STONE
ELONGATION RATIO (q)	0.68	0.72	0.57	0.63	0.47
FLATNESS RATIO (p)	0.60	0.70	0.37	0.48	0.48
SPHERICITY	0.86	0.88	0.69	0.76	0.72
ANGULARITY	1350	150	750	1450	1450
ROUNDNESS	0.2	0.7	0.4	0.2	0.2
ROUGHNESS	800	100	300	800	800
AGGREGATE INFLUENCE FACTOR	500	3700	1675	150	50
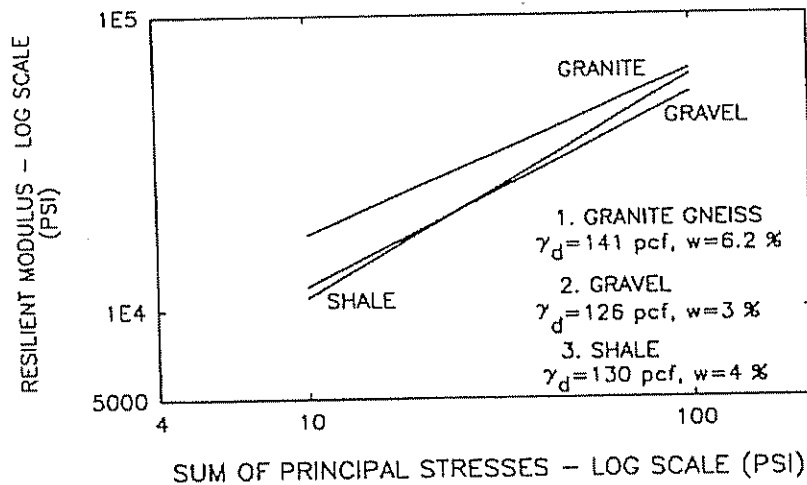
Figure 2.10: Influence of material type and state of stress on resilient modulus for median gradation. From Barksdale and Itani [1989]

2.4 Summary of the Literature and Relation to Research

Based on the literature presented, the following main conclusions can be drawn.

1. Unbound granular materials show a complex elastoplastic behavior when subjected to repeated loadings such as those generated by moving traffic. Many researchers have studied the behavior of these materials at the macroscopic level, but understanding the true nature of their response at a microscopic, particulate level is a great challenge yet to be overcome.

2. According to the literature available, the resilient behavior of granular materials, defined by resilient modulus and Poisson's ratio, is affected by factors such as stress level, density, grading, fines content, maximum grain size, aggregate type, particle shape, moisture content, stress history and number of load applications.

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Aggregate characteristics, including shape, angularity, surface roughness, and roundness have an important influence on the resilient and permanent response of an unbound aggregate.

3. Modeling is an important requirement in dealing with material performance. As described in the above literature, many investigations conducted have outlined different procedures for predicting the resilient response in granular materials. This is further evidence of the complexities that overshadow this research area.

Several authors seem to agree that the resilient response is influenced most by the level of applied stresses, and the amount of moisture present in the material. An increasing moisture content, particularly at high degrees of saturation, has been shown to result in a marked reduction of resilient modulus. The influence of the other factors on resilient response of granular materials is rather unclear. Various authors do not always agree on the nature of the impact of these factors and different, or even completely opposite, conclusions are often drawn.

Chapter 3

EXPERIMENTAL DESIGN, MATERIAL SELECTION

3.1 Experimental Program

The experimental program consists of mainly laboratory tests. Representative samples of subgrade materials were obtained from various proposed and ongoing highway construction projects in the New Castle, Kent and Sussex Counties in Delaware. Gradation and Atterberg limit tests were initially conducted on the various materials by the Materials and Research division of the Delaware Department of Transportation (DelDOT). The next phase of the laboratory testing, which included the standard Proctor compaction tests as well as the CBR tests, were conducted in the Pavement Materials laboratory at the University of Delaware. A brief description of the tests are outlined later in this chapter.

3.2 Geologic Material

Ten typical Delaware subgrade soils were used for the study. The subgrade materials consisted of 3 types of A-1-b soils, 5 types of A-2-4 soils, 1 type of A-4 soil, and 1 type of A-7-5 soil. Generally, all the soils were granular materials with the exception of one from New Castle County, Route 202, which contained some sizable amount of clay materials. Compaction characteristics were determined in the laboratory

using the standard AASHTO (T-99) method.   Table 3.1 presents a summary of the

unbound materials used for the experimental program.   In the table, the ten soils are

classified according to the designations of the AASHTO classification system.   These

soils are most commonly found as supporting layers of pavement systems in Delaware.

The basic properties of the tested soils are shown in Table 3.2.

Table 3.1: Identification and Classification of Materials for Experimental Program

County	I.D.	Location	Classification	Soil Type
New Castle	I	Way Road	A-1-b (0)	Gravely Sand
	II	Route 202	A-7-5 (18)	Clayey soil
	III	SR 1	A-1-b (0)	Sandy Soil
Kent	IV	Woodfield	A-2-4 (0)	Clayey Gravel
	V	Moore's Meadow	A-1-b (0)	Gravely Sand
Sussex	VI	Oak Crest Farms	A-4 (0)	Silty soil
	VII	Rehoboth Yatch/Country Club	A-2-4 (0)	Silty Sand
	VIII	SR 30	A-2-4 (0)	Silty Sand
	IX	Bayshore Route 22	A-2-4 (0)	Silty Sand
	X	Route 113	A-2-4 (0)	Silty Soil

Table 3.2: Summary of Basic Properties of Tested Granular Materials

County		New Castle			Kent		Sussex				
Sample I.D.		I	II	III	IV	V	VI	VII	VIII	IX	X
Classification		A-1-b (0)	A-7-5 (18)	A-1-b (0)	A-2-4 (0)	A-1-b(0)	A-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4(0)	A-2-4(0)
Sieve Analysis	Passing, #4 (%)	83.5	97.1	87.2	98.6	97.0	99.7	98.9	98.2	99.3	97.6
	Passing #10 (%)	69.2	91.8	82.5	95.5	91.6	99.3	92.7	92.7	95.3	96.5
	Passing, #40 (%)	34.2	84.2	47.3	71.3	33.6	87.6	55.3	64.7	95.3	74.7
	Passing, #200 (%)	11.5	72.1	18.9	30.4	14.0	36.0	20.1	23.6	12.9	16.2
	% Sand and Gravel	88.46	27.91	81.06	69.62	85.98	63.94	79.92	76.40	87.05	83.78
Liquid Limit (LL)		0	59	0	21	0	16	0	0	0	0
Plastic Limit (PL)		0	36	0	18	0	0	0	0	0	0
Plasticity Index (PI)		NP	23	NP	3	NP	NP	NP	NP	NP	NP

NP = nonplastic

3.3   Standard Proctor Compaction Test

Prior to performing the CBR test, a laboratory compaction test was conducted to obtain the moisture-density relationships for all the soil samples. The standard Proctor compaction test was used which is based on the compaction of the soil fraction passing No. 4 U.S. sieve according to AASHTO T-99 test designation. The equipment required for the compaction test is listed below.

- Compaction mold with base plate and collar
- No. 4 U.S. sieve
- Standard Proctor hammer (5.5 lb)
- Moisture cans
- Steel straight edge to smooth sample ends flush with mold
- Balance sensitive up to 0.01 lb
- Balance sensitive up to 0.1 g
- Large mixing pan and large spoon for dispensing soil
- Drying oven

The Proctor compaction mold is 4 inches in diameter and 4.584 inches in height. There is a base plate and an extension that can be attached to the top and bottom of the mold respectively. The inside of the mold is 1/30 ft^3.

About 10 lb of air-dry soil on which the compaction test was to be conducted was obtained for each of the ten samples. All the soil lumps were broken down as determined visually and sieved through the No. 4 U.S. sieve. The minus-4 materials were collected in a large pan and enough water was added and mixed thoroughly to bring the moisture

content to about 5%. Each soil was then compacted in the mold in three layers with 25 blows per layer, using the 5.5 lb compaction rammer dropping 1.0 ft onto the soil, to provide a nominal compaction energy of 12,400 ft-lbf/ft^3 for each test. The compacted sample was then broken down to the No. 4 sieve size and water-content samples taken. More water, about 2% moisture (by weight) was added to the soil and remixed thoroughly, and the process of compacting a mold repeated. This sequence was repeated a sufficient number of times that a curve of dry unit weight vs. water content can be drawn which has a zero slope (a maximum value) with sufficient points on either side of the maximum unit weight point to accurately define its location. The maximum ordinate value represents the maximum dry density and the water content at which this value occurs is termed the optimum moisture content.

3.4 Specimen Preparation for CBR Test

The primary factors affecting the stiffness characteristics of granular materials is water content, compaction method, and effort. Accordingly, any logical method of compaction that produces the desired dry density is suitable for specimen preparation. A dynamic compaction method by hand rammer using a specified compactive effort was used. All the test samples were prepared in a sequence of mixing and compacting. All mixes were prepared in an aluminium pan that provided enough space for mixing and compacting two CBR molds (approximately 30 lb of 0.75-inches maximum-size material) of soil at a time. A measured amount of potable water was added in small doses during mixing. The amount of water to be added was determined from the optimum water content of the samples as given by the moisture-density curves. Mixing was done

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using a large spoon and this continued until a uniform mix was obtained. Next, the specimens were made up as described in Section 3.6.

3.5   CBR Test Apparatus

The apparatus that was used for CBR testing consists of the following:

- A Humboldt compression machine model H-4454.100 with a capacity of at least 10000 lbf (44.5kN) and equipped with a movable base capable of traveling at a uniform rate of 0.05 in/min. for use in forcing the penetration piston into the specimen.

- CBR equipment consisting of 6.026 in diam. x 7.018 in height CBR compaction mold with collar, and spacer disc 5.938 in diam. x 2.416 in height.

- A 10.0 lbf (4.5 kg) compaction rammer as specified by ASTM Test Method D 1557.

- An expansion-measuring apparatus with an adjustable metal stem and perforated metal plate with dial gauge reading to 0.001 in.

- Annular and slotted surcharge weights each having masses of 2.27±0.02kg.

3.6   Test Procedure

Ten sets of granular materials each weighing approximately 14 kg of 0.75 in (19 mm) maximum size material were prepared to compact two CBR molds of soil at the optimum moisture content as determined using the standard AASHTO (T-99) compaction test method. After curing the soil for a more uniform water distribution, a controlled compaction test was conducted for the soil according to Method D of ASTM D 1557.

This required five layers of soil to be compacted in the mold at 56 blows with each lift of material using a 10 lb hammer. For each test the first mold was compacted for immediate penetration testing and the second mold soaked for a period of 96 hours with a surcharge approximately 5.72 kg. The surcharge simulates the effects of the thickness of road construction overlying the layer being tested. Soaking of test specimens serves as a precaution to allow for moisture content increase in the soil due to flooding or elevation of the water table. The test on the soaked sample provides:

- Information concerning expected soil expansion beneath the pavement when the soil becomes saturated.

- An indication of strength loss from field saturation.

Penetration testing was accomplished in the compression machine using a strain rate of approximately 0.05 in/min. Readings of load versus penetration were taken at each 0.02 in of penetration to include the value of 0.2 in, and then at each 0.1 inch increment thereafter until the total penetration was 0.5 in. The results were then plotted as illustrated in Appendices A, B and C, after which the CBR values were obtained for each specimen.

Chapter 4

DATA ANALYSIS AND INTERPRETATION

4.1   Test Results on Granular Materials

The experimental investigation that provided data for developing empirical relationships between CBR and basic soil properties included basic soil testing, CBR and compaction tests on ten different types of soils commonly found in Delaware. Basic soil tests such as sieve analysis, the Atterberg test, and the standard Proctor tests were conducted on each type of soil. These tests provide the necessary physical parameters for the statistical analysis. Proctor tests were conducted according to AASHTO (T-99). Table 4.1 presents moisture content and dry density levels obtained for each soil type as a result of standard Proctor tests. Levels A, B, and C represent dry-of-optimum points on a standard Proctor compaction curve. Level D represents the optimum point and Levels E and F represent wet-of-optimum on the Proctor test curve. The optimum moisture points under level D were used as targets when compacting specimens for the bearing ratio tests. The results of the Proctor test compaction curves for the various counties are shown in Appendices D, E, and F.

Table 4.1: Moisture Content and Dry Density Levels

Moisture Levels		A	B	C	D	E	F
I	Moisture Content:	7.21	8.40	9.61	10.0	11.48	12.89
	Dry Density:	117.53	121.49	125.08	125.40	122.44	118.79
II	Moisture Content:	18.12	19.76	20.90	22.0	23.42	24.80
	Dry Density:	88.64	93.19	97.02	98.20	95.77	92.50
III	Moisture Content:	8.33	9.10	10.18	10.40	12.34	13.92
	Dry Density:	120.74	123.0	125.25	125.5	122.04	119.30
.IV.	Moisture Content:	10.74	11.30	12.18	12.80	13.30	15.25
	Dry Density:	118.66	119.11	119.90	120.50	120.21	116.10
V	Moisture Content:	5.71	7.04	7.89	9.20	11.25	13.18
	Dry Density:	117.49	122.18	125.41	128.03	124.58	117.95
VI	Moisture Content:	4.35	5.20	6.73	7.20	8.22	10.74
	Dry Density:	118.74	120.70	123.4	123.70	123.08	120.82
VII	Moisture Content:	4.56	6.28	7.52	8.20	9.34	11.52
	Dry Density:	120.22	125.89	131.14	132.25	130.05	124.28
VIII	Moisture Content:	6.20	7.41	8.61	9.40	10.19	11.83
	Dry Density:	120.90	125.80	130.38	131.0	130.41	125.28
IX	Moisture Content:	6.18	7.80	9.76	11.50	12.01	15.37
	Dry Density:	114.71	116.33	118.35	120.40	119.99	114.41
X	Moisture Content:	7.97	8.57	9.88	10.30	13.17	15.05
	Dry Density:	114.75	118.0	120.68	121.0	117.43	114.73

Note: Dry Density in lb/ft^3

The CBR test was conducted on each soil type at the optimum moisture content (OMC) level. Specimens for the CBR test were compacted according to ASTM D 1557 (Methods B & D). After the CBR test was completed, the CBR values were obtained by computing the penetration stress in kilopascals, and plotting a curve of penetration resistance (stress) vs. penetration for both the freshly compacted and the soaked samples. Appendices A, B, and C illustrates the stress-penetration curves for the samples in the various Counties. The curves for both samples (soaked and unsoaked) were plotted on the same graph and clearly identified so that one may readily observe the effect of soaking the sample. The penetration resistance for 0.10 and 0.20 inches were obtained from the curve and the corresponding two CBR numbers computed by dividing the penetration tresses by the standard stresses of 1000 psi (6,900 kPa) and 1500 psi

(10,300kPa) respectively, and multiplying by 100. The desired CBR value was chosen as the one corresponding to the greater of the computed values at 0.10 inches and 0.20 inches penetration. Typical CBR values range from less than 5 for soft clays up to 80 for dense sandy gravel [Robert Day 2001].

