THE EFFECTS OF MATERIAL PARAMETERS ON DYNAMIC CONE PENETROMETER RESULTS FOR FINE-GRAINED SOILS AND GRANULAR MATERIALS

By

AZMI BIN HASSAN

Bachelor of Eng. (Tech) Honors in Civil Engineering
University of Wales
Institute of Science & Technology
June, 1980

Master of Science in Civil Engineering
Oklahoma State University
Stillwater, Oklahoma
December, 1987

Submitted to the Faculty of the Graduate College
of the Oklahoma State University
in partial fulfillment of the requirements
for the Degree of
DOCTOR OF PHILOSOPHY
December, 1996

THE EFFECTS OF MATERIAL PARAMETERS ON DYNAMIC CONE PENETROMETER RESULTS FOR FINE-GRAINED SOILS AND GRANULAR MATERIALS

Thesis Approved:

Mark E Ayers

Thesis Adviser

James B. Nevels Jr.

Phillip L. Manke

Carol L. Goad

Gerald D. Oberlender

Thomas C. Collins

Dean of the Graduate College

ACKNOWLEDGMENTS

The author wishes to express his deep gratitude and sincere appreciation to his thesis adviser, Dr. Michael Ayers; and to other committee members, Dr. Carla Goad, Dr. Phillip Manke, Dr. James Nevels, and Dr. Garold Oberlender for their invaluable assistance, counsel, encouragement, and guidance during the research program.

The author is also grateful to the Public Works Department of Malaysia for nominating him for the award of a scholarship from the Government of Malaysia, which provided financial support during the author's stay at Oklahoma State University.

Special acknowledgment is extended to the author's parents, Hassan Ahmad and Meriam Hj. Awang, for their love of education and knowledge, and for their encouragement to continue his education to this level.

The author is exceptionally grateful to his wife, Azimah Arifin, for her encouragement, love, and patience during this period of study; and children, Azhar, Norazhani, Zakiran, Nornabilah, Nornashuha, and Hamdan, for their understanding.

The assistance of Ms. Charly Fries in managing and editing the manuscript is gratefully acknowledged.

TABLE OF CONTENTS

LIST OF TABLES

Table | Page

LIST OF FIGURES

Figure		Page

CHAPTER I

INTRODUCTION

1.1 Background

The design of highway and airport pavements is made up of two broad categories: the design of the paving mixtures commonly called mix design and the structural design of the pavement components generally referred to as pavement design (1). The latter is applicable in this study. Three important factors that affect pavement design are subgrade soil characteristics, assumed drainage conditions, and expected traffic loading. For a pavement design procedure to provide a reliable thickness requirement, an accurate and representative material characterization technique will be required (2). Such a technique would be even more beneficial if it could also be performed rapidly.

In the evaluation of pavements, sometimes the existing base course and subbase need to be characterized to facilitate the decision on the nature of the rehabilitation strategy. Even though Non-Destructive Testing (NDT) procedures are generally used in the evaluation of rehabilitation projects, the use of a rapid testing method to verify the findings is regularly carried out. One of the methods that can be utilized for this evaluation purpose is the Dynamic Cone Penetrometer (DCP) test procedure (3).

Following the advent of the American Association of State Highways and Transportation Officials (AASHTO) design procedure in 1986, the resilient modulus test gained importance in the characterization of pavement support material, i.e. fine-grained soils and granular materials in the subgrade, subbase and base course layers. However, the resilient modulus test procedure requires three to four hours to perform. The DCP is a reliable and proven device whose data have been correlated to California Bearing Ratio (CBR), unconfined compressive strength, and shear strength values.

However, correlation studies with the resilient modulus have yet to be carried out. This study is primarily an attempt to investigate whether data from the DCP test can be meaningfully correlated to the resilient modulus of fine-grained soils. A significant correlation between the DCP values and the resilient modulus should expedite pavement design and pavement evaluation procedures. Additionally, it is also an investigation of how selected factors affect DCP values in fine-grained soils and granular materials.

1.2 Materials Characterization in Pavement Design

1.2.1 Subgrade Soil

The importance of subgrade soil characterization in pavement design is less critical compared to other areas of civil engineering such as the soil investigations for foundations in high rise buildings, bridges, tunnels, dams, etc. The consequences of the failure of these structures are generally more critical and could possibly lead to loss of lives. Nevertheless, in the United States, Oglesby and Hicks (1) noted that serious investigations of soil properties in the subgrade began in the early 1930s. The increase in vehicle speeds brought demands for higher design standards requiring deeper cuts and higher fills. A scientific basis in characterizing soil was initiated with the development of the Public Roads Administration Classification System using grain size as the basis of categorization. The present AASHTO classification system was subsequently developed from this early method. Now most transportation agencies have established detailed procedures for investigating subgrade soil properties and incorporating these data in their design procedures (1).

1.2.2 Subbase and Base Courses

The test methods used to evaluate the subbase and base course materials are dependent upon the pavement design procedure employed. The parameter of the materials can either be based upon a fundamental property such as the strength/ deformation characteristics of the CBR or the input variables for layer theory solution such as the resilient modulus. Strength/deformation tests such as plate loading, triaxial, CBR, etc. are generally those that define the material strength against deformation. Other input variables suitable for the layer theory solution include the complex modulus and dynamic stiffness. In this respect, correlation to strength coefficients have been established so that these parameters can be easily determined and incorporated as design inputs. The layer coefficients used in the structural number concept in the 1986 AASHTO design procedure is an example of this.

One of the characterizing values that has been correlated to the strength coefficient of the subbase or the base course is the resilient modulus, M_r (2). The M_r is believed to be more representative than the CBR of the condition the material is subjected to in real situations. The load to failure under field conditions is through dynamically repeated loads similar to the resilient modulus testing rather than single load to failure reminiscent of the CBR.

1.2.3 Pavement Design Procedures

There are a number of methods that may be used in the structural design of pavements. They may vary from the application of engineering judgment which depends on the expertise and the experience of the engineer or empirical methods, rational methods such as the Strategic Highway Research Program (SHRP) procedure and mechanistic means. Mechanistic models are based on properties of materials and allows qualitative evaluation of the effects of unusual loading, wheel configurations, materials and different layer thickness without constructing test sections. Pavements may be classified as either flexible or rigid. A typical flexible pavement structure is shown in Figure 1; Figure 2 shows a typical rigid pavement structure (4).

1.2.4 Flexible Pavement

Flexible pavements normally consist of several material layers or courses above the subgrade soil. A wide variety of materials and layer combinations may be used in the lower layers, i.e the subbase and the base course. The thickness of the respective layers varies with the quality of materials and the design procedure used. Oglesby and Hicks (1) classified a flexible pavement structure as having a thin asphalt wearing course with layers of subbase and base course protecting the subgrade from being overstressed. The

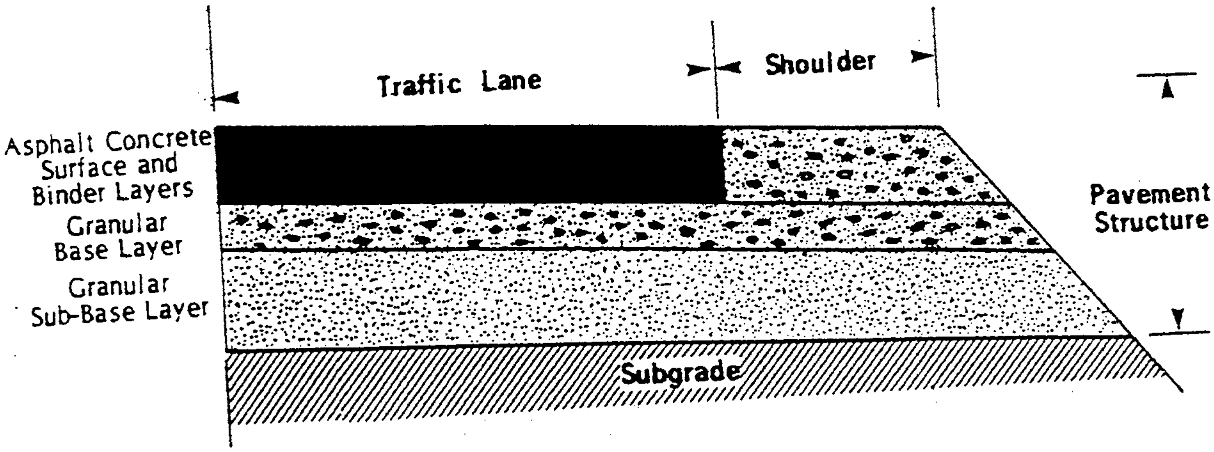


Figure 1. Typical Flexible Pavement Structure (4)

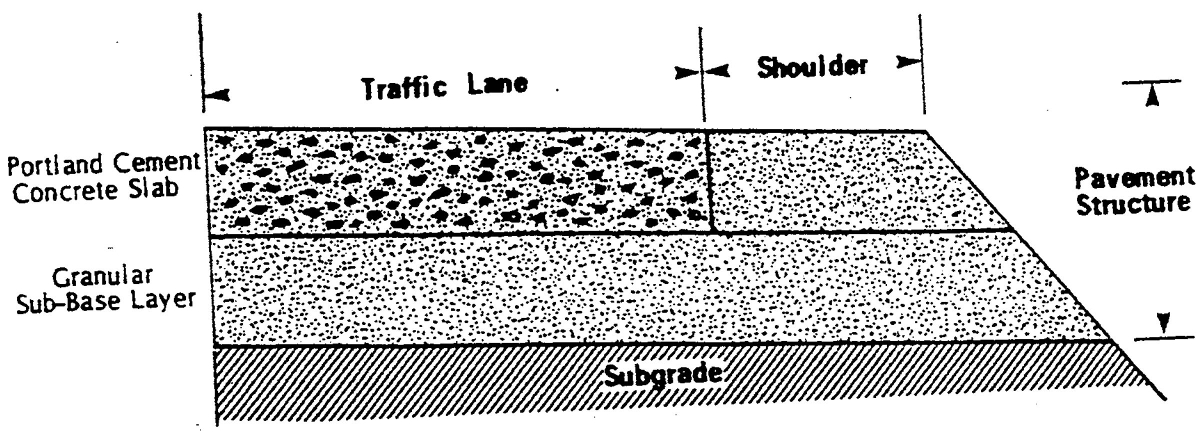


Figure 2. Typical Rigid Pavement Structure (4)

pavement strength is developed through the build-up of relatively thick layers of subbase and the base course. It generally consists of the subbase, base course, asphaltic concrete binder course and asphaltic concrete wearing course layers (see Figure 1).

Flexible pavement design methods currently in use include the 1986 AASHTO design procedure, the 1972 AASHTO design procedure, the Asphalt Institute procedure, the Corps of Engineers—California Bearing Ratio (CBR) procedure, the California Department of Transportation (CALTRAN) procedure, and a few mechanistic methods. A literature search and interviews conducted in North America by the National Research Council revealed that 51 of the 55 state transportation agencies employ either the 1972 or 1986 AASHTO design procedure.

The 1986 AASHTO procedure, which is empirical in nature, requires the expected equivalent number of 18 kips cumulative axles loads, the assumed drainage type, the resilient modulus of the subgrade soil, and the layer coefficients of the upper pavement layers as design inputs. The layer coefficient of a pavement layer is a value that relates to its strength to resist deformation. They are determined from direct laboratory testing or through correlation to other strength parameters such as resilient modulus. These data are entered in a nomograph which provides the required structural number as the end result. The structural number can be translated into the summation of the product of thickness and layer coefficient for each layer (7).

In the Asphalt Institute procedure, the subgrade soil is characterized by the resilient modulus. However, the procedure can also use other soil characterization techniques, i.e the California Bearing Ratio (CBR). The design procedure uses nomographs with the number of 18-kip cumulative axle loads and subgrade soil strength as input variables. There are three kinds of pavement structure that can be used, i.e. full depth of asphaltic concrete, and asphalt concrete wearing surface with either emulsified base or granular base (8). In a survey conducted among states in North America, it was found that only Kansas and Nova Scotia are using the Asphalt Institute method. Kansas uses the resilient modulus and Nova Scotia uses the CBR as the subgrade soil characterizing method (5).

The Corps of Engineers design procedure uses the CBR value while the California Department of Transportation procedure uses the stabilometer resistance value (R-value) as the subgrade soil characterizing method. The less commonly used Group Index procedure uses gradation and classification of the soil as the primary method of characterizing the subgrade soil (2). Among the state Departments of Transportation that use the mechanistic approach, a majority use either the resilient modulus or the CBR in characterizing the subgrade soil. It is apparent that the prevalent subgrade soil characterizing method used in the flexible pavement design procedures is the CBR followed closely by the resilient modulus. This study was conducted to evaluate pavement design procedures used by the various state departments of transportation and found that many of these agencies expressed their intention to adopt the resilient modulus in the future. Table 1 is a summary of the design procedures employed by the various departments of transportation in North America (5).

1.2.5 Rigid Pavement

Rigid pavements are made of portland cement concrete and are sometimes referred to as concrete pavements. A portland cement concrete slab may or may not have a base course. The slab has a high modulus of elasticity and rigidity which tends to distribute applied loads over a wide area of the subgrade enabling the slab to provide a substantial portion of the structural capacity. The major factor in the design of rigid pavement is the required thickness of the concrete pavement itself. Minor variations in the subgrade and base strength have little impact on the structural capacity of the pavement.

The primary rigid pavement design methods include the 1972 and 1986 AASHTO procedures, the Portland Cement Association (PCA) procedure, the California Department of Transportation (CALTRAN) method, and a variety of mechanistic methods. The majority of departments of transportation in the United States use the 1986 AASHTO procedure. The remaining states, with the exception of California and Kentucky, use the 1972 AASHTO procedure and the PCA procedure; California uses the CALTRAN procedure and Kentucky uses a mechanistic design method (5).

TABLE 1

FLEXIBLE PAVEMENT DESIGN PROCEDURES [5]

Agency	Design Procedure(s)	Shoulder Design
Alabama	AASHTO (1986)	
Alaska	Alaska	Standard Thickness
Arizona	AASHTO (1986)	Full Depth
Arkansas	AASHTO (1986)	Standard Thickness
California	Caltrans	2% Mainline Traffic
Colorado	AASHTO (1986) with Colorado	Full Depth
Connecticut	AASHTO (1986)	Full Depth
Delaware	AASHTO (1986)	2.5% Mainline Traffic
Florida	AASHTO (1972)	Standard Thickness
Georgia	AASHTO (1972)	Standard Thickness
Hawaii	Caltrans	Full Depth
Idaho	Caltrans	Full Depth
Illinois	AASHTO (1972), Mechanistic (full depth)	Standard Thickness
Iowa	AASHTO (1986), AASHTO (1972)	Standard Thickness
Kansas	Asphalt Institute	10% Mainline Traffic
Kentucky	Kentucky (mechanistic)	10-20% Mainline Traffic
Louisiana	AASHTO (1972)	Standard Thickness
Maine	AASHTO (1972)	Standard Thickness
Maryland	AASHTO (1986) modified	10% Mainline Traffic
Massachusetts	AASHTO (1972)	Full Depth
Michigan	AASHTO (1972)	Full Depth
Minnesota	Mn DOT (AASHTO (1986), Asphalt Institute used as checks)	Standard Thickness (rural), Full Depth (urban)
Missouri	AASHTO (1986)	Standard Thickness, Full Depth
Montana	AASHTO (1972) Back Calc. Mr	Full Depth
Nebraska	AASHTO (1986)	Standard Thickness
Nevada	AASHTO (1972)	Full Depth
New Hampshire	AASHTO (1972)	Standard Thickness
New Jersey	AASHTO (1972), AASHTO (1986)	Full Depth, 10% Mainline Traffic

TABLE 1. (CONTINUED)

Agency	Design Procedure(s)	Shoulder Design
New Mexico	AASHTO (1972)	20% Mainline Traffic
New York	NYSDOT	Standard Thickness, Full Depth
North Carolina	AASHTO (1972)	3% Mainline Traffic
North Dakota	AASHTO (1972)	Standard Thickness
Oklahoma	AASHTO (1986)	Full Depth
Ohio	AASHTO (1986)	Standard Thickness
Oregon	AASHTO (1986), Asphalt Institute, Mechanistic	Full Depth
Pennsylvania	AASHTO (1972)	Standard Thickness
Rhode Island	AASHTO (1986)	Full Depth
South Carolina	AASHTO (1972)	Standard Thickness
South Dakota	AASHTO (1986)	Standard Thickness
Tennessee	AASHTO (1972)	2% Mainline Traffic
Texas	Texas FPS	Full Depth
Utah	AASHTO (1972)	Full Depth
Vermont	AASHTO (1972)	Standard Thickness
Virginia	AASHTO (1972), VDOT	2.5% Mainline Traffic
Washington	WSDOT (mechanistic), AASHTO (1986)	10% Mainline Traffic
West Virginia	AASHTO (1986)	Full Depth
Wisconsin	AASHTO (1972)	Standard Thickness, Full Depth
Wyoming	AASHTO (1972)	Full Depth
Alberta	RTAC Prototype Method, Asphalt Institute	Full Depth
British Columbia	Canadian Good Roads Association	Standard Thickness
Nova Scotia	Asphalt Institute	Standard Thickness
Ontario	Ontario Pavt. Anal. of Cost (OPAC)	Standard Thickness
Quebec	AASHTO (1986)	Full Depth
Saskatchewan	Shell (mechanistic)	10% Mainline Traffic

For characterizing the subgrade soil, the 1972 AASHTO procedure relies on the CBR method while the 1986 AASHTO procedure makes use of the resilient modulus of the composite subgrade and the subbase layers. The Westergaard modulus of subgrade reaction (k) is the subgrade support variable required in the PCA method. For the mechanistic method employed in the state of Kentucky, a CBR value is used to characterize the subgrade soil. The CALTRAN procedure uses the stabilometer resistance value, commonly called the R-value, to characterize the soil. Table 2 is a summary of the various design methods for rigid pavement employed by the various departments of transportation in North America (5).

1.2.6 Group Index

The Group Index is a soil characterizing method which uses grain size and Atterberg Limits to classify the soil. Its use as a soil characterizing method is now limited to the states of Wisconsin and Missouri, and is only marginally utilized elsewhere since the introduction of the AASHTO design procedure (5). Interestingly, these two states adopted the 1986 AASHTO pavement design procedure in which the recommended soil strength parameters is the resilient modulus. This suggests that some form of correlation may have been established by these states between the Group Index and the resilient modulus to enable the design procedure to be used in this manner. In the past, the Group Index soil characterizing method was widely used in design methods such as the U.S. Corps of Engineers method.

1.2.7 California Bearing Ratio (CBR)

The CBR, the most commonly used soil characterizing procedure, is used to determine the subgrade, subbase, and base course strength (5). It is generally used to determine the subgrade soil strength in the design of interstate highways as well as low volume roads using the appropriate design procedures discussed in the previous sections. Occasionally, when designing lower volume roads, agencies rely on correlation to either the ASTM, AASHTO, or FAA soil classification system. Some agencies allow correlation of CBR with other soil strength parameters including the penetration values

TABLE 2

RIGID PAVEMENT DESIGN PROCEDURES (5)

Agency	JPCP	RCP	CRCP	Shoulder Design
Alabama	AASHTO (1986)	AASHTO (1986)	AASHTO (1986)	Full Depth
Arizona	AASHTO (1986)	AASHTO (1986)	AASHTO (1986)	Full Depth
Arkansas	AASHTO (1986)	AASHTO (1986)	AASHTO (1986)	Full Depth
California	Caltrans	NA	NA	2% Mainline Traffic
Colorado	AASHTO (1986)	NA	NA	Full Depth
Connecticut	AASHTO (1986) PCA, ACPA	PCA, ACPA	AASHTO (1986)	Full Depth
Delaware	AASHTO (1986)	AASHTO (1986)	NA	Standard Thickness
Florida	AASHTO (1972)	NA	NA	3% Mainline Traffic
Georgia	AASHTO (1972)	NA	AASHTO (1972)	Full Depth Taper
Hawaii	PCA	NA	NA	Full Depth
Idaho	AASHTO (1972)	AASHTO (1972)	AASHTO (1972)	Full Depth
Illinois	Illinois DOT	AASHTO (1972)	AASHTO (1972)	Full Depth Taper
Iowa	AASHTO (1986) PCA	NA	NA	Full Depth
Kansas	AASHTO (1986)	PCA	NA	Full Depth Taper
Kentucky	Mechanistic[1]	NA	NA	10% + Mainline Traffic
Louisiana	AASHTO (1986)	NA	NA	Standard Thickness
Maine	AASHTO (1972)	NA	NA	Standard Thickness
Maryland	NA	AASHTO (1986)	AASHTO (1986)	Full Depth
Michigan	AASHTO (1986)	AASHTO (1986)	NA	Full Depth Taper
Minnesota	AASHTO (1972)	AASHTO (1972)	NA	Standard Thickness
Missouri	MHTD	MHTD	AASHTO (1986)	Full Depth
Montana	PCA	NA	NA	Full Depth
Nebraska	AASHTO (1986)	NA	NA	Full Depth
Nevada	AASHTO (1972)	NA	NA	Full Depth
New Jersey	NA	NA	Standard Thickness AASHTO (1986) PCA	Full Depth 10% Mainline Traffic
New Mexico	AASHTO (1986)	NA	NA	Full Depth
New York	NYSDOT	NYSDOT	NA	Full Depth
North Carolina	AASHTO (1972)	NA	NA	Full Depth
North Dakota	AASHTO (1972)	NA	NA	Standard Thickness
Ohio	AASHTO (1986)	AASHTO (1986)	AASHTO (1986)	Full Depth
Oklahoma	AASHTO (1986)	AASHTO (1986)	AASHTO (1986)	Full Depth
Oregon	AASHTO (1986)	NA	AASHTO (1986)	Full Depth
Pennsylvania	AASHTO (1972)	AASHTO (1972)	AASHTO (1972)	Standard Thickness
Rhode Island	AASHTO (1986)	AASHTO (1986)	AASHTO (1986)	Full Depth
South Carolina	AASHTO (1972)	NA	NA	Standard Thickness
South Dakota	AASHTO (1986)	NA	AASHTO (1986)	Standard Thickness
Tennessee	AASHTO (1986)	NA	NA	Full Depth
Texas	AASHTO (1986)	AASHTO (1986)	AASHTO (1986)	Full Depth
Utah	AASHTO (1986)	NA	NA	Full Depth
Virginia	AASHTO (1986) PCA	AASHTO (1986) PCA	AASHTO (1986) PCA	Full Depth Taper
Washington	AASHTO (1986)	NA	NA	Full Depth
West Virginia	NA	AASHTO (1986)	NA	Standard Thickness
Wisconsin	AASHTO (1972)	NA	NA	Standard Thickness
Wyoming	AASHTO (1986)	NA	NA	Full Depth
Ontario	Ontario, PCA	NA	NA	Full Depth Full Depth Taper
Quebec	PCA, AASHTO (1986)	NA	NA	Full Depth

[1]AASHTO (1986) for comparison.

from the DCP or the cone penetration tests. The determination of CBR values for the subgrade soil can either be performed in the laboratory or in the field. AASHTO T193-93 is used for the laboratory testing of CBR using either one of the two procedures depending on the nature of the soil.

