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CHARACTERIZATION OF SUBGRADE MATERIALS FROM LOCAL SOURCES FOR USE IN THE NIGERIAN EMPIRICAL-MECHANISTIC PAVEMENT ANALYSIS AND DESIGN SYSTEM

BY

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TITLE PAGE

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**A THESISSUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES, AHMADU BELLO UNIVERSITY, ZARIA
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**DEPARTMENT OF CIVIL ENGINEERING,
FACULTY OF ENGINEERING,
AHMADU BELLO UNIVERSITY,
ZARIA, NIGERIA**

JANUARY, 2016

DECLARATION

I declare that the work in this thesisentitled "**Characterization of Subgrade Materials from Local Sources for Use in the Nigerian Empirical-Mechanistic Pavement Analysis and Design**" has been carried out by me in the Department of Civil Engineering, Faculty of Engineering, Ahmadu Bello University, Zaria. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this thesiswas previously presented for another degree or diploma at this or any other Institution.

ABDULFATAI ADINOYIMURANA		
Name of Student	Signature	Date

CERTIFICATION

This thesisentitled "CHARACTERIZATION OF SUBGRADE MATERIALS FROM LOCAL SOURCES FOR USE IN THE NIGERIAN EMPIRICAL-MECHANISTIC PAVEMENT ANALYSIS AND DESIGN" by ABDULFATAI ADINOYIMURANA meets the regulations governing the award of the degree of Doctor of Philosophy (PhD) in Civil Engineering of the Ahmadu Bello University, and is approved for its contribution to knowledge and literary presentation.

Dr. A.T. Olowosulu Chairman, Supervisory Committee	Signature	Date
Dr. M. Joel Member, Supervisory Committee	Signature	Date
Dr. A.O. Eberemu Member, Supervisory Committee	Signature	Date
Dr. Y.D. Amartey Head of Department	Signature	Date
Prof. Kabir Bala Dean, School of Postgraduate Studies	Signature	Date

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DEDICATION

This work is dedicated to Almighty Allah (SWT) and His beloved Prophet Muhammad (SAW).

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ABBREVIATIONS AND SYMBOLS

AASHO American Association of State Highway Officials

AASHTO American Association of State Highway and Transportation Officials

ANOVA Analysis of Variance

ASTM American Society for Testing and Materials

CBR California Bearing Ratio

CL Lean Clayey

FHWA Federal Highway Administration

GC Clayey Gravel

GI Group Index

HMA Hot Mix Asphalt

HRIS Highway Research Information Service

k_i Resilient Modulus Model Parameters (Constitutive Equations Coefficient)

LL Liquid Limit

LTPP Long-Term Pavement Performance

LVDT Linear Variable Differential Transducer

MDD Maximum Dry Density

M-E Mechanistic-Empirical

MEPDG Mechanistic-Empirical Pavement Design Guide

M_r resilient modulus

MTS Master Test Section

MTSs Master Test Sections

NCHRP National Cooperative Highway Research Program

NEMPADS Nigerian Empirical Mechanistic Pavement Analysis and Design System

OMC	Optimum Moisture Content
PEU	Pavement Evaluation Unit
P_a	atmospheric pressure
PL	Plastic Limit
QA	Quality Assurance
QC	Quality Control
R-value	Resistance value
RLT	Repeated Load Triaxial
S	degree of saturation
SEE	Standard Error of Estimate
SHA	State Highway Agency
SHRP	Strategic Highway Research Program
SSV	Soil Support Value
TP	Trial Pit
TRB	Transportation Research Board
UCS	Unconfined Compressive Strength
USCS	Unified Soil Classification System
VIF	Variance Inflation Factor
ε_r	resilient(recoverable) strain
σ_c	confining pressure
σ_d	deviator stress
σ_1	major principal stress
σ_2	intermediate principal stress
σ_3	minor principal stress/confining pressure
θ	bulk stress

σ_{oct}	octahedral stress
τ_{oct}	octahedral shear stress
w_c	moisture content
q_u	unconfined compressive strength
a	initial tangent modulus
γ_s	dry density
γ_{dr}	dry density/maximum dry density
c_u	uniformity coefficient
α	significance level

ABSTRACT

Subgrade materials from different locations in Nigeria were characterized for use in the Mechanistic – Empirical Pavement Design. The basic soil index properties of the subgrade soil materials from Master Test Section (MTS) 1 that identified the material response to external stimuli of traffic loading and environmental conditions were obtained in the laboratory.Three samples each from the MTSs making a total eighteen (18) samples were obtained and subjected to laboratory test to determine their basic physical properties. Testing include particle size distribution, Atterberg limits, specific gravity, compaction characteristics, Unconfined Compression test and the California Bearing Ratio tests. The samples were classified according to American Association of State Highway and Transportation Officials (AASHTO) and the Unified Soil Classification System (USCS).The AASHTO soil classification shows that the subgrade soil samples obtained from the MTS 1 were eitherclayey soil (A-6 and A-7-6) or silty soils (A-4, A-5 and A-2-4). The AASHTO soil classification generally showed that the subgrade samples were "Fair to Poor" in subgrade properties for use as construction materials.The USCS soil classification indicated that most of the samples were lean clay soil with gravel (CL) except few that were either clayey gravel (GC) or clayey sand (SC).This showed that the subgrade samples were mostly clay soil. The California bearing ratio (CBR) values were generally less than 3%. This implies that the strength of the subgrades were poor for engineering construction purposes.Resilient modulus constitutive equation for estimating the resilient modulus of Nigerian subgrade soils was adopted through evaluation of existing resilient modulus constitutive equations using the repeated load triaxial test result conducted on Nigerian subgrade soils. From the evaluation, the National Cooperative Highway Research Programresilient modulus constitutive equation was adopted for purposes of predicting resilient modulus of Nigerian subgrade soils.Comprehensive statistical analysis using multiple linear regression used to develop correlations between basic soil properties and the resilient modulus model parameters.The correlation showed that the resilient modulus model parameters (k_i) can be estimated from basic soil properties.These correlations developed can be used to estimate the resilient modulus of the compacted subgrade soils with reasonable accuracy and can be utilized to estimate level 2 resilient modulus input for Mechanistic – Empirical Pavement Design.

CHAPTER ONE

INTRODUCTION

1.1 Background Information

One of the problems faced by the pavement engineer is inadequatecharacterization ofmaterial properties of existing service pavements. The information is vital in the assessmentof pavement structural capacity to accommodate thegrowing traffic for future overlaying and development of rehabilitative recommendations and maintenance strategy, based on routine structural evaluation(Viswanathan, 1989).

Pavements fail for different reasons; poor design, poor materials and poor construction methods are the most common. The pavement foundation (subgrade) represents one of the key elements in the pavement design; its behavior will influence the overall pavement performance. Subgrade soils are subjected to repeated loads due to heavy traffic, which can cause deformations and distress of the overlying structures. To improve and standardize design procedures, the American Association of State Highway and Transportation officials (AASHTO) published the Guide for Design of Pavement Structures (AASHTO, 1986) in which the use of resilient modulus was adopted as the principal soil property contributing to the design of flexible pavements (Ibrahim, 2013).

Material characterization is vital to pavement analyses and has received increasing focus as it forms a critical component in recent improvements to engineering practices in terms of pavement design. This pertains to all aspects of pavement engineering—analysis, design, construction, Quality Control(QC)/Quality Assurance(QA), pavement management, and rehabilitation. At each stage during the life of a project, the influence of several fundamental construction material parameters on the long-term performance of the pavement has been recognized. There is a greater emphasis on optimizing the performance of concrete pavements, which involves a detailed understanding of the variables that affect pavement behaviour and the properties of concrete that correspond to the desired performance. Consequently, there is the need for more information about material properties so that they can be characterized accurately for predicting performance or for verifying their quality during the construction phase. With limited resources for performing laboratory and field tests to determine material properties, the need for a secondary means to obtain these material property values (i.e., through correlations or predictive models based on data from routine or less expensive tests) is obvious. Additionally, the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) offers users the option of using inputs obtained through correlations.The MEPDG defines level 1 inputs as those obtained from actual laboratory resilient modulus testing which is conducted to characterize the subgrade soil. In Level 1 design/analysis, the MEPDG requires input of the regression constants of the stress-dependent constitutive equation for resilient modulus of a particular unbound material (subgrade soil or base aggregate). Constitutive equation coefficients (*k*-values) are usually obtained from the regression analysis of resilient modulus test data for an actual soil/aggregate sample(Hossain, 2010). The MEPDG defines level 2 inputs as those obtained from correlations between the primary inputs (level 1 measured) and other parameters that are material-specific or are measured through simpler tests(Rao *et al.*, 2012). The MEPDG defines level 3 inputs as those obtained fromtypical resilient modulus values which are used based on soil classification(Hossain, 2010).

Subgrade characterization allows for the design of a proper foundational support for the pavement. On the other hand, base/sub-base materials provide structural capacity to the pavement. Therefore, both subgrade and base/sub-base material characterization is needed to design adequate pavement structure for expected traffic (Hossain, 2008). Recommendation of layer types and their dimensions were established based on American Association of State Highway Officials (AASHO) road tests performed during the 1950s. The often-used soil strength parameters in pavement design practice are California Bearing Ratio (CBR) value, Hveem *R* value, and Soil Support Value (SSV). All these soil parameters are based on the failures of subgrade soil specimens in the laboratory conditions. However, flexible pavements seldom fail owing to subgrade strength failures during their service life(Huang, 1993).

The CBR test istaken as direct measure of the strength of the in-situ subgrade material. Despite concerns regarding the limitedaccuracy of this test, it is utilized on the basis that is widely used and accepted by both theorists andpractitioners(Abiola *et al*., 2012).

Empirical test parameters such as the CBR, R-value, etc., are used to characterize subgrade soil and base/sub-base aggregate. Resilient modulus testing, a basis for the mechanistic approach, was later incorporated into the AASHTO design guide for subgrade soils characterization, but most departments of transportation are still using empirical relations based on the CBR. Although the resilient modulus was incorporated in 1986, the basic pavement design process still depends on the results of the American Association of State Highway Officials (AASHO) road test, which were limited to a particular soil and environmental condition. The resilient modulus test is the test recommended to characterize subgrade soil in the 1993 AASHTO design guide and both subgrade soil and aggregate base for pavement design in the MEPDG. The resilient modulus test requires significant resources including a high level of technical capability (Hossain, 2010).

The resilient modulus is analogous to the elastic modulus used in elastic theories and is defined as a ratio of deviatoric stress to resilient or elastic strain experienced by the material under repeated loading conditions that simulate traffic loading. Most bases and subgrades are not elastic and they experience permanent deformation under repeated loads (Uzan, 2004). However, because loads applied in the laboratory test for resilient modulus are small when compared with ultimate loads at failure and also the result of the application of a large number of cycles of loading that reduces the plastic deformation, the deformation measured during test cycles is considered as completely recoverable or elastic and hence the recovered deformations are used to estimate the resilient modulus or elastic modulus (or simply modulus or stiffness)(NCHRP, 2008).

Some agencies consider the cost, time, complication, and sampling resolution required for meaningful resilient modulus testing to be too cumbersome for its application in less critical projects. Regardless of project size, it is often difficult to predict and consequently reproduce the in-situ conditions, usually with respect to state of stress, further complicating the use of resilient modulus testing. Because of this, correlations are desired for estimating resilient modulus, especially for use (or verification of default values) associated with mechanistic-empiricalpavement design Level 2 design/analysis. A common method to predict resilient modulusvalue is through the use of correlations with other soil test properties such as the CBR. Another approach is to use the constitutive equation with the *k*-values (resilient modulus parameters) estimated from soil index properties through further regression equations. The use of soil properties to determine the regression constants presents the concern of multi-co-linearity effects, in which a strong correlation exists among and between the explanatory variables (Yau and Von Quintus, 2002).

Nigerian Empirical Mechanistic Pavement Analysis and Design System (NEMPADS) is a framework for mechanistic-empirical pavement design for tropical climate (Olowosulu, 2005). An advantage of this approach is the modular nature, which allows for adaptation of new development into pavement design without change to the process. NEMPADSconsists of two parts.Part 1 consists of the development of input values, which include traffic, climate and material. Geotechnical analysis is also performed in this part to determine the strength and stiffness of the sub-grade. Part 2 of the design process is structural response analysis. Each component of the design procedure is developed specifically for Nigeria environment. The material characterization, load characterization and mechanistic analysis and behaviour functions developed for overlay design procedure were adopted for the new method(Claros *et al*., 1986).

Master Test Section(MTS) are section chosen to fill the factorial experiment and also took into account the soil type and history, pavement layer homogeneity, layer thickness and field operational considerations for Nigerian soils. The soils tested in this study for the correlation of basic soil properties with the resilient modulus constitutive model were selected to provide a general representation of typical subgrade soils located in MTS 1 (Kano-Kaduna region) of Nigeria. There are six sites within MTS 1 namely MTS 1 – 1, MTS 1 – 2, MTS 1 – 3, MTS 1 – 4, MTS 1 – 5 and MTS 1 – 6 with route numbers A236, A11, A2, A235, A125 and A2 respectively(Claros *et al*., 1986).

1.2 Statement of the Problem

Pavement structure is normally supported by the underlying soil and their structural design depends on the condition of the soil. Hence, the need to characterizing the supporting soil layer, also known as the subgrade, a critical component for pavement design and, the performance and life of the pavement structure(Hossain, 2008). The design and evaluation of pavement structures on base and subgrade soils requires a significant amount of supporting data such as traffic loading characteristics, base, subbase and subgrade material properties, environmental conditions and construction procedures. Currently, empirical correlations developed between field and laboratory material properties are used to obtain highway performance characteristics (Barksdale *et al*., 1990). These correlations do not satisfy the design and analysis requirements since they neglect all possible failure mechanisms in the field. Most of these methods, which use CBR and SSV, do not represent the conditions of a pavement subjected to repeated traffic loading. Recognizing this deficiency, the 1986 and the subsequent 1993 AASHTO design guides (AASHTO, 1986; AASHTO, 1993)recommended the use of resilient modulus for characterizing base and subgrade soils and for designing flexible pavements. The resilient modulus accounts for soil deformation under repeated traffic loading with consideration of seasonal variations of moisture conditions(Titi *et al*., 2006). The resilient modulus test requires significant resources including a high level of technical capability (Hossain, 2010). Because of this, correlations are desired for estimating resilient modulus, especially for use (or verification of default values) associated with mechanistic-empirical pavement design Level 2 design/analysis.

1.3 Justification of theStudy

For the design of flexible pavement structure, there is the need to characterize the supporting soil layer, also known as the subgrade, a critical component for pavement design and, the performance and life of the pavement structure. The AASHTO design guides recommended the use of resilient modulus which accounts for soil deformation under repeated traffic loading with consideration of seasonal variations of moisture conditions for characterizing subgrade soils. The resilient modulus test requires significant resources including a high level of technical capability. Because of this, correlations are desired for estimating resilient modulus, especially for use (or verification of default values) associated with mechanistic-empirical pavement design Level 2 design/analysis. There is a need to develop a correlations for estimating resilient modulus of Nigerian subgrade soils for mechanistic-empirical pavement design Level 2 and 3 design/analysis.

1.4 Aim and Objectives of the Study

1.4.1 Aim of the study

The aim of this research is to characterise subgrade materials from Local sources for use in the Nigerian Empirical-Mechanistic Pavement Analysis and Design System (NEMPADS).

1.4.2 Objectives of the study

The following specific objectives are identified for successful accomplishment of this study:

1. Examine the state of the practice regarding the use of the resilient modulus in pavement analysis/design.
2. Obtain subgrade soil samples from Master Test Section (MTS) 1 and determine their basic soil index propertiesviz: soil gravity; Atterberg limits; specific gravity; compaction characteristics;unconfined compression strength and the CBR values.
3. Evaluate available resilient modulus equations (models) and proposed an equation for estimating resilient modulus of Nigerian subgrade soils.
4. Develop and specify default resilient modulus parameters(k-values) values ofNigerian subgrade soils for inputs in level 3 analysis.

5. Evolve correlations with basic soil index properties for estimating resilient modulus of Nigerian subgrade soil for inputs in level 2 analysis.

1.5 Scope of the Study

The scope of this study was to establish correlation relationships between the measured soil resilient modulus data using laboratory triaxial test and the basic soil properties. The theoretical relationship from past researchwas evaluated based on the experimental studies. Using the resilient modulus data from the work of Claros *et al*, 1986, a resilient modulus constitutive equation was adopted for Nigerian subgrade soils and its resilient modulus parameters was obtained through regression analysis. The laboratory-testing program was conducted on selected subgrade soils from MTS 1 to evaluate their basic engineering properties. Comprehensive statistical analysis was performed to develop correlations between basic soil properties and the resilient modulus model input parameters.This study was delimited to developing resilient modulus equations for Nigerian subgrade soils, developingdefault resilient modulus parameters (*k*-values)for level 3 analysis anddeveloping correlation equations with basic soil engineering properties of Nigeria subgrade soils for estimating resilient modulus for level 2 analysis.

CHAPTER TWO

LITERATURE REVIEW

2.1 Flexible Pavement

Pavements are structures used for the purpose of carrying vehicular traffic safely and economically (Adu-Osei, 2001). Pavement design is a continually changing field as a result of the dynamic interactions of the development of improved concepts and more and better data collection systems. In early stages of highway and airport development, pavement design was a rule-of-thumb procedure based on past experiences. With the increase in vehicular traffic (both weight and number of vehicles), more and more attention has been directed toward fundamental concepts. Empiricism is necessary to relate fundamental concepts and empirical definitions of failure of a pavement system (Southgate *et al*., 1976).

Historically, flexible pavement design practices were typically based on empirical procedures, which recommend certain base, subbase, and surface layer types and their thicknesses based on the strength of the subgrade (Huang, 1993).

Flexible Pavement is usually composed of several asphalt concrete layers, a granular base course and a soil subgrade. For mechanistic design of pavement systems based on elastic theory, a modulus of elasticity must be designated for each design layer including the soil subgrade(Smolen, 2003).

A conventional flexible pavement consists of a prepared subgrade or foundation and layers of sub-base, base and surface courses (AASHTO, 1993).

The sub grade is the foundation layer over whichthe roadway is being constructed and all the loads whichcome onto the pavement are eventually supported by it. Insome cases, this layer is normally considered to be thenatural in situ; in other cases, the term sub grade is appliedto include compacted soil existing in a cut section or theupper layer of an embankment section(Abiola *et al*., 2012).

For the roadbed soils, the seasonal variation of resilient moduli is considered and used directly to determine the design or effective roadbed soil resilient modulus. However, seasonal variation of the resilient moduli for pavement materials is not used or considered in the design process, even though the resilient modulus of pavement materials can vary substantially throughout the year(Von Quintus and Killingsworth, 1997).

These layers are selected to spread traffic loads to a level that can be withstood by the subgrade without failure. The surface course consists of a mixture of mineral aggregates cemented by a bituminous material. The base and sub-base course usually consists of unbound granular materials. In flexible pavements, and especially for thinly surfaced pavements, the unbound granular layers serve as major structural components of the pavement system(Adu-Osei, 2001).

A good performance for a flexible pavement is achieved by stability, load distribution characteristics and durability. It is essential for a designprocess to acquire the exact material properties, loadingcharacteristics and seasonal conditions(Viswanathan, 1989).The performance of pavements depends to a large extent on the strength and stiffness of the subgrades. Subgrades play an important role in imparting structural stability to the pavement structure as it receives loads imposed upon it by road traffic. Traffic loads need to be transmitted in a manner that the subgrade-deformation is within elastic limits, and the shear forces developed are within safe limits under adverse climatic and loading conditions. The subgrade comprises of unbound earth materials such as gravel, sand, silt and, clay that influence the design and construction of roads. The assessment of properties of soil subgrades, in terms of density, soilstiffness, strength, and other in-situ parameters is vital in the design of roads, and their performance(Rao *et al*., 2008).

When designing pavements, the characteristic of the subgrade upon which the pavement is placed is an essential design parameter that is considered. Subgrade materials are typically characterized by their resistance to deformation under load, which can be a measure of their strength or stiffness. In general, the more resistant to deformation a subgrade is the more load it can support before reaching a critical deformation value. A basic subgrade stiffness/strength characterization is resilient modulus. Both the AASHTO 1993 Design Guide and the mechanistic based design methods use the resilient modulus of each layer in design process(Sheng, 2010).

The objective of pavement design is to provide a structural and economical combination of materials such that it serves the intended traffic volume in a given climate over the existing soil conditions for a specified time interval. Traffic volume, environmental loads, and soil strength determine the structural requirements of a pavement, and failure to characterize any of them adversely affects the pavement performance. Traffic is estimated from present traffic and traffic growth projections. Climatic conditions are incorporated in the design by accounting for their effects on material properties. The subgrade may be characterized in the laboratory or by field tests or both. It is essential that methods adopted to characterize reflect the actual subgrade's role in the pavement structure, and the frequency of the sampling should account for spatial variation in the field. All pavements derive their ultimate support from the underlying subgrade: therefore, knowledge of basic soil mechanics is essential (George, 2004).

Most of the flexible pavements fail owing to either excessive rutting or cracking of pavement layers as a result of fatigue, temperature changes, and/or softening caused by the surface layer cracking(Barksdale, 1972; Brown, 1974; Brown, 1996). Since flexible pavements do not fail as a result of soil strength failure, the 1986 AASHTO interim pavement design guide and subsequently the 1993 AASHTO pavement design guide recommended the use of a soil parameter known as the resilient modulus to replace strengthbased parameters such as CBR and SSV(Brickman, 1989; Mohammad *et al*., 1994; Maher *et al*., 2000). Several other investigations also refer to this modulus parameter as resilient modulus in their studies(NCHRP, 2008).

Material properties that contribute to pavement distress include modulus and shear strength, which are greatly influenced by moisture content, density, and gradation. Failures in flexible pavements resulting from poor performance of granular layers are manifested as permanent deformation (rutting), fatigue and longitudinal cracking, depressions, corrugations and frost heave. Failures in rigid pavements resulting from poor performance of granular layers include pumping, faulting, cracking, corner breaks, and fatigue cracking (Saeed*et al*., 2001).

A pavement design requires characterization of the component materials in addition to the support soil. The subgrade is the underlying soil, and its characterization allows for the design of a proper foundational support for the pavement (Hossain, 2010).

Of all the pavement components, the subgrade support condition is the vital that factor contribute to the performance of the pavement and maximum effort should be stressed in arriving at the material factor, i.e. the resilientmodulus of the subgrade material. About 60% of the deformation occurs in thesubgrade which is resilient in nature. Thus, the characterization of fine grainedsoils response to loading is an important consideration in pavement design andperformance(Viswanathan, 1989).

2.2 Pavement Design Methods

Knowledge concerning the characteristics of natural materials such as subgrade soils is still relatively limited. Many of the pavement design procedures presently employed remain empirically based. They were often developed from experience with existing roads, supplemented with the analysis of test sections and a few major research projects like the well-known AASHO Road Test (Powell*et al*., 1984).

Test methods for characterisation of the mechanical properties of the subgrade are often still empirically based and only yield a rough estimate of the fundamental material parameters required for pavement design. Consequently, material specifications too are mainly based on experience and practical considerations. Choosing the correct road construction materials and having a full understanding of their material properties and performance under traffic loading is paramount for the successful utilisation of the road. This understanding is essential for the design of new roads as well as the addition of layers to roads requiring strengthening during rehabilitation works. The empirical approach to material characterisation and pavement design has been used for many years; continual revision of these methods with newly gathered experience has improved on many early shortcomings. These empirical test and design methods form a sound basis for pavement design. Because of their empirical nature, they are often very well implemented and, most importantly, simple in nature. The testing techniques require only standard laboratory equipment and often the pavement design techniques use charts from which the pavement design for a given set of circumstances can be obtained(Gillett, 2001).

2.2.1 Conventional roadway design

A number of testing methods are available to the designer to aid in roadway design. The CBR test is a common empirical test method used in the design of pavements. The CBR tests are typically conducted after soaking the compacted soil sample (Vogrig *et al*., 2003).

The laboratory CBR-test is used throughout the world as a quick means of characterizing qualitatively the bearing capacity of soils and unbound base and subbase materials. The CBR value still is an input value to many pavement design procedures(Araya, 2011).

The design of roadway structures is also based sometimes on the resistance, or *R*-value. *R*-value test was first formulated in 1930's and used to determine the stability of field and laboratory samples of bituminous pavements. Later, it was modified to determine the resistance of subgrade materials and used in the design of pavements (Chadbourn *et al*., 2002). A number of parameters such as exudation pressure, expansion pressure, and resistance values are required to determine *R*-value. This method is useful in determining required thicknesses of material above a sub-soil that will prevent swelling. The *R*-values estimate a soils ability to resist lateral deformation under applied vertical loads (ASTM D 2844, 2003). The *R*-values are sometimes converted to resilient modulus values and used in the design of pavements(Garber and Hoel, 2009).

A more widely used recent test method on which pavement designs are based is the resilient modulus value. It is defined as the ratio between repeated deviator stress and resilient strain. The laboratory testing procedures for determining the resilient modulus values is time consuming and needs expensive equipment and highly trained personnel (Vogrig *et al*., 2003).

The design of pavements can also be based on index properties of the soil. Testing of index properties is relatively simple and inexpensive. Correlation of the index properties of a soil to more mechanistic property such as resilient modulus has the potential to provide a reasonable and cost effective means of pavement design (Zeghal, 2001).

2.2.2 Empirical and mechanistic design methods

An empirical design approach is one that is based solely on the results of experiments or experience. Observations are used to establish correlations between the inputs and the outcomes of a processe.g., pavement design and performance. These relationships generally do not have a firm scientific basis, although they must meet the tests of engineering reasonableness(Charles and Carvalho, 2007).

Methods which make use of the calculated stresses and strains within the pavement, together with studies of the effect of these stresses and strains on the pavement materials (mechanistic behaviour) are usually called ‘mechanistic methods’, ‘theoretical methods’ or ‘analytical methods’. The two methods (empirical and mechanistic) are complimentary(Araya, 2011; Rolt, 2004).

Empirical methods require theoretical understanding to help extend them to different conditions, whilst mechanistic methods require empirical information for calibration.Neither method is ideal on its own, but the combination of the two provides a competent basis for design namely the ‘Mechanistic-Empirical’ (M-E) method(Araya, 2011).

Mechanistic-Empirical (M-E) methods represent one step forward from empirical methods. The induced state of stress and strain in a pavement structure due to traffic loading and environmental conditions is predicted using theory of mechanics(Charles and Carvalho, 2007).

The major drawback of empirical methods is that they only operate within the limits of the experience on which they are based. It is thus desirable to develop more general analytical design procedures. These analytical methods should be based on the capability to calculate stress, strain or deflection in a pavement subjected to an external load providing pavement response that can subsequently be interpreted in terms of long-term pavement performance such as cracking and rutting(Sweere, 1990).

Recent advances in mechanistic-empirical modelling for pavements make it now possible to predict pavement performance in terms of fundamental engineering properties of the layer materials. The mechanistic-empirical modelling approach couples laboratory characterization of material properties and the theories of mechanics with empirical observations of distresses in field pavement sections (Gervas, 2003).

In both empirical and M-E design systems, material property inputs are essential to characterize pavement behaviour and to predict pavement responses, such as the magnitudes of stress, strain, and displacement, when subjected to applied traffic loads and environmental conditions. Furthermore, major pavement distresses are associated directly with the material properties of a component (or layer) of the pavement structure(Rao *et al*., 2012).

2.3 Mechanistic-Empirical Pavement Design Guide (MEPDG)

The proposed MEPDG procedure, introduced in National Cooperative Highway Research Program (NCHRP) Project 1-37A (NCHRP, 2004), is an improved methodology for pavement design and evaluation of paving materials. This is because the MEPDG procedure provides better capability for predicting pavement performance using mechanistic analyses to determine stresses and strains and empirical models to predict performance. To accomplish this improved prediction capability, the MEPDG procedure requires fundamental material properties (Apeagyei and Diefenderfer, 2011).

The MEPDG adopts a hierarchical input level scheme to accommodate the designer's knowledge of the input parameter. Inputs can be provided at three different levels. Level 1 inputs represent the input parameter and typically are obtained from a projectspecific data collection or test effort. Level 2 represents a moderate level of knowledge of the input parameter and is often calculated from correlations with other site-specific data or a less expensive measure. Level 3 represents the least knowledge of the input parameter and is based on default values. The MEPDG therefore supports the use of level 2 or 3 data in the absence of level 1 laboratory test data(Rao *et al*., 2012).

Unlike currently used empirical pavement design methods, this new procedure depends heavily on the characterization of the fundamental engineering properties of paving materials (Flintsch *et al*., 2007).

In Mechanistic-Empiricalapproach for pavement design, four major inputs are utilized to predict pavement responses and ultimately pavement performance enabling the selection of a cross-section meeting the specified requirements. M-E design enables the mechanical properties of the selected materials to be used in conjunction with empiricalperformance information and site conditions (traffic and climate), as shown in Figure 2.1.

Inputs include an initial pavement structure, climatic data, traffic volume and weight distributions, and material properties of the Hot Mix Asphalt (HMA), base and subgrade materials. In M-E pavement design, accurate representation of material characteristics is imperative to a successful and reliable design. In an M-E approach, accurate material characterization is vital in successfully predicting pavement responses and ultimately pavement performance(Robbins, 2009).

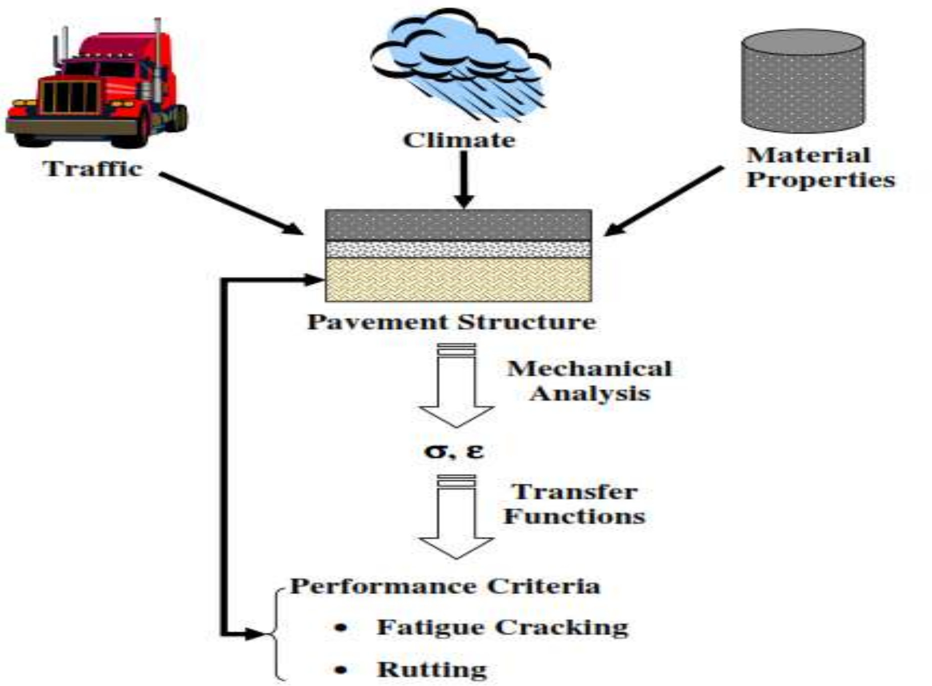


Figure 2.1: Mechanistic-Empirical Pavement Design Framework
Source: (Robbins, 2009)

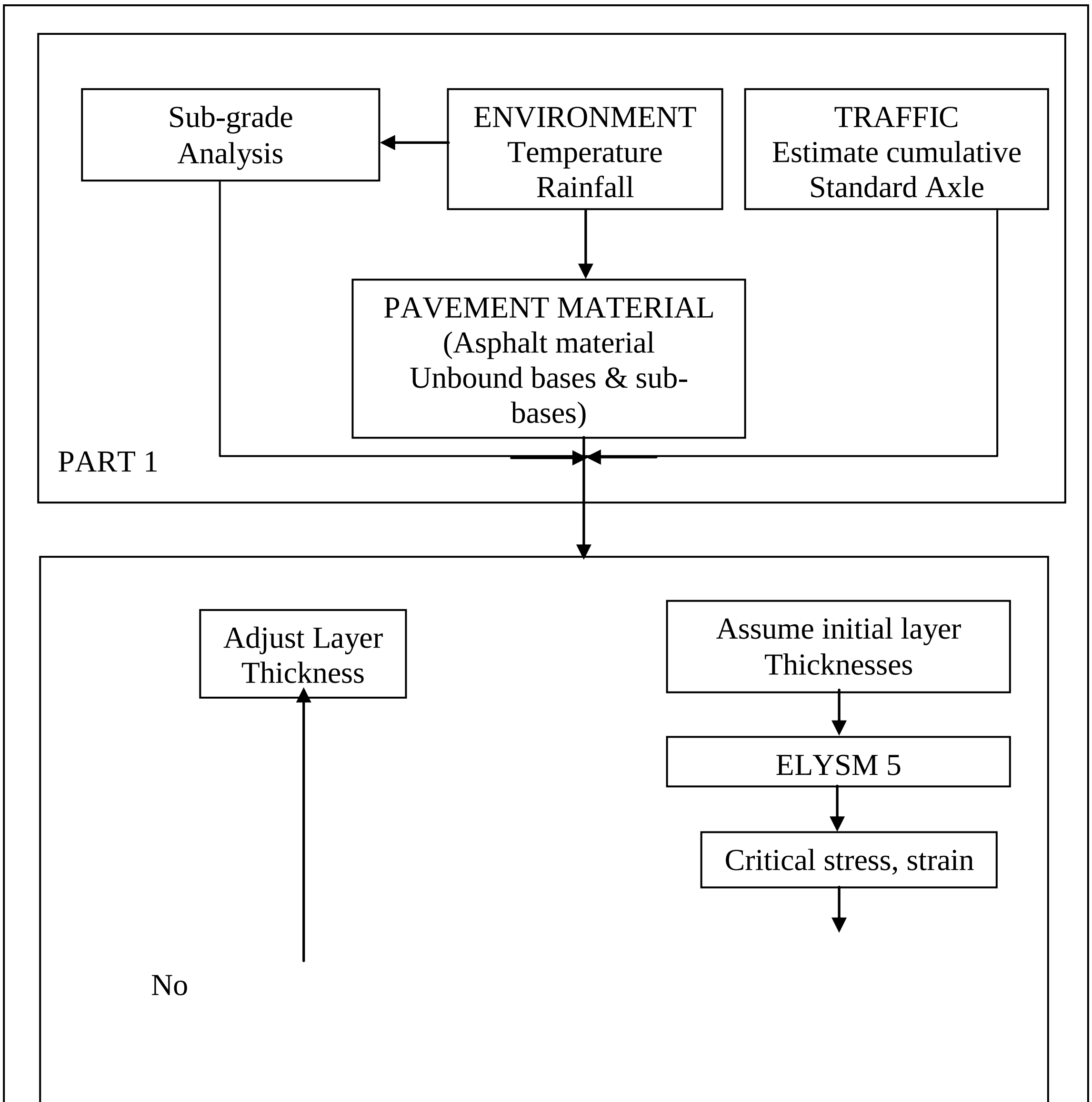
MEPDG considers traffic, structural features, materials, construction, and climate far more than ever before. It uses a hierarchical approach to determine design inputs. Depending on the desired level of accuracy of input parameter, three levels of input are provided from Level 1 (highest level of accuracy) to level 3 (lowest level of accuracy). Depending on the criticality of the project and the available resources, the designer has the flexibility to choose any one of the input levels for the design as well as use a mix of levels(Halil *et al*., 2009).

The three identified levels of design input parameters provides the pavement designer with flexibility in achieving pavement design with available resources based on the significance of the project. The three levels of input parameters apply to traffic characterization, material properties, and environmental conditions(Titi *et al*., 2006). Level 1 input parameters are measured directly and are considered site or project specific. This level requires the greatest amount of testing and data collection. Level 2 input parameters generally are less detailed data sets that are used with correlations or regressions to estimate the corresponding Level 1 parameters. This level of input data requires less testing and data collection efforts. Level 3 input parameters are either "best estimate" or default values and require the least testing and data collection(Diefenderfer, 2010).

2.4 Nigerian Empirical Mechanistic Pavement Analysis and Design System

Nigerian Empirical Mechanistic Pavement Analysis and Design System (NEMPADS) is a framework for mechanistic-empirical pavement design for tropical climate (Olowosulu, 2005). A flowchart depicting this framework is as shown in Figure 2.2, which was based on similar flow charts developed by other researchers(Timm, *et al*., 1998). An advantage of this approach is the modular nature, which allows for adaptation of new development into pavement design without change to the process. For example, as better mechanistic-based load-deformation models become available, they may simply be substituted into the procedure to yield a more accurate prediction of stresses and strains(Olowosulu, 2005; Olowosulu *et al*., 2011).

'NEMPADS' consists of two parts.Part 1 consists of the development of input values, which include traffic, climate and material. Geotechnical analysis is also performed in this part to determine the strength and stiffness of the sub-grade. Part 2 of the design process is structural response analysis. It also shows the step-by-step procedure of M-E, starting with an assumed initial layer thickness through to selection of the optimum layer thickness. The analysis is an iterative trial-and-error solution. Initially with assumed layer thickness, the critical stresses and strains are computed using the Elastic Layered System Analysis (ELYSM) 5 computer program. These are then compared with relevant failure criteria.When any criterion is exceeded, the thicknesses are adjusted. This procedure is repeated until all failure criteria are satisfied. Each component of the design procedure is developed specifically for Nigerian environment.



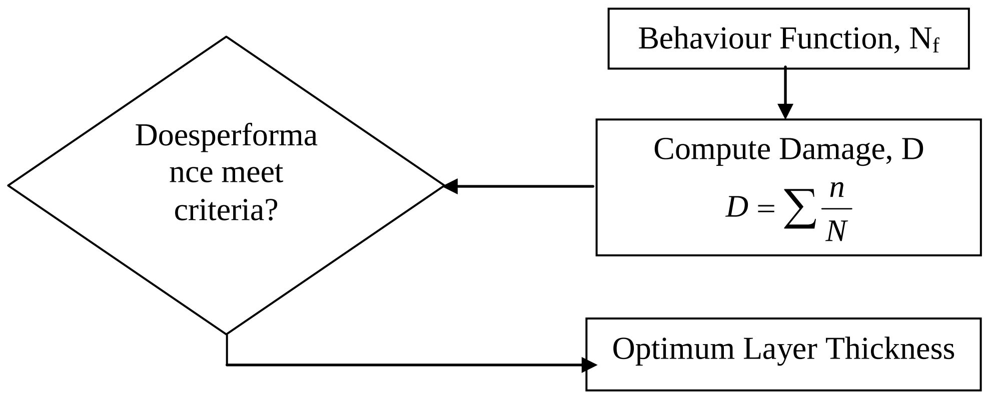


Figure 2.2: Mechanistic-Empirical Pavement Design Procedure
Source: (Olowosulu, 2005; Olowosulu*et al*., 2011)

The material characterization, load characterization and mechanistic analysis and behaviour functions developed for overlay design procedure (Claros*et al*., 1986) were adopted for the new method.The key feature of the procedure presented in Figure 2.2 is an iterative loop to determine whether performance meets failure criteria specified and after accumulated damage has been computed, the designer must evaluate and decide if the layer thicknesses should be changed(Olowosulu, 2005; Olowosulu *et al*., 2011).

Miner's hypothesis (Miner, 1945) was used to quantify accumulating damage, in terms of rutting or fatigue, over the life span of the pavement. The expression simply adds the damage done in the particular seasons under given load configurations. When the damage exceeds unity, the pavement has been under-designed and thicknesses are increased. If the damage is much less than unity, the pavement has been over-designed and thicknesses are decreased. An optimum design is achieved when the damage is near but not exceeding unity (Olowosulu, 2005; Olowosulu*et al*., 2011).

2.5 Resilient Modulus

The resilient modulus, defined as the repeated deviator stress (axialstress in unconfined tests divided by the recoverable strain and the resilientPoisson's ratio, defined as the ratio of recoverable lateral stress to axial strain hasbeen used to highlight the properties of granular and fine grained materials. Theresilient modulus test is a repetitive load test performed in either an unconfined ortriaxial state ofstress(Viswanathan, 1989).

The resilient modulus is an important parameter which describes the mechanical behaviour of unbound granular materials. However, this parameter can be determined from physical properties (Dione *et al*., 2013).

The resilient modulus is a fundamental engineering material property that describes the non-linear stress-strain behaviour of pavement materials under repeated loading. It is defined as the ratio of the maximum cyclic stress to the recoverable resilient (elastic) strain in a repeated dynamic loading(Mohammad *et al*., 2007).
It is a measure or estimate of the elastic modulus of the material at a given stress or temperature. Mathematically it is expressed as the ratio of applied deviator stress to recoverable strain (George, 2004).

Resilient Modulus is a key value in pavement design. Performance of resilient modulus tests is difficult, expensive and time consuming and hence many researchers are developing mathematical models that satisfactorily predicts resilient modulus values without the necessity of a laboratory test. It is very important for a mathematical model to accommodate new data as it becomes available (Ibrahim, 2013).

Resilient Modulus is the failure of a flexible pavement structure supported on a subgrade soil and subjected to repeated traffic loading; this can occur through two primary mechanisms - collapse of the pavement structure or cracking of the surface of the pavement. A collapse of the pavement structure can occur due to large plastic (permanent) deformations in the subgrade soils. However, even when the loads on the pavement are not excessive but nominal, the pavement surface can crack due to fatigue, caused by the reversal of elastic strains at any location in the pavement system. As a result of repeated loads such as those caused by moving traffic, cohesive soils in the subgrade incur repeated elastic deformations. When these deformations exceed a threshold value, premature fatigue failure of the flexible pavement through cracking of the pavement surface occurs (Ibrahim, 2013).

Subgrade soil characterization in terms of resilient modulus has become crucial for pavement design. For a new design, resilient modulus values are generally obtained by conducting repeated load triaxial tests on reconstituted/undisturbed cylindrical specimens. Because the test is complex and time-consuming, in-situ tests would be desirable if reliable correlation equations could be established. Alternately, resilient modulus can be obtained from correlation equations involving stress state and soil physical properties. Several empirical equations have been suggested to estimate the resilient modulus(George, 2004).

2.6 Determination of Resilient Modulus of Soils

Resilient modulus is the primary material property that is used to characterize the roadbed soil and other structural layers for flexible pavement design in the 1986 and 1993 AASHTO Guide for Design of Pavement Structures. It is mathematically defined as the applied stress (or deviator stress change for triaxial testing of unbound materials) divided by the “recoverable” strain that occurs when the applied repeated-load is removed from the test specimen. Resilient modulus is generally measured in the laboratory using repeated-load triaxial and/or indirect tensile tests depending on the type of material being tested (Von Quintus and Killingsworth, 1998).

The Repeated Load Triaxial (RLT) test is specified for determining the resilient modulus by AASHTO T 294(AASHTO T294-94, 1995): “Resilient Modulus of

Unbound Granular Base/Subbase Materials and Subgrade Soils-SHRP Protocol P 46," and by AASHTO T 307(AASHTO T307, 2003): "Determining the Resilient Modulus of Soils and Aggregate Materials". The RLT test consists of applying a cyclic load on a cylindrical specimen under constant confining pressure (σ_3 or σ_c) and measuring the axial recoverable strain (ε_r). The RLT test setup is shown in Plate I(Titi *et al*., 2006). The system consists of a loading frame with a crosshead mounted hydraulic actuator. A load cell is attached to the actuator to measure the applied load. The soil sample is housed in a triaxial cell where confining pressure is applied. As the actuator applies the repeated load, sample deformation is measured by a set of Linear Variable Differential Transducers (LVDT). A data acquisition system records all data during testing(Titi *et al*., 2006).

Plate I: Repeated Load Triaxial Test Setup (Instron 8802 Dynamic Materials Test System)
Source:(Titi *et al*., 2006)

The resilient modulus determined from the RLT test is defined as the ratio of the repeated axial deviator stress to the recoverable or resilient axial strain expressed as Equation (2.1):

$$M_r = \frac{\sigma_d}{\varepsilon_r} \quad (2.1)$$

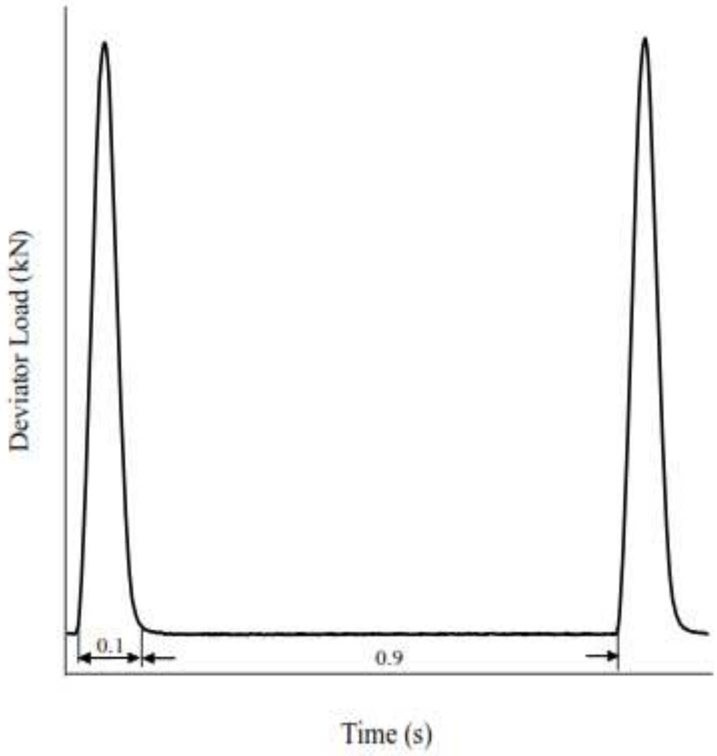
where

M_r is the resilient modulus,

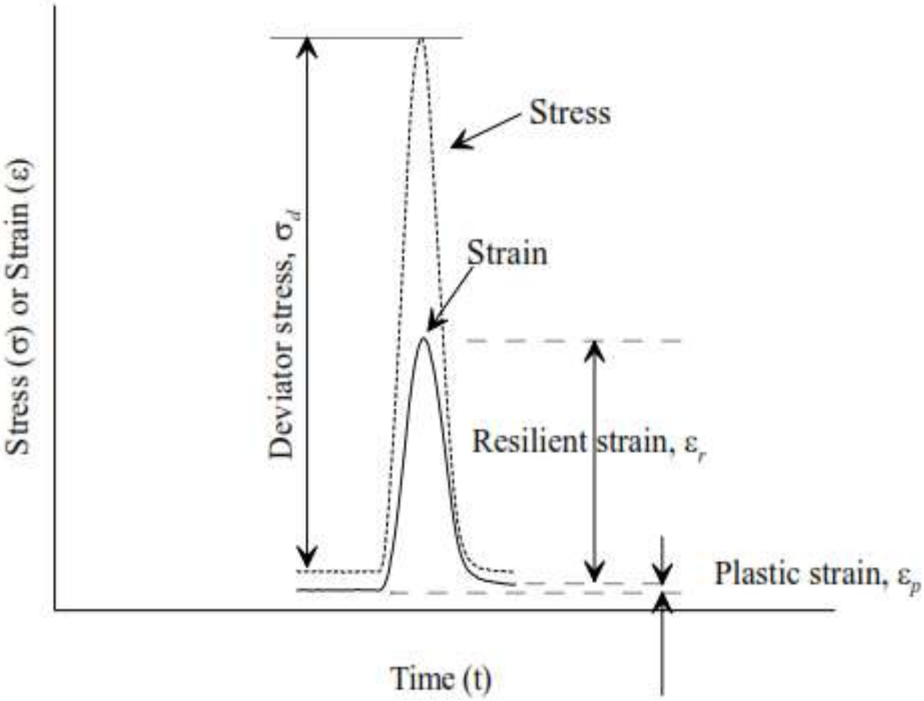
σ_d is the deviator stress (cyclic stress in excess of confining pressure),

and ε_r is the resilient (recoverable) strain in the vertical direction.

Figure 2.3 depicts a graphical representation of the definition of resilient modulus from a repeated load triaxial test.



(a) Shape and duration of repeated load



(b) Stresses and strains of one load cycle

Figure 2.3: Definition of the Resilient Modulus in a Repeated Load Triaxial Test
Source:(Titi *et al*., 2006)

AASHTO provided standard test procedures for determination of resilient modulus using the repeated load triaxial test, which include AASHTO T 292(AASHTO T292-91, 1994), AASHTO T 294 and AASHTO T 307(Titi *et al*., 2006).

The resilient modulus of pavement materials is typically determined using the RLT test.This test requires well trained personnel and expensive laboratory equipment. In addition, it is considered to be relatively time consuming. Therefore, highway agencies tried to seek different alternatives. Various empirical correlations have been used to determine resilient modulus in the last three decades. The resilient modulus of subgrade soils is related to several parameters, such as the SSV, the *R*-value, the CBR, and the Texas triaxial classification value. However, these parameters do not represent the dynamic load behaviour under moving vehicles(Mohammad *et al*., 2007). The standard procedure for obtaining resilient modulus is a RLT test at a constant confining pressure.

There is not a singular resilient modulus value for a particular soil but rather the modulus is a function of the stress state. The standard test produces a range of resilient modulus values in a series of stress conditions(Smolen, 2003).

The resilient modulus test is inherently complicated, time consuming, and expensive. For these reasons, most commercial and design laboratories will not conduct these tests but instead rely on empirical relationships. Therefore, it has been recommended that alternative tests be developed to approximate resilient modulus(Smolen, 2003; Mohammad *et al*., 2007).

The resilient modulus calculated from one set of loading conditions is independent of the loading history of the material. The same specimencan be used for determining the effects on wide range of loading conditions.The interpretation of test results need some mathematical modelling torepresent a useful relationship between deviatoric stresses versus resilient modulus.It has to be demonstrated that low deviatoric stress levels gives a high resilientmodulus and high deviatoric stress gives a low resilient modulus values(Viswanathan, 1989).

The results of resilient modulus testing are required for Level 1 pavement design where a high volume of traffic is expected. Because of the complexity of resilient modulus testing, conducting the test for the other two levels of pavement design, for which traffic volume is relatively low and safety concerns are less intense, has not been recommended.

In Level 1 design/analysis, the MEPDG requires input of the regression constants of the stress-dependent constitutive equation for resilient modulus of a particular unbound material (subgrade soil or base aggregate). This ensures a more accurate assessment of the modulus during the analysis over the design period including seasonal variation and varying stress conditions. Constitutive equation coefficients (*k*-values) are usually obtained from the regression analysis of resilient modulus test data for an actual soil/aggregate sample(Hossain, 2010).

Regardless of project size, it is often difficult to predict and consequently reproduce the in-situ conditions, usually with respect to the state of stress, further complicating the use of resilient modulus testing. Because of this, correlations are desired for estimating resilient modulus, especially for use (or verification of default values) associated with MEPDG Level 2 design/analysis. A common method to predict a resilient modulus value is to use the stress-dependent constitutive equation with the *k*-values estimated from soil index properties through further regression equations. MEPDG Level 3 design/analysis also requires a specific resilient modulus value as input(Hossain, 2010).

2.7 Factors Affecting Resilient Modulus of Subgrade Soils

Factors that influence the resilient modulus of subgrade soils include physical condition of the soil (moisture content and unit weight), stress level and soil type. Many studies have been conducted to investigate these effects on the resilient modulus. The effect of some of these factors on the resilient modulus of subgrade soils is significant (Titi*et al.*, 2006).

The factors that may affect the resilient modulus of granular material was analysed and found that the following factors may have a significant influence on the stress-deformation characteristics under short duration repeated loads: stress level (confining pressure), degree of saturation, dry density, fine content, and load frequency and duration(Sheng, 2010).

Repeated load triaxial test depend on soil gradation, compaction method, specimen size and testing procedure(Zaman*et al*., 1994).

Resilient modulus that range between 14, 000 and 140, 000kN/m^2 can be obtained for the same fine-grained subgrade soil by changing parameters such as stress state or moisture content(Li and Selig, 1994).

The resilient modulus is a stress-dependent soil property as it is a measure of soil stiffness. The most significant loading conditionfactor that affects resilient modulus response is the stress level. Loading characteristics, factors such as stress duration, stress frequency, sequence of load and number of stress repetitions necessary to reach an equilibrium-resilient strain response have little effect on resilient modulus response(Rada and Witczak, 1981).

In general, the increase in the deviator stress results in decreasing the resilient modulus of cohesive soils due to the softening effect. The increase of the confinement results in an increase in the resilient modulus of granular soils. It was reported that the confining pressure has more effect on material stiffness than deviator or shear stress(Lekarp *et al*., 2000).

Laboratory investigations on the effect of stress history on the resilient modulus results showed that the resilient modulus increased with the increase of the repeated number of loads. This increase was mainly attributed to the reduction in moisture content of the soil. However, (Pezo*et al*., 1992; Nazarian and Feliberti, 1993) reported that specimen conditioning affected the resilient modulus of the specimen and indicated that stress history plays an important role in the modulus of soils.

Most laboratory studies on subgrade soils and unbound materials show that the resilient modulus increased with the increase of the confining stress(Rada and Witczak, 1981; Pezo and Hudson, 1994). The effect of an AASHO road test on subgrade soil was summarized and found that the resilient modulus decreased as moisture increased. Resilient modulus of fine-grained soils does not depend on the confining pressure and confining pressures in the upper soil layers under pavements are normally less than 35 kN/m^2(Thompson and Robnett, 1979). In general, the effect of confining stress is more significant in granular soils than in fine-grained soils. For granular materials, the increase in confining pressure can significantly increase the resilient modulus (Rada and Witczak, 1981).

Laboratory resilient modulus tests were conducted on pavement materials to characterize their behaviour under seasonal frost conditions, and to provide input necessary for modelling the materials with the mechanistic pavement design and evaluation procedure. It was observed that the resilient modulus of cohesive soils is significantly influenced by the deviator stress. Also, the resilient modulus of fine-grained soils decreases with the increase of the deviator stress. For granular materials, the resilient modulus increases with increasing deviator stress, which typically indicates strain hardening due to reorientation of the grains into a denser state (Maher *et al*., 2000).

It was found that low clay content and high silt content results in lower resilient modulus values and that low plasticity index and liquid limit, low specific gravity, and high organic content result in lower resilient modulus (Thompson and Robnett, 1979). Other research results indicated that the resilient modulus generally decreased when the amount of fines increases (Lekarp *et al*., 2000). Also, it was noticed that an increase in maximum particle size leads to an increase in the resilient modulus(Janoo and Bayer II, 2001).

2.8 Resilient Moduli Correlations

Simple correlation equations have been reported to predict resilient modulus from standard CBR. The correlation of CBR with resilient moduluswas studied and proposed an empirical relationship was proposed as shown in Equation (2.2) (George, 2004; Putri*et al*., 2012):

$$M_R(N/mm^2) = 10.34 \times CBR \quad (2.2)$$

This correlation is only for fine grained non expansive soils with a soaked CBR < 100% (AASHTO, 1993). The lower and upper bound values of the constant of proportionality ranged between 750 and 3,000, respectively. This equation provides reasonable estimates of resilient modulus for fine-grained soils with a CBR value of 10 or less(George, 2004).

Also, a correlation of CBR with resilient moduluswas proposed by *(*Powell *et al*., 1984)as shown in Equation (2.3)(Putri *et al*., 2012):

$$M_R(N/mm^2) = 17.6 \times CBR^{0.64} \quad (2.3)$$

Thus, the correlation between resilient modulus and CBR developed by (NAASRA, 1979) has been divided into two parts as shown in Equations (2.4) and (2.5)(Putri *et al*., 2012).

For CBR less than 5,

$$M_R(N/mm^2) = 16.2 \times CBR^{0.7} \quad (2.4)$$

Then, for CBR more than 5,

$$M_R(N/mm^2) = 22.4 \times CBR^{0.5} \quad (2.5)$$

A very promising approach for obtaining resilient modulus for use in design, for at least most agencies, is to determine values of resilient modulus using generalized empirical relationships with statistically relevant, easy to measure physical properties of the material. Considering the large variation in resilient modulus along the route and important design changes in moisture with time, the use in design of empirical resilient modulus relationships is considered to be justified. A number of state agencies have already developed generalized resilient modulus relationships for use in design, particularly for cohesive subgrade soils. Statistically based equations, graphs or charts would then be developed for each class of materials for the range of properties routinely used in design within the region of interest(Sheng, 2010).

Different types of correlations are used to estimate the resilient properties of subgrades and bases. Currently, two approaches are followed to analyse resilient moduli test data. One is to develop relationships between resilient moduli values and various soil properties or different in situ test-related parameters. Statistical regression tools are usually adopted for this exercise. The other one is to analyse the resilient moduli data with a formulation that accounts for confining or deviatoric or both stress forms. This formulation usually contains several model constant parameters. Once these parameters are determined, they are correlated with different sets of soil properties. These correlations are termed here as semi-empirical or indirect correlations (NCHRP, 2008).

One of the first studies of the resilient properties of soil with the objective of developing correlation equations for predicting resilient modulus from basic soil test data was developed in 1985(Carmichael and Stuart, 1985). They used the Highway Research Information Service (HRIS) database and developed two regression models, one for fine-grained soils and another for coarse-grained soils. Regression models were developed for individual soil types according to the Unified Soil Classification System. Variables used in the models for coarse-grained soils included moisture content and bulk stress. For fine-grained soils, plasticity index, confining, and deviatoric stresses were used as predictive variables(Rao *et al*., 2012).

Equation (2.6) presents the model for coarse-grain soils.

$$LogM_R = 0.523 - 0.025(w_c) + 0.544(log\theta) + 0.173(SM) + 0.197(GR) \quad (2.6)$$

where,

M_R = Resilient Modulus,

w_c = moisture content, %;

θ = bulk stress (σ_1+σ_2+σ_3),

SM = 1 for *SM* soils (Unified Soil Classification)

= 0 otherwise; and

GR = 1 for *GR* soils (*GM, GW, GC* or *GP*)

= 0 otherwise.

A different equation (Equation 2.7) was derived for fine-grain soils:

$$M_R = 37.431 - 0.4566(PI) - 0.6179(w_c) + 0.1424(P_{200}) + 0.0012(\sigma_3) - 0.0022(\sigma_d) + 36.722(CH) + 17.097(MH) \quad (2.7)$$

where,

PI = plasticity index, %;

P_{200} = percentage passing #200 sieve;

σ_3 = confining stress, N/mm^2;

σ_d = deviator stress, N/mm^2;

CH = 1 for *CH* soil

= 0 otherwise (for *MH*, *ML* or *CL* soil); and

MH = 1 for *MH* soil

= 0 otherwise (for *CH*, *ML* or *CL* soil).

Drumm *et al*., (1990)conducted a resilient modulus study on cohesive soils based on AASHTO soil classification. The authors tried to establish a simple procedure for modulus testing. They used deviator stress as the main model parameter. The model coefficients were derived as functions of liquid limit of soil, degree of saturation, and unconfined compressive strength(Rao *et al*., 2012).Their result showed that unconfined compressive strength, q_u, is a better property to predict resilient modulus. Accordingly, they correlated the soil index properties and the initial tangent modulus obtained from unconfined compression test to the resilient modulus. A statistical model developed with a nonlinear relationship between resilient modulus and the deviator stress is as shown in Equation (2.8):

$$M_R(N/mm^2) = 6.89476\left(\frac{a^{'} + b^{'}\sigma_d}{\sigma_d}\right) for \sigma_d > 0 \tag{2.8}$$

where,

$$a^{'} = 318.2 + 0.0023(q_u) + 0.73(\%Clay) + 2.26(PI) - 0.00004(\gamma_s) - 2.19(S) - 0.304(P_{200}) \tag{2.9}$$

$$b^{'} = 2.10 + 0.00039(1/a) + 0.0007(q_u) + 0.09(LL) - 0.10(P_{200}) \tag{2.10}$$

q_u = unconfined compressive strength, N/mm^2;

$1/a$ = initial tangent modulus, N/mm^2, obtained from unconfined compression tests;

%Clay = percent clay;

LL = liquid limit, %;

S = degree of saturation; and

γ_s = dry density, N/mm^2.

It was concluded that a similar relationship could be established for soils other than those investigated and might be helpful to agencies that lack the capability for complex repeated load testing.

Two resilient modulus regression models, one each for fine-grained and coarse-grained Mississippi soils, were developed by George(George, 2004). Variables used to predict fine-grained soil resilient modulus included soil dry density, liquid limit, moisture content, and percentage passing the No. 200 sieve. Variables for the coarse-grained soil resilient modulus model included dry density, moisture content, and percentage passing the No. 200 sieve.The two models are presented in Equations (2.11) and (2.12)(Rao *et al.*, 2012):

Fine-grain soil:

$$M_R(MN/m^2) = 16.75((LL/w_c\gamma_{dr})^{2.06} + (P_{200}/100)^{-0.59})(2.11)$$

Coarse-grain soil:

$$M_R(MN/m^2) = 307.4(\gamma_{dr}/w_c)^{0.86}(P_{200}/logc_u)^{-0.46}(2.12)$$

where,

γ_{dr} = dry density/maximum dry density; and

c_u = uniformity coefficient

The resilient modulus has been predicted using a two-step approach. First, models to predict parameters k_1, k_2, and k_3 of the constitutive equation are developed. Next, the constitutive equation is used to estimate the resilient modulus. Several researchers (Von Quintus and Killingsworth, 1998; Dai and Zollars, 2002; Santha, 1994)developed prediction equations by regressing the coefficients of selected constitutive equations and relating them to soil physical properties. It was observed that the most influential parameters are moisture content, liquid limit, plasticity index, and percent passing the No. 200 sieve(Rao *et al*., 2012).