The stress-penetration response in the bearing ratio tests (unsoaked) on the granular materials with a maximum penetration of 0.5 inches is shown in Figure 4.1. The deformation at each point is the average penetration during the cycle. The results show that the CBR values generally increase with increase in penetration of the plunger (piston).
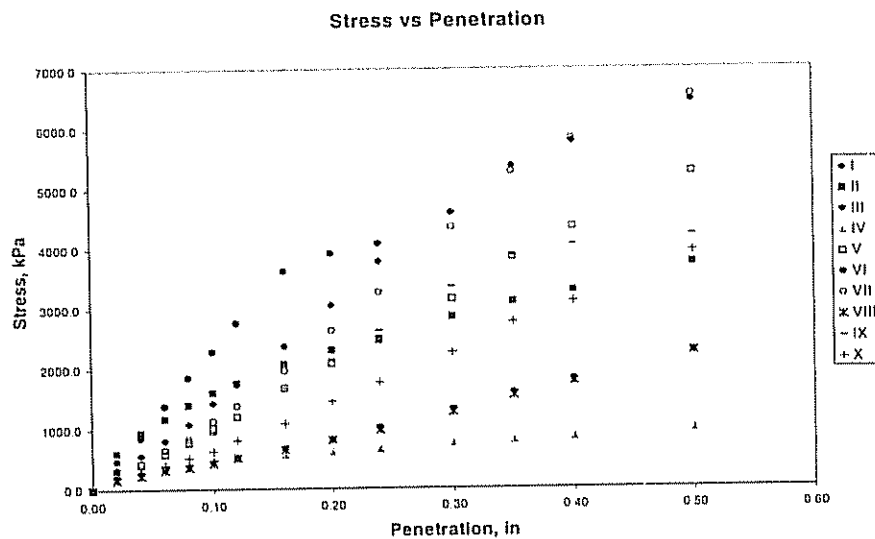
Figure 4.1: Plot of Stress versus Penetration for CBR test data (Unsoaked)

CBR vs Penetration (Unsoaked)
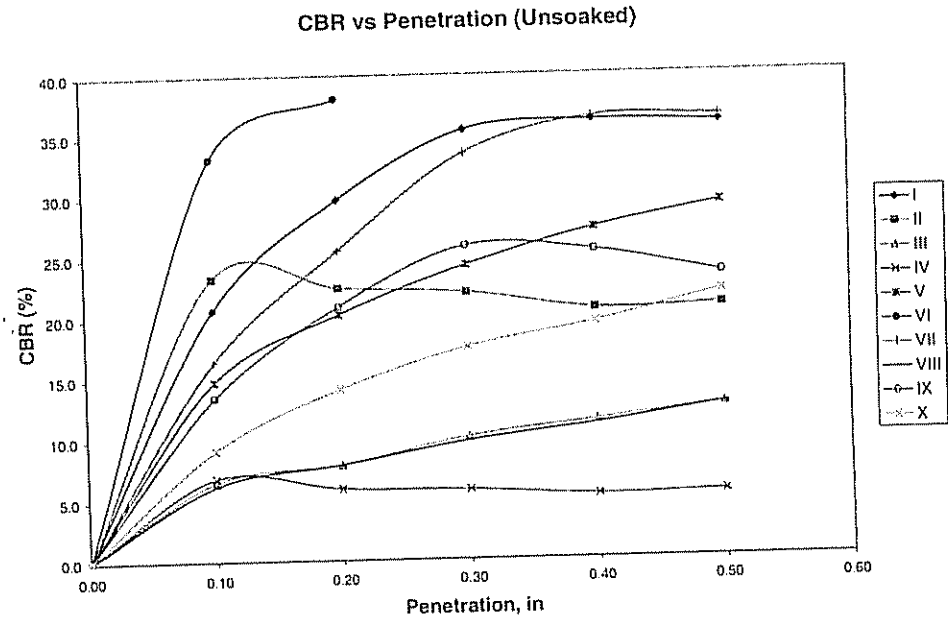
Figure 4.2: Relationship between CBR and Penetration

Sample VI produced the highest CBR value, but however the stress-penetration curve collapsed after a penetration of 0.20 in. This can be considered to be the boundary of the soil strength distinguishing between soft and adequate support. This value is referred to as the threshold strength. The threshold strength could be seen to be rather different for the different soils, as illustrated in Figure. 4.2 above. The test results of the CBR and the standard Proctor compaction tests are shown in Table 4.2.

Table 4.2: Summary of laboratory bearing ratio test results

County		New Castle			Kent		Sussex				
Sample I.D.		I	II	III	IV	V	VI	VII	VIII	IX	X
Classification		A-1-b (0)	A-7-5 (18)	A-1-b(0)	A-2-4 (0)	A-1-b (0)	A-4 (0)	A-2-4 (0)	A-2-4(0)	A-2-4(0)	A-2-4(0
Compaction Test	w_{opt} (%)	10	22	10.4	12.8	9.2	7.2	8.2	9.4	11.5	10.3
	$\gamma_{d(max)}$ (lb/ft³)	125.4	98.2	125.5	120.5	128	123.7	132.3	131	120.4	121
Bearing Ratio Test	w_U (%)	10.23	22.45	11.20	12.79	9.06	5.74	7.29	9.4	10.91	10.75
	$CBR_{(U)}$	29.7	23.3	7.9	6.9	20.2	38.1	25.5	7.9	11.05	11.15
	w_S (%)	10.25	28.15	11.7	13.73	10.89	9.59	7.78	10.20	11.05	11.0
	$CBR_{(S)}$	28.6	6.2	6.4	4.6	12.8	18.5	23.8	6.8	20.7	11.0
	γ_{wet} (lb/ft³)	147.17	123.76	144.11	141.24	147.30	134.37	149.67	145.92	138.18	140.12
	γ_d (lb/ft³)	133.51	101.07	129.63	125.51	135.06	127.05	139.49	133.37	124.59	126.06
	% water absorbed	0.41	5.0	0.21	0.45	0.64	4.73	0.61	0.11	0.44	0.55
	% reduction in CBR	3.7	73.4	19	33.3	36.68	51.4	6.7	13.9	1	22

w_{opt} = optimum moisture content

γ_{wet} = Wet Density

γ_d = Dry Density

$\gamma_{d(max)}$ = Maximum Dry Density

w_U = Moisture content of test specimen (Unsoaked)

w_S = Moisture content of test specimen (Soaked)

$CBR_{(U)}$ = California bearing ratio (Unsoaked)

$CBR_{(S)}$ = California bearing ratio (Soaked)

From Table 4.2 above, it was observed that the optimum moisture for the range of soils tested were generally within 10±2.8% with the exception of sample II which had an optimum moisture of 22%. This material being a clayey gravel has a sizable amount of clay minerals present and thus able to form a minisci, which enables the particles to attract water to their surfaces and this also goes to explain why it has such a high liquid limit. From the moisture-density relationships, the materials exhibiting slightly higher maximum dry densities have lower optimum moisture contents. As expected, sample II

which had the lowest maximum dry density recorded a very high optimum moisture. The CBR values obtained for the unsoaked samples varied from 6.9 at 12.8 percent moisture to 38.1 at 5.74 percent moisture. The CBR values obtained after soaking were generally lower than the unsoaked CBR values with respect to the compaction moisture content. Since both results are very sensitive to the compaction moisture content, a slight increase in the moisture content would have resulted in a very small difference between the soaked and unsoaked CBR values, whereas a slight decrease in moisture content in relation to the optimum would have resulted in a large difference between the soaked and unsoaked CBR values. Obviously, when the moisture content is greater than the OMC, the increasing moisture content has a greater influence on the decreasing of the CBR values. This may be because the soaked specimens were all compacted above the OMC which produces smaller dry densities than the unsoaked specimens that were compacted nearly at their respective OMC. Also the decrease in CBR values could be attributed to the decrease in matric suction with increasing moisture content. Both factors are detrimental to the strength of the materials.

Analysis of the soaked samples reveals that the recorded percentage water absorbed over the 4-day period were generally found to be less than 1 percent with the exception of samples II and VI which recorded 5 and 4.73 percent respectively. These are considered to be very significant, and it shows that an increase in the field moisture content could soften the fine-grained soils and therefore lead to a loss in strength than originally measured when the samples becomes saturated. This assertion is confirmed from the percentage reduction in CBR values obtained for samples II and VI which

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recorded reductions of 73.4 and 51.4 percent respectively. This means that the undrained

shear strength, and hence CBR, becomes progressively less as the water content of the

soil increases above optimum moisture content. Samples I and IX recorded the lowest

percentage reduction in CBR as 3.7 and 1 percent respectively. This indicates that field

saturation of both materials would result in negligible strength loss and as such they

could be considered for use as drainage blankets in a pavement structure for purposes of

improving subsurface drainage. Generally, the materials that were utilized in the testing

program did not produce any observed swellings due to significantly lower amounts of

clay and silt fractions present in the various samples.

4.2 Indirect Procedures Determining Resilient Modulus using CBR Test

4.2.1 Prediction of CBR from Standard Test Data

Variables (physical properties) that were considered for the development of the

model include:

- Optimum moisture content

- Maximum dry density

- Plasticity Index

- Liquid Limit

- % passing No. 40 sieve

- % sand and gravel

- Normalized moisture content, and

- Normalized dry density

The normalized moisture content is defined as the ratio of the moisture content of the test specimen to the optimum moisture content. It will be less than 1 for moisture contents that are less than optimum, equal to one for moisture contents at optimum, and greater than one for moisture contents that are higher than optimum. Similarly, the normalized dry density is defined as the ratio of the dry density at a particular moisture content to the optimum dry density. Relationships were developed for all materials tested between the CBR and each of these variables to identify any possible trends that might be useful in making a reasonable prediction of the CBR value. This was done for the unbound materials representing the different counties, and also on the range of materials used for the State for both the soaked and unsoaked test specimens (see Appendix G). For instance, for the range of materials tested for Sussex County, the maximum dry densities varied from about 120 to 133 lb/ft^3. There was also no significant change in the CBR values for most of the samples from the same set of materials in the County, with the exception of sample VI which had a reduction in CBR of about 51%. This could be attributed to the fact that the test sample contained a sizeable amount of fines and had a considerably high swell potential which makes it capable of absorbing moisture in a saturated condition and thereby leading to a reduction in strength under loading. This material would appear to be initially stiff, but its performance under vehicular loading in a saturated condition might lead to a rapid deterioration of the pavement. Similarly, from the plot of CBR versus normalized moisture content for all the unsoaked samples, it could be seen that four of the samples were compacted above the targeted moisture levels, four were below their respective optimum moisture contents, and the remaining

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two were compacted approximately at their optimum moisture levels. These have been identified as possible areas that could be sources of discrepancies in the data.

The CBR data were reviewed in detail to identify anomalies or any potential errors in the data. The test results were then analyzed by performing a multiple regression using Stat Graphics statistical analysis system software relating the physical properties of the test specimen to the CBR used in the model equation, in which CBR was taken as the dependent variable. A backward stepwise selection procedure was adopted in which the independent variables were removed from the model one at a time starting with the variable with the least statistical significance. The next step was correlating the CBR with basic soil properties and providing the correlations in the form of empirical functions of density, moisture content, Atterberg limits, and other soil properties. Using the best subsets of data found in the analysis of the individual soil types, a general equation was sought that would encompass all soil types. Combining all the soil types to accomplish this objective did not yield reasonable results. It was decided to create two broad classes of soils based on the material type--granular materials and silt-clay materials. The cohesive soil (sample I.D. II) was identified as an outlier and was subsequently excluded from the analysis. Thus, only the granular subgrade soils were analyzed in this study. Regression analysis on the CBR correlations produced the following equations with a coefficient of determination, R^2 of 0.923 and 0.745 for both the unsoaked and soaked specimens respectively. A correlation matrix describing the relationship between the independent variables is illustrated in Table 4.3.

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The model can be written as:

$$CBR = CBR(PI, LL, \gamma_{d(max)}, w_{opt}, P_{No. 40}, \%sg)$$

$$CBR_{(U)} = 6.41208PI + 1.39473LL - 0.50419\gamma_{d(max)} - 10.40921w_{opt} + 0.14250P_{No. 40}$$
$$+ 2.12479\%sg \qquad (4.1)$$

$$(R^2 = 0.923, \; SE = 6.357)$$

$$CBR_{(S)} = 3.77699PI + 0.53895LL - 0.40215\gamma_{d(max)} - 5.49309w_{opt} - 0.11179P_{No. 40}$$
$$+ 1.36669\%sg \qquad (4.2)$$

$$(R^2 = 0.745, \; SE = 9.01)$$

where

w_{opt} = optimum moisture content,

$\gamma_{d(max)}$ = maximum dry density (lb/ft^3),

LL, PI = liquid limit and plastic limit, respectively,

$P_{No. 40}$ = percentage passing No. 40 sieve,

$\%_{sg}$ = percentage sand and gravel,

$CBR_{(U)}$ = California bearing ratio (Unsoaked), and

$CBR_{(S)}$ = California bearing ratio (Soaked).