1.2.8 Modulus of Subgrade Reaction

The Westergaard modulus of subgrade reaction (k) is the characterization method used in the Portland Cement Association (PCA) design procedure. It is defined as the load in pounds per square inch (psi) on a circularly loaded area of 30 inches diameter divided by the deflection in inches corresponding to that load. It is expressed in psi per inch or more commonly in pounds per cubic inch. This is not only a cumbersome and time consuming method but is also comparatively expensive to run. Whenever possible, agencies will try to characterize the soil through other methods. Correlating it to other characterizing methods such as the CBR and also to the various soil classification systems have expedited the use of this parameter in the design process (9).

1.2.9 Stabilometer Resistance Value

Generally, agencies which follow the California Department of Transportation design procedure use the stabilometer resistance value (R-value) and the material cohesion as determined by the cohesiometer as inputs to design the pavement thickness. The determination of the R-value requires the use of the stabilometer. This test method provides the resistance to deformation which is expressed as a function of the ratio of the transmitted lateral pressure to the applied vertical pressure of 160 psi on a compacted cylindrical material sample. In this design procedure, the required value for the other input parameter, the cohesion may sometimes be assumed from soil classification (2).

1.2.10 Gradation And Soil Classification Systems

The soil classification systems suitable for engineering purposes include: the Unified Soil Classification System (ASTM D2487, D2488) and the AASHTO Classification

System. The design procedure employed by Alaska Department of Transportation use the AASHTO Classification System as the primary design input. These classification systems have been correlated to common subgrade soil characterization methods such as the resilient modulus, CBR, R-value, and modulus of subgrade reaction. Generally, correlations based on soil classification are used as a design input only in the design of secondary or low volume roads. Figure 3 shows the interrelationship between the various soil characterization methods and common soil strength parameters (9).

1.3 Material Characterization in Pavement Evaluation and Rehabilitation

The evaluation of existing pavements to ascertain the type of rehabilitation required necessitates determining the strength of the subgrade, subbase, and base courses. Nondestructive testing (NDT) methods are suitable for these determinations under most circumstances and are more widely used for the design of overlay, reconstruction or rehabilitation projects than for the design of new pavements (3). The objective of the NDT is to evaluate the structural response of the pavement to dynamic loads similar to those imposed by high speed truck traffic. The collected data are then used to ascertain the properties of the pavement layers as estimated through back calculation from the measured deflection. These properties are then used to determine the remaining pavement structural capacity. From this knowledge, reconstruction and rehabilitation actions can be programmed to suit the financial constraints (10).

There are three broad categories of NDT methods differentiated by load type: impulse, steady state dynamic and static. The impulse NDT device is the most recently developed and best simulates the load from a moving tire. The Falling Weight Deflectometer (FWD) is the best example of this type of equipment which includes: Dynatest FWD, Phoenix FWD and KUAB FWD. Examples of the steady state dynamic loading type are the Dynaflect, the Road Rater, and the Waterways Experimental Station (WES) 16 kip vibrator. The Benkelman Beam is the most widely used of the static or the slow moving load devices (3).

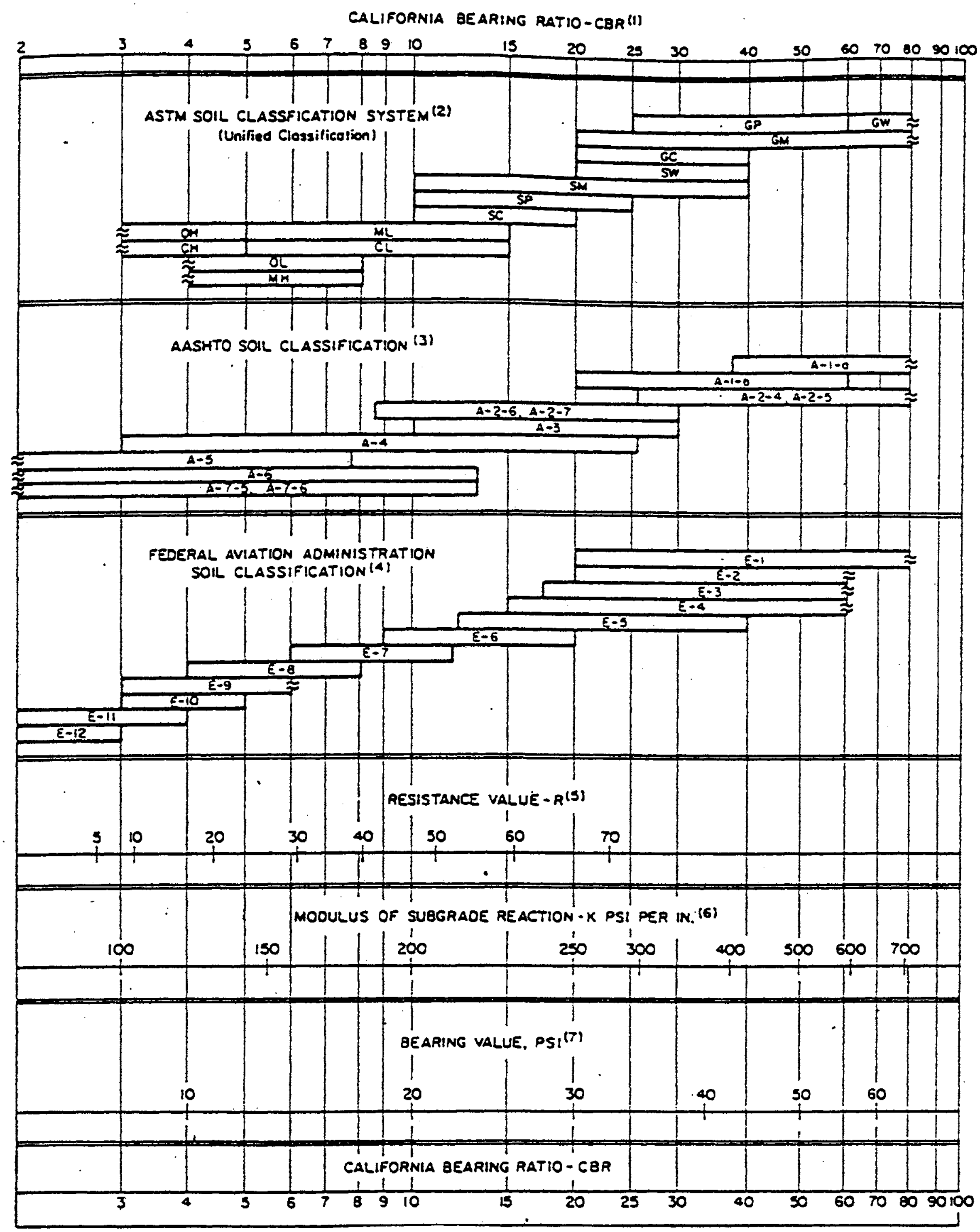


Figure 3. Interrelation of Soil Classifications and Various Strength Characteristics (10)

The estimated material properties provided by NDT methods are frequently verified by some type of destructive testing method, to confirm that the backcalculated strength is comparable to the laboratory/field test determined value. The DCP, which is inexpensive, rapid and easy to use apparatus is a suitable method to determine the pavement layer strength (3). Otherwise some form of a boring procedure has to be performed.

1.4 Resilient Modulus

The resilient modulus is a measure of the elastic property of the soil that recognizes certain nonlinear characteristics. It is defined as the modulus of the elastic rebound or resiliency of the soil. Numerically, it is the ratio of the deviator stress (σ_d) to the resilient or recoverable strain (E_r).

$$M_r = \sigma_d / E_r$$

Resilient modulus values may either be obtained through laboratory testing, back calculation programs using deflection measurements, and correlation to other strength parameters such as the CBR and R-value. However, AASHTO recommends user agencies to use the testing device for the determination of the resilient modulus (6).

Generally, for fine-grained soils, the effect of deviator stress, density and moisture content are controlling factors in the determination of the resilient modulus. Deviator stress is the applied vertical which is the numerical difference between the principal stress in the vertical and horizontal directions. Numerous research studies have shown that confining pressure does not affect the resilient modulus of fine-grained soil. The resilient modulus has been found to decrease with increasing deviator stress to a certain value after which it increases very slightly with the increase in deviator stress. Generally, increasing moisture content decreases the resilient modulus especially for A-4 to A-6 soil while increasing density increases the resilient modulus of the soil (6).

For granular materials, it was found that confining pressure and density affect the resilient modulus values. Increasing confining pressures and increasing density increase the resilient modulus values. Additionally, resilient modulus values are also found to increase with increasing angularity of the soil particles and decreasing saturation (6).

The laboratory procedure for determining the resilient modulus consists of the application of more than 15 loading steps in a cyclic triaxial test on a material sample. At each loading step the confining pressure and/or the deviatoric stress is changed. The latest guideline in resilient modulus testing is provided by the Method of Test for Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils—SHRP Protocol P46 (AASHTO T 294-94). The test procedure for estimating the resilient modulus of a particular material depends on the material type, whether granular (type I) or cohesive (type II). Type I soils undergo higher confining and deviator stresses as they can tolerate higher deformation. The deformation is evaluated by displacement measured by the linear variable differential transformers (LVDTs) located outside the chamber.

Subgrade soils, unbound granular layers, or slightly stabilized layers can also be tested using the original resilient modulus test method (AASHTO T 274) while stiff materials, asphaltic concrete, or stabilized bases can be tested using the repeated-load indirect tensile test (ASTM D 4123).

To evaluate existing pavement conditions and to conduct rehabilitation studies, the resilient modulus may be obtained using one of the numerous nondestructive pavement tests mentioned previously. Deflection data are used to obtain the resilient modulus with back calculation computer programs. Computer programs such as MODULUS, EVERCALC, and BOUSDEF are available for this purpose.

To assist the use of resilient modulus as an input in pavement design, correlation to other parameters such as CBR, R-value, and various soil classification systems such as the Unified Soil Classification system. In the design of rigid pavements, the resilient modulus is correlated with the modulus of subgrade reaction (k-value).

Resilient modulus values of materials for primary state highways or the interstate highways are generally obtained through direct laboratory testing rather than through correlation (9). In order to facilitate obtaining a reliable, less expensive and a faster method for the characterization of subgrade soils or subbase/base course materials, penetration testing procedures will be reviewed and their potential to be correlated with the resilient modulus will be studied.

1.5 Problem Statement

While there are established correlation between resilient modulus and other soil characterization methods, there remain numerous reasons to doubt their use. Many factors including density, moisture content, rate of load application, stress history, and the degree of confinement for granular soils affect the strength/deformation properties of the soil. The correlation of the CBR with the resilient modulus is not based on a comparable philosophy since it is a comparison of a static to a repeated load dynamic testing procedure. The practice of correlating the resilient modulus with soil gradation is not reliable, as it is claimed that the resilient modulus is also influenced by the parent material of the soil prior to it being weathered down to finer particles.

The resilient modulus test is a tedious and time consuming procedure since its determination requires performing repeated load triaxial tests. In practice, the speed at which subgrade soil strength data needs to be gathered may sometimes be crucial as highway designs must be completed by a specified deadline. Consequently, the number of locations to perform the resilient modulus test along the route of a project may have to be reduced since it will take between three to four hours to conduct a single test satisfactorily.

The resilient modulus is an expensive test method costing approximately $300 per test, and because of this, the cost to characterize the subgrade soil for a typical project may become prohibitive. Some agencies reduce the number of tests required for adequate characterization in an effort to reduce costs. The accuracy of the soil characterizing process thus becomes doubtful. Such actions can sometimes result in increased life cycle costs of the project due to premature failures caused by poor design.

It has been reported that the resilient modulus is very sensitive to variations in testing conditions, especially in granular materials. A slight change in confining pressure has been reported to produce a significant change in its value. The sensitive nature of this characterization procedure has led to dissatisfaction by many users.

The DCP testing appears to be a promising alternative in the characterization of pavement materials if it can be meaningfully correlated to the resilient modulus. DCP testing in granular materials has produced useful correlation with such parameters as CBR and shear strength. However, studies of DCP testing on fine-grained soils have drawn contrasting conclusions, especially when applied to soils which are highly plastic or near saturation. Whether similar trends would prevail when applied to correlation with the resilient modulus have not been adequately investigated.

1.6 Purpose of the Study

A review of state-of-the-art penetration testing procedures will be undertaken in this study. Five types of penetration testing procedures will be investigated. Their history, equipment description, testing procedure, general use, and suitability for the characterization of pavement materials will be discussed.

The DCP equipment will be adapted to allow testing samples under confined conditions with the goal of developing correlation with resilient modulus values. Establishing a reliable correlation between the DCP values and resilient modulus for fine-grained soil will be the primary goal of this study. Whether the correlation is meaningful when applied to fine-grained soil in a plastic state or with a relatively high water content, such as those below the water table, will be evaluated.

The factors that influence DCP values for fine-grained soils that will be investigated include density, moisture content, gradation, and the confining pressure. How some of these factors affect the relationship of the DCP values to the resilient modulus values will be addressed as well as the sensitivity of these variables. The influence of factors such as coefficient of uniformity, maximum aggregate size, and confining pressure on DCP values of granular materials will also be investigated.

1.7 Objectives

The objectives of the study are:

1. To review the existing penetration testing procedures: Standard Penetration Testing (SPT), Cone Penetration Testing (CPT), Dilatometer, Pressuremeter, and DCP testing. Their history, testing procedure, general application and previous studies relating to their application in the characterization of materials for pavement design will be reviewed. In addition, the availability of correlation of resilient modulus with values determined from these methods will be assessed. The relative advantages and disadvantages of each method will be summarized and discussed.

2. To modify the DCP apparatus and triaxial cell for the application of confining pressure when conducting DCP testing to simulate the effect of lateral stress at depth. The confinement level will enable correlation of the DCP values with the resilient modulus to be made on a similar confining pressure level.

3. To experimentally determine DCP values for 15 fine-grained soil samples under the following circumstances:

    a. Confining pressure = 3 psi

    b. Moisture content = optimum and optimum +20%

    c. Density = maximum dry density corresponding to moisture states

4. To investigate the effects of the moisture content on the DCP values of fine-grained soils and to determine if there is a significant statistical difference when comparing the values at each moisture content level for the 15 soils. The approach is to use a paired t-test and evaluate the significance at the 5% significance level.

5. To determine the DCP values for 7 randomly picked samples from the 15 fine-grained soils at approximately 100% maximum and 110% maximum dry densities under the following conditions:

    a. Confining pressure level = 0 psi

    b. Moisture content = optimum

    c. Density = 100% maximum and 110% maximum dry density

6. To investigate the effects of the increase in density on the DCP values of fine-grained soils. Paired t-tests will be used to determine whether there is a significant difference when tested at the 5 % significance level.

7. To experimentally determine DCP values for six fine-grained soils under the following circumstances:

   a. Confining pressure = 0, 15, and 30 psi

   b. Soil types = A-6 and A-7

   c. Moisture content = optimum

   d. Density = maximum dry density

8. To investigate the effects of confining pressure and AASHTO soil classification types on the DCP values and to determine if there is a statistical difference when comparing these values for each confining pressure levels and soil type .

9. To correlate the DCP values with the resilient modulus for fine-grained soils at the following conditions:

   a. Confining pressure = 3 psi

   b. Moisture content = optimum and optimum +20%

10. To develop a general regression equation for fine-grained soils between resilient modulus as the dependent variable and the DCP values as the independent variable at the following conditions:

    a. Confining pressure = 3 psi

    b. Moisture content = optimum and optimum +20%

11. To experimentally determine DCP values for 6 granular materials:

    a. Aggregate grain size = sand, 3/8-inch maximum aggregate size, and 3/4-inch maximum aggregate size

    b. Confining pressures = 6, 12, and 18 psi

12. To investigate the effects of maximum aggregate size and confining pressure on the DCP values for the granular materials. A contrast analysis will be carried out to determine any significant trend the confining pressure had on the DCP values.

13. To correlate DCP values with the coefficient of uniformity and maximum aggregate size of granular materials tested at confining pressures of 6, 12, and 18 psi.

1.8 Scope

The study covers both fine-grained soils and granular materials found in the state of Oklahoma. Fine-grained soils come from various parts of Oklahoma selected in a previous landmark study of soils across the state (11). The selection of fine-grained soils can be considered to be based on a stratified random sampling which had been designed to suit that study. The choice of the two moisture states selected in the study was to enable the correlation with available data obtained from resilient modulus testing. The fine-grained soil samples investigated were all remolded samples. Granular materials and aggregates were randomly selected based upon typical conditions found in construction sites across the state. Two sets of granular materials were used in this study, one supplied by the Oklahoma Department of Transportation (ODOT) and the other supplied by Oklahoma State University (OSU). The samples were randomly selected on the basis of maximum aggregate size.

1.9 Limitations

A limitation of this study is the limited number of data that can be used in the correlation of the resilient modulus with the DCP values for the fine-grained soils. The resilient modulus tests were determined by ODOT for a previous study. Samples used in the present study were restricted to the available samples from that study. Possible variation in the experimental design was also restricted by this situation.

The height of the exterior sample mold used in preparing the specimen is only 16.5 inches high and this limits the effective sample height to approximately 12 inches. Invariably, this restricts the readings that can be obtained from the DCP tests for weaker soils. The second moisture content level that DCP testing was carried out for the fine-grained soils, i.e. at 20% of the optimum moisture content above the optimum moisture content (referred to as optimum moisture content +20%) made the soils too soft. This limits the number of DCP readings that can be determined in view of the restricted sample height. The moisture content was chosen in order to be consistent with the resilient modulus testing.

1.10 Rationale

The choice of this study for the dissertation is very challenging to the student because of his work involvement in highway design, construction, and maintenance for the Public Works Department in Malaysia. In dealing with his daily work, decisions regarding subgrade soil, base or subbase material utilization, pavement design and pavement rehabilitation techniques have to be made. Invariably, the assessment of soil strength is a vital element in this field. In this respect, the subgrade soil strength, subbase, and base course characterization method plays a significant role in the decision making process. Consequently, the evaluation of the DCP as a possible apparatus to be used to rapidly characterize the subgrade soil is very challenging.

The government of Malaysia devotes a significant proportion of its annual infrastructure budget on new highway projects, rehabilitation, and normal maintenance of existing highways and roads. Sometimes the financial allocation may have to be adjusted in a manner that road projects need to be constructed or completed within a short period of time. Obviously, designs are supposed to be done quickly. Thus the availability of a device that can be used to rapidly characterize the materials to be incorporated in the design is a tremendous boost.