2.9 Resilient Modulus Constitutive Models

The concept of resilient modulus has been used to explain the nonlinear stress-strain characteristics of subgrade soils. During the past two decades, several constitutive models have been proposed by many researchers for modelling resilient moduli of soils and aggregates. No stress or deformation analysis can be meaningful unless a correct constitutive equation describing the actual behaviour of the material has been used in the analysis(NCHRP, 2008).

Mathematical models are generally used to express the resilient modulus of subgrade soils such as the bulk stress model and the deviatoric stress model. These models were utilized to correlate resilient modulus with stresses and fundamental soil properties. A valid resilient modulus model should represent and address most factors that affect the resilient modulus of subgrade soils(Titi*et al*., 2006).

Several other models were reported in the literature, which use both stresses (either confining and deviatoric stresses or bulk or octahedral stresses) that are functions of confining and deviatoric stresses.The most general form of a three-parameter model is as shown in Equation (2.13)(Ooi *et al*., 2006):

$$M_R = k_1 P_a [f(c)]^{k_2} [g(s)]^{k_3} \quad (2.13)$$

where

$f(c)$ is a function of confinement

$g(s)$ is a function of shear and

k_1, k_2, and k_3 are constants

The effects of confinement in these models can be expressed in terms of the minor principal stress (σ_3), bulk stress (θ), or octahedral stress (σ_{oct} = θ/3), while the parameter options for modelling the effects of shear include the deviatoric stress or octahedral shear stress (τ_{oct}). The three-parameter models represented by the Equation (2.13) are more versatile and apply to all soils (NCHRP, 2008).

2.9.1 Uzan's three-parameter resilient modulus constitutivemodel

Uzan, (1985) studied and discussed different existing models for estimating resilient modulus. The Uzan equation was developed as a combination of the bulk and deviator stress models in an effort to improve the predicted response of resilient modulus test results by including both axial and shear effects.He developed a model to overcome the bulk stress model limitations by including the deviatoric stress to account for the actual field stress state. The model defined the resilient modulus as shown in Equation (2.14)(Uzan, 1985):

$$M_R = k_1 \theta^{k_2} {\sigma_d}^{k_3} \quad (2.14)$$

By normalizing the resilient modulus and stresses in the above model, it can be written as shown in Equation (2.15)(Titi*et al*., 2006):

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a}\right)^{k_3} \quad (2.15)$$

where

P_a is the atmospheric pressure(NCHRP, 2008).

Uzan also suggested that the above model can be used for all types of soils. By setting k_3 to zero the bulk model is obtained, and the semi-log model can be obtained by setting k_2 to zero(Titi*et al*., 2006).

2.9.2 Witczak and Uzan'sthree-parameter resilient modulus constitutive model

An equation similar to Uzan's model using the octahedral shear stress instead of the deviator stress was developed by Witczak and Uzan as shown in Equation (2.16)(Witczak and Uzan, 1988):

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a}\right)^{k_3} \text{ (2.16)}$$

where

σ_1 = major principal stress

σ_2 = intermediate principal stress = σ_3 for M_R test on cylindrical specimen.

$$\tau_{oct} = \frac{1}{3}((\sigma_1 - \sigma_2)^2(\sigma_2 - \sigma_3)^2(\sigma_3 - \sigma_1)^2)^{1/2} \text{(2.17)}$$

This formulation is recommended in the 1993 AASHTO design guide (NCHRP, 2008).

2.9.3 Pezo's three-parameter resilient modulus constitutive model

An equation similar to Uzan's model using the confining pressure instead of the bulk stress was recommended by Pezo as shown in Equation (2.18)(Pezo, 1993):

$$M_R = k_1 P_a \left(\frac{\sigma_3}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a}\right)^{k_3} \text{ (2.18)}$$

2.9.4 Ni et al'sthree-parameter resilient modulus constitutive model

An equation similar to Pezo's model using the confining pressureand deviator stress in a three-parameter formulation was recommended by Ni et al as shown in Equation (2.19)(Ni *et al*., 2002):

$$M_R = k_1 P_a \left(1 + \frac{\sigma_3}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{P_a}\right)^{k_3} \text{ (2.19)}$$

The three models (Equations 2.16, 2.18, and 2.19) predict zero resilient modulus when a confining pressure of zero is used in those formulations. Hence, the attributes are revised in Ni *et al*. (2002), such that they will work for a wide range of stresses(NCHRP, 2008).

2.9.5 Ooi et al'sthree-parameter resilient modulus constitutive model

Ooi et al slightly modified the equation recommended by Ni et al using the bulk stress, octahedral shear stressand deviator stress in a three-parameter formulation into two models as shown in Equations (2.20) and (2.21)(Ooi *et al*., 2004):

$$M_R = k_1 P_a \left(1 + \frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{P_a}\right)^{k_3} \quad (2.20)$$

$$M_R = k_1 P_a \left(1 + \frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{k_3} \quad (2.21)$$

2.9.6 NCHRP three-parameter resilient modulus constitutive model

An equation similar to Ooi et al's model using the octahedral shear stress and bulk stress was recommended by the NCHRP project 1-28 A as shown in Equation (2.22):

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{k_3} \quad (2.22)$$

This expression is a simplification of the five-parameter model (NCHRP, 2008; Titi*et al*., 2006).

2.10 Concluding Remarks

Resilient modulus of subgrade soil is an important material property, a requisite parameter to input in the pavement design equation, generally determined in the laboratory byperforming a repeated load triaxial test procedure. Because the test is complex and time consuming several user agencies now estimate designresilient modulus from correlation equations developed from soil physical properties. A cursory study of the equations suggest that bulk, confiningand deviator stresses describe the nonlinear behaviour of soil, the resilient modulus constitutive models provided a better representation of laboratory measurements of resilient modulus and equations suggest that soil index properties such as material passing No. 200 sieve, Atterberg limits, moisture and dry density significantly affect resilient modulus.

2.11 Statistical Analysis

The coefficient of multiple determination was used as a primary measure to select the best correlation. However, a high R^2 does not necessarily imply that the regression model is a good one. Adding a variable to the model may increase R^2 (at least slightly) whether the variable is statistically significant or not. This may result in poor predictions of new observations. The significance of the model and individual regression coefficients were tested for each proposed model. In addition, the independent variables were checked for multi-collinearity to insure the adequacy of the proposed models (Titi*et al.*, 2006).

2.11.1 Selection of best subset of regressors for a model (C_p)

A statistical term to select the best subset of regressors for a model and an indicator of the collinearity of a regression model. Mallows' C_p is often used as the criterion for selecting the most appropriate sub-model of p regressors (or independent variables) from a full model of *k* regressors, $p<k$. The C_p term that is used in a step-wise regression process helps avoid an over-fit model by identifying the best subset of only the important predictors of the dependent variable. The relationship for C_p is as shown in Equation (2.23) (Rao*et al.*, 2012).

$$C_p = (n-p)\frac{MSE_r}{MSE_f} - n + 2p \tag{2.23}$$

where

n = the sample size

MSE_r = the mean square error for the regression for the smaller model of p regressors and is expressed as shown in Equation (2.24):

$$MSE_r = \sum_{i=1}^{n} (y_i - y_{ri})^2 \quad (2.24)$$

MSE_f is the mean square error for the regression on the full model of k regressors.

2.11.2 Variance Inflation Factor (VIF)

A statistical term to evaluate the multicollinearity of the model (i.e., it tracks the interaction effects of the regressors identified). Generally, VIF can be regarded as the inverse of tolerance. The square root of VIF indicates how much larger the standard error is compared with what it would be if that variable is uncorrelated with the other independent variables in the equation. If y is regressed on a set of x variables x_1 to x_k, VIFs of all x variables should be created in the following manner:

For variable x_j, VIF is the inverse of $(1 - R^2)$ from the regression of x_j on the remainder of the x variables. In other words, x_j regressed on $x_1 \ldots x_j - 1$, $x_{j+1} \ldots x_k$, produces a regression with R^2 as R_j^2. The relationship for VIF is asshown in Equation (2.25):

$$VIF(x_j) = \frac{1}{(1 - R_j^{\,2})} \quad (2.25)$$

VIF is always greater than 1. A VIF value of 10 indicates that 90 percent of x_j is not explained by the other x variables. A common rule of thumb is that if VIF for any variable is greater than 5, multicollinearity exists for that variable and should be excluded from the model. However, in cases where the parameter is either known to correlate well or other variables do not provide a reasonable model, a cut-off value of 10 is acceptable but less preferred(Rao *et al*., 2012).

2.11.3 *p*-value

A probability calculation to ascertain the significance of the regressor in the equation. The *p*-value should be limited to the level of confidence desired in the model.Typically, aconfidence level of 95 percent is used, such that the values should remain below 0.05. However, in rare cases, this sometimes is limited to 0.1(Rao *et al*., 2012).

2.11.4 Coefficient of multiple determination(R^2)

A statistic that indicates the goodness of fit of a model and describes how closely the regression line fits the data points.R^2 is the coefficient of determination and is the square of the sample correlation coefficient computed between the outcomes and their predicted values, or, in the case of simple linear regression, between the outcome and the values being used for prediction. R^2 values vary from zero to 1 and are expressed as a percentage. An R^2 of x percent indicates x percent of the variation in the response variable can be explained by the explanatory variable, and $(100 - x)$ percent can be explained by unknown variability. The higher the value of this term, the greater the predictive ability of the model. It is the most commonly used statistic to evaluate the quality of fit achieved with a model. From the standpoint of using R^2 to select a model, while relationships with higher values are desirable, it is not to be treated as the ultimate criterion to establish the model. R^2 needs to be interpreted with reasonable caution and needs to be combined with the information from the other statistical parameters discussed in this section. In fact, it is not the first check to select a model; instead, it should serve as the final check to establish the model. The statistical parameters discussed previously do not individually optimize a model; instead, these parameters need to be evaluated in combination to derive the most accurate model. Furthermore, it is imperative in establishing a model that both statistical and engineering aspects be balanced. The accuracy of the model needs to be verified for technical/engineering validity by evaluating each variable in the model and confirming that the observed trends are as expected (verified in literature) and that the effect of the independent variable on the predicted variable is reasonable (verified through sensitivity analyses)(Rao *et al*., 2012).

2.11.5 Test for significance of the model

The significance of the model is tested using the *F*-test to insure a linear relationship between k_i and the estimated regression coefficients (independent variables). For testing hypotheses on the model (Titi*et al*., 2006):

H_0: $\beta_1 = \beta_2 = \text{---} = \beta_k = 0$; H_a: $\beta_i \neq 0$ for at least one i

where

H_0 is the null hypothesis, andH_a is the alternative hypothesis.

The test statistic is as shown in Equation (2.26):

$$F_0 = \frac{SS_R/p}{SS_E/(n-p-1)} \quad (2.26)$$

where

SS_R is the sum of squares due to regression,

SS_E is the sum of squares due to errors,

n is the number of observations and

p is the number of independent variables

H_0 is rejected if $F_0 > F_{\alpha,p,n-p-1}$

where, α is the significance level (used as 0.05 for all purposes in this study).

2.11.6 Test for significance of individual regression coefficients

The hypotheses for testing the significance of individual regression coefficient β_i is based on the *t*-test and is given by(Titi *et al*., 2006):

H_0: $\beta_i = 0$; H_a: $\beta_i \neq 0$

The test statistic is as shown in Equation (2.27):

$$\hat{t}_0 = \frac{\widehat{\beta_i}}{\sqrt{\hat{\sigma}^2 C_{ii}}} \quad (2.27)$$

where

C_{ii} is the diagonal element of $(X^{/}X)^{-1}$ corresponding to $\widehat{\beta_i}$ (estimator of β_i),

$\hat{\sigma}$ is estimator for the standard deviation of errors,

X (n,p) is matrix of all levels of the independent variables,

$X^{/}$ is the diagonal X matrix,

n is the number of observations, and

p is the number of independent variables

H_0 is rejected if $/t_0/ > t_{\alpha/2,\mathrm{n}-p-1}$

CHAPTER THREE

MATERIALS AND METHODS

3.1 Brief Description of Study Area

The geographical regions designated for testing during Phase I of the Nigerian Trunk Road Study were re-evaluated and expanded to provide better sampling of the pavement materials, environmental conditions, and soils types in Nigeria. The entire Federal Trunk Road Network was divided into six regions with similar soil types and climate. The six regions are shown in Table 3.1. The field experimental program was designed to characterize thein-situ load carrying capacity of pavement layers on selected types of pavement soils in Nigeria.

Table 3.1: Six Testing Regions for Pavement Design Procedure in Nigeria

Region	States
1	Kaduna, Kano
2	Bauchi, Borno
3	Benin, Eastern Kwara, Lagos, Ogun, Ondo, Oyo
4	Federal Capital Territory, Western Kwara, Niger, Sokoto
5	Benue, Yola, Plateau
6	Anambra, Cross River, Imo, Rivers

Source:(Claros *et al*., 1986)

A series of tests were conducted on several flexible pavement structures inNigeria to achieve the set objectives.The locations were evenly spread to appropriately represent different soil conditions in Nigeria. Table 3.2 shows the Master Test Sectionsfor Pavement Design Procedure in Nigeria(Claros *et al*., 1986).The Master Test Sections (MTSs) were chosen to fill the factorial experiment and also took into account the soil type and history, pavement layer homogeneity, layer thickness and field operational considerations. The programme was designed to obtain information on properties of subgrade materials, the traffic loading to which the highways are subjected and the relative performance of the highways.

Table 3.2: Master Test Sectionsfor Pavement Design Procedure in Nigeria

MTS Number	Route No.	Distance from First Node (km)	From	To
			Node Number and Name	
1-1	A236	35	241 Zaria (A126)	394 A236/State Road
1-2	A11	4	239 Katabu	353 Pambegua
1-3	A2	40	237 Kaduna (A235)	711 Kaduna-Niger Border
1-4	A235	11	237 Kaduna (A235)	584 Kachia
1-5	A125	11	238 Kaduna	346 S.B. Gwari
1-6	A2	2	239 Katabu	460 Zaria (A236)
2-1	A4	5	277 Maiduguri (F256	295 Biu
2-2	A3	8	310 Potiskum	276 Damaturu
2-3	A4	7	295 Biu	277 Maiduguri (F256)
2-4	A3	80	278 Maiduguri (A4)	279 Dikwa
2-5	A345	2	270 Bauchi (A345)	374 Gombe
2-6	A3	7	270 Bauchi (A345)	720 Plateau-Bauchi State Bor
7-1 (SS)	A3	25	277 Maiduguri (F256	276 Damaturu
3-1	A2	2	232 Lokoja (A123)	233 Lokoja (A233)
3-2	A121	1	299 Ijebu-ode 3	298 Shagamu 3
3-3	A122	10	302 Oluku	734 Bendel-Ondo State Bord
3-4	A122	27	207 A1/State Road	325 Ikire
3-5	A121	18	302 Oluku	744 Bendel-Ondo State
3-6	A2	20	224 Efferun	Border 225 Sapele
4-1	A125	20	549 Ushiba	735 Kaduna-Niger Border
4-2	A2	40	236 Abuja (A124)	711 Kaduna-Niger Border
4-3	A1	130	707 Niger-Sokoto Bo	214 Kagara (A1/A125)
4-4	A124	4	212 A1/(A124)	341 Bida
4-5	A124	20	236 Abuja (A124) (S	342 Lambata
4-6	A1	3	214 Kagara (A1/A12	522 Bokani
5-1	A3	9	268 Bukuru	719 Kaduna State Border
5-2	A3	32	262 Makurdi	717 Plateau Border
5-3	A4	20	290 Wukari	291 Jalingo 1
5-4	A236	16	269 Jos 1	355 Fustum Mata
5-5	A4	12	289 Katsina Ala A12	534 Zaki Biam
5-6	A3	14	262 Makurdi	261 Aliade
6-1	A3	18	258 9 Milestone Corn	307 Oji River
6-2	A343	3	256 Emene	587 Nkalagu
6-3	A3	10	258 9 Milestone Corn	307 Oji River
6-4	A343	27	256 Emene	587 Nkalagu
6-5	F229	20	429 F229/F103 (PH 3	503 Elele
6-6	A6	6	312 Owerri 1	313 Umu Uvo

Source:(Claros *et al*., 1986)

The soils tested in the course of this study for the correlation of basic soil properties with the resilient modulus equations (models) were selected to provide a general representation of typical subgrade soils found in Kano-Kaduna region (MTS 1). Plate II shows the Master Test Section (MTS) 1. These MTS were selected in connection with a study by Claros and his research team in 1986, to develop procedure for overlay design method.

Plate II: Map Showing Roads in Master Test Section 1 of Nigeria
Source: Google map

The MTS 1 consist of six sections as shown in Table 3.3. Three (3) samples each was obtained at 500mm depth from each of the six sections. Plate III showed the picture of the researcher obtaining soil samples from the study area. A total of eighteen (18) soil samples were obtained and tested in the laboratory to determine their basic soil index properties.

Table 3.3: Description of Master Test Section (MTS) 1

MTS	Route Number	Distance from First Node (km)	Node Number and Name
1 – 1	A236	35	241 Zaria (A126) to 394 A236/State Road
1 – 2	A11	4	239 Katabu to 353 Pambegua
1 – 3	A2	40	237 Kaduna (A235) to 711 Kaduna-Niger Border
1 – 4	A235	11	237 Kaduna (A235) to 584 Kachia
1 – 5	A125	11	238 Kaduna to 346 S.B. Gwari

1 – 6	A2	2	239 Katabu to 460 Zaria (A236)

Source: (Claros*et al*., 1986)

3.2 Laboratory Testing Program on the Soil Samples from MTS 1

The soil samples obtained from the six sections within MTS 1 were tested in the Soil and Geology laboratory in Civil Engineering Department of Kaduna Polytechnic, Kaduna to determine their basic physical properties. Plate IV showed the picture of part of the soil samples taken to the soil laboratory for test. The properties include physical properties and compaction characteristics, Unconfined Compressive Strength and the California Bearing Ratio.

Plate III: The Researcher Obtaining Subgrade Soil Samples from MTS 1

3.2.1 Physical properties and compaction characteristics of the soil samples

The laboratory experimental program was designed to evaluate the physical properties and compaction characteristics of the subgrade soil samples. Evaluation of soil properties and identification and classification of the soil samples are important steps to accomplish the research objective since the resilient modulus is highly influenced by soil properties (Ping *et al*., 2000; Ping and Ling, 2008).

Plate IV: Soil Samples obtained from MTS 1 in the Soil Laboratory

Samples of soil wassubjected to laboratory experimental program which includes the test materials and the physical property analysis to achieve the set objectives.Soil index properties are used extensively by engineers to discriminate between the different kinds of soil within a broad category, e.g. clay will exhibit a wide range of engineering properties depending upon its composition. Classification tests to determine index properties will provide the engineer with valuable information when the results are compared against empirical data relative to the index properties determined. The subgrade soil samples was subjected to standard laboratory tests using standard test procedures (AASHTO, 2003; BS 1377, 1990) to determine their physical properties and compaction characteristics. The soil index properties determined in the laboratory include Atterberg limits, soil gradationand the specific gravity tests. The subgrade soil samples was then classified using Unified Soil Classification system and AASHTO Soil Classification systemincluding group index (GI).The compaction characteristics involve the determination of the maximum dry density, and the optimum moisture content of the subgrade soil samples in accordance with the standard testing procedures.Also obtained was the Natural Moisture Content tests were conducted on the soil samples in accordance with AASHTO T265 testing procedures(AASHTO T265, 2003).

3.2.1.1 *Determination ofAtterberg limits of the soil samples*

Atterberg limits were determined using the testing procedures(AASHTO T89, 2003; ASTM D 4318, 2000; BS 1377, 1990; AASHTO T90, 2003). The Liquid Limit was obtained by plotting the flow curve of the soil samples.The flow curve was made by plotting the individual moisture contents and the number of blows required to close the grooves, on a semi-logarithmic graph, for each of the points. A best-fit straight line was then drawn through the points. The moisture content corresponding to the intersection of the best-fit line with the 25-tap line was recorded as the Liquid Limit.

The plastic limit was determined by alternately pressing together and rolling into a 3.2-mm diameter thread a small portion of plastic soil until its water content was reduced to a point at which the thread crumbles and can no longer be pressed together and re-rolled. The water content of the soil at this point was reported as the plastic limit.

The plasticity index was calculated as the difference between the liquid limit and the plastic limit. The PL and PI are to be reported to the nearest whole number.The plasticity index of the subgrade soil samples was calculated using Equation (3.1):

$$PI = LL - PL \qquad (3.1)$$

3.2.1.2 *Determination oflinear shrinkage of the soil samples*

The linear shrinkage was determined in accordance with the testing procedures(AASHTO T92, 2003; BS 1377, 1990; ASTM D 427, 1999). On cooling, the length of the soil samples was measured with a ruler and the linear shrinkage was calculated using Equation (3.2):

$$Linear\ Shrinkage = \frac{(Initial\ length - Dried\ length)}{Initial\ length} \times 100 \qquad (3.2)$$

3.2.1.3 *Determination ofparticle size distribution of the soil samples*

The analysis of soils by particle size provides a useful engineering classification system from which a considerable amount of empirical data can be obtained. Soil gradationwas conducted on the soil samples in accordance with testing procedures(AASHTO T88, 2003; BS 1377, 1990; ASTM D 422, 1998).

3.2.1.4 *Determination ofspecific gravity of the soil samples*

The specific gravity tests was conducted on the s soil samples in accordance with testing procedures(AASHTO T100-03, 2004; BS 1377, 1990; ASTM D 854, 2000).The specific gravity is calculated using Equation (3.3):

$$G_s = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_1)} \quad (3.3)$$

3.2.1.5 *Determination ofsoilclassification of the soil samples*

The soil samples was classified using Unified Soil Classification System (USCS) and AASHTO Soil Classification.The USCS was in accordance with American Society for Testing and Materials (ASTM) standard(ASTM D 2487-98, 2000).The AASHTO Soil Classification system was in accordance with AASHTO M145(AASHTO M145-91, 2004).

3.2.1.6 *Determination ofcompaction characteristics of the soil samples*

The soil samples was also subjected to standard compaction test to determine their compaction characteristics in accordance with the standard testing procedures(AASHTO T99, 2003; ASTM D 698, 2000; BS 1377, 1990). The bulk density for each compacted layer was calculated using Equation (3.4):

$$\rho = \frac{m_2 - m_1}{1000} \quad (3.4)$$

The dry density was also calculated using Equation (3.5):

$$\rho_d = \frac{100\rho}{(100 + w)} \quad (3.5)$$

where

w is the moisture content of each compacted layer.

Maximum dry density and optimum moisture content for the soil or soil-aggregate mixture was determined by plotting the relationship between dry density and moisture content, and the points are connected with a smooth line to develop a moisture-density curve. The coordinates of the peak of the curve was taken as the MDD and the OMC of the soil.

3.2.1.7 *Determination ofnatural moisture content of the soil samples*

Natural moisture content tests were conducted on the subgrade soil samples in accordance with testing procedures(AASHTO T265, 2003; ASTM D 2216, 1999; BS 1377, 1990). The natural moisture content (as collected from the site) is calculated as the average of the three oven dried samples given by Equation (3.6):

$$w = \frac{M_2 - M_3}{M_3 - M_1} \times 100 \qquad (3.6)$$

where *w* is the moisture content in percentage.

3.2.2 Determination ofCalifornia bearing ratio of the soil samples

The CBR test is commonly used to determine the suitability of a soil as a subgrade or subbase for highway and runway design and construction(Putri, *et al*., 2012). The test was carried out in accordance with standard testing procedures (AASHTO T193, 2003; ASTM D 1883, 1999; BS 1377, 1990). Results of stress (load) versus penetration depth were plotted to determine the CBR for each specimen. The CBR at the specified density was determined from the graph of CBR versus dry unit weight.The CBR value was determined at penetration 2.5 *mm* penetration or 5.0 *mm* penetration. The CBR was calculated using Equation (3.7).

$$CBR = \frac{Measured\ load}{Standard\ load} \times 100\% \qquad (3.7)$$

where

Standard load = 13.24 *kN* of 2.5 *mm* penetration, and

Standard load = 19.96 *kN* of 5.0 *mm* penetration.

3.2.3 Determination ofunconfined compressive strength of the soil samples

Unconfined compressive strength tests were conducted on the subgrade soil samples immediately after samplecompaction in accordance with the testing procedures(AASHTO T208-96, 2004; ASTM D 2166, 2000; BS 1377, 1990). The UCS was determined as a relationship between the failure load and the surface area of specimen.It was taken as the maximum load attained per unit area during the performance of a test. A continuous stress-strain response was recorded to produce a complete stress-strain diagram. Samples were prepared using the Proctor hammer (Proctor).

The failure load was calculated using Equation (3.8):

$$failure\ load\ (P) = R \times C_r \times (100 - \varepsilon\%) \times 1000 \quad (3.8)$$

where

R = load ring reading at strain ε

C_r = Mean calibration of load ring

ε = strain percent

The axial strain, ε, was obtained to the nearest 0.1 %, for the applied load was calculated using Equation (3.9):

$$\varepsilon = \frac{\Delta L}{L_0} \quad (3.9)$$

where

ΔL = length change of specimen as read from deformation indicator, mm, and

L_0 = initial length of test specimen, mm

The average cross-sectional area, A, for the applied load was obtained using Equation (3.10).

$$A = \frac{A_0}{(1 - \varepsilon)} \quad (3.10)$$

where:

A_0 = initial average cross-sectional area of the specimen, mm^2, and

ε = axial strain for the given load, %.

The compressive stress, σ_c, to three significant figures, or nearest 1 kN/mm^2, for the applied load is obtained using Equation (3.11),

$$\sigma_c = \frac{P}{A} \quad (3.11)$$

where:

P = the applied load, 1 kN/mm^2,

A = corresponding average cross-sectional area mm^2.

3.3 Evaluation of Resilient Modulus Model for Nigerian Soils

The resilient modulus model is a general constitutive equation that was developed by researchers for implementation in the Design of New and Rehabilitated Pavement Structures. The resilient modulus model can be used for all types of subgrade materials. The resilient modulus, the bulk stress, the octahedral shear stress and the deviator stress are normalized in theresilient modulus models by the atmospheric pressure. This will result in non-dimensional model parameters.

Statistical analysis based on multiple linear regression was utilized to determine the resilient modulus model parameters k_1, k_2 and k_3. Microsoft excel was used to perform the analysis.The resilient modulus is treated as the dependent variable, while bulk, the octahedral shear and the deviator stresses are used as the independent variables for both the fine and granular soils. The analysis was carried out for each soil type to evaluate the model parameters (k_1, k_2 and k_3) from the results of the 15 stress combinations for fine soils applied during repeated load triaxial test (15 load sequences for fine soils).

The seven resilient modulus models developed by several researchers were evaluated in this work. Based on the results of repeated triaxial loading tests performed on 56 fine-grained and 22 coarse-grained Nigeriasoils by Claros *et al*, (1986), its results were used to test the resilient modulus constitutive modelsproposed by Uzan, Witczak and Uzan, Pezo, Ni *et al*,Ooi *et al*, and NCHRP. The stress ratios used in describing the soils behaviourwere bulk, confiningand deviator stresses log-log model.

In order to determine the model parameters (k_1, k_2 and k_3), the resilient modulus models was transformed.Table 3.4 showed the summary of the transformed resilient modulus constitutive models considered in the research.

Table 3.4: Transformed Resilient Modulus Constitutive Equations

Resilient Modulus Constitutive Model	Transformed Resilient Modulus Constitutive Equation	Equation Number
Uzan(Uzan, 1985)	$log\left(\frac{M_R}{P_a}\right) = logk_1 + k_2 log\left(\frac{\theta}{P_a}\right) + k_3 log\left(\frac{\sigma_d}{P_a}\right)$	Equation 3.12
Witczak and Uzan(Witczak & Uzan, 1988)	$log\left(\frac{M_R}{P_a}\right) = logk_1 + k_2 log\left(\frac{\theta}{P_a}\right) + k_3 log\left(\frac{\tau_{oct}}{P_a}\right)$	Equation 3.13
Pezo(Pezo, 1993)	$log\left(\frac{M_R}{P_a}\right) = logk_1 + k_2 log\left(\frac{\sigma_3}{P_a}\right) + k_3 log\left(\frac{\sigma_d}{P_a}\right)$	Equation 3.14
Ni et al(Ni, et al., 2002)	$log\left(\frac{M_R}{P_a}\right) = logk_1 + k_2 log\left(1 + \frac{\sigma_3}{P_a}\right) + k_3 log\left(1 + \frac{\sigma_d}{P_a}\right)$	Equation 3.15
Ooi et al(Ooi, et al., 2004)	$log\left(\frac{M_R}{P_a}\right) = logk_1 + k_2 log\left(1 + \frac{\theta}{P_a}\right) + k_3 log\left(1 + \frac{\sigma_d}{P_a}\right)$	Equation 3.16
Ooi et al(Ooi, et al., 2004)	$log\left(\frac{M_R}{P_a}\right) = logk_1 + k_2 log\left(1 + \frac{\theta}{P_a}\right) + k_3 log\left(1 + \frac{\tau_{oct}}{P_a}\right)$	Equation 3.17
NCHRP(NCHRP, 2008)	$log\left(\frac{M_R}{P_a}\right) = logk_1 + k_2 log\left(\frac{\theta}{P_a}\right) + k_3 log\left(1 + \frac{\tau_{oct}}{P_a}\right)$	Equation 3.18

As shown in Table 3.4, the resilient modulus model proposed by Uzan (Equation 2.15) was transformed to Equation (3.12); the resilient modulus model proposed by Witczak and Uzan (Equation 2.16) was transformed to Equation (3.13); the resilient modulus model proposed by Pezo (Equation 2.18) was transformed to Equation (3.14); the resilient modulus model proposed by Ni et al(Equation 2.19) was transformed to Equation (3.15); the resilient modulus model proposed by Ooi et al(Equations2.20 and 2.21) were transformed to Equations (3.16) and (3.17) respectively and the resilient modulus model proposed by NCHRP (Equation 2.22) was transformed to Equation (3.18).

3.4 Correlations of Model Parameters with Basic Soil Properties

Regression models describe a functional relationship between two or more variables in mathematical formula. First-order regression models exist when the relationship between the dependent and independent variables are linear in nature. Regression is one of the most widely usedand powerful analysis techniques used formodel development. Multiple linear regressionattempts to model the relationship betweentwo or more explanatory variables and aresponse variable by fitting a linear equationto observed data.The general multiple linear regression model is expressed as(Sandefur, 2003; Owolabi *et al.*, 2012; Owolabi and Abiola, 2011):

$$Y_i = \beta_0 + \beta_1 X_{i,1} + \beta_2 X_{i,2} + \cdots + \beta_{p-1} X_{i,p-1} + \epsilon_i \quad (3.19)$$

where:

Y_i = dependent variables

β_0, β_1,β_2 …β_{p-1}= regression parameters

$X_{i,1}$, $X_{i,2}$… $X_{i,p-1}$= independent predictor variable

ϵ_i = independent error term

The error term, ϵ_i is usually assumed to be normally distributed with a constant variance for all observations, X_i(Sandefur, 2003).

The resilient modulus model parameters k_1, k_2 and k_3 were determined for all soil types. These parameters were then correlated to fundamental soil properties using regression analysis. Soil properties of the same materials were incorporated in the regression constants of the models. The values of resilient modulus model parameters (k_1, k_2 and k_3) were alternatively used as dependent variables while various fundamental soil properties were treated as independent variables. Various combinations of soil properties (independent variables) were used in the regression analysis.The general multiple linear regression model is expressed as:

$$k_i = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_k x_k + \epsilon \qquad (3.20)$$

where:

k_i = the dependent variable for the regression, (model parameters k_1, k_2 and k_3)

β_0 = intercept of the regression plane

β_i = regression coefficient

x_i = the independent or regressor variable

ϵ = random error

3.5 Selection of Basic Soil Properties

A preliminary list of basic soil properties was prepared for developing predictive models based on inputs required for the NEMPADS and their level of significance in performance prediction. The soils are classified broadly as unbound materials (include both coarse-grained and fine-grained soils) which have different mechanical behaviour in response to applied stress states.

The resilient modulus is used to evaluate the stiffness of bound/unbound materials. Factors that affect resilient modulus are stress state, soil type and the environmental conditions of the soil that influence the soil physical state (unit weight and moisture content). Stress state is expressed in the resilient modulus model by including bulk and octahedral stresses. The soil type and the current soil physical condition should be included in attempted correlations in order to obtain valid estimation/prediction of the resilient modulus.Sets of independent variables are specified to reflect soil type and current soil physical condition. Independent variables available from basic soil testing that represent soil type and current soil physical conditionand combinations of variables were also included.

The goal of the regression analysis is to identify the best subset of independent variables that results in accurate correlation between resilient modulus model parameters k_i and basic soil properties. Several combinations of regression equations were attempted and evaluated based on the criteria of the coefficient of multiple determination (R^2), the significance of the model and the significance of the individual regression coefficients.

In this study, a correlation matrix was used as a preliminary method for selecting material properties used in the regression analysis models. The magnitude of each element in the correlation matrix indicates how strongly two variables (whether independent or dependent) are correlated. The degree of correlation is expressed by a number that has a maximum value of one for highly correlated variables, and zero if no correlation exists. This was used to evaluate the importance of each independent variable (soil property) among other independent variables to the dependent variable (model parameters k_i).

3.6 Resilient Modulus Model Development

Regression analysis was conducted on the results of the fine-grained soils. Basic soil index properties were used to obtain correlations with the resilient modulus model parameters k_1, k_2, and k_3. The information collected from literature was used to identify the independent variables or index properties used to predict the material engineering properties. Each correlation was examined from both physical and statistical points of view. It was envisioned that more than one prediction model might be required or might be derived with the data available. If the model was not consistent with the observed behaviour of soils, it was rejected. Many attempts were made in which basic soil properties were included.

CHAPTER FOUR

RESULTS AND ANALYSIS

4.1 Physical Properties and Compaction Characteristics of the Soil Samples

This section presented the results of the physical properties of the soil samples. The physical properties are Atterberg limits, linear shrinkage, particle size distribution, specific gravity, soil classification, compaction characteristics and natural moisture content of the subgrade soil samples.

4.1.1 Atterberg limits test results of the soil samples

Figures A.1 through A.18showed the flow curve for the soil samples in Appendix A.Data sheets for the Atterberg Limits tests for the soil samples were as presented in Tables A.1 through A.18 in Appendix A. Table 4.1 show the summary ofAtterberg limits test resultsof the soil samples.

Table 4.1: Summary of Atterberg Limits Test Resultsof the Soil Samples

Master Test Section	Liquid Limit (LL) (%)	Plastic Limit (PL) (%)	Plasticity Index (PI) (%)
MTS 1-1	37	20	17
	39	24	15
	33	20	13
MTS 1-2	36	24	12
	29	21	8
	37	18	19
MTS 1-3	39	25	14
	36	26	10
	31	18	13
MTS 1-4	45	29	16
	42	25	17
	32	22	10
MTS 1-5	42	28	14
	41	31	10
	41	27	14
MTS 1-6	30	20	10
	35	21	14
	39	23	16

Source: Laboratory results

4.1.2 Linear shrinkage test results of the soil samples

Table 4.2 presents the summary of linear shrinkage test resultsof the soil samples.

Table 4.2: Summary of Linear Shrinkage Test Resultsof the Soil Samples

Master Test Section	Linear Shrinkage (%)
MTS 1-1	10.00
	8.57
	11.43
MTS 1-2	10.00
	8.57
	9.29
MTS 1-3	10.71
	7.14
	9.29
MTS 1-4	8.57
	10.71
	9.29
MTS 1-5	7.86
	8.57
	7.14
MTS 1-6	8.57
	8.57
	10.71

Source: Laboratory results

4.1.3 Particle size distribution test results of the soil samples

Figures A.19 through A.36 in Appendix A show the particle size distribution curve for the soil samples. Table 4.3 showed the summary of sieve analyses tests results of the soil samples.

4.1.4 Specific gravity test results of the soil samples

Specific gravity tests were conducted on the soil samples and the results were summarized in Table 4.4. Data sheets for the specific gravity tests for the soil samples were presented in Tables A.55 through A.72in Appendix A.

Table 4.3: Summary of Sieve AnalysesTests Results of the Soil Samples

Master Test	Passing #200	Passing #40	Passing #4	Clay	Silt (%)

Section	(%)	(%)	(%)	(%)	
MTS 1-1	72.42	79.86	86.38	49.76	22.72
	77.76	94.12	99.64	46.88	30.88
	48.66	55.08	63.78	29.60	19.04
MTS 1-2	22.32	26.14	33.30	8.48	13.76
	73.38	96.42	99.94	37.28	36.16
	75.84	91.84	99.16	42.88	32.96
MTS 1-3	66.34	73.96	91.86	39.84	26.56
	43.76	50.88	73.14	27.36	16.48
	55.94	68.82	91.38	33.44	22.40
MTS 1-4	88.22	92.70	97.64	69.28	18.88
	92.70	95.96	99.52	48.80	48.16
	85.76	93.32	99.22	61.60	24.16
MTS 1-5	57.82	61.06	74.62	18.08	39.68
	69.08	74.10	87.20	42.72	26.40
	71.36	75.08	84.68	50.08	21.28
MTS 1-6	57.60	64.54	81.58	17.28	40.32
	50.48	59.34	73.32	18.08	32.32
	49.76	54.70	69.32	30.88	18.88

Source: Laboratory results

Table 4.4: Summary of Specific Gravity Tests Results of the Soil Samples

MTS	Specific Gravity	Soil Type
MTS 1-1	2.51	A-6
	2.44	A-6
	2.49	A-6
MTS 1-2	2.57	A-2-4
	2.46	A-4
	2.51	A-4
MTS 1-3	2.46	A-4
	3.31	A-4
	2.51	A-6
MTS 1-4	2.54	A-7-6
	2.55	A-7-6
	2.55	A-4
MTS 1-5	2.58	A-7-6
	2.50	A-5
	2.53	A-7-6
MTS 1-6	2.58	A-4
	2.46	A-6
	2.47	A-4

Source: Laboratory results

4.1.5 Soil classification results of the soil samples

The soil samples were classified using USCS and AASHTO Soil Classification system. Table 4.5 summarized theUSCS and the AASHTO Classification of the soil samples.

Table 4.5: The USCS and the AASHTO Classification Results of the Soil Samples

Master Test Section	USCS	AASHTO
MTS 1-1	CL (Lean clay with gravel)	A-6 (clayed soils)
	CL (Lean clay with gravel)	A-6 (clayed soils)
	GC (Clayed Gravel)	A-6 (clayed soils)
MTS 1-2	GC (Clayed Gravel)	A-2-4 (Silty or clayed gravel sand)
	CL (Lean clay with gravel)	A-4 (Silty soils)
	CL (Lean clay with gravel)	A-4 (Silty soils)
MTS 1-3	CL (Gravelly lean clay)	A-4 (Silty soils)
	SC (Clayed Sand)	A-4 (Silty soils)
	CL (Gravelly lean clay)	A-6 (clayed soils)
MTS 1-4	CL (Lean clay)	A-7-6 (clayed soils)
	CL (Lean clay)	A-7-6 (clayed soils)
	CL (Lean clay)	A-4 (Silty soils)
MTS 1-5	CL (Gravelly lean clay)	A-7-6 (clayed soils)
	CL (Gravelly lean clay)	A-5 (Silty soils)
	CL (Lean clay with gravel)	A-7-6 (clayed soils)
MTS 1-6	CL (Gravelly lean clay)	A-4 (Silty soils)
	CL (Gravelly lean clay)	A-6 (clayed soils)
	GC (Clayed Gravel)	A-4 (Silty soils)

Source: Laboratory results

4.1.6 Compaction characteristics of the soil samples

Table 4.6 showed the summary of compaction characteristics test results of the soil samples with its soil type. Figures A.37 through A.54 in Appendix A show the compaction curves for the soil samples.

4.1.7 Natural moisture content of the soil samples

Table 4.7 present theNWC test results of the soil samples with its Soil Type. Tables A.37 through A.54 in Appendix A show the Data sheets for the NWC computation of the soil samples.

Table 4.6: Summary of Compaction Characteristics Test Results of the Soil Samples

Master Test Section	Optimum Moisture Content (%)	Maximum Dry Density (kN/m^3)	Soil Type
MTS 1-1	17.6	16.27	A-6
	18.7	15.98	A-6
	13.3	18.48	A-6
MTS 1-2	16.2	16.64	A-2-4
	20.0	16.18	A-4
	21.0	15.97	A-4
MTS 1-3	15.6	17.68	A-4
	16.2	17.70	A-4
	17.8	16.12	A-6
MTS 1-4	22.3	15.18	A-7-6
	21.6	15.16	A-7-6
	18.5	15.75	A-4
MTS 1-5	18.8	16.72	A-7-6
	18.9	15.82	A-5
	21.1	16.18	A-7-6
MTS 1-6	13.9	17.68	A-4
	21.0	15.85	A-6
	16.6	17.30	A-4

Source: Laboratory results

Table 4.7: Summary of Natural Moisture Content Test Results of the Soil Samples

MTS	Natural Moisture Content (%)	Soil Type
MTS 1-1	2.98	A-6
	5.55	A-6
	2.35	A-6
MTS 1-2	0.94	A-2-4
	0.77	A-4
	0.74	A-4
MTS 1-3	12.38	A-4
	14.31	A-4
	7.51	A-6
MTS 1-4	2.97	A-7-6
	2.42	A-7-6
	2.27	A-4
MTS 1-5	7.76	A-7-6
	7.74	A-5
	10.36	A-7-6
MTS 1-6	1.25	A-4
	1.61	A-6
	1.34	A-4

Source: Laboratory results

4.2 California Bearing Ratio of the Soil Samples

The summary of the California Bearing Ratio values obtained on the soil samples were summarized in Table 4.8.

Table 4.8: Summary of California Bearing RatioResults of the Soil Samples

Master Test Section	California Bearing Ratio (%)	
	UNSOAKED	SOAKED
MTS 1-1	2.30	2.86
	1.02	1.36
	4.52	1.58
MTS 1-2	2.11	1.80
	1.51	2.28
	1.51	1.42
MTS 1-3	0.64	1.80
	4.32	2.77
	1.36	2.44
MTS 1-4	1.44	1.06
	2.27	1.96
	0.79	1.35
MTS 1-5	3.77	5.88
	0.98	7.83
	12.82	0.90
MTS 1-6	1.81	1.89
	1.71	2.56
	0.81	2.16

Source: Laboratory results

The results of stress (load) versus penetration depth were plotted to determine the CBR for each specimen. Figures B.1 through B.36in Appendix B showed the relationship between the stress (load) versus penetration depth(both soaked and unsoaked) for the soil samples.

4.3 Unconfined Compressive Strength (UCS) Test of the Soil Samples

Table 4.9 present thesummary of UCS test results of the soil samples with its soil type.Data sheets for the computation of the unconfined compressive strength tests of the soil samples were presented in Tables A.19 through A.36in Appendix A.

Table 4.9: Unconfined Compressive Strength Test Results of the Soil Samples

Master Test Section	UCS (kN/m^2)	Shear Strength(kN/m^2)	Soil Type
MTS 1-1	350.2	175.1	A-6
	70.7	35.35	A-6
	549.2	274.6	A-6
MTS 1-2	211.6	105.8	A-2-4
	114.3	57.15	A-4
	234.2	117.1	A-4
MTS 1-3	39.9	19.95	A-4
	298.9	149.45	A-4
	392.8	196.4	A-6
MTS 1-4	17.8	8.9	A-7-6
	366.2	183.1	A-7-6
	183.1	91.55	A-4
MTS 1-5	398.0	199.0	A-7-6
	424.5	212.3	A-5
	185.2	92.6	A-7-6
MTS 1-6	44.3	22.2	A-4
	238.6	119.3	A-6
	534.6	267.3	A-4

Source: Laboratory results

4.4 Statistical Analysis of Resilient Modulus of Nigerian Soil

The statistical summary of the resilient modulus of Nigerian soils (Claros, et al., 1986)are shown in Tables4.10 – 4.13.

Table 4.10 presents the mean resilient modulus values, standard deviation, and coefficient of variation for the fine-grained soil obtained from MTS 1 and 2. Table 4.11 presents the mean resilient modulus values, standard deviation, and coefficient of variation of the resilient modulus of the fine-grained soil obtained from MTS 3 and 4. Table 4.12 presents the mean resilient modulus values, standard deviation, and coefficient of variation of the resilient modulus of the fine-grained soil obtained from MTS 5 and 6.Table 4.13 presents the mean resilient modulus values, standard deviation, and coefficient of variation of the resilient modulus of the coarse-grained soil obtained from MTS 1 – 6.

Table 4.10: Statistical Summary of M_r of Fine-Grained Soilfrom MTS1 and 2

Test Sequence	Confining Stress (kPa)	Deviator stress (kPa)	Resilient Modulus(kN/m^2) for MTS 1			Resilient Modulus(kN/m^2) for MTS 2		
			Mean	SD	COV	Mean	SD	COV
1	0	7.38	103663	48751.7	0.47	62634	18566.9	0.30
2	0	14.09	106338	38562.2	0.36	72285	20568.5	0.28
3	0	29.08	111466	35739.9	0.32	85211	24163.1	0.28
4	0	57.25	118357	41619.4	0.35	99771	29349.6	0.29
5	0	71.64	121103	44967.1	0.37	105205	31552.4	0.30
6	20.68	7.14	103574	49434.6	0.48	62180	18487.1	0.30
7	20.68	14.09	106338	38562.2	0.36	72285	20568.5	0.28
8	20.68	28.48	111287	35681	0.32	84804	24034.4	0.28
9	20.68	56.78	118259	41506.2	0.35	99575	29272.7	0.29
10	20.68	71.40	121060	44912.6	0.37	105121	31517.2	0.30
11	41.37	6.42	103323	51700.9	0.50	60761	18245.3	0.30
12	41.37	13.49	106102	39041.9	0.37	71584	20403.4	0.29
13	41.37	29.68	111644	35805.5	0.32	85612	24290.7	0.28
14	41.37	57.25	118357	41619.4	0.35	99771	29349.6	0.29
15	41.37	68.77	120583	44313.3	0.37	104188	31129.9	0.30

Table 4.11: Statistical Summary of $M_r$of Fine-Grained SoilfromMTS3 and 4

Test Sequence	Confining Stress (kPa)	Deviator stress (kPa)	Resilient Modulus(kN/m^2) for MTS 3			Resilient Modulus(kN/m^2) for MTS 4		
			Mean	SD	COV	Mean	SD	COV
1	0	7.38	194320	84496	0.43	118241	46355.2	0.39
2	0	14.09	175790	61931.7	0.35	115092	32483.3	0.28
3	0	29.08	159158	43590.6	0.27	113899	20437.3	0.18
4	0	57.25	146925	33307.0	0.23	115267	23831.8	0.21
5	0	71.64	143501	31601.1	0.22	116305	28838.6	0.25
6	20.68	7.14	195378	85826.8	0.44	118452	47083.7	0.40
7	20.68	14.09	175790	61931.7	0.35	115092	32483.3	0.28
8	20.68	28.48	159583	44017.8	0.28	113897	20636.4	0.18
9	20.68	56.78	147058	33387.9	0.23	115234	23674.6	0.21
10	20.68	71.40	143550	31620.0	0.22	116288	28753.2	0.25
11	41.37	6.42	198820	90184.7	0.45	119159	49407.7	0.41
12	41.37	13.49	176919	63259.5	0.36	115244	33381.8	0.29
13	41.37	29.68	158746	43178.2	0.27	113904	20256.6	0.18
14	41.37	57.25	146925	33307.0	0.23	115267	23831.8	0.21
15	41.37	68.77	144105	31844.9	0.22	116093	27821.3	0.24

Table 4.12:Statistical Summary of $M_r$of Fine-Grained Soilfrom MTS5 and 6

Test	Confini	Devia	Resilient Modulus(kN/m^2)	Resilient Modulus(kN/m^2)

Sequence No.	ng Stress	tor stress	for MTS 5			for MTS 6		
			Mean	SD	CV	Mean	SD	CV
1	0	7.38	66108	32240.4	0.49	101840	38260.3	0.38
2	0	14.09	76419	36278.3	0.47	105981	31818.8	0.30
3	0	29.08	90102	41710.7	0.46	113746	32515.1	0.29
4	0	57.25	105366	47989.8	0.46	124059	38769.6	0.31
5	0	71.64	111029	50399.9	0.45	128158	41733.4	0.33
6	20.68	7.14	65621	32049.8	0.49	101700	38783.9	0.38
7	20.68	14.09	76419	36278.3	0.47	105981	31818.8	0.30
8	20.68	28.48	89673	41537.9	0.46	113476	32396.8	0.29
9	20.68	56.78	105161	47903.7	0.46	123913	38667.2	0.31
10	20.68	71.40	110942	50362.2	0.45	128093	41685.8	0.33
11	41.37	6.42	64096	31452.8	0.49	101302	40575.5	0.40
12	41.37	13.49	75674	35985.5	0.48	105619	32029.5	0.30
13	41.37	29.68	90525	41881.0	0.46	114013	32635.8	0.29
14	41.37	57.25	105366	47989.8	0.46	124059	38769.6	0.31
15	41.37	68.77	10997	49945.4	0.45	127382	41161.6	0.32

Table 4.13:Statistical Summary of $M_r$of Coarse-Grained Soilfrom MTS1 – 6

Test Sequence No.	Confining Stress, σ_3	Deviator stress, σ_d	Resilient Modulus(kN/m^2) for MTS 1-6		
			Mean	SD	COV
1	34.47	6.89	75938.47	55636.36	0.73
2	34.47	14.69	91621.76	73649.80	0.80
3	34.47	36.27	115941.45	104364.06	0.90
4	34.47	72.84	140410.95	137032.67	0.98
5	34.47	119.60	161957.27	166405.68	1.03
6	68.95	6.42	74633.43	54231.64	0.73
7	68.95	14.69	91621.76	73649.80	0.80
8	68.95	35.67	115418.87	103680.67	0.90
9	68.95	68.04	137746.59	133427.45	0.97
10	68.95	116.61	160755.32	164760.47	1.02
11	103.42	6.54	74961.11	54582.74	0.73
12	103.42	14.39	91138.75	73068.70	0.80
13	103.42	35.67	115424.84	103688.48	0.90
14	103.42	70.44	139093.29	135248.65	0.97
15	103.42	125.60	164312.62	169630.89	1.03
16	103.42	149.58	173104.51	181680.23	1.05
17	137.90	5.58	72148.86	51606.01	0.72
18	137.90	14.69	91621.76	73649.80	0.80
19	137.90	34.47	114373.98	102316.65	0.89
20	137.90	46.46	123963.66	114944.48	0.93
21	137.90	116.61	160755.32	164760.47	1.02
22	137.90	173.56	181091.31	192633.40	1.06

4.5 Determination of Resilient Modulus Model Parameters for Nigerian Soils

The statistical summary of the resilient modulus of Nigerian soils presented in Tables 4.10 through 4.13 were used in evaluating the resilient modulus parameters of the Nigerian soils. The seven resilient modulus equations proposed by several researchers presented in literature were evaluated for the Nigerian soils. For the evaluation, the seven transformed resilient modulus equations (Equations 3.12 through 3.18) presented in Table 3.4 was used alongside the statistical summary of the resilient modulus of Nigerian soils.

4.5.1 Determination of resilient modulus model parameters for fine-grained soil

The statistical summary of the resilient modulus of Nigerian fine-grained soils presented in Tables 4.10 through 4.12 were used todetermine the resilient modulus parameters of the fine-grained soils. The six out of seven resilient modulus equations presented in literature was used for the evaluation of the fine-grained soils. The resilient modulus equation proposed by Pezo, (1993) was not used because of zero confining stress for fine-grained soils. The transformed resilient modulus equations presented in Table 3.4 was used for the evaluation.Tables4.14through 4.19presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the resilient modulus equations evaluated. Tables C.1 through C.6in the appendix presents the resilient modulus model parameters $k_i$obtained forfine-grained soilusing the equations evaluated.

Table 4.14 presents the statistical summary of the resilient modulus parameters of fine-grained soilobtained from the evaluation of Uzan model.

Table 4.14: Statistical Summary of $k_i$for the Fine-Grained Soil using Uzan's Model

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error

k_1	1320.92	1230.78	21.9186	2107.274	422.720	1.0000
k_2	-5E-17	-4E-17	-3.1E-16	1.3E-16	9E-17	3.6E-15
k_3	0.2045	0.1931	0.0042	0.4042	0.1040	3.6E-15

The $k_i$values obtained from Uzan model were presented in histograms in Figure 4.1a-c.

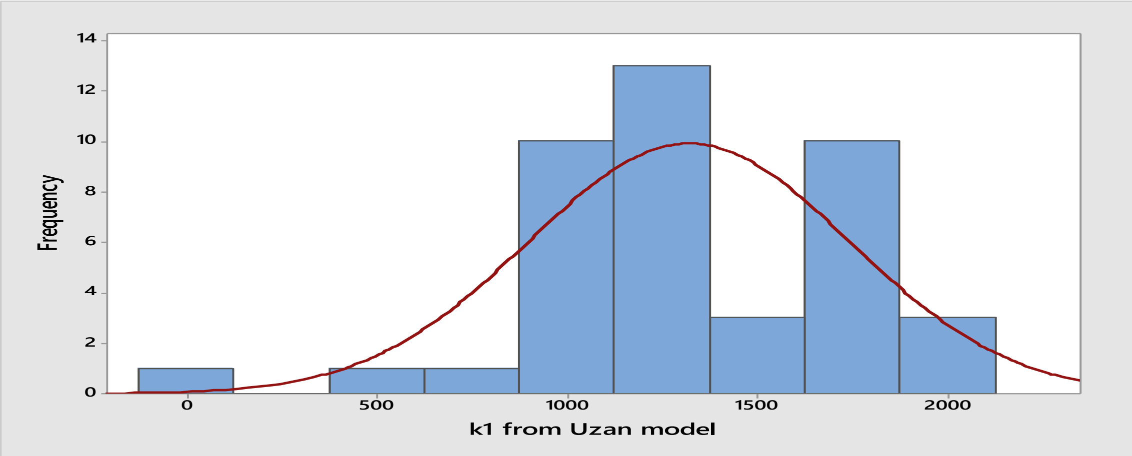


Figure 4.1a: $k_1$ofthe Fine-Grained Soil Samples using the Uzan's Model

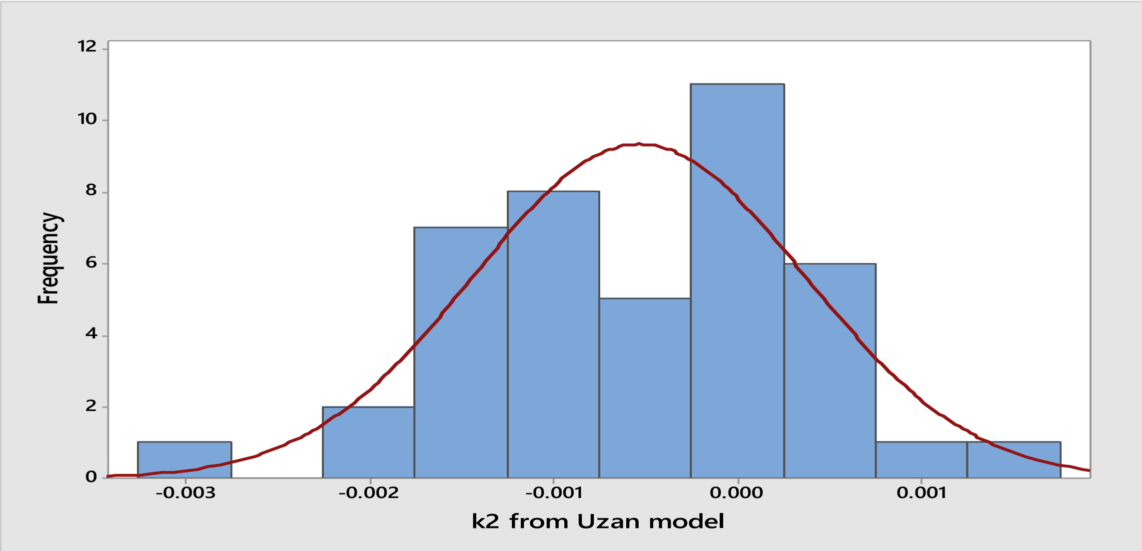


Figure 4.1b: $k_2$ofthe Fine-Grained Soil Samples using the Uzan's Model

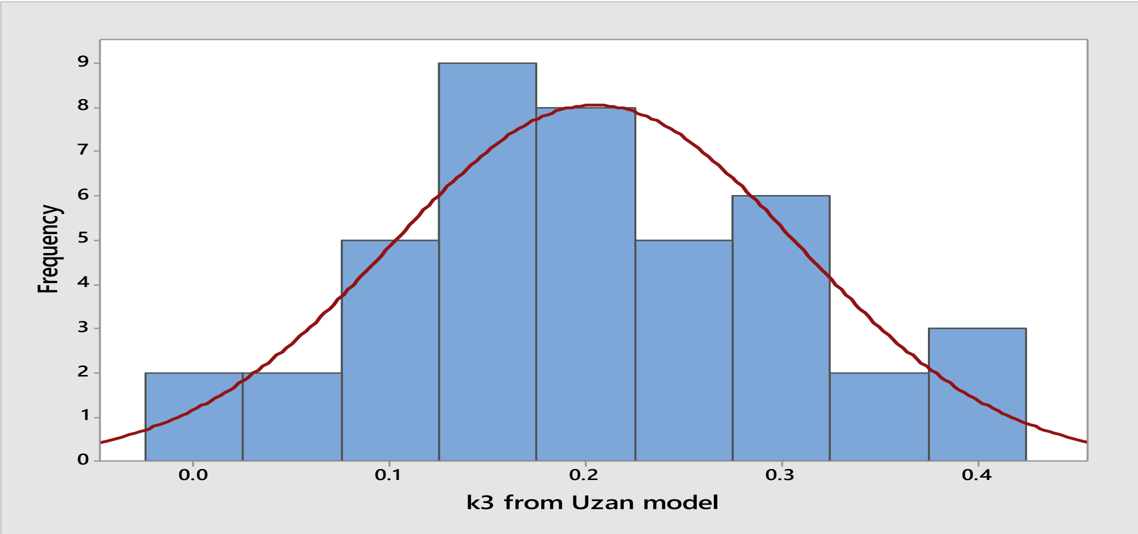


Figure 4.1c: k_3 of the Fine-Grained Soil Samples using the Uzan's Model

Table 4.15 presents the statistical summary of the resilient modulus parameters of fine-grained soil obtained from the evaluation of Witczak and Uzan equation.

Table 4.15: Statistical Summary of $k_i$for the Fine-Grained Soil using Witczak and Uzan'sModel

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	1548.03	1463.34	27.9011	2576.806	527.610	1.0000
k_2	-9E-18	0.0000	-2.7E-16	1.88E-16	8.5E-17	3.6E-15
k_3	0.2045	0.1931	0.0042	0.4042	0.1040	3.8E-15

The histograms presenting the resilient modulus model parameters k_i obtained from the evaluation using the Witczak and Uzan model are presented in Figures 4.2a-c.