Table 4.3: Correlation Matrix for Coefficients Estimates

	w_{opt}	$\gamma_{d(max)}$	LL	PI	$P_{No. 40}$	$\%sg$
w_{opt}	1.0000	0.5121	0.1749	-0.8247	-0.5322	-0.8340
$\gamma_{d(max)}$	0.5121	1.0000	-0.2314	-0.2965	-0.5884	-0.8860
LL	0.1749	-0.2314	1.0000	-0.5965	-0.4765	0.1372
PI	-0.8247	-0.2965	-0.5965	1.0000	0.5760	0.5773
$P_{No. 40}$	-0.5322	-0.5884	-0.4765	0.5760	1.0000	0.5398
$\%sg$	-0.8340	-0.8860	0.1372	0.5773	-0.8054	1.0000

From Table 4.3, a strong inverse correlation was observed between the percent sand and gravel and the maximum dry density. A fairly good correlation was also observed between the percent passing sieve No. 40 and the plasticity index. There were, however, no correlation between the liquid limit and the percent sand and gravel. From the above correlations, it appears that the CBR is mainly governed by moisture content, dry density, percent passing No. 40 sieve, percent gravel and sand, liquid limit, and plastic limit. This is a widely accepted occurrence, and in the present study, this occurrence proved to be the case again. Some deviations were detected in the results but there do not appear to be any major trends in the data that would cause the equations to be rejected. The resulting statistical model was used to back-predict the CBR for each of the soils in the test program. A comparison between the measured CBR results and predicted values is shown in Figure 4.3.
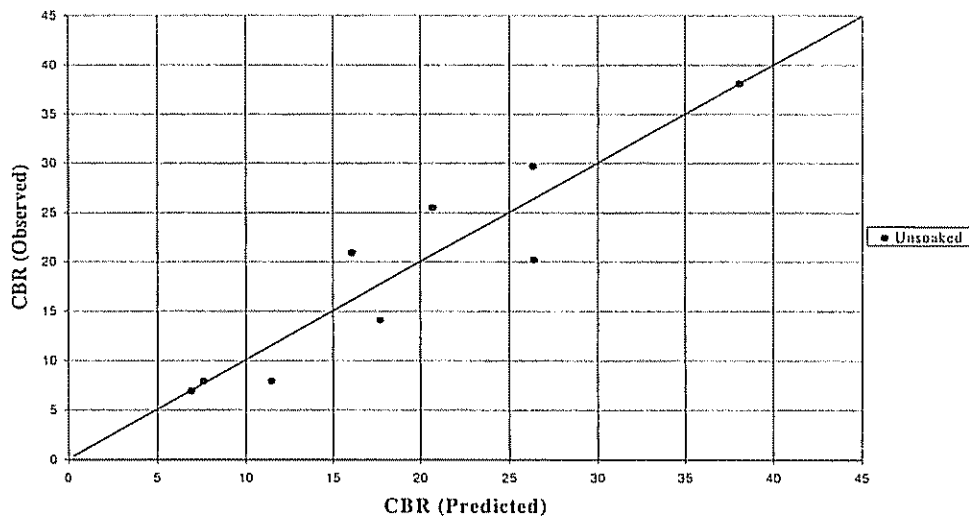
Figure 4.3: CBR test data and prediction (Unsoaked)

In view of the variations that might be associated with the sampling and testing of the pavement materials, the proposed model provides a satisfactory prediction of CBR and may be sufficient for pavement design procedures where only levels 2 and 3, mechanistic-empirical methods are required.

4.2.2   Validation of Proposed Model

To verify the reliability of the model developed, soil samples of a range of material types--clayey silts to fine to medium sands--were obtained by the Delaware Department of Transportation (DelDOT) and given to independent consultants for testing using the requisite AASHTO and ASTM test methods. In all, 15 materials were used and they were sampled from the New Castle County on Route 141 roadway improvements. For the limited range of soils tested, the model response was compared with the observed response. The actual values compared favorably with the predicted values from the general model developed. It was noted that the overall CBR behavior is well represented, particularly at $\gamma_{d(max)}$ ranges between 118 and 135 lb/ft^3. This indicates that for the soils investigated in this study, the errors in the approximate method are of the same order as the variability of measured CBR between specimens of the same soil. Possible anomalies in the results from the tested soils could be due to different testing procedures, and variability in physical properties of the tested soils. For most cases, the method yields a fairly good estimate of CBR from compaction tests, grain-size distribution, and index properties.

4.2.3   Correlation Consideration for Resilient Modulus

CBR is a measure of the stiffness or resistance to deformation of a particular material and it is often correlated with resilient modulus where resilient modulus is equal to a constant times the CBR. Resilient modulus values for soils must be developed to take advantage of the AASHTO Design guide, as well as the mechanistic procedures which rely solely on the resilient modulus. Since not all agencies have access to the necessary equipment to the resilient modulus test data required for immediate use in design projects, some general correlations have been defined under the National Cooperative Highway Research Program (NCHRP) Project 1-37A to be used to relate results from common tests to the resilient modulus of roadbed soils. An accepted approximate correlation is defined in equation 4.3.

$$M_R = 2550(CBR)^{0.64} \quad psi \tag{4.3}$$

This equation is, however, only applicable to Level 2 and 3 approaches to new and rehabilitation designs for the M-E methods as discussed in chapter 1. Typical CBR values range from about less than 5 to a value of around 80.

Chapter 5

SUMMARY AND CONCLUSIONS

5.1   Summary

The elastic or resilient modulus of pavement materials is an important material property in any mechanistically based design/analysis procedure for flexible pavements. In fact, the resilient modulus (M_R) is the material property required for the 1993 American Association of State Highway and Transportation Officials (AASHTO) Design Guide, which is an empirically based design procedure, and is the primary material input parameter for the 2002 AASHTO Mechanistic-Empirical Design Guide. With the introduction of the resilient modulus value into the AASHTO design process, transportation agencies have been encouraged to conduct appropriate testing to develop resilient modulus values for soils in their area. However, not all agencies have access to the necessary equipment to develop the resilient modulus data required for immediate use in design projects. In such cases, while specific resilient modulus data are developed, some general correlations can be used to relate results from common tests to the resilient modulus of the roadbed soils.

A procedure is described to relate the soil-index properties, grain size distribution, dry density and moisture content obtained from Proctor compaction tests to CBR. One

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broad class of soils--coarse-grained soils--was identified and created to yield reasonable results, and after several trial combinations a general model equation was developed for the soil class. A model is described and demonstrated for unbound pavement materials from selected locations throughout the state of Delaware. A second model which provides a general relationship for resilient modulus as a function of CBR has been adopted, based on recommendations by AASHTO for incorporation in mechanistic-empirical design procedures for levels 2 and 3 pavement design criteria. Both models are utilized to predict the resilient modulus test data for the range of soils tested, and shown to provide a good characterization of response for the soils investigated. Such relationships can be developed for other subgrade soils, and may be useful to agencies such as DelDOT that lack the capability for high-production repeated-load testing. The analysis presented suggests that the relationship between the soil properties and M_R depends somewhat on the predominant soil series of an area, and the methodologies presented could therefore be easily adapted by DelDOT to local conditions. This study, thus, provides an indirect estimate of resilient modulus and requires only data obtained from conventional Proctor compaction tests, grain size distribution, and index properties. These tests are easy to perform, inexpensive, and less time consuming. Additionally, low level of expertise is required to conduct such tests. Breaking the data into subgroups by material or soil type did improve on the regression statistics. Thus, the primary result from this study is that the resilient modulus can be reasonably predicted from the range of physical properties that are included in the data. As a larger data base of resilient moduli data is incorporated into the proposed model, refinements can be expected.

5.2   Conclusions

This study yields a valuable information about the physical properties and strength characteristics of unbound pavement materials in Delaware. The engineering parameters indicate that the granular materials will perform adequately in many highway applications as long as proper quality control is maintained. The following specific conclusions can be drawn:

- The multiple linear regression model developed in this study includes the most important factors that have influences on CBR and hence the M_R values. The models presented could be adopted for use in resilient modulus estimates for levels 2 and 3 M-E design procedures applicable to both new and rehabilitated pavement designs. Certainly, caution must be used when using these or any regression equations to ensure that the input information is within the inference space from which the equations were developed.

- CBR values are very sensitive to the compaction moisture content of the sample. The optimum moisture content has a great effect on the soil behavior. Moisture contents on the wet side of optimum is associated with reduced shear strengths as exhibited by the soaked CBR values, and this ultimately affects the desired resilient modulus estimates which might lead to overly conservative designs of pavement thicknesses with significant economic loss.

- Since the quality and constraints of the facilities and testing procedures vary, sometimes greatly, between different laboratories, the engineer should always use these equations with engineering judgment.

• Finally, the result of this research could be used to develop material databases for updating pavement design input values and built upon as information becomes available.

APPENDIX A

STRESS-PENETRATION RELATIONSHIPS FOR CBR TEST DATA

(NEW CASTLE COUNTY)

Sample I.D. I - Way Road
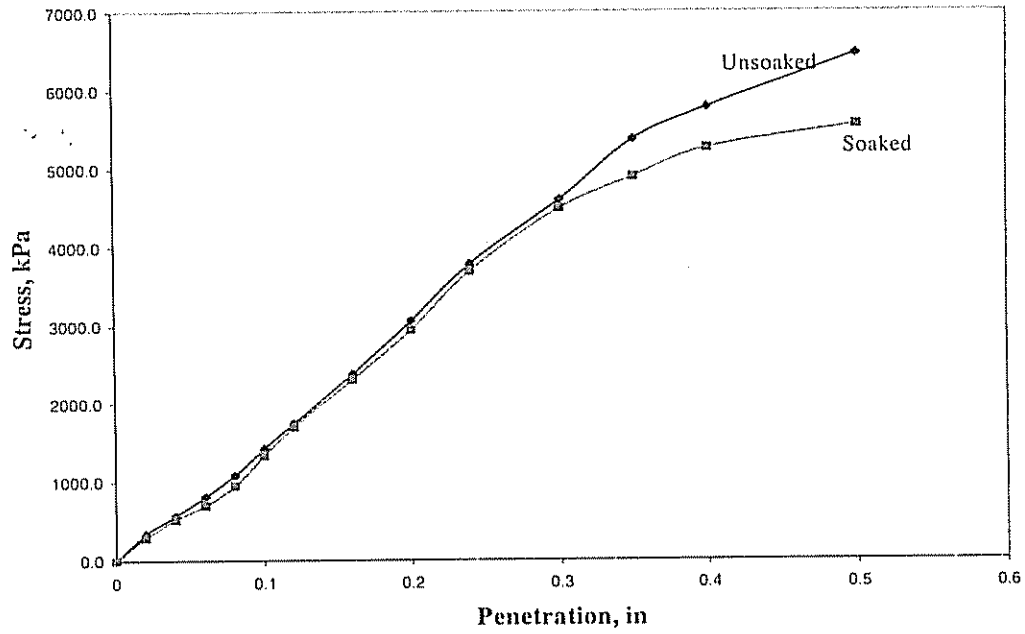
Sample I.D. II - Route 202
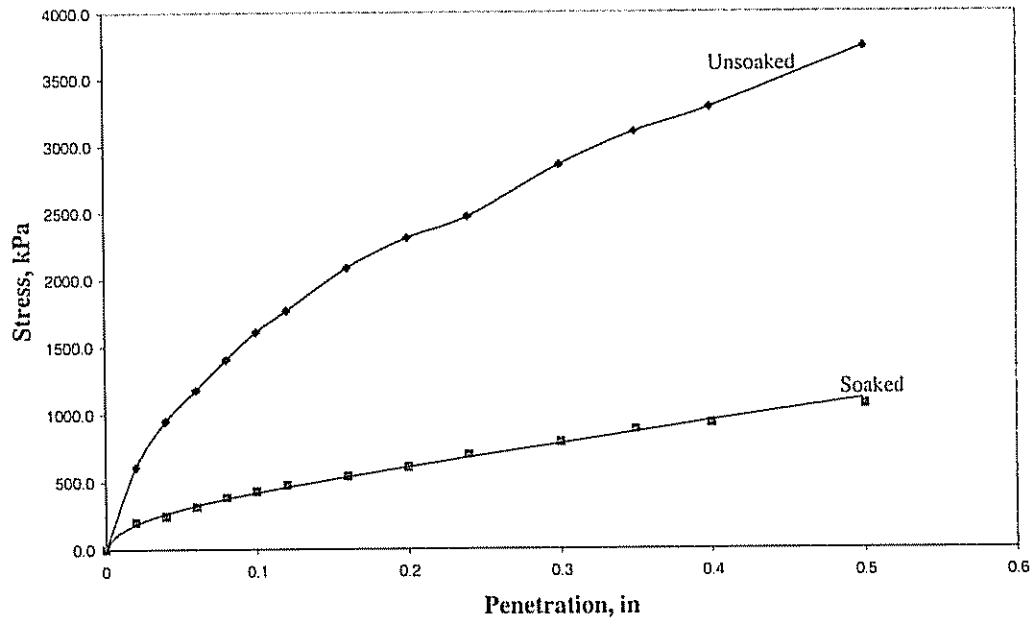
Sample I.D. III - SR 1
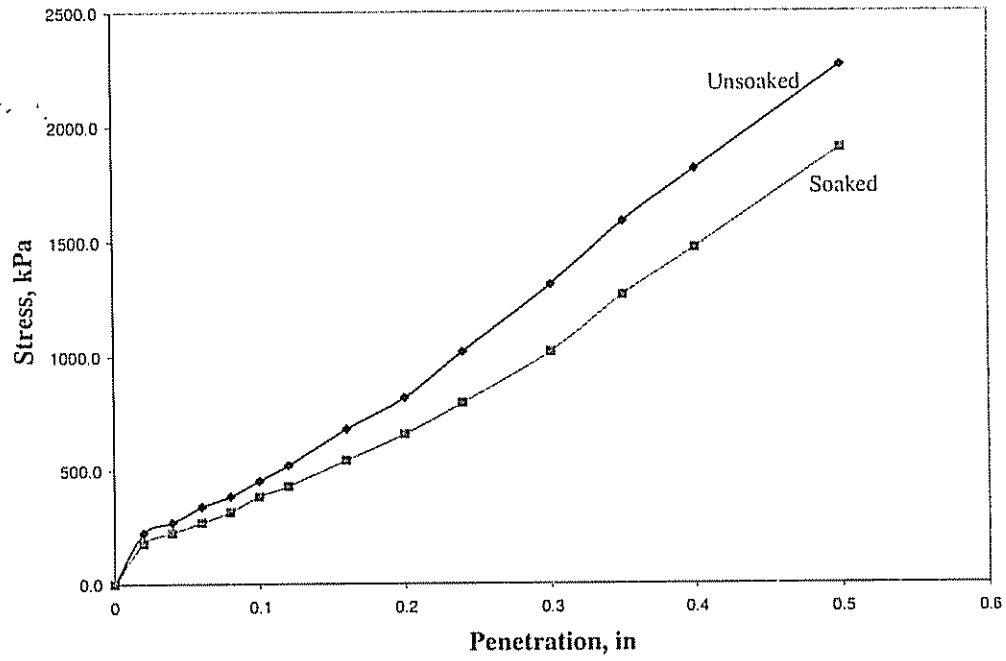
APPENDIX B