CHAPTER II

PENETRATION METHODS

2.1 Soil Investigation Techniques

2.1.1 Soil Conservation Map

Soil conservation survey maps are widely used in the U.S. These maps are based on pedologic classification of soils found within a specific area. They provide reliable soil data for cross country routes. The design of low volume or minor roads may sometimes be based on this kind of data. Such maps may also serve as a good starting point if the need arises for a thorough soil investigation. Alternatively, only confirmatory tests may be needed to validate the soil map data as soon as the route of a specific project is finalized. These data are regularly updated by the relevant authorities (1). The accuracy of this method is approximately 80% as determined by the presence of soil types which are different than those predicted.

2.1.2 Direct Testing

The resilient modulus and the CBR value are design input parameters for the AASHTO method and the Corps of Engineers-CBR method, respectively (5). For large-scale projects such as interstate highways, state highway agencies generally perform the actual test to determine these parameters. The determination of either the resilient modulus or the CBR is a highly detailed procedure which takes several days to complete. This includes the time it takes for field sampling, sample preparation and the actual testing. However, in view of the available correlations between these inputs and various soil classifications, agencies sometimes use such correlations for the resilient modulus or the CBR design inputs on small scale projects (2).

2.1.3 Geophysical Methods

Geophysical methods are indirect methods which rely on the principle of physics to estimate soil properties. Some of the procedures included in this category are seismic refraction, electrical resistivity, gravimetric survey, magnetic surveys and ground penetrating radar. Generally, geophysical methods are able to provide layer identification, stratification and subsurface irregularities which may be useful in the evaluation of a small scale reconstruction or rehabilitation projects. These methods may show bedrock location and general information on its rippibility can be inferred (12). However, using data obtained from geophysical methods for pavement design may be impractical and is considered too imprecise by many engineers. It is recommended that borings and test excavations be carried out in conjunction with geophysical methods. Measurements from geophysical methods can be made rapidly and even though the cost of this equipment is generally not prohibitive, they may require a high level of skill to operate. Precision of measurement is high in all methods, but the accuracy of interpretation and inferences depends very much on the experience of the interpreter (13). Some highway agencies rely on such methods for reconstruction and rehabilitation projects as well as for preliminary investigations (2).

2.1.4 Boring and Sampling

Borings and samplings are occasionally used in the soil investigation for pavement design to determine soil classification or for obtaining samples to be tested for the required strength/deformation parameters. Generally, soil can be classified under either the textural, AASHTO, FAA, or Unified Soil Classification system. Sieve analysis and the Atterberg Limits of these soil samples are required for classification. This method of soil or material characterization can also be useful when unusual circumstances arise such as the need to confirm a doubtful strata determined through either geophysical methods or the penetration methods. Only subgrade soil that is within two ft of the planned subgrade elevation are generally tested (13). Soil classifications have also been used as inputs in the design of secondary roads through the relevant correlation.

2.1.5 Nondestructive Testing

This group of tests which was discussed in the previous section provides engineers with an overall pavement characterization without physical sample testing. However, NDT methods are more widely used for the design of overlay or rehabilitation projects than for the design of new pavement. NDT evaluation techniques are based on their type of loading, i.e impulse, steady state dynamic, or static (3, 15).

The methods mentioned in sections 2.1.1 through 2.1.5 are some of the techniques that can be considered for soil investigation for pavement design for soil or material characterization. However, it is the potential of the rapid investigation methods such as the standard penetration test (SPT), the dynamic cone penetrometer (DCP), the dilatometer test, the pressuremeter test, and the cone penetration test (CPT) which are of major interest. Current literature reveals penetration tests have been widely used on secondary or low volume roads but on major facilities such as airport runaways, their employment has been primarily experimental. Technological developments may one day make this group of tests useful in the design and evaluations of major highways as well.

2.2 Penetration Testing Methods

This discussion focuses on the standard penetration test, the cone penetration test, the dilatometer and the pressuremeter. A more detailed review of the DCP is included in the next chapter. Penetration methods are used mostly for identification and stratification purposes; however, the information from some of these tests has been correlated to strength parameters used in pavement design. The major contribution is through the correlation they provide to the bearing capacity parameters of the soil or other materials in the pavement, i.e., the resilient modulus or the CBR. Only the South African State Highway Agency of Transvaal and The Victoria Country Roads Board of Australia have developed a relevant pavement design procedure making use of the DCP values directly for low volume roads (14). The wider application of the penetration methods occurs in the design of bridge foundations, buildings and other structures where the penetration values of the soil are more directly used to evaluate the allowable bearing capacity of the soil and its related expected settlement.

The world recognition of penetration methods has been reflected by many conferences held specifically on this particular topic. The European Symposium on Penetration Testing (ESCOPT) in Stockholm, Sweden, in 1974 and its subsequent event in Amsterdam, Holland, in 1982 and the First International Symposium on Penetration Testing in Orlando, Florida, in 1988 are notable events in this area. In addition, international and regional conferences on soil mechanics have devoted whole sessions to this topic. It is anticipated that more widespread use of penetration methods will come about as their level of sophistication increases.

2.3 Standard Penetration Tests

2.3.1 History Description and Procedure

The Standard Penetration Test (SPT) is an empirical test developed in 1927 designed by Fletcher and Mohr (17). The SPT number is defined as the number of blows required to drive a 2-inch sampler 12 inches into the ground by a 140-pound weight falling through 30 inches. According to Fletcher, the first recorded version of the test was carried out in 1902 by Lt. Colonel Charles R. Gow when a 1-inch open drive sampler was driven into the ground by a 110-pound hammer. The present method involves driving a standard sampler a distance of 18 inches from the base of a bore hole of which the first 6 inches are considered the seater. Basically, the original concept still prevails. Interestingly, the standard of the United States, A.S.T.M. 1586-67, became the pioneer in which many other countries follow, some with minor modifications (16, 18).

The description of the equipment is best illustrated with the aid of Figure 4. The equipment consists of a boring element, a tubular steel sampler called the split spoon sampler, the hammer assembly, and a steel drive rod connecting the hammer assembly to the sampler. Sometimes mud or water is used in the procedure to facilitate the boring process.

The procedure of determining the SPT number begins with the driving of a steel casing to the required depth from which borings are to be made. Water is then forced inside this casing which pushed the soil bits out. The mast of the rig is then hoisted into place and the rod with the sampler together with the hammer equipment are positioned

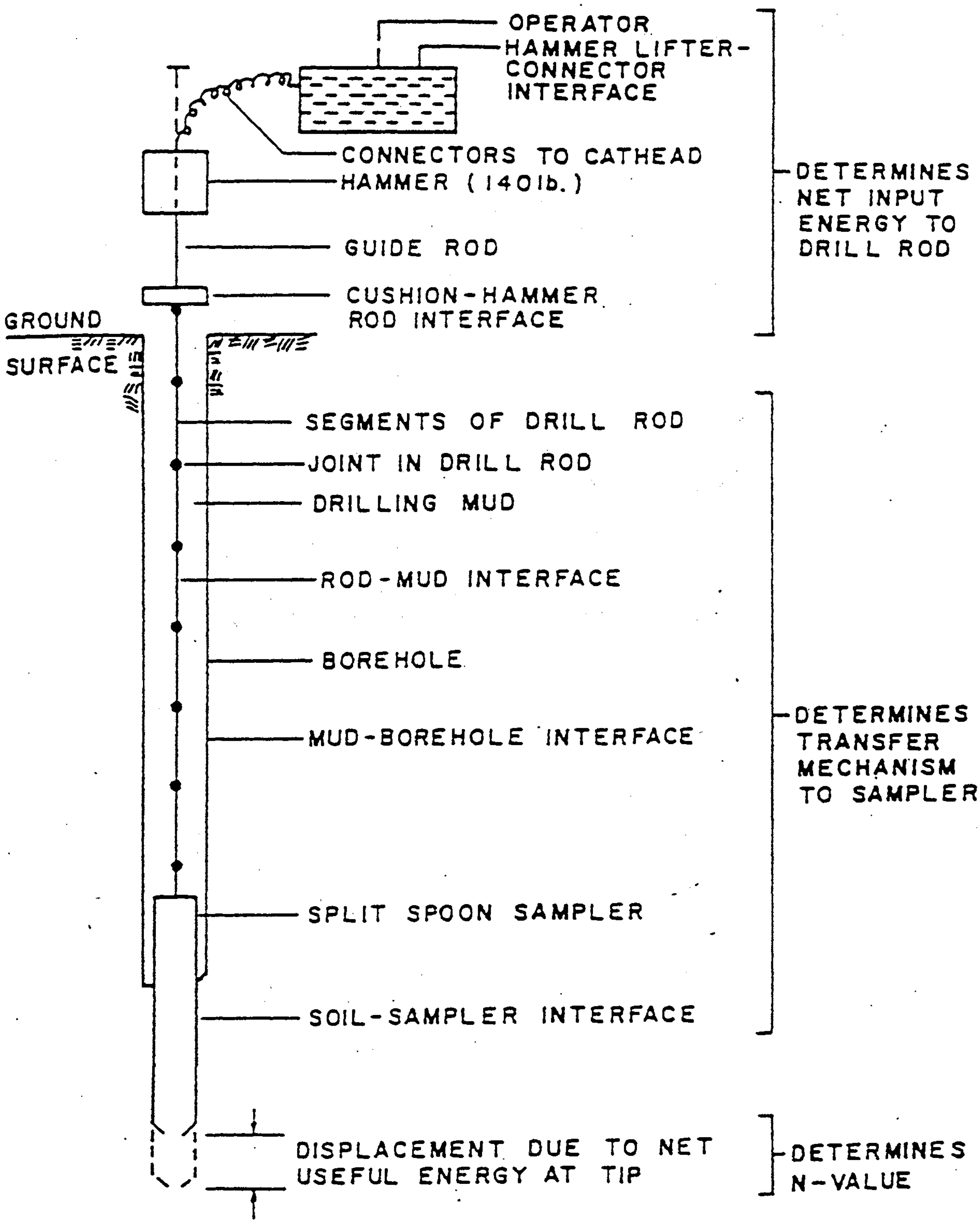


Figure 4. Schematic Overview of Interacting Components of Standard Penetration Test (12)

before the driving process begins. The hammer is first driven about six inches into the ground to serve as the seater. This allows the material, made loose while being driven, to be recompacted again. Upon completion of the seater, the hammer is then driven at 3 hammer drops per minute to reach the next 12 inches. The number of drops required to reach the 12 inches is known as the penetration resistance value of the material being tested (18).

2.3.2 General Uses

The SPT has generally been used to determine soil parameters for granular materials. Only occasionally has the test procedure been used on fine-grained soils. Its use in rocks, gravel, and silt is reportedly less encouraging. According to the Army Corps of Engineers manual (13), apart from identification purposes, the engineering properties that can be obtained using the SPT equipment are the determination of the shear strength, bearing capacity, mass deformability, relative density, and liquefaction susceptibility assessment. Correlations to these properties and procedures to obtain them have been developed and are widely used. For pavement design purposes, the identification and stratification of the subgrade materials are correlated to the relevant strength criteria if the resilient modulus or the CBR have not been directly determined. Adjustments are needed due to the influence of relative density, the water table, and the presence of saturated fine and silty sand. In this respect, some researchers have expressed doubts whether the SPT can be applied to clay and cohesive soil. However, the consensus among most authorities is that the SPT, when used with care, is acceptable in all types of soil. The user must be aware of its limitations and precautions.

The SPT has become the principal means of determining the permissible ground pressure to be used for the design of shallow foundations both in sand and in clay. However, many countries use the SPT for evaluating a wide range of materials. Table 3 summarizes the usage, practice, and application of the SPT in various countries as presented at the First International Symposium for Penetration Testing in Orlando, Florida, in 1988 (18).

2.3.3 Correlation of SPT to CBR as Input for Pavement Design

A study was conducted in Israel in 1987 to determine the relationship between penetration resistance (N) of the SPT and the CBR value various soils. In the study, two correlation equations were postulated, one for CBR values of up to 30 and the other for CBR values exceeding 100. However, only one equation was reported (19).

The equation for soils whose CBR exceeds 100 was given by

$$\log CBR = 2.2 - 2.1\ [\log (0.309\ SPT)]$$

where SPT is 305/N, and N is the number of standard blows to a depth of 12 inches.

SPT measurements were performed on the main runway of Ben Gurion Airport for the granular subbase and base course materials, and a correlation was made to determine the CBR values. When these values were compared with direct CBR measurements taken earlier, they were reported to be quite satisfactory. Since the coefficient of correlation and standard error of the estimate was not mentioned in this study, the precision of the work cannot be assessed. However, this study which was conducted in granular subbase and base course materials does serve to reinforce the applicability of the SPT to cohesionless soils (19).

2.3.4 Correlation of the SPT With the Resilient Modulus

In a study of railroad subgrades at four sites in various parts of the United States, Bukoski et al. (20) correlated the SPT values with the resilient modulus. Two SPT tests and several static and cyclic unconsolidated undrained triaxial test were carried out to determine the resilient modulus for each case. Their correlation is shown in Figure 5. The study was prompted by the need for a speedier test method to verify the bearing capacity of an existing railroad track. In this experiment, the two values of the resilient modulus were averaged from two static tests to compensate the bias due to the decrease of the ratio of the cyclic shear stress to the mean stress with depth. Since there was no presentation of the statistical analysis, it is difficult to say how well the regression line fits the strength of coefficient of correlation or the value of the standard error of the estimate (20).

TABLE 3

USAGE AND PRACTICES OF SPT IN SOME COUNTRIES [18]

Country	Usage		Materials range	Practice								
	Widely	Occasionally	S = sands & silts C = clays G = gravels WR = weak rocks	Own standards	Other basis	Main types of hammer	Solid cone	Wash boring	Percussion	Auger	Rotary bit	Casing/mud
Australia	●		S.C.G & WR Interbedded soil/rock. Suspect in gravels.	●		trip	●		●	●	●	●
Brazil	●		S.C & G	●		manual		●		●	●	●
Canada	●		Glacial till etc.	●		manual & trip						
Czechoslavakia		●	S & C	●		manual & trip					●	m
Greece		●	S & C		T & P48 EM74	manual					●	c
India	●		S & C	●		manual		●			●	m
Israel	●		S & WR Interbedded soil/rock. Kurkar		ASTM 67	trip		●		●		m
Italy		●	S.C & G		ASTM 67	trip						
Japan	●		S.C & G	●		manual & trip					●	●
Norway		●	S & G		T&P48						●	c
Poland		●		●								
Portugal	●		S.C & G Drilling mud in loose sands	●		manual & trip	●		●			c
South Africa	●		S.C.G & WR Spring retainer for sands		ASTM 67	manual & trip	●	●	●	●	●	m
Spain	●		S.C & G Solid cone for gravels		T&P48	manual	●		●	●	●	c
Turkey	●		S.C & G, also over-consolidated clays		ASTM 67	manual			●		●	
United Kingdom	●		S.C.G & WR Solid cone for G & WR	●		trip	●		●	●	●	c
U.S.A.	●		S.C & G	●		manual & trip		●		●	●	●
17. Totals:	12	5		10	manual—11 trip—10		5	5	6	7	12	–

Notes: T&P 48 – Terzaghi, K. & R.B. Peck. 1948. *Soil Mechanics in Engineering Practice*
EM 74 – Earth Manual 1974
ASTM 67 – D 1586:1967. Standard method for Penetration Test and Split-barrel Sampling of soils

2.3.5 Limitations

One of the major limitations of the SPT is its suspected validity when used for silt, rocks, or gravel. Since many projects involve these types of soil, this may limit the equipment's usefulness. Second, variability in the SPT readings may occur if it is accompanied by poor boring methods and inconsistent energy losses during impact. Third, hammer drop height is generally based upon visual judgment, making the reliability and competence of the operator a very critical aspect of the test. In a similar respect, the operator must ensure the rod is vertical to obtain reliable results. The correlation provided by the SPT is empirical in nature and as such its reliability is dependent upon the quality and amount of data collected. This correlation is also affected by factors such as moisture content of the soil and depth at which the measurements are made (18).

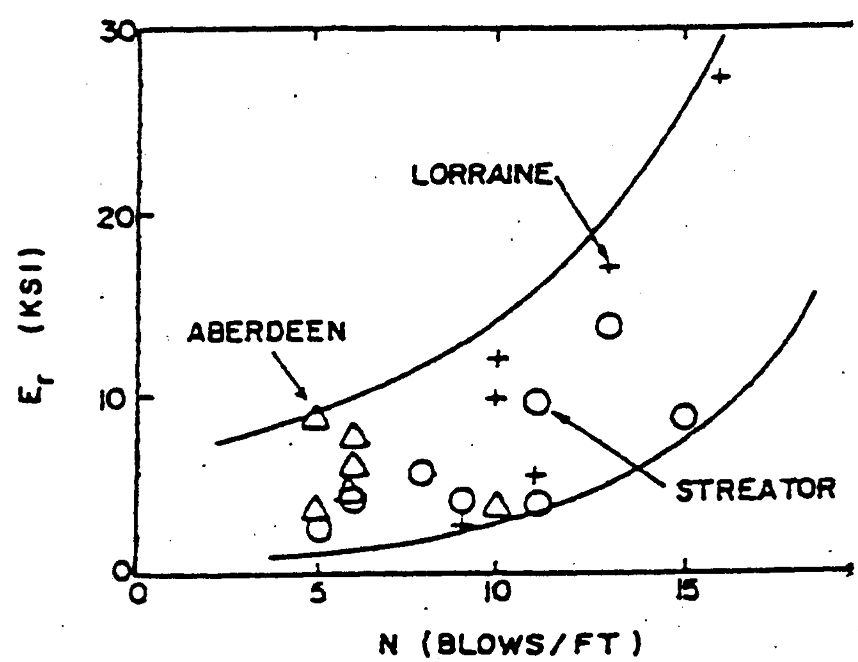


Figure 5. Relationship of SPT Values to Resilient Modulus [20]

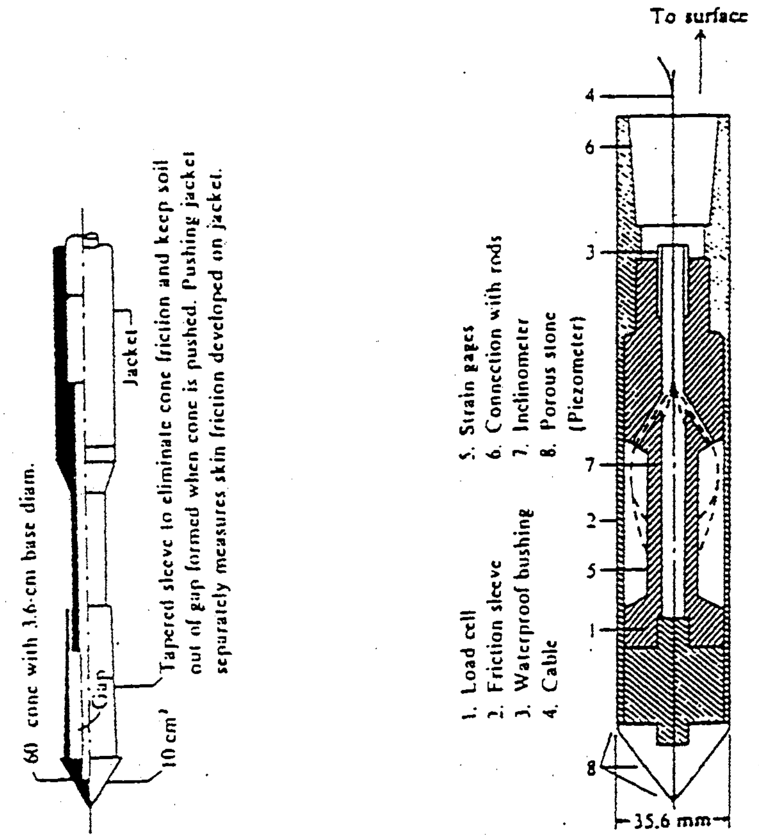
2.4 Cone Penetration Test

2.4.1 History Description and Procedure

The Cone Penetration Test (CPT) device is an instrument that has been widely used for the last three decades to classify soil and measure its strength. It is a simple static device that enables soil parameters to be rapidly obtained. The equipment which consists of a series of cylindrical rods with a cone called the penetrometer tip at one end. The cone contains a device that enables the measurement of the tip resistance and the side or the shaft friction as the cone is pushed into the soil. Generally, cone penetrometers are either electrically or mechanically operated. Electrical cone penetrometers use electrical device such as strain gauge built into the tip while the mechanical penetrometers use inner rods to operate an inner tip to measure the advancing rate. Another type that has recently been added is the hydraulic or pneumatic penetrometers which uses pneumatic devices built into the tip for the measurement of the penetration rate (21).

Figure 6 is an illustration of the types of CPT devices discussed above. Since the advantage of the nonstop measuring capability of the electrical type is tremendous, presently most CPT devices are of the electrical type. Recent developments in CPT technology includes accessories which have been added to measure other parameters such as pore water pressure and water quality (22).

The procedure begins with the positioning of the rig or the machine used to thrust the device into the ground. The cone attached to a series of rods is pushed into the ground at a constant rate, usually at 20 mm per second. Measurements of the tip resistance and the side friction are made. The ratio of the side friction to the tip resistance, termed the friction ratio can be evaluated from these measurements. Calibration of measurements of the tip resistance and the friction ratio allows the determination of the soil classification and also the subsurface ground profiles.