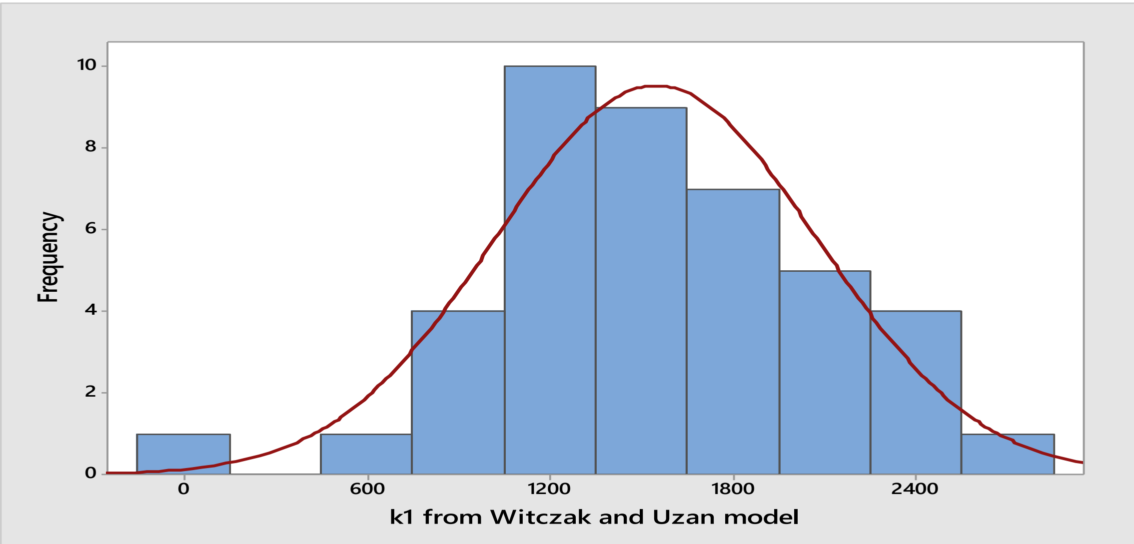


Figure 4.2a: $k_1$ofthe Fine-Grained Soil Samples using the Witczak and Uzan's Model

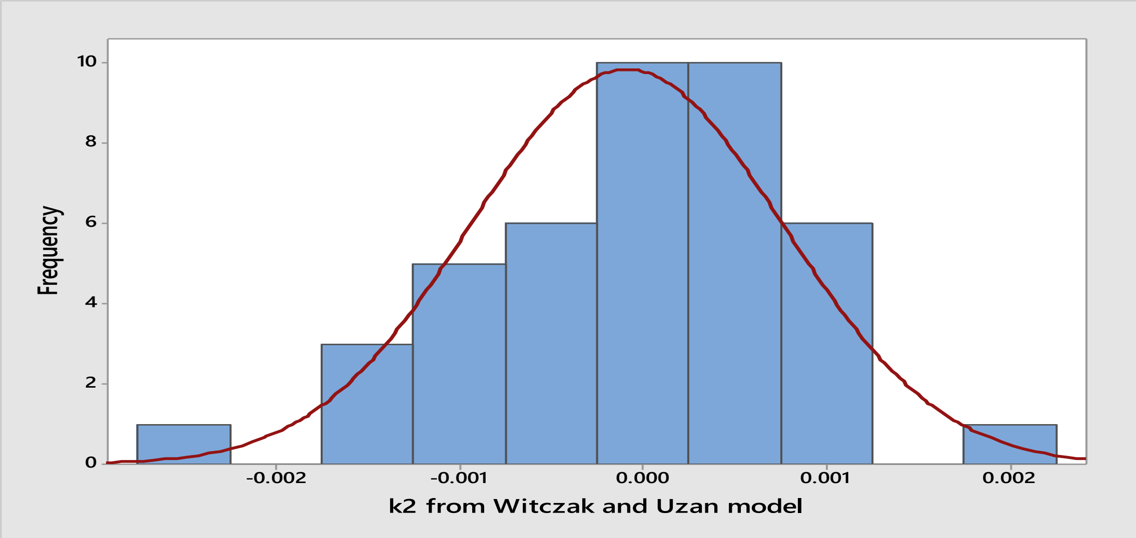


Figure 4.2b: $k_2$ofthe Fine-Grained Soil Samples using the Witczak and Uzan's Model

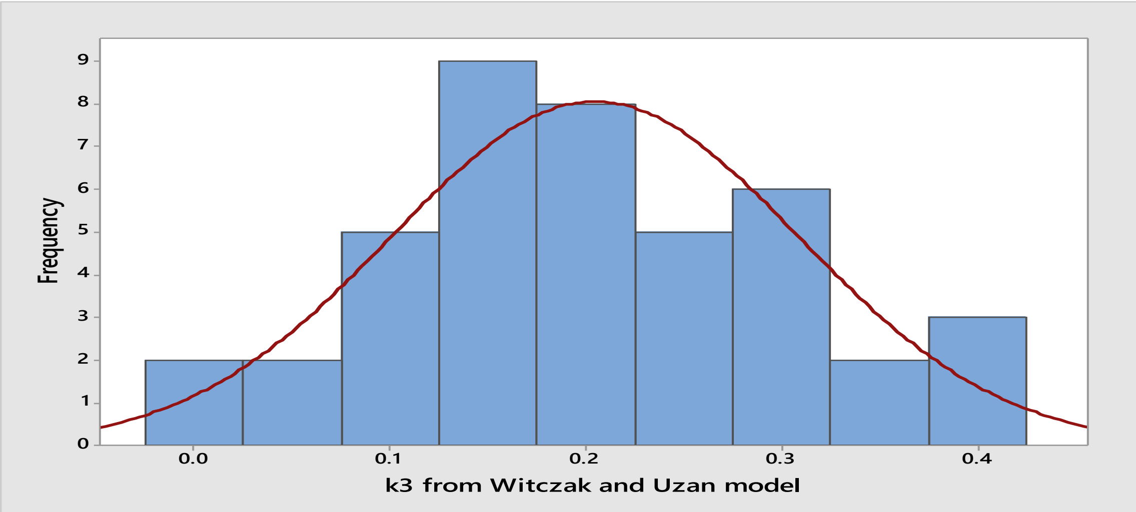


Figure 4.2c: $k_3$ofthe Fine-Grained Soil Samples using the Witczak and Uzan's Model

Tables4.16 presents the statistical summary of the resilient modulus parameters of fine-grained soilobtained from the evaluation of Ni et al model.

Table 4.16: Statistical Summary of $k_i$for the Fine-Grained Soil using Ni et al's Model

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	781.001	748.621	9.1845	1430.387	296.915	3.0067
k_2	-0.0092	-0.0087	-0.0182	-0.0002	0.0047	3.4579
k_3	0.9693	0.9150	0.0200	1.9158	0.4929	2.7137

Figure 4.3a-cpresent the histogram of the resilient modulus model parameters k_i obtained from the evaluation ofNi et al model.

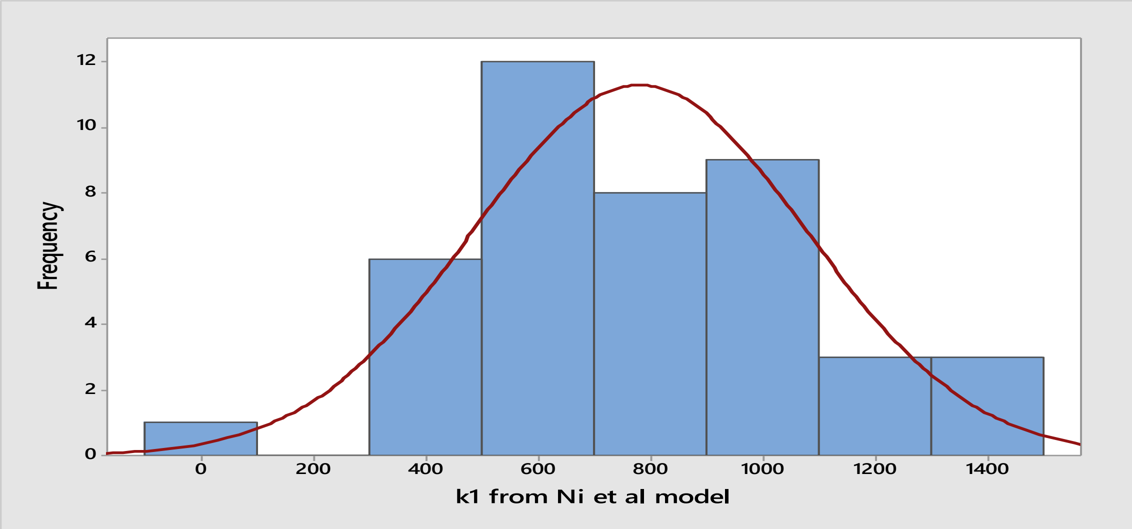


Figure 4.3a: $k_1$ofthe Fine-Grained Soil Samples using the Ni *et al*'s Model

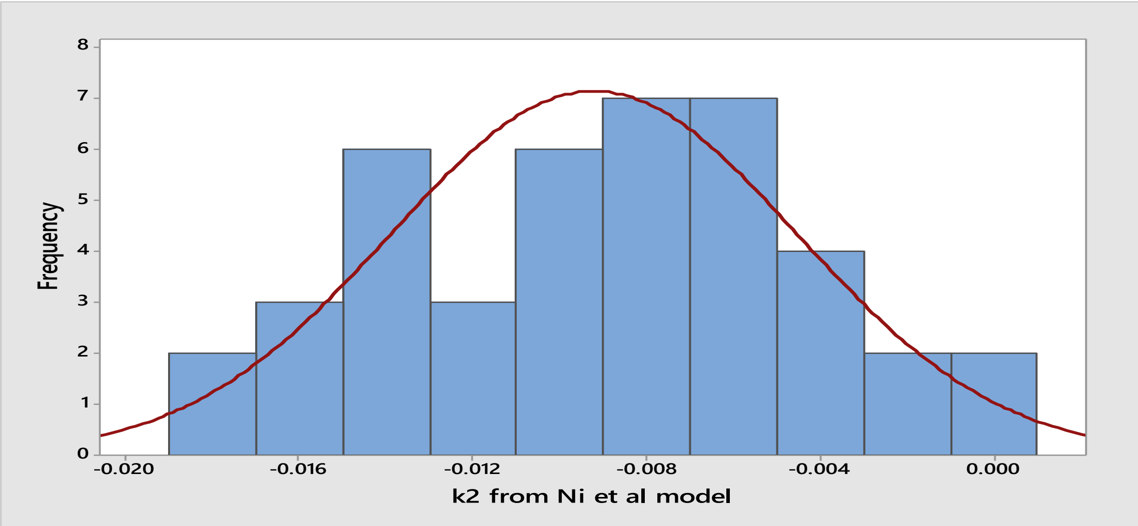


Figure 4.3b: $k_2$ofthe Fine-Grained Soil Samples using the Ni *et al*'s Model

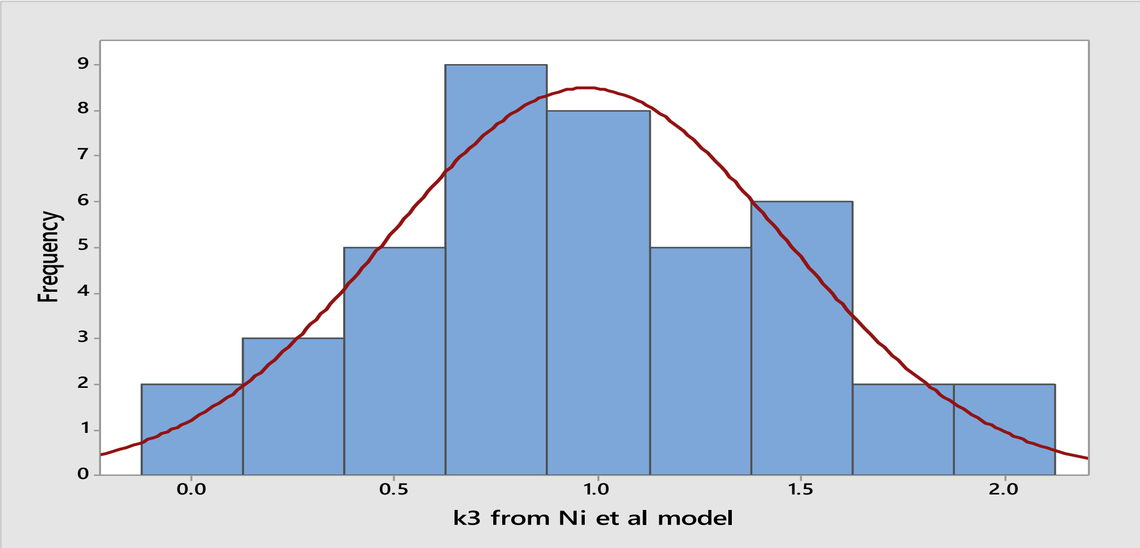


Figure 4.3c: $k_3$ofthe Fine-Grained Soil Samples using the Ni *et al*'s Model

Tables4.17 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the evaluation of Ooi et al A resilient modulus equations.

Table 4.17: Statistical Summary of $k_i$for the Fine-Grained Soil using Ooi et al's Model A

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	781.924	750.168	9.2038	1430.843	296.985	3.2722
k_2	-0.0070	-0.0066	-0.0138	-0.0001	0.0036	1.7799
k_3	0.9744	0.9198	0.0202	1.9259	0.4955	2.9995

Figure 4.4a-c present the histogram of the resilient modulus model parameters k_i obtained using the Ooi et alA resilient modulus model.

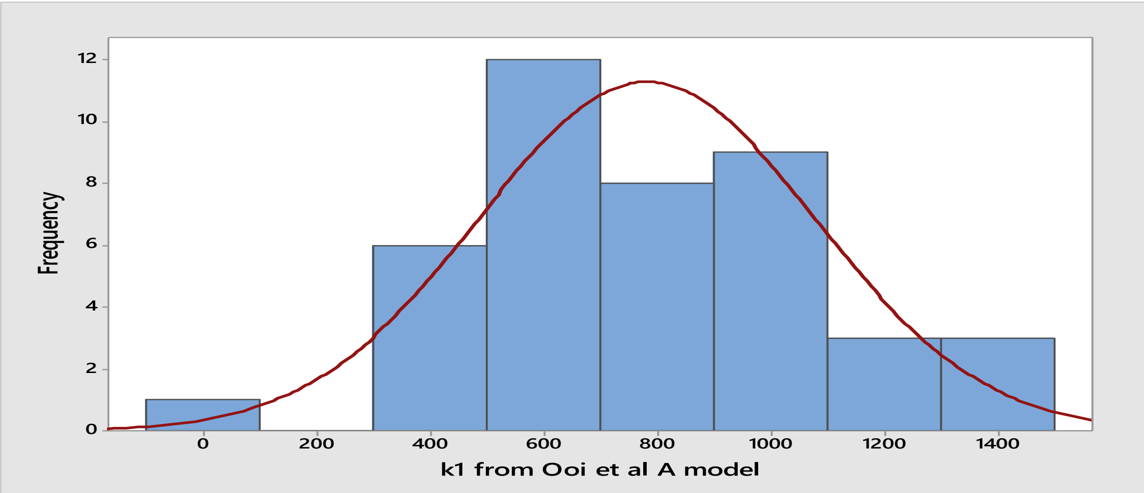


Figure 4.4a: $k_1$ofthe Fine-Grained Soil Samples using the Ooi *et al*'s Model A

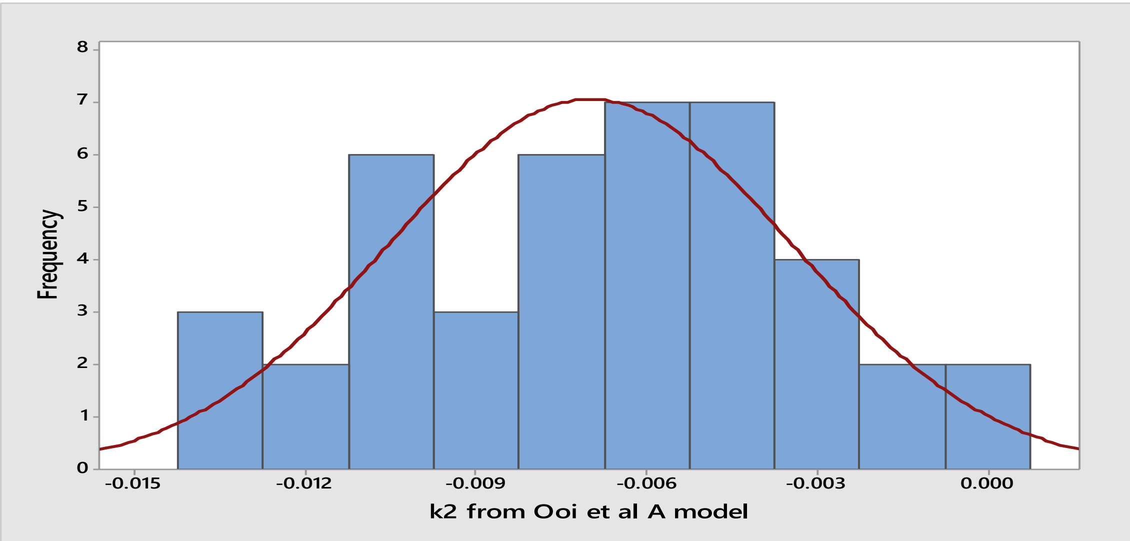


Figure 4.4b: $k_2$ofthe Fine-Grained Soil Samples using the Ooi *et al*'s Model A

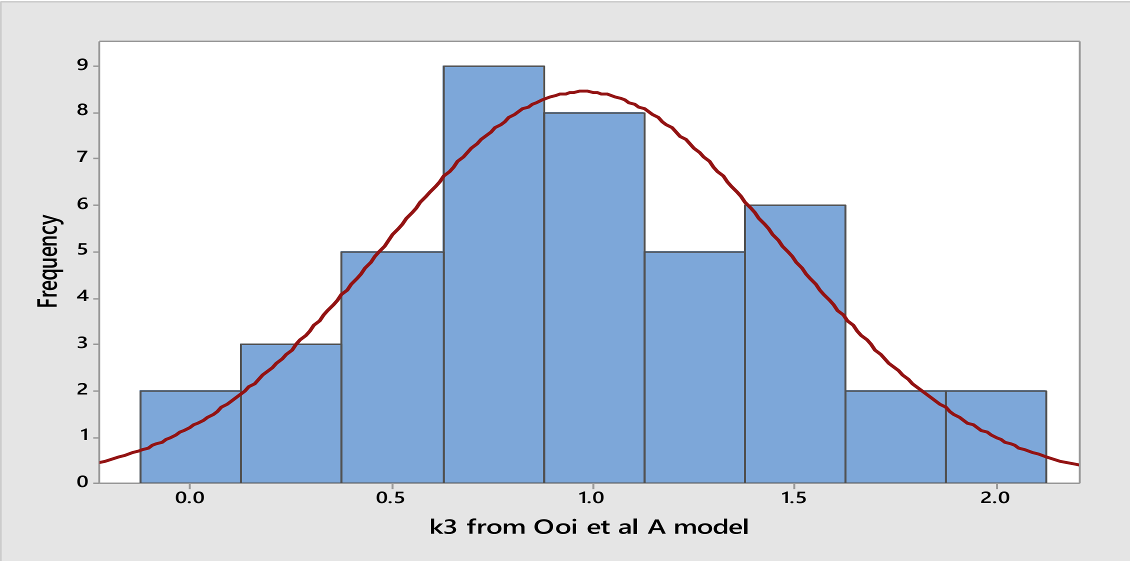


Figure 4.4c: $k_3$ofthe Fine-Grained Soil Samples using the Ooi *et al*'s Model A

Tables4.18 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the evaluation of Ooi et al B resilient modulus equations.

Table 4.18: Statistical Summary of $k_i$for the Fine-Grained Soil using Ooi et al's Model B

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	791.406	763.520	9.4025	1435.499	297.729	3.6492
k_2	-0.0053	-0.0050	-0.0105	-0.0001	0.0027	1.9633
k_3	1.7720	1.6728	0.0367	3.5025	0.9012	6.0592

Figure 4.5a-c present the histogram of the resilient modulus model parameters k_i obtained using the Ooi et al B resilient modulus model.

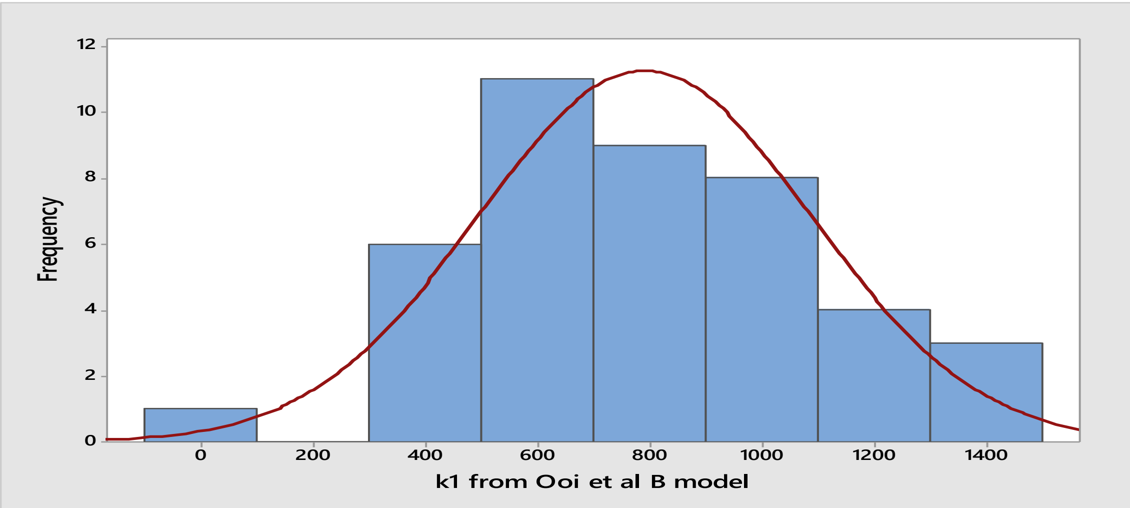


Figure 4.5a: $k_1$ofthe Fine-Grained Soil Samples using the Ooi *et al*'s Model B

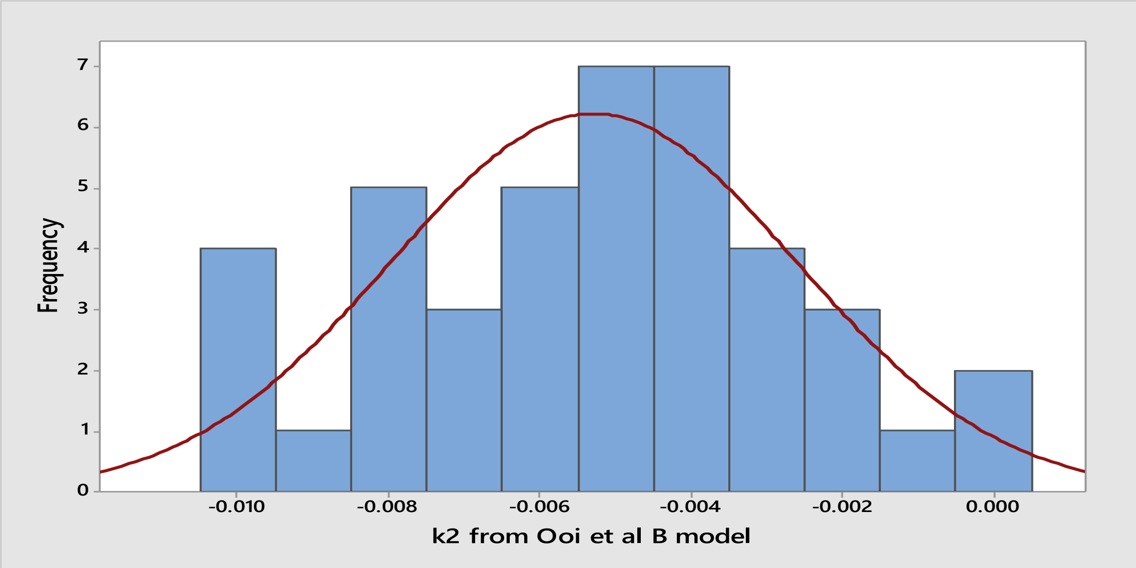


Figure 4.5b: $k_2$ofthe Fine-Grained Soil Samples using the Ooi *et al*'s Model B

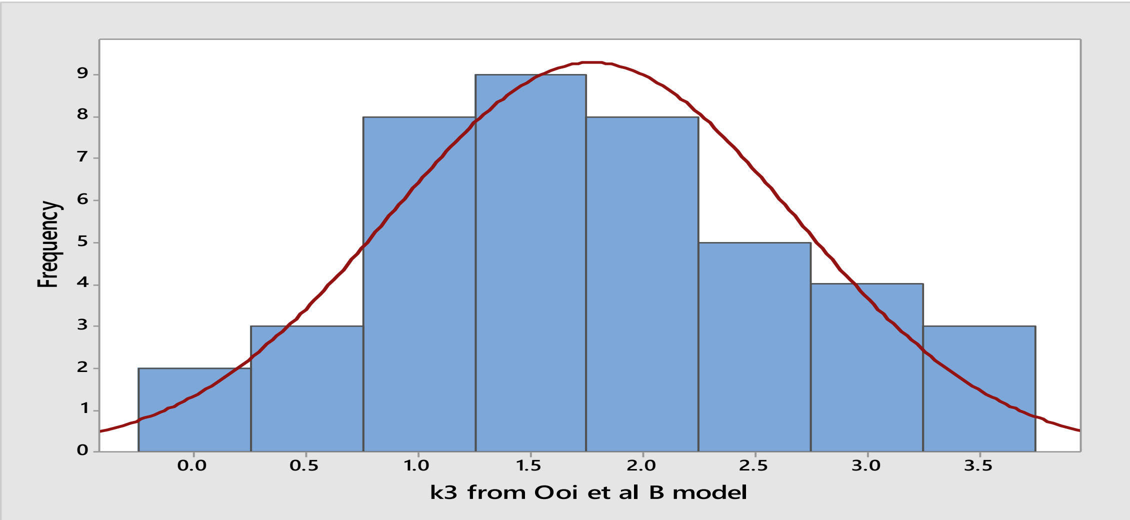


Figure 4.5c: $k_3$ofthe Fine-Grained Soil Samples using the Ooi *et al*'s Model B

Tables4.19 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the evaluation of NCHRPresilient modulus equation.

Table 4.19: Statistical Summary of $k_i$for the Fine-Grained Soil using NCHRP's Model

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	857.780	803.924	9.3952	1435.330	291.160	2.7412
k_2	0.0018	0.0018	4E-05	0.0038	0.0010	0.5701
k_3	1.6647	1.5941	0.0363	3.4729	0.8818	5.3643

Figure 4.6a-c present the histogram of the resilient modulus model parameters k_i obtained using the Ooi et al A resilient modulus model.

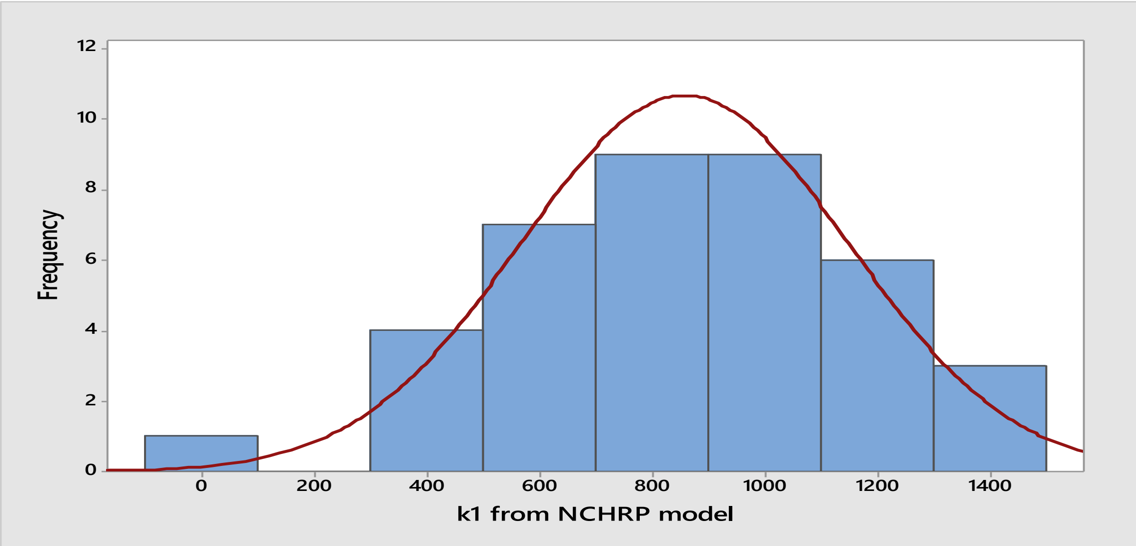


Figure 4.6a: $k_1$ofthe Fine-Grained Soil Samples using the NCHRP's Model

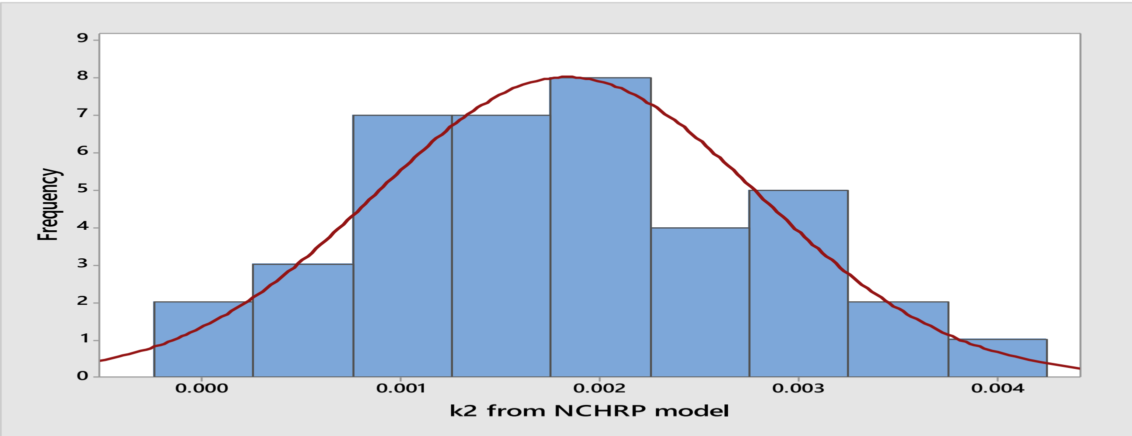


Figure 4.6b: $k_2$ofthe Fine-Grained Soil Samples using the NCHRP's Model

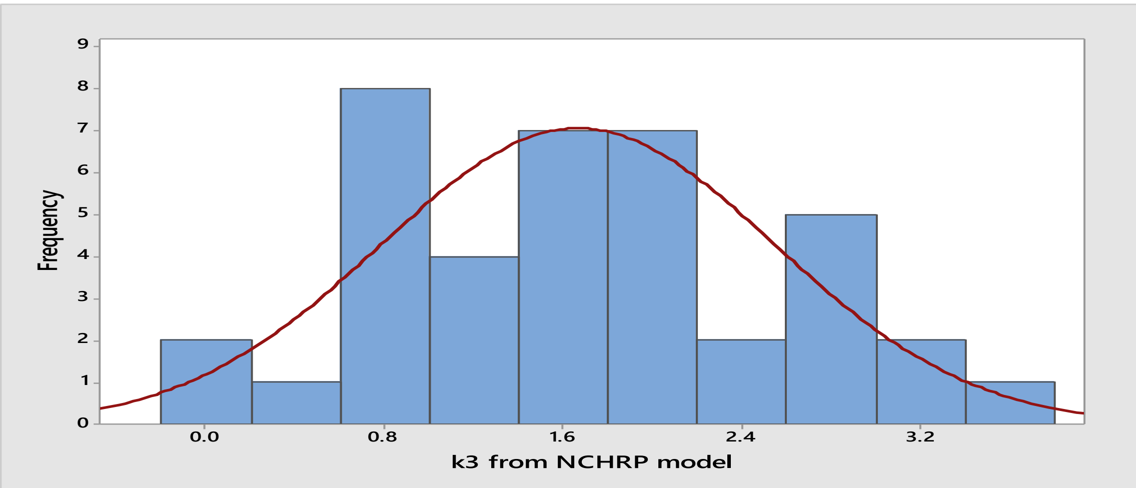


Figure 4.6c: $k_3$ofthe Fine-Grained Soil Samples using the NCHRP's Model

Histograms showing the distribution of resilient modulus model parameters (k_1, k_2, and k_3) values obtained for fine-grained soils were presented in Figures 4.7 through 4.9.

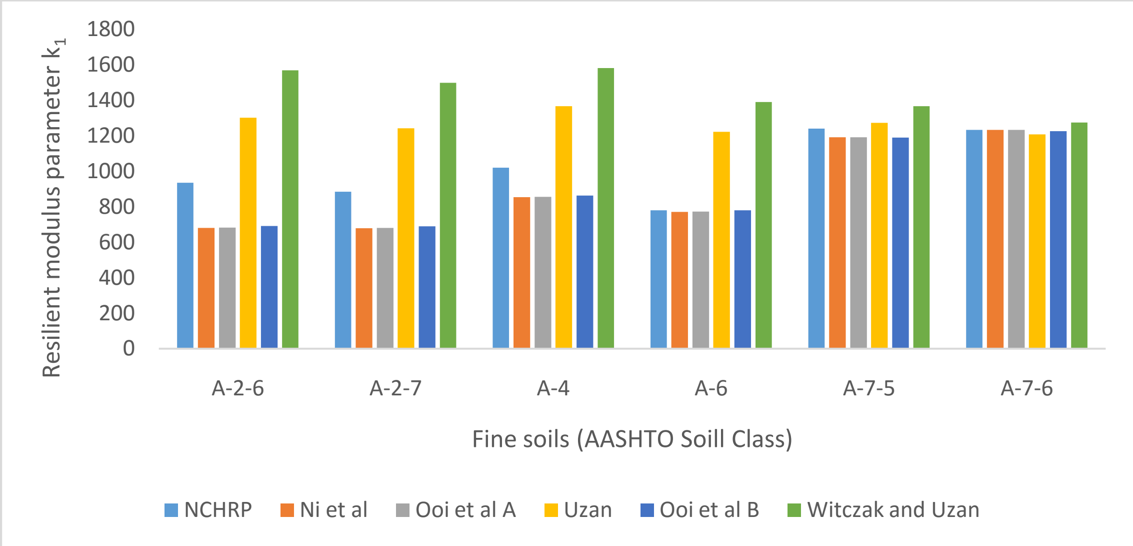


Figure 4.7: Resilient Modulus Parameters$k_1$ofFine-GrainedSoilsincluded in the Models

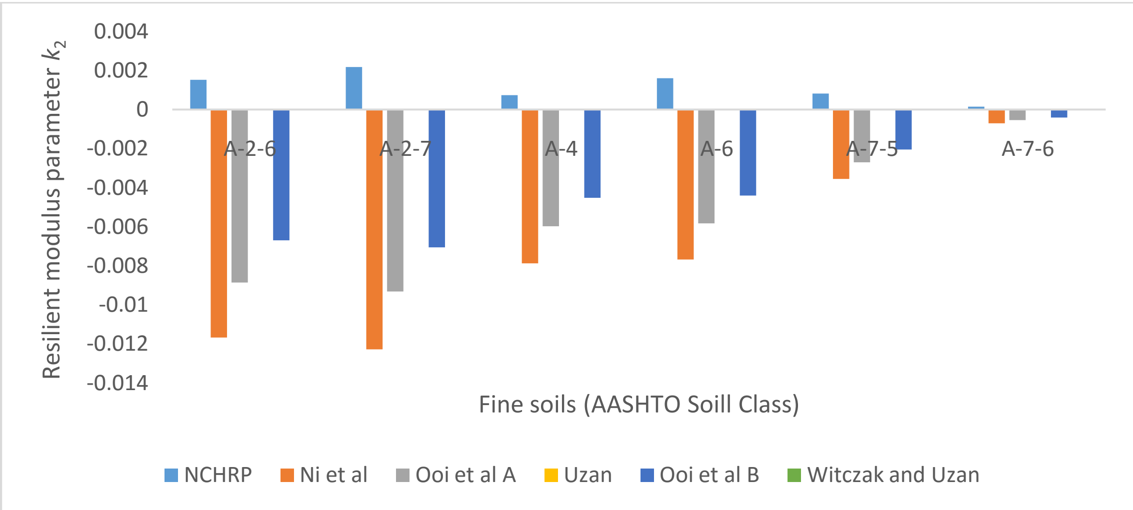


Figure 4.8: Resilient Modulus Parameter $k_2$ofFine-Grained Soilsincluded in the Models

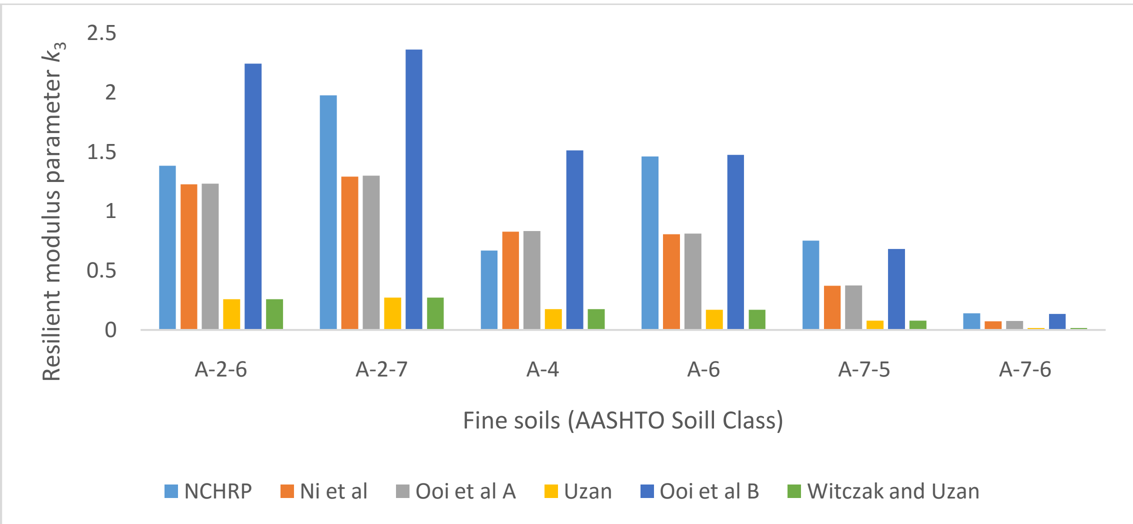


Figure 4.9: Resilient Modulus Parameters$k_3$ofFine-Grained Soilsincluded in the Models

4.5.2 Determination of resilient modulus model parameters for coarse-grained soil

The statistical summary of the resilient modulus of coarse-grained soils presented in Table 4.13 was used in evaluating the resilient modulus parameters of the coarse-grained soils. The seven resilient modulus equations presented in literature was used for the evaluation of coarse-grained soils. The transformed resilient modulus equations was used for the evaluation. Tables4.20 through 4.26 presents the statistical summary of the resilient modulus parameters of coarse-grained soils. Tables C.7 through C.13 in the appendix presents the resilient modulus model parameters $k_i$for the coarse-grained.

Table 4.20 presents the statistical summary of the resilient modulus parameters of the coarse-grained soils obtained from the evaluation of Uzan resilient modulus equation.

Table 4.20: Statistical Summary of $k_i$of the Coarse-Grained SoilSamplesusing theUzan's Model

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	1522.5	1166.44	657.896	7551.296	1538.68	1.0000
k_2	2.8E-17	4.4E-17	-3.0E-16	3.5E-16	1.6E-16	3E-15
k_3	0.2807	0.2543	0.0295	1.1014	0.2255	1.E-15

Figure 4.10a-c present the histogram of the resilient modulus model parameters k_i obtained using the Uzan resilient modulus model.

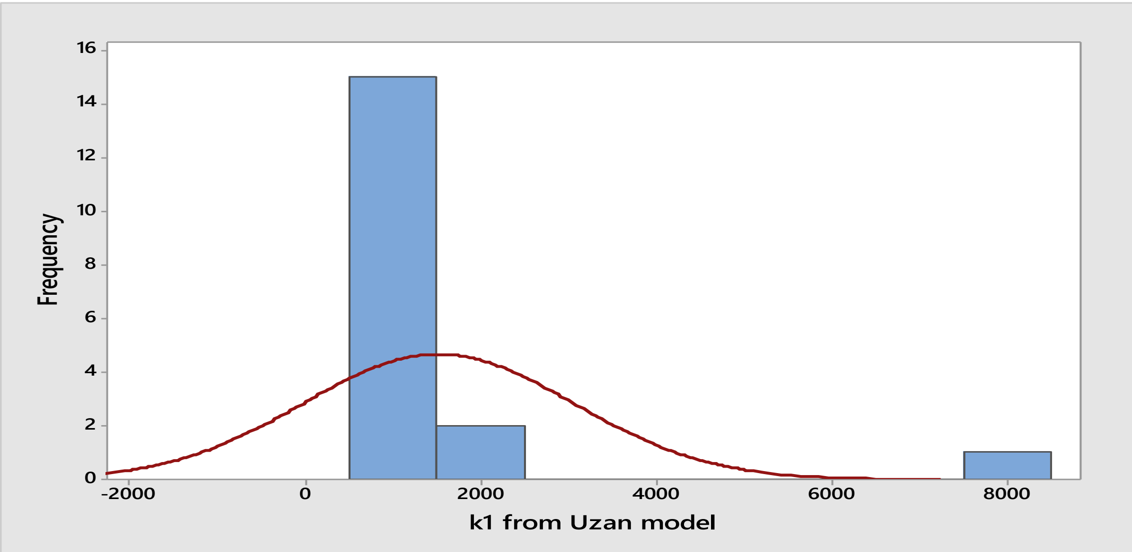


Figure 4.10a: $k_1$ofthe Coarse-Grained Soil Samples using the Uzan's Model

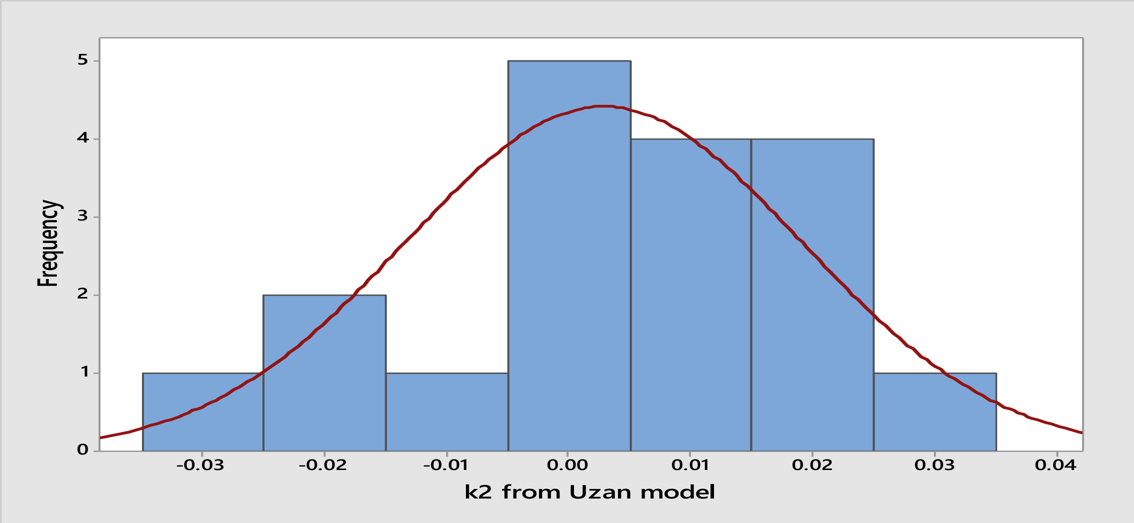


Figure 4.10b: $k_2$ofthe Coarse-Grained Soil Samples using the Uzan's Model

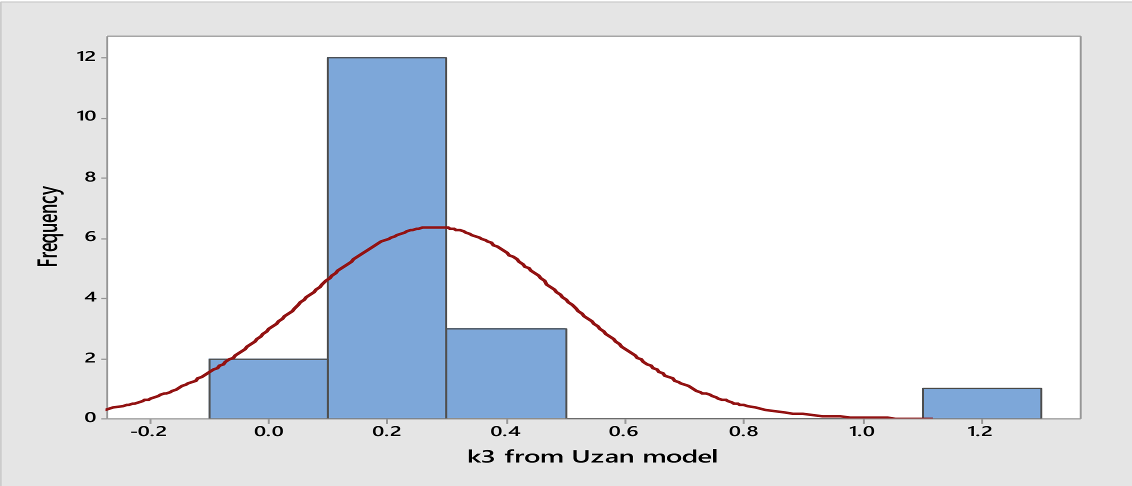


Figure 4.10c: $k_3$ofthe Coarse-Grained Soil Samples using the Uzan's Model

Table 4.21 presents the statistical summary of the resilient modulus parameters of coarse-grainedsoils obtained from the evaluation of Witczak and Uzan equation.

Table 4.21: Statistical Summaryof $k_i$for the Coarse-Grained SoilSamples using theWitczak and Uzan'sModel

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	1909.2	1410.11	843.4426	10058.81	2068.54	1.0000
k_2	3.0E-17	4.6E-17	-3.0E-16	3.53E-16	1.6E-16	3E-15
k_3	0.2807	0.2543	0.0295	1.1014	0.2255	1.E-15

Figure 4.11a-c present the histogram of the resilient modulus model parameters k_i obtained using the Witczak and Uzan resilient modulus model.

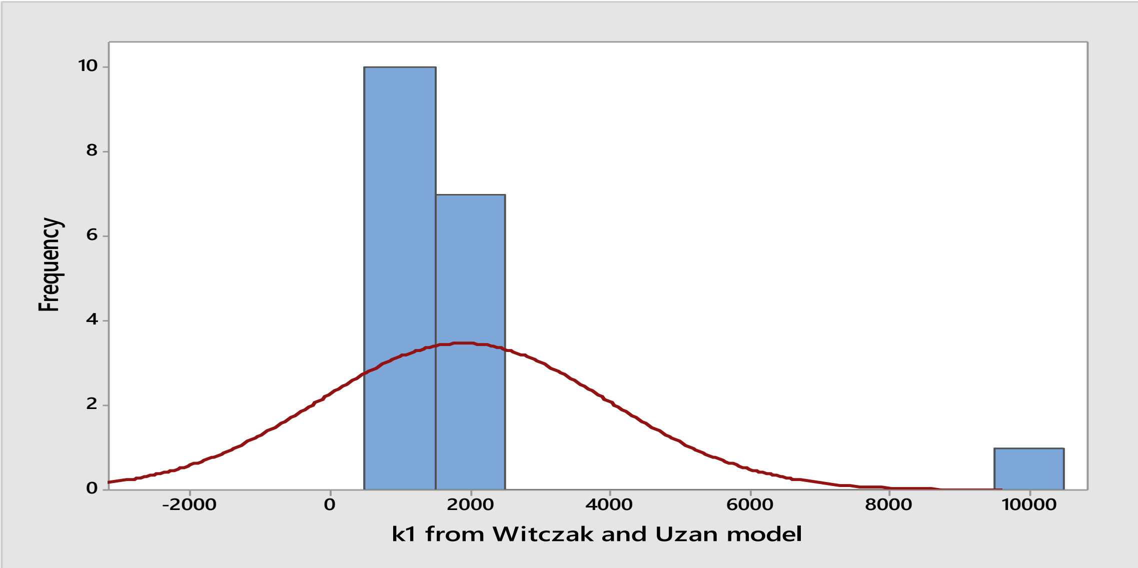


Figure 4.11a: $k_1$ofthe Coarse-Grained Soil Samples using the Witczak and Uzan's Model

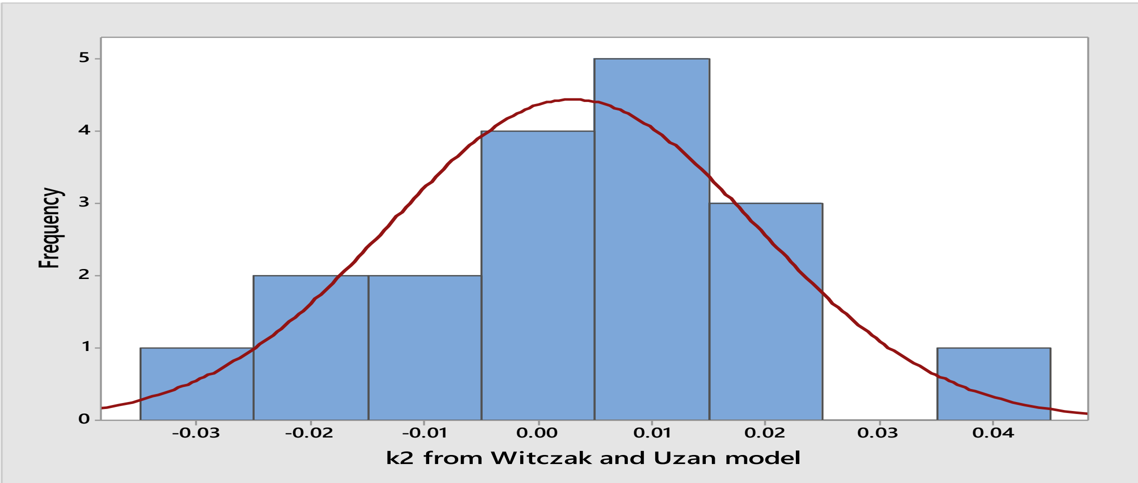


Figure 4.11b: $k_2$ofthe Coarse-Grained Soil Samples using the Witczak and Uzan's Model

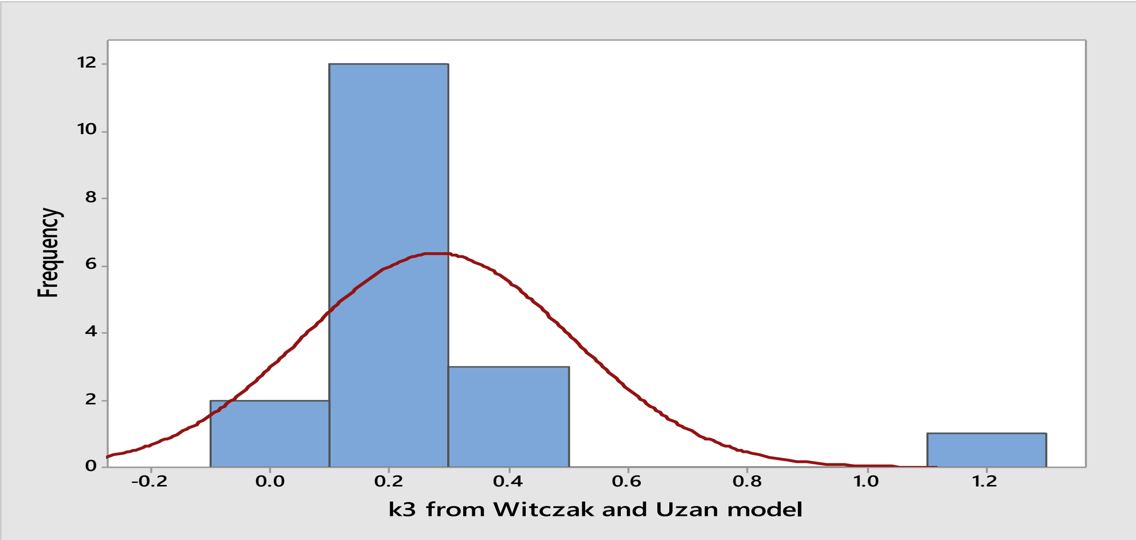


Figure 4.11c: $k_3$ofthe Coarse-Grained Soil Samples using the Witczak and Uzan's Model

Table 4.22 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Pezo resilient modulus equations.

Table 4.22: Statistical Summaryof $k_i$for the Coarse-Grained Soil Samples using thePezo'sModel

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	1522.50	1166.44	657.8956	7551.296	1538.68	1.0000
k_2	1.7E-17	2.8E-17	-2.2E-16	2.80E-16	1.3437	2.E-15
k_3	0.2807	0.2543	0.0295	1.1014	0.2255	10E-16

Figure 4.12a-c present the histogram of the resilient modulus model parameters k_i obtained using the Pezo resilient modulus model.

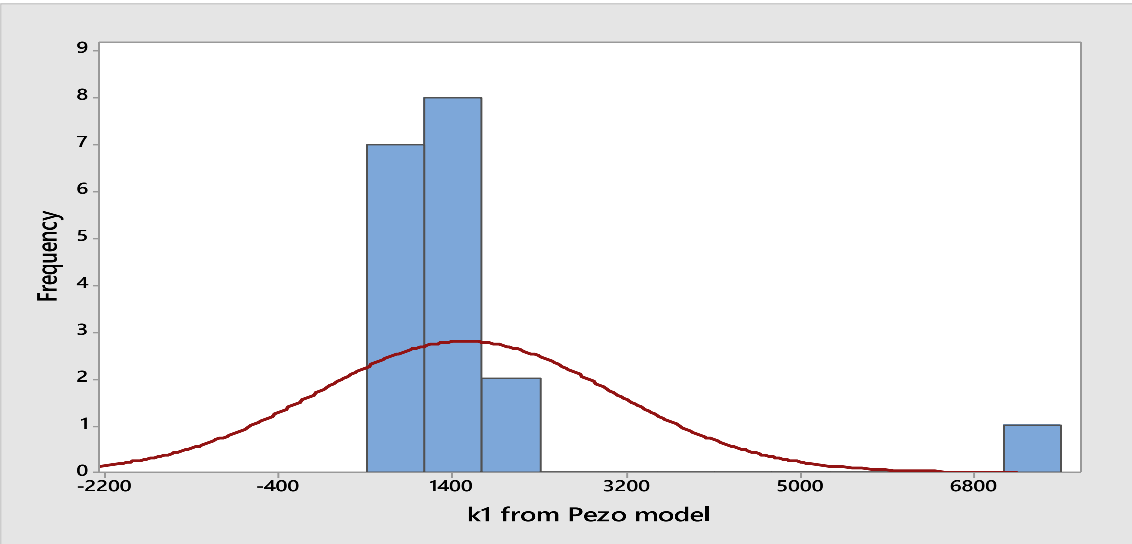


Figure 4.12a: $k_1$ofthe Coarse-Grained Soil Samples using the Pezo's Model

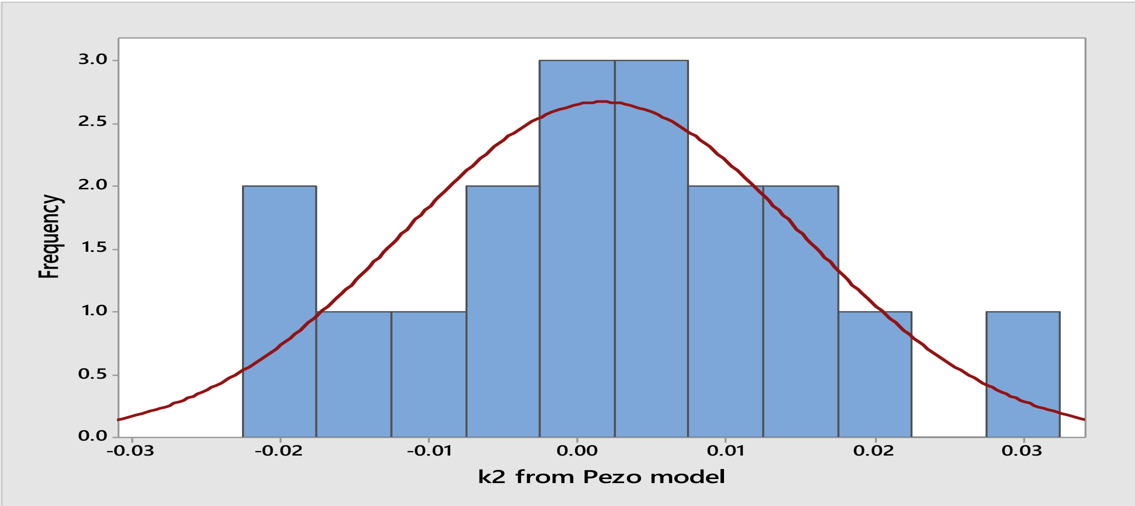


Figure 4.12b: $k_2$ofthe Coarse-Grained Soil Samples using the Pezo's Model

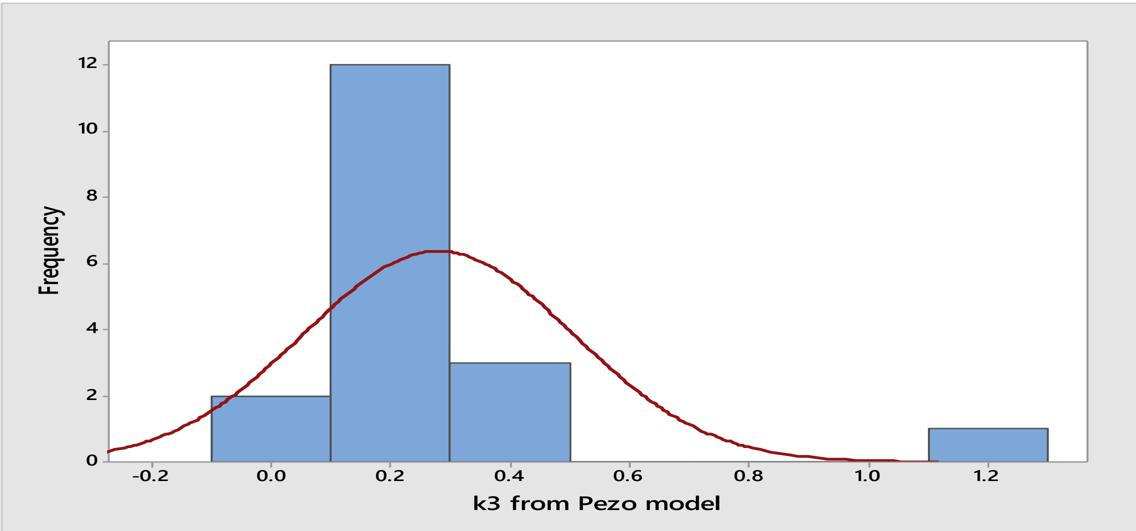


Figure 4.12c: $k_3$ofthe Coarse-Grained Soil Samples using the Pezo's Model

Table 4.23 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Ni et al resilient modulus equations.

Table 4.23: Statistical Summaryof $k_i$of the Coarse-Grained SoilSamplesusing theNi et al'sModel

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	807.98	715.1792	47.8460	3047.611	614.150	3.3769
k_2	-0.0506	-0.0458	-0.1984	-0.0053	0.0406	1.8022
k_3	0.9849	0.8925	0.1037	3.8649	0.7914	1.2478

Figure 4.13a-c present the histogram of the resilient modulus model parameters k_i obtained using the Ni et al resilient modulus model.