STRESS-PENETRATION RELATIONSHIPS FOR CBR TEST DATA

(KENT COUNTY)

Sample I.D. IV - Woodfield
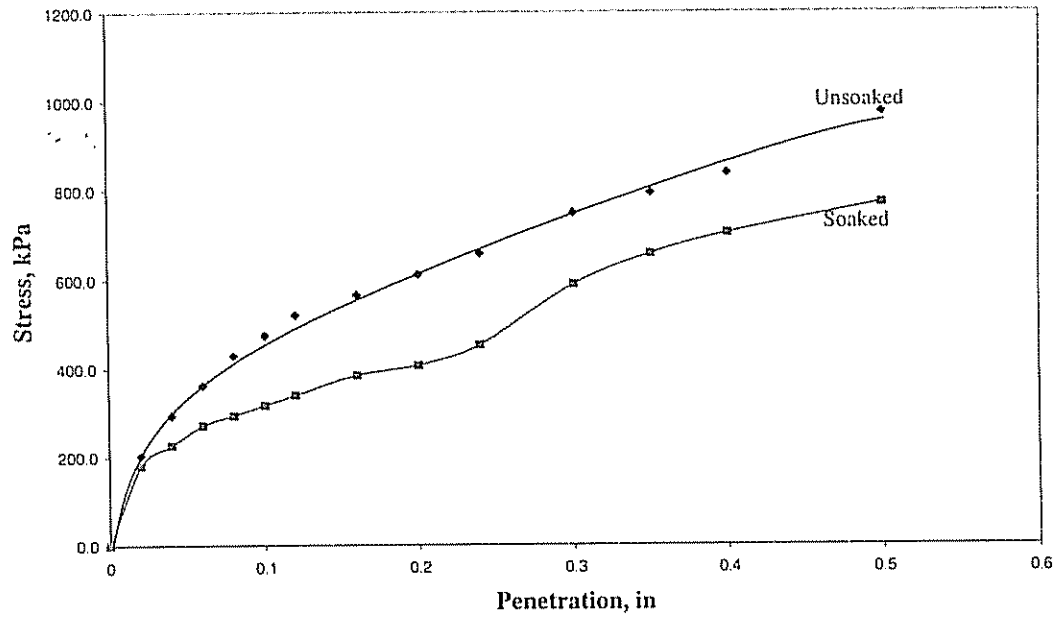
Sample I.D. V - Moore's Meadow

113
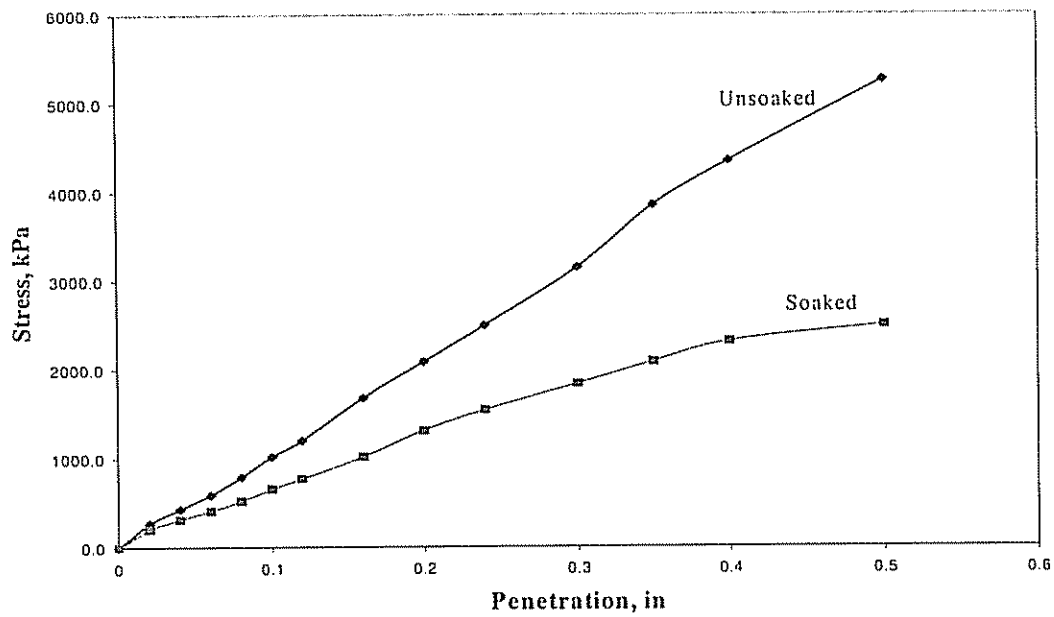
APPENDIX C

STRESS-PENETRATION RELATIONSHIPS FOR CBR TEST DATA

(SUSSEX COUNTY)

Sample I.D. VI - Oak Crest Farm
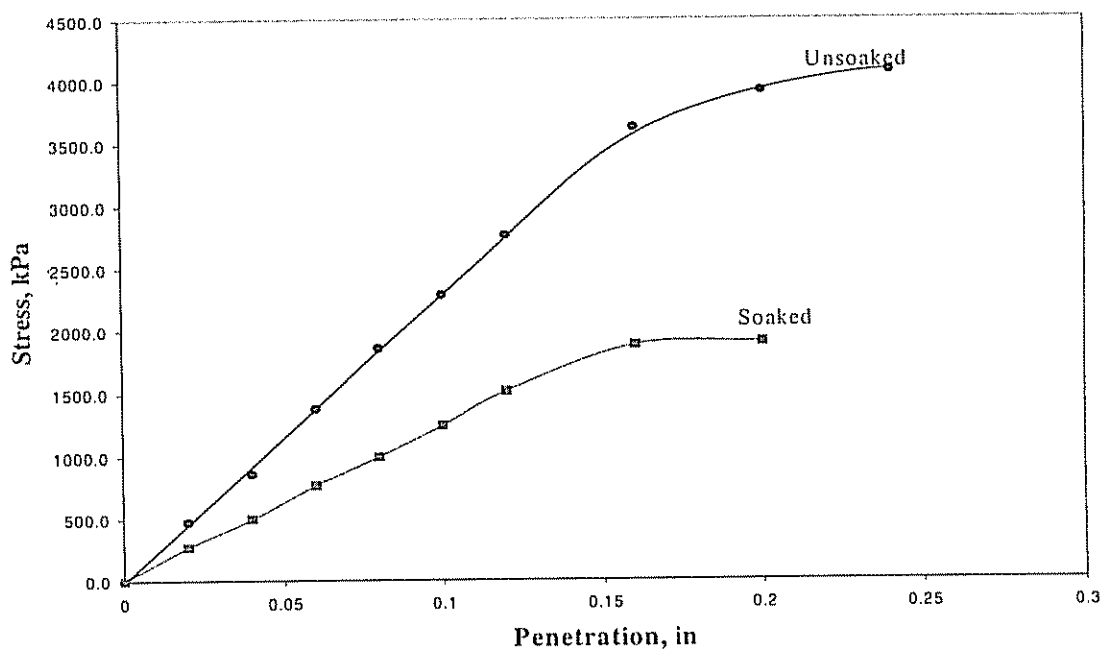
Sample I.D. VII - Rehoboth Beach/Country Club
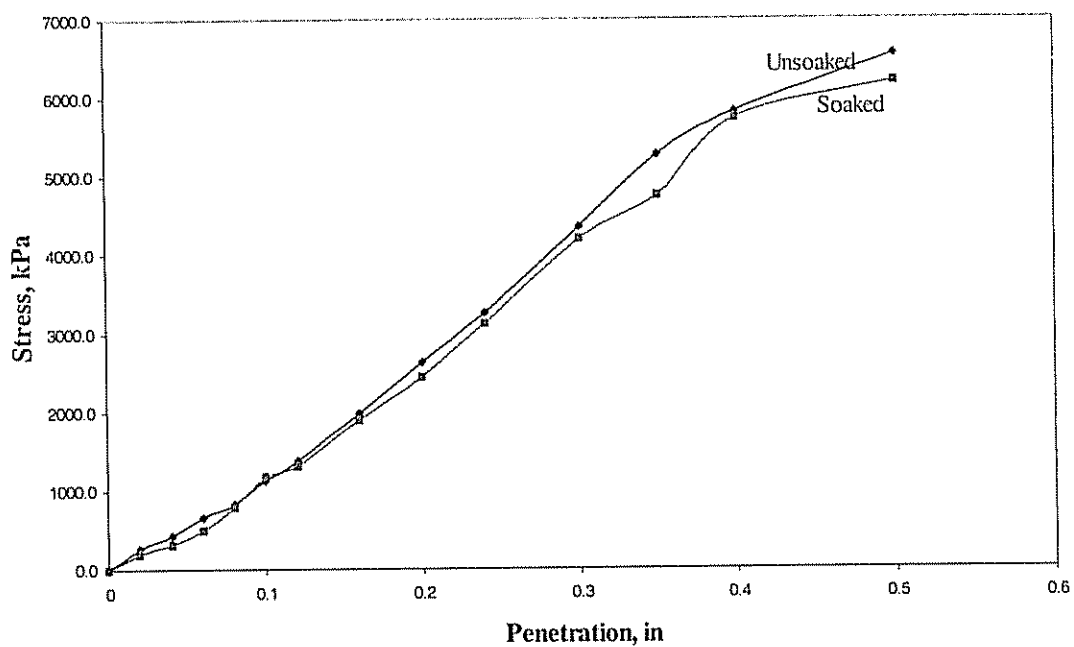
Sample I.D. VIII - SR 30

Unsoaked

Soaked
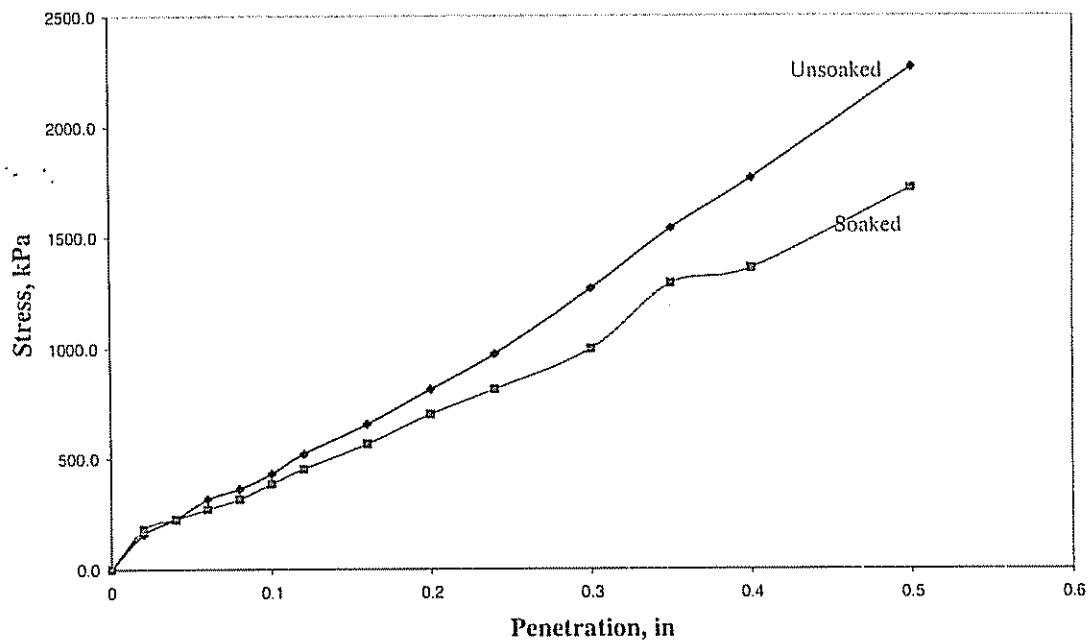
Sample I.D. IX- Bayshore (Route 22)

Unsoaked
Soaked

116
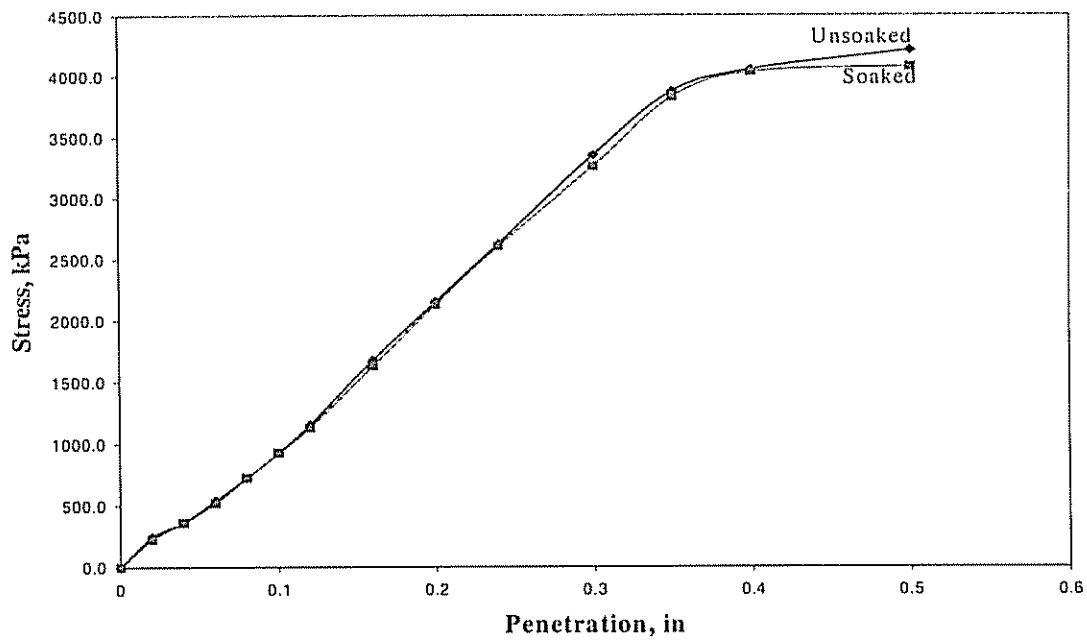
Sample I.D. X - Route 113
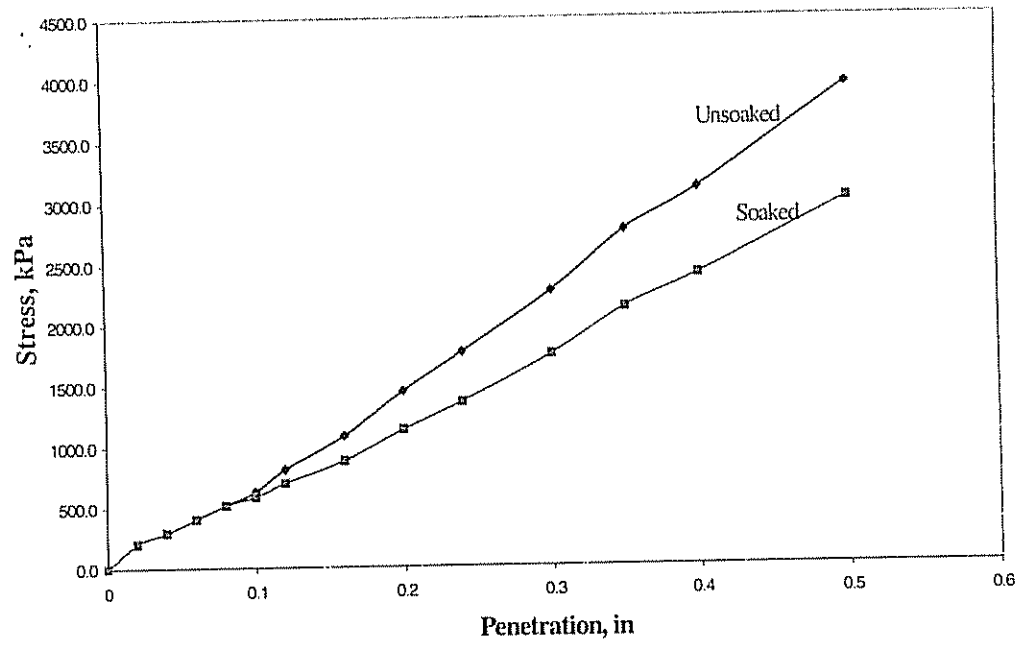
APPENDIX D