Mechanical Electric

Figure 6. Schematic of Mechanical and Electric Cone Penetrometers [21]

2.4.2 General Use

Generally the CPT has been used in the soil classification and its determination of soil profile, the determination of strength parameters such as the angle of friction, undrained shear strength and its density. It is well suited for both the fine-grained soils and granular materials soil, silt, and even peat. CPT devices have been used to predict bearing capacity of piles and are sometimes used to check compaction. Since there are many CPT types, each has its advantages and disadvantages. The fact that CPT devices are simple, inexpensive, and will allow continuous data collection with depth makes this equipment very attractive.

Mitchell (22) reported that the CPT has become the most popular penetration test for the investigation of soils (that do not contain gravel and other obstructions) which can be penetrated using a reaction load of up to 20 tons. Recent technical developments in the CPT area have enabled extra features to be added. The dual range penetrometer which allows for the variation of tip resistance sensitivity has been developed—one for clay and the other for sand. Also, there is now a cableless cone system that does not require a cable for data transmission. Other devices that have recently been developed are acoustic cones that utilize acoustic signals and lateral stress cones which can be used to measure lateral earth pressure. Additionally, CPT devices with piezometers to measure pore water pressure and other devices that will allow water quality to be gauged have also been developed.

2.4.3 Correlation of CPT to Resilient Modulus

The Florida Department of Transportation has conducted a study to correlate the CPT data with resilient modulus. The results were excellent for the subbase and base course materials (coefficient of correlation of around 0.9) but relatively poorer for the subgrade (coefficient of correlation of around 0.7). The data in Table 4 represent these findings. The poorer coefficient of correlation in the subgrade soil may be attributed to the natural variability of the soil as compared to the more homogeneous subbase and base course materials (23).

TABLE 4

CORRELATION COEFFICIENT BETWEEN RESILIENT MODULUS (M_r) AND CONE RESISTANCE (q_c) (23)

	Dynaflect Moduli			FWD Moduli		
Layer	Regression Equation	R^2	n	Regression Equation	R^2	n
Base	$E_2 = 15.02\ q_c$	0.88	11	$E_2 = 8.97\ q_c$	0.92	11
Subbase	$E_3 = 14.23\ q_c$	0.92	14	$E_3 = 8.49\ q_c$	0.88	14
Subbase	$E_4 = 10.87\ q_c$	0.71	12	$E_4 = 11.32\ q_c$	0.67	12
All	$M_r = 14.21\ q_c$	0.87	37	$M_r = 9.13\ q_c$	0.83	37

In another study to correlate CPT data to the resilient modulus on a railroad bed, Bukoski and Selig (20) found the correlation to be generally good except for sites having a high proportion of gravel. In each of the four sites that were verified, seven CPT tests were conducted. Additionally, a total of 21 static and 10 cyclic undrained triaxial tests were performed at the field water content, enabling the resilient modulus to be obtained. The correlation between the CPT and resilient modulus is shown in Figure 7.

2.4.4 Correlation of CPT to CBR

There are no reported studies that correlate the CPT values with CBR and nothing was found on the use of CPT data directly in pavement design.

2.4.5 Limitations

Correlations of CPT results with engineering parameters and performance should be viewed with caution. Meigh (24) advised that correlations vary with locality and therefore are site specific. Also, care should be exercised when interpreting results as there are cones which have different cone angles. The findings of Bukoski (20) and the

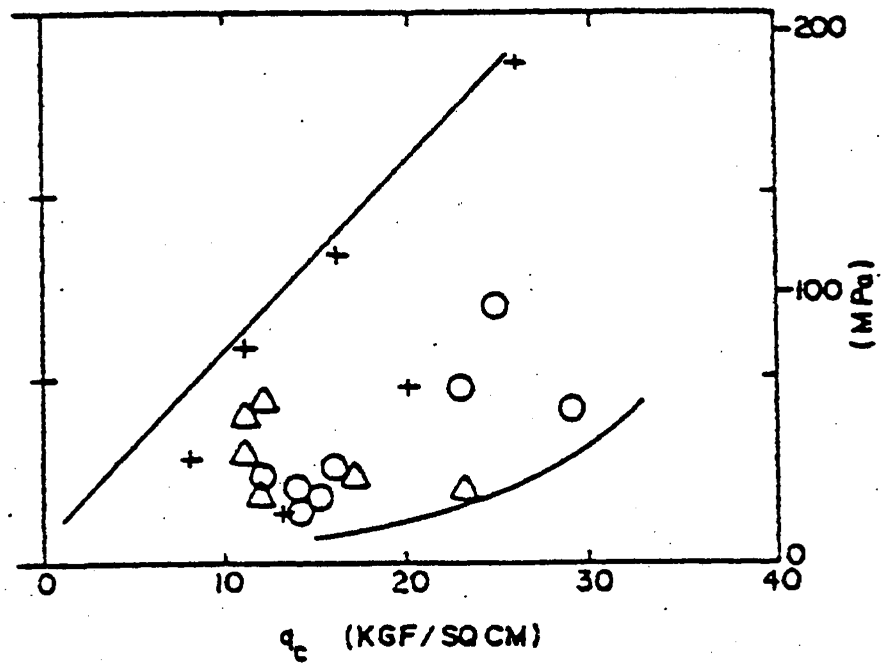


Figure 7. Correlation of Resilient Modulus to CPT Values [20]

recommendations of Mitchell (22) indicate the CPT works well with most types of soil except for gravel. However, since the CPT method does not lead to the recovery of samples, it must normally be supplemented by boring procedures (25).

The use of CPT device requires careful handling in ensuring the straightness of the rod since rod bending may greatly affect the results. Additionally, regular inspection must be done to ensure the satisfactory condition of the cone, friction sleeve and the shaft. When dealing with the electric penetrometer, experience has shown that it must be temperature compensated. In this respect, when there is a tremendous amount of change in temperature, it has been advised that the readings be discarded (25).

2.5 Dilatometer

2.5.1 History Description and Procedure

The flat plate dilatometer or the Marchetti dilatometer refers to the same equipment invented by Marchetti in 1980 (26). The dilatometer, which is sometimes called the DMT, is an instrument that primarily measures the load deformation characteristics of soils. Figure 8 is a schematic of the instrument (12) . It enables readings such as the pushing pressure, lift-off pressure, and also the pressure to push the membrane or blade one mm into the soil. From these values, parameters such as the material index, the horizontal index, and the dilatometer modulus can be determined. The dilatometer possesses most of the preferred qualities of an in situ test device, i.e it is simple, rugged, nonelectronic and gives reproducible results (26).

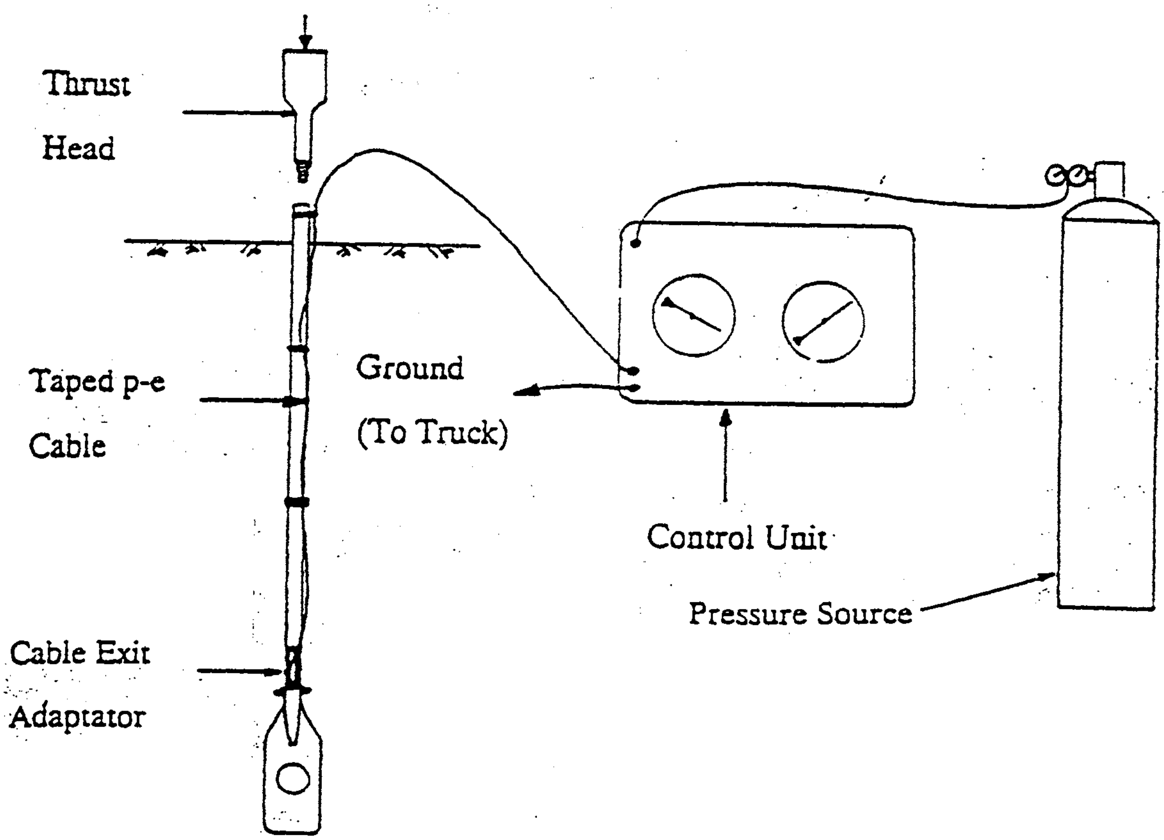


Figure 8. Schematic of the Dilatometer (12)

2.5.2 General Uses

The dilatometer has been used in a broad range of cohesive and cohesionless materials. However, its use in gravelly deposits may cause damage to the blade or diaphragm. One of its main uses is to provide an estimate of soil parameters such as the undrained strength, its compressibility and the lateral stress ratio, K_o. Since it can be carried out from depths as shallow as 8 inches, it can be particularly useful for checking pavement subgrades.

One of the data provided by the dilatometer is the material index which can be considered as a quick way of identifying the soil type. This is useful in the input for the design of secondary and low volume roads. Another use of the Dilatometer is the deformation characteristics of the soil. In this respect, the constrained and elastic modulus can be developed from the Dilatometer Modulus (E_D) which is obtained from the equation,

$$\text{Dilatometer Modulus } (E_D) = 3.47\,(P_1 - P_o)$$

where P_o is pressure to cause initial lift off of the membrane, and P_1 is pressure to cause 1 mm expansion of the membrane.

2.5.3. Correlation of Dilatometer Modulus to Resilient Modulus, CBR, and Modulus of Subgrade Reaction (M_r)

In pavement design and evaluation, the Resilient Modulus, CBR, and Modulus of Subgrade Reaction (M_R) have been correlated with the Dilatometer Modulus. Tweneboah et al. (23) found the relationship of Dilatometer Modulus with the Resilient Moduli obtained by the Dynaflect and the Falling Weight Deflection (FWD) method and the average coefficient of correlation was in the range of 0.8. Interestingly, in the same study, the correlation coefficient for cone resistance from the CPT is found to be slightly higher. Table 5 shows the comparison of these correlations.

TABLE 5

CORRELATION BETWEEN RESILIENT MODULUS (M_r) AND DILATOMETER MODULUS (E_D) [23]

	Dynaflect Moduli			FWD Moduli		
Layer	Regression Equation	R^2	n	Regression Equation	R^2	n
Base*	—	—	—	—	—	—
Subbase	$E_3 = 4.52\ E_D$	0.76	14	$E_3 = 2.64\ E_d$	0.82	14
Subbase	$E_4 = 2.24\ E_D$	0.72	12	$E_4 = 1.83\ E_d$	0.89	12
All	$M_r = 3.77\ E_D$	0.70	26	$M_r = 2.37\ E_d$	0.81	26

*DMT not conducted in base course layer.

2.5.4 Limitations

One disadvantage that has been discussed is the limitation of the dilatometer when used in gravel deposits. Apart from the possibility of providing a false reading, it might also damage the equipment. Also it has been reported that on entering a soft or saturated cohesive material, the blade is capable of creating a cavity expansion failure condition leading to the creation of a large excess pore water pressure. This provides an accurate estimation of the undrained shear strength from the initial pressure required to cause the lift-off of the membrane, the P_o value. However, in stiffer overconsolidated material, further expansion of the diaphragm is required to cause failure which leads to the belief that estimates of the undrained shear strength by this method is lower when compared to other methods (26).

2.6 Pressuremeter

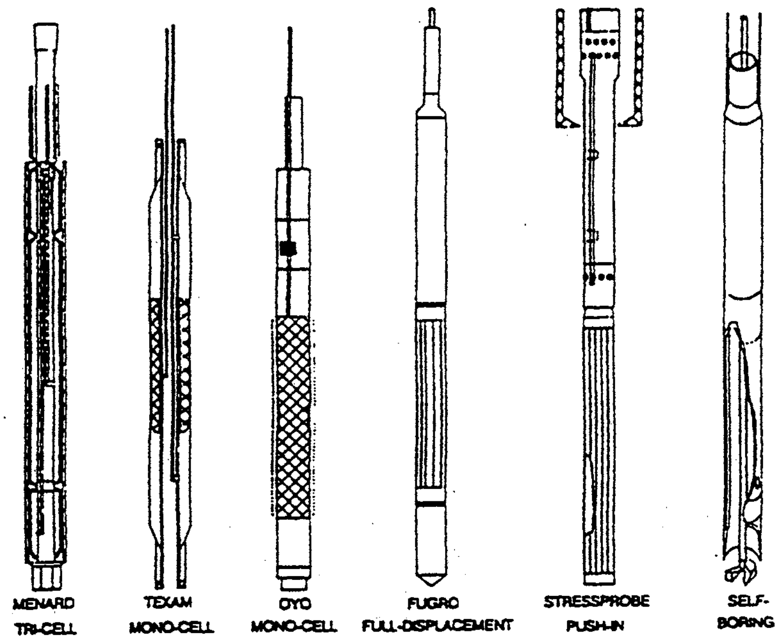
2.6.1 History Description and Procedure

The pressuremeter is an in situ device that has a rubber membrane which expands cylindrically when pressured with fluids. It was developed by Menard in 1950 (27). Currently, pressuremeters can be divided into four general categories: Prebored, Self Boring, Full Displacement, and Push In according to the method by which the probe is inserted. Figure 9a shows diagrams of the various pressuremeter types. Generally, pressuremeters consist of three components—a cylindrical probe, the control unit, and the tubing that connects the two as illustrated in Figure 9b (27).

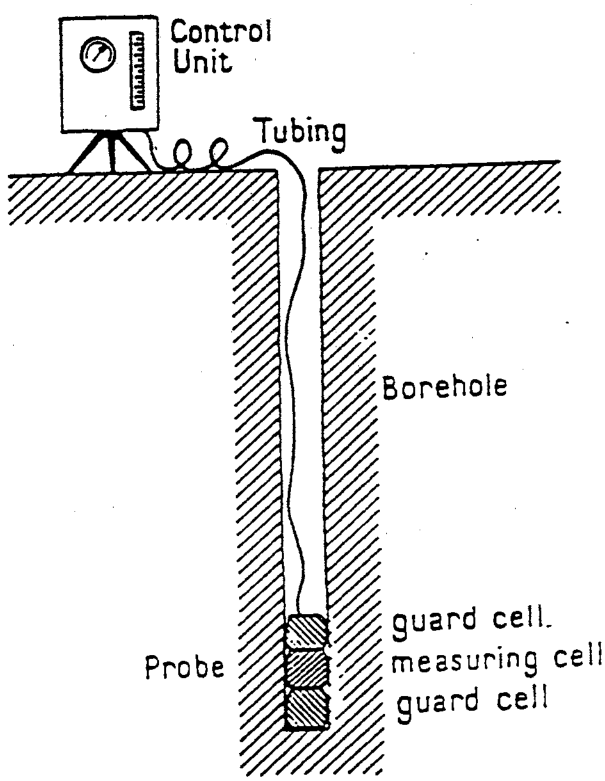
The pressuremeter is a versatile device that can be used in nearly all types of soil. The operating principle in all types of pressuremeter equipment involves the expansion of a cylindrical device against the sides of a borehole under increments of pressure. It can be operated either by controlling the stress or strain. During testing the probe is inflated, creating a pressure against the wall of the bore hole. Throughout the test, the pressure is recorded from a pressure gauge while the increase in volume is recorded through the indicator in the control unit. The test results are expressed as pressure versus volumetric or radial strain and can be used to estimate the soil strength required for engineering design in both theoretical and empirical procedures. The pressuremeter will generally cause some soil disturbance except when using the self-boring type (27).

2.6.2 General Uses

The main purpose of the pressuremeter is to gather soil deformation data due to the various applied pressures at specific depths. From these readings, soil parameters such as compressibility, undrained shear strength for clays or weak rocks, angle of shearing resistance for sands, bearing capacity, settlement potential, and in situ horizontal stress can be inferred. According to Schmertman (17), the use of the pressuremeter offers three advantages over other in situ tests. First, the assumed model is well suited for the behavior of real soil. Second, the test, when properly conducted, can provide an estimate of the in situ horizontal stress of the soil. Finally, stress and strain data of the



(a) Various Pressuremeter Types



(b) Menard Pressuremeter Operation

Figure 9. Schematics of Pressuremeter Types and Operation (27)

soil in the direction perpendicular to the axis of the expanding cavity can also be determined from the test.

2.6.3 Correlation to Pavement Strength Parameters

Pressuremeters have been correlated to other material strength characterizing methods. The idea of using the pressuremeter for pavement design can also be attributed to Menard. Transport Canada and the Texas Department of Transportation jointly funded a research project to evaluate the use of the pressuremeter in pavement design (28). This project revealed that the pressuremeter can provide a profile of moduli which will allow an assessment of the pavement stiffness. Additionally, the pavement pressuremeter test can measure the subgrade reaction value to be used in the design of rigid pavements. In overlay design, the profile of limit pressure during the test can be used to assess the maximum load carrying capacity of a pavement. By repeating the inflation and deflation of the probe, the magnitude of the permanent strains in the pavement and its rate of deterioration can be evaluated. This project report showed the potential of the pressuremeter in pavement design and evaluation (28).

A study to correlate the resilient modulus, as determined by the pressuremeter, and the CBR in the state of Texas compared very well with similar studies. The pressuremeter equipment used in this study was the TEXAN PMT. Six unload/reload cycles at 10, 20, 30, 60 ,120 and 240 seconds were conducted. It was concluded that except for the 10-second cycle length, the apparatus can be accurately used to determine the resilient modulus. In conducting this study, the researchers were driven by the philosophy that the resilient modulus may be determined from four types of pavement pressuremeter loading sequences. The sequences are variations of stress and strain levels, loading rate, and number of cycles. One important conclusion from this study was that the pressuremeter could be used to determine the resilient modulus. However, since the resilient modulus as determined by the pressuremeter is governed by the loading rate, designers must be specific about the rate it was conducted. The loading rate is reflected by the cycle time during the test (30).

2.6.4 Limitations

Pressuremeter testing requires a high standard of operator skill and site supervision. In predrilled holes, required by some pressuremeters, installation methods have yet to be standardized in order to make the test reproducible. Another disadvantage reported is the uncertainty of the soil drainage condition around the expanding cavity in fine-grained soil. Additionally, bore hole disturbance can cause a decrease of as high as 50% of the pressuremeter deformation modulus. New types of pressuremeters are now being developed to reduce this disturbance (27). Pressuremeter devices are highly sophisticated and thus contain electronic and mechanical components that often lead to difficulties in calibration and measurement (29).

CHAPTER III

THE DYNAMIC CONE PENETROMETER (DCP)

3.1 Background

In principle, the Dynamic Cone Penetrometer (DCP) is similar to the SPT device except the sampler attachment to the rod being driven into the ground is replaced by a cone. It is unlike the Cone Penetration Test device in that the shaft or side friction effect is not required in its measurement. The DCP test results are reported either in terms of the penetration rate which is the advancement of the equipment from a single fall of the sliding weight or the number of blows per a defined depth of penetration. The single fall is sometimes called a blow.

At the present time there is no uniform world standard for the DCP configuration, only a recognized set of procedures agreed upon by member countries which attended The World Symposium on Penetration Testing in Florida in 1988. These countries seemed to indicate it was not necessary to have a world standard because of the varied practice which has been established in many countries and standardizing them would run contrary to the body of knowledge which has been developed (31).

Against this background, it is not unusual that even the terminology is varied. At the 1988 symposium, the term Dynamic Probing was referred to the DCP. To help classify the various types of DCP at this symposium, the hammer weight was classified under four categories—DPL representing the light category of hammer mass of less than 10 kg, DPM representing the medium category of hammer mass between 10 and 40 kg, DPH representing the heavy category of hammer mass between 40 and 60 kg, and DPSH representing the super heavy category of hammer mass exceeding 60 kg (31).