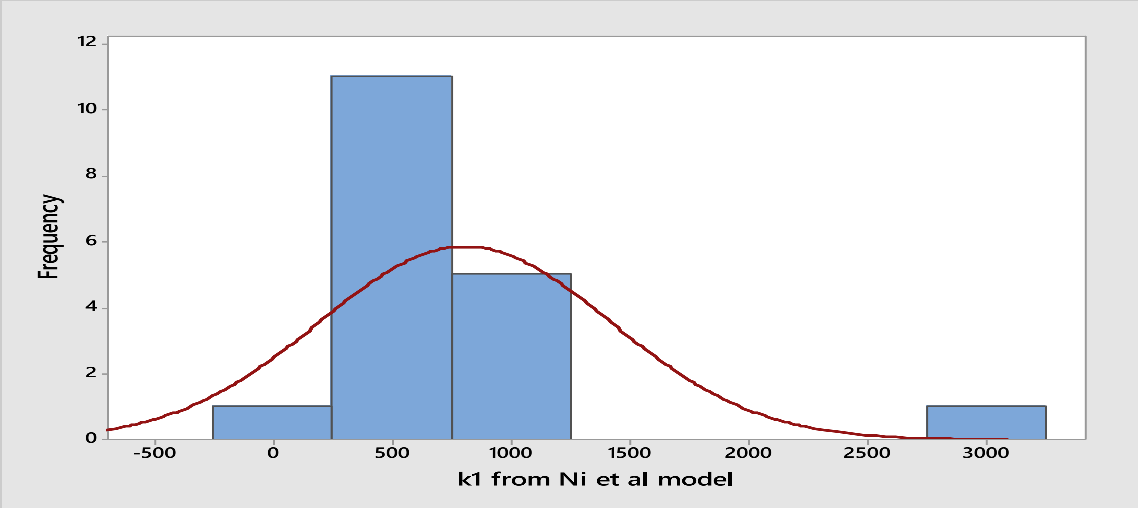


Figure 4.13a: $k_1$ofthe Coarse-Grained Soil Samples using the Ni et al's Model

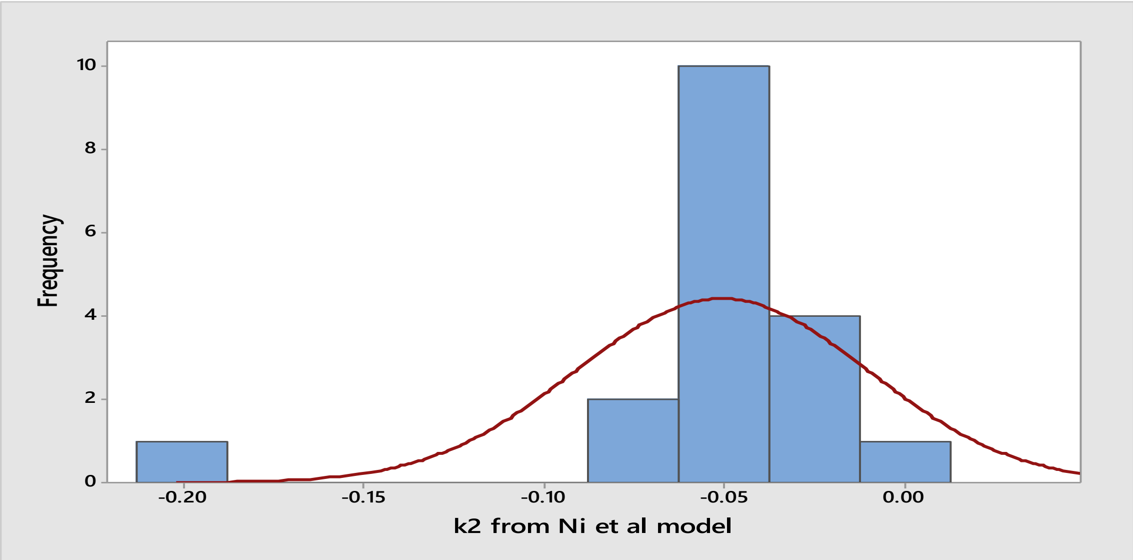


Figure 4.13b: $k_2$ofthe Coarse-Grained Soil Samples using the Ni et al's Model

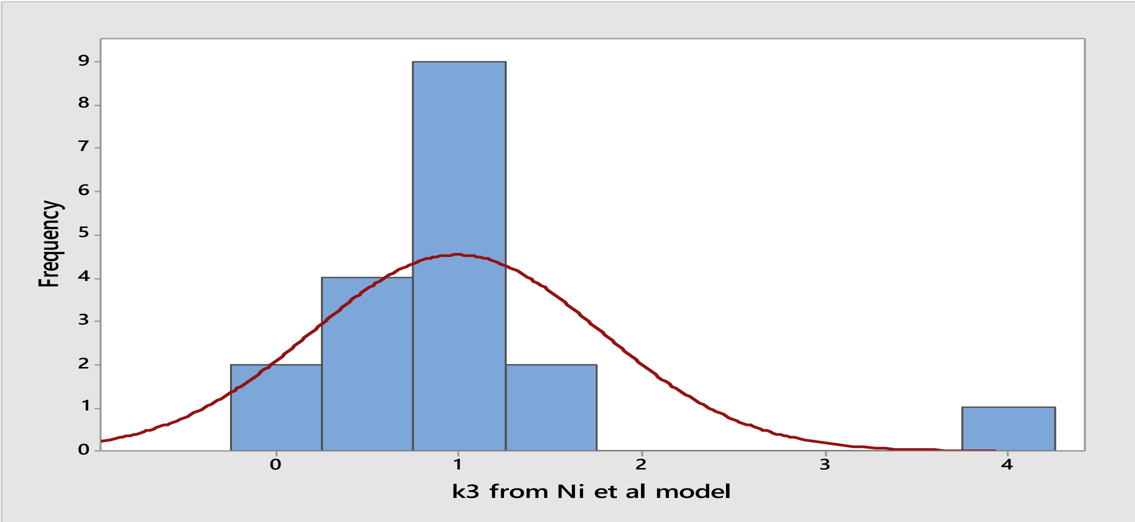


Figure 4.13c: $k_3$ofthe Coarse-Grained Soil Samples using the Ni et al's Model

Table 4.24 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Ooi et al A resilient modulus equations.

Table 4.24: Statistical Summaryof $k_i$of the Coarse-Grained SoilSamplesusing theOoi et al's Model A

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	816.94	722.8025	50.2513	3099.802	624.354	5.0619
k_2	-0.0353	-0.0319	-0.1383	-0.0037	0.0283	1.2629
k_3	0.9985	0.9048	0.1051	3.9182	0.8023	1.3911

Figure 4.14a-c present the histogram of the resilient modulus model parameters k_i obtained using the Ooi et al A resilient modulus model.

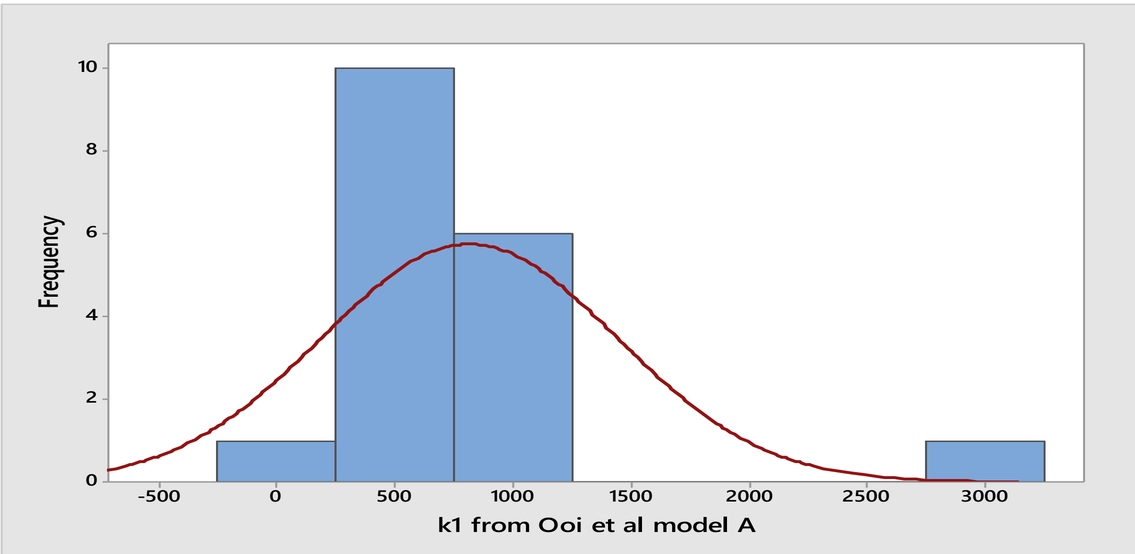


Figure 4.14a: $k_1$ofthe Coarse-Grained Soil Samples using the Ooi et al's Model A

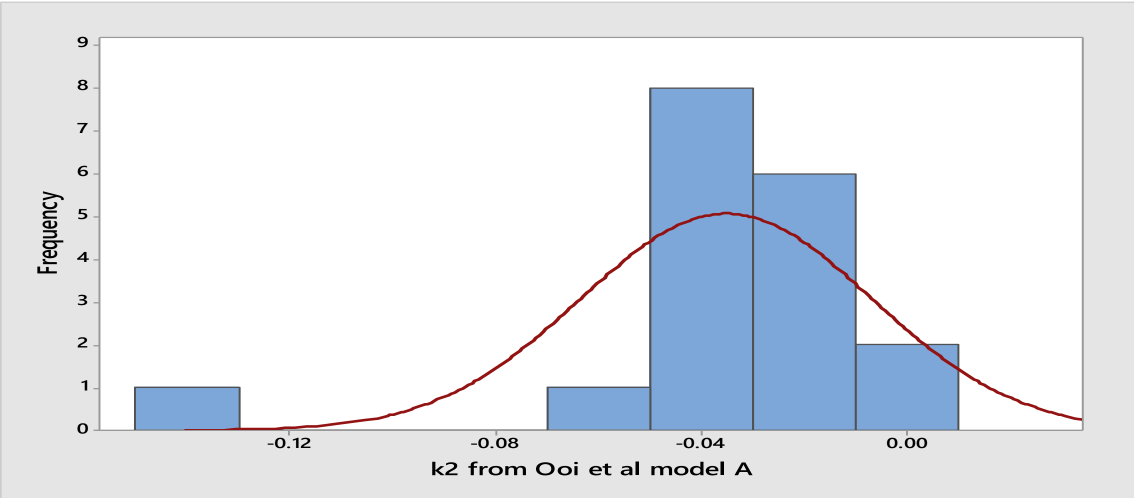


Figure 4.14b: $k_2$ofthe Coarse-Grained Soil Samples using the Ooi et al's Model A

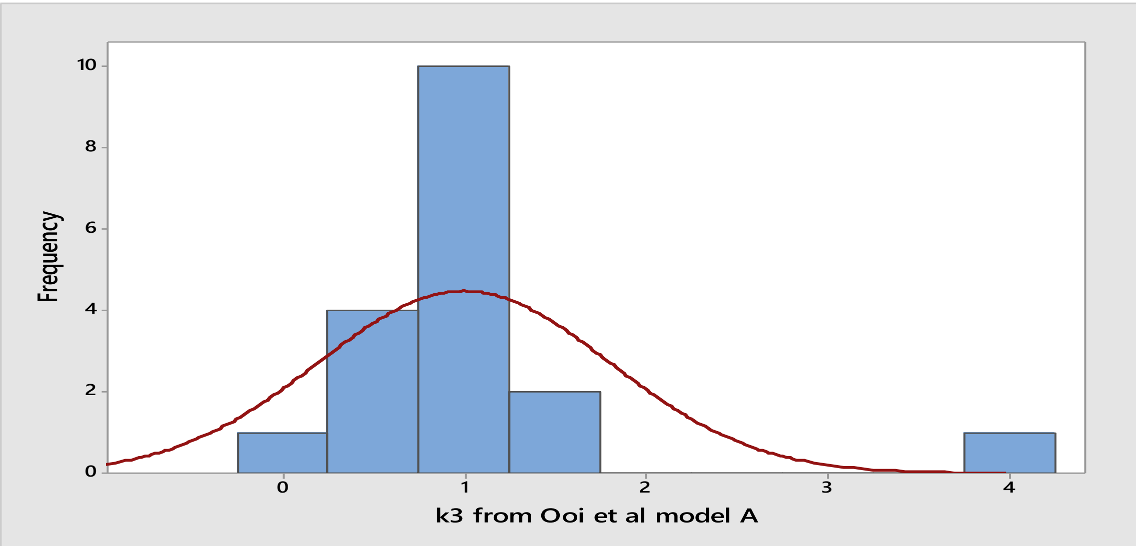


Figure 4.14c: $k_3$ofthe Coarse-Grained Soil Samples using the Ooi et al's Model A

Table 4.25 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Ooi et al B resilient modulus equations.

Table 4.25: Statistical Summaryof $k_i$of the Coarse-Grained SoilSamplesusing theOoi et al's Model B

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	841.31	743.4510	57.2416	3242.764	652.451	6.4635
k_2	-0.0353	-0.0320	-0.1384	-0.0037	0.0283	1.4516
k_3	1.6692	1.5127	0.1757	6.5504	1.3413	2.7114

Figure 4.15a-c present the histogram of the resilient modulus model parameters k_i obtained using the Ooi et al B resilient modulus model.

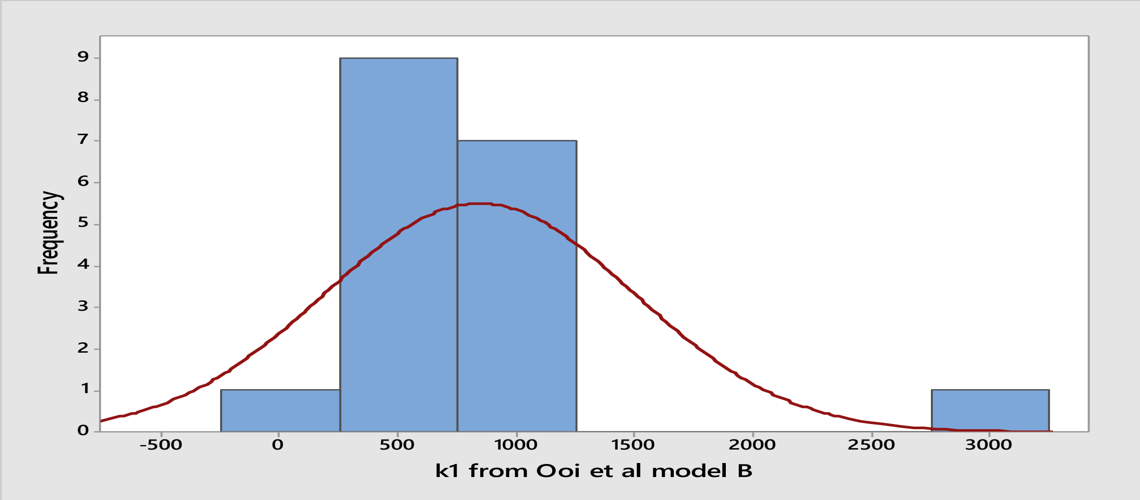


Figure 4.15a: $k_1$ofthe Coarse-Grained Soil Samples using the Ooi et al's Model B

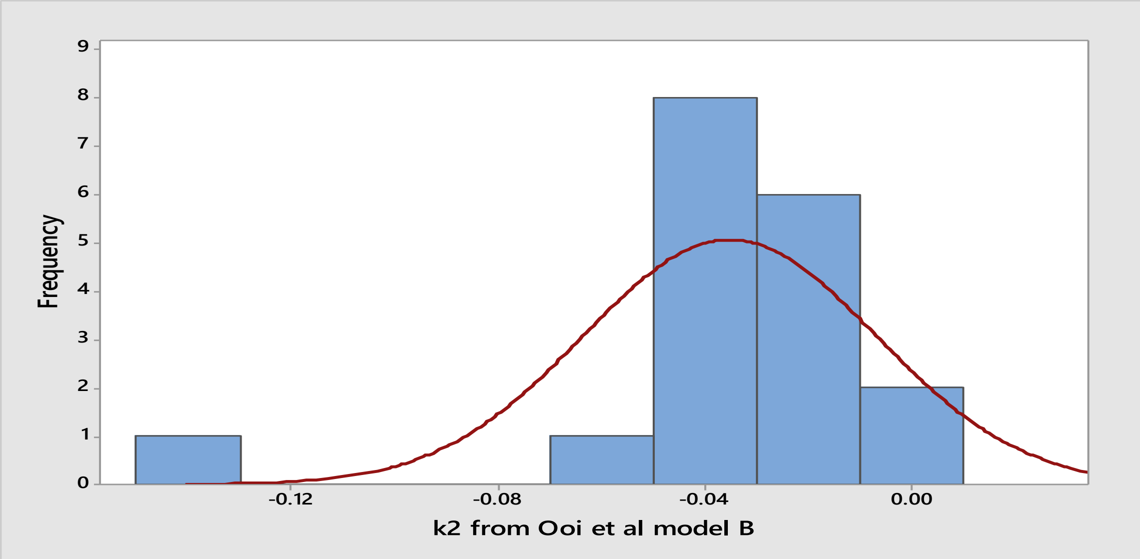


Figure 4.15b: $k_2$ofthe Coarse-Grained Soil Samples using the Ooi et al's Model B

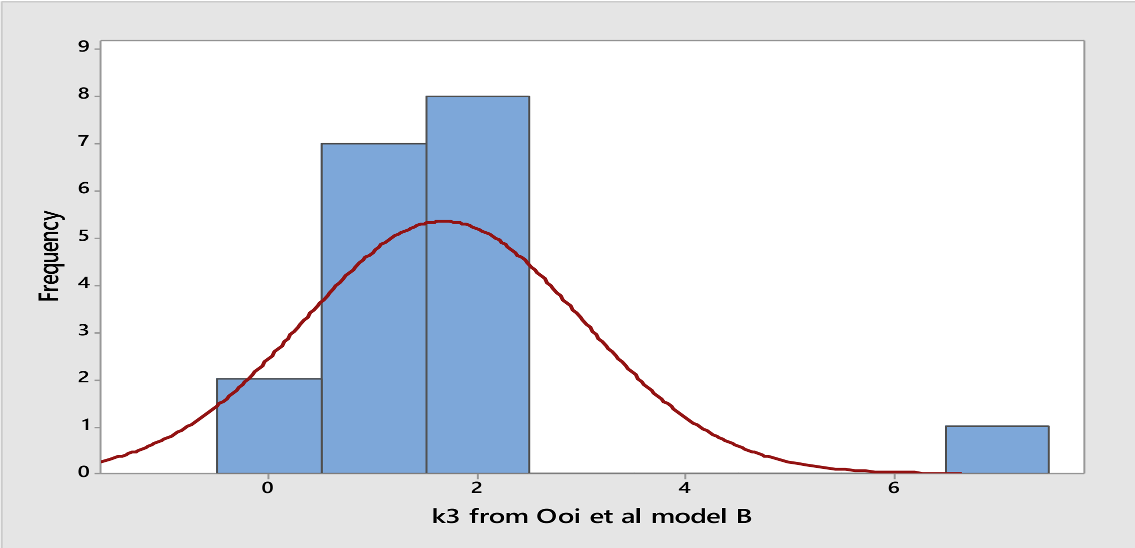


Figure 4.15c: $k_3$ofthe Coarse-Grained Soil Samples using the Ooi et al's Model B

Table 4.26 presents the statistical summary of the resilient modulus parameters ofcoarse-grained soils obtained from the evaluation of NCHRP resilient modulus equation.

Table 4.26: Statistical Summaryof $k_i$for the Coarse-Grained SoilSamples using theNCHRP's Model

Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
k_1	822.11	727.1963	51.6800	3130.028	630.277	2.9391
k_2	-0.0200	-0.0181	-0.0784	-0.0021	0.0161	1.0275
k_3	1.6625	1.5066	0.1750	6.5241	1.3359	2.7006

Figure 4.16a-c present the histogram of the resilient modulus model parameters k_i obtained using the NCHRP resilient modulus model.

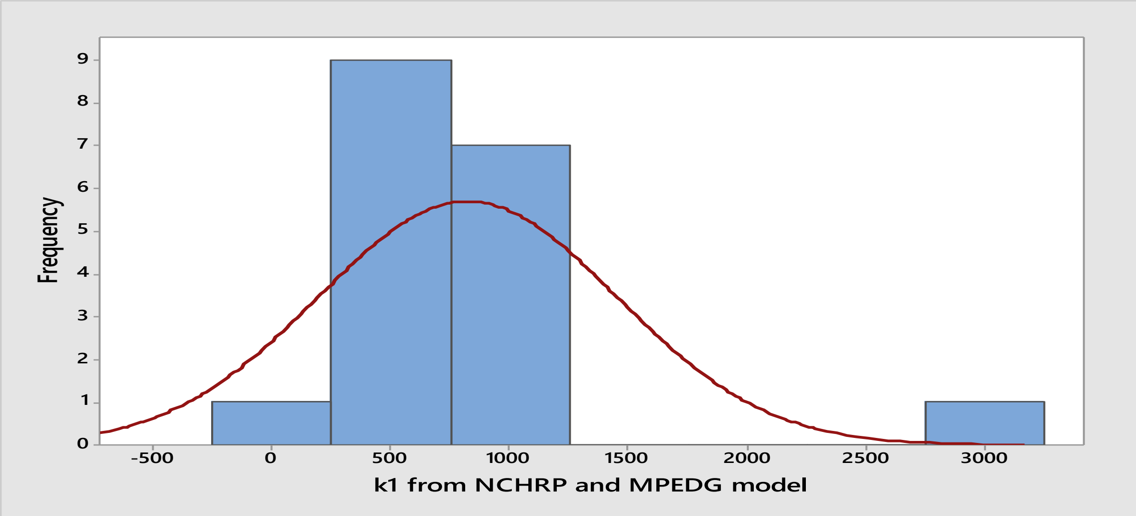


Figure 4.16a: $k_1$ofthe Coarse-Grained Soil Samples using the NCHRP's Model

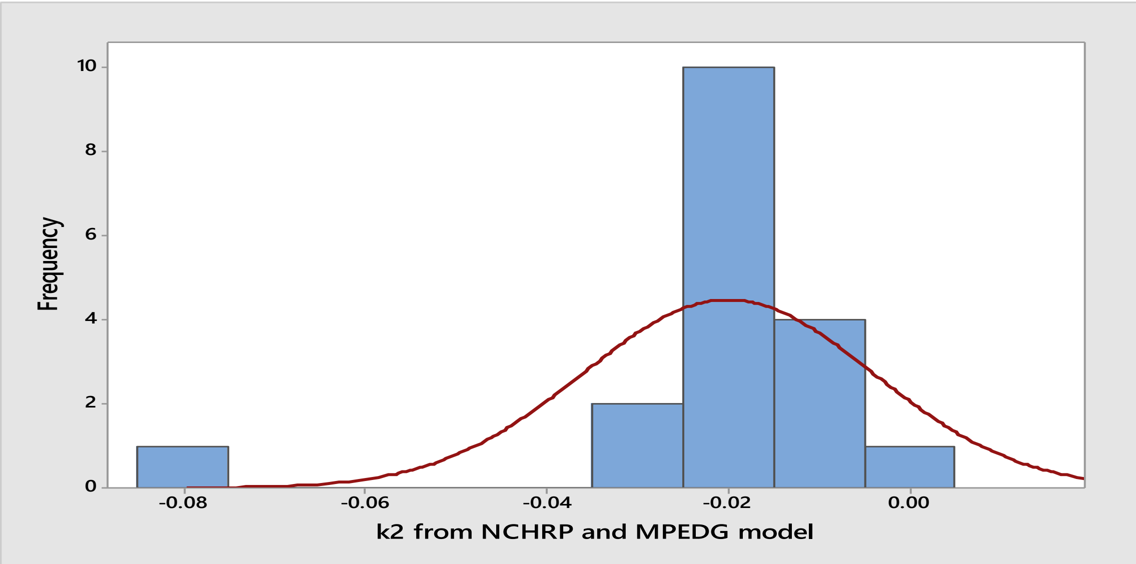


Figure 4.16b: $k_2$ofthe Coarse-Grained Soil Samples using the NCHRP's Model

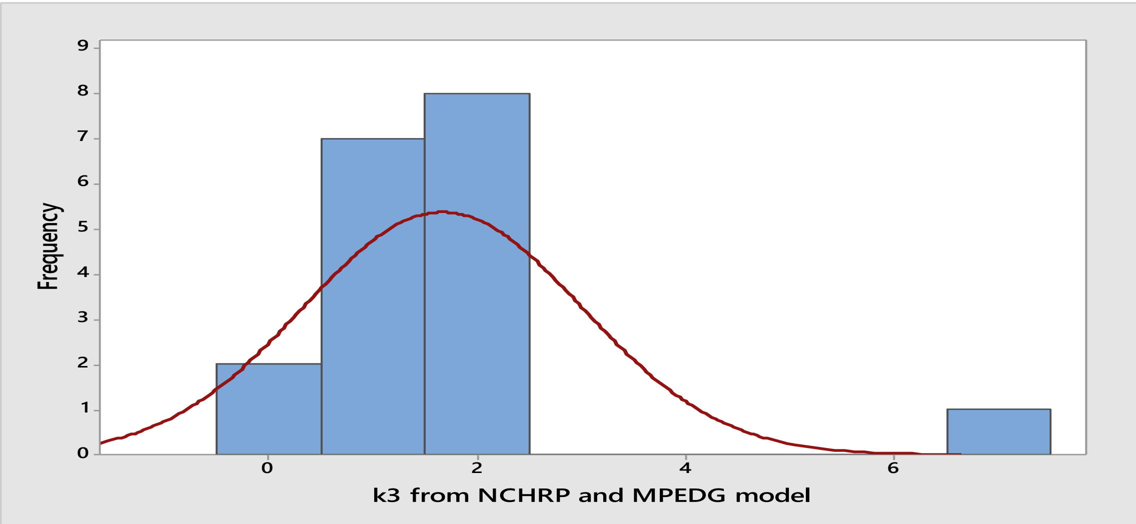


Figure 4.16c: $k_3$ofthe Coarse-Grained Soil Samples using the NCHRP's Model

Histograms showing the distribution of k_1, k_2, and k_3 values by soil class for the sevenresilient modulus equations that model resilient modulus behaviour for coarse-grained soilsused in this studyare as shown in Figures 4.17 through 4.19.

Figure 4.17 showed the relationship between resilient modulus parameter $k_1$obtained for coarse-grainedsoils and the resilient modulus equations evaluated in this research.

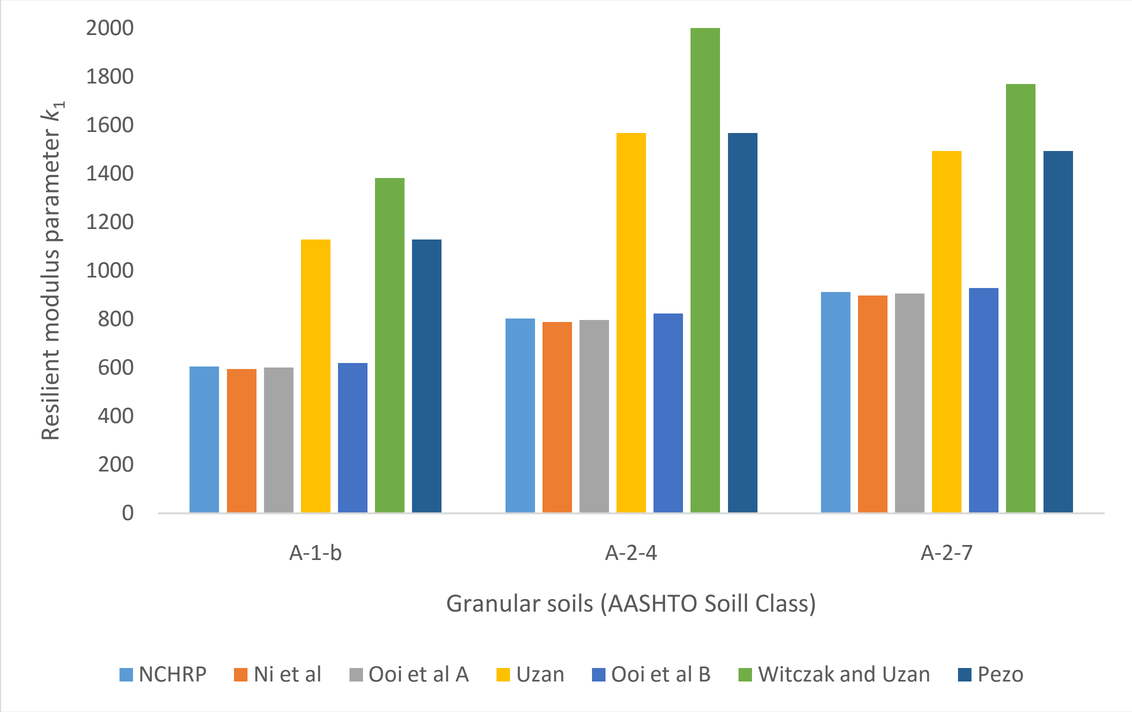


Figure 4.17: Resilient Modulus Parameter k_1 for Coarse-Grained Soils included in the Models

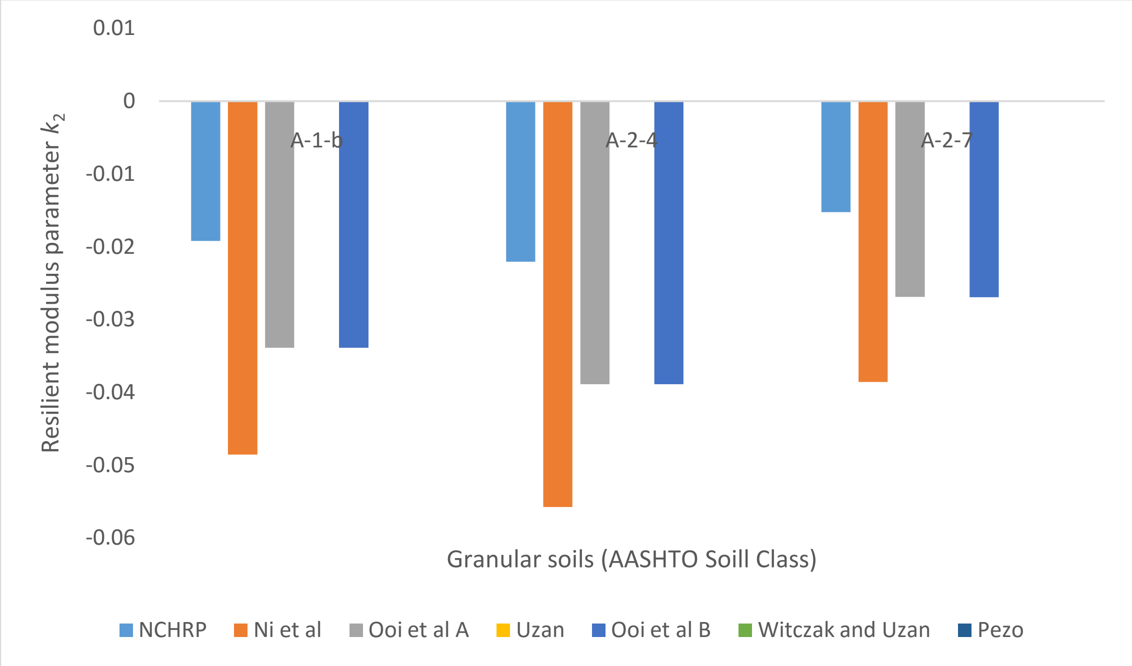


Figure 4.18: Resilient Modulus Parameter k_2 for Coarse-Grained Soils included in the Models

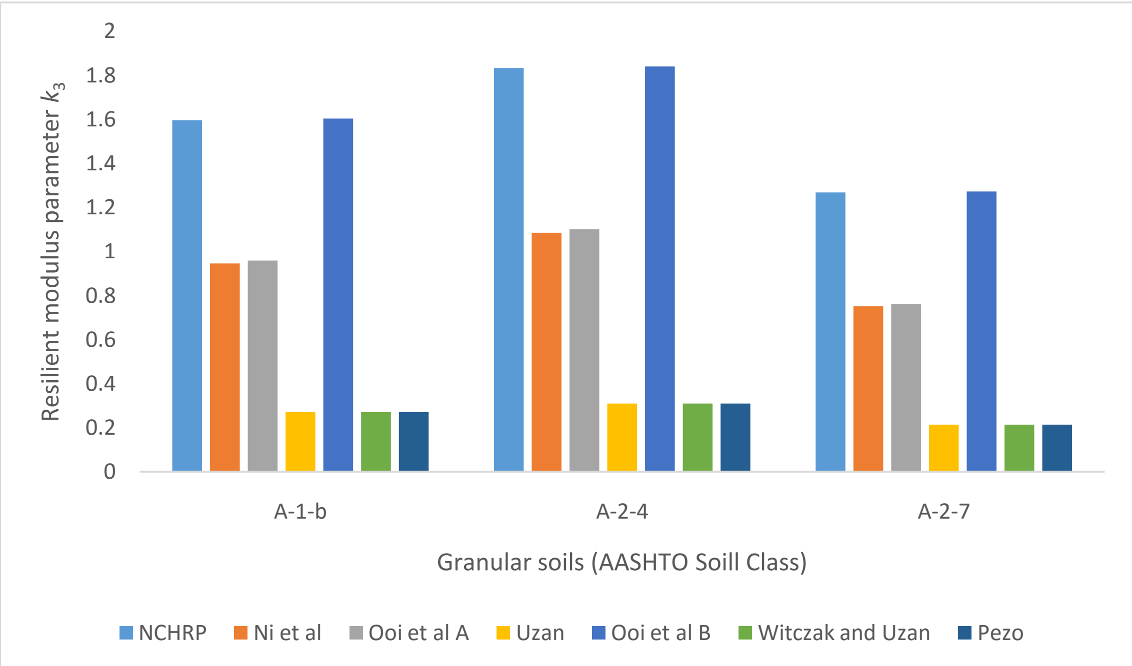


Figure 4.19: Resilient Modulus Parameter k_3 for Coarse-Grained Soils included in the Models

4.6 Statistical Analysis Results

This section discusses the statistical analyses performed to develop the predictive models and the sensitivity analyses used to validate the models. All statistical analyses were performed using the Microsoft Excel software program. The analysis was made in which fine-grained and coarse-grained soils data were used to develop correlations between the resilient modulus model parameters (k_1, k_2 and k_3) and selected soil properties separately and independently. Direct mathematical correlation between dependent and independent variables was adopted for the data formulation.

Table 4.27 presents a summary of the soil constituents based on particle size analysis.

Table 4.27: Constituents of the Subgrade Soil Samples from Master Test Section 1

MTS	Passing No. 200 (%)	% Clay Content	% Silt Content
MTS 1 – 1	72.42	49.76	22.72
	77.76	46.88	30.88
	48.66	29.60	19.04
MTS 1 – 2	22.24	8.48	13.76
	73.38	37.28	36.16
	75.84	42.88	32.96
MTS 1 – 3	66.34	39.84	26.56
	43.76	27.36	16.48
	55.94	33.44	22.40
MTS 1 – 4	88.22	69.28	18.88
	92.70	48.80	48.16
	85.76	61.60	24.16
MTS 1 – 5	57.82	18.08	39.68
	69.08	42.72	26.40
	71.36	50.08	21.28
MTS 1 – 6	57.60	17.28	40.32
	50.48	18.08	32.32
	49.76	30.88	18.88

4.7 Resilient Modulus Model DevelopmentResults for the SoilSamples

Tables 4.28-4.30 present summaries of the regression analysis results in which models to estimate k_1, k_2, and k_3 from basic soil properties were obtained. Tables D.1 through D.12 presents the summary output of the regression analysis of the models. Figures4.20 – 4.22 depicts comparisons between k_i values obtained from analysis of the results of the repeated load triaxial test (considered herein as measured values) and k_i values estimated from basic soil properties using the proposed correlations (Tables 4.28 – 4.30).

Table 4.28: Correlations between the Resilient Modulus Model Parameter k_1 and Basic Soil Properties for Fine-Grained Soils

Variable	k_1 correlations coefficients		
	Model 1	Model 2	Model 3
Intercept	97782.5176	63459.6000	5767.778
P_{200} (%)	-86.6774	-62.1078	-
P_4 (%)	-14.7650	-	-
P_{40} (%)	-25.4001	-26.5277	-15.6282
%Clay	100.5425	74.9752	11.97018
%Silt	105.5295	77.01873	10.23872
PL	57.4900	45.00211	19.20555
NMC (%)	-8.2568	-	-
OMC (%)	-301.8208	-195.942	-67.693
y_d (kg/m^3)	39.2704	25.79185	-
MDD (kg/m^3)	-55.9263	-36.2635	-1.78929
NMC/OMC	12541.0184	7707.483	300.9741
y_d/MDD	-64152.3227	-42352	-878.393
P_{200}/NMC	21.1872	15.60331	8.534821
$(NMC/OMC)*(y_d/MDD)$	-9865.3090	-6140.77	-
R^2	0.99	0.90	0.79

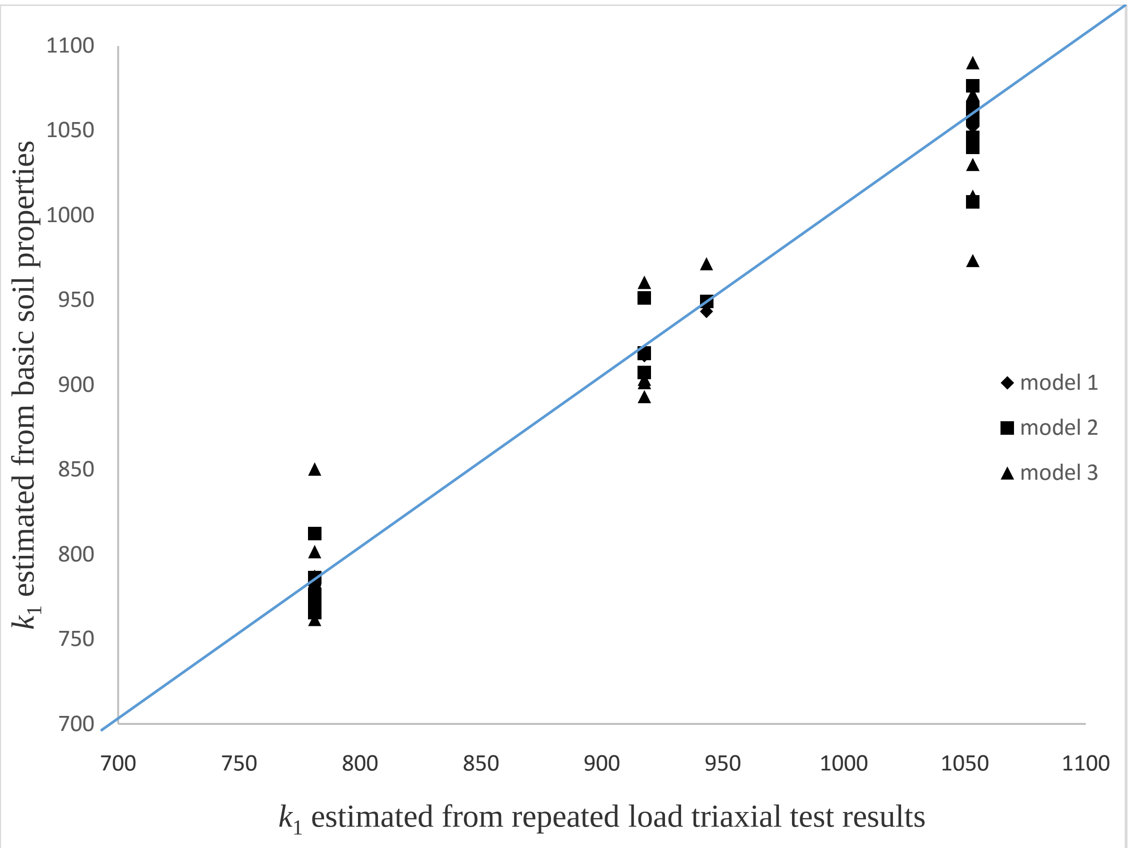


Figure 4.20: Comparison of Resilient Modulus Model Parameters (k_1) Estimated from Soil Properties and k_1 determined from Results of RLTTest on Fine-GrainedSoils

Table 4.29: Correlations between the Resilient Modulus Model Parameter k_2 and Basic Soil Properties for Fine-Grained Soils

Variable	k_2 correlations coefficients			
	Model 1	Model 2	Model 3	Model 4
Intercept	0.0278	0.0281	0.0309	0.0235
P_4 (%)	-1.2E-05	-1.6E-05	-	-
P_{40} (%)	-	7.63E-06	-	-
%Clay	6.85E-06		-	-
%Silt	1.02E-05	4.27E-06	4.07E-06	-
LL	-	1.55E-05	1.76E-05	1.49E-05
PL	-3E-05	-3.9E-05	-3.3E-05	-3.2E-05
NMC (%)	-	-0.0001	-0.0001	-8.6E-05
OMC (%)	-0.0001	-3.2E-05	-	-
y_d (kg/m^3)	1.15E-05	1.08E-05	1.45E-05	1.07E-05
MDD (kg/m^3)	-1.5E-05	-1.5E-05	-1.8E-05	-1.3E-05
NMC/*OMC*	0.0024	0.0022	0.0022	0.0016
y_d/*MDD*	-0.0197	-0.0187	-0.0245	-0.0181
R^2	0.67	0.88	0.67	0.62

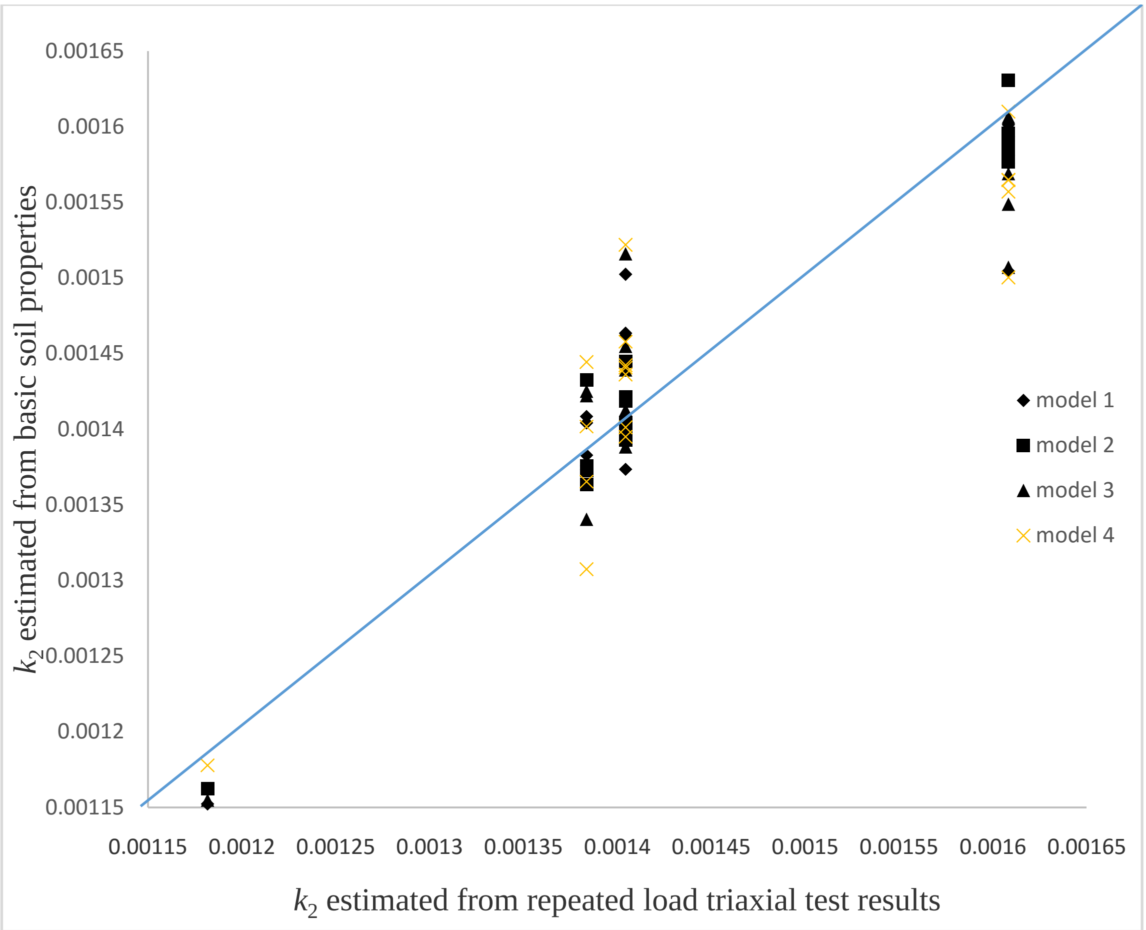


Figure 4.21: Comparison of Resilient Modulus Model Parameters (k_2) Estimated from Soil Properties and k_2 determined from Results of RLTTest on Fine-GrainedSoils

Table 4.30: Correlations between the Resilient Modulus Model Parameter k_3 and Basic Soil Properties for Fine-Grained Soils

Variable	k_3 correlations coefficients			
	Model 1	Model 2	Model 3	Model 4
Intercept	4.2623	15.8009	19.1569	29.3710
P_{200} (%)	-	-0.0048	0.0057	-
P_4 (%)	-0.0231	-0.0097	-0.0104	-0.0081
P_{40} (%)	0.0129	0.0104	-	-
%Clay	-	-	-	0.0034
%Silt	-	0.0025	-	0.0076
PL	-0.0315	-0.0332	-0.0275	-0.0343
LL	-	0.0111	-	0.0138
NMC (%)	-	-0.0817	-0.0879	-0.1164
OMC (%)	-0.0289	-	-	-0.0193
y_d (kg/m^3)	-0.0014	0.0063	0.0074	0.0123
MDD (kg/m^3)	-	-0.0080	-0.0097	-0.0159
NMC/OMC	0.2414	1.4827	1.7219	2.2724
y_d/MDD	1.7394	-10.7471	-12.7855	-21.0219
P_{200}/NMC	-	-0.0024	-	-

R^2	0.59	0.95	0.63	0.83

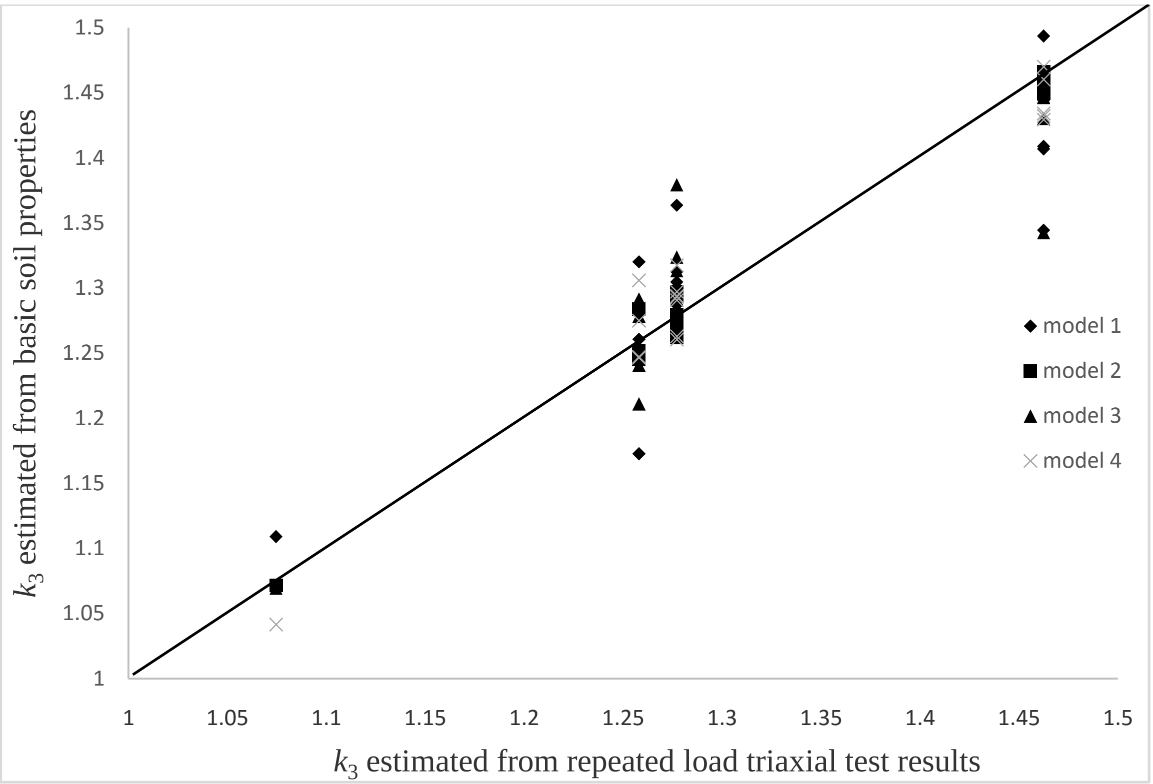


Figure 4.22: Comparison of Resilient Modulus Model Parameters (k_3) Estimated from Soil Properties and k_3 determined from Results of RLTTest on Fine-GrainedSoils

CHAPTER FIVE

DISCUSSION

5.1 Physical Properties and Compaction Characteristics of the Soil Samples

This section discussed the results of the physical and compaction properties of the soil samples. The physical properties are Atterberg limits, linear shrinkage, particle size distribution, specific gravity, soil classification, compaction characteristics and natural moisture content of the soil samples.

5.1.1 Atterberg limits of the soil samples

Table 4.1 showed the summary of theAtterberg limits test resultsof the soil samples. As shown in Table 4.1, the liquid limit of the soil samples ranged from 29 to 45 with plasticity index rangingfrom8 and 19. Table 4.1 indicated that most of the soil samples are materials with a high liquid limit and plasticity index. This provide an indication of the presence of clay in the soil samples and thereby will be unsuitable for many construction applications.

Table 4.2 presents the linear shrinkage test resultsof the soil samples. Table 4.2 showed that shrinkage can be significant in clays but less in silts.

5.1.2 Particle size distribution of the soil samples

Table 4.3 showed the summary of the sieve analyses tests results of the soil samples. As shown in Table 4.3, the percent of silt content in the soil samples ranged from 13.76% to 48.16% and the percent of clay content in the soil samples ranged from 8.48% to 69.28%. Also, Table 4.3 showed that most of the soil samples had more than 50% finer materials with smaller fraction of sand present and have particle sizes smaller than 75 µm. This indicate that the soil samples contains more of clayed and silty materials than gravel and/or sand. This provides a useful information for the engineering classification of the soil.

5.1.3 Specific gravity of the soil samples

Specific gravity tests were conducted on the soil samples and the results are summarized in Table 4.4. As shown in Table 4.4, the specific gravity for A-4 group ranged from 2.46 to 3.31. The specific gravity for A-2-4 and A-5 groups were2.57 and 2.50 respectively. The specific gravity for A-6 group ranged between 2.44 to 2.51. In the case of A-7-6 group, the specific gravity ranged from 2.53 to 2.58. The specific gravity in Table 4.4was used in calculating the degree of saturation for the soil samples. The specific gravity was also used to calculate the density of the soil. Table 4.4 showed that the specific gravity of the soil samples are greater than one, thereby do not require special treatment.

5.1.4 Soil classification of the soil samples

The soil samples was classified using USCS and AASHTO Soil Classification. Table 4.5 summarized theUSCS and the AASHTO Classification of the soil. According to USCSand the AASHTO Soil Classification system, the soil type for each of the soil sample was identified on the basis of the results of Atterberg limit, and particle size distribution tests (see Table 4.1 and Figures A.1 through A.18). For the USCS, most of the soil samples belongs to the group of CL (low plasticity clay). The USCS identifies that the soil samples are fine-grained soils and mostly clay. The AASHTO soil classification shows that the soil samples belongs to Clayed soil (A-6 and A-7-6) or silty soils (A-4, A-5 and A-2-4) group. From Table 4.5, the soil classification generally showed that the soil samples were “fair to poor” in subgrade properties.

5.1.5 Compaction characteristics of the soil samples

Table 4.6 showed the summary of thecompaction test results of the soil samples with its soil type. From Table 4.6, OMC for the soil ranged between 13.3% and 22.3%. This implies that most of the soil samples have OMC values higher than the standard OMC values of 16.6% for lean clay. Based on the OMC values showed in Table 4.6, it can be concluded that most of the soil samples had fat clay (CH) in its content. The MDD for soil samples ranges between 15.16kN/m^3 and 18.48kN/m^3. This indicated that the MDDof most of the soil samples are within the standardMDDof 17.15kN/m^{3}specified for soil samples with lean clay (CL). This also shows that there was presence of fat clay (CH) within the soil samples. This provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to assure that the required compaction and water contents are achieved.

5.1.6 Natural moisture content of the soil samples

Table 4.7 present theNWC test results of the soil samples with its Soil Type. From Table 4.7, the NWC for A-4 group ranged between 0.7% and 14.3%. NWC for A-2-4and A-5 groups were0.9% and 7.7% respectively. NWC for A-6 group ranged between 1.6% and7.5%. NWC for A-7-6 group was the highest (2.4% to and 10.4%). The water content is one of the most significant index properties used in establishing a correlation between soil behaviour and its index properties. Also, from Table 4.7, it can concluded that for the fine-grained (cohesive) soils, the consistency of the soil type depends on its water content.

5.2 California Bearing Ratio of the Soil Samples

The summary of the California Bearing Ratio values obtained on the soil samples are summarized in Table 4.8. Table 4.8 showed that the CBR values of the soil samples were generally less than 3%. This implies that generally the soil samples are suitable for use as subgrades for highway construction except if capping is applied.

The resilient modulus equations stated in Equations 2.1 through 2.4 were used to obtained resilient modulus for the soil samples obtained from the Master Test Section 1. Table 5.1 presents the resilient modulus values of the soil samples obtained from the Master Test Section 1 using California Bearing Ratio.

Table 5.1: Resilient modulus values obtained using CBR for the Samples from MTS 1

MTS	Heukelom and Klomp		Powell et al		Putri, et al (CBR<5)		Putri, et al (CBR>5)	
	Unsoaked	Soaked	Unsoaked	Soaked	Unsoaked	Soaked	Unsoaked	Soaked
MTS 1-1	23,787	29,579	29,993	34482	29,022	33,804	33,971	37,882
	10,549	14,065	17,824	21428	16,426	20,091	22,623	26,123
	46,746	16,341	46,217	23586	46,571	22,314	47,623	28,156
MTS 1-2	21,822	18,616	28,383	25638	27,322	24,446	32,538	30,053
	15,617	23,580	22,912	29826	21,617	28,845	27,526	33,823
	15,617	14,686	22,912	22028	21,617	20,707	27,526	26,693
MTS 1-3	6,619	18,616	13,227	25638	11,853	24,446	17,920	30,053
	44,678	28,648	44,898	33783	45,118	33,056	46,558	37,281
	14,065	25,235	21,428	31149	20,091	30,247	26,123	34,990
MTS 1-4	14,893	10,963	22,226	18269	20,911	16,874	26,880	23,062
	23,477	20,271	29,742	27074	28,756	25,947	33,749	31,360
	8,170	13,962	15,135	21327	13,736	19,987	19,910	26,026
MTS 1-5	38,990	60,812	41,150	54691	41,016	55,986	43,493	54,317
	10,135	80,979	17,374	65693	15,973	68,415	22,175	62,680
	132586	9,308	90,066	16452	96,613	15,048	80,203	21,251
MTS 1-6	18,719	19,547	25,729	26451	24,541	25,295	30,136	30,795
	17,685	26,476	24,810	32121	23,584	31,281	29,292	35,840
	8,377	22,339	15,380	28811	13,978	27,774	20,160	32,921

5.3 Unconfined Compressive Strength (UCS) Test of the Soil Samples

Table 4.9 present the UCS test results of the soil samples with its soil type. As shown in Table 4.9, the UCS for A-4 group werefound to range from 39.9 kN/m^2 to 534.6 kN/m^2. The UCS for A-6 group ranged between 70.7 kN/m^2 to 549.2 kN/m^2. In the case of A-7-6 group, the UCS ranged from 17.8 kN/m^2 to 398.0 kN/m^2. The primary purpose of the unconfined compression test was to quickly obtain the approximate compressive strength of soils that possess sufficient cohesion to permit testing in the unconfined state. This test method provides an approximate value of the strength of cohesive soils in terms of total stresses.

5.4 Statistical Analysis of Resilient Modulus of Nigerian Soil Samples

The statistical summary of the resilient modulus of Nigerian soil samples (Claros, et al., 1986)are shown in Tables4.10 – 4.13. From Table 4.10,the coefficient of variation ranges between 0.32 and 0.50% for fine-grained soils obtained from MTS 1 and from 0.28 to 0.30% for fine-grained soils obtained from MTS 2. This indicates that each soil specimen showed consistent behaviour during each test. Inspection of Table 4.10 indicates that the resilient modulus of the fine-grained soils obtained from MTS 1 and 2 increases with the increase in the deviator stress (σ_d) under constant confining pressure (σ_c). It also showed that the resilient modulus increases with the increase in confining pressure under constant deviator stress.

Table 4.11 presents the mean resilient modulus values, standard deviation, and coefficient of variation of the resilient modulus of the soil samples obtained from MTS 3 and 4. The coefficient of variation ranges between 0.22 and 0.45% for fine-grained soils obtained from MTS 3 and from 0.18 to 0.41% for fine-grained soils obtained from MTS 4. This indicates that fine-grained soils obtained from MTS 3 and 4 showed consistent behaviour during each test sequence.Inspection of Table 4.11 indicates that the resilient modulus of the fine-grained soils obtained from MTS 3 and 4 decreases with increase in the deviator stress (σ_d) under constant confining pressure (σ_c). The fine-grained soil samples obtained from MTS 3 and 4 typically display a decreases in resilient modulus with increasing deviator stress, thisis in consistent with other researchers (Sandefur, 2003; Titi*et al*., 2006). Also, Table 4.11 showed that the confining stress has a less significant effect than deviator stress for the fine-grained soils samples obtained from MTS 3 and 4.

Table 4.12 presents the mean resilient modulus values, standard deviation, and coefficient of variation of the resilient modulus of fine-grained soilsobtained from MTS 5 and 6. The coefficient of variation ranges between 0.45 and 0.49% for the fine-grained soils obtained from MTS 5 and from 0.29 to 0.40% forfine-grained soils obtained from MTS 6. This indicates that each soil specimen showed consistent behaviour during each test sequence. Inspection of Table 4.12 indicates that the resilient modulus of the subgrade soils (fine) obtained from MTS 5 and 6 increases with the increase of the deviator stress (σ_d) under constant confining pressure (σ_c). Moreover, the resilient modulus increases with increase in confining pressure under constant deviator stress.

Table 4.13 presents the mean resilient modulus values, standard deviation, and coefficient of variation of the resilient modulus of coarse-grained soilsobtained from MTS 1 – 6. The coefficient of variation ranges between 0.72 and 1.06% for specimens. This indicates that each soil specimen showed consistent behaviour during each test sequence. Inspection of Table 4.13 indicates that the resilient modulus of coarse-grained soilsincreases with the increase of the deviator stress (σ_d) under constant confining pressure (σ_c). Moreover, the resilient modulus increases with the increase of confining pressure under constant deviator stress.

5.5 Constitutive Equation Coefficients of Nigerian Soils

This section presents the discussion of resilient modulus model parameters (constitutive equation coefficients) evaluated for Nigerian soils.

5.5.1 Constitutive equation coefficients of fine-grained soil

The statistical summary of the resilient modulus of fine-grained soils presented in Tables 4.10 through 4.12 were used in evaluating the resilient modulus parameters of the fine-grained soils. Tables4.14 through 4.19 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the resilient modulus equations.

Table 4.14 presents the statistical summary of the resilient modulus parameters of fine-grained soilobtained from the evaluation of Uzan model.Table 4.14showed that the magnitude of k_1 was always greater than zero. This implies that resilient modulus should always be greater than zero. Also, the parameter k_2 which, is related to the confining stress,was close to zero since the resilient modulus of the fine-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. From the evaluation, the resultant resilient modulus constitutive equation for fine-grained soils for A-2-6, A-2-7, A-4, A-5, A-6, A-7-5 and A-7-6 using Uzan's resilient modulus model are presented in Equations 5.1a – 5.1g respectively.

$$M_R = 1303.0.96P_a \left(\frac{\sigma_d}{P_a}\right)^{0.2589} \quad (5.1a)$$

$$M_R = 1243.699 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.2726} \quad (5.1b)$$
$$M_R = 1367.853 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.1747} \quad (5.1c)$$
$$M_R = 663.841 P_a \left(\frac{\sigma_d}{P_a}\right)^{-0.1251} \quad (5.1d)$$
$$M_R = 1224.321 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.1702} \quad (5.1e)$$
$$M_R = 1274.613 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.0788} \quad (5.1f)$$
$$M_R = 1208.953 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.0156} \quad (5.1g)$$

Observing equations 5.1a – 5.1g, the terms for the confining stress is missing comparing to the Uzan resilient modulus constitutive equation. This is because the values of k_2 tends to zero. This showed that the confining stress has a less significant effect than deviator stress for fine-grained soils.

Table 4.15 presents the statistical summary of the resilient modulus parameters of fine-grained soil obtained from the evaluation of Witczak and Uzan resilient modulus equations.Table 4.15showed that $k_1$the magnitude of k_1 was always greater than zero. This implies that resilient modulus should always be greater than zero. The parameter $k_2$were close to zero since the resilient modulus were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero which showed that the resilient modulus increases with increase in deviator stress at constant confining stress. From the evaluation, the resultant resilient modulus constitutive equation for fine-grained soils for A-2-6, A-2-7, A-4, A-5, A-6, A-7-5 and A-7-6 using Witczak and Uzan's resilient modulus model are presented in Equations 5.2a – 5.2g respectively.

$$M_R = 1570.418 P_a \left(\frac{\tau_{oct}}{P_a}\right)^{0.2589} \quad (5.2a)$$
$$M_R = 1499.621 P_a \left(\frac{\tau_{oct}}{P_a}\right)^{0.2726} \quad (5.2b)$$

$$M_R = 1583.395P_a\left(\frac{\tau_{oct}}{P_a}\right)^{0.1747} \quad (5.2c)$$
$$M_R = 602.885P_a\left(\frac{\tau_{oct}}{P_a}\right)^{-0.1251} \quad (5.2d)$$
$$M_R = 1391.513P_a\left(\frac{\tau_{oct}}{P_a}\right)^{0.1702} \quad (5.2e)$$
$$M_R = 1368.769P_a\left(\frac{\tau_{oct}}{P_a}\right)^{0.0788} \quad (5.2f)$$
$$M_R = 1275.309P_a\left(\frac{\tau_{oct}}{P_a}\right)^{0.0156} \quad (5.2g)$$

Observing Equations 5.2a – 5.2g, the terms for the confining stress is missing comparing to the Witczak and Uzan's resilient modulus constitutive equation. This is because the values of k_2 tends to zero. This showed that the confining stress has a less significant effect than deviator stress for fine soils.