STANDARD PROCTOR COMPACTION CURVES

(NEW CASTLE COUNTY)

Sample I - Way Road

Optimum moisture = 10.0%

Max. dry unit weight = 125.40 lb/ft³
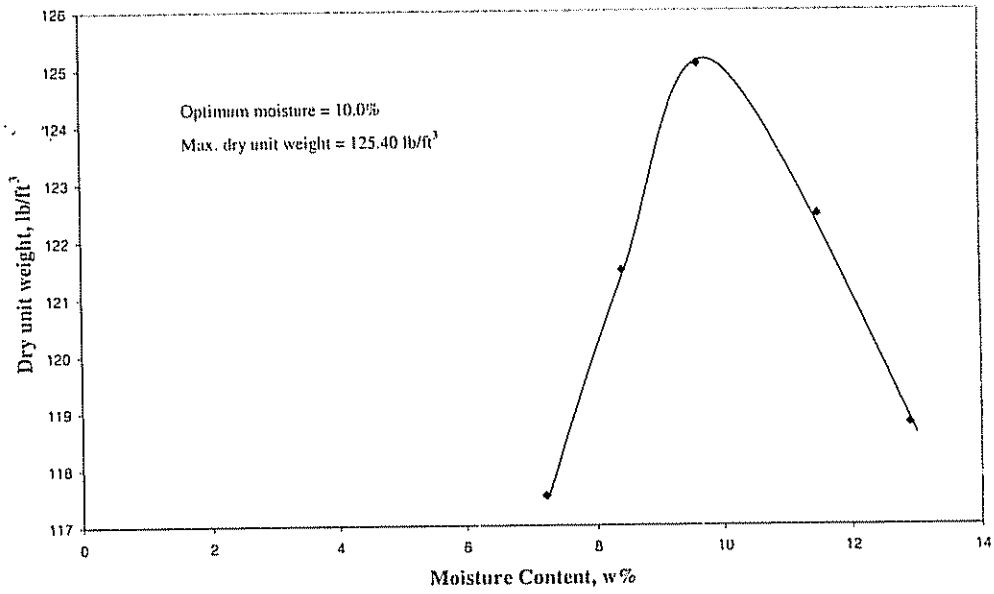
Sample II - Route 202 (North End)

Optimum moisture = 22.0%

Max. dry unit weight = 98.20 lb/ft³

Sample III - SR 1

Optimum moisture = 10.40%

Max. dry unit weight = 125.50 lb/ft³

(y-axis) Dry unit weight, lb/ft³

(x-axis) Moisture Content, w %
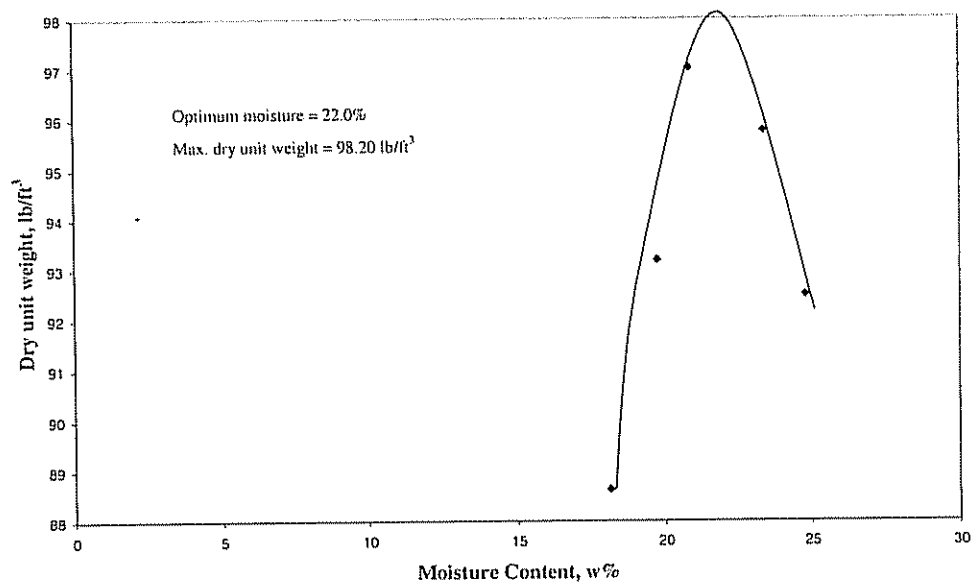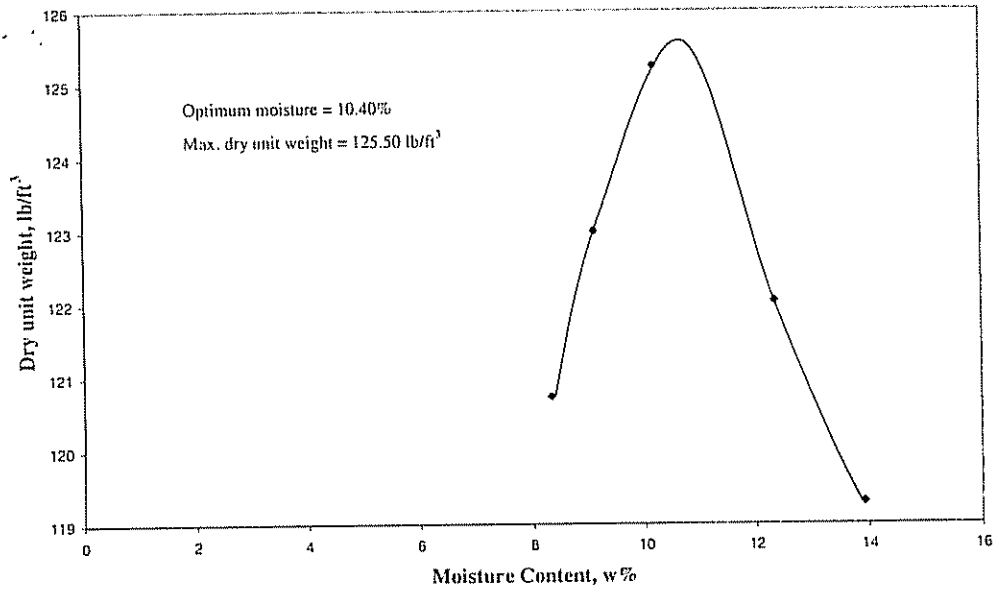
APPENDIX E

STANDARD PROCTOR COMPACTION CURVES

(KENT COUNTY)

Sample IV - Woodfield

Optimum moisture = 12.80%

Max. dry unit weight = 120.50 lb/ft^3

Sample V - Moore's Meadow

Optimum moisture = 9.20%

Max. dry unit weight = 128.03 lb/ft^3
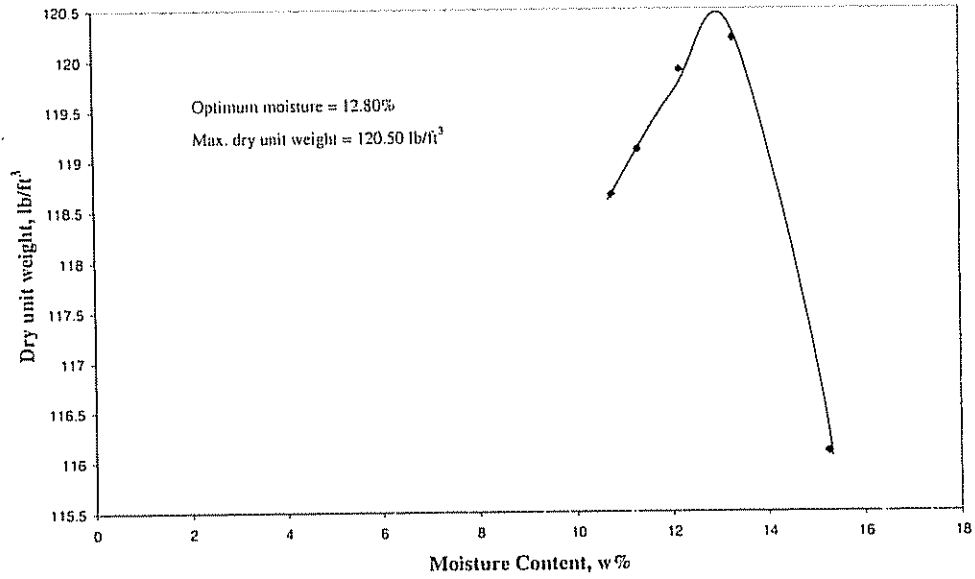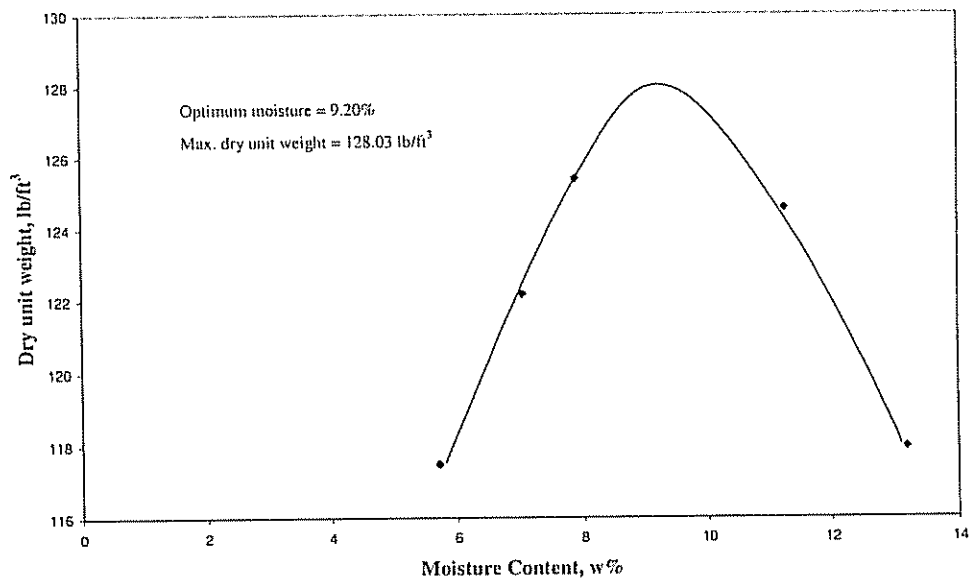
APPENDIX F

STANDARD PROCTOR COMPACTION CURVES

(SUSSEX COUNTY)

Sample VI - Oak Crest Farms

Optimum moisture = 7.20%

Max. dry unit weight = 123.70 lb/ft³
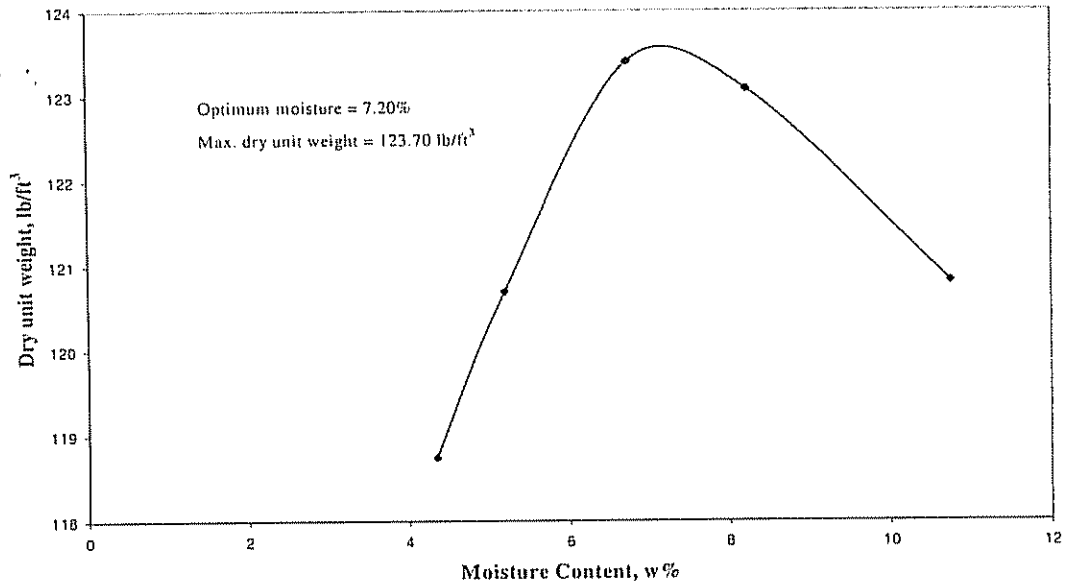
Sample VII - Rehoboth Yatch/Country Club