The DCP test resembles both the SPT and the CPT. It involves dynamic testing as does the SPT and uses a cone tip comparable to that used in the CPT. Although there are similarities, the primary difference between the DCP test and the CPT is the dynamic and the static nature of the tests. Historically, the beginnings of both those methods are similar. The DCP test became as well known as the other methods after the First World War and its use grew more widespread after the Second World War when European consultants introduced it internationally (31).

Development of the hand-held DCP is credited to A. J. Scala of Australia in the mid-1950s. Pavement design procedures in Australia then did not specifically require in situ strength tests of the subgrade soils because of the time and complexity of available test methods. The device Scala developed included a 20-lb drop hammer falling a distance of 20 inches. A 5/8-inch diameter rod calibrated in 2-inch increments was used to determine the penetration. The configuration used a 30° included angle cone tip. Scala conducted tests correlating CBR with DCP data and proposed a pavement design procedure based on this correlation. Use of this DCP device was adopted by the Country Roads Board, Victoria, and gained widespread acceptance (32).

The next generation of DCP equipment was developed by D. J. Van Vuuren from South Africa. Basically it was similar to the DCP apparatus developed by Scala except the weight of the drop hammer was changed to 22 lbs and the drop distance was changed to 18.1 inches. The shaft diameter measured 0.63 inch while the included angle remained at 30°. The development was prompted by the need to alleviate problems associated with performing field CBR tests. In the ensuing study, the CBR/DCP correlation resulted in a better correlation when compared to CBR/CPT correlation. Additionally, Van Vuuren concluded that the DCP is suited for use with soils having CBR values of 1 to 50 (33).

The present version of the DCP used in this study was developed by E. G. Kleyn for the Transvaal Roads Department, South Africa. Van Vuuren's basic design was utilized in Kleyn's work; however, the hammer weight was reduced to 17.6 lbs and the height of the drop was increased to 22.6 inches. Kleyn studied two cone angle configurations of

30° and 60°. The cone angle utilized in this study is based on the 30° included angle. Kleyn's work focused on the development of the generalized DCP/CBR correlation for the full range of materials tested (34).

The schematic diagram shown in Figures 10 and 11 are illustrations of the various DCP configurations available. The DCP shown in Figure 10 was used in this study. Since there is no universal standard specification for the equipment, the DCP illustrated is currently the most commonly used in its weight range. The device consists of a DCP hammer assembly and a graduated steel rod with a cone tip affixed to its end.

Consistent with the various types of DCP devices, the procedure to measure the penetration rate depends on the mass of the hammer and whether the equipment is hand or machine operated. At the start of the test, the rod must be straight and vertical. This requirement may sometimes be assisted by preboring. Deformation of the rod must be avoided as this may cause not only the diversion of the driving energy but also additional skin friction along the rods and thus false results. Driving the rods by the hammer is generally done at driving rates of between 15-30 blows per minute. In cohesionless soils, the driving rate can be increased to 60 blows per minute. The procedure to allow the hammer weights to freely fall from the designated height which enables the driving of this rod must be carefully adhered to for consistency. Results are generally reported as penetration resistance versus penetration depth (31).

3.2 General Uses

The DCP has been developed in response to various problems and uncertainties associated with in situ evaluations, especially when dealing with soils which are difficult to sample. The main uses are for determination of bearing capacity, settlement potential, and to a limited extent classification of the soil being tested.

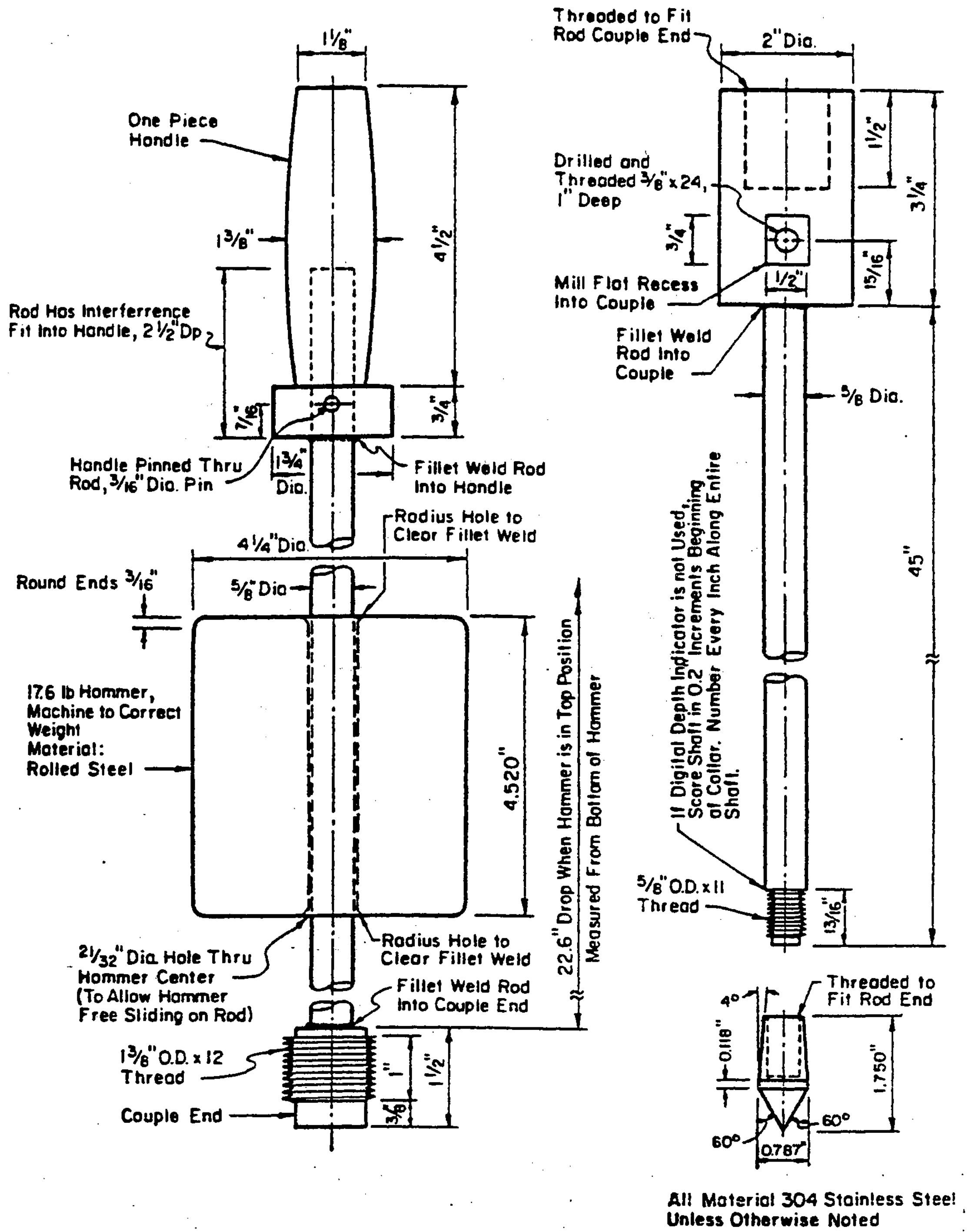
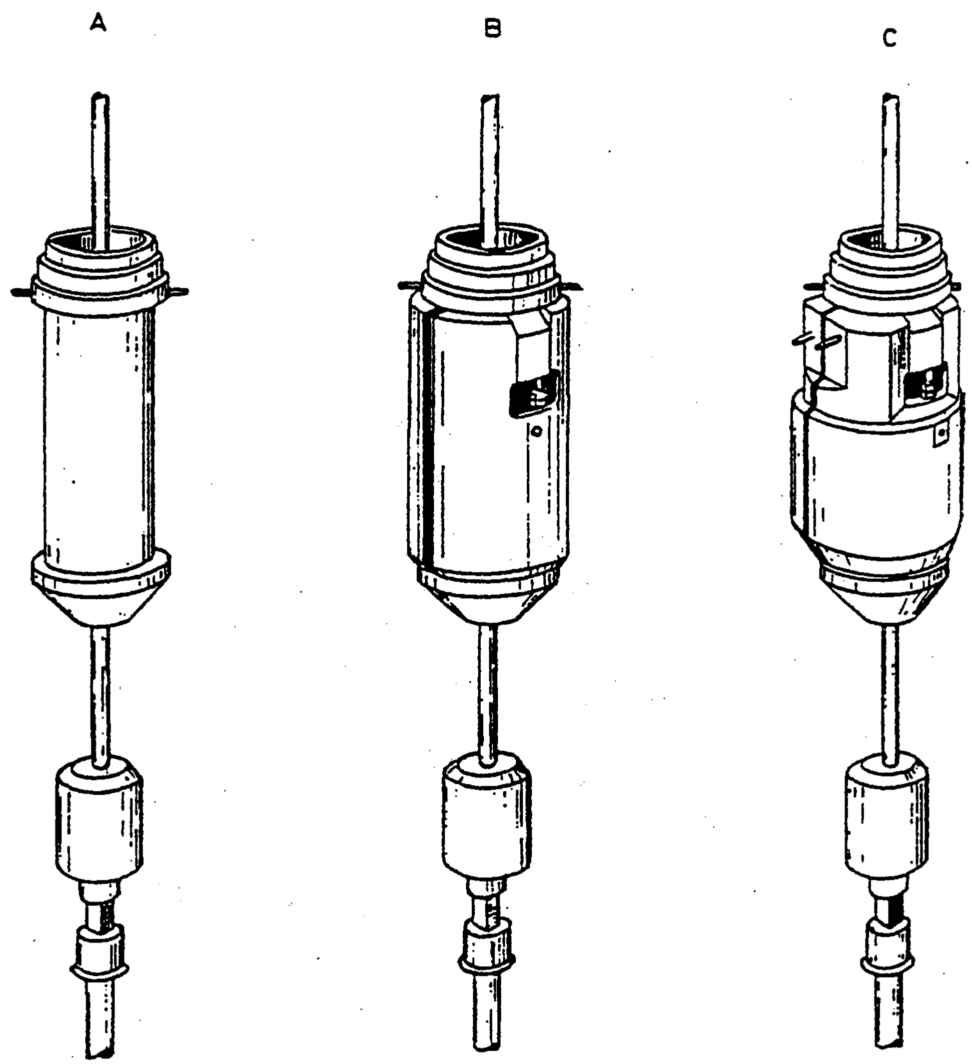


Figure 10. Schematic of Dynamic Cone Penetrometer Used (14)



A. Hammer Drop Weight of 30 kg

B. Hammer Drop Weight of 60 kg

C. Hammer Drop Weight of 90 kg

Figure 11. Sermes Dynamic Penetrometer (21)

According to Sangelerat (21), the DCP test is very reliable in the calculation of bearing capacity of shallow and deep foundations in granular materials. The dynamic resistance of driven piles in granular materials is very close to that measured by the DCP. However, he expressed concern if the DCP is used to calculate the bearing capacity of fine-grained cohesive soil especially if it is lower than the water table. This is due to the side friction now becoming a major component of the dynamic resistance and the contribution of the effect of pore water pressure.

Sangelerat advised that only relative information regarding classification and stratification should be inferred from the penetration rate of the DCP. This is even more critical for cohesive soil. If the layer is beneath the water table, it is felt that it is impossible to determine if an increase in the penetration rate is due to the increase in point resistance, side friction, or change in pore pressure. However, the technical committee at the International Symposium on Penetration Testing in which Sangelerat is also a member recommended the DCP test should only be used as a general assessment of the stratification to confirm the layering of the subsoil, while the soil type itself should be determined from other boring or sampling (31, 35). In other words, the DCP test may be used for all types of soil for the confirmation of a stratification around the vicinity of a position which has been determined by other methods.

3.2.1 DCP Value as a Direct Input in Pavement Design

Kleyn et al. (34) reported the development of a DCP-based pavement design method for thin surfaced unbound gravel pavements in South Africa. A pavement design model was developed and subsequently correlated with the heavy vehicle simulator (HVS) for a number of pavement sections. The South African development was presented through a paper which introduced the concept of the DCP structural number, called the DSN. The DCP structural number provides the layer thickness through the equation

$$\text{Layer DSN} = h/DN$$

where h is the layer thickness, and DN is the DCP test results in terms of mm/blow. The DSN is equal to the number of blows to penetrate a layer, while the pavement DSN is the summation of the individual layer DSN values which made up the pavement. The limiting depth for a pavement DSN was determined to be 800 mm, assuming that stresses at depths greater than 800 mm were insignificant. The percent DSN (X-AXIS) was then plotted against the depth (Y-AXIS) to obtain a pavement strength balance (PSB) curve. An example of the PSB curve is shown in Figure 12. Typical PSB curves used in South Africa are shown in Figure 13. The PSB curve is then compared to curves obtained from field evaluations of various types of pavement conditions using the HVS. In this case DCP values are used as a direct design input to obtain the pavement thickness using this PSB curve. This procedure is currently restricted to low volume roads in South Africa and has prompted other studies to increase its applicability. Another procedure incorporating the direct use of DCP values, developed in Victoria, Australia, was also reported but the details were too vague for presentation here (14).

3.3 Theoretical Basis of the DCP

The theoretical analysis of the DCP device has been the subject of many studies. A summary of the findings of the committee on dynamic cone penetrometer testing during the 1988 Symposium on Penetration Testing stated that it is questionable to evaluate the DCP procedure on a purely theoretical or analytical basis, and many others also believed the analysis of the stress state at the cone tip is a complex subject (16). Numerous studies have attempted to determine stresses in a granular material induced by the advancement of a cone penetrometer tip. A literature search on the study of the theoretical analysis of fine-grained or cohesive soil revealed no such research was conducted. The study that was found which could reasonably be used to explain stress conditions brought about by the DCP was the case of the static cone (14).

For granular materials, the advancement of a static cone was investigated by Meier and Baladi (36). A cone penetration model was developed and was partially verified by laboratory studies. Although the advancement of a dynamic cone is a much more complex problem; many of the conclusions drawn in this study are noteworthy. The derivation of this penetration model assumes a cone length (L), base diameter (2R), and

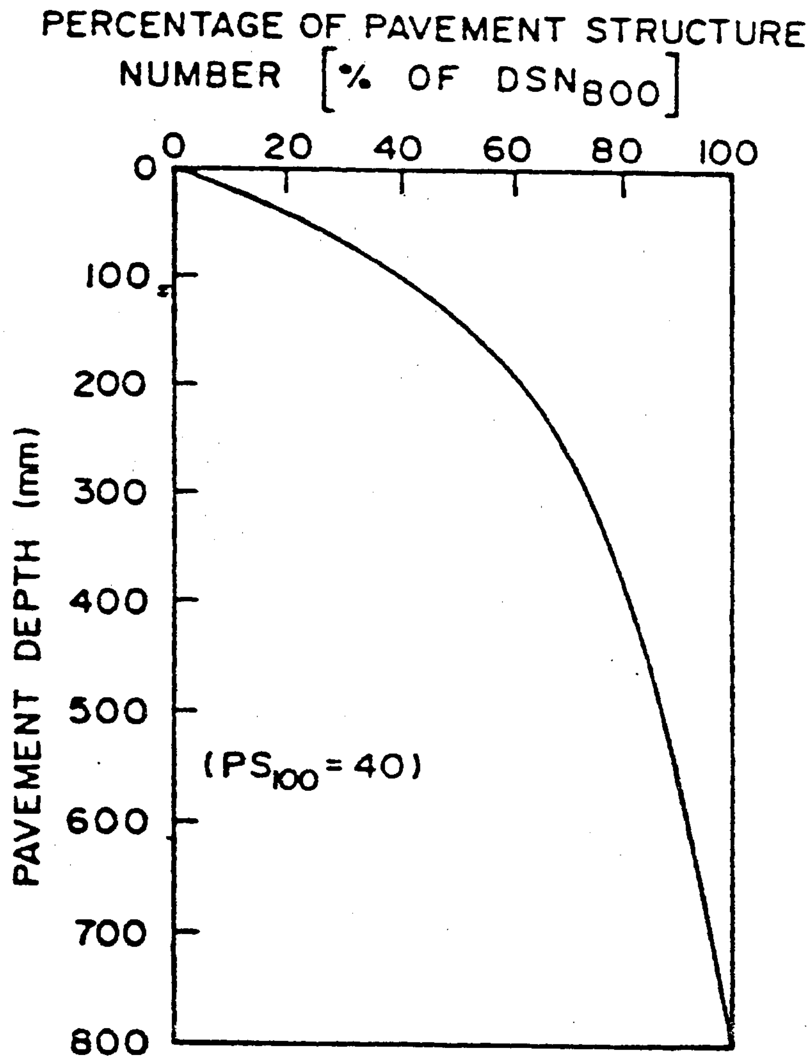


Figure 12. Example of a Pavement Strength-Balance Curve (34)

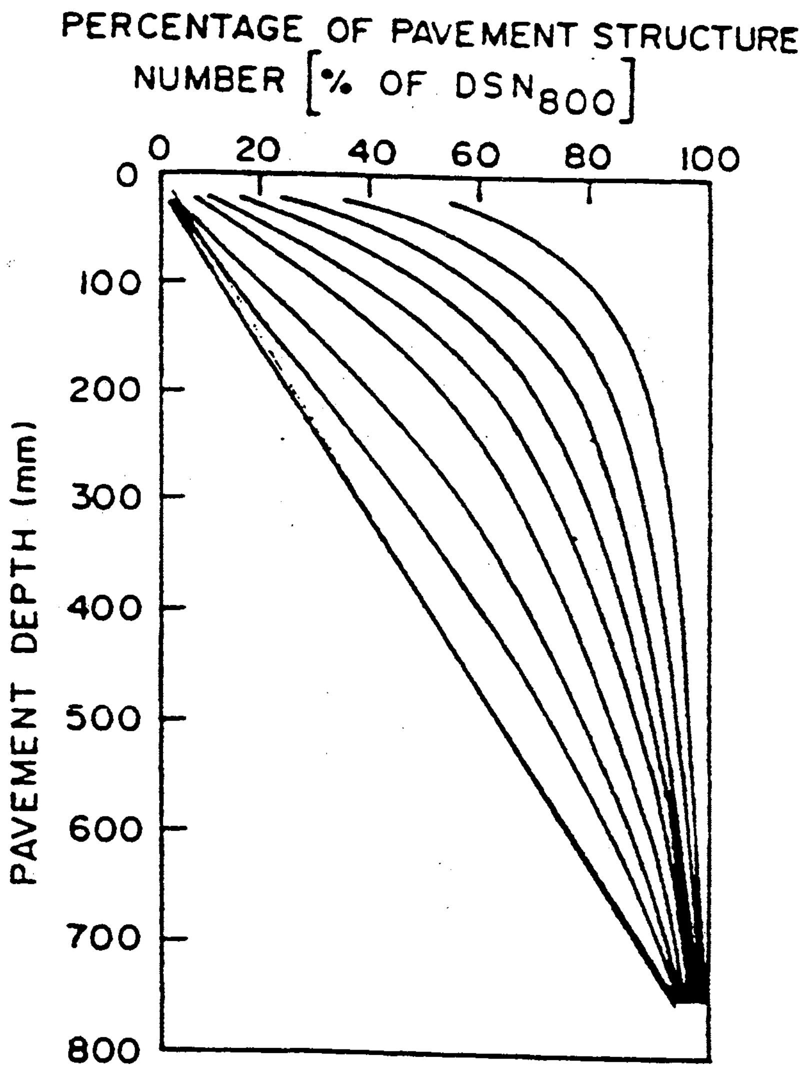


Figure 13. Pavement Strength-Balance Curves For Typical Pavements (34)

an included (apex) angle of 2. The cone is assumed to be penetrating a rate-independent isotropic granular medium which satisfies Mohr-Coulomb failure criteria.

The advancement of a cone through a granular medium results in shear failure in the surrounding material. It is assumed that the shear strength of the material is mobilized in resisting the cone penetration. The shear strength may be expressed as:

$$S = C + \sigma \tan\phi$$

where

S = shear strength;

C = apparent cohesion (assumed zero for noncohesive granular materials);

σ = normal stress; and

ϕ = apparent angle of internal friction;

assuming normal stress is equivalent to the internal pressure required to expand a spherical cavity in an unbounded, elastic-plastic media. The expression for normal stress (for granular materials with $\tan\phi > 0$) is:

$$\sigma = 3\,(q + C\cot\phi)\left(\frac{1+\sin\phi}{1-\sin\phi}\right)\left(\frac{\overline{G}}{C+q\tan\phi}\right)^{m} - C\cot\phi$$

where

q = hydrostatic stress;

$\overline{G}$ = apparent shear modulus; and

m = $[(4 * \sin\phi)/(3\,(1 + \sin\phi))]$.

If the cone tip is at a depth (z) plus the cone length (L) below the test surface, the normal and shear stresses acting on a portion of the cone (dL) may be calculated. If the in situ stress is assumed to be hydrostatic, the following equation is applicable:

$$q = (z + L - \eta)\gamma$$

where h is the distance from the cone tip to the center of the differential element of the cone being analyzed, and γ is density of the granular material.