Tables4.16 presents the statistical summary of the resilient modulus parameters of fine-grained soil obtained from the evaluation of Ni et al model.Table 4.16showed that the magnitude of k_1 was always greater than zero. This showed that resilient modulus should always be greater than zero. The parameter $k_2$were always less than zero since the resilient modulus of the fine-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$showed that the resilient modulus increases with increase in deviator stress at constant confining stress. From the evaluation, the resultant resilient modulus constitutive equation for fine-grained soils for A-2-6, A-2-7, A-4, A-5, A-6, A-7-5 and A-7-6 using Ni et al's resilient modulus model are presented in Equations 5.3a – 5.3g respectively.

$$M_R = 681.205P_a\left(1+\frac{\sigma_3}{P_a}\right)^{-0.0117}\left(1+\frac{\sigma_d}{P_a}\right)^{1.2271} \quad (5.3a)$$
$$M_R = 679.963P_a\left(1+\frac{\sigma_3}{P_a}\right)^{-0.0123}\left(1+\frac{\sigma_d}{P_a}\right)^{1.2920} \quad (5.3b)$$
$$M_R = 854.863P_a\left(1+\frac{\sigma_3}{P_a}\right)^{-0.0079}\left(1+\frac{\sigma_d}{P_a}\right)^{0.8279} \quad (5.3c)$$
$$M_R = 952.346P_a\left(1+\frac{\sigma_3}{P_a}\right)^{0.0056}\left(1+\frac{\sigma_d}{P_a}\right)^{-0.5928} \quad (5.3d)$$

$$M_R = 771.841P_a\left(1+\frac{\sigma_3}{P_a}\right)^{-0.0077}\left(1+\frac{\sigma_d}{P_a}\right)^{0.8067} \quad (5.3e)$$
$$M_R = 1192.951P_a\left(1+\frac{\sigma_3}{P_a}\right)^{-0.0036}\left(1+\frac{\sigma_d}{P_a}\right)^{0.3734} \quad (5.3f)$$
$$M_R = 1234.351P_a\left(1+\frac{\sigma_3}{P_a}\right)^{-0.0007}\left(1+\frac{\sigma_d}{P_a}\right)^{0.0738} \quad (5.3g)$$

Tables4.18 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the evaluation of Ooi et al A resilient modulus equations.

Table 4.17showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter k_2 which, is related to the confining stress,were always less than zero since the resilient modulus of the fine-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. From the evaluation, the resultant resilient modulus constitutive equation for fine-grained soils for A-2-6, A-2-7, A-4, A-5, A-6, A-7-5 and A-7-6 using Ooi et al A's resilient modulus model are presented in Equations 5.4a – 5.4g respectively.

$$M_R = 682.240P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0089}\left(1+\frac{\sigma_d}{P_a}\right)^{1.2336} \quad (5.4a)$$
$$M_R = 680.872P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0093}\left(1+\frac{\sigma_d}{P_a}\right)^{1.2989} \quad (5.4b)$$
$$M_R = 855.726P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0060}\left(1+\frac{\sigma_d}{P_a}\right)^{0.8323} \quad (5.4c)$$
$$M_R = 951.494P_a\left(1+\frac{\theta}{P_a}\right)^{0.0043}\left(1+\frac{\sigma_d}{P_a}\right)^{-0.5959} \quad (5.4d)$$
$$M_R = 772.700P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0058}\left(1+\frac{\sigma_d}{P_a}\right)^{0.8110} \quad (5.4e)$$
$$M_R = 1192.767P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0027}\left(1+\frac{\sigma_d}{P_a}\right)^{0.3754} \quad (5.4f)$$
$$M_R = 1233.733P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0005}\left(1+\frac{\sigma_d}{P_a}\right)^{0.0742} \quad (5.4g)$$

Tables4.18 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the evaluation of Ooi et al B resilient modulus equations.Table 4.18showed that the magnitude of k_1 was always greater than zero. This indicated that the resilient modulus should always be greater than zero. The parameter $k_2$were always less than zero since the resilient modulus were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. From the evaluation, the resultant resilient modulus constitutive equation for fine-grained soils for A-2-6, A-2-7, A-4, A-5, A-6, A-7-5 and A-7-6 using Ooi et al B's resilient modulus model are presented in Equations 5.5a – 5.5g respectively.

$$M_R = 692.888 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0067} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{2.2433} \quad (5.5a)$$

$$M_R = 690.223 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0071} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{2.3621} \quad (5.5b)$$

$$M_R = 864.599 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0045} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{1.5136} \quad (5.5c)$$

$$M_R = 942.862 P_a \left(1 + \frac{\theta}{P_a}\right)^{0.0032} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{-1.0837} \quad (5.5d)$$

$$M_R = 781.503 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0044} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{1.4749} \quad (5.5e)$$

$$M_R = 1190.983 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0020} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{0.6826} \quad (5.5f)$$

$$M_R = 1227.605 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0004} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{0.1349} \quad (5.5g)$$

Tables4.19 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained from the evaluation of NCHRPresilient modulus equation.Table 4.19showed that the magnitude of k_1 was always greater than zero. This showed that the resilient modulus should always be greater than zero. The parameter $k_2$were always greater than zero since the resilient modulus were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. From the evaluation, the resultant resilient modulus constitutive equation for fine-grained soils for A-2-6, A-2-7, A-4, A-5, A-6, A-7-5 and A-7-6 using NCHRP's resilient modulus model are presented in Equations 5.6a – 5.6g respectively.

$$M_R = 935.584P_a\left(\frac{\theta}{P_a}\right)^{0.0015}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.3830} \quad (5.6a)$$

$$M_R = 885.922P_a\left(\frac{\theta}{P_a}\right)^{0.0022}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.9776} \quad (5.6b)$$

$$M_R = 1020.646P_a\left(\frac{\theta}{P_a}\right)^{0.0007}\left(1+\frac{\tau_{oct}}{P_a}\right)^{0.6679} \quad (5.6c)$$

$$M_R = 943.172P_a\left(\frac{\theta}{P_a}\right)^{-0.0012}\left(1+\frac{\tau_{oct}}{P_a}\right)^{-1.0745} \quad (5.6d)$$

$$M_R = 781.183P_a\left(\frac{\theta}{P_a}\right)^{0.0016}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.4624} \quad (5.6e)$$

$$M_R = 1241.608P_a\left(\frac{\theta}{P_a}\right)^{0.0008}\left(1+\frac{\tau_{oct}}{P_a}\right)^{0.7515} \quad (5.6f)$$

$$M_R = 1234.664P_a\left(\frac{\theta}{P_a}\right)^{0.0002}\left(1+\frac{\tau_{oct}}{P_a}\right)^{0.1399} \quad (5.6g)$$

Histograms showing the distribution of resilient modulus model parameters (k_1, k_2, and k_3) values obtained for fine-grained soils are presented in Figures 4.7 through 4.9.

Figure 4.7 showed the relationship between resilient modulus parameter $k_1$obtained for fine-grained soils and the resilient modulus equations evaluated in this study. Figure 4.7 showed that the magnitude of k_1 was always greater than zero since the resilient modulus should always be greater than zero. This is in line with what had been stated in literatures be several researchers (Titi *et al.*, 2006; Santha, 1994; Rao *et al.*, 2012; Yau and Von Quintus, 2002). This also agrees with the statement that the coefficient k_1 is proportional to Young's modulus, it should always be positive as resilient modulus can never be negative(George, 2004). In conclusion, all the resilient modulus equations evaluated in this study can be adopted for the fine-grained soils.

Figure 4.8 showed the relationship between resilient modulus parameter $k_2$obtained for fine-grainedsoils through the evaluation of the six resilient modulus equations evaluated in this work. From Figure 4.8, it is observed that the parameter $k_2$was generally less than zero for all the models evaluated except that proposed by NCHRP. As it was reported by several researchers(Santha, 1994; Rao *et al*., 2012; Yau and Von Quintus, 2002; George, 2004; Titi *et al*., 2006), the coefficient k_2 should be positive, because increase in volumetric stress produces stiffening or hardening of the material, thereby yielding higher modulus and that the resilient modulus decreases with the increase in the deviator stress.In conclusion, the resilient modulus equation proposed by NCHRP should be adopted for the fine-grained soils.

Figure 4.9 showed the relationship between resilient modulus parameters$k_3$obtained for fine-grainedsoils and the resilient modulus models evaluated in this study. Figure 4.9 showed that the parameter $k_3$was also greater than zero since the resilient modulus increases with increase in the confining stress.The coefficient k_3 should be negative because an increase in the shear stress softens the material, thereby yielding lower modulus(George, 2004). In conclusion, all the resilient modulus equations evaluated in this research can be adopted for the fine-grained soils.

Based on the evaluation of the resilient modulus equations for fine-grained soils, it was observed that the resilient modulus equation adopted by NCHRP was the best in determining resilient modulus for Nigerian fine-grained soils. This expression has gained considerable acceptance over time. The strength of this equation is its ability to handle two of the resilient modulus dependencies, stress-state and material type. Santha

(1994) compared a log-log bulk stress model to NCHRP resilient modulus equation in modelling Georgia granular subgrade soils and concluded that the model provided a better representation of laboratory measurements of resilient modulus(Rao *et al*., 2012).

For level 3 analysis, the resilient modulus parameters presented in Table 5.2 should be used as the default values for the corresponding soil types in NCHRP equation to determine the resilient modulus of Nigerian fine-grained soils.

Table 5.2: Default Constitutive Equation Coefficients(k_i)Values for Fine-GrainedSoils

Soil Type	Default Values of k_i		
	k_1	k_2	k_3
A-2-6	935.584	0.0015	1.3830
A-2-7	885.922	0.0022	1.9776
A-4	1020.646	0.0007	0.6679
A-5	943.172	-0.0012	-1.0745
A-6	781.183	0.0016	1.4624
A-7-5	1241.608	0.0008	0.7515
A-7-6	1234.664	0.0002	0.1399

5.5.2 Constitutive equation coefficients for coarse-grained soil

The statistical summary of the resilient modulus of coarse-grained soils presented in Table 4.13 was used in the evaluation. Tables4.20 through 4.26 presents the statistical summary of the resilient modulus parameters of coarse-grained soils.

Table 4.20 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Uzan resilient modulus equation.Table 4.20showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter $k_2$were close to zero since the resilient modulus of the investigated granular soils were constant with increase in the confining stress at constant deviator stress. The parameter

$k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. These are in consistent with the mean values of resilient modulus presented in Table 4.13. From the evaluation, the resultant resilient modulus constitutive equation for coarse-grained soils for A-1-b, A-2-4 and A-2-7 using Uzan's resilient modulus model are presented in Equations 5.7a – 5.7c respectively.

$$M_R = 1128.265 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.2695} \quad (5.7a)$$

$$M_R = 1567.500 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.3094} \quad (5.7b)$$

$$M_R = 1493.391 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.2140} \quad (5.7c)$$

Observing equations 5.7a – 5.7c, the terms for the confining stress is missing comparing to the Uzan resilient modulus constitutive equation. This is because the values of k_2 tends to zero. This showed that the confining stress has a less significant effect than deviator stress for granular soils.

Table 4.21 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Witczak and Uzan resilient modulus equation.Table 4.21showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter $k_2$were close to zero since the resilient modulus of the coarse-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. These are in consistent with the mean values of resilient modulus presented in Table 4.13. From the evaluation, the resultant resilient modulus constitutive equation for coarse-grained soil for A-1-b, A-2-4 and A-2-7 using Witczak and Uzan's resilient modulus model are presented in Equations 5.8a – 5.8c respectively.

$$M_R = 1381.746P_a\left(\frac{\tau_{oct}}{P_a}\right)^{0.2695} \quad (5.8a)$$
$$M_R = 2011.393P_a\left(\frac{\tau_{oct}}{P_a}\right)^{0.3094} \quad (5.8b)$$
$$M_R = 1769.317P_a\left(\frac{\tau_{oct}}{P_a}\right)^{0.2140} \quad (5.8c)$$

Observing Equations 5.8a – 5.8c, the terms for the confining stress is missing comparing to the Witczak and Uzan's resilient modulus constitutive equation. This is because the values of k_2 tends to zero. This showed that the confining stress has a less significant effect than deviator stress for granular soils.

Table 4.22 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Pezo resilient modulus equations.Table 4.22showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter $k_2$were close to zero since the resilient modulus of the coarse-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. These are in consistent with the mean values of resilient modulus presented in Table 4.13. From the evaluation, the resultant resilient modulus constitutive equation for the coarse-grained soils for A-1-b, A-2-4 and A-2-7 using Pezo's resilient modulus model are presented in Equations 5.9a – 5.9c respectively.

$$M_R = 1128.265P_a\left(\frac{\sigma_d}{P_a}\right)^{0.2695} \quad (5.9a)$$
$$M_R = 1567.500P_a\left(\frac{\sigma_d}{P_a}\right)^{0.3094} \quad (5.9b)$$

$$M_R = 1493.391 P_a \left(\frac{\sigma_d}{P_a}\right)^{0.2140} \quad (5.9c)$$

Observing Equations 5.9a – 5.9c, the terms for the confining stress is missing comparing to the Pezo's resilient modulus constitutive equation. This is because the values of k_2 tends to zero. This showed that the confining stress has a less significant effect than deviator stress for granular soils. Equations 5.9a – 5.9c are the same with that of Equations 5.8a – 5.8c from Witczak and Uzan's resilient modulus constitutive equation except in their k_1 values.

Table 4.23 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Ni et al resilient modulus equations.Table 4.23showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter $k_2$were always less than zero and negative since the resilient modulus of the coarse-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. These are in consistent with the mean values of resilient modulus presented in Table 4.13. From the evaluation, the resultant resilient modulus constitutive equation for the coarse-grained soils for A-1-b, A-2-4 and A-2-7 using Ni et al's resilient modulus model are presented in Equations 5.10a – 5.10c respectively.

$$M_R = 594.127 P_a \left(1 + \frac{\sigma_3}{P_a}\right)^{-0.0485} \left(1 + \frac{\sigma_d}{P_a}\right)^{0.9457} \quad (5.10a)$$
$$M_R = 787.860 P_a \left(1 + \frac{\sigma_3}{P_a}\right)^{-0.0557} \left(1 + \frac{\sigma_d}{P_a}\right)^{1.0856} \quad (5.10b)$$
$$M_R = 899.048 P_a \left(1 + \frac{\sigma_3}{P_a}\right)^{-0.0385} \left(1 + \frac{\sigma_d}{P_a}\right)^{0.7509} \quad (5.10c)$$

Table 4.24 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Ooi et al A resilient modulus equations.Table 4.24showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter $k_2$were always less than zero and negative since the resilient modulus of the coarse-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. These are in consistent with the mean values of resilient modulus presented in Table 4.13. From the evaluation, the resultant resilient modulus constitutive equation for the coarse-grained soils for A-1-b, A-2-4 and A-2-7 using Ooi et al A's resilient modulus model are presented in Equations 5.11a – 5.11c respectively.

$$M_R = 601.301 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0339} \left(1 + \frac{\sigma_d}{P_a}\right)^{0.9587} \quad (5.11a)$$

$$M_R = 797.293 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0389} \left(1 + \frac{\sigma_d}{P_a}\right)^{1.1006} \quad (5.11b)$$

$$M_R = 907.205 P_a \left(1 + \frac{\theta}{P_a}\right)^{-0.0269} \left(1 + \frac{\sigma_d}{P_a}\right)^{0.7613} \quad (5.11c)$$

Table 4.25 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of Ooi et al B resilient modulus equations.Table 4.25showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter $k_2$were always less than zero and negative since the resilient modulus of the coarse-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. These are in consistent with the mean values of resilient modulus presented in Table 4.13. From the evaluation, the resultant resilient modulus constitutive equation for the coarse-grained soils for A-1-b, A-2-4 and A-2-7 using Ooi et al B's resilient modulus model are presented in Equations 5.12a – 5.12c respectively.

$$M_R = 620.772P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0339}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.6028} \quad (5.12a)$$
$$M_R = 823.015P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0389}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.8400} \quad (5.12b)$$
$$M_R = 929.313P_a\left(1+\frac{\theta}{P_a}\right)^{-0.0269}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.2727} \quad (5.12c)$$

Table 4.26 presents the statistical summary of the resilient modulus parameters of coarse-grained soils obtained from the evaluation of NCHRP resilient modulus equations.Table 4.26showed that the magnitude of k_1 was always greater than zero. This implies that the resilient modulus should always be greater than zero. The parameter $k_2$were always less than zero and negative since the resilient modulus of the coarse-grained soils were constant with increase in the confining stress at constant deviator stress. The parameter $k_3$was always greater than zero. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress. These are in consistent with the mean values of resilient modulus presented in Table 4.13. From the evaluation, the resultant resilient modulus constitutive equation for the coarse-grained soils for A-1-b, A-2-4 and A-2-7 using NCHRP's resilient modulus model are presented in Equations 5.13a – 5.13c respectively.

$$M_R = 605.439P_a\left(\frac{\theta}{P_a}\right)^{-0.0192}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.5964} \quad (5.13a)$$
$$M_R = 802.746P_a\left(\frac{\theta}{P_a}\right)^{-0.0220}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.8326} \quad (5.13b)$$
$$M_R = 911.908P_a\left(\frac{\theta}{P_a}\right)^{-0.0152}\left(1+\frac{\tau_{oct}}{P_a}\right)^{1.2676} \quad (5.13c)$$

Histograms showing the distribution of k_1, k_2, and k_3 values by soil class for the coarse-grained soilswere as shown in Figures 4.17 through 4.19.Figure 4.17 showed the relationship between resilient modulus parameter $k_1$obtained for coarse-grained soils and the resilient modulus equations evaluated in this study. Figure 4.17 showed that the magnitude of k_1 was always greater than zero since the resilient modulus should always be greater than zero.This is in line with what had been stated in literatures be several researchers (Titi*et al*., 2006; Santha, 1994; Rao *et al*., 2012; Yau and Von Quintus, 2002).This also agrees with the statement that the coefficient k_1 is proportional to Young's modulus, it should always be positive as resilient modulus can never be negative(George, 2004). In conclusion, all the resilient modulus equations evaluated in this research can be adopted for coarse-grained soils.

Figure 4.18 showed the relationship between resilient modulus parameter $k_2$obtained for coarse-grained soils through the evaluation of resilient modulus equations. From Figure 4.18, it was observed that the parameter $k_2$were always less than zero and negative. This implies that the resilient modulus decreases with the increase in the deviator stress.The coefficient k_2 should be negative because an increase in the shear stress softens the material, thereby yielding lower modulus(George, 2004).

Figure 4.19 showed the relationship between resilient modulus parameters$k_3$obtained for coarse-grained soils and the resilient modulus models evaluated in this study. Figure 4.19 showed that the parameter $k_3$was also greater than zero since the resilient modulus increases with increase in the confining stress.

Based on the evaluation of the resilient modulus equations for the coarse-grained soils, it was observed that the resilient modulus equation adopted by NCHRP was the best in determining resilient modulus for coarse-grained soils. Therefore, the model was the one chosen for coarse-grained soils. This expression has gained considerable acceptance over time. The strength of this equation is its ability to handle two of the resilient modulus dependencies, stress-state and material type. Santha (1994) compared a log-log bulk stress model to NCHRP resilient modulus equation in modelling Georgia granular subgrade soils and concluded that the model provided a better representation of laboratory measurements of resilient modulus(Rao *et al*., 2012). For level 3 analysis, the resilient modulus parameters presented in Table 5.3 should be used as the default values for the corresponding soil types in NCHRP equation to determine the resilient modulus of coarse-grained soils.

Table 5.3: Default Constitutive Equation Coefficients(k_i)Values for Coarse-Grained Soils

Soil Type	Default Values of k_i		
	k_1	k_2	k_3
A-1-b	605.439	-0.0192	1.5964
A-2-4	802.746	-0.0220	1.8326
A-2-7	911.908	-0.0152	1.2676

5.6 Correlation Equations forNigerianFine-Grained Soils

Tables 4.28-4.30 present summaries of the regression analysis results in which models to estimate k_1, k_2, and k_3 from basic soil properties were obtained. Figures4.20 – 4.22 depicts comparisons between k_i values obtained from analysis of the results of the repeated load triaxial test and k_i values estimated from basic soil properties using the proposed correlations. Examination of Tables 4.28 – 4.30 shows that these models are consistent with the natural behaviour of the soils. The magnitudes of R^2 for $k_1$correlations range between 0.79 and 0.99, which is considered acceptable. Lower R^2values were obtained for k_2 and $k_3$as shown in Tables 4.29 and 4.30. Figures4.20 – 4.22 alsodemonstrate the good estimation capability of these models. It should be emphasized that these models were obtained based on statistical analysis on test data that are limited to fine-grained soils.

As part of model development, various combinations of model forms (i.e., mathematical relationships) and transformation of dependent/independent variables were evaluated to determine which combination resulted in the best prediction model. Both linear and nonlinear statistical techniques were utilized for model development and calibration of the mathematical equations. In general, using the dependent and independent variables (transformed or otherwise) and mathematical equations representing the model forms identified above, the iterative process in selecting a tentative model was performed as follows: Selection of best combination of independent variables, Selection of submodels, Maximize R^2 and Minimize error.

The combinations of model forms and transformation of dependent/independent variables from Tables 4.28 through 4.30 were presented in Equation 5.14 through Equation5.16. Based on the statistical analysis of the fine-grained soils, the resilient modulus model parameters (k_i) can be estimated from basic soil properties using the following equations:

$$\begin{aligned}k_1 = {} & 5767.78 - 15.63P_{40} + 11.97(\%Clay) + 10.24(\%Silt) + 19.21(PL) \\ & - 67.69(OMC) - 1.79(MDD) + 300.97\left(\frac{NMC}{OMC}\right) - 878.39\left(\frac{\gamma_d}{MDD}\right) \\ & + 8.53\left(\frac{P_{200}}{NMC}\right) \quad (5.14)\end{aligned}$$

$$\begin{aligned}k_2 = {} & 0.1522 - 7.3\times10^{-5}P_{40} + 8.8\times10^{-5}P_{200} + 3.4\times10^{-5}(\%Silt) + 4.4 \\ & \times10^{-5}(LL) - 0.0002(PL) - 0.0008(NMC) + 0.0002(OMC) + 7.9 \\ & \times10^{-5}(\gamma_d) - 8.9\times10^{-5}(MDD) + 0.0151\left(\frac{NMC}{OMC}\right) \\ & - 0.1354\left(\frac{\gamma_d}{MDD}\right) \quad (5.15)\end{aligned}$$

$$\begin{aligned}k_3 = {} & 15.8009 - 0.0048(P_{200}) - 0.0097(P_4) + 0.0104(P_{40}) + 0.0025(\%Silt) \\ & - 0.0332(PL) + 0.0111(LL) - 0.0817(NMC) + 0.0063(\gamma_d) \\ & - 0.0080(MDD) + 1.4826\left(\frac{NMC}{OMC}\right) - 10.7471\left(\frac{\gamma_d}{MDD}\right) \\ & - 0.0024\left(\frac{P_{200}}{NMC}\right) \quad (5.16)\end{aligned}$$

where

P_4 is the percent passing sieve #4

P_{40} is the percent passing sieve #40

P_{200} is the percent passing sieve #200

$LL$is the liquid limit

$PL$is the plastic limit

*%Silt*is the amount of silt

*%Clay*is the amount of clay

NMC is the natural moisture content

γ_d is the dry density

OMC is the optimum moisture content

MDD is the maximum dry density

This model was fitted to all soil samples with resilient modulus available for a range of confining and deviator stresses. Model fitting was done individually for each subgradematerial sample in the PEU database. For each of the soil samples, calculated k_1, k_2, and k_3 and the constitutive equation were used to predict resilient modulus at the lab test confining and deviator stresses for comparison. The results of the comparison showed a good fit of predicted and measured resilient modulus with a high R^2 value and low standard error of estimate (SEE).

Model prediction accuracy and reasonableness were evaluated by reviewing the plot of predicted and measured resilient modulus for all individual resilient modulus test values used in model development.

CHAPTER SIX

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

Subgrade materials from different locations in Nigeria were characterized for use in the Mechanistic – Empirical Pavement Design. The basic soil index properties of the subgrade soil materials from Master Test Section (MTS) 1 that identified the material response to external stimuli of traffic loading and environmental conditions were obtained in the laboratory. Three samples each from the MTSs making a total eighteen (18) samples were obtained and subjected to laboratory test to determine their basic physical properties. Testing include particle size distribution, Atterberg limits, specific gravity, compaction characteristics, Unconfined Compression test and the California Bearing Ratio tests. The samples were classified according to American Association of State Highway and Transportation Officials (AASHTO) and the Unified Soil Classification System (USCS). The AASHTO soil classification shows that the subgrade soil samples obtained from the MTS 1 were eitherclayey soil (A-6 and A-7-6) or silty soils (A-4, A-5 and A-2-4). The USCS soil classification indicated that most of the samples were lean clay soil with gravel (CL) except few that were either clayey gravel (GC) or clayey sand (SC).The California bearing ratio (CBR) values were generally less than 3%. Resilient modulus constitutive equation for estimating the resilient modulus of Nigerian subgrade soils was adopted through evaluation of existing resilient modulus constitutive equations using the repeated load triaxial test result conducted on Nigerian subgrade soils. Comprehensive statistical analysis using multiple linear regression used to develop correlations between basic soil properties and the resilient modulus model parameters.The correlation showed that the resilient modulus model parameters (k_i) can be estimated from basic soil properties.

6.2 Conclusions

Based on the results of this study, the following conclusions was reached:

1. The AASHTO soil classification generally showed that the subgrade soil samples were "Fair to Poor" in subgrade properties.
2. The USCS soil classification indicated that the samples were mostly clay soil.
3. The CBR values were generally less than 3%. This implies that the strength of the subgrades were poor, therefore, capping is required.
4. Based on the compaction characteristics, it can be concluded that most of the samples had fat clay (CH) in its content.
5. The Equation of resilient modulus adopted by NCHRP should be adopted for Nigerian subgrade soils for level 3 resilient modulus input for the NEMPADS.
6. The equations that correlate resilient modulus model parameters (k_1, k_2, and k_3) to basic soil properties for fine-grained soil can be utilized to estimate level 2 resilient modulus input for the NEMPADS.
7. The equations developed in this research were based on statistical analysis of laboratory test results that were limited to the soil physical conditions specified.

6.3 Recommendations

Based on the results of this study, the followings was recommended:

1. It was recommended that the resilient modulus of the subgrade soil could be determined from basic soil index properties through correlation.
2. The use of the resilient modulus parameters(default values) in the absence of any basic soil testing when designing low volume roads as indicated by AASHTO.
3. Further research should be carried out to develop correlation equations subgrade soils from the five other Master Test Sections in Nigeria.

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APPENDIX A

RESULTS OF BASIC SOIL INDEX PROPERTIES OF THE SOIL SAMPLES

KADUNA POLYTECHNIC
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SOIL AND GEOLOGY LABORATORY

ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: ZARIA-STATE ROAD (A236)
SAMPLE NO: TP1 LEFT CH: 35+000
Table A.1: Atterberg Limits of Soil SamplesfromMTS 1-1 TP1

No. of blows	11	16	20	38
Container No.	23	60T	98	52
Wt. of wet soil and container………(g)	16.70	16.80	17.60	17.40
Wt. of dry soil and container……(g)	13.20	13.30	14.10	13.70
Wt. of container……(g)	4.10	4.10	3.90	3.90
Wt. of moisture (Wm)…(g)	3.50	3.50	3.50	3.70
Wt. of dry soil (Wd)……(g)	9.10	9.20	10.20	9.80
Moisture contents (100Wm/Wd)…(%)	38.46	38.04	34.31	37.76

Container No.	112	19
Wt. of wet soil and container………(g)	9.20	9.00
Wt. of dry soil and container……(g)	8.30	8.20
Wt. of container……(g)	4.00	4.00
Wt. of moisture (Wm)…(g)	0.90	0.80
Wt. of dry soil (Wd)……(g)	4.30	4.20
Moisture contents (100Wm/Wd)…(%)	20.93	19.05
Average moisture contents (m)…(%)	19.99	

LL = 37.00%, PL = 19.99%, PI = 17.01%, SL= 10.00%

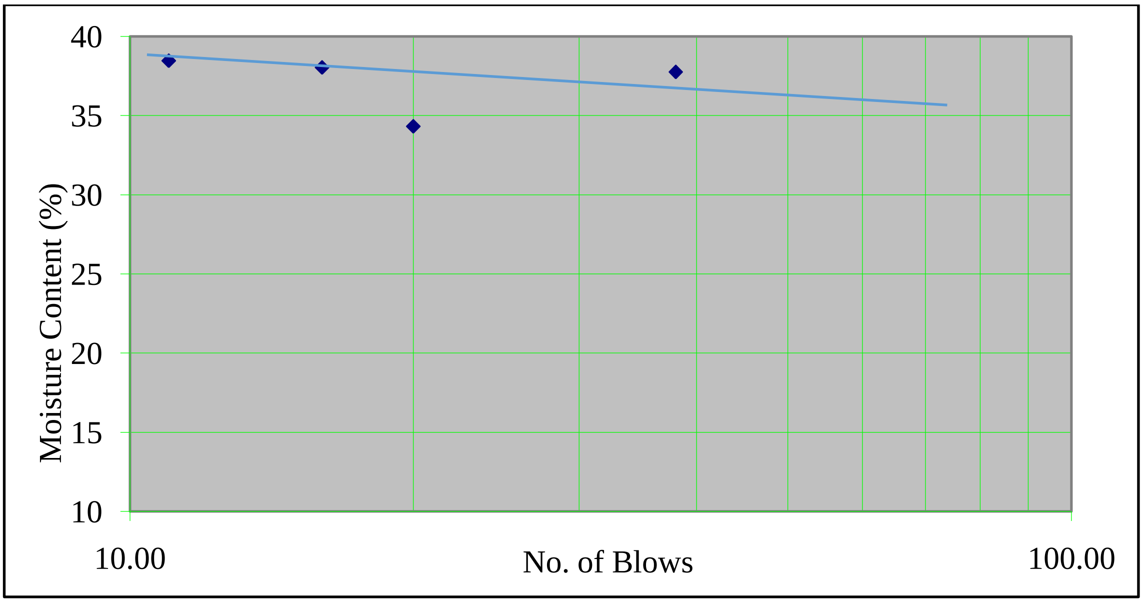

Figure A.1: Flow Curve of Soil SamplesfromMTS 1-1 TP1

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-1 ZARIA-STATE ROAD (A236)
SAMPLE NO: TP2 LEFT CH: 35+000
Table A.2: Atterberg Limits of Soil SamplesfromMTS 1-1 TP2

No. of blows	10	14	22	38
Container No.	89	26	112	60T
Wt. of wet soil and container………(g)	15.80	17.00	16.10	16.30
Wt. of dry soil and container……(g)	12.30	13.10	12.90	12.90
Wt. of container……(g)	3.90	4.00	4.10	4.10
Wt. of moisture (Wm)…(g)	3.50	3.90	3.20	3.40
Wt. of dry soil (Wd)……(g)	8.40	9.10	8.80	8.80
Moisture contents (100Wm/Wd)…(%)	41.67	42.86	36.36	38.64

Container No.	23	98
Wt. of wet soil and container………(g)	8.80	9.10
Wt. of dry soil and container……(g)	7.80	8.20
Wt. of container……(g)	4.10	4.10
Wt. of moisture (Wm)…(g)	1.00	0.90
Wt. of dry soil (Wd)……(g)	3.70	4.10
Moisture contents (100Wm/Wd)…(%)	27.03	21.95
Average moisture contents (m)…(%)	24.49	

LL = 39.00%, PL = 24.49%, PI = 14.51%, SL = 8.57%

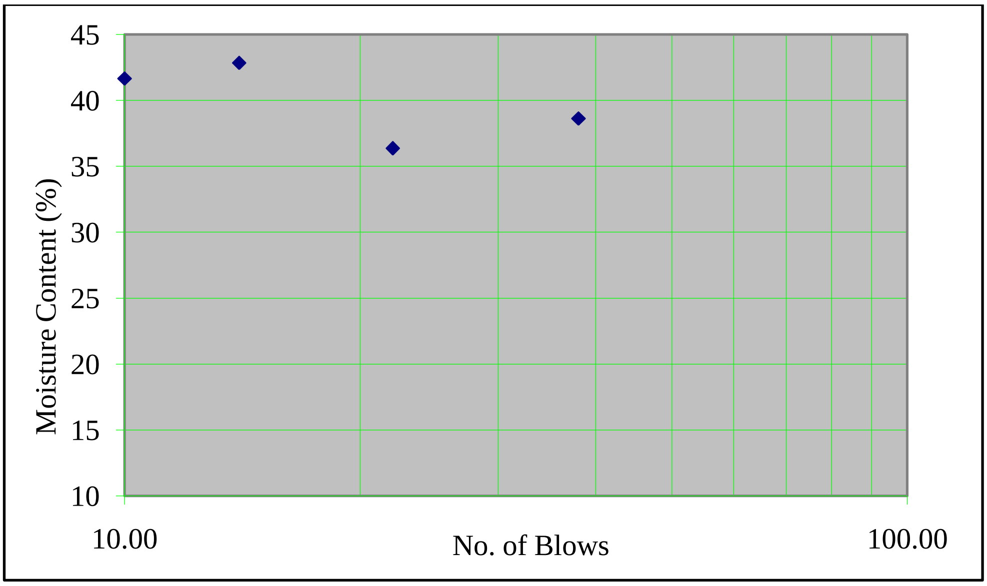

Figure A.2: Flow Curve of Soil SamplesfromMTS 1-1 TP2

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-1 ZARIA-STATE ROAD (A236)
SAMPLE NO: TP3 LEFT CH: 35+000
Table A.3: Atterberg Limits of Soil SamplesfromMTS 1-1 TP3

No. of blows	10	15	24	28
Container No.	26	29	55	56G
Wt. of wet soil and container………(g)	17.00	14.80	17.30	17.50
Wt. of dry soil and container……(g)	13.40	12.00	14.00	13.90
Wt. of container……(g)	4.20	3.90	4.00	3.50
Wt. of moisture (Wm)…(g)	3.60	2.80	3.30	3.60
Wt. of dry soil (Wd)……(g)	9.20	8.10	10.00	10.40
Moisture contents (100Wm/Wd)…(%)	39.13	34.57	33.00	34.62

Container No.	102	89
Wt. of wet soil and container………(g)	9.20	9.00
Wt. of dry soil and container……(g)	8.30	8.20
Wt. of container……(g)	4.10	4.00
Wt. of moisture (Wm)…(g)	0.90	0.80
Wt. of dry soil (Wd)……(g)	4.20	4.20
Moisture contents (100Wm/Wd)…(%)	21.43	19.05
Average moisture contents (m)…(%)	20.24	

LL = 32.50%, PL = 20.24%, PI = 12.26%, SL= 11.43%

Figure A.3: Flow Curve of Soil SamplesfromMTS 1-1 TP3

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-2 KATABU-PAMBEGUA ROAD (A11)
SAMPLE NO: TP1 RIGHT CH: 4+000
Table A.4: Atterberg Limits of Soil SamplesfromMTS 1-2 TP1

No. of blows	12	20	42	
Container No.	89	79	71H	
Wt. of wet soil and container………(g)	16.50	16.00	17.20	
Wt. of dry soil and container……(g)	13.10	12.90	13.80	
Wt. of container……(g)	3.90	4.00	4.10	
Wt. of moisture (Wm)…(g)	3.40	3.10	3.40	
Wt. of dry soil (Wd)……(g)	9.20	8.90	9.70	
Moisture contents (100Wm/Wd)…(%)	36.96	34.83	35.05	

Container No.	48	74
Wt. of wet soil and container………(g)	9.00	9.50
Wt. of dry soil and container……(g)	8.00	8.50
Wt. of container……(g)	4.00	4.00
Wt. of moisture (Wm)…(g)	1.00	1.00
Wt. of dry soil (Wd)……(g)	4.00	4.50
Moisture contents (100Wm/Wd)…(%)	25.00	22.22
Average moisture contents (m)…(%)	23.61	

LL = 36.00% , PL = 23.61% , PI = 12.39% , SL= 10.00%

Figure A.4: Flow Curve of Soil SamplesfromMTS 1-2 TP1

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-2 KATABU-PAMBEGUA ROAD (A11)
SAMPLE NO: TP2 LEFT CH: 4+000
Table A.5: Atterberg Limits of Soil Samples fromMTS 1-2 TP2

No. of blows	11	15	20	28
Container No.	TP3A	120	16	PL9
Wt. of wet soil and container………(g)	17.50	17.20	17.00	16.30
Wt. of dry soil and container……(g)	14.30	14.10	14.00	13.40
Wt. of container……(g)	3.90	4.00	4.00	3.30
Wt. of moisture (Wm)…(g)	3.20	3.10	3.00	2.90
Wt. of dry soil (Wd)……(g)	10.40	10.10	10.00	10.10
Moisture contents (100Wm/Wd)…(%)	30.77	30.69	30.00	28.71

Container No.	91	67Z
Wt. of wet soil and container………(g)	8.90	9.10
Wt. of dry soil and container……(g)	8.10	8.20
Wt. of container……(g)	3.90	4.10
Wt. of moisture (Wm)…(g)	0.80	0.90
Wt. of dry soil (Wd)……(g)	4.20	4.10
Moisture contents (100Wm/Wd)…(%)	19.05	21.95
Average moisture contents (m)…(%)	20.50	

LL = 29.00%, PL = 20.50%, PI = 8.50%, SL = 8.57%

Figure A.5: Flow Curve of Soil SamplesfromMTS 1-2 TP2

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-2 KATABU-PAMBEGUA ROAD (A11)
SAMPLE NO: TP3 LEFT CH: 4+000
Table A.6: Atterberg Limits of Soil SamplesfromMTS 1-2 TP3

No. of blows	12	20	28	
Container No.	80	KA	54V	
Wt. of wet soil and container.........(g)	14.80	15.80	17.10	
Wt. of dry soil and container......(g)	11.80	12.60	13.60	
Wt. of container......(g)	3.90	3.90	4.00	
Wt. of moisture (Wm)...(g)	3.00	3.20	3.50	
Wt. of dry soil (Wd)......(g)	7.90	8.70	9.60	
Moisture contents (100Wm/Wd)...(%)	37.97	36.78	36.46	

Container No.	77M	82
Wt. of wet soil and container.........(g)	9.50	8.90
Wt. of dry soil and container......(g)	8.60	8.20
Wt. of container......(g)	4.00	4.00
Wt. of moisture (Wm)...(g)	0.90	0.70
Wt. of dry soil (Wd)......(g)	4.60	4.20
Moisture contents (100Wm/Wd)...(%)	19.57	16.67
Average moisture contents (m)...(%)	18.12	

LL = 37.00%, PL = 18.12%, PI = 18.88%, SL = 9.29%

Figure A.6: Flow Curve of Soil SamplesfromMTS 1-2 TP3

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-3 KADUNA-ZARIA ROAD (A2)
SAMPLE NO: TP1CENTRE CH: 40+000
Table A.7: Atterberg Limits of Soil SamplesfromMTS 1-3 TP1

No. of blows	11	19	23	27
Container No.	91	75	48	120
Wt. of wet soil and container………(g)	14.80	15.90	15.00	15.20
Wt. of dry soil and container……(g)	11.60	12.50	11.90	12.10
Wt. of container……(g)	4.00	3.90	4.00	4.20
Wt. of moisture (Wm)…(g)	3.20	3.40	3.10	3.10
Wt. of dry soil (Wd)……(g)	7.60	8.60	7.90	7.90
Moisture contents (100Wm/Wd)…(%)	42.11	39.53	39.24	39.24

Container No.	22	79
Wt. of wet soil and container………(g)	9.00	9.00
Wt. of dry soil and container……(g)	8.00	8.00
Wt. of container……(g)	3.90	4.00
Wt. of moisture (Wm)…(g)	1.00	1.00
Wt. of dry soil (Wd)……(g)	4.10	4.00
Moisture contents (100Wm/Wd)…(%)	24.39	25.00
Average moisture contents (m)…(%)	24.70	

LL = 38.50%, PL = 24.70%, PI =13.80%, SL = 10.71%

Figure A.7: Flow Curve of Soil SamplesfromMTS 1-3 TP1

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-3 KADUNA-ZARIA ROAD (A2)
SAMPLE NO: TP2LEFT CH: 40+000
Table A.8: Atterberg Limits of Soil SamplesfromMTS 1-3 TP2

No. of blows	12	18	27	44
Container No.	5	75	11	54V
Wt. of wet soil and container………(g)	17.50	15.50	16.50	16.50
Wt. of dry soil and container……(g)	14.20	12.80	13.50	13.70
Wt. of container……(g)	3.90	4.10	3.90	4.00
Wt. of moisture (Wm)…(g)	3.30	2.70	3.00	2.80
Wt. of dry soil (Wd)……(g)	10.30	8.70	9.60	9.70
Moisture contents (100Wm/Wd)…(%)	32.04	31.03	31.25	28.87

Container No.	20	22
Wt. of wet soil and container………(g)	9.60	9.90
Wt. of dry soil and container……(g)	8.70	9.00
Wt. of container……(g)	3.90	3.90
Wt. of moisture (Wm)…(g)	0.90	0.90
Wt. of dry soil (Wd)……(g)	4.80	5.10
Moisture contents (100Wm/Wd)…(%)	18.75	17.65
Average moisture contents (m)…(%)	18.20	

LL = 31.00%, PL = 18.20%, PI = 12.8%, SL= 7.14%

Figure A.8: Flow Curve of Soil SamplesfromMTS 1-3 TP2

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-3 KADUNA-ZARIA ROAD (A2)
SAMPLE NO: TP3 RIGHT CH: 40+000
Table A.9: Atterberg Limits of Soil SamplesfromMTS 1-3 TP3

No. of blows	10	18	34	
Container No.	D	49	90	
Wt. of wet soil and container………(g)	16.30	16.60	17.60	
Wt. of dry soil and container……(g)	12.90	13.20	14.10	
Wt. of container……(g)	4.10	4.10	4.00	
Wt. of moisture (Wm)…(g)	3.40	3.40	3.50	
Wt. of dry soil (Wd)……(g)	8.80	9.10	10.10	
Moisture contents (100Wm/Wd)…(%)	38.64	37.36	34.65	

Container No.	54V	5
Wt. of wet soil and container………(g)	9.00	8.80
Wt. of dry soil and container……(g)	8.00	7.80
Wt. of container……(g)	4.10	4.00
Wt. of moisture (Wm)…(g)	1.00	1.00
Wt. of dry soil (Wd)……(g)	3.90	3.80
Moisture contents (100Wm/Wd)…(%)	25.64	26.32
Average moisture contents (m)…(%)	25.98	

LL = 35.98%, PL = 25.98%, PI = 10.00%, SL= 9.29%

Figure A.9: Flow Curve of Soil SamplesfromMTS 1-3 TP3

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-4 KADUNA-KACHIA ROAD (A235)
SAMPLE NO: TP1LEFT CH: 11+000
Table A.10: Atterberg Limits of Soil SamplesfromMTS 1-4 TP1

No. of blows	12	16	20	27
Container No.	67Z	98	KJ	47A
Wt. of wet soil and container………(g)	17.00	16.00	17.20	14.80
Wt. of dry soil and container……(g)	13.60	12.60	13.90	12.10
Wt. of container……(g)	4.10	4.00	3.90	4.10
Wt. of moisture (Wm)…(g)	3.40	3.40	3.30	2.70
Wt. of dry soil (Wd)……(g)	9.50	8.60	10.00	8.00
Moisture contents (100Wm/Wd)…(%)	35.79	39.53	33.00	33.75

Container No.	TP3A	PLA
Wt. of wet soil and container………(g)	9.00	8.50
Wt. of dry soil and container……(g)	8.10	7.60
Wt. of container……(g)	4.00	3.40
Wt. of moisture (Wm)…(g)	0.90	0.90
Wt. of dry soil (Wd)……(g)	4.10	4.20
Moisture contents (100Wm/Wd)…(%)	21.95	21.43
Average moisture contents (m)…(%)	21.69	

LL = 31.70%, PL = 21.69%, PI =10.01%, SL = 8.57%

Figure A.10: Flow Curve of Soil SamplesfromMTS 1-4 TP1

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-4 KADUNA-KACHIA ROAD (A235)
SAMPLE NO: TP2 RIGHT CH: 11+000
Table A.11: Atterberg Limits of Soil SamplesfromMTS 1-4 TP2

No. of blows	15	19	30
Container No.	74	13	67Z
Wt. of wet soil and container………(g)	16.50	16.00	16.40
Wt. of dry soil and container……(g)	12.60	12.30	12.50
Wt. of container……(g)	4.00	4.00	4.10
Wt. of moisture (Wm)…(g)	3.90	3.70	3.90
Wt. of dry soil (Wd)……(g)	8.60	8.30	8.40
Moisture contents (100Wm/Wd)…(%)	45.35	44.58	46.43

Container No.	89	48
Wt. of wet soil and container………(g)	9.00	8.80
Wt. of dry soil and container……(g)	7.90	7.70
Wt. of container……(g)	4.10	4.00
Wt. of moisture (Wm)…(g)	1.10	1.10
Wt. of dry soil (Wd)……(g)	3.80	3.70
Moisture contents (100Wm/Wd)…(%)	28.95	29.73
Average moisture contents (m)…(%)	29.34	

LL = 44.50%, PL = 29.34%, PI = 15.16%, SL = 10.71%

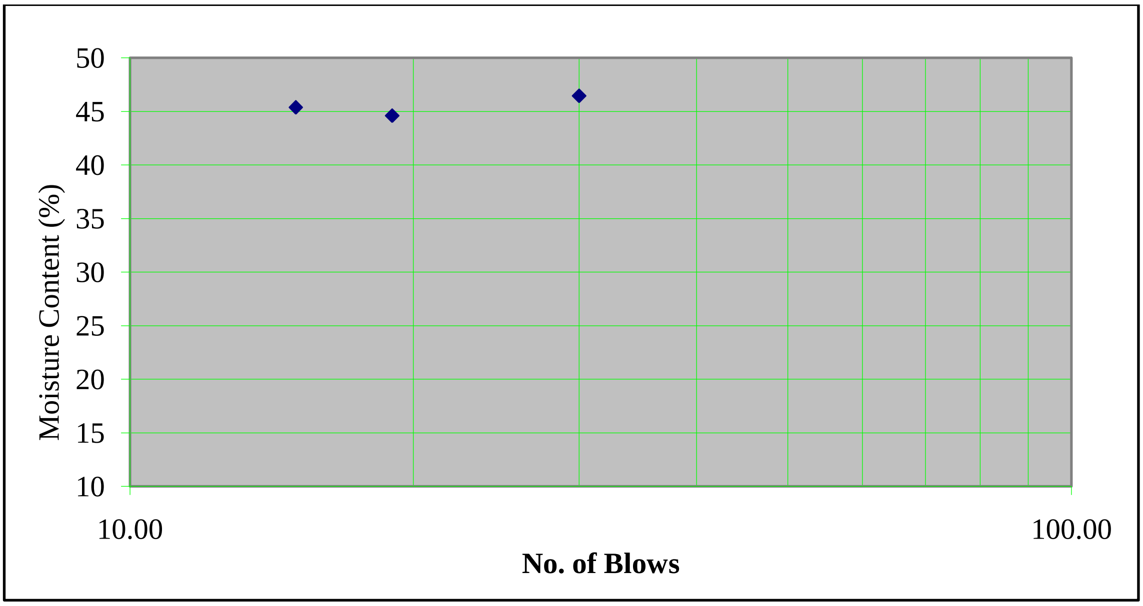

Figure A.11: Flow Curve of Soil SamplesfromMTS 1-4 TP2

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-4 KADUNA-KACHIA ROAD (A235)
SAMPLE NO: TP3 RIGHT CH: 11+000
Table A.12: Atterberg Limits of Soil SamplesfromMTS 1-4 TP3

No. of blows	10	13	22	26
Container No.	31	54	77M	71H
Wt. of wet soil and container………(g)	16.40	17.50	16.20	16.90
Wt. of dry soil and container……(g)	12.60	13.40	12.60	13.20
Wt. of container……(g)	4.10	4.00	4.00	4.20
Wt. of moisture (Wm)…(g)	3.80	4.10	3.60	3.70
Wt. of dry soil (Wd)……(g)	8.50	9.40	8.60	9.00
Moisture contents (100Wm/Wd)…(%)	44.71	43.62	41.86	41.11

Container No.	16	30
Wt. of wet soil and container………(g)	9.00	9.10
Wt. of dry soil and container……(g)	8.00	8.10
Wt. of container……(g)	4.10	3.90
Wt. of moisture (Wm)…(g)	1.00	1.00
Wt. of dry soil (Wd)……(g)	3.90	4.20
Moisture contents (100Wm/Wd)…(%)	25.64	23.81
Average moisture contents (m)…(%)	24.73	

LL = 41.50%, PL = 24.73%, PI = 16.77%, SL = 9.29%

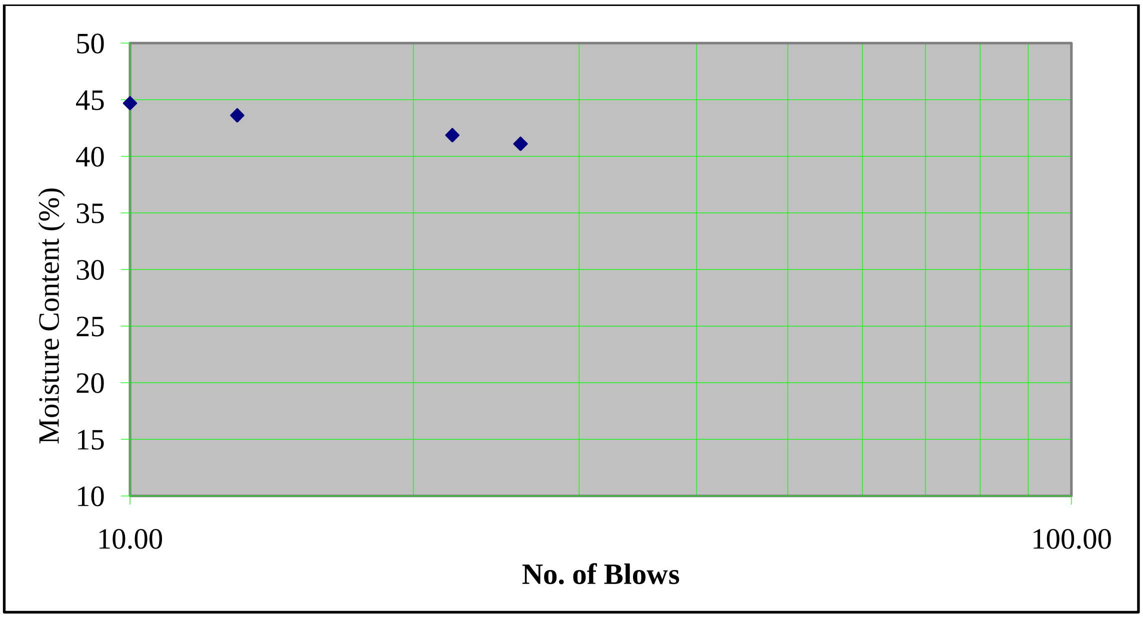

Figure A.12: Flow Curve of Soil SamplesfromMTS 1-4 TP3

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-5 KADUNA-BIRNIN GWARI ROAD (A125)
SAMPLE NO: TP1 RIGHT CH: 11+000

Table A.13: Atterberg Limits of Soil SamplesfromMTS 1-5 TP1

No. of blows	11	21	34	
Container No.	91	90	120	
Wt. of wet soil and container………(g)	16.30	16.40	16.30	
Wt. of dry soil and container……(g)	12.40	12.60	12.80	
Wt. of container……(g)	3.90	4.00	4.10	
Wt. of moisture (Wm)…(g)	3.90	3.80	3.50	
Wt. of dry soil (Wd)……(g)	8.50	8.60	8.70	
Moisture contents (100Wm/Wd)…(%)	45.88	44.19	40.23	

Container No.	47A	79
Wt. of wet soil and container………(g)	9.00	9.00
Wt. of dry soil and container……(g)	8.00	7.80
Wt. of container……(g)	3.90	4.00
Wt. of moisture (Wm)…(g)	1.00	1.20
Wt. of dry soil (Wd)……(g)	4.10	3.80
Moisture contents (100Wm/Wd)…(%)	24.39	31.58
Average moisture contents (m)…(%)	27.98	

LL = 42.00%, PL = 27.98%, PI = 14.02%,SL=7.86%

Figure A.13: Flow Curve of Soil SamplesfromMTS 1-5 TP1

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-5 KADUNA-BIRNIN GWARI ROAD (A125)
SAMPLE NO: TP2LEFT CH: 11+000
Table A.14: Atterberg Limits of Soil SamplesfromMTS 1-5 TP2

No. of blows	12	18	25	32
Container No.	30	71H	74	89
Wt. of wet soil and container………(g)	15.20	16.20	16.30	15.40
Wt. of dry soil and container……(g)	11.70	12.60	12.70	12.20
Wt. of container……(g)	4.20	4.20	3.90	4.00
Wt. of moisture (Wm)…(g)	3.50	3.60	3.60	3.20
Wt. of dry soil (Wd)……(g)	7.50	8.40	8.80	8.20
Moisture contents (100Wm/Wd)…(%)	46.67	42.86	40.91	39.02

Container No.	16	13
Wt. of wet soil and container………(g)	8.50	9.00
Wt. of dry soil and container……(g)	7.60	7.90
Wt. of container……(g)	4.10	4.10
Wt. of moisture (Wm)…(g)	0.90	1.10
Wt. of dry soil (Wd)……(g)	3.50	3.80
Moisture contents (100Wm/Wd)…(%)	25.71	28.95
Average moisture contents (m)…(%)	27.33	

LL = 40.91%, PL = 27.33%, PI =13.58%, SL = 8.57%

Figure A.14: Flow Curve of Soil SamplesfromMTS 1-5 TP2

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-5 KADUNA-BIRNIN GWARI ROAD (A125)
SAMPLE NO: TP3 RIGHT CH: 11+000

Table A.15: Atterberg Limits of Soil SamplesfromMTS 1-5 TP3

No. of blows	14	19	29
Container No.	78	82	KA
Wt. of wet soil and container………(g)	15.90	15.30	16.20
Wt. of dry soil and container……(g)	12.30	12.00	12.60
Wt. of container……(g)	4.20	4.00	3.90
Wt. of moisture (Wm)…(g)	3.60	3.30	3.60
Wt. of dry soil (Wd)……(g)	8.10	8.00	8.70
Moisture contents (100Wm/Wd)…(%)	44.44	41.25	41.38

Container No.	55	34
Wt. of wet soil and container………(g)	9.10	9.00
Wt. of dry soil and container……(g)	7.90	7.80
Wt. of container……(g)	3.90	4.10
Wt. of moisture (Wm)…(g)	1.20	1.20
Wt. of dry soil (Wd)……(g)	4.00	3.70
Moisture contents (100Wm/Wd)…(%)	30.00	32.43
Average moisture contents (m)…(%)	31.22	

LL = 41.22%, PL = 31.22%, PI = 10.00%, SL= 7.14%

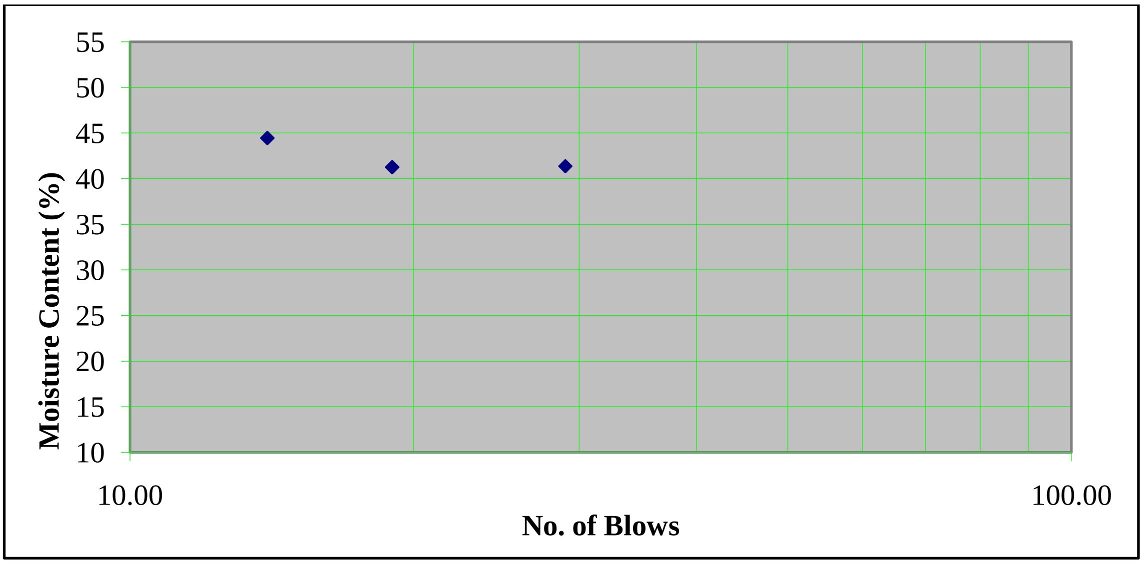

Figure A.15: Flow Curve of Soil SamplesfromMTS 1-5 TP3

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-6 KATABU-ZARIA ROAD (A2)
SAMPLE NO: TP1 RIGHT CH: 16+000
Table A.16: Atterberg Limits of Soil SamplesfromMTS 1-6 TP1

No. of blows	11	15	19	30
Container No.	49	116	D	A3
Wt. of wet soil and container………(g)	17.20	16.00	17.00	17.50
Wt. of dry soil and container……(g)	13.40	12.80	13.80	14.10
Wt. of container……(g)	4.10	4.10	4.00	3.60
Wt. of moisture (Wm)…(g)	3.80	3.20	3.20	3.40
Wt. of dry soil (Wd)……(g)	9.30	8.70	9.80	10.50
Moisture contents (100Wm/Wd)…(%)	40.86	36.78	32.65	32.38

Container No.	99	34
Wt. of wet soil and container………(g)	9.20	9.50
Wt. of dry soil and container……(g)	8.40	8.50
Wt. of container……(g)	4.00	4.00
Wt. of moisture (Wm)…(g)	0.80	1.00
Wt. of dry soil (Wd)……(g)	4.40	4.50
Moisture contents (100Wm/Wd)…(%)	18.18	22.22
Average moisture contents (m)…(%)	20.20	

LL = 30.00%, PL = 20.20%, PI = 9.80%, SL= 8.57%

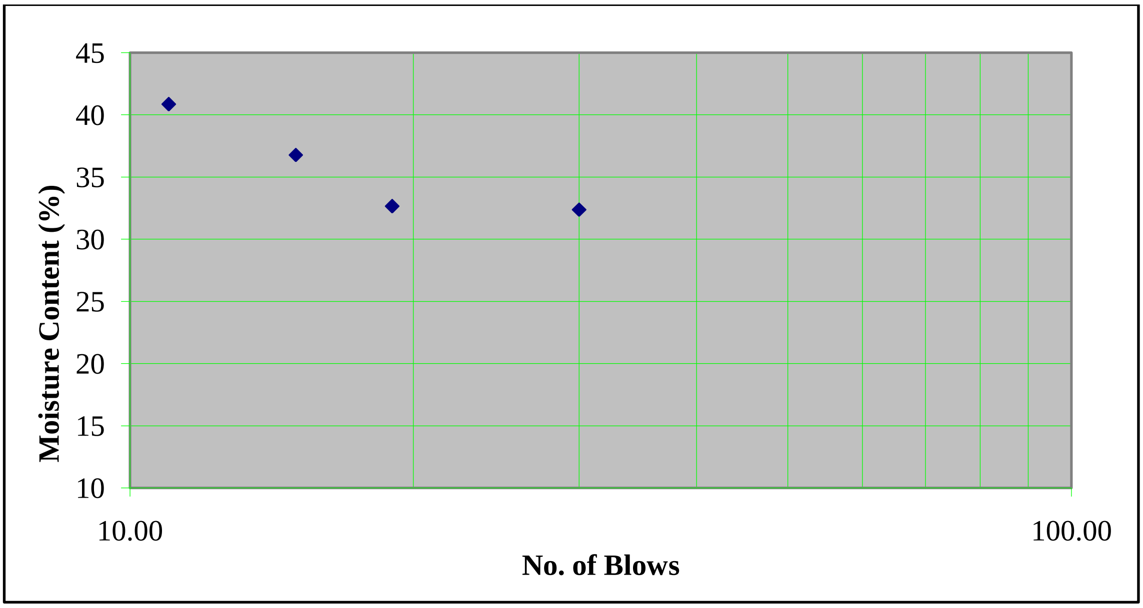

Figure A.16: Flow Curve of Soil Samples fromMTS 1-6 TP1

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ATTERBERG'S LIMITS TEST RESULTS SHEET

LOCATION: MTS 1-6 KATABU-ZARIA ROAD (A2)
SAMPLE NO: TP2 CENTRE CH: 16+000
Table A.17: Atterberg Limits of Soil Samples fromMTS 1-6 TP2

No. of blows	10	13	19	27
Container No.	47A	98	75	KJ
Wt. of wet soil and container………(g)	16.30	16.00	17.20	17.50
Wt. of dry soil and container……(g)	13.00	12.70	13.50	14.00
Wt. of container……(g)	4.00	4.00	4.00	4.00
Wt. of moisture (Wm)…(g)	3.30	3.30	3.70	3.50
Wt. of dry soil (Wd)……(g)	9.00	8.70	9.50	10.00
Moisture contents (100Wm/Wd)…(%)	36.67	37.93	38.95	35.00