Optimum moisture = 8.2%

Max. dry unit weight = 132.25 lb/ft³

124

Sample VIII - SR 30

Optimum moisture = 9.40%

Max. dry unit weight = 131.0 lb/ft³
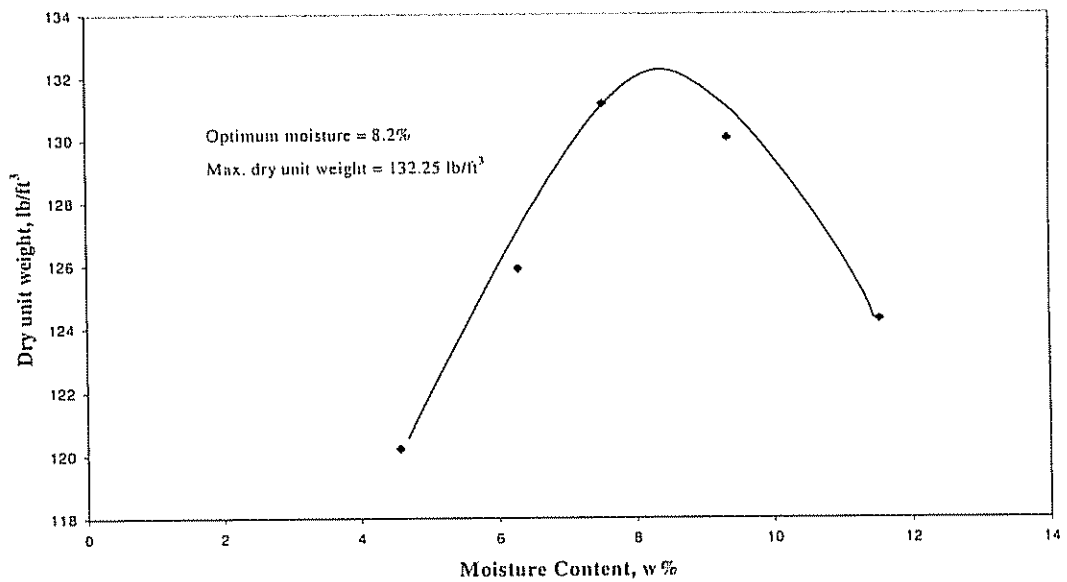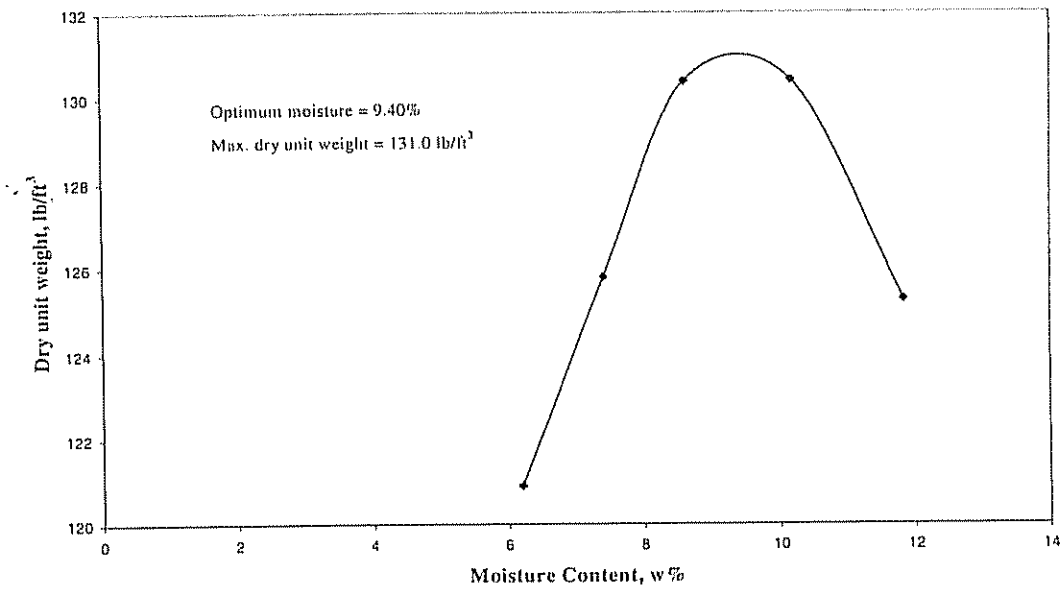
Sample IX - Bayshore(Route 22)

Optimum moisture = 11.50%

Max. dry unit weight = 120.40 lb/ft³

125

Sample X - Route 113

Optimum moisture = 10.30%
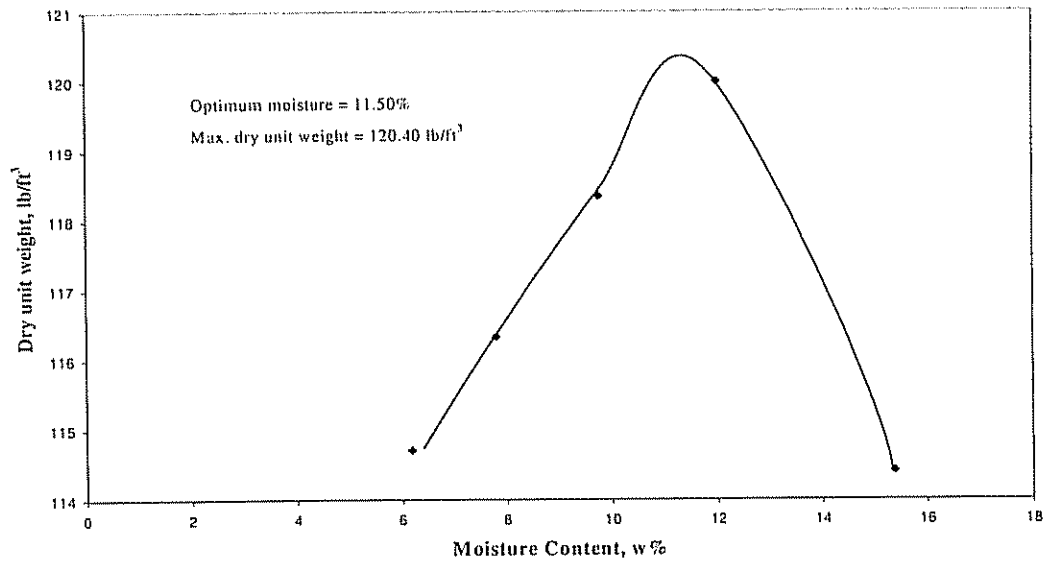
Max. dry unit weight = 121.0 lb/ft^3
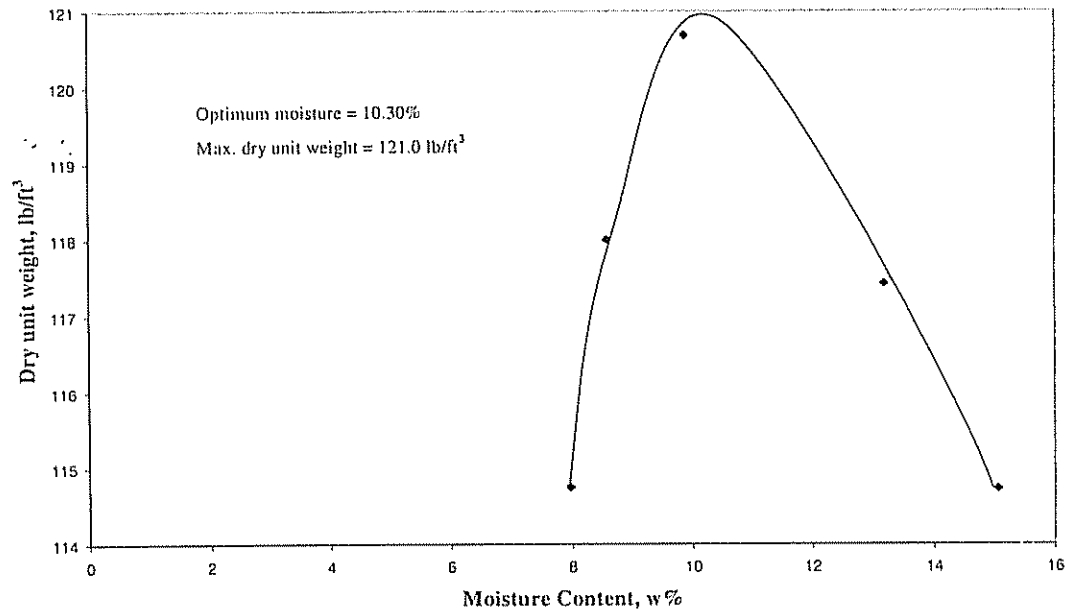
APPENDIX G

RELATIONSHIP BETWEEN CBR AND PHYSICAL PROPERTIES OF

UNBOUND MATERIALS

CBR vs OMC (New Castle County)
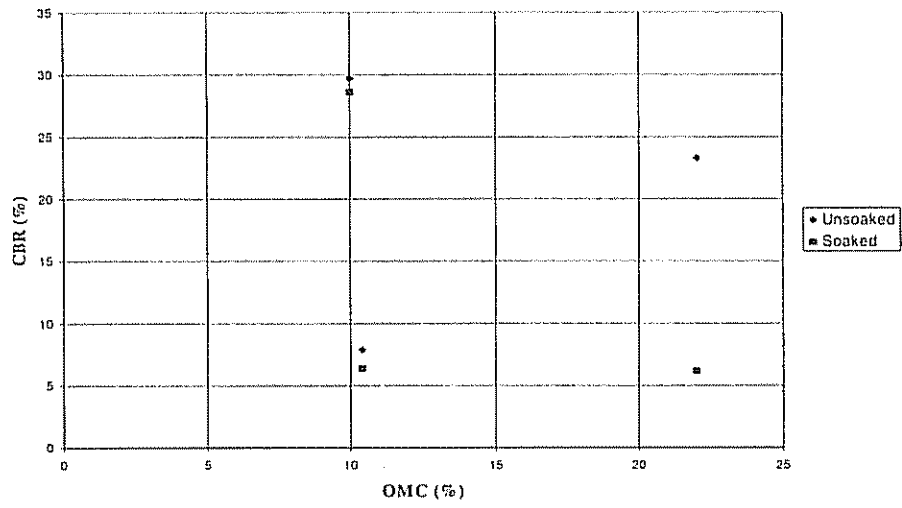
CBR vs OMC (Kent County)

128
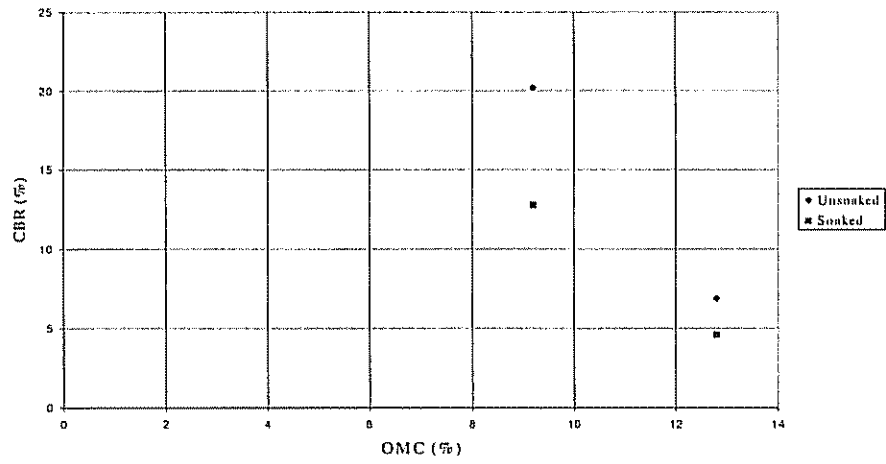
CBR vs OMC (Sussex County)
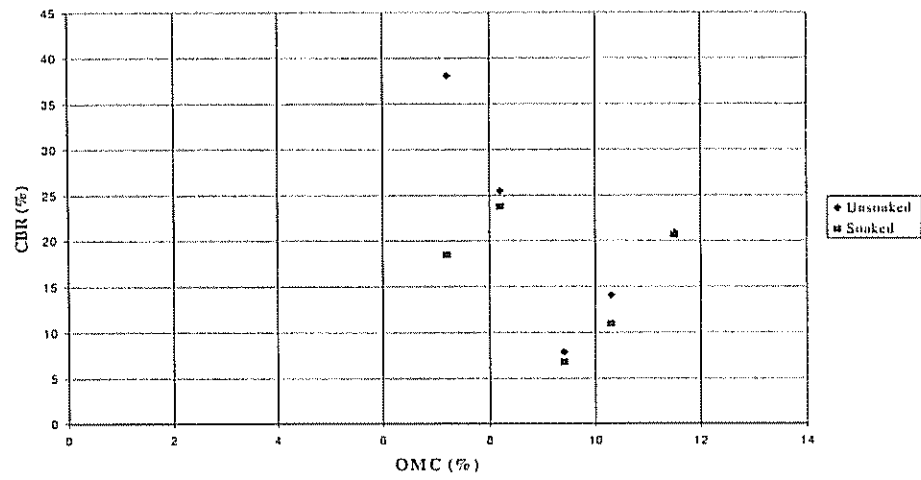
CBR vs MDD (New Castle County)