The normal and shear stress resisting the penetration of a cone tip have been stated in terms of C, ϕ, γ, and depth $(z + L - \eta)$. The objective of this study by Meier and Baladi (36) was to develop a theoretical basis for relating U.S. Army Corps of Engineers

Waterways Experiment Station (WES) cone test results, i.e. Cone Index (CI) to the engineering properties of soil, C and ϕ. The concept of CI was first seriously considered by Freitag in the investigation of penetration tests for soil measurements (37).

The equations listed above are based on a differential element of the cone length (dL). Integration over the entire cone surface (of length L) is required to obtain a "practical" relationship between CI and C and ϕ. Integration of the aforementioned equations results in the following relationship:

$$CI = 6\ \overline{G}^{m} \frac{\tan\alpha}{\tan\phi} \frac{1+\sin\phi}{3-\sin\phi} \left[\frac{\tan\alpha + \tan\phi}{(R\gamma\tan\phi)^2} \right] \Omega$$

where

$$\Omega = \frac{[(Z+L)\gamma\tan\phi]^{3-m} - [Z\gamma\tan\phi + (3-m)L\gamma\tan\phi](Z\gamma\tan\phi)^{2-m}}{(2-m)(3-m)}$$

R = radius of the cone base; and

α = one-half of the apex angle of the cone tip.

The CI is thus expressed in terms of C, ϕ, γ, $\overline{G}$, depth of penetration (z), and cone geometry. It should be noted that the "idealized" equations presented do not consider boundary conditions present in actual cone testing. The theoretical equations based on cavity expansion in an unbounded medium require modification for the relative confinement due to adjacent materials and the free surface effect (due to unbounded test surface).

Near surface penetration results in a upward flow of material adjacent to the cone. This material displacement reduces the penetration resistance of the cone, thereby giving misleading test values. The surface effect decreases nonlinearly to a depth of approximately 9L for the WES cone, i.e. surface effects are not present at depths greater than approximately 9L. In the case of cohesive or mixed soil, the free surface effect is negligible (36).

Therefore, to account for the free surface effect in granular materials, the apparent shear modulus, $\overline{G}$, is proposed to vary with depth according to the following equation (36):

$$\overline{G} = 0.5 \left[A + \frac{1 - B \exp(-Cz)}{1 + B \exp(-Cz)} \right] G$$

where A, B, and C are experimentally evaluated constants; and G is the shear modulus (obtained from triaxial testing).

A, B, and C are related to cone geometry and only to a small extent related to the material properties. Limited testing using the WES cone has resulted in the following approximations (36):

$A = 0.986$

$B = 100$

$C = 0.55 \text{ in.}^{-1}$

The equations developed in the WES study as reported by Meier and Baladi (36) were derived for a "static" cone and are therefore not directly applicable to "dynamic" cone testing. However, many of the factors that affect the static cone should also affect the DCP in a similar manner.

Ayers (14) reported the following observations on his DCP testing of granular materials related to the above study:

1. The overburden effect, analogous to (q) the hydrostatic stress, has a significant effect on cone penetration. In his study, Ayers found this effect to be an overwhelming factor in the DCP/shear strength. Figure 14 illustrates this effect, i.e. as depth of penetration increases, rate of cone penetration decreases.

2. Displacement of near surface material is common to static and dynamic penetrometer testing. The free surface effects were found to be a factor to a depth of approximately 9L for the WES cone. The large near surface values of penetration rate (PR) illustrates this effect in DCP testing as found in his study.

3. The geometry of the cone, i.e. diameter, length, and included cone angle is a critical factor. A difference in test values of approximately 15% is noted for a DCP apparatus with a 30° cone tip versus a 60° cone tip.

4. The "accurate" determination of in-place shear strength using a static or dynamic cone penetrometer may not be achievable at depths greater than approximately

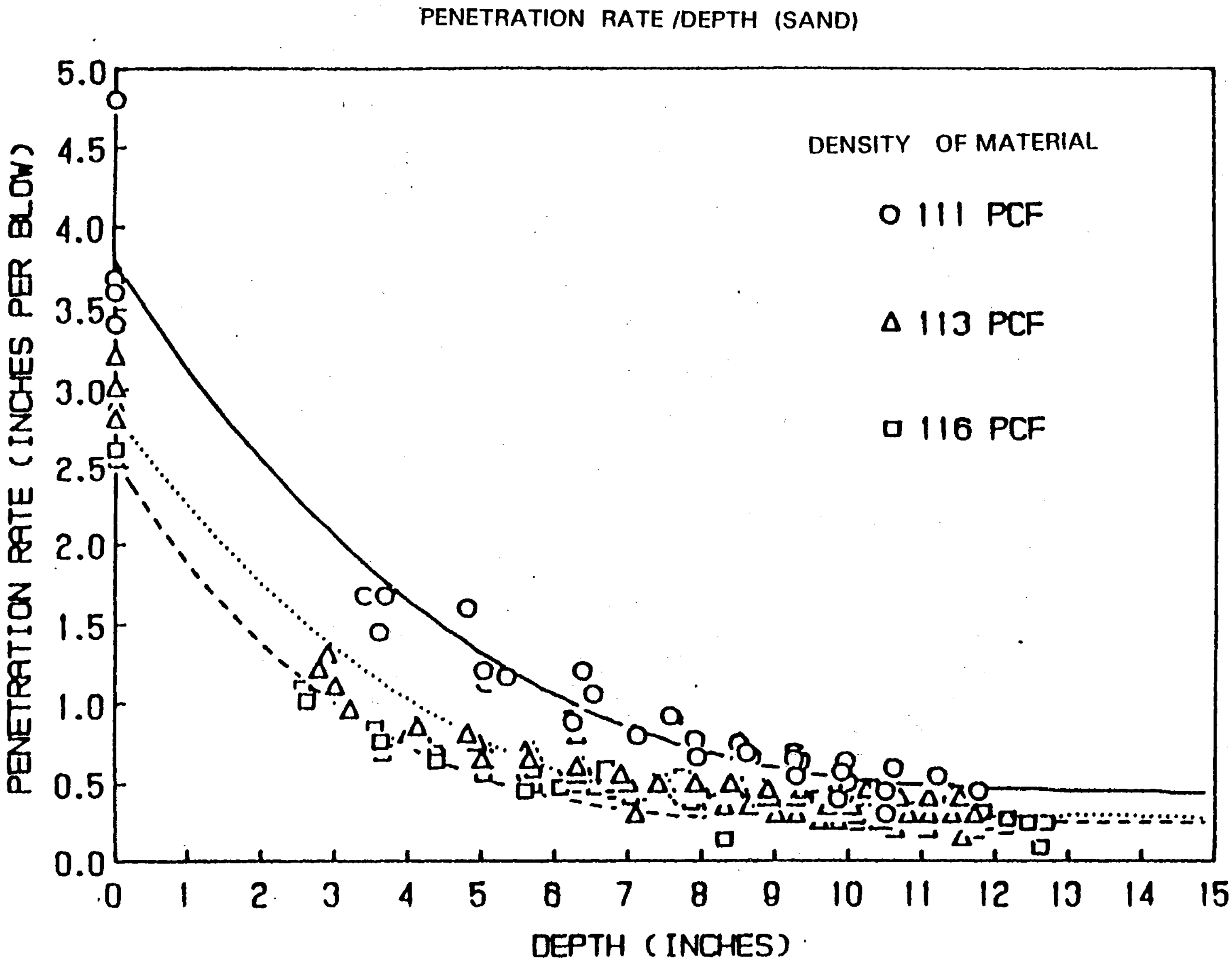


Figure 14. Effect of Overburden on DCP Values (14)

24 to 36 inches. The effect of the overburden tends to overshadow the effects of material properties for both cohesive and granular materials.

A study by Alersma (38) was also based on the advancement of a static cone and may not be directly applicable to the DCP. This study utilized the optical stress/strain analysis procedures to determine the effects of the cone penetration. A study by Harrison (39) and Scala (32) who devised a mathematical model to estimate the CBR based on the penetration rate and characteristics of the DCP demonstrated weaknesses in that the detailed cone geometry, material characteristics, and boundary conditions are not addressed.

The theoretical equations developed assumed no energy loss within the system in a manner that all momentum is transferred to the cone tip. As such, the complexity of the cone tip analysis cannot be overstated. It is doubtful that any theoretical analysis of the cone tip will have any practical implications for field use.

3.4 DCP Correlation Studies

3.4.1 Soil and Materials Factors Affecting DCP Values

Many studies have been conducted to determine the general trends and behavior of DCP values with regard to various soil and materials factors. Factors such as soil type, density, gradation, maximum aggregate size, and moisture content have been known to affect the DCP values in various ways.

One of the findings by Kleyn (34) in his comparative study of CBR with DCP values was that increased moisture content increased the DCP values and reduced the CBR value. Ayers (14) confirmed this finding. Gradation properties such as the coefficient of uniformity, and D10 values have also been studied to determine their effect on the DCP values. An increase in the percentage of the fines generally decreases the DCP value for the same target density. Similarly, an increase in the density for a similar gradation or individual material type decreases the DCP value. The effect of the maximum size aggregate was indirectly studied by Ayers (14) and found to be not significant. Kleyn also concluded that gradation, density, moisture content, and plasticity were important material properties affecting the DCP values (34).

3.4.2 Correlation of DCP Values With CBR Values

Numerous transportation agencies around the world use DCP data as input in determining pavement thickness since it is correlated with CBR values. There are approximately 30 regression equations between CBR and DCP values encompassing a wide range of material types. Table 6 shows 20 of these regression equations between DCP and CBR values in the form of the following equation (14):

$$\log CBR = A - B \log (DCP)^{C}$$

where A, B, and C are regression coefficients.

Chua (40) reported the three regression equations presented in Figure 15 have all been used satisfactorily as pavement design inputs. These equations were developed for granular materials/cohesive soils in both soaked and unsoaked conditions.

In Ireland, McGrath et al. (41) reported satisfactory results with the use of the DCP. The Swedish standard DCP was purchased and several experiments were conducted on a variety of Irish soils. A significant correlation existed between DCP values and other parameters of the Irish study which included cohesive soils. However, they expressed reservations when testing cohesive soils in a highly plastic state. This finding is consistent with the opinion of Sanglerat who is dubious of DCP readings when taken below the water table. It seems that many investigators are uncertain about DCP use in saturated soils.

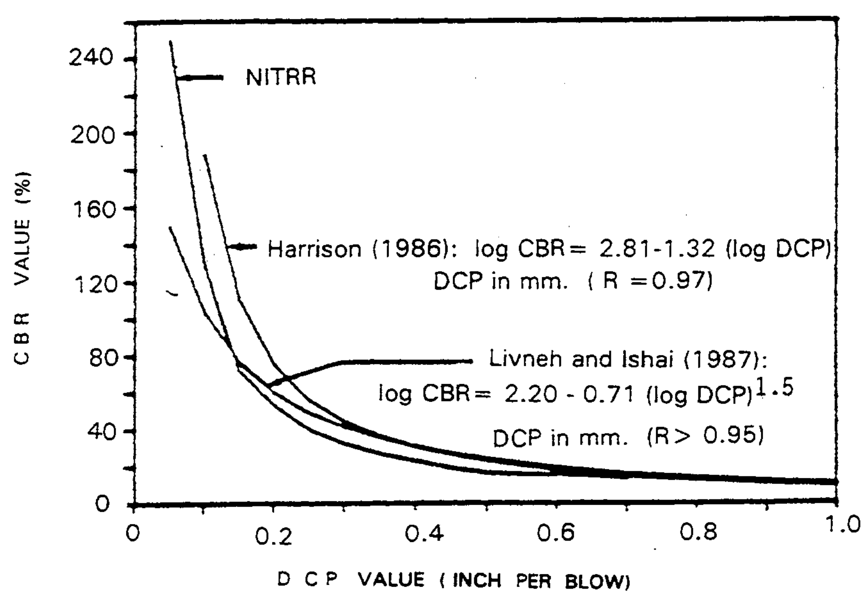


Figure 15. Correlation Between DCP and CBR Values (40)

TABLE 6

DCP/CBR CORRELATION EQUATIONS (14)

Equation Number	"A" Coefficient	"B" Coefficient	"C" Coefficient	Material Type
1	2.555	1.145	1.0	All
2	2.810	1.320	1.0	All
3	2.060	0.310	1.0	s
4	2.340	0.868	1.0	c
5	1.985	0.726	1.0	c
6	2.378	0.944	1.0	c
7	2.497	1.020	1.0	m, c
8	2.840	1.210	1.0	m
9	2.380	0.973	1.0	c
10	2.555	1.135	1.0	x (1)
11	2.775	1.135	1.0	x (2)
12	2.940	1.169	1.0	x
13	3.170	1.415	1.0	x
14	2.222	0.576	1.0	c (3)
15	3.070	1.250	1.0	c (3)
16	2.200	0.710	1.5	All
17	2.317	0.858	1.0	All
18	2.580	1.310	1.0	All
19	2.390	1.260	1.0	All (1)
20	2.605	1.269	1.0	All (2)

Notes: s = sand, m = silt, c = clay
All = materials as indicated in reference
x = unspecified material type
1 = samples confined in CBR mold
2 = unconfined samples
3 = expansive (clay)

In a study of four penetration testing methods, Livneh et al. (19) correlated DCP to CBR for both granular materials and fine-grained soils and established the following relationship:

$$\log CBR = 2.20 - 0.71 (\log DCP)^{1.5} + 0.075$$

where the coefficient of simple determination, $R^2 = 0.96$, and 0.075 is introduced as a correction factor to the earlier developed equation.

Number of samples tested, N = 74.

The DCP for this study was defined as the ratio of penetration (in terms of mm) and number of blows. One of the important findings of this study was that the coefficient of variation for the CBR for a given material was higher than that which was obtained by the DCP. This is noteworthy, as a higher reliability was obtained from the simpler device, the DCP.

3.4.3 Relationship of DCP Values With Unconfined Compressive Strength

Bester and Hallat (42) produced a graphical relationship between the DCP values with the unconfined compressive strength as shown in Figure 16. A general trend of a reduction in unconfined compressive strength corresponds to an increase in the DCP value was observed. The relationship was developed for use with a broad range of materials.

3.4.4 Correlation of DCP to Shear Strength

In a study by Ayers (14) of granular materials, the relationship between the DCP value and shear strength was investigated. Using multiple linear regression analysis, the regression equations for various gradations of granular materials at three levels of confinement were established. A general relationship between shear strength of granular materials and DCP values is not possible because shear strength of granular materials is affected by confining pressure. Therefore, the relationship investigated in the study related the deviator stress (σ_d) at various confining pressures as the dependent variable and the penetration rate (PR) of the DCP as the single independent variable. A single DCP value is determined for each material and target density. The single DCP value

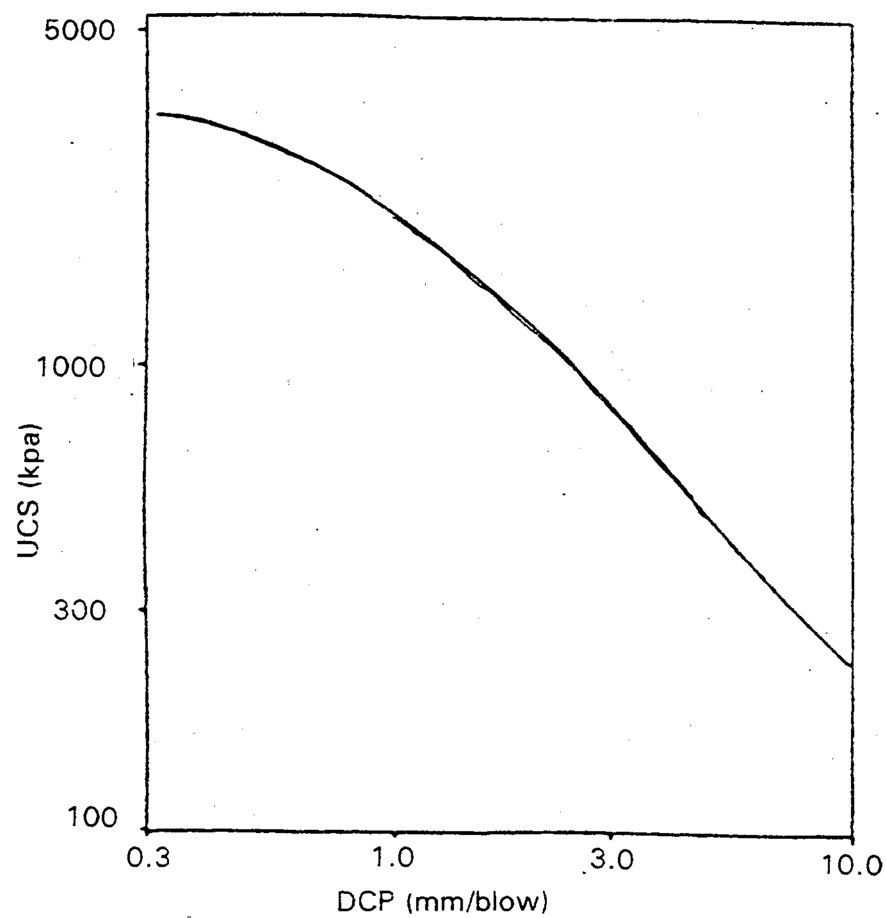


Figure 16. Relationship Between Unconfined Compressive Strength and DCP Values (42)

was assumed valid for three shear test results of the material at 5, 15, and 30 psi confining pressures in the development of the relationship. Inclusion of other independent variables such as density, void ratio, and coefficient of uniformity were performed when considering multiple independent variables. Table 7 is a sample of the developed equations for one independent variable; Table 8 is a sample of the developed equations for multiple independent variables.

3.4.5 Correlation of DCP Values to Resilient Modulus Values

A review of the literature reveals that there has only been one reported study to correlate resilient modulus to DCP data. It has been reported that the Minnesota Department of Transportation is in the process of conducting a related study. However, the details of their findings are still not known. The lack of such a study may be due to the difficulty to create the confinement pressure in conducting the DCP test which is controlled in the resilient modulus determination. Second, as reported by Sangelerat and McGrath, cohesive soils may exhibit increased DCP values in the presence of moisture as demonstrated in soils below water table.

3.5 Limitations

The fact that there is no universal standard makes comparative study of DCP testing difficult, although this apparently does not hinder progress in the development of the equipment. Thirteen countries have established standards of their own and are making progress in improving the DCP instrument. Categorization into four DCP hammer weight ranges at the First World Symposium on Penetration Testing may eradicate some problems in the advancement of DCP technology (31). When used in gravel and other granular base course materials, damage to the DCP cone tip is possible and may limit its usefulness. The DCP will give acceptable readings when used in coarse materials unless the cone bears directly on stone (34).

The doubts expressed by some investigators on its use in clay or cohesive soil are now being refuted as new studies show it can be used with confidence in this type of soil as well. However, as stated by some researchers, care must be used when dealing with

TABLE 7

SAMPLE OF ONE VARIABLE REGRESSION EQUATION (14)

Material	Confining Pressure (psi)	Regression Equation	Coefficient of Determination	Standard Error of Estimate
Sand	5	$\sigma_d = 41.3 - 12.8\,PR$	0.998	0.3
Sand	15	$\sigma_d = 100.4 - 23.4\,PR$	0.998	0.5
Sand	30	$\sigma_d = 149.6 - 12.7\,PR$	0.978	0.9
All Dense	5	$\sigma_d = 94.5 - 40.0\,PR$	0.441	28.5
Graded	15	$\sigma_d = 154.6 - 57.7\,PR$	0.438	41.3
Materials	30	$\sigma_d = 192.2 - 64.2\,PR$	0.466	49.4

TABLE 8

SAMPLE OF MULTIPLE LINEAR VARIABLE REGRESSION EQUATIONS (14)

Material	Confining Pressure (psi)	Regression Equation	Coefficient of Determination	Standard Error of Estimate
Dense	5	$\sigma_d = 94.5 - 40.1\,PR$	0.441	28.5
Graded		$\sigma_d = 137 - 24.9\,PR - 208.3\,e$	0.564	26.9
Materials		$\sigma_d = 132.4 - 31.4\,PR + 0.2e - 208.7\,Cu$	0.584	28.4
		$\sigma_d = 2777 - 105.7\,PR - 18.0e + 1.6\,Cu - 1575\,\gamma$	0.859	18.1
Dense	15	$\sigma_d = 154.5 - 57.6\,PR$	0.438	41.3
Graded		$\sigma_d = 197 - 42.5\,PR - 208.1\,e$	0.497	41.6
Materials		$\sigma_d = 1808 - 59.5\,PR - 10.8\,e - 1028\,\gamma$	0.710	34.2
		$\sigma_d = 4474 - 148.7\,PR - 29.1\,e - 1.8\,\gamma - 2411\,Cu$	0.893	22.7
Dense	30	$\sigma_d = 209.4 - 75.7\,PR$	0.466	49.4
Graded		$\sigma_d = 280.5 - 50.4\,PR - 348.3\,e$	0.589	47.1
Materials		$\sigma_d = 2093 - 69.5\,PR - 12.2\,e - 1271\,\gamma$	0.761	38.3
		$\sigma_d = 5392 - 179.9\,PR - 34.8\,e + 2.2\,\gamma - 2983\,Cu$	0.941	21.2

σ_d = deviator stress; PR = penetration rate; e = void ratio; Cu = coefficient of uniformity; and γ = density of soil.

cohesive soils that are below the water table or in a plastic state. One conclusion of the study in Ireland (41) was the unsuitability for its use in soil type identification, especially if the depth was 15 ft below the surface. In view of this, its application in pavement design and evaluation may not be adversely affected since only depths of up to 2 ft below the subgrade level are of great concern to pavement designers.