Container No.	13	22
Wt. of wet soil and container………(g)	9.00	9.30
Wt. of dry soil and container……(g)	8.20	8.30
Wt. of container……(g)	4.00	3.90
Wt. of moisture (Wm)…(g)	0.80	1.00
Wt. of dry soil (Wd)……(g)	4.20	4.40
Moisture contents (100Wm/Wd)…(%)	19.05	22.73
Average moisture contents (m)…(%)	20.89	

LL = 35.00%, PL = 20.89%, PI = 14.11%, SL = 8.57%

Figure A.17: Flow Curve of Soil Samples fromMTS 1-6 TP2

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ATTERBERG'S LIMITS TEST RESULTS SHEET

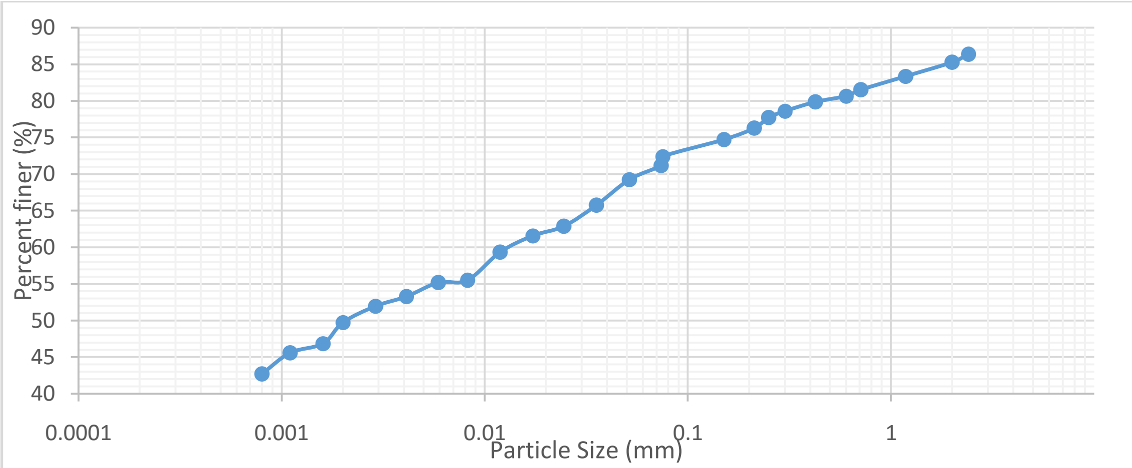
LOCATION: MTS 1-6 KATABU-ZARIA ROAD (A2)
SAMPLE NO: TP3 LEFT CH: 16+000
Table A.18: Atterberg Limits of Soil Samples fromMTS 1-6 TP3

No. of blows	12	18	25	36
Container No.	178	20	5	90
Wt. of wet soil and container………(g)	17.40	17.30	17.20	16.20
Wt. of dry soil and container……(g)	13.50	13.60	13.50	12.90
Wt. of container……(g)	3.90	4.10	3.90	4.10
Wt. of moisture (Wm)…(g)	3.90	3.70	3.70	3.30
Wt. of dry soil (Wd)……(g)	9.60	9.50	9.60	8.80
Moisture contents (100Wm/Wd)…(%)	40.63	38.95	38.54	37.50

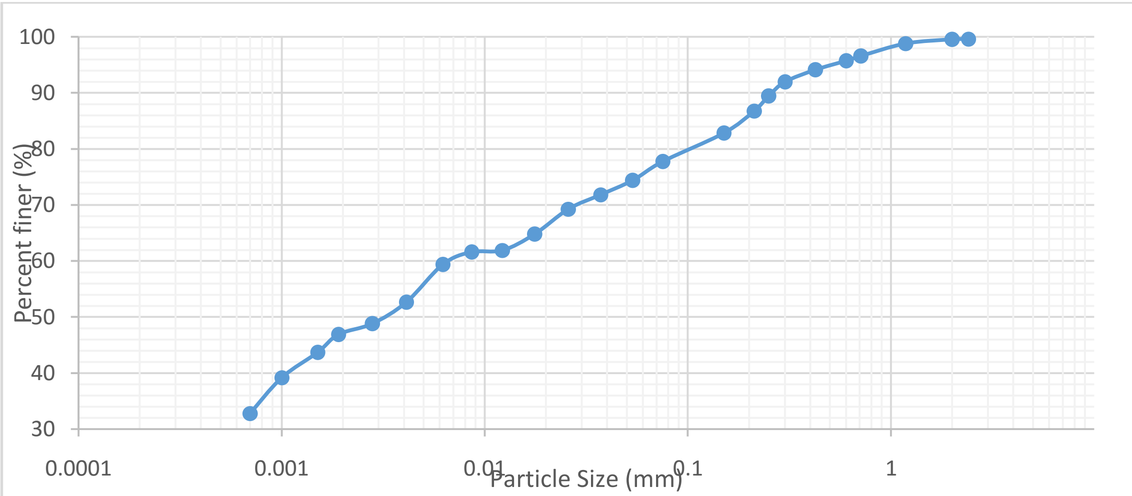
Container No.	30	78
Wt. of wet soil and container………(g)	8.60	9.10
Wt. of dry soil and container……(g)	7.70	8.20
Wt. of container……(g)	3.90	4.20
Wt. of moisture (Wm)…(g)	0.90	0.90
Wt. of dry soil (Wd)……(g)	3.80	4.00
Moisture contents (100Wm/Wd)…(%)	23.68	22.50
Average moisture contents (m)…(%)	23.09	

LL = 38.54%, PL = 23.09%, PI = 15.45%, SL= 10.71%

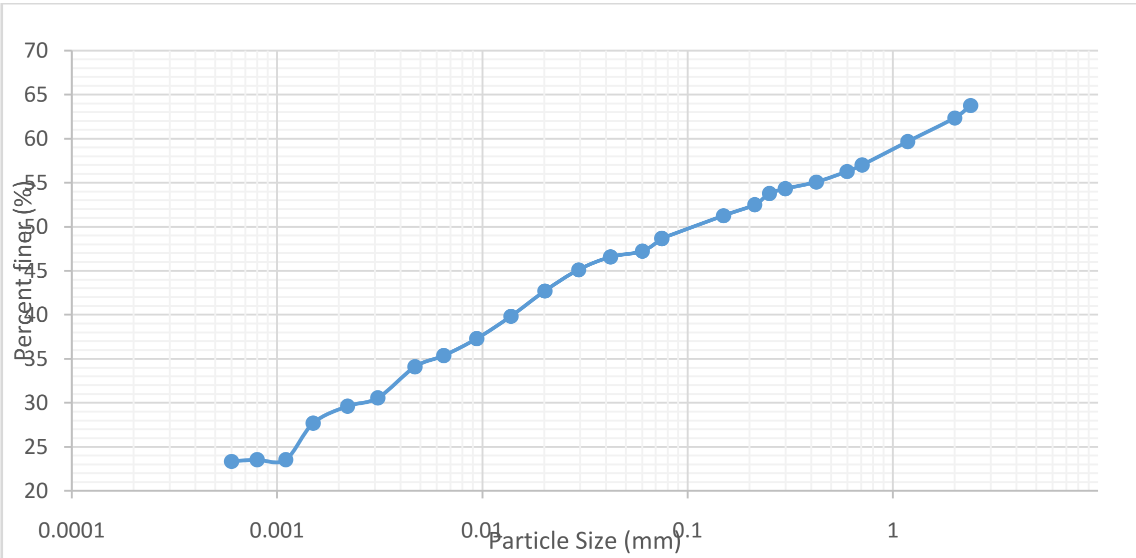
Figure A.18: Flow Curve of Soil Samples fromMTS 1-6 TP3



MTS 1-1 TP1

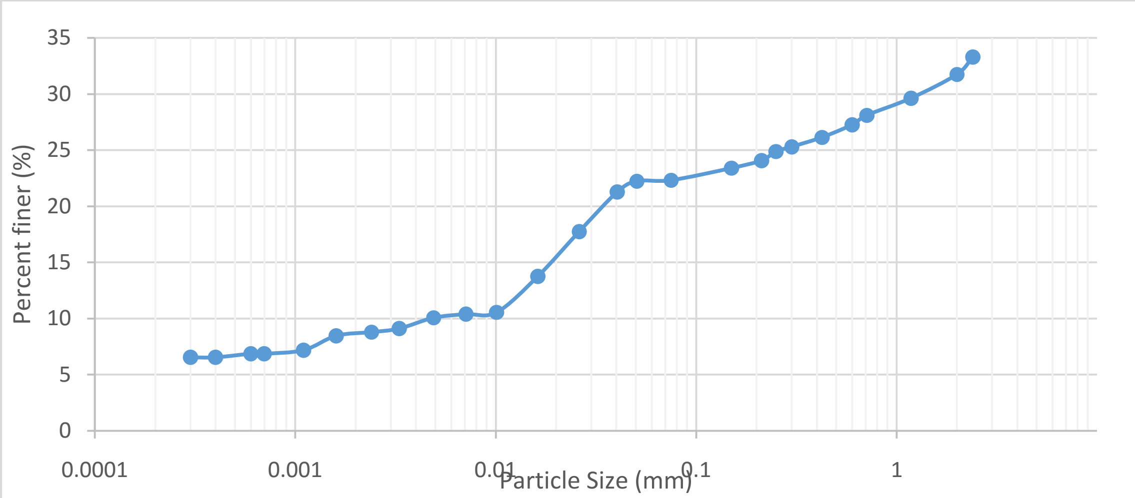


MTS 1-1 TP2

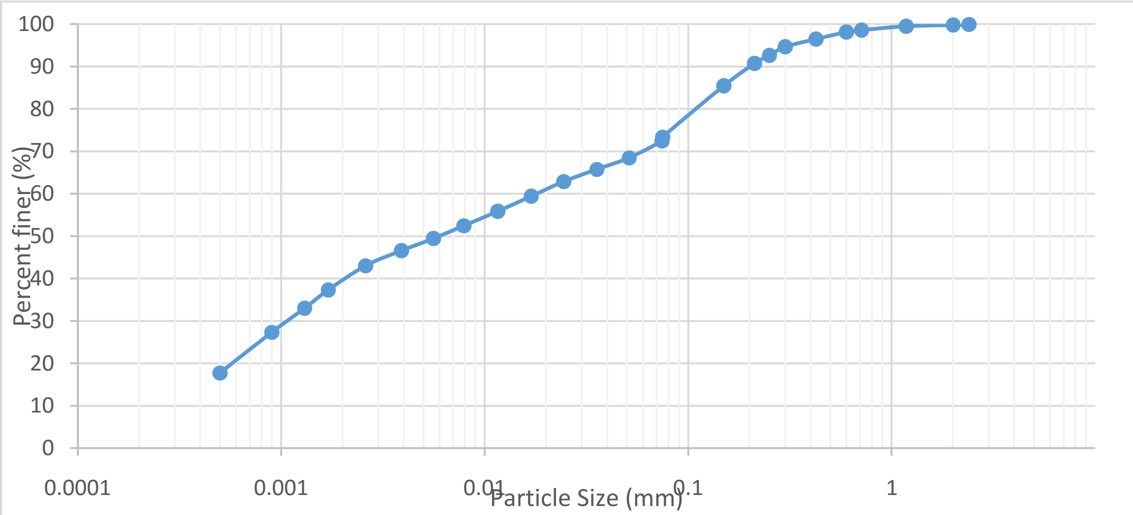


MTS 1-1 TP3

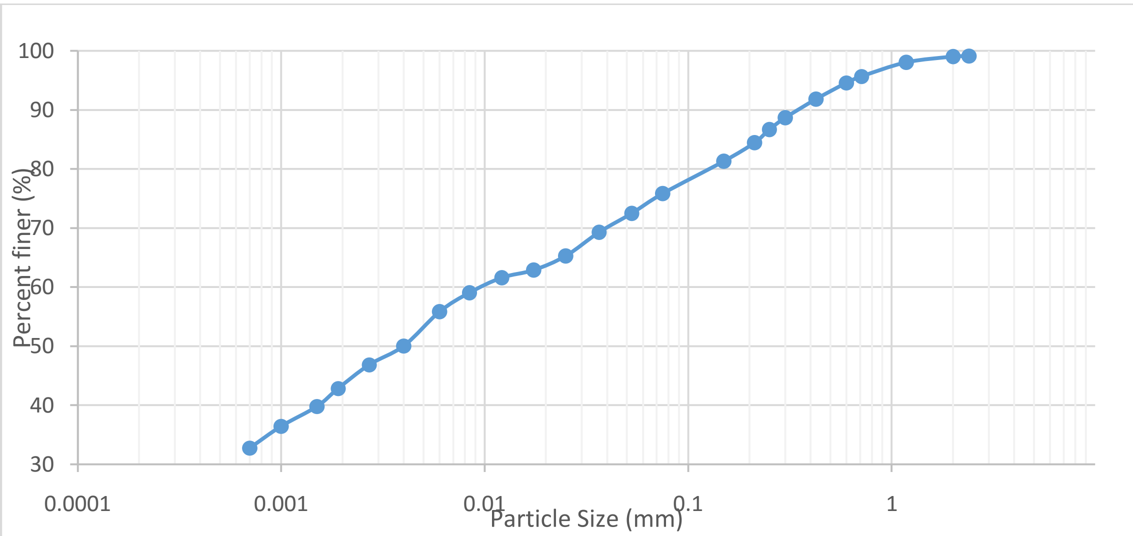
Figure A.19: Particle Size Distribution Curve of Soil Samples fromMTS 1-1



MTS 1-2 TP1

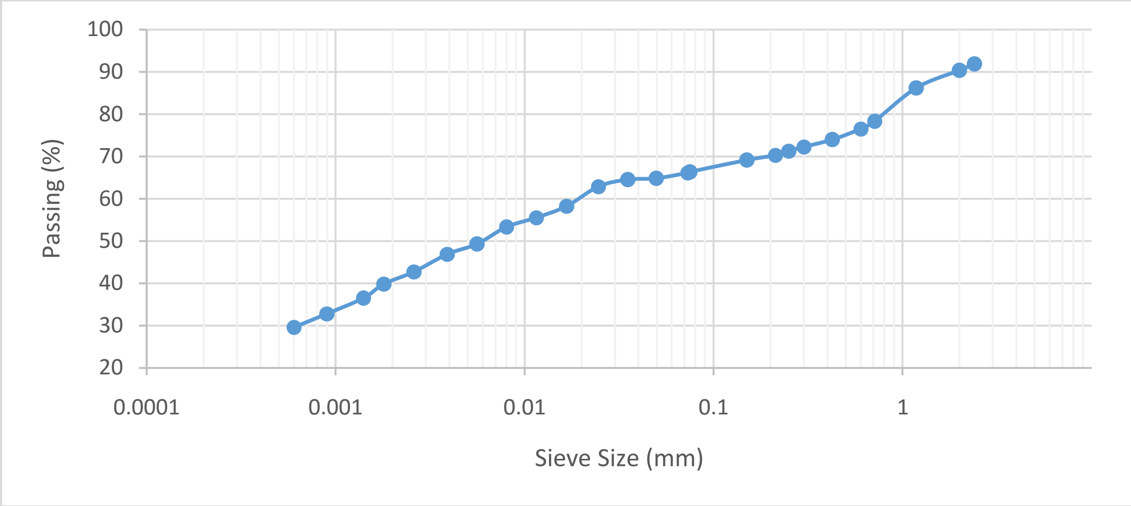


MTS 1-2 TP2

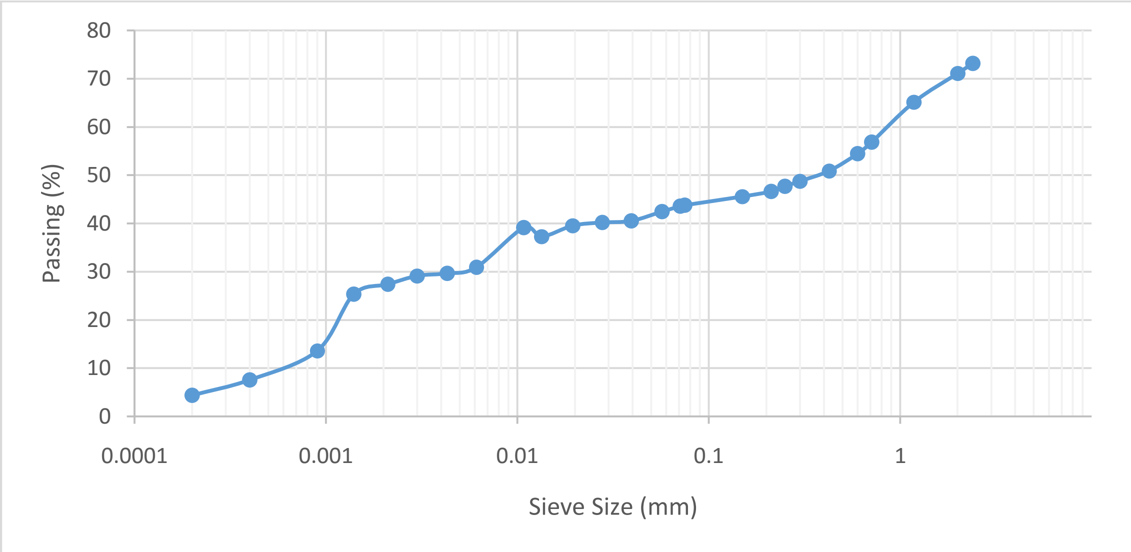


MTS 1-2 TP3

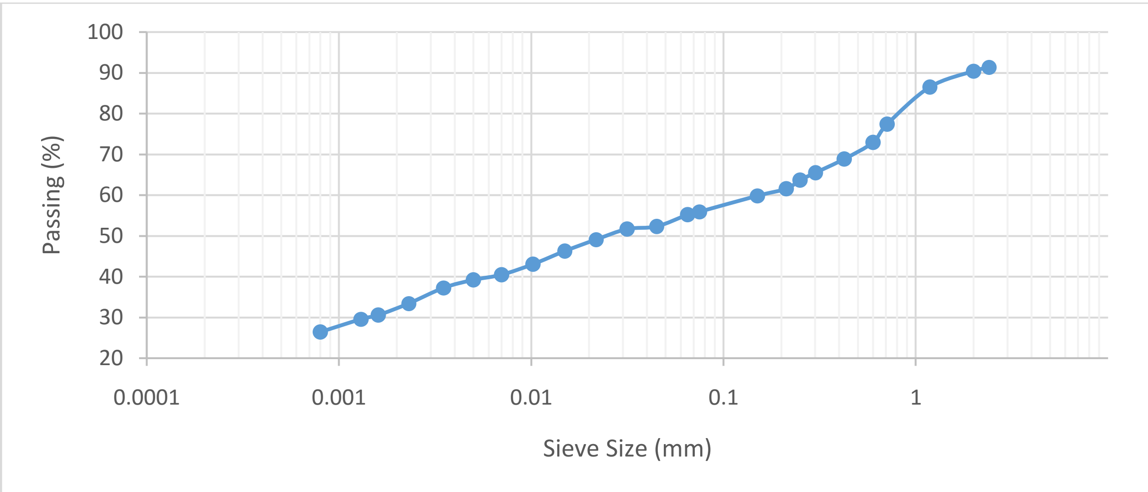
Figure A.20: Particle Size Distribution Curve of Soil Samples fromMTS 1-2



MTS 1-3 TP1

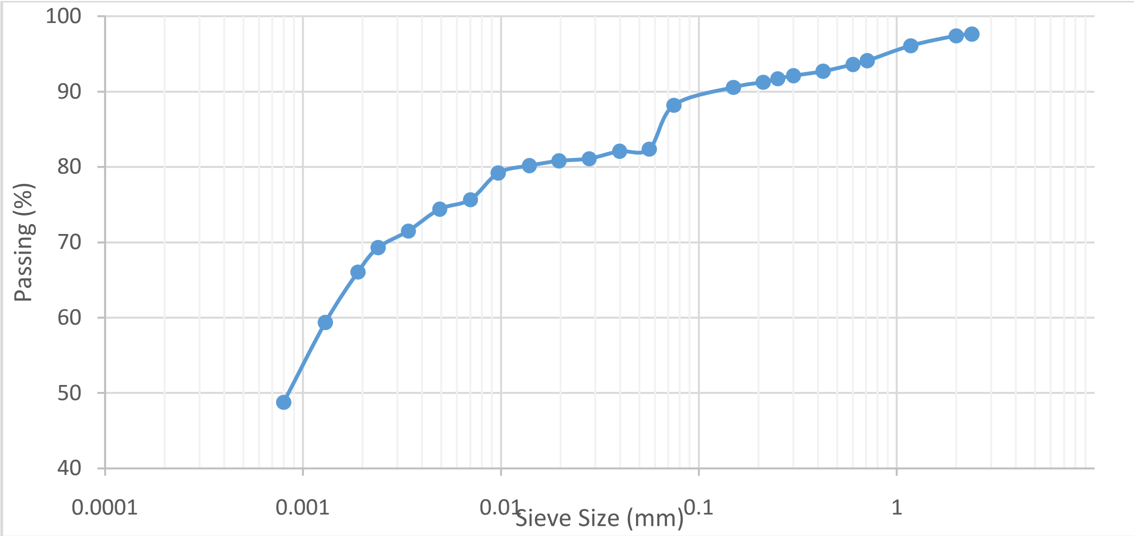


MTS 1-3 TP2

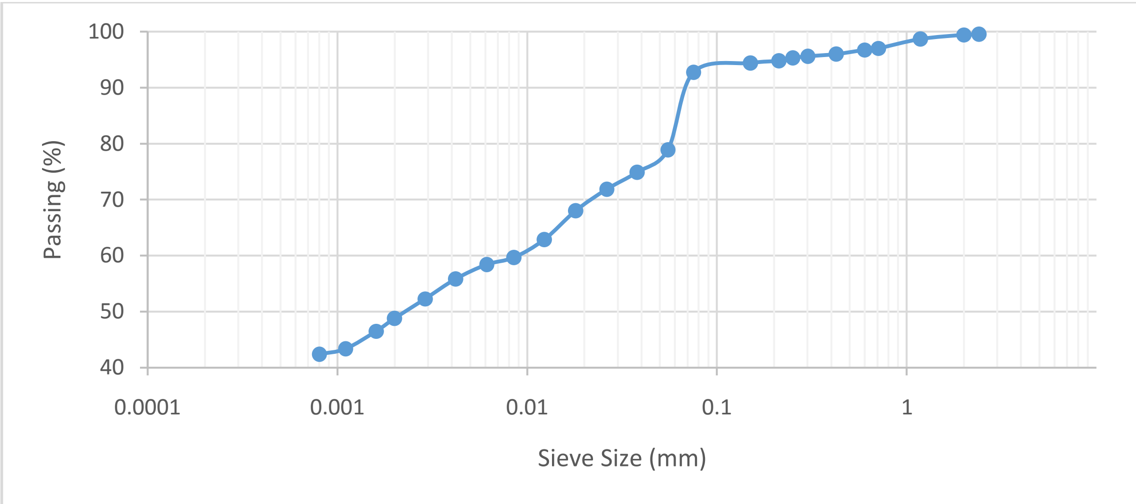


MTS 1-3 TP3

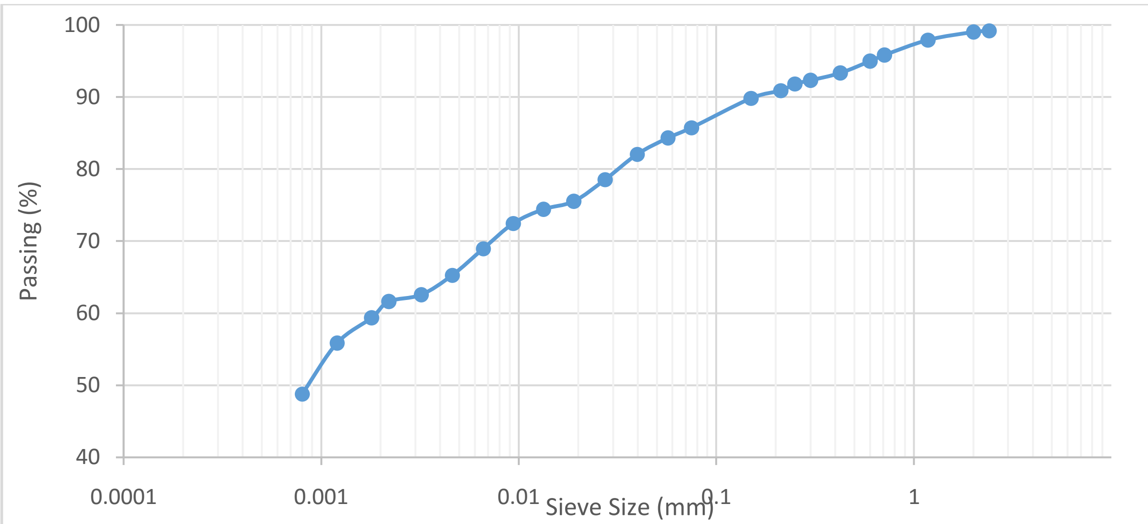
Figure A.21: Particle Size Distribution Curve of Soil Samples fromMTS 1-3



MTS 1-4 TP1

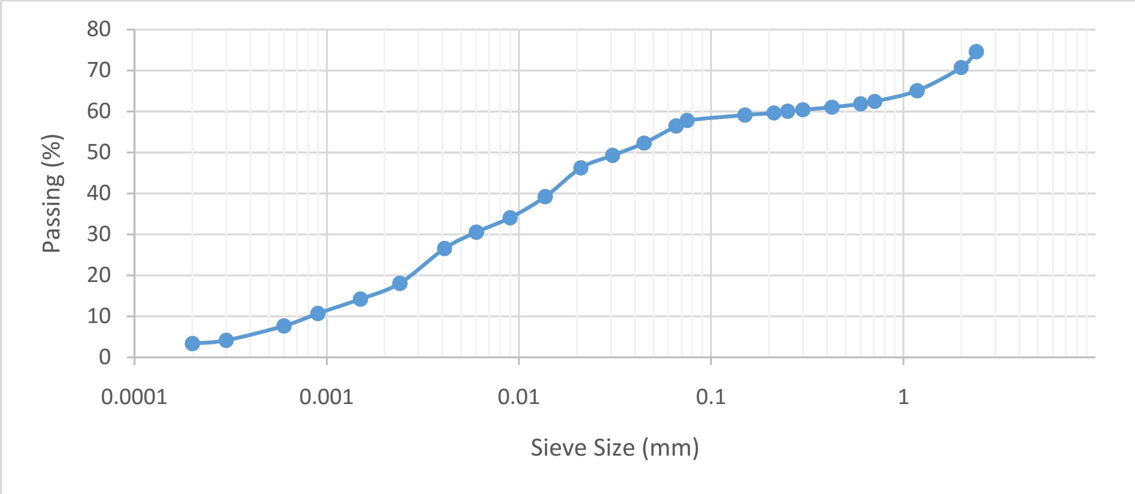


MTS 1-4 TP2

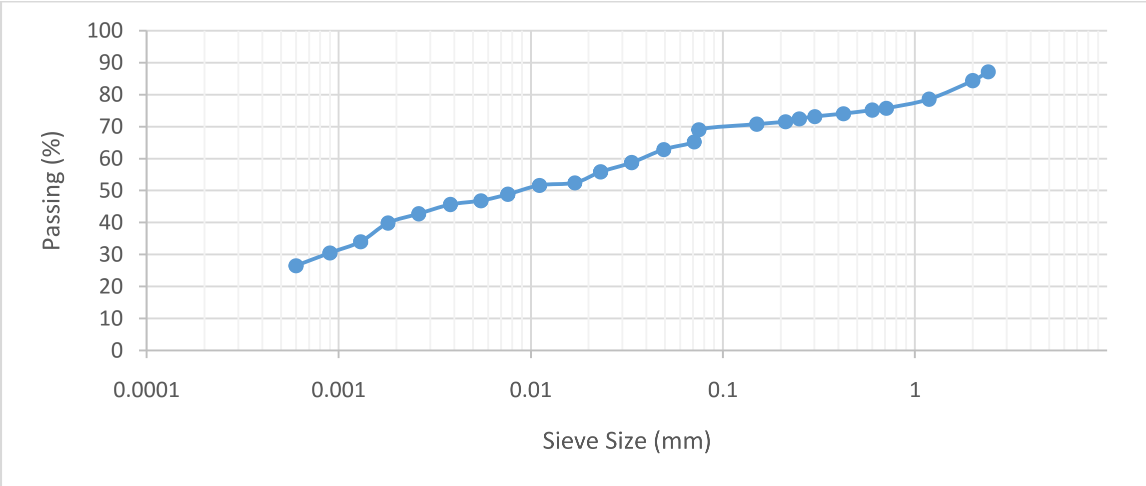


MTS 1-4 TP3

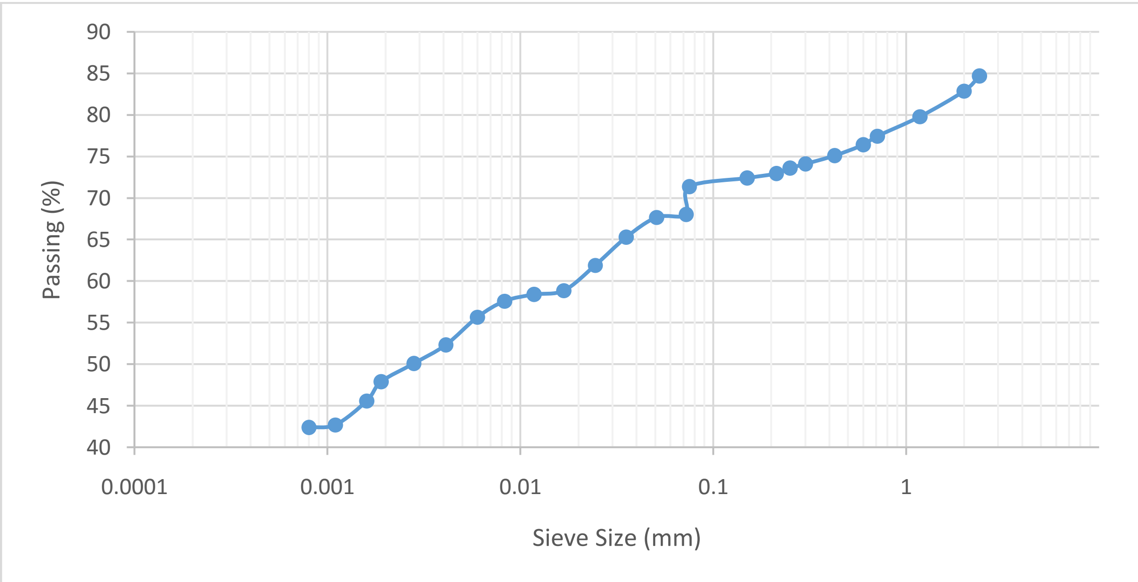
Figure A.22: Particle Size Distribution Curve of Soil Samples fromMTS 1-4



MTS 1-5 TP1

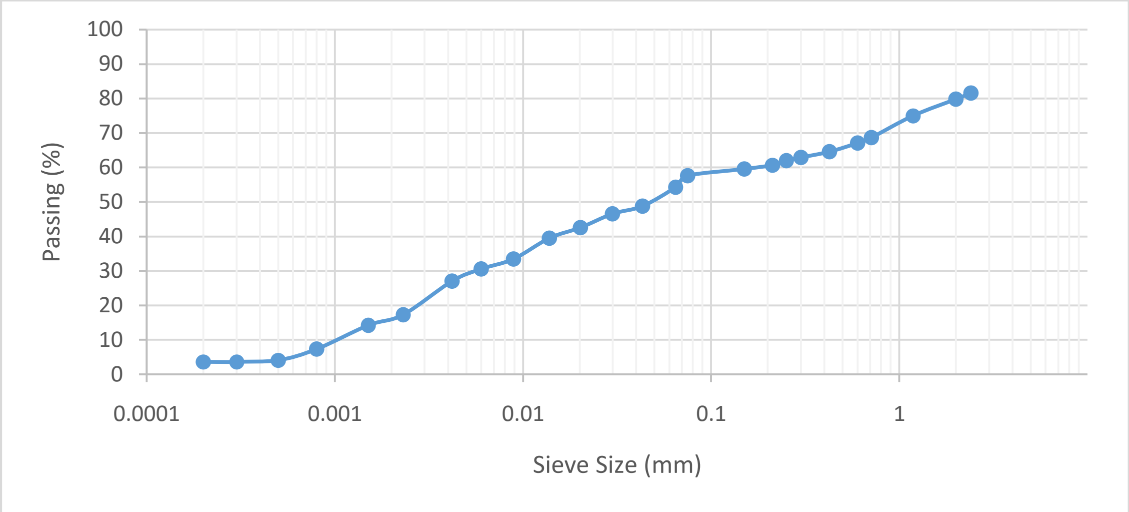


MTS 1-5 TP2

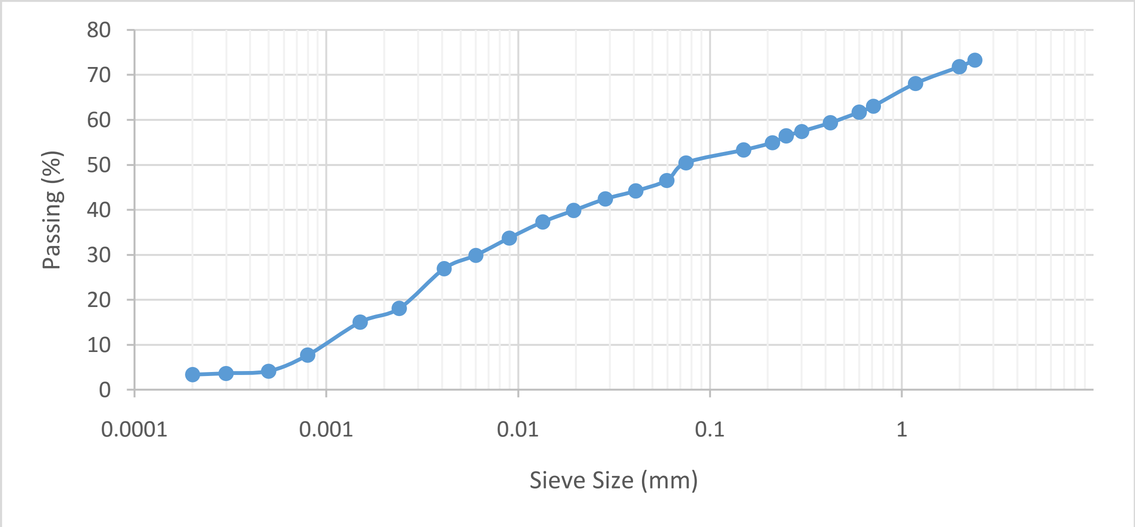


MTS 1-5TP3

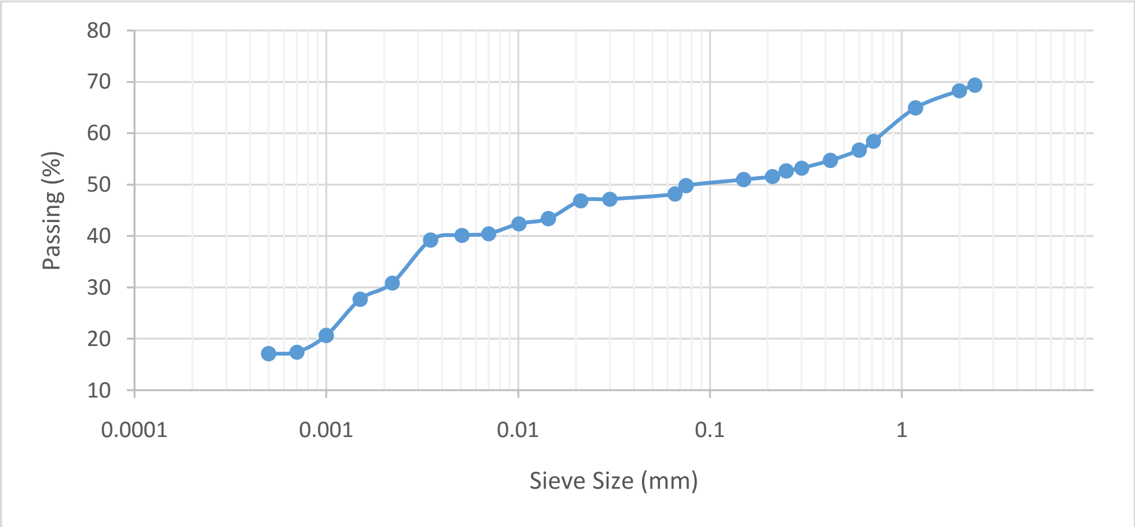
Figure A.23: Particle Size Distribution Curve of Soil Samples fromMTS 1-5



MTS 1-6 TP1

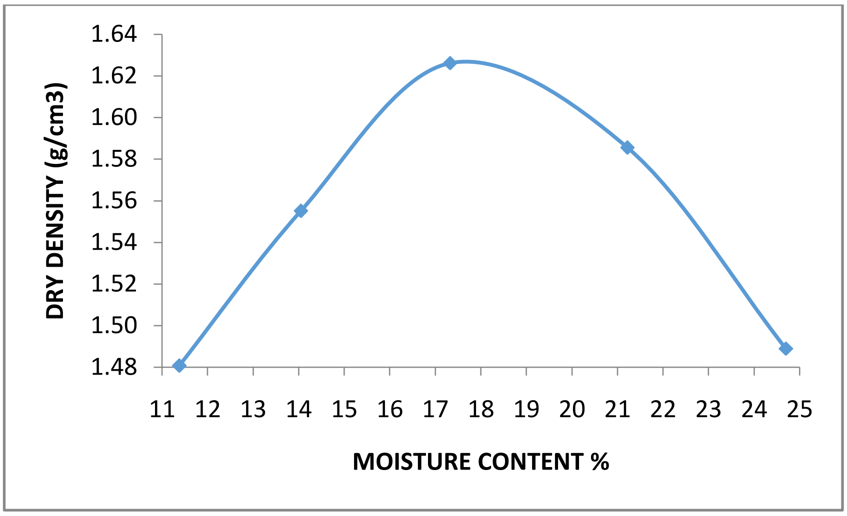


MTS 1-6 TP2

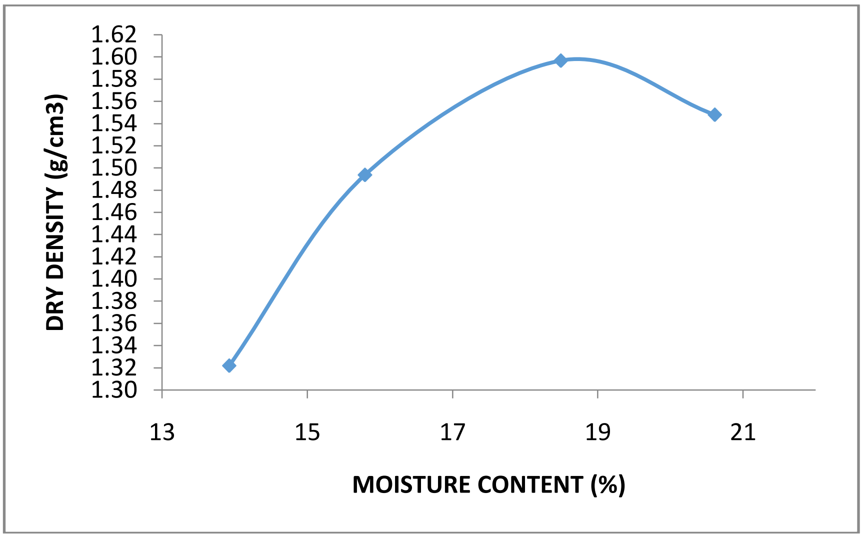


MTS 1-6 TP3

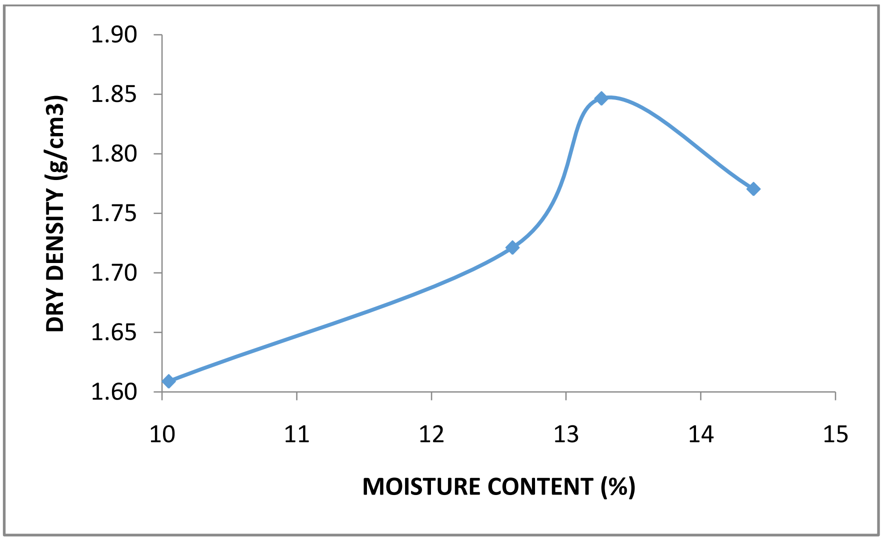
Figure A.24: Particle Size Distribution Curve of Soil Samples fromMTS 1-6



MTS 1-1 TP1

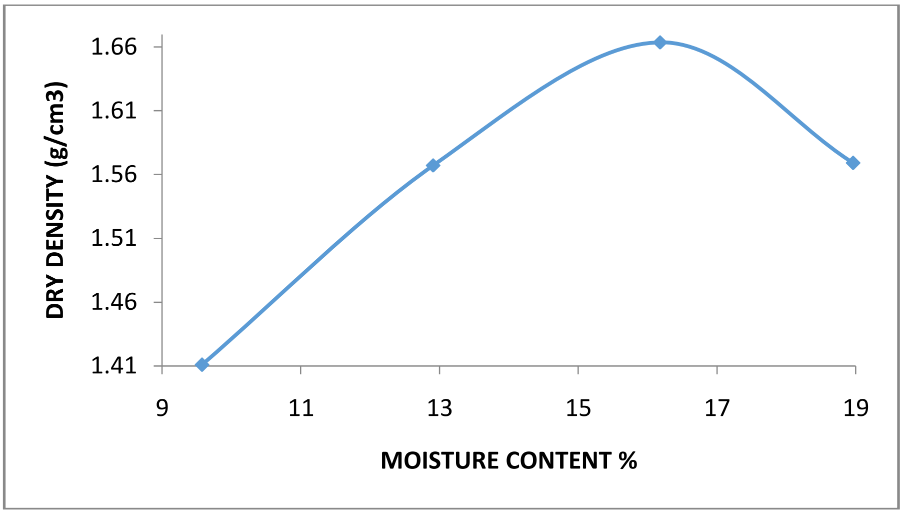


MTS 1-1 TP2

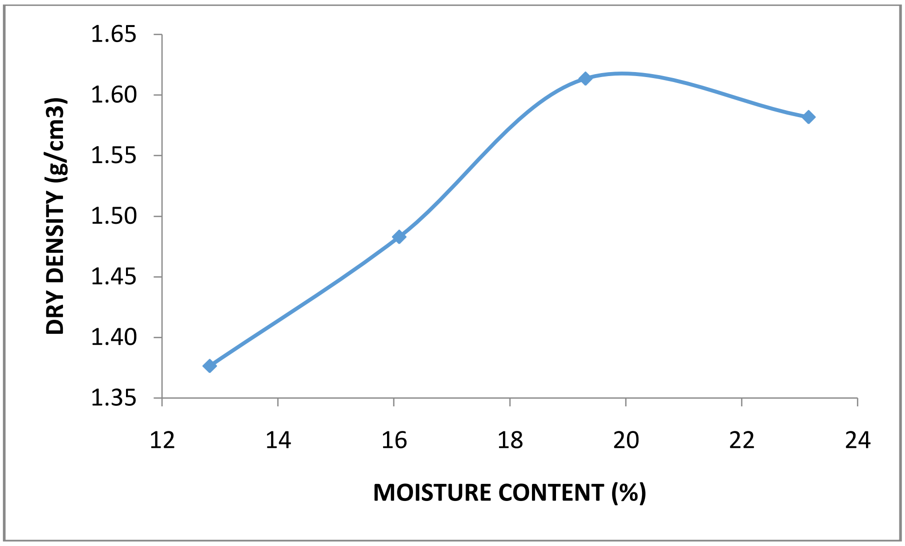


MTS 1-1 TP3

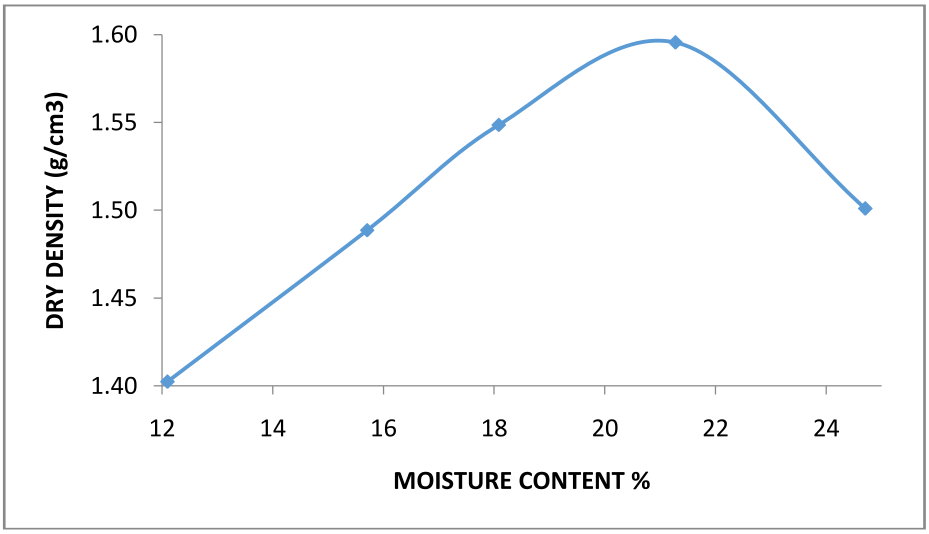
Figure A.25: Graph of Dry Density against Moisture Content for MTS 1-1



MTS 1-2 TP1

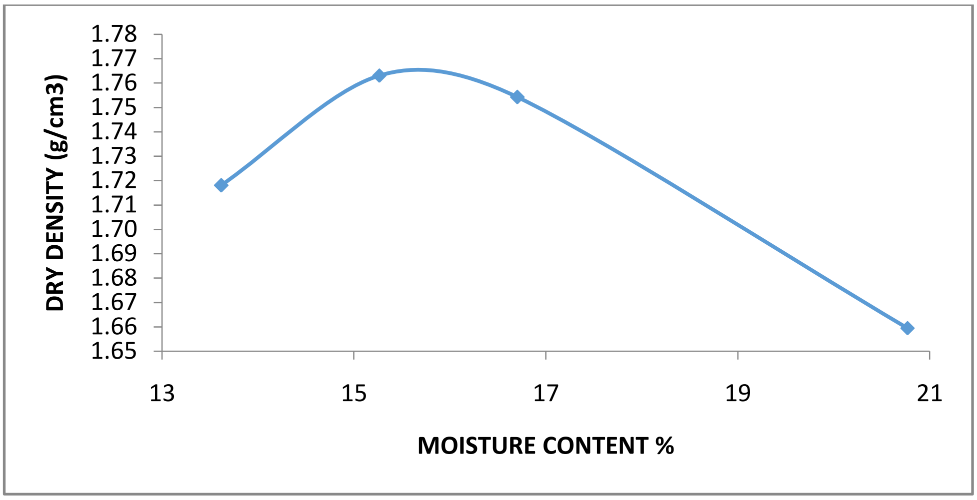


MTS 1-2 TP2

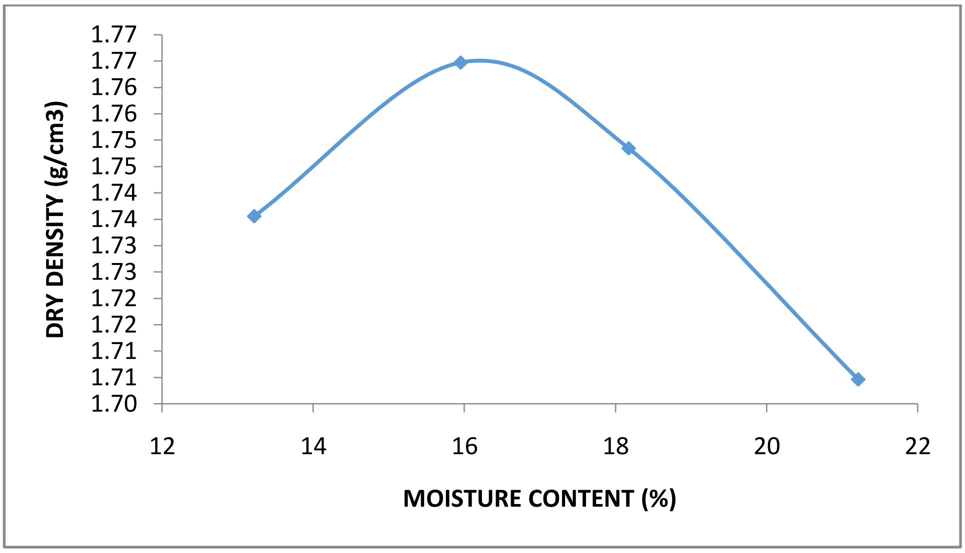


MTS 1-2 TP3

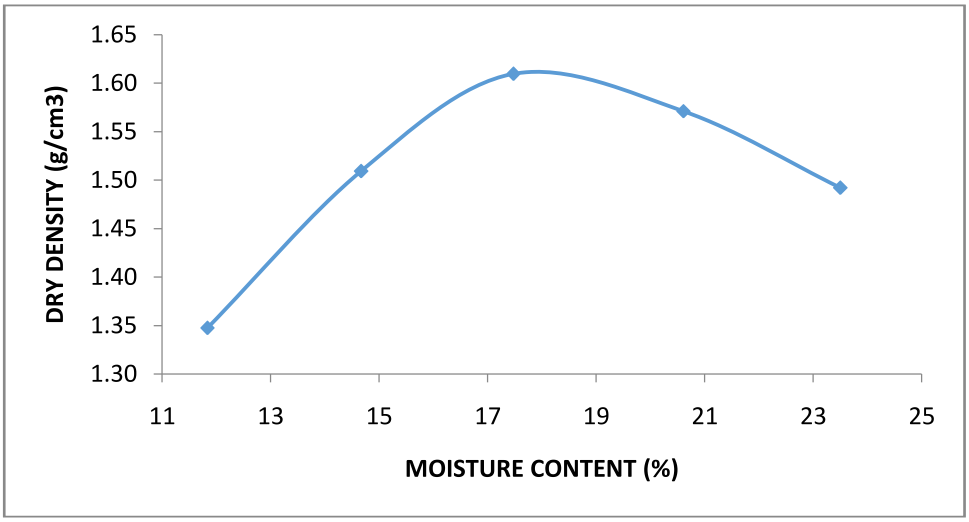
Figure A.26: Graph of Dry Density against Moisture Content for MTS 1-2



MTS 1-3 TP1

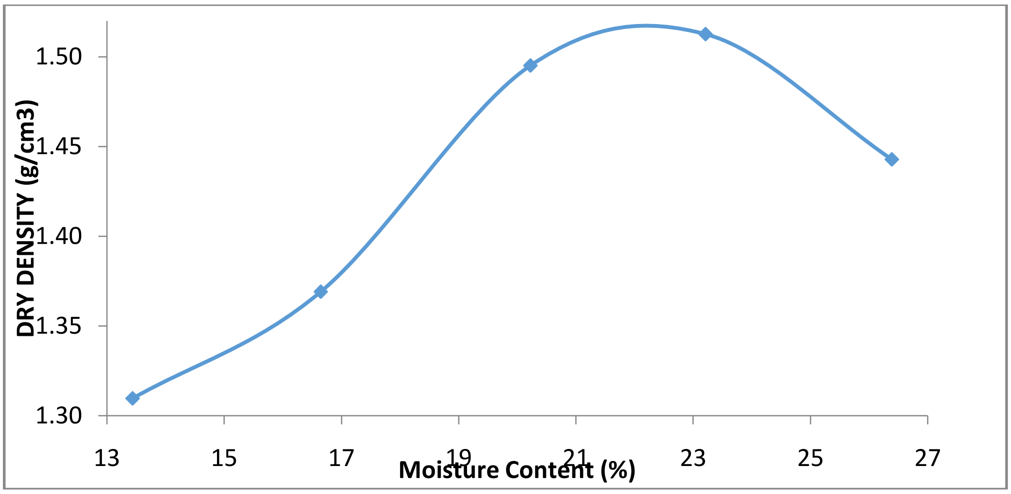


MTS 1-3 TP2

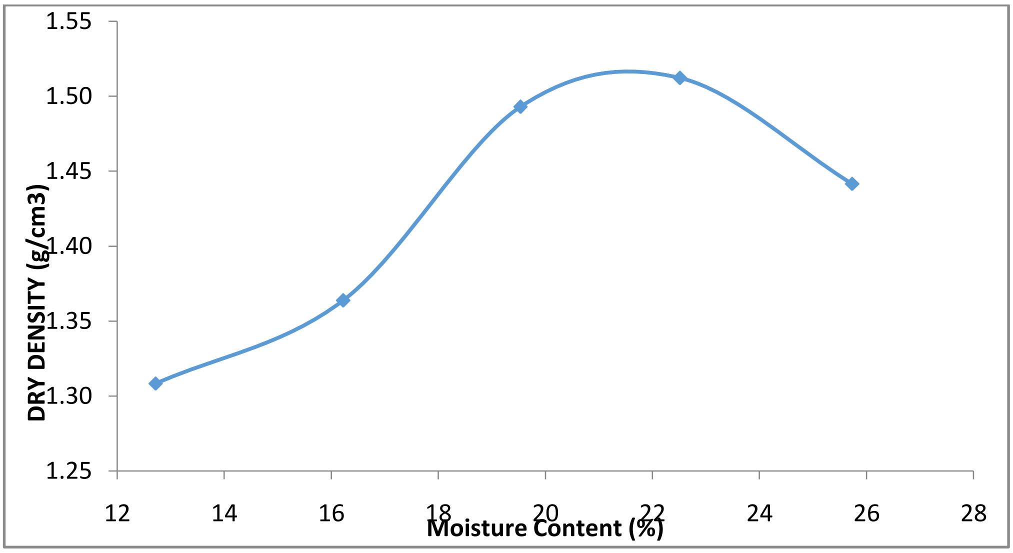


MTS 1-3 TP3

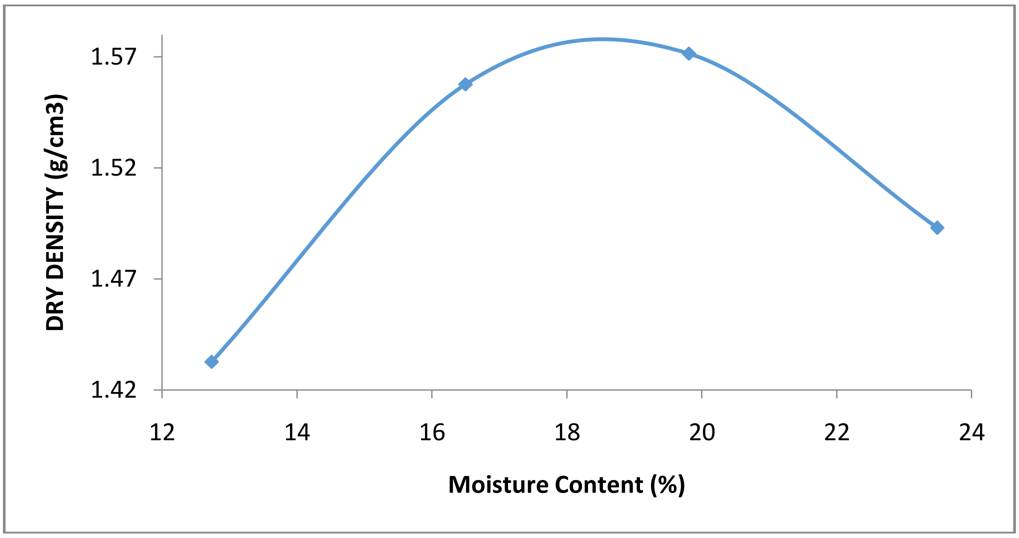
Figure A.27: Graph of Dry Density against Moisture Content for MTS 1-3



MTS 1-4 TP1

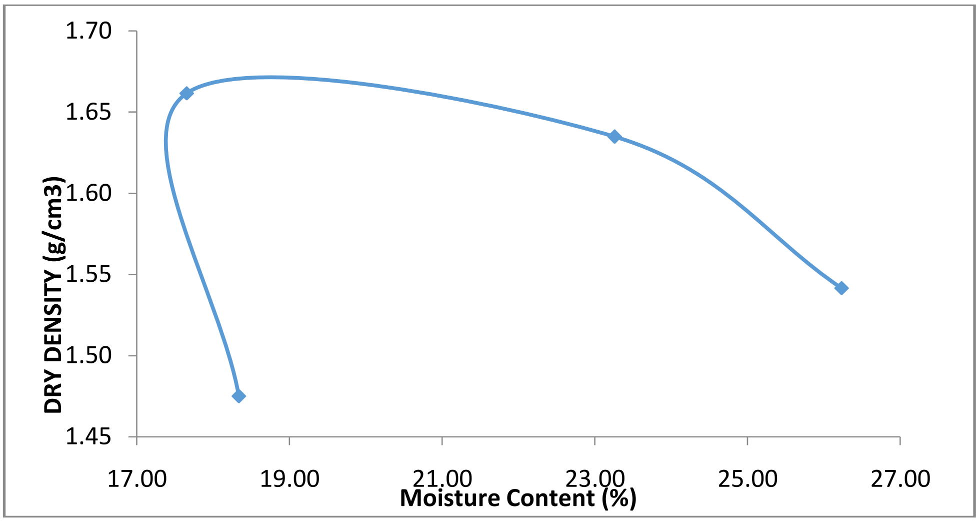


MTS 1-4 TP2

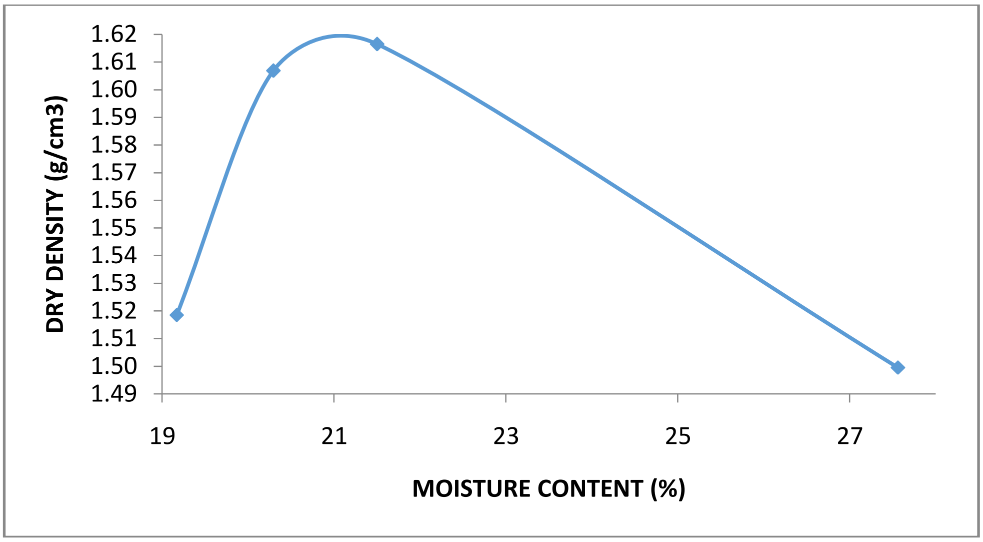


MTS 1-4 TP3

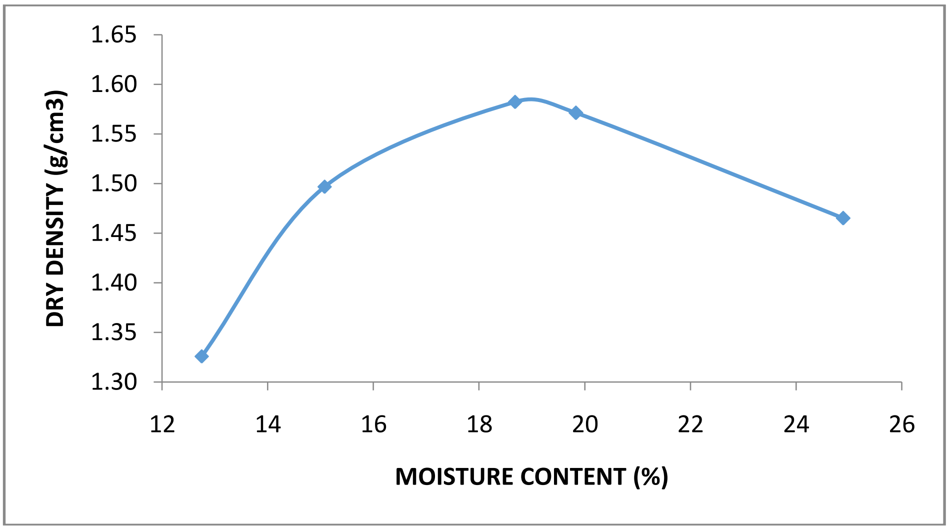
Figure A.28: Graph of Dry Density against Moisture Content for MTS 1-4