129
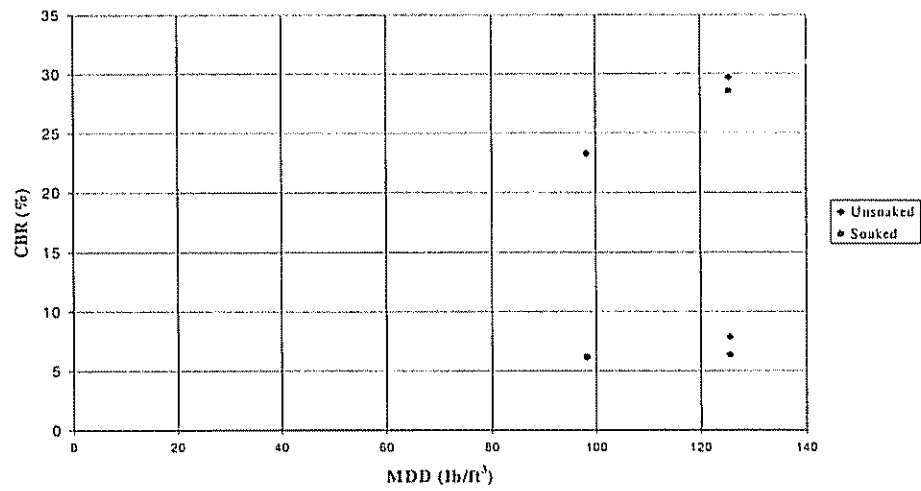
CBR vs MDD (Kent County)
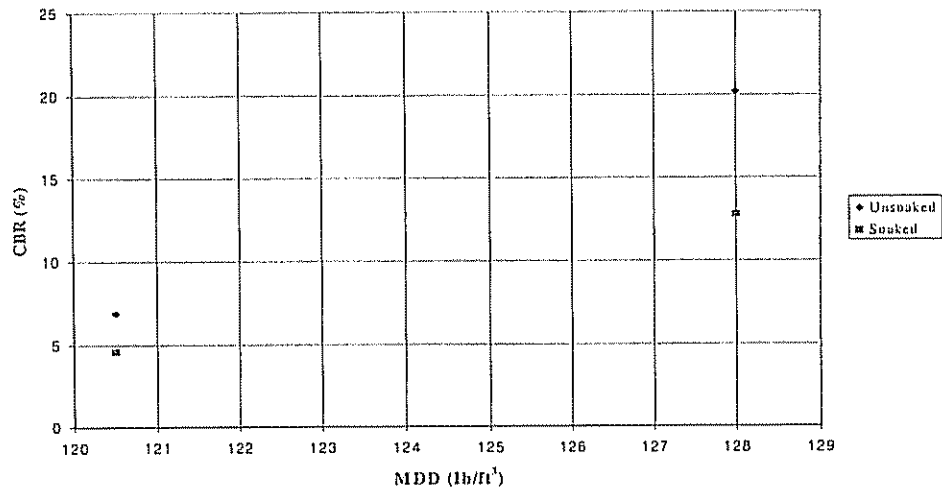
CBR vs MDD (Sussex County)

130
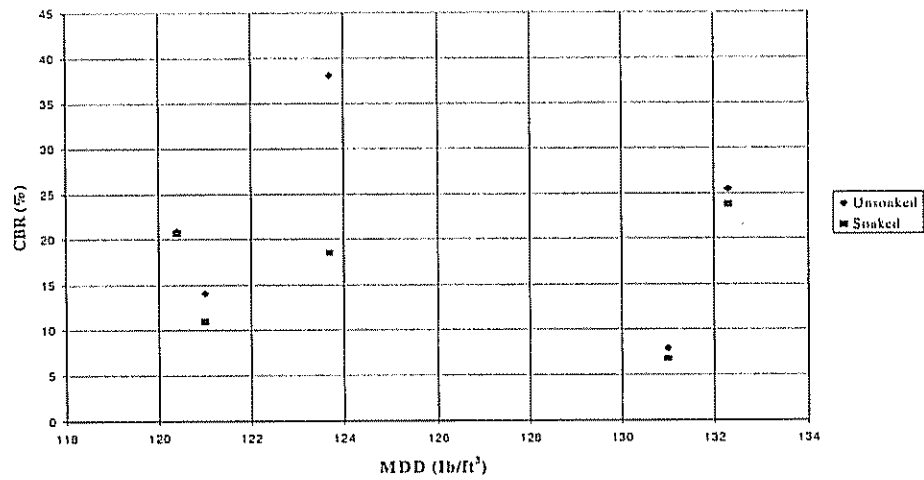
CBR vs % Passing #40 (New Castle County)
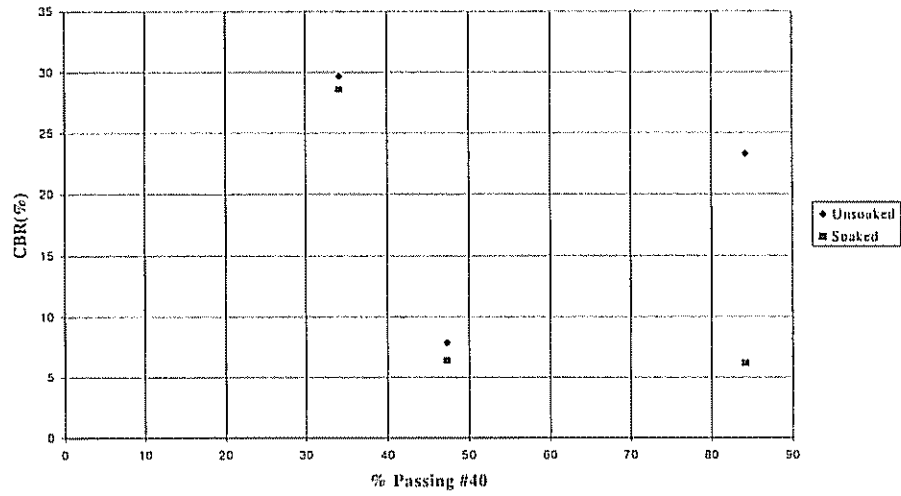
CBR vs % Passing #40 (Kent County)

131
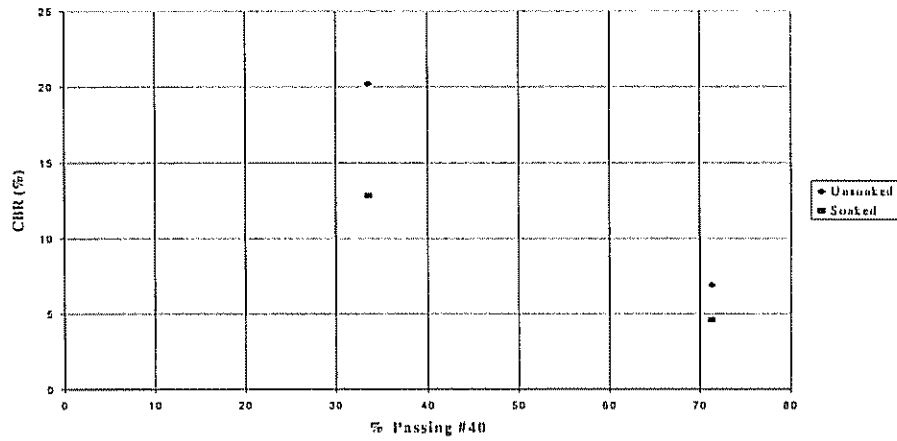
CBR vs % Passing #40 (Sussex County)
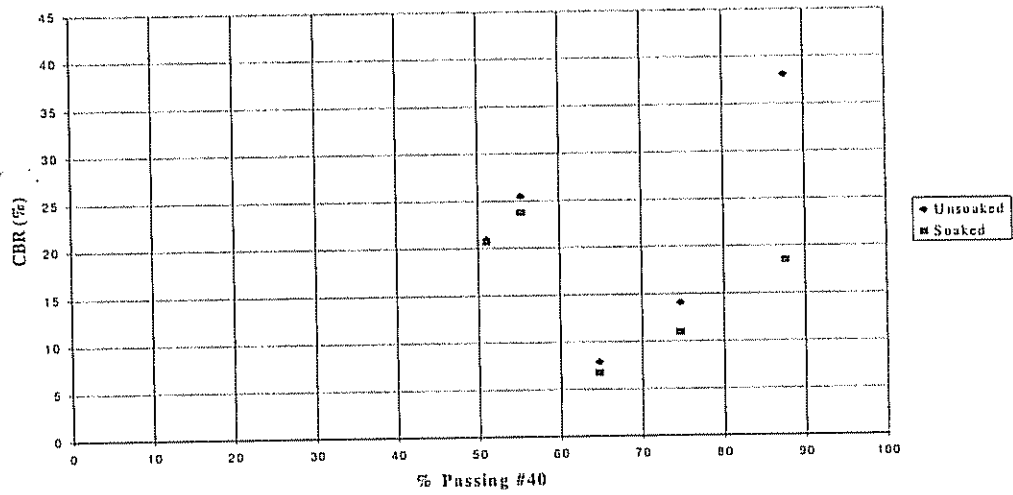
CBR vs. % Sand and Gravel (New Castle County)
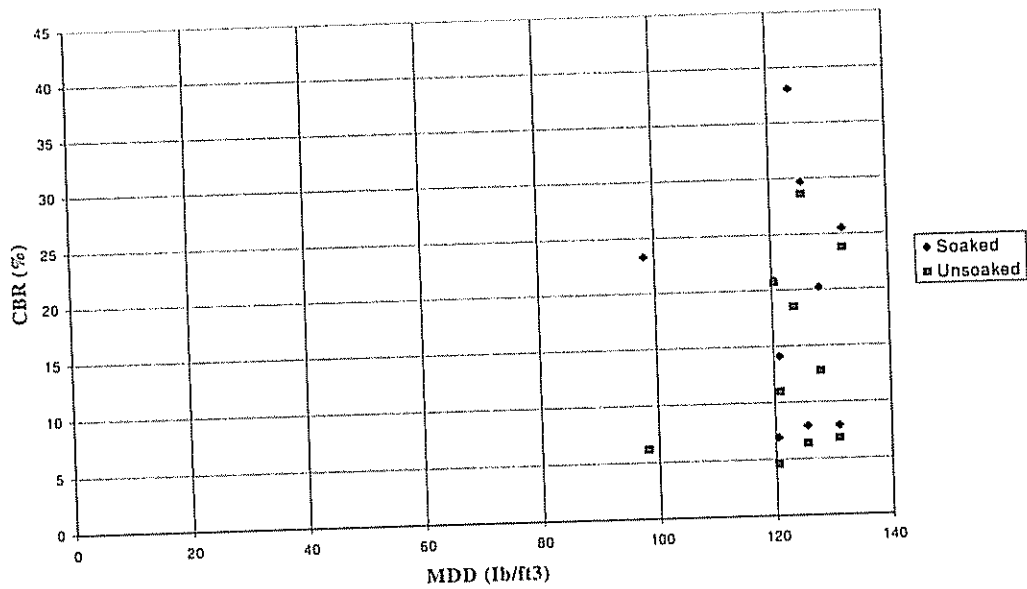

132
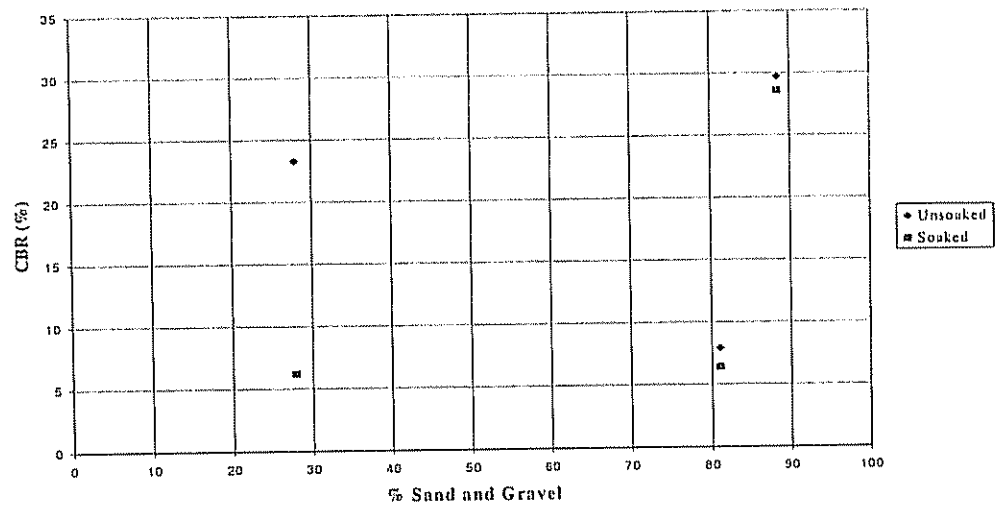
CBR vs. % Sand and Gravel (Kent County)
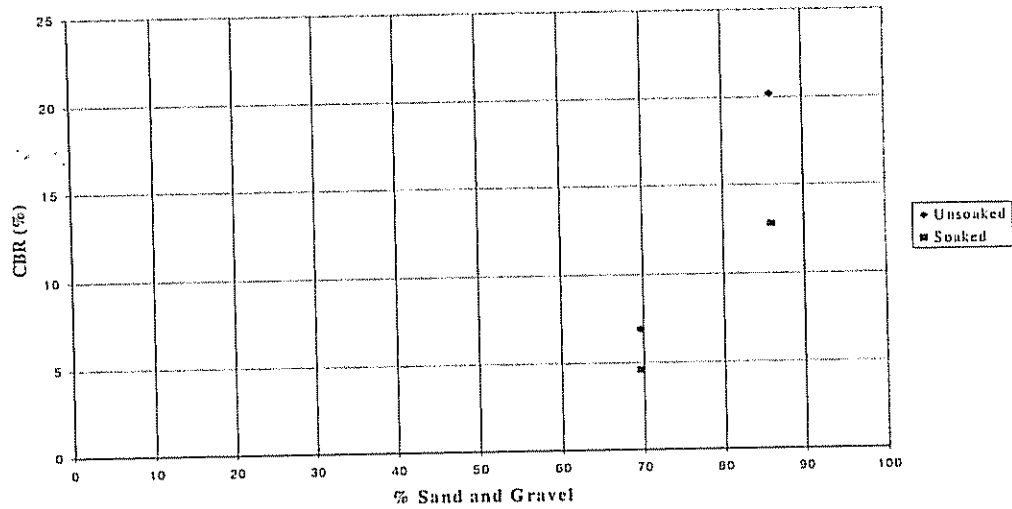
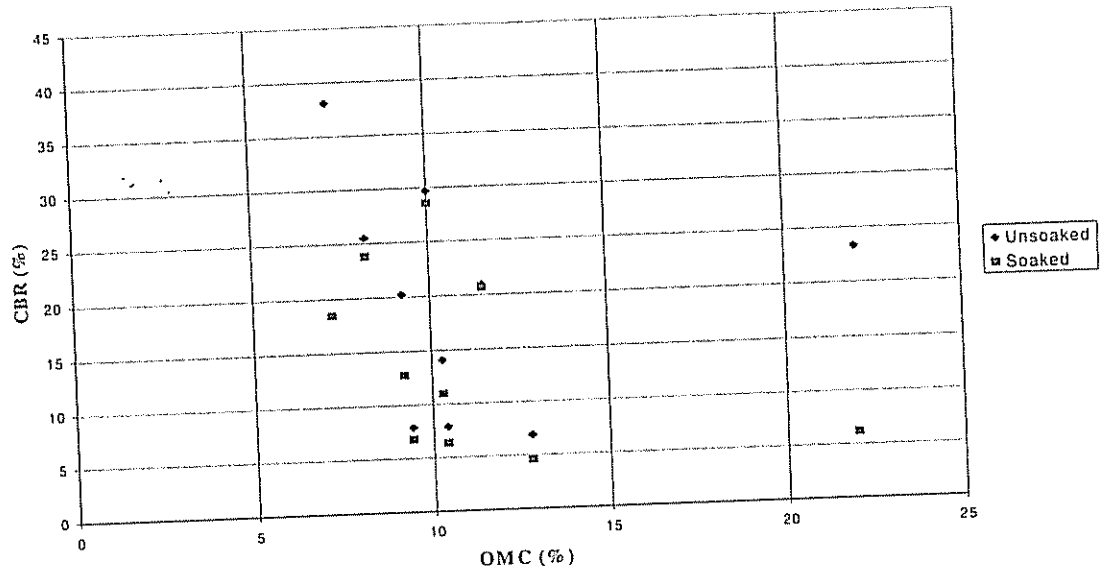**CBR vs. % Sand and Gravel (Sussex County)**