In the execution of the test, variability of the driving rate and inconsistent height of fall are inherent in this test. This testing variability together with the material variability associated with soils, gravel, and stones have made users cautious of DCP readings (31).

CHAPTER IV

EXPERIMENTAL PROCEDURE

4.1 Equipment and Instrument Adaptation

The primary objectives of this study are to establish a relationship between the resilient modulus and the DCP values for fine-grained soils, and to investigate the effects of certain factors on DCP values of fine-grained soils and granular materials. The main problem in establishing any correlation between these values is the dissimilarity in the two test procedures.

Special equipment was developed to mold cylindrical test specimens (6 inches diameter and 12 inches high) and a triaxial cell was modified to permit the application of confining pressure to these specimens while the DCP test was conducted. The split mold is shown schematically in Figure 17. The triaxial cell with a test specimen and the DCP cone and rod in place for testing is shown in Figure 18. Modification of the DCP test apparatus was not required.

Air pressure was used to provide confinement for the sample during DCP testing. The major problem associated with the triaxial cell was loss of confinement pressure due to leaks. Sealing the DCP rod as it passed through the linear bearing was accomplished with multiple seals as shown in Figure 19.

4.1.1 Compaction Equipment for Fine-Grained Soils

Fine-grained soil samples were molded in the previously described split mold. Compaction was accomplished with the standard AASHTO manually operated rammer (weight = 5.5 pounds; drop = 12 inches) using 25 blows per 3 inch lift of soil or until target height was reached.

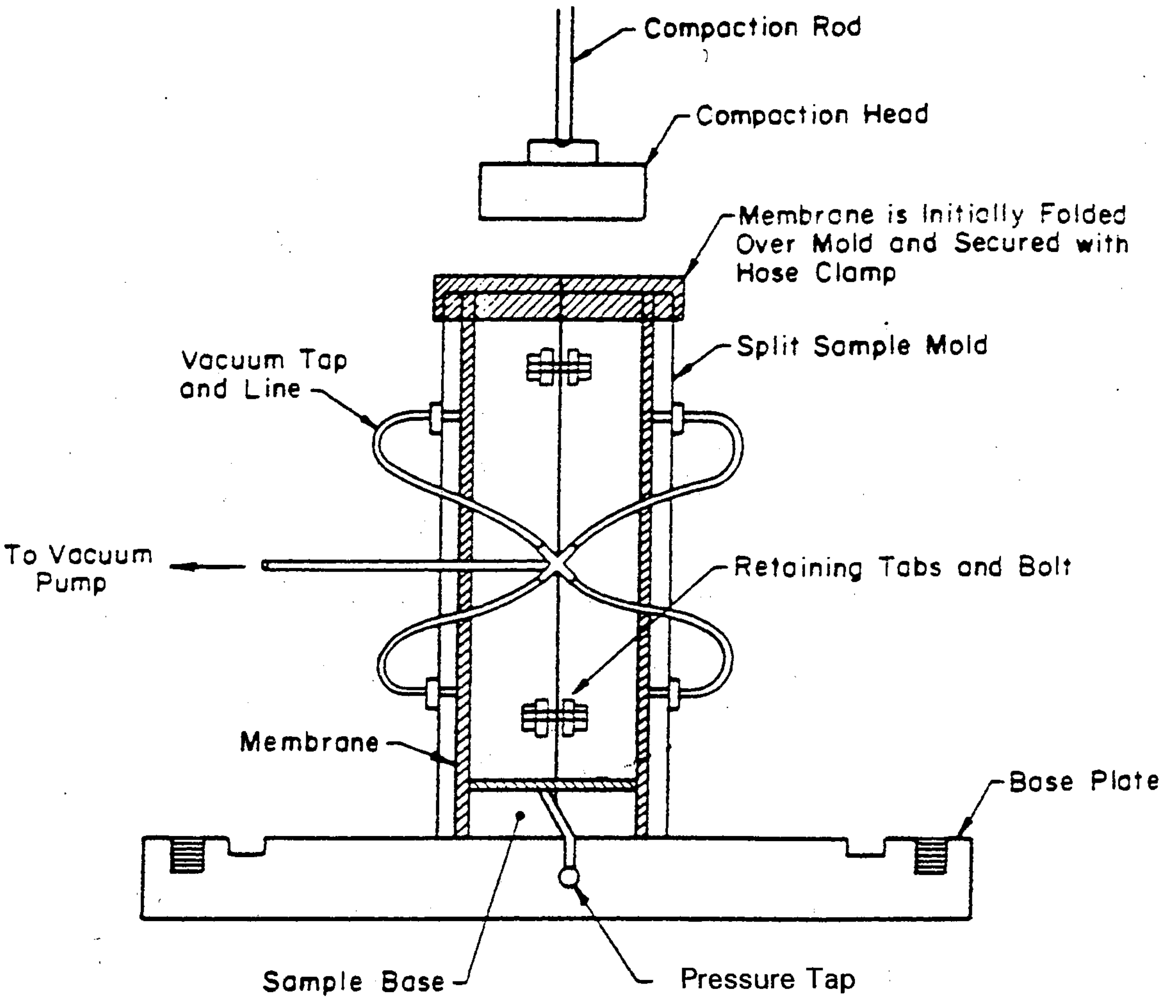


Figure 17. Mold and Triaxial Cell Base Plate Assembly

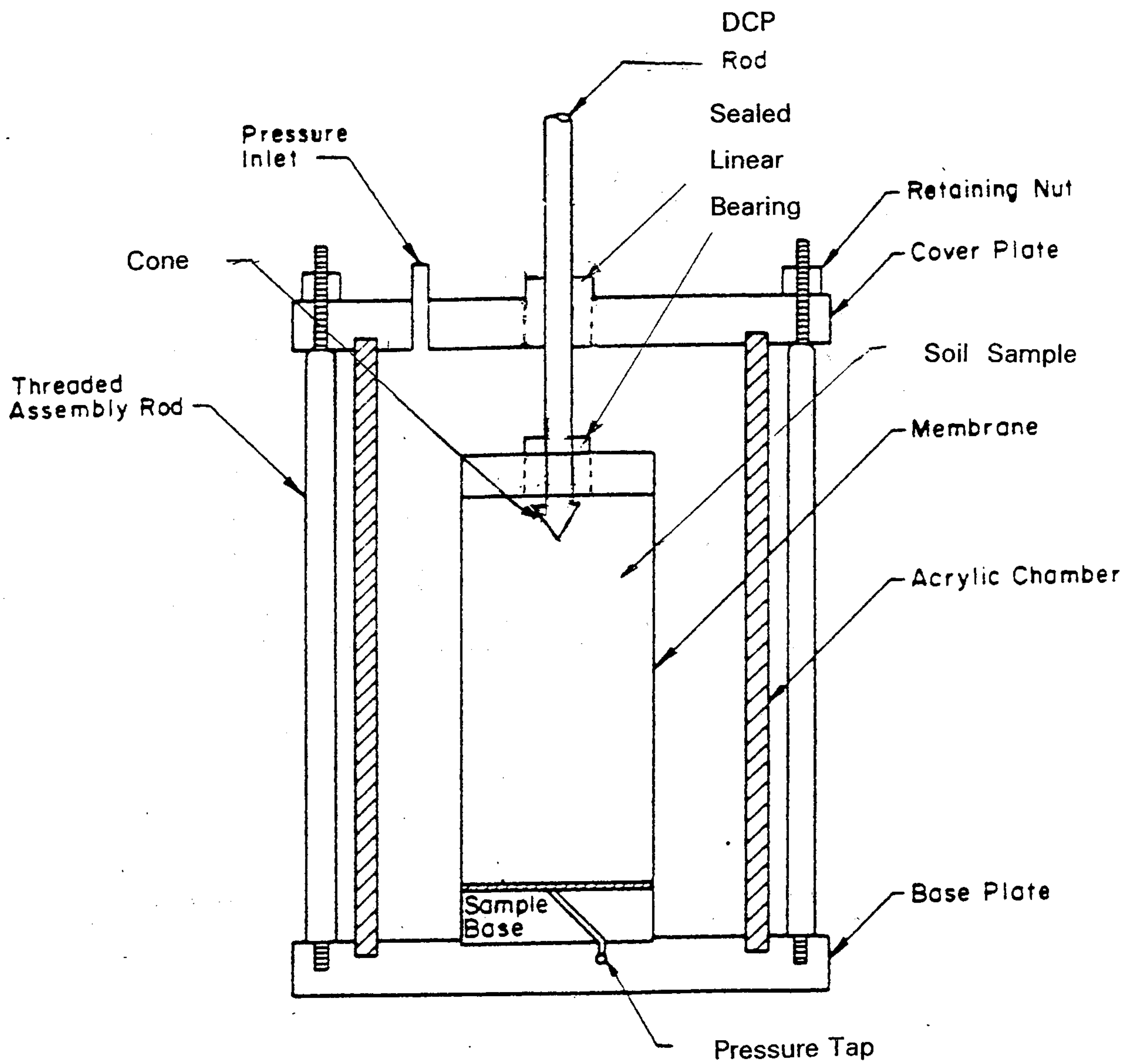


Figure 18. Triaxial Cell With Sample and DCP Apparatus in Place

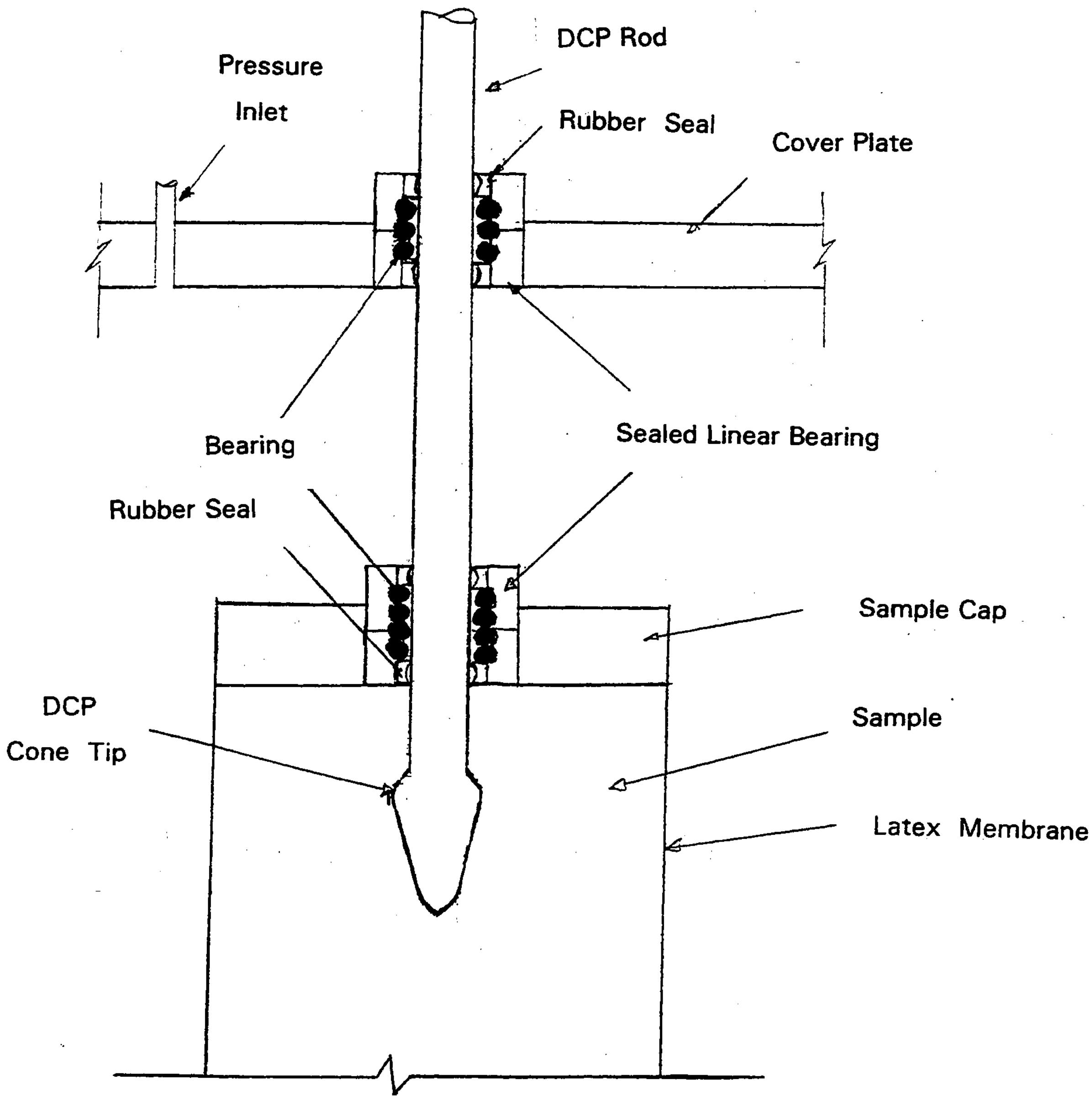


Figure 19. Schematic of Multiple Seal With DCP Rod Passing the Linear Bearing

4.1.2 Compaction Equipment for Granular Materials

The vibratory compactor was fabricated by OSU. It consisted of a metal frame, a Bosch electric jack hammer (Model No. 0611304034), a power winch, and a spring-mounted compaction base as schematically shown in Figure 20. The compaction head was a steel plate 1-in. thick and 5.75 in. in diameter. The weight of the compaction head was 15.5 lbs. A 15-second vibration time was used for each lift of soil during compaction.

4.2 Materials Description

4.2.1 Fine-Grained Soils

The source of the samples and the corresponding AASHTO soil classification are shown in Appendix A, Table 17. They are naturally occurring Oklahoma soils furnished by ODOT. The number of samples tested and treatments received varied according to objectives of the study as follows:

Objective 3
- a. Number tested: 15 samples (30 tests)
- b. Moisture content: from optimum to optimum +20%
- c. Density: maximum dry density
- d. Confining pressure: 3 psi

Objective 5
- a. Number tested: 7 samples (14 tests)
- b. Moisture content: optimum
- c. Density: 100% and 110% maximum dry density
- d. Confining pressure: 0 psi

Objective 7
- a. Number tested: 6 samples (18 tests)
- b. Moisture content: optimum
- c. Density: maximum dry density
- d. Confining pressure: 0, 15, and 30 psi

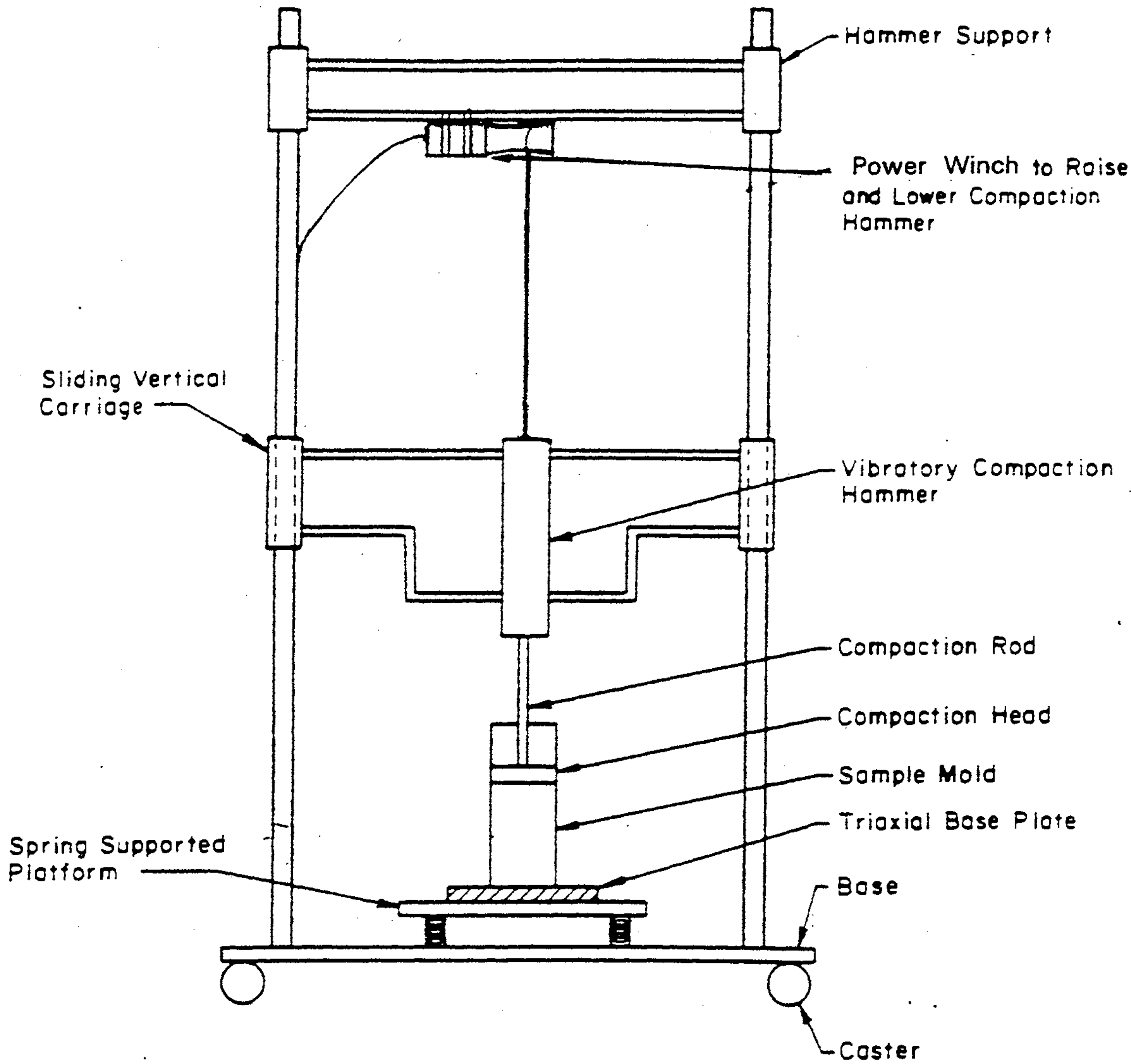


Figure 20. Compaction Apparatus for Granular Materials

4.2.2 Granular Materials

ODOT and OSU each supplied three categories of granular material samples to be tested in this study. The three categories of materials include sand, 3/8-inch maximum size aggregates, and 3/4-inch maximum size aggregates. The materials supplied by ODOT and OSU consisted of crushed limestone with varying gradation. The gradation charts and coefficient of material uniformity are shown in Appendix B, Figures 36 through 41. The samples and treatments were stipulated in Objective 12. These were:

a. Number tested: 6 samples (18 tests)

b. Moisture content: oven dry state

c. Density: maximum dry density

d. Confining pressure: 6, 12, and 18 psi

4.3 Test Procedure

The resilient modulus testing was conducted by the ODOT Central Materials Laboratory according to AASHTO T 292-91. The target moisture content and confining pressures of the fine grained soils were established by ODOT. The granular material characteristics were established during DCP testing at the OSU Asphalt Materials Laboratory.

4.3.1 Fine-Grained Soils

All DCP tests were conducted at the OSU Asphalt Materials Laboratory. For fine-grained soils, tests were conducted to assess the effects of moisture content, density, soil classification, and confining pressure. The DCP tests were conducted at two moisture states corresponding to those used in the resilient modulus testing by ODOT to facilitate the development of a relationship between DCP values and resilient modulus.

The material preparation procedure was as follows:

1. The moisture content of the fine-grained soils as received from ODOT was determined.

2. An appropriate amount of water was added to the soil samples to achieve optimum moisture content, as determined by ODOT tests.

3. An estimate of the required weight of soil for each lift necessary to prepare the 12 inch sample in 4 lifts was made. First, the approximate sample dimension of 12 inches high and 6 inches in diameter was used to calculate the volume of the required soil. The required weight of the sample at the density corresponding to its optimum moisture content was determined. This was then divided by 4 to obtain the approximate weight of the soil for each lift of soil.

4. Similar procedures were used to prepare the soils samples at 20% above optimum moisture.

The sample compaction procedure was as follows:

1. The 0.025 inch thick latex membrane was placed inside the split sample mold with both its top and bottom secured to the mold as shown Figure 17.