MTS 1-5 TP1

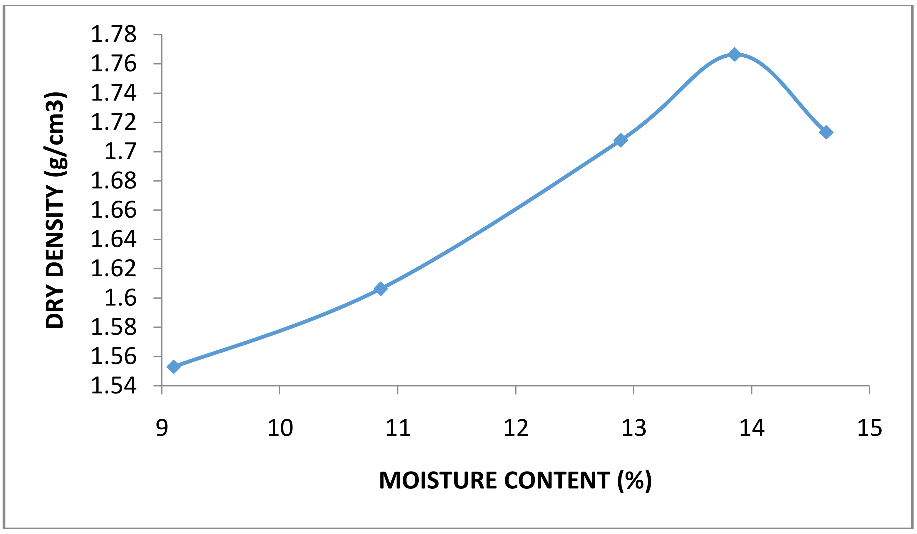


MTS 1-5 TP2

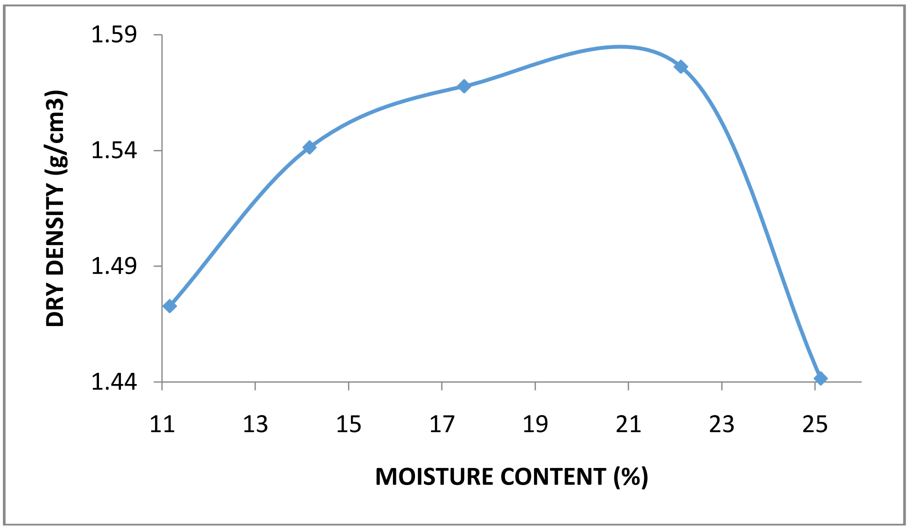


MTS 1-5 TP3

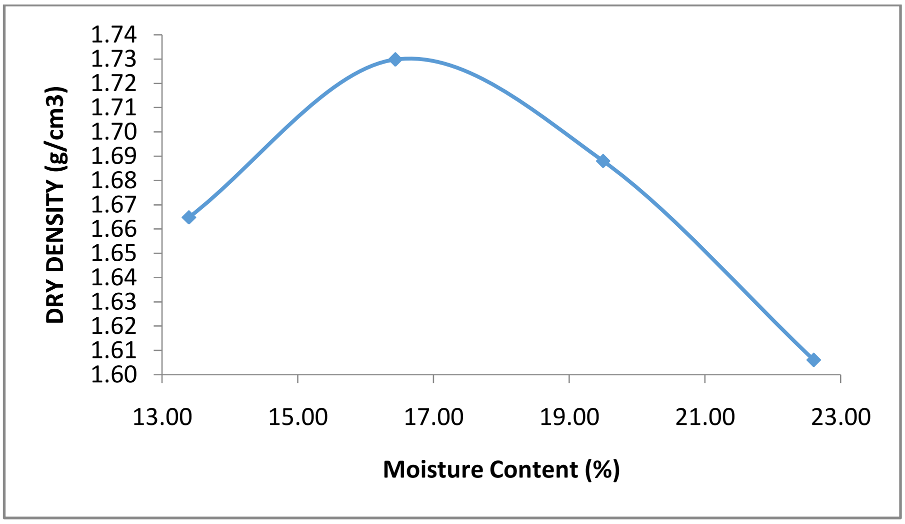
Figure A.29: Graph of Dry Density against Moisture Content for MTS 1-5



MTS 1-6 TP1



MTS 1-6 TP2



MTS 1-6 TP3

Figure A.30: Graph of Dry Density against Moisture Content for MTS 1-6

Table A.19: Data Sheet for UCSof the Soil Samples fromMTS 1

MTS 1 – 6 TP3

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
61	420	1156.43	1.4	106.68	1140.24	76.2	10135.7	114024	98.6	534.641

MTS 1 – 1 TP1

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
40	450	1157.6	1.5	114.3	1140.24	76.2	10135.7	114024	98.5	350.229

MTS 1 – 6 TP2

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
27	180	1147.12	0.6	45.72	1140.24	76.2	10135.7	114024	99.4	238.564

MTS 1 – 2 TP3

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
27	720	1168.28	2.4	182.88	1140.24	76.2	10135.7	114024	97.6	234.244

MTS 1 – 1 TP3

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
62	105	1144.24	0.35	26.67	1140.24	76.2	10135.7	114024	99.65	549.192

MTS 1 – 3 TP1

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
45	540	1161.14	1.8	137.16	1140.24	76.2	10135.7	114024	98.2	392.807

MTS 1 – 3 TP2

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
4.5	90	1143.67	0.3	22.86	1140.24	76.2	10135.7	114024	99.7	39.8807

MTS 1 – 4 TP1

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
21	570	1162.32	1.9	144.78	1140.24	76.2	10135.7	114024	98.1	183.123

MTS 1 – 3 TP3

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
34	330	1152.92	1.1	83.82	1140.24	76.2	10135.7	114024	98.9	298.903

MTS 1 – 5 TP2

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
48	150	1145.97	0.5	38.1	1140.24	76.2	10135.7	114024	99.5	424.541

MTS 1 – 4 TP2

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
42	570	1162.32	1.9	144.78	1140.24	76.2	10135.7	114024	98.1	366.247

MTS 1 – 2

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
24	240	1149.44	0.8	60.96	1140.24	76.2	10135.7	114024	99.2	211.631

MTS 1 – 2 TP2

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
13	330	1152.92	1.1	83.82	1140.24	76.2	10135.7	114024	98.9	114.287

MTS 1 – 1 TP2

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
8	165	1146.55	0.55	41.91	1140.24	76.2	10135.7	114024	99.45	70.7213

MTS 1 – 5 TP1

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
21	240	1149.44	0.8	60.96	1140.24	76.2	10135.7	114024	99.2	185.177

MTS 1 – 4 TP3

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
2	45	1141.95	0.15	11.43	1140.24	76.2	10135.7	114024	99.85	17.7514

MTS 1 – 6 TP1

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
5	75	1143.1	0.25	19.05	1140.24	76.2	10135.7	114024	99.75	44.3341

MTS 1 – 5 TP3

Pi	X	A	E	dL	A'	L'	constant	C	D	UCS
45	150	1145.97	0.5	38.1	1140.24	76.2	10135.7	114024	99.5	398.007

Table A.20: Data Sheet forNMC of the Soil Samples from MTS 1

MTS 1 – 6 TP3

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
A2	4.7	31	30.6	25.9	0.4	1.54440154	1.342543
3A	4.7	31.3	31	26.3	0.3	1.14068441	

MTS 1 – 3 TP3

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
47	3.9	38.8	36.7	32.8	2.1	6.40243902	7.5089118
80	3.9	39.2	36.4	32.5	2.8	8.61538462	

MTS 1 – 5 TP3

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
18	4.1	38.1	35	30.9	3.1	10.0323625	10.355987
11	4	38.2	34.9	30.9	3.3	10.6796117	

MTS 1 – 5 TP1

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
116	4.2	37.4	35.2	31	2.2	7.09677419	7.7571413
28	3.9	36.1	33.6	29.7	2.5	8.41750842	

MTS 1 – 3 TP2

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%

78	4.2	36.1	32.1	27.9	4	14.3369176	14.311316
54	4	35.2	31.3	27.3	3.9	14.2857143	

MTS 1 – 5 TP2

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
68	4	37.9	35.5	31.5	2.4	7.61904762	7.7403414
P7	4	38.3	35.8	31.8	2.5	7.86163522	

MTS 1 – 3TP1

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
88	4	39.8	35.8	31.8	4	12.5786164	12.379052
82	3.9	38.9	35.1	31.2	3.8	12.1794872	

MTS 1 – 1 TP3

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
20	5.4	36.1	35.7	30.3	0.4	1.32013201	2.3549813
39	5.4	35.9	34.9	29.5	1	3.38983051	

MTS 1 – 1 TP2

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
12	5.5	34.1	32.4	26.9	1.7	6.3197026	5.5495572
94	5.4	33.9	32.6	27.2	1.3	4.77941176	

MTS 1 – 6 TP2

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
30A	5.4	33.7	33.3	27.9	0.4	1.43369176	1.609703
2W	5.4	33.9	33.4	28	0.5	1.78571429	

MTS 1 – 1 TP1

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
92	4.7	32.2	31.3	26.6	0.9	3.38345865	2.9832422
33	4.7	32.5	31.8	27.1	0.7	2.58302583	

MTS 1 – 6 TP1

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
52	5.4	33.8	33.4	28	0.4	1.42857143	1.2538541
29	5.4	33.5	33.2	27.8	0.3	1.07913669	

MTS 1 – 2 TP3

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
87	5.3	33	32.7	27.4	0.3	1.09489051	0.7404954
AH	4.4	30.4	30.3	25.9	0.1	0.38610039	

MTS 1 – 2 TP1

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
47	4.7	31.7	31.5	26.8	0.2	0.74626866	0.9391721
10	4.7	31.5	31.2	26.5	0.3	1.13207547	

MTS 1 – 2 TP2

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
116	5.4	31.9	31.7	26.3	0.2	0.76045627	0.7663285
86	5.4	31.5	31.3	25.9	0.2	0.77220077	

MTS 1 – 4 TP1

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
117	5.4	24.6	23.9	18.5	0.7	3.78378378	2.9671607
3	5.4	24.4	24	18.6	0.4	2.15053763	

MTS 1 – 4 TP2

CON NO	WT OF EMPTY CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
77	4.8	36.1	35.4	30.6	0.7	2.2875817	2.4217461
202	5.5	37.6	36.8	31.3	0.8	2.55591054	

MTS 1 – 4 TP3

CON NO	WT OF EMPT CONT.(g)	WT OF CONT+WET SOIL (g)	WT OF CONT +DRY SOIL(g)	WT OF DRY SOIL(g)	WT OF MOIST(g)	M.C.%	N.M.C.%
94	5.4	38.6	37.9	32.5	0.7	2.15384615	2.2745279
83B	5.4	39.6	38.8	33.4	0.8	2.39520958	

Table A.21: Data sheet forSpecific Gravity of Soil Samples from MTS 1

MTS 1 – 1 TP2

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	59.4	99	81.4	29.8	51.8	39.6	12.2	2.44262

MTS 1 – 2 TP2

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	56.9	97.6	81.4	27.3	51.8	40.7	11.1	2.45946

MTS 1 – 3 TP2

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	53.1	97.8	81.4	23.5	51.8	44.7	7.1	3.30986

MTS 1 – 5 TP3

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	52.6	95.3	81.4	23	51.8	42.7	9.1	2.52747

MTS 1 – 3 TP3

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	59.2	99.2	81.4	29.6	51.8	40	11.8	2.50847

MTS 1 – 1 TP1

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	59	99.1	81.4	29.4	51.8	40.1	11.7	2.51282

MTS 1 – 6 TP2

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	63.5	101.5	81.4	33.9	51.8	38	13.8	2.45652

MTS 1 – 1 TP3

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	60	99.6	81.4	30.4	51.8	39.6	12.2	2.4918

MTS 1 – 6 TP1

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	58.7	99.2	81.4	29.1	51.8	40.5	11.3	2.57522

MTS 1 – 3 TP2

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	58.1	98.3	81.4	28.5	51.8	40.2	11.6	2.4569

MTS 1 – 5 TP2

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	55.4	96.9	81.4	25.8	51.8	41.5	10.3	2.50485

MTS 1 – 6 TP3

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	59.2	99	81.4	29.6	51.8	39.8	12	2.46667

MTS 1 – 2 TP3

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	57.7	98.3	81.4	28.1	51.8	40.6	11.2	2.50893

MTS 1 – 2 TP1

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	58.1	98.8	81.4	28.5	51.8	40.7	11.1	2.56757

MTS 1 – 5 TP1

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	58	98.8	81.4	28.4	51.8	40.8	11	2.58182

MTS 1 – 4 TP1

M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	57.8	98.5	81.4	28.2	51.8	40.7	11.1	2.54054

MTS 1 – 4 TP2

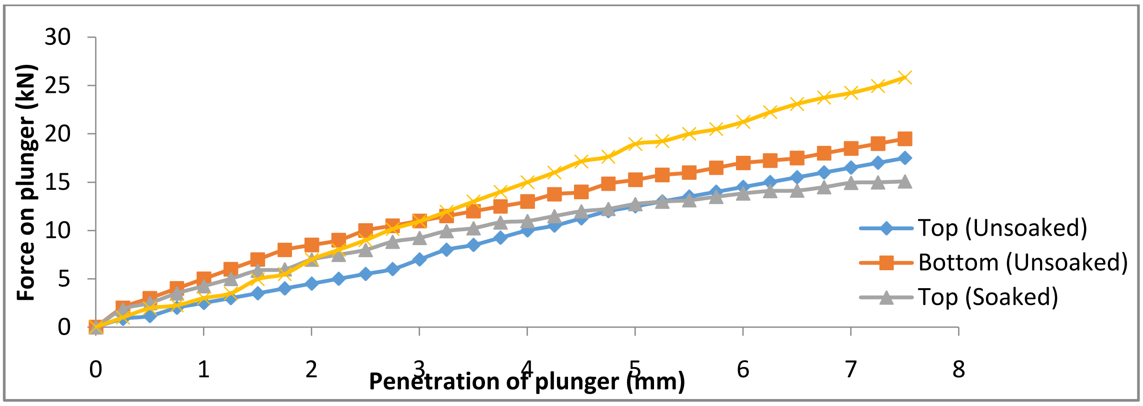
M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	57.4	98.3	81.4	27.8	51.8	40.9	10.9	2.55046

MTS 1 – 4 TP3

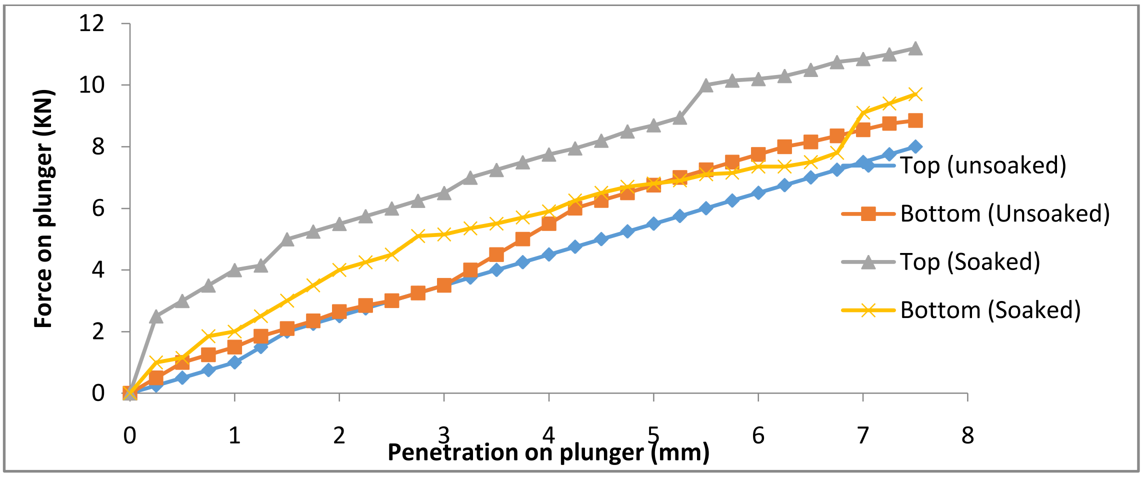
M1	M2	M3	M4	M2-M1	M4-M1	M3-M2	B23	GS
29.6	56.9	98	81.4	27.3	51.8	41.1	10.7	2.5514

APPENDIX B

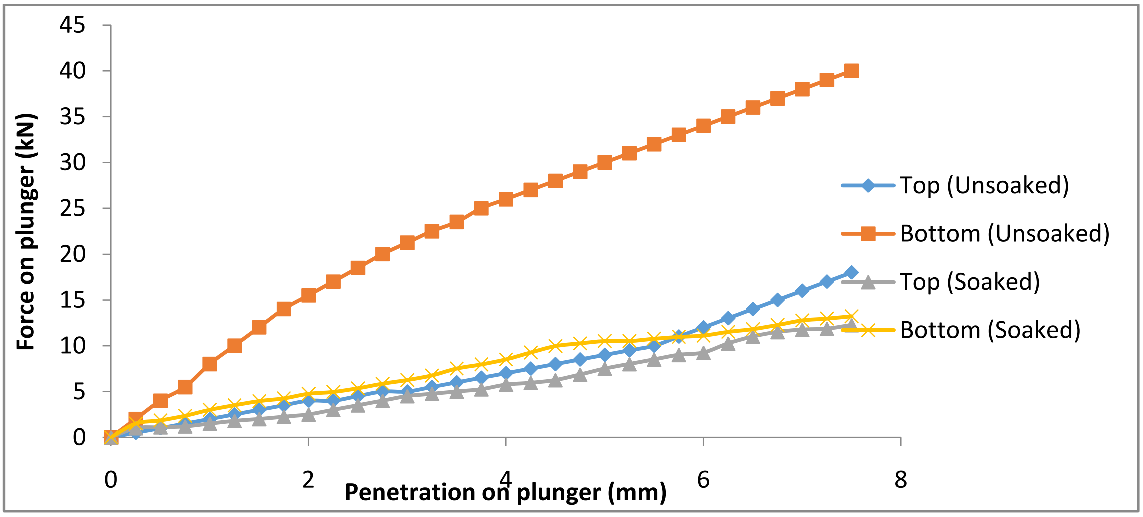
CALIFORNIA BEARING RATIO TESTS OF THE SOIL SAMPLES



MTS 1-1 TP1

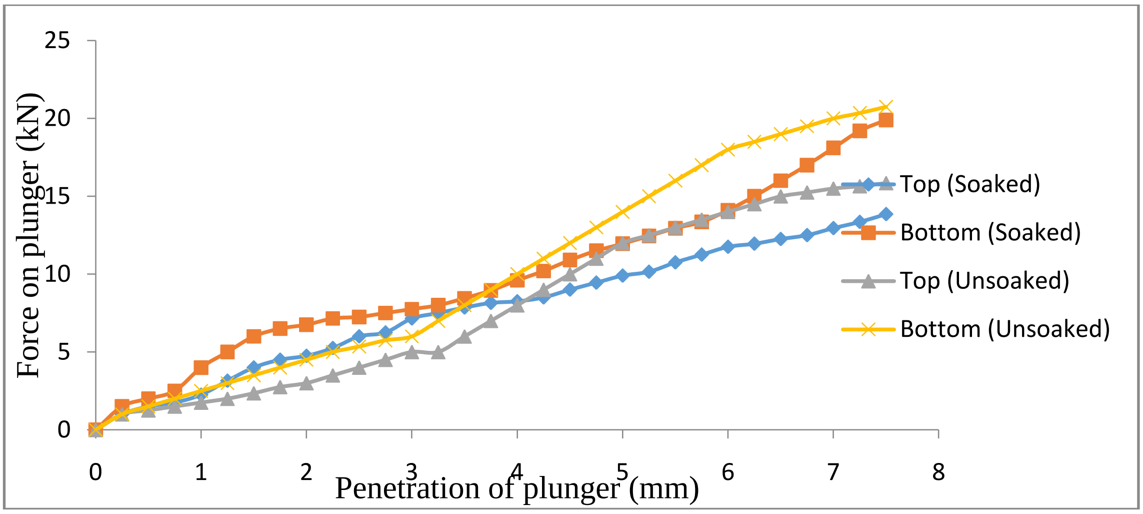


MTS 1-1 TP2

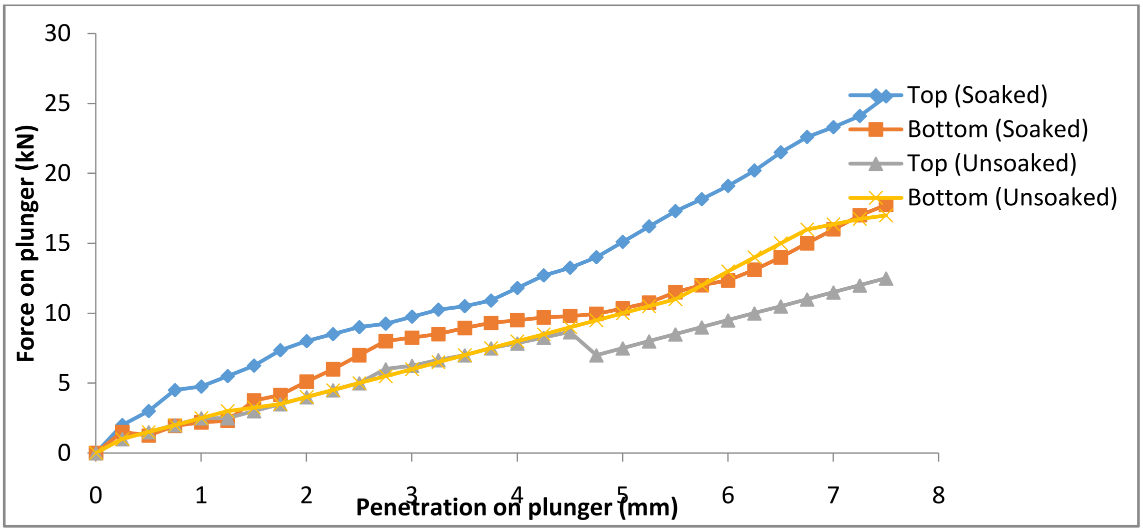


MTS 1-1 TP3

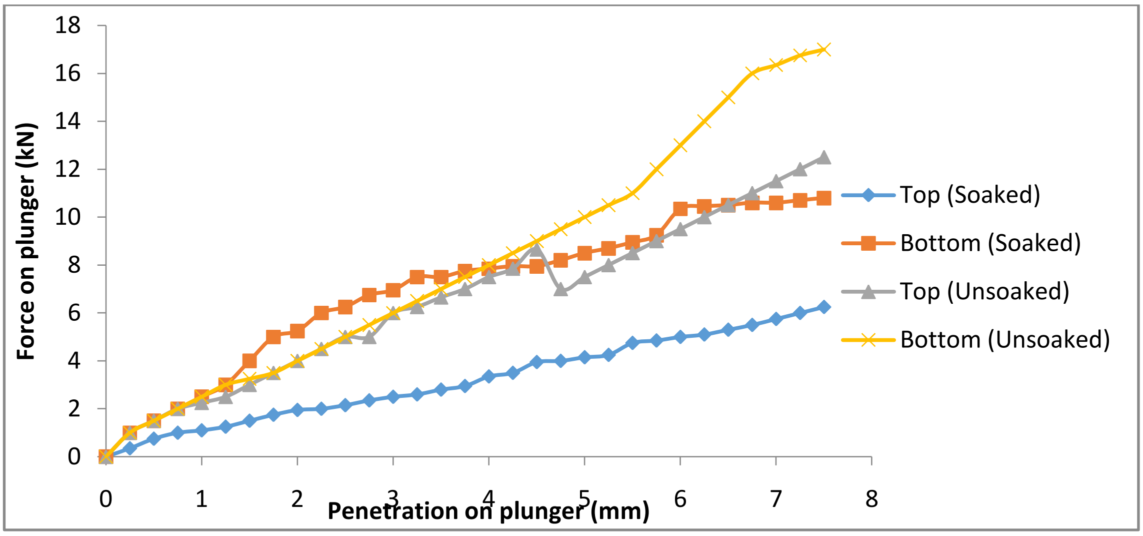
Figure B.1: Force against Penetration of Plunger of Soil Samples fromMTS 1-1



MTS 1-2 TP1

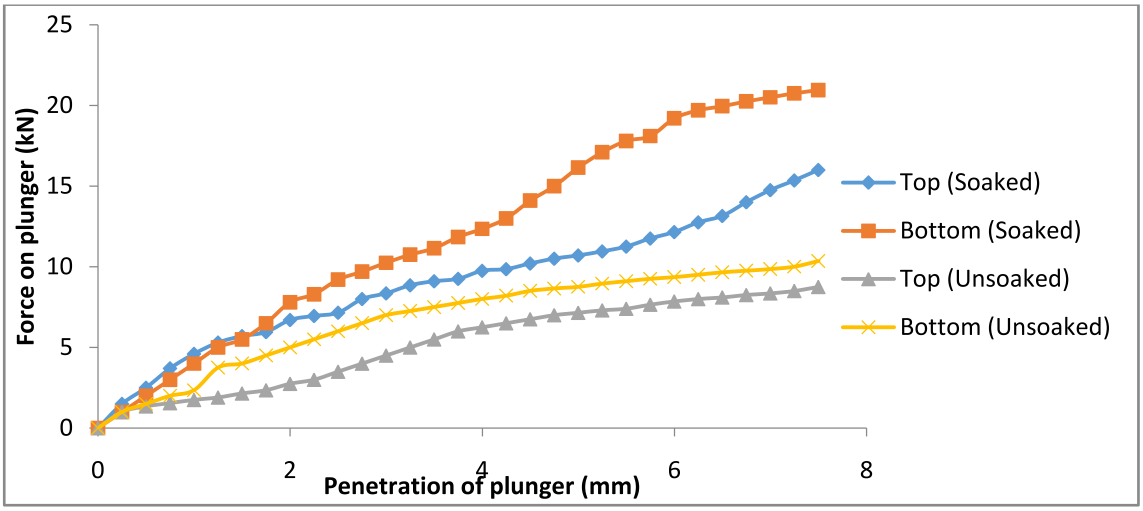


MTS 1-2 TP2

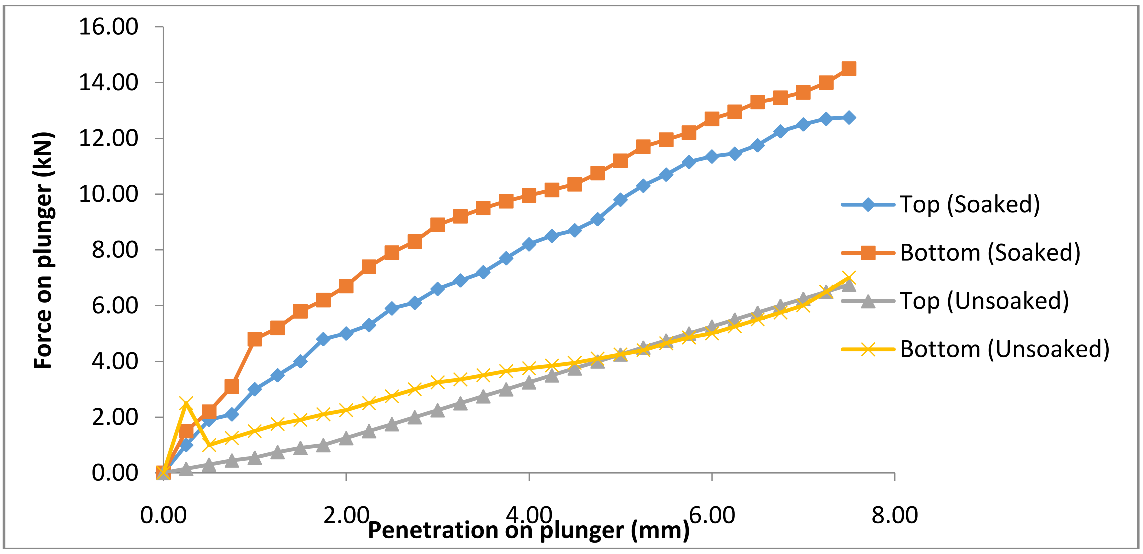


MTS 1-2 TP3

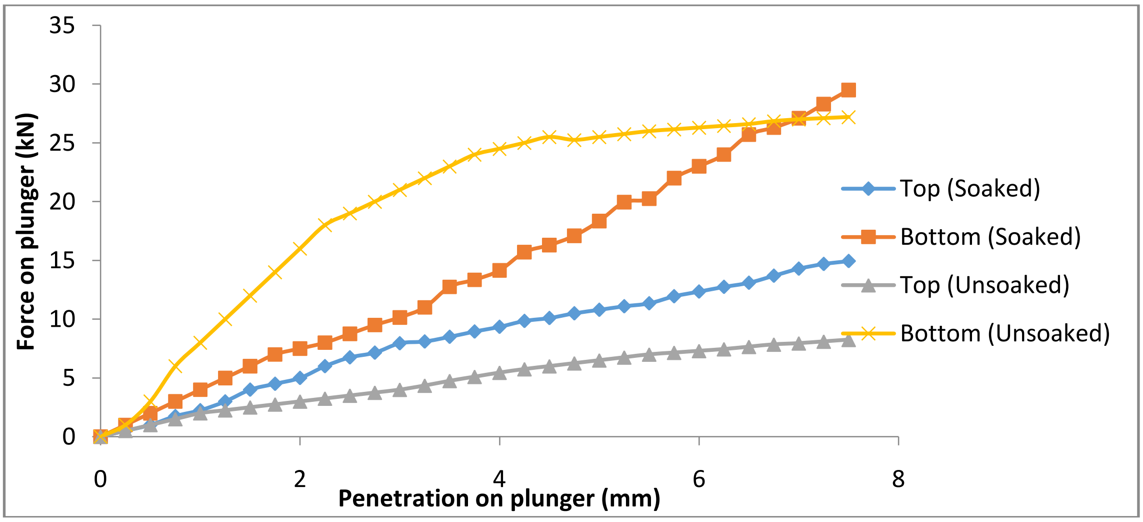
Figure B.2: Force against Penetration of Plunger of Soil Samples fromMTS 1-2



MTS 1-3 TP1

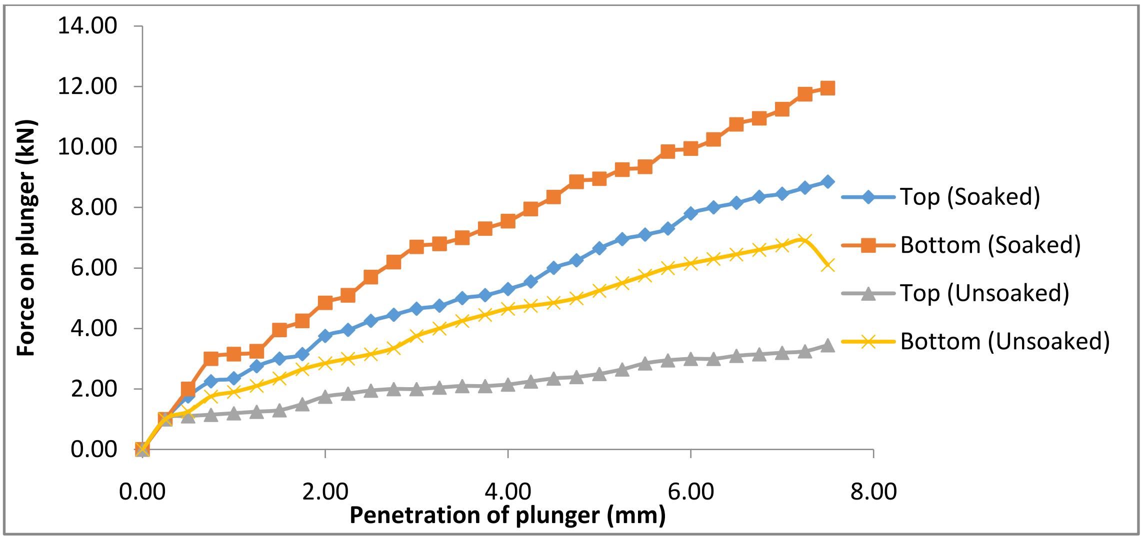


MTS 1-3 TP2

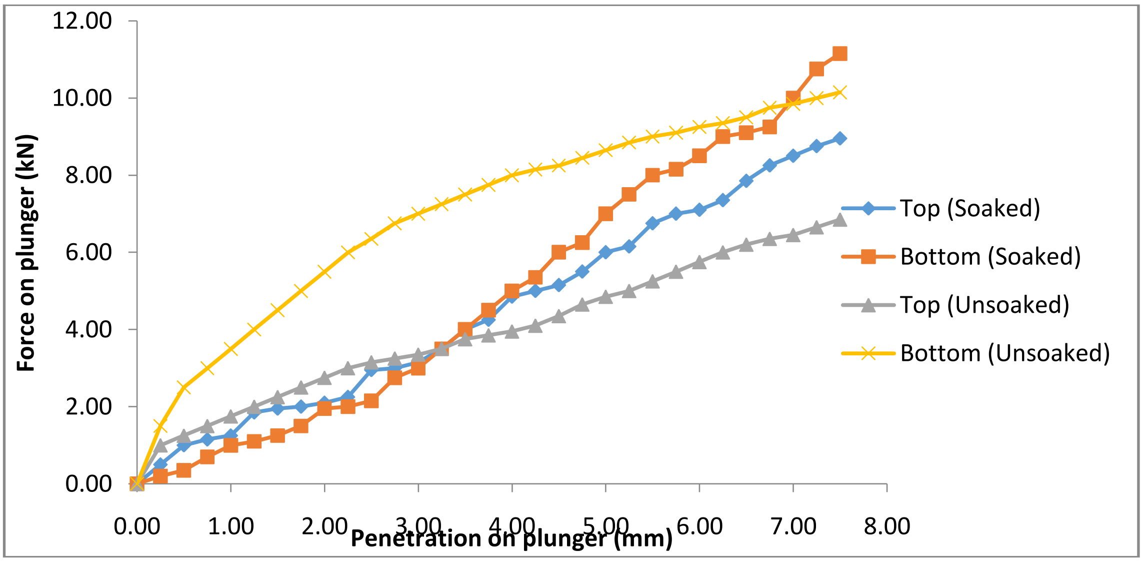


MTS 1-3 TP3

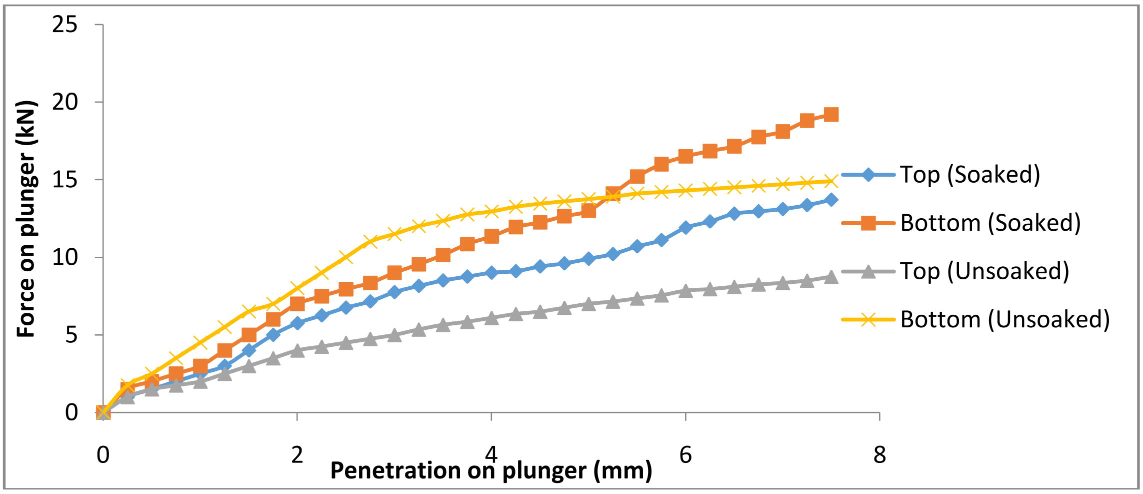
Figure B.3: Force against Penetration of Plunger of Soil Samples fromMTS 1-3



MTS 1-4 TP1

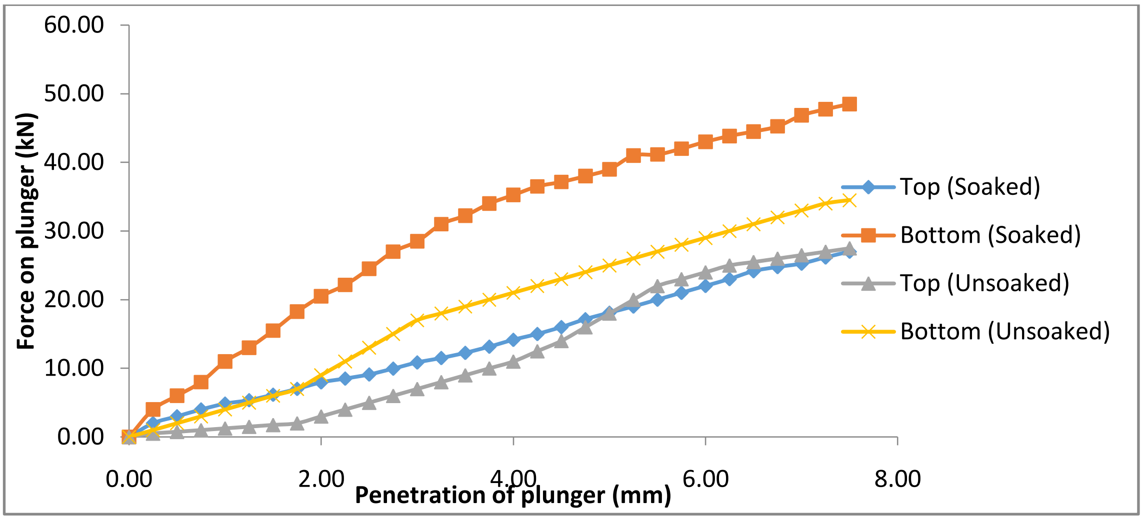


MTS 1-4 TP2

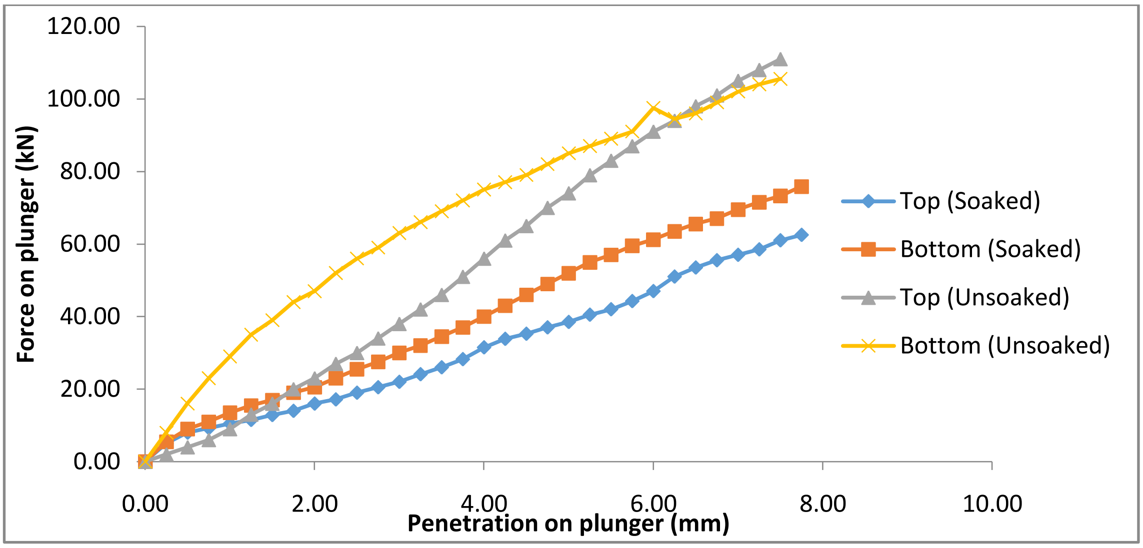


MTS 1-4 TP3

Figure B.4: Force against Penetration of Plunger of Soil Samples fromMTS 1-4



MTS 1-5 TP1



MTS 1-5 TP2

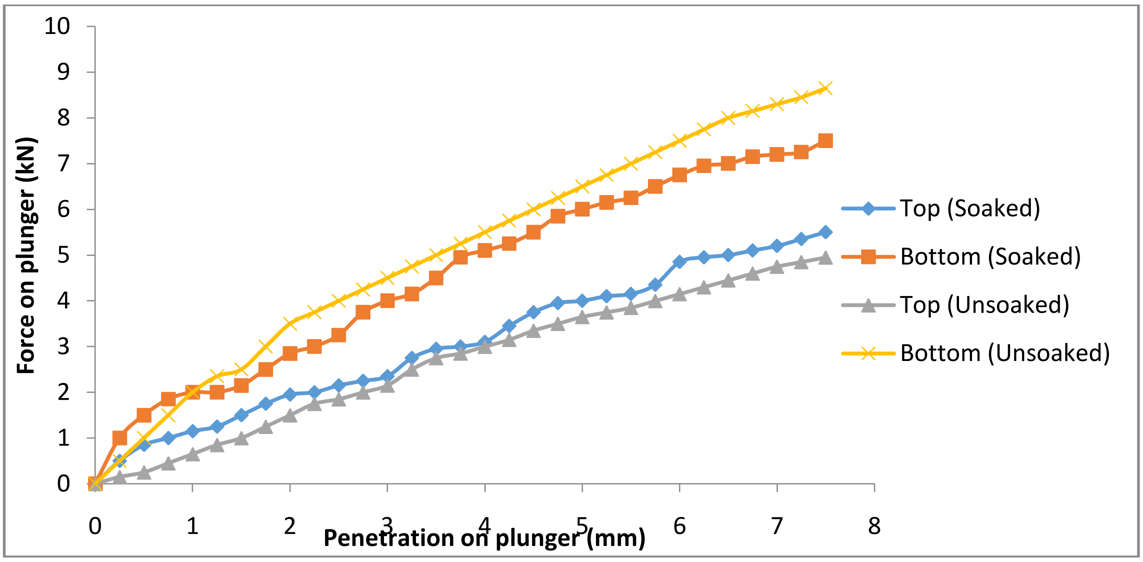
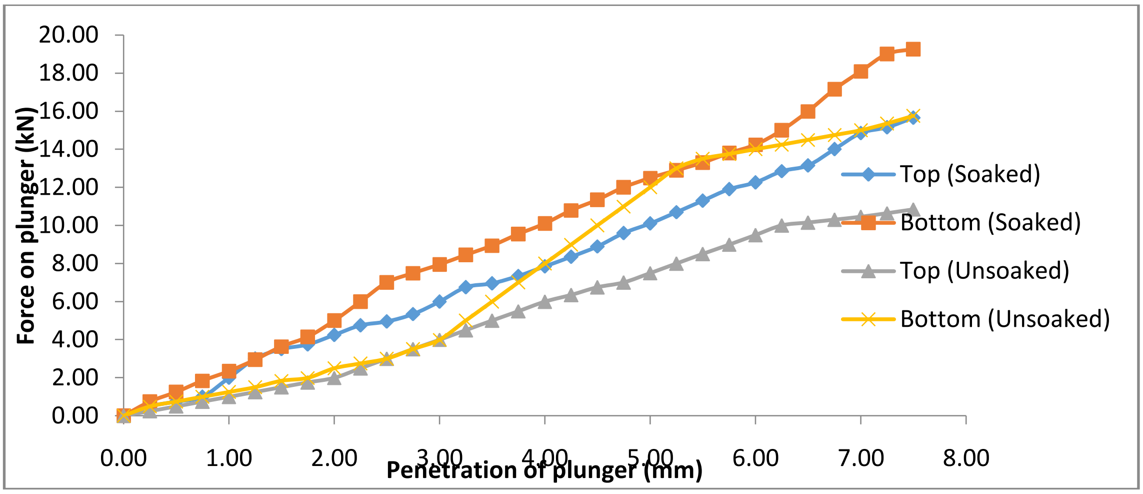
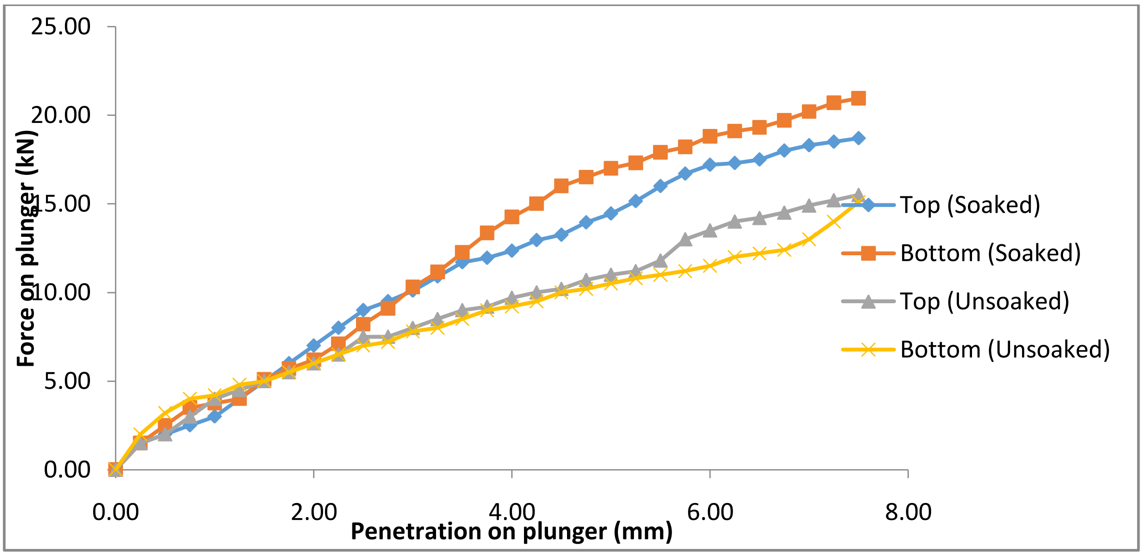


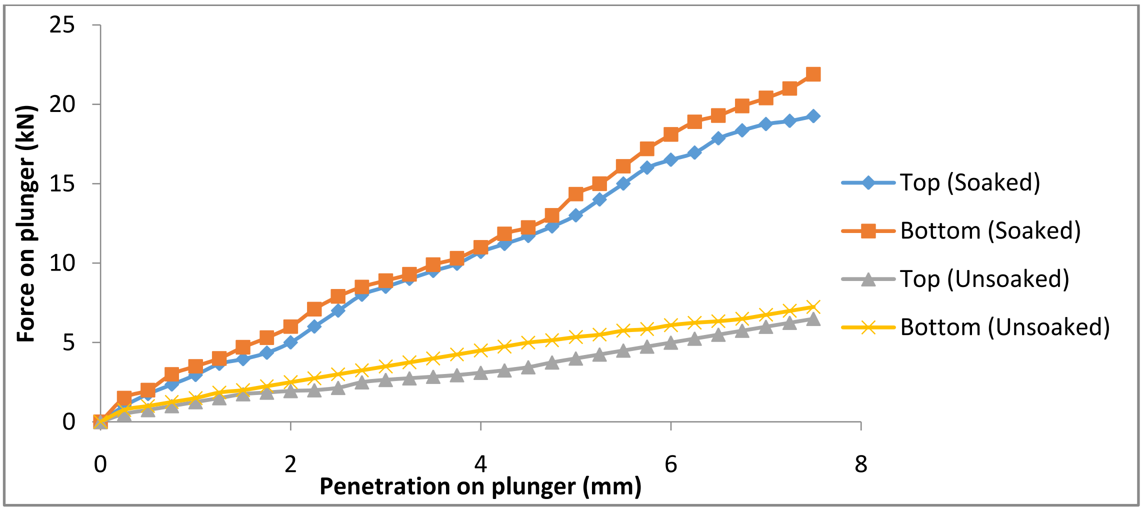
Figure B.5: Force against Penetration of Plunger of Soil Samples fromMTS 1-5



MTS 1-6 TP1



MTS 1-6 TP2



MTS 1-6 TP3

Figure B.6: Force against Penetration of Plunger of Soil Samples fromMTS 1-6

APPENDIX C

RESILIENT MODULUS MODEL PARAMETERS k_i OF NIGERIAN SOILS

Table C.1: Statistical Data for k_i of the Fine-GrainedSoil using Uzan's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	999.12	8.44E-17	0.0042	1.55E-16	9.89E-17	1.04E-16
2	1173.62	-1.66E-17	0.0057	1.25E-16	7.95E-17	8.40E-17
3	1632.72	-3.91E-17	0.0488	1.46E-16	9.29E-17	9.81E-17
4	749.44	1.56E-17	0.0600	1.59E-16	1.01E-16	1.07E-16
5	985.60	7.43E-17	0.0772	1.32E-16	8.38E-17	8.85E-17
6	1431.55	7.82E-18	0.0950	1.32E-16	8.39E-17	8.86E-17
7	1728.70	5.48E-17	0.0995	9.53E-17	6.07E-17	6.41E-17
8	1285.46	-1.10E-16	0.1023	1.51E-16	9.63E-17	1.02E-16
9	1850.62	-1.41E-16	0.1156	1.50E-16	9.55E-17	1.01E-16
10	1105.67	-8.61E-17	0.1273	1.15E-16	7.35E-17	7.76E-17
11	618.41	3.91E-17	0.1410	9.47E-17	6.03E-17	6.36E-17
12	1232.32	5.48E-17	0.1431	1.66E-16	1.06E-16	1.12E-16
13	1530.39	-7.82E-18	0.1432	1.05E-16	6.67E-17	7.03E-17
14	940.77	-7.82E-18	0.1489	1.48E-16	9.42E-17	9.95E-17
15	1166.21	2.35E-17	0.1500	1.41E-16	8.97E-17	9.47E-17
16	1097.93	-1.88E-16	0.1676	1.12E-16	7.14E-17	7.54E-17
17	1224.32	-3.91E-17	0.1702	1.17E-16	7.42E-17	7.84E-17
18	1639.35	-1.56E-17	0.1749	1.18E-16	7.49E-17	7.91E-17
19	913.28	-1.10E-16	0.1770	1.08E-16	6.84E-17	7.23E-17
20	1813.93	-1.56E-17	0.1855	1.74E-16	1.11E-16	1.17E-16
21	1526.92	-6.26E-17	0.1920	1.36E-16	8.67E-17	9.16E-17
22	1147.69	-1.56E-17	0.1941	1.52104E-16	9.67669E-17	1.02E-16
23	1208.67	-1.41E-16	0.2104	1.69709E-16	1.07967E-16	1.14E-16
24	1760.38	-3.13E-17	0.2117	1.5703E-16	9.99007E-17	1.06E-16
25	2098.37	-2.03E-16	0.2236	1.201E-16	7.64062E-17	8.07E-17
26	1179.33	-7.82E-17	0.2236	1.10263E-16	7.01478E-17	7.41E-17
27	1265.34	3.13E-17	0.2358	1.63503E-16	1.04019E-16	1.10E-16
28	1054.45	-1.72E-16	0.2410	1.33212E-16	8.47479E-17	8.95E-17
29	1973.66	1.25E-16	0.2453	1.5956E-16	1.0151E-16	1.07E-16
30	1023.68	-7.82E-17	0.2540	1.4859E-16	9.4531E-17	9.98E-17
31	2107.27	-1.10E-16	0.2675	9.80706E-17	6.23913E-17	6.59E-17
32	963.13	-3.13E-16	0.2884	1.37144E-16	8.7249E-17	9.21E-17
33	1360.06	-7.82E-17	0.3040	1.04249E-16	6.63216E-17	7.00E-17
34	1693.17	3.13E-17	0.3110	1.53407E-16	9.75953E-17	1.03E-16
35	1670.61	-1.56E-16	0.3171	1.10476E-16	7.02831E-17	7.42E-17
36	21.92	-7.82E-17	0.3209	6.51276E-17	4.14333E-17	4.38E-17
37	1833.91	-1.56E-16	0.3209	1.42648E-16	9.0751E-17	9.58E-17

38	1194.85	-1.56E-17	0.3409	9.3799E-17	5.96737E-17	6.30E-17
39	1834.07	-1.25E-16	0.3676	1.321E-16	8.40406E-17	8.88E-17
40	1229.24	-3.13E-17	0.3761	1.54423E-16	9.82419E-17	1.04E-16
41	957.01	0	0.4020	1.91468E-16	1.2181E-16	1.29E-16
42	1255.66	-1.56E-16	0.4042	1.04801E-16	6.66733E-17	7.04E-17
MEAN	**1320.92**	**-5.32E-17**	**0.2045**	**1.32917E-16**	**8.46E-17**	**8.93E-17**
MAX	**2107.27**	**1.25E-16**	**0.4042**	**1.91468E-16**	**1.22E-16**	**1.29E-16**
MIN	**21.92**	**-3.13E-16**	**0.0042**	**6.51276E-17**	**4.14E-17**	**4.38E-17**
STD DEV	**422.72**	**8.98E-17**	**0.1040**	**2.6807E-17**	**1.71E-17**	**1.80E-17**
TOTAL	**55478.79**	**-2.24E-15**	**8.5892**	**5.58252E-15**	**3.55E-15**	**3.75E-15**

Table C.2: Statistical Data for k_i of the Fine-Grained Soil using Witczak and Uzan's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	1002.302	8.56E-17	0.0042	2.2534E-16	9.9064E-17	1.05E-16
2	1178.680	-1.5E-17	0.0057	1.8077E-16	7.9470E-17	8.39E-17
3	1693.765	-2.7E-17	0.0488	2.1116E-16	9.2831E-17	9.80E-17
4	784.0354	3.13E-17	0.0600	2.2769E-16	1.001E-16	1.06E-16
5	1044.498	8.22E-17	0.0772	1.9175E-16	8.4297E-17	8.90E-17
6	1537.558	3.13E-17	0.0950	1.8356E-16	8.0695E-17	8.52E-17
7	1863.087	7.82E-17	0.0995	1.4478E-16	6.3648E-17	6.72E-17
8	1388.300	-8.6E-17	0.1023	2.1940E-16	9.6451E-17	1.02E-16
9	2018.723	-1.2E-16	0.1156	2.2699E-16	9.9790E-17	1.05E-16
10	1216.768	-4.7E-17	0.1273	1.6989E-16	7.4684E-17	7.89E-17
11	687.5839	8.61E-17	0.1410	1.4226E-16	6.2538E-17	6.60E-17
12	1372.339	9.39E-17	0.1431	2.3248E-16	1.0220E-16	1.08E-16
13	1704.411	3.13E-17	0.1432	1.5796E-16	6.9440E-17	7.33E-17
14	1052.207	3.13E-17	0.1489	2.2379E-16	9.8380E-17	1.04E-16
15	1305.468	6.26E-17	0.1500	2.1800E-16	9.5838E-17	1.01E-16
16	1245.456	-1.5E-16	0.1676	1.5308E-16	6.7297E-17	7.11E-17
17	1391.513	-2.4E-17	0.1702	1.8369E-16	8.0754E-17	8.53E-17
18	1869.800	1.56E-17	0.1749	1.6415E-16	7.2162E-17	7.62E-17
19	1043.331	-6.3E-17	0.1770	1.3173E-16	5.7909E-17	6.12E-17
20	2085.544	3.13E-17	0.1855	2.6927E-16	1.1837E-16	1.25E-16
21	1764.101	-1.6E-17	0.1920	1.8509E-16	8.1370E-17	8.59E-17
22	1328.059	0	0.1941	1.9981E-16	8.7840E-17	9.28E-17
23	1415.874	-7.8E-17	0.2104	2.5008E-16	1.0994E-16	1.16E-16
24	2064.140	1.56E-17	0.2117	2.2449E-16	9.8689E-17	1.04E-16
25	2482.536	-1.1E-16	0.2236	1.5939E-16	7.0072E-17	7.40E-17
26	1395.234	-1.6E-17	0.2236	1.6621E-16	7.3069E-17	7.72E-17
27	1510.801	9.39E-17	0.2358	2.2063E-16	9.6990E-17	1.02E-16
28	1263.977	-1.6E-16	0.2410	1.6680E-16	7.33E-17	7.74E-17
29	2373.429	1.88E-16	0.2453	2.4928E-16	1.10E-16	1.16E-16
30	1239.151	-1.6E-17	0.2540	2.0743E-16	9.12E-17	9.63E-17
31	2576.806	-4.7E-17	0.2675	1.1210E-16	4.93E-17	5.20E-17
32	1196.435	-2.7E-16	0.2884	2.0006E-16	8.79E-17	9.29E-17
33	1709.406	0	0.3040	1.8256E-16	8.03E-17	8.48E-17
34	2139.312	6.26E-17	0.3110	2.2472E-16	9.88E-17	1.04E-16
35	2120.576	-7.8E-17	0.3171	1.9128E-16	8.41E-17	8.88E-17
36	27.9011	0	0.3209	8.4853E-17	3.73E-17	3.94E-17
37	2334.462	-6.3E-17	0.3209	2.2998E-16	1.01E-16	1.07E-16
38	1544.071	4.69E-17	0.3409	1.5458E-16	6.80E-17	7.18E-17

39	2418.148	-1.3E-16	0.3676	1.7409E-16	7.65E-17	8.08E-17
40	1631.039	6.26E-17	0.3761	2.1102E-16	9.2768E-17	9.80E-17
41	1294.832	6.26E-17	0.4020	3.1753E-16	1.3959E-16	1.47E-16
42	1701.73	-6.3E-17	0.4042	1.61E-16	7.08E-17	7.48E-17
MEAN	**1548.03**	**-8.8E-18**	**0.2045**	**1.94E-16**	**8.51E-17**	**8.99E-17**
MAX	**2576.81**	**1.88E-16**	**0.4042**	**3.18E-16**	**1.40E-16**	**1.47E-16**
MIN	**27.90**	**-2.7E-16**	**0.0042**	**8.49E-17**	**3.73E-17**	**3.94E-17**
STD DEV	**527.61**	**8.52E-17**	**0.1040**	**4.33E-17**	**1.91E-17**	**2.01E-17**
TOTAL	**65017.38**	**-3.7E-16**	**8.5892**	**8.13E-15**	**3.57E-15**	**3.77E-15**

Table C.3: Statistical Data fork_i of the Fine-Grained Soilusing Ooi et al's model (A)