133
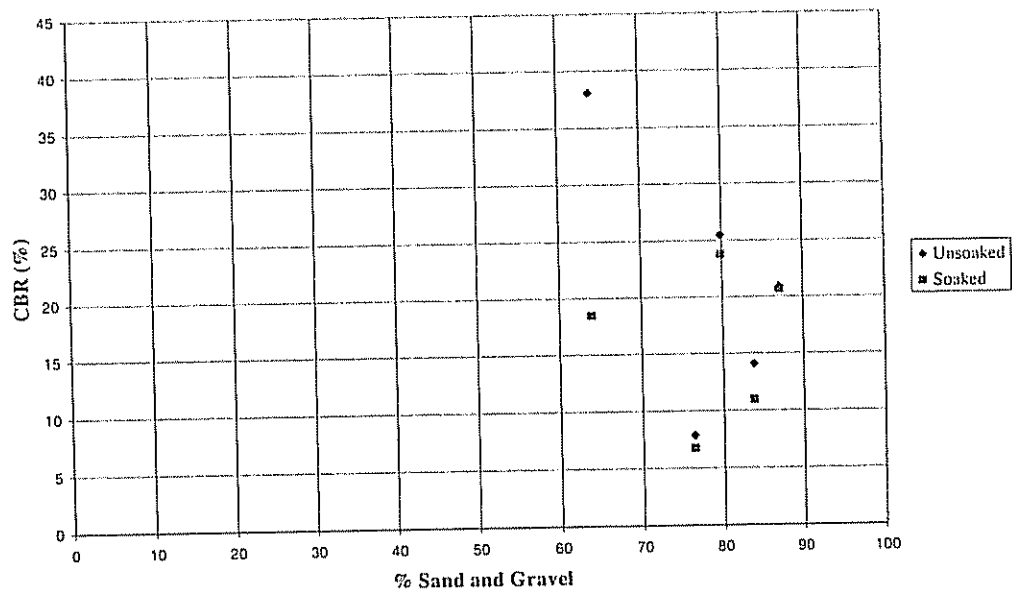
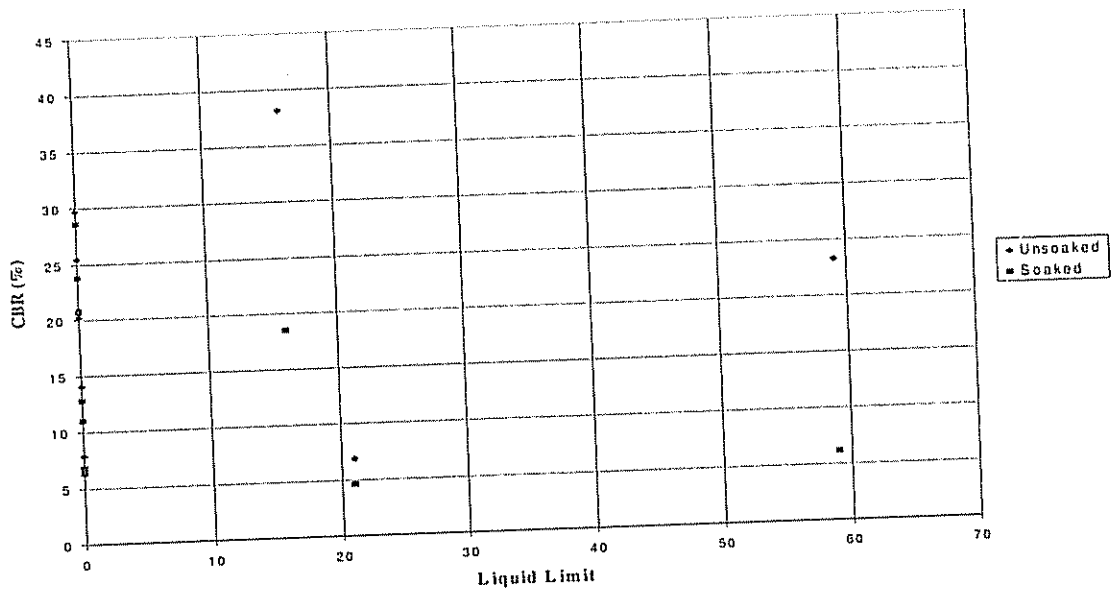## CBR vs % Passing #40 Sieve (*Delaware)
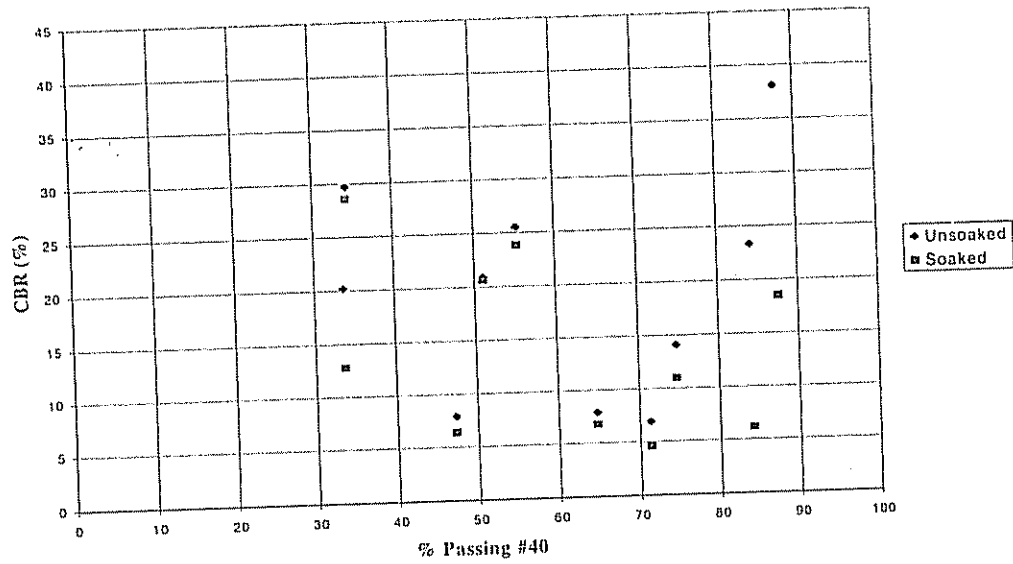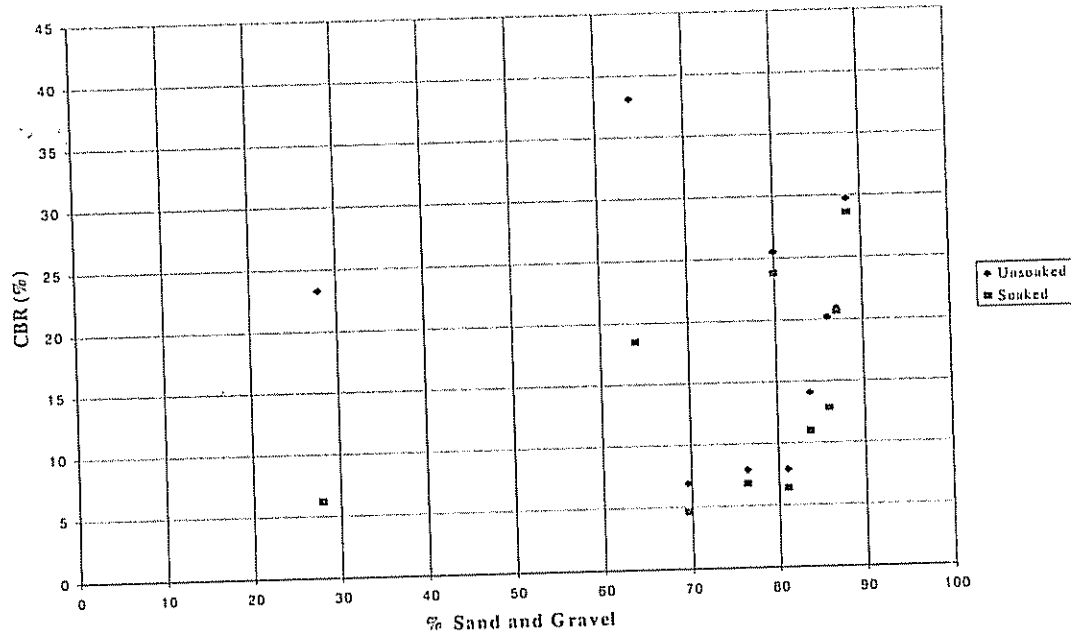

CBR vs MDD (*Delaware)

134

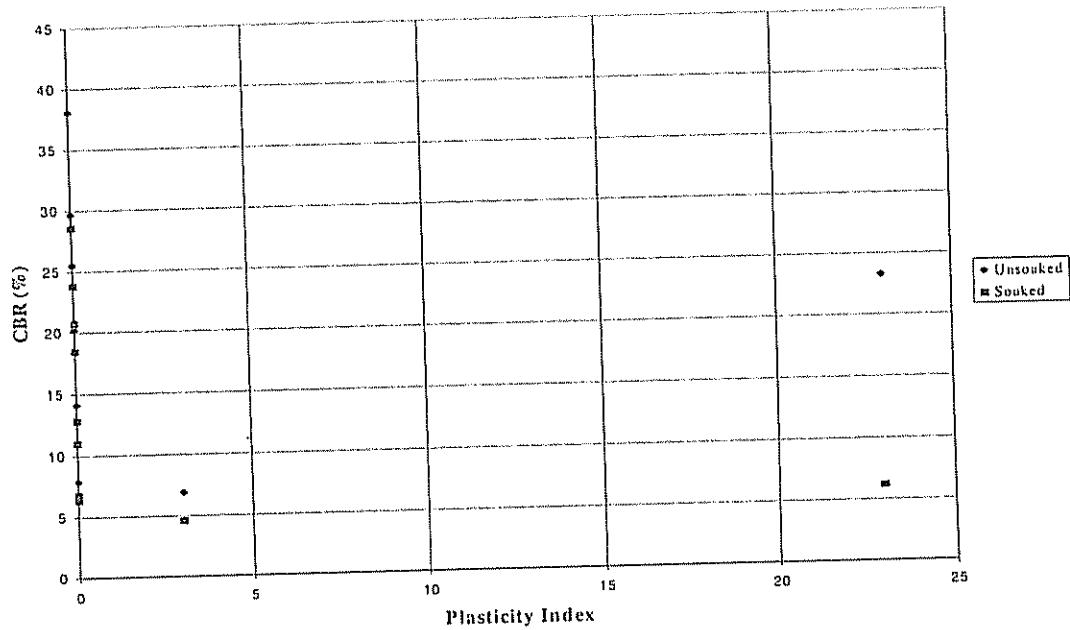## CBR vs OMC (*Delaware)

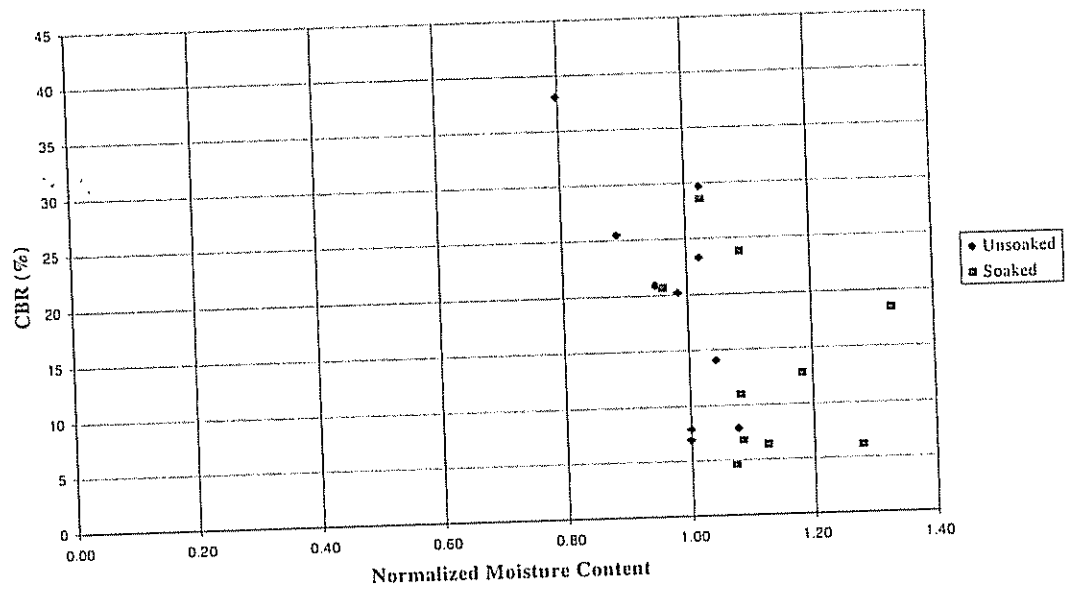## CBR vs. Liquid Limit

CBR vs. % Sand and Gravel

CBR vs. Plasticity Index

136

CBR vs. Normalized Moisture Content

137

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