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	987.7555433	-0.00015	0.020155	0.000584	0.000877	0.001477
2	1155.607361	-0.000196	0.027254	0.000790	0.001185	0.001998
3	1430.842671	-0.001669	0.232564	0.006737	0.010115	0.017045
4	637.2044117	-0.00205	0.285881	0.008281	0.012434	0.020953
5	799.9530898	-0.00264	0.367738	0.010652	0.015994	0.026952
6	1107.276965	-0.003249	0.452597	0.013110	0.019685	0.033172
7	1320.728728	-0.003405	0.474324	0.013740	0.020630	0.034764
8	974.7116786	-0.003500	0.487617	0.014125	0.021208	0.035738
9	1353.792538	-0.00395	0.550844	0.015956	0.023958	0.040372
10	783.6226118	-0.00436	0.606639	0.017572	0.026384	0.044462
11	422.3689314	-0.00482	0.671819	0.019461	0.029219	0.049239
12	836.9041271	-0.00489	0.681825	0.019750	0.029655	0.049972
13	1039.018935	-0.00490	0.682349	0.019766	0.029677	0.050011
14	629.0277874	-0.00509	0.709270	0.020545	0.030848	0.051984
15	777.3633011	-0.00513	0.714702	0.020703	0.031084	0.052382
16	697.7652202	-0.00573	0.798750	0.023137	0.034740	0.058542
17	772.6996093	-0.00582	0.810996	0.023492	0.035273	0.059439
18	1021.595554	-0.00598	0.833342	0.024139	0.036244	0.061077
19	565.8603251	-0.00606	0.843491	0.024433	0.036686	0.061821
20	1098.331433	-0.00635	0.884038	0.025608	0.038449	0.064793
21	908.5345655	-0.00657	0.914818	0.026499	0.039788	0.067049
22	679.0212433	-0.00664	0.924824	0.026789	0.040223	0.067782
23	684.2659254	-0.00720	1.002488	0.029039	0.043601	0.073474
24	993.1922996	-0.00724	1.008539	0.029214	0.043864	0.073918
25	1146.459165	-0.00765	1.065144	0.030854	0.046326	0.078066
26	644.3325540	-0.00765	1.065144	0.030854	0.046326	0.078066
27	668.8729663	-0.00806	1.123320	0.032539	0.048856	0.082330
28	549.5542045	-0.00824	1.148287	0.033262	0.049942	0.084160
29	1016.842378	-0.00839	1.168585	0.033850	0.050825	0.085648
30	515.0902726	-0.00869	1.210228	0.035057	0.052636	0.088700
31	1022.369792	-0.00915	1.274456	0.036917	0.055430	0.093407
32	441.5406505	-0.00987	1.374276	0.039808	0.059771	0.100723
33	597.8015008	-0.01040	1.448462	0.041957	0.062998	0.106160
34	730.2627286	-0.01064	1.481814	0.042924	0.064448	0.108605
35	708.6682399	-0.01085	1.511070	0.043771	0.065721	0.110749
36	770.0740594	-0.01098	1.528985	0.044290	0.066500	0.112062
37	9.2038028	-0.01098	1.528985	0.044290	0.066500	0.112062
38	475.2629371	-0.01166	1.624469	0.047056	0.070653	0.119060
39	678.7410573	-0.01257	1.751590	0.050738	0.076182	0.128377

40	444.6339715	-0.01286	1.791852	0.051904	0.077933	0.131328
41	322.7245907	-0.01375	1.915400	0.055483	0.083306	0.140383
42	420.9124403	-0.01382	1.925930	0.055788	0.083764	0.141155
MEAN	**781.9236230**	**-0.00699**	**0.974401**	**0.028225**	**0.042379**	**0.071416**
MAX	**1430.842671**	**-0.00015**	**1.925930**	**0.055788**	**0.083764**	**0.141155**
MIN	**9.2038028**	**-0.01382**	**0.020155**	**0.000584**	**0.000877**	**0.001477**
STD DEV	**296.9852522**	**0.003557**	**0.495545**	**0.014354**	**0.021553**	**0.036319**
TOTAL	**32840.79**	**-0.29376**	**40.92486**	**1.185464**	**1.779937**	**2.999457**

Table C.4: Statistical Data for k_i of the Fine-Grained Soil Ooi et al's Model (B)

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	988.03367	-0.00011	0.036653	0.000638	0.000967	0.002984
2	1156.0474	-0.00015	0.049564	0.000862	0.001308	0.004035
3	1435.4985	-0.00126	0.422936	0.007356	0.011157	0.034432
4	639.75412	-0.00155	0.519897	0.009043	0.013715	0.042326
5	804.07291	-0.00200	0.668760	0.011632	0.017642	0.054446
6	1114.299605	-0.00246	0.823083	0.014316	0.021713	0.067009
7	1329.508579	-0.00258	0.862595	0.015004	0.022755	0.070226
8	981.3735182	-0.00265	0.886770	0.015424	0.023393	0.072194
9	1364.249659	-0.00299	1.001754	0.017424	0.026426	0.081556
10	790.2912579	-0.00329	1.103221	0.019189	0.029103	0.089816
11	426.3513157	-0.00365	1.221757	0.021251	0.032230	0.099467
12	844.9131185	-0.00370	1.239953	0.021567	0.032710	0.100948
13	1048.969801	-0.00371	1.240906	0.021584	0.032735	0.101026
14	635.2909504	-0.00385	1.289863	0.022435	0.034027	0.105011
15	785.1629968	-0.00388	1.299741	0.022607	0.034287	0.105816
16	705.5941921	-0.00434	1.452591	0.025266	0.038320	0.118260
17	781.5030155	-0.00440	1.474860	0.025653	0.038907	0.120072
18	1033.55722	-0.00453	1.515498	0.026360	0.039979	0.123381
19	572.56704	-0.00458	1.533955	0.026681	0.040466	0.124884
20	1111.978771	-0.00480	1.607693	0.027963	0.042411	0.130887
21	920.2191485	-0.00497	1.663669	0.028937	0.043888	0.135444
22	687.8502093	-0.00502	1.681865	0.029254	0.044368	0.136925
23	693.9154896	-0.00545	1.823104	0.031710	0.048094	0.148424
24	1007.283504	-0.00548	1.834108	0.031902	0.048384	0.149320
25	1163.6446	-0.00579	1.937048	0.033692	0.051100	0.157701
26	653.99111	-0.00579	1.937048	0.033692	0.051100	0.157701
27	679.45132	-0.00610	2.042847	0.035532	0.053891	0.166314
28	558.44024	-0.00624	2.088251	0.036322	0.055088	0.170010
29	1033.5773	-0.00635	2.125164	0.036964	0.056062	0.173016
30	523.87212	-0.00657	2.200895	0.038281	0.058060	0.179181
31	1040.7336	-0.00692	2.317699	0.040313	0.061141	0.188690
32	450.0987836	-0.00746	2.499229	0.043470	0.065930	0.203469
33	610.0201801	-0.00787	2.634143	0.045817	0.069489	0.214453
34	745.5360932	-0.00805	2.694797	0.046872	0.071089	0.219391
35	723.7856791	-0.00821	2.748000	0.047797	0.072493	0.223722
36	786.6982682	-0.00830	2.780580	0.048364	0.073352	0.226375
37	9.402492704	-0.00830	2.780580	0.048364	0.073352	0.226375
38	486.1708427	-0.00882	2.954226	0.051384	0.077933	0.240512
39	695.5530747	-0.00951	3.185406	0.055405	0.084032	0.259333

40	455.9036244	-0.00973	3.258625	0.056679	0.085963	0.265294
41	331.4759133	-0.01040	3.483307	0.060587	0.091890	0.283586
42	432.3899186	-0.01046	3.502456	0.060920	0.092395	0.285145
MEAN	**791.4055038**	**-0.00529**	**1.772026**	**0.030822**	**0.046746**	**0.144266**
MAX	**1435.498521**	**-0.00011**	**3.502456**	**0.060920**	**0.092395**	**0.285145**
MIN	**9.402492704**	**-0.01046**	**0.036653**	**0.000638**	**0.000967**	**0.002984**
STD DEV	**297.7293914**	**0.002692**	**0.901188**	**0.015675**	**0.023774**	**0.073368**
TOTAL	**33239.03116**	**-0.22229**	**74.42510**	**1.294513**	**1.963348**	**6.059156**

Table C.5: Statistical Data fork_i of the Fine-Grained Soil using Ni et al's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	987.72825	-0.00019	0.020049	0.000542	0.001703	0.001336
2	1155.5642	-0.00026	0.027111	0.000733	0.002303	0.001807
3	1430.3866	-0.00220	0.231341	0.006256	0.019650	0.015421
4	636.95475	-0.00271	0.284378	0.007690	0.024155	0.018956
5	799.54994	-0.00348	0.365804	0.009892	0.031072	0.024384
6	1106.5902	-0.00428	0.450217	0.012175	0.038242	0.030011
7	1319.8703	-0.00449	0.471830	0.012759	0.040078	0.031452
8	974.06037	-0.00461	0.485053	0.013117	0.041201	0.032333
9	1352.7707	-0.00521	0.547948	0.014817	0.046543	0.036526
10	782.97124	-0.00574	0.603449	0.016318	0.051258	0.040226
11	421.98014	-0.00636	0.668287	0.018072	0.056765	0.044548
12	836.12228	-0.00645	0.678241	0.018341	0.057611	0.045211
13	1038.0475	-0.00646	0.678762	0.018355	0.057655	0.045246
14	628.41650	-0.00671	0.705541	0.019079	0.059929	0.047031
15	776.60208	-0.00676	0.710944	0.019225	0.060388	0.047391
16	697.00163	-0.00756	0.794551	0.021486	0.067490	0.052964
17	771.84106	-0.00767	0.806732	0.021815	0.068525	0.053776
18	1020.4292	-0.00788	0.828961	0.022416	0.070413	0.055258
19	565.20642	-0.00798	0.839056	0.022689	0.071270	0.055931
20	1097.0012	-0.00836	0.879390	0.023780	0.074696	0.058620
21	907.39595	-0.00866	0.910008	0.024608	0.077297	0.060660
22	678.16096	-0.00875	0.919962	0.024877	0.078143	0.061324
23	683.32624	-0.00948	0.997217	0.026966	0.084705	0.066474
24	991.82015	-0.00954	1.003237	0.027129	0.085216	0.066875
25	1144.7864	-0.01008	1.059544	0.028652	0.089999	0.070628
26	643.39245	-0.01008	1.059544	0.028652	0.089999	0.070628
27	667.84379	-0.01063	1.117414	0.030217	0.094914	0.074486
28	548.68985	-0.01086	1.142250	0.030888	0.097024	0.076142
29	1015.2148	-0.01106	1.162441	0.031434	0.098739	0.077487
30	514.23645	-0.01145	1.203865	0.032554	0.102258	0.080249
31	1020.5852	-0.01206	1.267755	0.034282	0.107685	0.084508
32	440.70963	-0.01300	1.367050	0.036967	0.116119	0.091127
33	596.61571	-0.01370	1.440846	0.038963	0.122387	0.096046
34	728.78087	-0.01402	1.474024	0.039860	0.125205	0.098257
35	707.20184	-0.01430	1.503125	0.040647	0.127677	0.100197
36	768.46172	-0.01447	1.520946	0.041129	0.129191	0.101385
37	9.1845324	-0.01447	1.520946	0.041129	0.129191	0.101385
38	474.20579	-0.01537	1.615928	0.043697	0.137259	0.107717
39	677.11330	-0.01657	1.742382	0.047117	0.148000	0.116146

40	443.54317	-0.01695	1.782431	0.048200	0.151402	0.118816
41	321.87834	-0.01812	1.905330	0.051523	0.161841	0.127008
42	419.80267	-0.01822	1.915804	0.051806	0.162731	0.127706
MEAN	**781.00103**	**-0.00922**	**0.969278**	**0.026211**	**0.082332**	**0.064611**
MAX	**1430.3866**	**-0.00019**	**1.915804**	**0.051806**	**0.162731**	**0.127706**
MIN	**9.1845324**	**-0.01822**	**0.020049**	**0.000542**	**0.001703**	**0.001336**
STD DEV	**296.91487**	**0.004688**	**0.492940**	**0.013330**	**0.041871**	**0.032859**
TOTAL	**32802.044**	**-0.38717**	**40.70969**	**1.100849**	**3.457926**	**2.713676**

Table C.6: Statistical Data for k_i of the Fine-Grained Soilusing NCHRP's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	988.0236255	4.00E-05	0.036343	0.000564	0.000319	0.003003
2	1156.031497	5.41E-05	0.049145	0.000763	0.000432	0.004060
3	1435.330126	0.000461	0.419363	0.006513	0.003682	0.034649
4	803.923763	0.000729	0.663111	0.010299	0.005823	0.054788
5	803.923763	0.000729	0.663111	0.010299	0.005823	0.054788
6	1114.04523	0.000898	0.816130	0.012676	0.007167	0.067431
7	1114.04523	0.000898	0.816130	0.012676	0.007167	0.067431
8	1329.190507	0.000941	0.855309	0.013284	0.007511	0.070668
9	981.1321552	0.000967	0.879280	0.013657	0.007721	0.072649
10	981.1321552	0.000967	0.879280	0.013657	0.007721	0.072649
11	1363.870629	0.001092	0.993292	0.015427	0.008722	0.082069
12	790.0494548	0.001203	1.093901	0.016990	0.009606	0.090382
13	844.6225681	0.001352	1.229479	0.019096	0.010796	0.101583
14	1048.608802	0.001353	1.230424	0.019110	0.010805	0.101662
15	635.0636933	0.001407	1.278968	0.019864	0.011231	0.105672
16	705.3099492	0.001584	1.440321	0.022370	0.012648	0.119003
17	781.183368	0.001608	1.462402	0.022713	0.012842	0.120828
18	1033.122832	0.001653	1.502697	0.023339	0.013195	0.124158
19	572.3234692	0.001673	1.520997	0.023623	0.013356	0.125670
20	1111.482999	0.001753	1.594113	0.024759	0.013998	0.131711
21	1111.482999	0.001753	1.594113	0.024759	0.013998	0.131711
22	687.5293885	0.001834	1.667659	0.025901	0.014644	0.137787
23	693.5646676	0.001988	1.807704	0.028076	0.015874	0.149358
24	1006.771179	0.002000	1.818616	0.028246	0.015970	0.150260
25	1163.01955	0.002112	1.920686	0.029831	0.016866	0.158693
26	653.6398155	0.002112	1.920686	0.029831	0.016866	0.158693
27	679.0664222	0.002228	2.025591	0.031460	0.017787	0.167361
28	1032.968157	0.002318	2.107212	0.032728	0.018504	0.174105
29	1032.968157	0.002318	2.107212	0.032728	0.018504	0.174105
30	1040.064772	0.002528	2.298121	0.035693	0.020180	0.189878
31	449.7868643	0.002725	2.478118	0.038489	0.021761	0.204750
32	723.2341863	0.002997	2.724787	0.042320	0.023927	0.225131
33	723.2341863	0.002997	2.724787	0.042320	0.023927	0.225131
34	786.0917346	0.003032	2.757092	0.042822	0.024210	0.227800
35	9.395243511	0.003032	2.757092	0.042822	0.024210	0.227800
36	485.7726133	0.003222	2.929271	0.045496	0.025722	0.242026
37	694.9387727	0.003474	3.158499	0.049056	0.027735	0.260965
38	455.4917262	0.003554	3.231099	0.050184	0.028373	0.266964
39	431.9700474	0.003820	3.472871	0.053939	0.030496	0.286940

MEAN	**857.7796487**	**0.001831**	**1.664744**	**0.025856**	**0.014618**	**0.137546**
MAX	**1435.330126**	**0.003820**	**3.472871**	**0.053939**	**0.030496**	**0.286940**
MIN	**9.395243511**	**4.00E-05**	**0.036343**	**0.000564**	**0.000319**	**0.003003**
STDEV	**291.1603892**	**0.000970**	**0.881792**	**0.013696**	**0.007743**	**0.072856**
TOTAL	**33453.4063**	**0.071406**	**64.92501**	**1.008381**	**0.570117**	**5.364310**

Table C.7: Statistical Data fork_i of the Coarse-Grained Soil using Uzan's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	1028.434861	6.28E-17	0.11807	1.72E-16	1.24E-16	5.26E-17
2	1079.459255	1.11E-16	0.07061	1.81E-16	1.30E-16	5.53E-17
3	1386.746395	-4.4E-17	0.259	2.41E-16	1.74E-16	7.36E-17
4	1227.40983	-1.4E-16	0.211	1.97E-16	1.42E-16	6.02E-17
5	884.6112598	1.77E-16	0.24967	2.35E-16	1.70E-16	7.20E-17
6	1163.925413	1.62E-16	0.2019	2.05E-16	1.48E-16	6.26E-17
7	1820.249703	1.54E-16	0.23137	1.40E-16	1.01E-16	4.28E-17
8	1365.814675	-2.3E-17	0.309	1.69E-16	1.22E-16	5.16E-17
9	1844.694986	-1.7E-16	0.27	1.89E-16	1.37E-16	5.79E-17
10	1267.075112	8.33E-17	0.26437	2.15E-16	1.55E-16	6.59E-17
11	1168.952527	2.44E-17	0.02954	1.85E-16	1.34E-16	5.67E-17
12	7551.295563	3.49E-16	0.38127	2.84E-16	2.05E-16	8.71E-17
13	736.1216769	-2.1E-16	0.18097	1.84E-16	1.33E-16	5.63E-17
14	657.895644	1.62E-17	1.10136	2.19E-16	1.58E-16	6.69E-17
15	873.4449307	-2.8E-17	0.41169	1.70E-16	1.23E-16	5.21E-17
16	1365.982742	-3.0E-16	0.27467	1.72E-16	1.24E-16	5.27E-17
17	854.8362832	2.07E-16	0.21784	1.66E-16	1.20E-16	5.08E-17
18	1128.264739	7.98E-17	0.26949	1.83E-16	1.32E-16	5.59E-17
MEAN	**1522.511977**	**2.84E-17**	**0.280657**	**1.95E-16**	**1.41E-16**	**5.96E-17**
MAX	**7551.295563**	**3.49E-16**	**1.10136**	**2.84E-16**	**2.05E-16**	**8.71E-17**
MIN	**657.895644**	**-3.0E-16**	**0.02954**	**1.40E-16**	**1.01E-16**	**4.28E-17**
STD DEV	**1538.675842**	**1.63E-16**	**0.225519**	**3.40E-17**	**2.45E-17**	**1.04E-17**
TOTAL	**27405.21559**	**5.11E-16**	**5.05182**	**3.51E-15**	**2.53E-15**	**1.07E-15**

Table C.8: Statistical Data for k_i of the Coarse-Grained Soil using Witczak and Uzan's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	1123.92981	6.40E-17	0.11807	2.10E-16	1.31E-16	5.53E-17
2	1138.32932	1.14E-16	0.07061	2.09E-16	1.30E-16	5.52E-17
3	1684.954066	-7.7E-17	0.259	2.45E-16	1.53E-16	6.47E-17
4	1438.478971	-1.4E-16	0.211	2.33E-16	1.45E-16	6.15E-17
5	1067.323931	1.87E-16	0.24967	2.95E-16	1.84E-16	7.78E-17
6	1354.774314	1.64E-16	0.2019	2.23E-16	1.39E-16	5.90E-17
7	2166.196543	1.09E-16	0.23137	1.70E-16	1.06E-16	4.48E-17
8	1723.110418	-6.6E-18	0.309	2.21E-16	1.38E-16	5.84E-17
9	2259.999051	-1.6E-16	0.27	2.37E-16	1.47E-16	6.24E-17
10	1545.778479	1.06E-16	0.26437	2.50E-16	1.55E-16	6.59E-17
11	1195.211665	2.72E-17	0.02954	2.15E-16	1.34E-16	5.67E-17
12	10058.81074	3.53E-16	0.38127	3.55E-16	2.21E-16	9.37E-17
13	843.4426171	-1.9E-16	0.18097	1.90E-16	1.18E-16	5.00E-17
14	1506.149486	-2.8E-17	1.10136	3.72E-16	2.31E-16	9.81E-17
15	1190.408608	-4.7E-18	0.41169	2.06E-16	1.28E-16	5.44E-17
16	1679.400016	-3.0E-16	0.27467	2.12E-16	1.32E-16	5.60E-17
17	1007.003209	2.22E-16	0.21784	2.01E-16	1.25E-16	5.31E-17
18	1381.745745	8.88E-17	0.26949	2.19E-16	1.36E-16	5.78E-17
MEAN	**1909.169277**	**2.96E-17**	**0.280657**	**2.37E-16**	**1.47E-16**	**6.25E-17**
MAX	**10058.81074**	**3.53E-16**	**1.10136**	**3.72E-16**	**2.31E-16**	**9.81E-17**
MIN	**843.4426171**	**-3.0E-16**	**0.02954**	**1.70E-16**	**1.06E-16**	**4.48E-17**
STD DEV	**2068.53983**	**1.62E-16**	**0.225519**	**5.32E-17**	**3.31E-17**	**1.40E-17**
TOTAL	**34365.04699**	**5.33E-16**	**5.05182**	**4.26E-15**	**2.65E-15**	**1.12E-15**

Table C.9: Statistical Data for k_i of the Coarse-Grained Soil using Pezo's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	1028.43	4.14E-17	0.11807	7.72E-17	1.04E-16	4.85E-17
2	1079.46	6.46E-17	0.07061	8.17E-17	1.11E-16	5.13E-17
3	1386.75	-4.2E-17	0.259	1.08E-16	1.46E-16	6.76E-17
4	1227.41	-1.2E-16	0.211	8.81E-17	1.19E-16	5.53E-17
5	884.611	1.05E-16	0.24967	1.07E-16	1.44E-16	6.71E-17
6	1163.93	1.51E-16	0.2019	9.09E-17	1.23E-16	5.71E-17
7	1820.25	1.37E-16	0.23137	6.21E-17	8.40E-17	3.90E-17
8	1365.81	-1.9E-17	0.309	7.55E-17	1.02E-16	4.74E-17
9	1844.69	-1.7E-16	0.27	8.32E-17	1.13E-16	5.23E-17
10	1267.08	6.50E-17	0.26437	9.65E-17	1.31E-16	6.06E-17
11	1168.95	1.54E-17	0.02954	8.31E-17	1.12E-16	5.22E-17
12	7551.3	2.80E-16	0.38127	1.28E-16	1.73E-16	8.05E-17
13	736.122	-1.8E-16	0.18097	8.19E-17	1.11E-16	5.15E-17
14	657.896	-2.3E-17	1.10136	9.79E-17	1.32E-16	6.15E-17
15	873.445	-5.5E-17	0.41169	7.57E-17	1.02E-16	4.76E-17
16	1365.98	-2.2E-16	0.27467	8.01E-17	1.08E-16	5.03E-17
17	854.836	1.81E-16	0.21784	7.38E-17	9.99E-17	4.64E-17
18	1128.26	8.67E-17	0.26949	8.12E-17	1.10E-16	5.10E-17
MEAN	**1522.51**	**1.66E-17**	**0.28066**	**8.73E-17**	**1.18E-16**	**5.48E-17**
MAX	**7551.3**	**2.80E-16**	**1.10136**	**1.28E-16**	**1.73E-16**	**8.05E-17**
MIN	**657.896**	**-2.2E-16**	**0.02954**	**6.21E-17**	**8.40E-17**	**3.90E-17**
STD DEV	**1538.68**	**1.34E-16**	**0.22552**	**1.54E-17**	**2.09E-17**	**9.70E-18**
TOTAL	**27405.2**	**2.99E-16**	**5.05182**	**1.57E-15**	**2.13E-15**	**9.87E-16**

Table C.10: Statistical Data for k_i of the Coarse-Grained Soil using Ooi et al's Model (A)

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	780.6010173	-0.01483	0.420048	0.037903	0.029517	0.032512
2	915.3621383	-0.00887	0.251203	0.022667	0.017652	0.019443
3	757.3852986	-0.03253	0.921423	0.083144	0.064748	0.071319
4	749.8794677	-0.02650	0.750658	0.067735	0.052749	0.058102
5	493.7815514	-0.03136	0.888231	0.080149	0.062416	0.068750
6	726.3673585	-0.02536	0.718283	0.064814	0.050474	0.055596
7	1060.408972	-0.02906	0.823126	0.074274	0.057841	0.063711
8	663.7448152	-0.03881	1.099304	0.099195	0.077248	0.085088
9	981.9471	-0.03391	0.960557	0.086675	0.067498	0.074348
10	683.401429	-0.03320	0.940528	0.084868	0.066091	0.072798
11	1091.031112	-0.00371	0.105092	0.009483	0.007385	0.008134
12	3099.801895	-0.04789	1.356413	0.122395	0.095315	0.104988
13	482.4009713	-0.02273	0.643822	0.058095	0.045241	0.049833
14	50.25133467	-0.13832	3.918219	0.353559	0.275333	0.303275
15	333.9610025	-0.05171	1.464636	0.132161	0.102920	0.113365
16	719.2375948	-0.03450	0.977171	0.088175	0.068666	0.075634
17	513.9816593	-0.02736	0.774992	0.069931	0.054459	0.059985
18	601.3008072	-0.03385	0.958743	0.086512	0.067371	0.074208
MEAN	**816.9358625**	**-0.03525**	**0.998469**	**0.090096**	**0.070162**	**0.077283**
MAX	**3099.801895**	**-0.00371**	**3.918219**	**0.353559**	**0.275333**	**0.303275**
MIN	**50.25133467**	**-0.13832**	**0.105092**	**0.009483**	**0.007385**	**0.008134**
STD DEV	**624.3539076**	**0.028324**	**0.802310**	**0.072396**	**0.056378**	**0.062100**
TOTAL	**14704.84553**	**-0.63448**	**17.97245**	**1.621736**	**1.262924**	**1.391092**

Table C.11: Statistical Data fork_i of the Coarse-Grained Soilusing Ooi et al's Model (B)

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	791.5766739	-0.01484	0.702227	0.043616	0.033927	0.06337085
2	923.0375202	-0.00888	0.419956	0.026084	0.020289	0.03789800
3	780.941805	-0.03255	1.540415	0.095676	0.074423	0.13901117
4	768.8260054	-0.02652	1.254933	0.077945	0.060630	0.11324848
5	508.5779043	-0.03138	1.484924	0.092230	0.071742	0.13400355
6	743.9188467	-0.02538	1.200810	0.074583	0.058015	0.10836431
7	1089.82349	-0.02908	1.376084	0.085469	0.066483	0.12418153
8	688.4475487	-0.03884	1.837792	0.114146	0.088790	0.16584731
9	1013.805946	-0.03394	1.605838	0.099740	0.077583	0.14491512
10	705.1044859	-0.03323	1.572353	0.097660	0.075966	0.14189337
11	1094.849085	-0.00371	0.175691	0.010912	0.008488	0.01585479
12	3242.763832	-0.04792	2.267622	0.140843	0.109556	0.20463625
13	492.8360703	-0.02275	1.076328	0.066851	0.052001	0.09713070
14	57.24157619	-0.13843	6.550392	0.406849	0.316471	0.59112488
15	350.6222373	-0.05175	2.448546	0.152081	0.118297	0.22096336
16	742.983152	-0.03452	1.633613	0.101465	0.078925	0.14742162
17	527.3944015	-0.02738	1.295614	0.080471	0.062595	0.11691967
18	620.7723097	-0.03387	1.602805	0.099551	0.077437	0.14464139
MEAN	**841.3068272**	**-0.03528**	**1.669219**	**0.103676**	**0.080646**	**0.15063480**
MAX	**3242.763832**	**-0.00371**	**6.550392**	**0.406849**	**0.316471**	**0.59112488**
MIN	**57.24157619**	**-0.13843**	**0.175691**	**0.010912**	**0.008488**	**0.01585479**
STD DEV	**652.4506685**	**0.028346**	**1.341285**	**0.083308**	**0.064802**	**0.12104113**
TOTAL	**15143.52289**	**-0.63497**	**30.04594**	**1.866171**	**1.451619**	**2.71142633**

Table C.12: Statistical Data fork_i of theCoarse-Grained Soilusing Ni et al's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	776.50712	-0.02127	0.414334	0.028443	0.042120	0.0291621
2	912.48815	-0.01272	0.247786	0.017010	0.025189	0.0174400
3	748.69923	-0.04665	0.908888	0.062392	0.092395	0.0639704
4	742.86582	-0.03800	0.740445	0.050829	0.075271	0.0521149
5	488.32148	-0.04497	0.876147	0.060145	0.089066	0.0616660
6	719.86530	-0.03637	0.708512	0.048637	0.072025	0.0498673
7	1049.5384	-0.04167	0.811928	0.055736	0.082538	0.0571461
8	654.67321	-0.05566	1.084349	0.074437	0.110231	0.0763199
9	970.21022	-0.04863	0.947489	0.065042	0.096319	0.0666873
10	675.40229	-0.04762	0.927732	0.063686	0.094310	0.0652968
11	1089.5967	-0.00532	0.103662	0.007116	0.010538	0.0072961
12	3047.6111	-0.06867	1.337960	0.091847	0.136013	0.0941699
13	478.52862	-0.03260	0.635064	0.043595	0.064559	0.0446978
14	47.845983	-0.19837	3.864914	0.265314	0.392894	0.2720250
15	327.89364	-0.07415	1.444711	0.099175	0.146865	0.1016834
16	710.49301	-0.04947	0.963877	0.066167	0.097985	0.0678408
17	509.01929	-0.03924	0.764448	0.052477	0.077711	0.0538043
18	594.12716	-0.04854	0.945700	0.064919	0.096137	0.0665614
MEAN	**807.98260**	**-0.05055**	**0.984886**	**0.067609**	**0.100120**	**0.0693194**
MAX	**3047.6111**	**-0.00532**	**3.864914**	**0.265314**	**0.392894**	**0.2720250**
MIN	**47.845983**	**-0.19837**	**0.103662**	**0.007116**	**0.010538**	**0.0072961**
STD DEV	**614.14962**	**0.040620**	**0.791396**	**0.054327**	**0.080451**	**0.0557009**
TOTAL	**14543.687**	**-0.90991**	**17.72795**	**1.216968**	**1.802164**	**1.24774959**

Table C.13: Statistical Data fork_i of the Coarse-Grained Soilusing NCHRP's Model

	Regression Coefficients			Standard Error		
S/N	k_1	k_2	k_3	k_1	k_2	k_3
1	782.9502833	-0.00841	0.699412	0.025197	0.024014	0.06311829
2	917.0086328	-0.00503	0.418273	0.015069	0.014361	0.03774695
3	762.394398	-0.01844	1.534241	0.055272	0.052678	0.13845717
4	753.917329	-0.01502	1.249903	0.045029	0.042915	0.11279715
5	496.9292463	-0.01777	1.478973	0.053281	0.050780	0.13346950
6	730.1094963	-0.01437	1.195997	0.043087	0.041064	0.10793244
7	1066.671801	-0.01647	1.370569	0.049376	0.047058	0.12368662
8	668.9853946	-0.02200	1.830426	0.065943	0.062847	0.16518635
9	988.7181455	-0.01922	1.599402	0.057620	0.054915	0.14433759
10	688.0152505	-0.01882	1.566051	0.056418	0.053770	0.14132788
11	1091.851694	-0.00210	0.174986	0.006304	0.006008	0.01579160
12	3130.028284	-0.02714	2.258533	0.081366	0.077546	0.20382071
13	484.6280016	-0.01288	1.072014	0.038620	0.036807	0.09674361
14	51.67986285	-0.07840	6.524137	0.235038	0.224004	0.58876906
15	337.4786649	-0.02931	2.438732	0.087858	0.083733	0.22008275
16	724.2832008	-0.01955	1.627065	0.058616	0.055865	0.14683409
17	516.8392535	-0.01551	1.290421	0.046489	0.044306	0.11645371
18	605.4392361	-0.01918	1.596380	0.057511	0.054811	0.14406495
MEAN	**822.1071208**	**-0.01998**	**1.662529**	**0.059894**	**0.057082**	**0.15003447**
MAX	**3130.028284**	**-0.00210**	**6.524137**	**0.235038**	**0.224004**	**0.58876906**
MIN	**51.67986285**	**-0.07840**	**0.174986**	**0.006304**	**0.006008**	**0.01579160**
STD DEV	**630.2771324**	**0.016054**	**1.335909**	**0.048127**	**0.045868**	**0.12055874**
TOTAL	**0.766660593**	**-0.80354**	**0.803540**	**0.803540**	**0.803540**	**0.80354028**

APPENDIX D

REGRESSION ANALYSIS OF RESILIENT MODULUS PARAMETERS

Table D.1: Regression Analysis Output of k_1 for Model 1

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.99997929
R Square	0.99995858
Adjusted R Square	0.99966867
Standard Error	2.12295453
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	14	217630.611	15545.04	3449.1379	0.00028988
Residual	2	9.01387183	4.5069359		
Total	16	217639.624			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	97782.5176	1532.577679	63.80265	0.000246	91188.368	104376.67	91188.3681	104376.667
P200 (%)	-86.677399	1.537224116	-56.38566	0.000314	-93.291540	-80.06326	-93.2915403	-80.0632572
P4 (%)	-14.764959	0.422482608	-34.94809	0.000818	-16.582755	-12.94716	-16.5827549	-12.9471630
P40 (%)	-25.400135	0.319544433	-79.48859	0.000158	-26.775024	-24.02525	-26.7750235	-24.0252461
Clay (%)	100.542457	1.489796442	67.48738	0.000220	94.132380	106.95253	94.1323804	106.952534
Silt (%)	105.529504	1.556063006	67.81827	0.000217	98.834305	112.22470	98.8343048	112.224702
PL	57.4900331	0.702317719	81.85759	0.000149	54.468204	60.511862	54.4682039	60.5118624
wc (%)	-8.2568408	1.833716175	-4.502791	0.045949	-16.146685	-0.366997	-16.1466847	-0.36699692
OMC (%)	-301.82085	4.013255522	-75.20599	0.000177	-319.08849	-284.5532	-319.088494	-284.553205

ys (kg/m3)	39.2703561	0.660326338	59.47113	0.000283	36.429201	42.111511	36.4292012	42.1115111
MDD kg/m3	-55.926325	0.907002113	-61.66063	0.000263	-59.828841	-52.02381	-59.8288405	-52.0238103
wc/OMC	12541.0184	224.7001812	55.81223	0.000321	11574.212	13507.825	11574.2115	13507.8252
ys/MDD	-64152.323	1058.093771	-60.63009	0.000272	-68704.933	-59599.71	-68704.9328	-59599.7127
P200/wc	21.1872435	0.211580038	100.1382	9.97E-05	20.276888	22.097599	20.2768881	22.0975990
(wc/OMC)*(ys/MDD	-9865.3090	180.6634732	-54.60600	0.000335	-10642.641	-9087.977	-10642.6412	-9087.97682

RESIDUAL OUTPUT

Observation	*Predicted k1*	*Residuals*
1	1054.44680	-1.088911113
2	1052.85362	0.504273731
3	1053.77061	-0.412716716
4	1052.51822	0.83966977
5	1053.48423	-0.126340194
6	1052.38252	0.9753759
7	1053.48335	-0.125455453
8	943.254533	-0.082490718
9	781.502669	-0.319300943
10	779.714921	1.468447049
11	781.555310	-0.371941878
12	782.230305	-1.046937061
13	780.431800	0.751567697
14	917.623087	-0.052903273
15	917.589902	-0.01971885
16	918.806896	-1.236712491
17	917.226089	0.344094542

Table D.2: Regression Analysis Output of k_1 for Model 2

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.98692751
R Square	0.97402592
Adjusted R Square	0.89610366
Standard Error	37.5931839
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	12	211986.635	17665.5529	12.4999713	0.01298788
Residual	4	5652.98989	1413.24747		
Total	16	217639.624			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	63459.6037	20398.9456	3.11092569	0.03583977	6823.05114	120096.156	6823.05114	120096.156
P200 (%)	-62.1077569	24.0122808	-2.58649969	0.06090828	-128.776536	4.56102256	-128.776536	4.56102256
P40 (%)	-26.5276578	4.90030501	-5.41347073	0.00564143	-40.1330856	-12.9222299	-40.1330856	-12.9222299
Clay (%)	74.9751926	23.0145138	3.25773524	0.03114732	11.0766583	138.873727	11.0766583	138.873727
Silt (%)	77.0187293	23.5645192	3.26841929	0.03083505	11.5931352	142.444323	11.5931352	142.444323
PL	45.0021122	10.7198871	4.19800245	0.01371784	15.2389342	74.7652902	15.2389342	74.7652902
OMC (%)	-195.941878	47.3143622	-4.14127695	0.01436162	-327.307607	-64.5761484	-327.307607	-64.5761484
ys (kg/m3)	25.7918452	9.19365248	2.80539701	0.04854388	0.26617376	51.3175166	0.26617376	51.3175166
MDD kg/m3	-36.2635212	12.3025751	-2.94763665	0.04206813	-70.4209456	-2.10609678	-70.4209456	-2.10609678
wc/OMC	7707.48304	3072.27080	2.50872516	0.06614729	-822.508175	16237.4743	-822.508175	16237.4743
ys/MDD	-42351.9665	14622.7324	-2.89631004	0.04428153	-82951.1802	-1752.75278	-82951.1802	-1752.75278

P200/wc	15.6033147	2.48941567	6.26786233	0.00330635	8.69158874	22.5150406	8.69158874	22.5150406
(wc/OMC)*(ys/MDD	-6140.76797	2596.60436	-2.36492246	0.07724716	-13350.0974	1068.56149	-13350.0974	1068.56149

RESIDUAL OUTPUT

Observation	*Predicted k1*	*Residuals*
1	1046.01751	7.340376838
2	1063.75661	-10.3987173
3	1056.76681	-3.40891504
4	1040.21184	13.14605515
5	1007.93979	45.41810604
6	1076.39957	-23.0416746
7	1045.11201	8.245881971
8	949.402787	-6.23074452
9	771.204280	9.979087765
10	776.297375	4.88599342
11	786.278234	-5.09486630
12	812.087964	-30.9045958
13	765.529545	15.65382257
14	951.405521	-33.8353377
15	918.739273	-1.16908976
16	907.121240	10.44894345
17	918.604510	-1.03432607

Table D.3: Regression Analysis Output of k_1 for Model 3

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.95275416
R Square	0.90774049
Adjusted R Square	0.78912112
Standard Error	53.55814557
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	9	197560.2997	21951.1444	7.65254874	0.006856315
Residual	7	20079.3247	2868.47496		
Total	16	217639.6244			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	5767.77791	1514.90599	3.80735039	0.00665081	2185.594462	9349.96135	2185.59446	9349.96135
P40 (%)	-15.6281609	4.09845135	-3.8131869	0.00660048	-25.3194584	-5.9368635	-25.319458	-5.9368635
Clay (%)	11.9701841	3.657336875	3.27292357	0.01361929	3.321956606	20.6184116	3.32195661	20.6184116
Silt (%)	10.2387196	3.683406931	2.77968734	0.02731054	1.528846245	18.9485930	1.52884625	18.9485930
PL	19.2055494	5.537818896	3.46807105	0.01043306	6.110688506	32.3004102	6.11068851	32.3004102
OMC (%)	-67.6929909	17.58514148	-3.8494425	0.00629699	-109.275243	-26.110739	-109.27524	-26.110739
MDD kg/m3	-1.78928924	0.596826854	-2.9980039	0.01999852	-3.20056050	-0.3780180	-3.2005605	-0.3780180
wc/OMC	300.974107	95.2311874	3.16045736	0.01591815	75.78813203	526.160083	75.7881320	526.160083
ys/MDD	-878.392738	264.0264906	-3.3269114	0.01264453	-1502.71618	-254.06930	-1502.7162	-254.06930
P200/wc	8.53482059	1.196517247	7.13305271	0.00018813	5.705506886	11.3641343	5.70550689	11.3641343

RESIDUAL OUTPUT

Observation	*Predicted k1*	*Residuals*
1	1043.503187	9.854704279
2	1071.354613	-17.9967225
3	1090.024289	-36.6663979
4	1030.091696	23.26619524
5	973.3991801	79.95871078
6	1069.183996	-15.8261046
7	1011.358529	41.99936225
8	971.5309758	-28.3589337
9	850.2677554	-69.0843875
10	761.6253583	19.55800968
11	787.034203	-5.85083498
12	773.7786666	7.404701361
13	801.5394421	-20.3560742
14	960.42527	-42.8550865
15	901.371667	16.19851657
16	903.2945093	14.27567421
17	893.0915163	24.47866719

Table D.4: Regression Analysis Output of k_1 for Model 4

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.8006336
R Square	0.6410141
Adjusted R Square	0.2820283
Standard Error	98.824054
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	8	139510.0753	17438.759	1.785625	0.214918384
Residual	8	78129.54911	9766.1936		
Total	16	217639.6244			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	-47452.953	14854.82152	-3.1944479	0.012718	-81708.2328	-13197.6731	-81708.2328	-13197.6731
P200 (%)	5.4639593	3.111613217	1.7559892	0.117160	-1.71143368	12.6393522	-1.71143368	12.6393522
Silt (%)	-6.9427931	4.037645527	-1.7195153	0.123837	-16.2536204	2.36803422	-16.2536204	2.36803422
(wc/OMC)*(ys/MDD)	5995.2581	2506.732376	2.3916626	0.043742	214.7228735	11775.7933	214.722874	11775.7933
OMC (%)	64.317547	27.42727611	2.3450213	0.047046	1.070134597	127.564959	1.07013460	127.564959
ys (kg/m3)	-23.319913	7.550635835	-3.0884701	0.014924	-40.7317103	-5.90811537	-40.7317103	-5.90811537
MDD kg/m3	30.042428	9.383978088	3.2014597	0.012585	8.402935728	51.6819203	8.40293573	51.6819203
wc/OMC	-7111.1578	2915.455354	-2.4391242	0.04062	-13834.2099	-388.105694	-13834.2099	-388.105694
ys/MDD	36350.5572	11716.93856	3.10239377	0.014613	9331.248385	63369.86592	9331.248385	63369.86592

RESIDUAL OUTPUT

Observation	*Predicted k1*	*Residuals*
1	1030.11071	23.24717922
2	947.593837	105.7640535
3	1085.47686	-32.1189653
4	1014.70562	38.65226658
5	900.248015	153.109876
6	1031.02075	22.33714255
7	1011.17601	42.18188274
8	976.820229	-33.6481866
9	912.401834	-131.218466
10	798.044257	-16.8608895
11	849.136435	-67.9530667
12	729.686542	51.49682616
13	844.19465	-63.0112829
14	988.72513	-71.1549426
15	969.71463	-52.1444483
16	904.06042	13.50976782
17	899.75893	17.81125304

Table D.5: Regression Analysis Output of k_2 for Model 1

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.96416357
R Square	0.92961139
Adjusted R Square	0.77475643
Standard Error	0.000307994
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	11	6.26402E-06	5.695E-07	6.0031105	0.03025508
Residual	5	4.74301E-07	9.486E-08		
Total	16	6.73832E-06			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	0.15217037	0.038385195	3.9642985	0.010697	0.0534981	0.2508427	0.0534981	0.2508427
P200 (%)	8.8335E-05	2.6941E-05	3.2788340	0.021983	1.908E-05	0.0001576	1.9081E-05	0.0001576
P40 (%)	-7.3246E-05	2.3274E-05	-3.147110	0.025462	-0.000133	-1.34E-05	-0.0001331	-1.342E-05
Silt (%)	3.3524E-05	1.25506E-05	2.6710717	0.044292	1.261E-06	6.579E-05	1.2612E-06	6.579E-05
LL	4.3517E-05	3.36449E-05	1.2934213	0.252394	-4.297E-05	0.0001300	-4.297E-05	0.0001300
PL	-0.00018760	3.67774E-05	-5.101037	0.003767	-0.000282	-9.31E-05	-0.000282	-9.306E-05
wc (%)	-0.0008213	0.000250107	-3.283789	0.021863	-0.001464	-0.000178	-0.001464	-0.0001784
OMC (%)	0.00024459	8.79683E-05	2.7804403	0.038882	1.846E-05	0.0004707	1.846E-05	0.0004707
ys (kg/m3)	7.8661E-05	1.9539E-05	4.0258345	0.010063	2.843E-05	0.0001289	2.843E-05	0.00012889
MDD kg/m3	-8.9346E-05	2.33066E-05	-	0.012204	-0.000149	-2.94E-05	-0.000149	-2.944E-05

			3.8335071					
wc/OMC	0.0150547	0.004503864	3.3426187	0.020494	0.0034772	0.0266323	0.0034772	0.0266323
ys/MDD	-0.13537718	0.032531263	-4.161449	0.008812	-0.219002	-0.051753	-0.219002	-0.0517529

RESIDUAL OUTPUT

Observation	*Predicted k2*	*Residuals*
1	0.001328574	7.59407E-05
2	0.001493918	-8.9403E-05
3	0.001578451	-0.000173936
4	0.001152127	0.000252387
5	0.001324864	7.96507E-05
6	0.001400535	3.97943E-06
7	0.001010745	0.000393769
8	-0.000964366	-0.000217426
9	0.001650546	-4.21602E-05
10	0.001539669	6.87165E-05
11	0.00185261	-0.000244224
12	0.00159071	1.76757E-05
13	0.00183027	-0.000221885
14	0.001459185	-7.55244E-05
15	0.001193077	0.000190584
16	0.001395791	-1.21297E-05
17	0.001389675	-6.01441E-06

Table D.6: Regression Analysis Output of k_2 for Model 2

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.832546532
R Square	0.693133727
Adjusted R Square	0.45445996
Standard Error	0.000479324
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	7	4.67056E-06	6.67222E-07	2.90410519	0.06958586
Residual	9	2.06776E-06	2.29751E-07		
Total	16	6.73832E-06			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	0.078896099	0.03519533	2.24166376	0.05170257	-0.00072127	0.15851347	-0.00072127	0.15851347
P200 (%)	-0.000343575	0.00017673	-1.94402324	0.08376670	-0.00074338	5.6225E-05	-0.00074338	5.6225E-05
Clay (%)	0.000344348	0.00017260	1.99509359	0.07716068	-4.6095E-05	0.00073479	-4.6095E-05	0.00073479
PL	-0.000108834	3.3705E-05	-3.22904867	0.01033839	-0.00018508	-3.2589E-05	-0.00018508	-3.2589E-05
Silt (%)	0.000340478	0.00016674	2.04199189	0.07153434	-3.671E-05	0.00071767	-3.671E-05	0.00071767
ys (kg/m3)	3.56932E-05	1.79757E-05	1.98563276	0.07834580	-4.9707E-06	7.6357E-05	-4.9707E-06	7.6357E-05
MDD kg/m3	-4.14481E-05	2.07466E-05	-1.9978297	0.07682114	-8.8380E-05	5.4839E-06	-8.8380E-05	5.4839E-06
ys/MDD	-0.064631907	0.03052943	-2.11703648	0.06334179	-0.13369427	0.00443045	-0.13369427	0.00443045

RESIDUAL OUTPUT

Observation	*Predicted k2*	*Residuals*
1	0.001783848	-0.000379333
2	0.001005477	0.000399037
3	0.001445603	-4.10884E-05
4	0.00110029	0.000304225
5	0.001341268	6.32462E-05
6	0.001597449	-0.000192935
7	0.001248734	0.000155781
8	-0.00034973	-0.000832062
9	0.001531656	7.67299E-05
10	0.001716726	-0.00010834
11	0.001704938	-9.65523E-05
12	0.002008438	-0.000400052
13	0.001406324	0.000202062
14	0.000852597	0.000531064
15	0.001367215	1.64457E-05
16	0.000810325	0.000573336
17	0.001655225	-0.000271565

Table D.7: Regression Analysis Output of k_2 for Model 3

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.91326459
R Square	0.83405221
Adjusted R Square	0.66810441
Standard Error	6.89582E-05
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	8	1.91198E-07	2.3900E-08	5.02599150	0.017364357
Residual	8	3.80419E-08	4.7552E-09		
Total	16	2.2924E-07			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	0.03092251	0.007864367	3.93197640	0.00434460	0.012787243	0.04905777	0.01278724	0.049057771
Silt (%)	4.06958E-06	2.70908E-06	1.50220204	0.17144579	-2.178E-06	1.0317E-05	-2.178E-06	1.0317E-05
LL	1.75857E-05	6.42113E-06	2.73872267	0.02549954	2.7785E-06	3.2393E-05	2.7785E-06	3.2393E-05
PL	-3.3434E-05	7.94714E-06	-4.2070974	0.00296761	-5.176E-05	-1.511E-05	-5.176E-05	-1.511E-05
wc (%)	-0.00011580	4.99281E-05	-2.3193016	0.04897279	-0.00023093	-6.639E-07	-0.0002309	-6.639E-07
ys (kg/m3)	1.45038E-05	3.9731E-06	3.65051522	0.00648953	5.3419E-06	2.3666E-05	5.3419E-06	2.3666E-05
MDD kg/m3	-1.7506E-05	4.79788E-06	-3.6486076	0.00650745	-2.857E-05	-6.442E-06	-2.857E-05	-6.442E-06
wc/OMC	0.00224130	0.000894569	2.50544684	0.03662985	0.000178415	0.00430418	0.00017842	0.00430418
ys/MDD	-0.02444618	0.006614571	-3.6958070	0.00607917	-0.03969941	-0.0091930	-0.0396994	-0.00919295

RESIDUAL OUTPUT

Observation	*Predicted k2*	*Residuals*
1	0.001454773	-5.0258E-05
2	0.001515999	-0.00011148
3	0.001388361	1.61532E-05
4	0.001439179	-3.4664E-05
5	0.001412689	-8.1739E-06
6	0.00141103	-6.5156E-06
7	0.001418771	-1.4256E-05
8	0.001154847	2.69444E-05
9	0.001548796	5.95897E-05
10	0.001597791	1.05954E-05
11	0.001568953	3.94331E-05
12	0.001606384	2.00167E-06
13	0.001507107	0.000101279
14	0.001425127	-4.1466E-05
15	0.001340411	4.325E-05
16	0.001422135	-3.8474E-05
17	0.001377616	6.04469E-06

Table D.8: Regression Analysis Output of k_2 for Model 4

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.951871955
R Square	0.906060218
Adjusted R Square	0.749493915
Standard Error	0.000324807
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	10	6.10532E-06	6.10532E-07	5.787070406	0.021712353
Residual	6	6.32996E-07	1.05499E-07		
Total	16	6.73832E-06			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	0.14535054	0.040096853	3.62498627	0.01103113	0.047237076	0.24346400	0.047237076	0.24346400
P200 (%)	0.000103617	2.55339E-05	4.05801598	0.00666543	4.11377E-05	0.00016610	4.11377E-05	0.00016610
P40 (%)	-8.55177E-05	2.24124E-05	-3.81563842	0.00880541	-0.000140359	-3.0677E-05	-0.000140359	-3.0677E-05
Silt (%)	3.10995E-05	1.30873E-05	2.37631351	0.05504431	-9.23942E-07	6.3123E-05	-9.23942E-07	6.3123E-05
PL	-0.000165215	3.4221E-05	-4.82788443	0.00291654	-0.000248951	-8.1479E-05	-0.000248951	-8.1479E-05
wc (%)	-0.000820386	0.000263759	-3.11035869	0.02083932	-0.001465782	-0.00017499	-0.001465782	-0.00017499
OMC (%)	0.000296727	8.2458E-05	3.59852152	0.01138670	9.49594E-05	0.00049849	9.49594E-05	0.00049849
ys (kg/m3)	7.60426E-05	2.04947E-05	3.71035469	0.00996542	2.58939E-05	0.00012619	2.58939E-05	0.00012619
MDD kg/m3	-8.52951E-05	2.4356E-05	-3.50202207	0.01279484	-0.000144892	-2.5698E-05	-0.000144892	-2.5698E-05

wc/OMC	0.015037376	0.004749701	3.16596248	0.01941778	0.003415276	0.02665948	0.003415276	0.02665948
ys/MDD	-0.130859294	0.034108754	-3.83653110	0.00859355	-0.214320407	-0.04739818	-0.214320407	-0.04739818

RESIDUAL OUTPUT

Observation	*Predicted k2*	*Residuals*
1	0.001480249	-7.57339E-05
2	0.001410362	-5.84709E-06
3	0.001533105	-0.00012859
4	0.001237251	0.000167263
5	0.001518525	-0.000114011
6	0.001474338	-6.98235E-05
7	0.000912294	0.000492221
8	-0.000925017	-0.000256774
9	0.001553244	5.51421E-05
10	0.001331034	0.000277351
11	0.001853343	-0.000244957
12	0.001575834	3.25516E-05
13	0.001867677	-0.000259291
14	0.001429086	-4.54254E-05
15	0.001190521	0.00019314
16	0.001325745	5.79158E-05
17	0.001458793	-7.5132E-05

Table D.9: Regression Analysis Output of k_3 for Model 1

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.877063833
R Square	0.769240968
Adjusted R Square	0.589761721
Standard Error	0.069707532
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	7	0.145782576	0.020826082	4.285960522	0.023471732
Residual	9	0.04373226	0.00485914		
Total	16	0.189514836			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	4.262315197	0.79141175	5.38571128	0.00044118	2.47201743	6.05261296	2.47201743	6.05261296
P40 (%)	0.012869351	0.00512582	2.51069043	0.0332730	0.00127394	0.02446477	0.00127394	0.02446477
P4 (%)	-0.02305569	0.00731722	-3.15088301	0.01172088	-0.03960838	-0.00650300	-0.03960838	-0.00650300
PL	-0.031476089	0.00668801	-4.70634287	0.00111021	-0.04660543	-0.01634675	-0.04660543	-0.01634675
OMC (%)	-0.028922109	0.0178514	-1.62015910	0.13964977	-0.06930478	0.01146056	-0.06930478	0.01146056
ys (kg/m3)	-0.00143753	0.00056184	-2.55859991	0.03075770	-0.00270851	-0.00016655	-0.00270851	-0.00016655
wc/OMC	0.241445466	0.13898419	1.73721539	0.11635921	-0.07295861	0.55584954	-0.07295861	0.55584954
ys/MDD	1.739362263	0.85394546	2.03685403	0.07213102	-0.19239659	3.67112111	-0.19239659	3.67112111

RESIDUAL OUTPUT

Observation	*Predicted k3*	*Residuals*
1	1.312681165	-0.035646534
2	1.287013338	-0.009978707
3	1.304592197	-0.027557566
4	1.271452559	0.005582072
5	1.363810135	-0.086775504
6	1.293790549	-0.016755918
7	1.300080942	-0.023046311
8	1.109056607	-0.034529503
9	1.493654263	-0.031252644
10	1.344616282	0.117785336
11	1.40887	0.053531618
12	1.406843982	0.055557637
13	1.456992341	0.005409278
14	1.172749207	0.085324507
15	1.260791073	-0.002717359
16	1.250831031	0.007242683
17	1.320246801	-0.062173086

Table D.10: Regression Analysis Output of k_3 for Model 2

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.994143755
R Square	0.988321806
Adjusted R Square	0.953287223
Standard Error	0.023522282
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	12	0.187301645	0.01560847	28.20989223	0.002754432
Residual	4	0.002213191	0.000553298		
Total	16	0.189514836			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	15.80089853	3.985183381	3.96491128	0.01660923	4.73625564	26.8655414	4.73625564	26.8655414
P200 (%)	-0.004817806	0.002556488	-1.88454102	0.13258377	-0.01191575	0.00228014	-0.01191575	0.00228014
P4 (%)	-0.009723589	0.002666461	-3.64662694	0.02183640	-0.01712687	-0.00232031	-0.01712687	-0.00232031
P40 (%)	0.010408592	0.003247788	3.20482538	0.03274988	0.00139129	0.01942590	0.00139129	0.01942590
Silt (%)	0.002546341	0.001021523	2.49269018	0.06729044	-0.00028986	0.00538255	-0.00028986	0.00538255
PL	-0.033154285	0.002846739	-11.6464071	0.00031070	-0.041058	-0.02525047	-0.041058	-0.02525047
LL	0.011098426	0.00245375	4.52304700	0.01063357	0.00428572	0.01791113	0.00428572	0.01791113
wc (%)	-0.081690707	0.020254296	-4.03325336	0.01569127	-0.13792565	-0.02545577	-0.13792565	-0.02545577
ys (kg/m3)	0.006261483	0.002019538	3.10045371	0.03620518	0.000654348	0.01186862	0.000654348	0.01186862

MDD kg/m3	-0.008033846	0.002423976	-3.31432498	0.02953474	-0.014763883	-0.00130381	-0.014763883	-0.00130381
wc/OMC	1.482627173	0.379750056	3.90421844	0.01747901	0.428271988	2.53698236	0.428271988	2.53698236
ys/MDD	-10.7471267	3.377010747	-3.18243782	0.03345711	-20.12321166	-1.37104174	-20.12321166	-1.37104174
P200/wc	-0.002426812	0.000488077	-4.97219560	0.00763956	-0.00378193	-0.00107169	-0.00378193	-0.00107169

RESIDUAL OUTPUT

Observation	*Predicted k3*	*Residuals*
1	1.279938961	-0.00290433
2	1.276888197	0.000146434
3	1.278546023	-0.001511392
4	1.270469707	0.006564924
5	1.297703671	-0.02066904
6	1.261862453	0.015172178
7	1.293968882	-0.016934251
8	1.071697055	0.002830049
9	1.466220195	-0.003818577
10	1.459963883	0.002437736
11	1.454352051	0.008049568
12	1.466456349	-0.004054731
13	1.449773329	0.012628289
14	1.245240439	0.012833275
15	1.251857356	0.006216359
16	1.283863334	-0.025789619
17	1.249270587	0.008803128

Table D.11: Regression Analysis Output of k_3 for Model 3

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.902339093
R Square	0.81421584
Adjusted R Square	0.628431679
Standard Error	0.066340838
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	8	0.154305982	0.019288248	4.382590193	0.025830556
Residual	8	0.035208855	0.004401107		
Total	16	0.189514836			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	19.15691356	5.84918529	3.275142197	0.01126929	5.668668092	32.6451590	5.668668092	32.6451590
P200 (%)	0.005730479	0.003289087	1.742270165	0.11963086	-0.00185417	0.01331513	-0.00185417	0.01331513
P4 (%)	-0.010380192	0.004185491	-2.48004187	0.03810917	-0.02003195	-0.00072843	-0.02003195	-0.00072843
PL	-0.027505795	0.007250633	-3.79357142	0.00528485	-0.04422579	-0.01078581	-0.04422579	-0.01078581
wc (%)	-0.087917281	0.044742709	-1.96495212	0.08500470	-0.19109415	0.01525959	-0.19109415	0.01525959
ys (kg/m3)	0.007373986	0.002939172	2.50886514	0.03643529	0.000596243	0.01415173	0.000596243	0.01415173
MDD kg/m3	-0.009697895	0.003535266	-2.74318668	0.02532418	-0.01785023	-0.00154556	-0.01785023	-0.00154556
wc/OMC	1.721942434	0.78755117	2.18645150	0.06025341	-0.09415382	3.53803869	-0.09415382	3.53803869
ys/MDD	-12.78552193	4.873595732	-2.62342686	0.03048778	-24.0240539	-1.54699002	-24.0240539	-1.54699002

RESIDUAL OUTPUT

Observation	*Predicted k3*	*Residuals*
1	1.313418099	-0.036383468
2	1.296306337	-0.019271706
3	1.262465597	0.014569034
4	1.323892826	-0.046858195
5	1.379416416	-0.102381785
6	1.276212518	0.000822113
7	1.299386756	-0.022352125
8	1.069455952	0.005071153
9	1.446480833	0.015920786
10	1.342634448	0.119767171
11	1.430388225	0.032013393
12	1.44902431	0.013377309
13	1.446949929	0.01545169
14	1.278398684	-0.020324969
15	1.211071615	0.047002099
16	1.240786432	0.017287283
17	1.291783497	-0.033709782

Table D.12: Regression Analysis Output of k_3 for Model 4

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.973239765
R Square	0.947195641
Adjusted R Square	0.831026051
Standard Error	0.044737477
Observations	17

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	11	0.179507627	0.016318875	8.153559312	0.015631621
Residual	5	0.010007209	0.002001442		
Total	16	0.189514836			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	29.37102075	5.18350653	5.666245539	0.002381334	16.04639302	42.69564848	16.04639302	42.69564848
P4 (%)	-0.00805008	0.00302989	-2.65688649	0.045052061	-0.01583866	-0.00026149	-0.01583866	-0.00026149
Clay (%)	0.003353755	0.002311808	1.450706427	0.206561188	-0.00258894	0.009296447	-0.00258894	0.009296447
Silt (%)	0.007613715	0.002941304	2.588551139	0.048922611	5.28532E-05	0.015174576	5.28532E-05	0.015174576
LL	0.013758314	0.004899088	2.808341567	0.037621695	0.001164806	0.026351822	0.001164806	0.026351822
PL	-0.03434047	0.005377557	-6.38588692	0.001394328	-0.04816392	-0.02051702	-0.04816392	-0.02051702
wc (%)	-0.11641301	0.033641909	-3.46035698	0.018035383	-0.20289229	-0.02993373	-0.20289229	-0.02993373
OMC (%)	-0.01926859	0.011626619	-1.65728209	0.158362179	-0.04915576	0.010618588	-0.04915576	0.010618588
ys (kg/m3)	0.012282678	0.002634201	4.662771686	0.005518318	0.005511249	0.019054107	0.005511249	0.019054107
MDD kg/m3	-0.01588914	0.003152621	-5.03997632	0.003966971	-0.02399321	-0.00778507	-0.02399321	-0.00778507

wc/OMC	2.272352247	0.607540611	3.740247494	0.013428109	0.710619388	3.834085106	0.710619388	3.834085106
ys/MDD	-21.0218853	4.37247595	-4.80777607	0.004850312	-32.2616925	-9.78207799	-32.2616925	-9.78207799

RESIDUAL OUTPUT

Observation	*Predicted k3*	*Residuals*
1	1.262937852	0.014096779
2	1.290628312	-0.01359368
3	1.260393626	0.016641005
4	1.297433204	-0.02039857
5	1.317476636	-0.04044201
6	1.2930466	-0.01601197
7	1.293843697	-0.01680907
8	1.041545317	0.032981787
9	1.46022141	0.002180208
10	1.432141044	0.030260575
11	1.429663424	0.032738195
12	1.470106154	-0.00770454
13	1.434171923	0.028229695
14	1.274874697	-0.01680098
15	1.247234182	0.010839533
16	1.30588102	-0.04780731
17	1.246473377	0.011600338