2. The membrane was drawn tightly against the sample mold via an external vacuum pump.

3. The first lift was placed inside the latex membrane and compaction was applied to achieve a sample height of approximately of 3 inches. The compaction was achieved using the standard proctor hammer at approximately 25 distributed blows per layer or until the target height of 3 inches for each lift was reached.

4. A similar procedure was repeated until the 4 lifts were compacted to the target density which corresponds to a sample height of 12 inches.

5. In order to compare the DCP values at two different densities, randomly selected soil samples at 110% density were also prepared. To achieve this density, additional compactive effort was applied to the sample while continuous height measurement were made until the target density was reached.

The test set-up procedure was as follows:

1. The sample height was recorded as a density verification measure.

2. Following sample preparation and compaction, the split mold was removed.

3. The DCP cone tip and sample cap was brought into contact with the sample. The cone tip was then allowed to slightly penetrate the sample.

4. The latex membrane was secured to the sample cap by overlapping the membrane and securing it with screw clamps. It was imperative that no leaks exist between the membrane and sample base and cap.

5. The triaxial cell was then assembled by carefully placing the acrylic chamber on the triaxial base plate, directing the DCP rod through the linear bearing in the triaxial cover plate and finally securing the cover plate to the acrylic chamber. Every effort was made to minimize movement of the DCP rod during this procedure.

6. The DCP slide hammer/handle assembly was then attached to the DCP anvil.

7. An air supply line was attached to the pressure inlet located in the triaxial cell cover plate. A pressure regulator was used to control the confining pressure.

8. An air pressure gauge was affixed to the pressure tap, located in the triaxial cell base plate, to verify the actual confining pressure.

9. The confining pressure was increased to its maximum value and an air pressure gauge attached to the sample drain was used to verify that no leaks existed.

10. If an air leak was detected, the triaxial cell was disassembled and the leak repaired. After all leaks were repaired, the confining pressure was increase and the tests began.

11. An initial reading of the calibrated DCP rod was taken as the baseline reading prior to actual testing.

The testing procedure was as follows:

1. The DCP slide hammer was raised until it came into contact with the handle and then released. A reading taken from the calibrated DCP rod was recorded as the depth of advancement of the DCP device for that blow.

2. This procedure was repeated until the cone tip contacted the sample base plate or until it was in close proximity to the base plate.

3. Penetration rates (inches/blow) throughout the sample depth were determined. The DCP values used in the analysis were determined by taking the average of the penetration rates determined at the middle of the sample, and at one inch above and one inch below this point.

4.3.2 Granular Materials

A set of material samples consisting of sand, 3/8 maximum size aggregate, and 3/4 maximum size aggregate was provided by OSU and ODOT for this study. DCP testing was performed on three categories of materials from both sources at the OSU Asphalt Materials Laboratory.

The materials preparation procedure was as follows:

1. All granular material samples were oven dried at 230°F to a constant oven dry moisture state.

2. A sieve analysis was performed on each material sample. The gradations and coefficient of uniformity (Cu) for the granular material are shown in Appendix B, Figures 36 through 41.

3. The estimated amount of material required to prepare a 12-inch high sample at the target density was weighed out and an additional 5% added to this estimate for the actual sample preparation.

The sample compaction procedure was as follows:

1. A latex membrane was placed inside the split sample mold with both its top and bottom secured to the mold as shown in Figure 17. The membrane was tightly drawn to the mold by the application of an external vacuum.

2. The samples were prepared in three lifts to promote uniform density and to minimize segregation.

3. The samples were compacted with a vibratory hammer for approximately 15 seconds for each lift. A schematic of the compaction apparatus is shown in Figure 20.

4. The sample height was measured following compaction of the final lift. The weight and height of the sample were used to determine the compacted density of the material.

The test set-up procedure was as follows:

1. Following the compaction of the sample, the rigid mold was removed. The DCP rod and cone assembly with the sample cover plate was placed on the sample. The latex membrane was secured as with the fine grained soils samples.

2. The sample generally required a second latex membrane over the first layer to seal off any ruptures which occured during the compaction process.

3. The triaxial cell was then assembled and a confining pressure applied to check for leaks. A leak in the membrane or cell would reduce the critical confining pressure. All air leaks were repaired prior to testing.

4. The application of confining pressure at the predetermined levels was similar to the fine-grained soil tests.

5. The DCP apparatus was assembled and an initial baseline reading taken.

The test procedure was as follows:

1. The test was performed similarly to the fine-grained soils as previously discussed.

2. Based on the penetration rates (inches per blow), DCP values for the three granular material categories at the three confining pressures were determined.

4.4 Experimental Design and Statistical Procedure

4.4.1 Fine-Grained Soils

DCP tests were performed on 15 fine-grained soil samples for which resilient modulus data were available. The confining pressure was held constant at 3 psi, while the moisture state was varied from optimum to 20% above optimum with a corresponding change in density. The effect of the moisture content on DCP values was analyzed using a paired t-test. An analysis of the difference in DCP values between the two moisture contents was made.

Seven randomly chosen samples were compacted to approximately 110% maximum dry density by increasing the compactive effort. A paired t-test was used to analyze whether a significant difference exists between the DCP values at 100% and 110% maximum dry density for the randomly chosen soil samples.

DCP tests were conducted on six soil samples chosen from the 15 available samples at optimum moisture content using 0, 15, and 30 psi confining pressures. The samples corresponded to an AASHTO A-6 or A-7 classification. The Analysis of Variance (ANOVA) was performed on these data to determine the joint effects of confining pressure and soil gradation and/or its individual effect on DCP values of fine-grained soils.

The coefficient of correlation between the resilient modulus and DCP values for fine-grained soils at the two moisture states was determined. A simple linear regression relationship between the resilient modulus and the DCP value was established at the two moisture states.

4.4.2 Granular Materials

Three categories of granular materials were evaluated according to their maximum grain size. The DCP values for these three categories from the two sources (OSU and ODOT) were determined in the oven dry state for consistency. Moisture content is not a significant factor in determining the DCP value of granular materials. The tests were conducted at three confining pressures including 6, 12, and 18 psi.

The experimental design was a split-plot design with a blocking factor at the whole plot level. The whole plot (or main unit) treatment was the category of aggregate with three levels. The three categories of aggregate were blocked according to the source of the materials. The subplot (subunit) treatment was confinement at the three levels.

The ANOVA procedure was used to determine whether confining pressure and maximum aggregate size yield significantly different DCP values of granular materials. A contrast analysis was also performed to determine if the effect of confining pressures on DCP values followed any significant trend. The coefficient of correlation between the coefficient of uniformity (Cu) and the maximum aggregate size of the materials with the DCP values at the three confining pressures was calculated to see if any association between the parameters existed.

CHAPTER V

ANALYSIS AND DISCUSSION OF RESULTS

All analyses were carried out using the Statistical Analysis System (SAS) software package. For comparisons, the 5% significance level was used.

5.1 Fine-Grained Soils

5.1.1 Effect of Increase in Moisture Content on DCP Values

The information on the physical and chemical properties, and clay mineralogy of fine-grained soils tested can be extracted from Appendix A, Table 17. Penetration data for each of the samples at optimum moisture content and at optimum moisture content +20% are given in Appendix A, Tables 18 and 19. Limited data are available for the optimum +20% sample due to high DCP values corresponding to the soft soil. The DCP values for all the soil samples tested at the two moisture states are presented in Table 9.

The initial penetration value shown for some samples indicates an unusually high reading due to either the movement of sample cover plate during assembly of the triaxial cell or improper initial seating of the DCP cone tip prior to testing. The overall results were not adversely affected since the DCP values used in the analysis were based upon penetration rates in the middle of the sample.

The DCP values for each of the soil samples at the two moisture contents used in the analysis of the effects of moisture content on DCP values are summarized in Table 10. Figure 21 graphically shows the DCP values of the samples tested at the two moisture states. The output of the statistical analysis is shown in Appendix A, Figure 31. Comparison of the DCP values for the two moisture contents was done by a paired t-test. The mean difference [(DCP values at optimum moisture content) – (DCP values at optimum moisture content +20%)] of –2.225 is significant. (D = -2.2250, Sd = 1.4801,

TABLE 9

DCP VALUES OF FINE-GRAINED SOILS AT TWO MOISTURE CONTENTS

Soil Description	DCP Values at Optimum Moisture Content	DCP Values at Optimum Moisture Content +20%
Woodward	3.9	6.1
Heiden2	3.2	8.1
Landslide	4.6	5.4
Bengal	5.1	8.1
Clebit	5.0	6.0
Quinlan	4.7	5.5
Heiden1	4.7	8.2
Durant1	2.7	4.3
Carnasaw	3.9	N/A
Dalhart	5.3	N/A
Osage	9.5	N/A
St.Paul	6.7	N/A
Darnell	5.8	N/A
Dougherty	3.8	N/A
Durant2	5.4	N/A

N/A—Data not available due to high penetration rates.

TABLE 10

DCP VALUES OF FINE-GRAINED SOILS ANALYZED FOR THE EFFECT OF INCREASED MOISTURE CONTENT

Soil Description	DCP Values at Optimum Moisture Content	DCP Values at Optimum Moisture Content +20%
Woodward	3.9	6.1
Heiden2	3.2	8.1
Landslide	4.6	5.4
Bengal	5.1	8.1
Clebit	5.0	6.0
Quinlan	4.7	5.5
Heiden1	4.7	8.2
Durant1	2.7	4.3

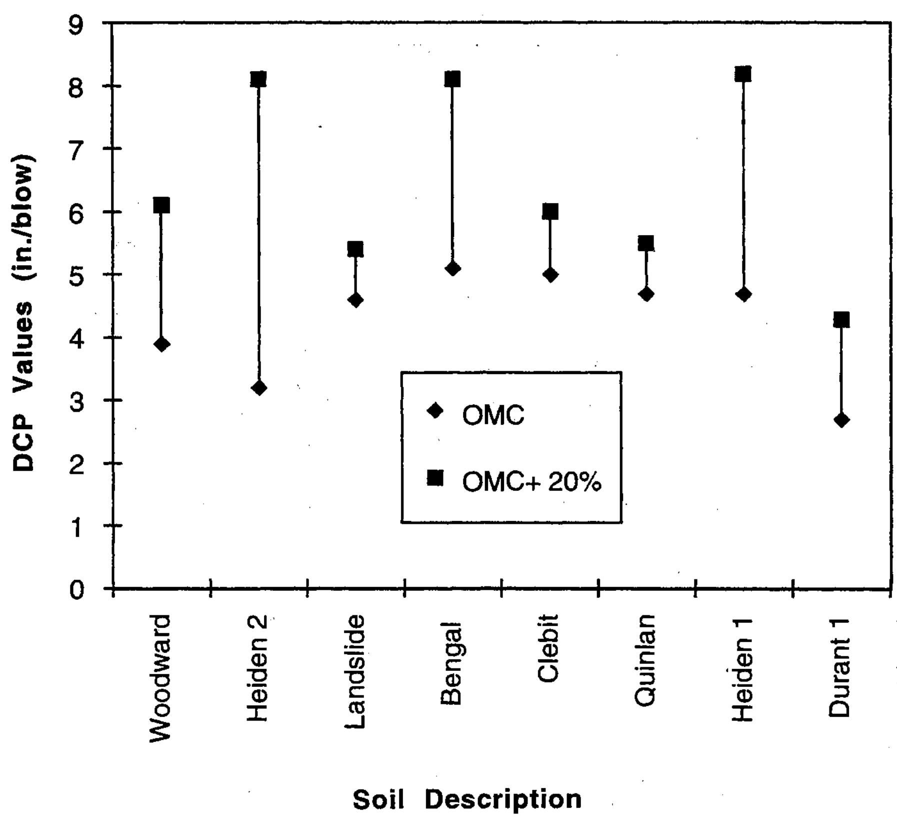


Figure 21. Graph of DCP Values at Two Moisture Contents

t = -4.2519, p = 0.0038). Since this difference is negative, the DCP values of a soil at optimum moisture content +20% is higher than the DCP values at optimum moisture content. Therefore, the DCP values for the fine-grained soils may increase by 1.0 to 3.5 inches per blow as estimated with 95% confidence when moisture content increases from optimum to optimum +20%.

The increase in DCP values with increasing moisture content for fine-grained soils can be explained in terms of the soil mineralogy. The interlayer water separating the mineral sheets is relatively thin, thereby promoting strong interlayer bonds (43). As the moisture content increases, the sheets are more widely separated, leading to a reduction in bond strength. Moisture facilitates the movement of DCP through fine-grained soil particles by added lubrication and a reduction in bonding forces of the mineral sheets. Therefore, an increased moisture content usually results in increased DCP values for a given soil.

It can be concluded that an increase in moisture content above the optimum significantly increases DCP values in fine-grained soils. This finding confirmed similar conclusions by Kleyn (34) and Harrison (40).

5.1.2 Effect of Increase in Density on DCP Values

The DCP penetration data for the seven soil samples at the two density levels are presented in Appendix A, Tables 20 and 21. The DCP values for each of the seven soil samples at the maximum dry density and 110% maximum dry density are summarized in Table 11. Two groups of soils corresponding to AASHTO A-6 and A-7 classification were tested. Figure 22 graphically shows DCP values of the samples tested at the two density states. The output of the analysis to evaluate the effects of increase in density on DCP values is shown in Appendix A, Figure 32.

Comparison of the DCP values at the two density levels was done by a paired t-test. The mean of the difference [maximum dry density – (110% maximum dry density)] of 1.9571 is significant (D = 1.9571, Sd = 0.5968, t = 8.6762, p = 0.0001). This showed that soils at higher densities have lower DCP values. The DCP values of the fine-grained soil may decrease by 1.4 to 2.5 inches per blow as estimated with 95% confidence when density increases from maximum to 110% maximum dry density.

The orientation of soil particles is changed when soils are being compacted to achieve higher densities. The optimum moisture content results in water film surrounding the particles. These water films lubricate the particles and allow reorientation into a denser configuration during compaction (44). At the optimum moisture content, a fine-grained soil compacted to a higher dry density generally has a lower DCP value. The densities investigated in this study are significant since specifications for field compaction are generally within this range.

This study confirms the findings by Van Vuuren (33), Kleyn (34), and Harrison (39) who concluded that, for a broad range of soil types, an increase in soil density significantly decreases the DCP values.

5.1.3 Effect of Soil Classification and Confining Pressure on DCP Values

The penetration data for each of the six fine-grained soil samples tested at 0, 15, and 30 psi confining pressures are presented in Appendix A, Tables 22 through 24. Table 12 shows the DCP values of these samples for the test carried out at the respective confining pressure. Figure 23 graphically shows the DCP values of the samples tested at the three confining pressures. The results of the analysis to study the effects of AASHTO soil classification and confining pressure on DCP values are shown in Appendix A, Figure 33.

There is no significant interaction between confining pressure and AASHTO soil classification in affecting DCP values ($p = 0.8622$). Confining pressure is also not found to be a significant factor in the determination of DCP values ($p = 0.2186$). However, AASHTO soil classification is found to be a significant factor in determining DCP values ($p = 0.001$). The observed means of the A-6 and A-7 soil types are 4.7111 and 3.2889, respectively (Least Significant Difference, LSD = 0.7206).

The tests showed that soils belonging to A6 and A7 AASHTO classification have different DCP values. It also showed that fine-grained soils tested at different confining pressure did not produce significantly different DCP values.

TABLE 11

DCP VALUES OF FINE-GRAINED SOILS TESTED AT MAXIMUM DRY DENSITY AND 110% MAXIMUM DRY DENSITY

Soil Description	DCP Values at 100% Density	DCP Values at 110% Density
Clebit	5.0	2.4
Durant1	2.9	2.1
Heiden2	5.5	3.0
Carnasaw	4.2	2.3
Dalhart	5.4	3.5
Bengal	5.1	3.3
Heiden1	4.8	2.6

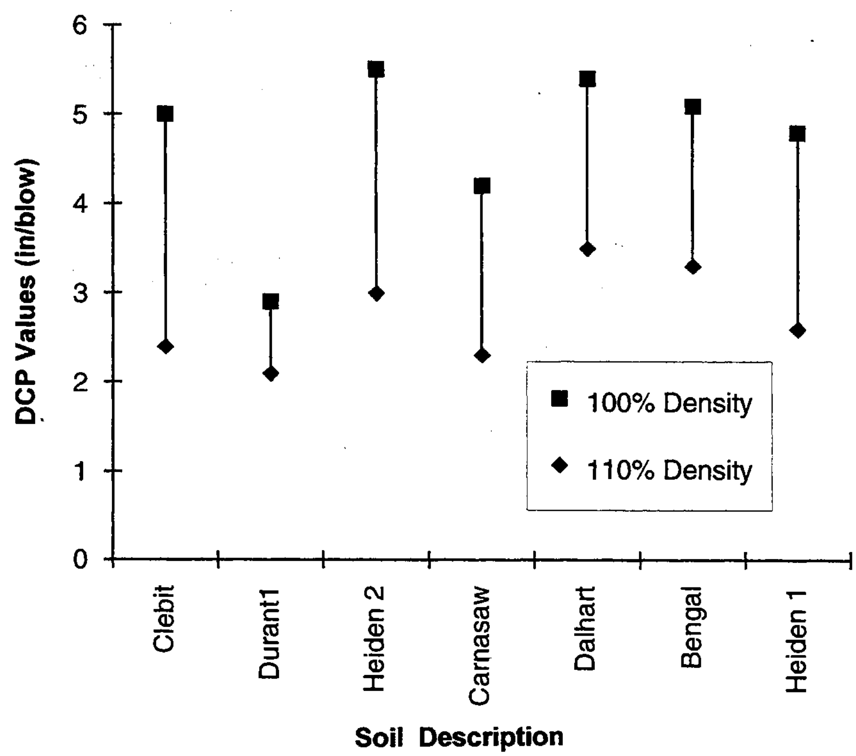


Figure 22. Graph of DCP Values at Two Density States

TABLE 12

DCP VALUES OF FINE-GRAINED SOILS TESTED AT
0, 15, AND 30 PSI CONFINING PRESSURES

Soil Description	Soil Type	0 psi	15 psi	30 psi
Clebit	A-6	5.0	4.7	4.9
Landslide	A-7	4.4	4.2	4.0
Carnasaw	A-7	4.2	2.4	2.4
Dalhart	A-6	5.4	4.9	4.6
Quinlan	A-6	4.6	4.4	3.9
Durant 1	A-7	2.9	2.8	2.3

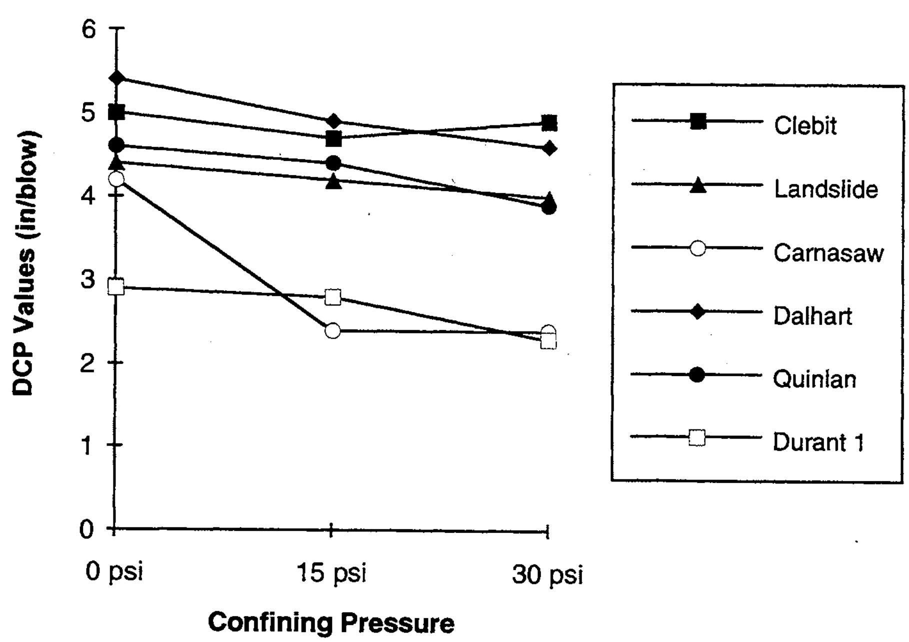


Figure 23. DCP Values Against Confining Pressure

The higher DCP values shown for the AASHTO A-6 soils compared to the A-7 soils are likely due to the particle size distribution and plasticity index. The AASHTO soil classification is a function of the particle size distribution and Atterberg Limits of fine-grained soils. The DCP value of a specific fine-grained soil is not significantly affected by the confining pressure if the moisture content and density are maintained. Increasing the confining pressure on a sample does not involve the reorientation of the soil as found in the compaction process. Compaction involves reorientation of the soil particles by relative movement resulting in an increased density. An increased confining pressure results in a restraint of soil particles within the membrane and not reorientation.

The theoretical basis for this observation is based on the basic shear strength equation for soils:

$$S = c + p \tan\phi$$

where

S = shear strength or penetration resistance in this case;

c = cohesion;

p = confining pressure; and

ϕ = angle of internal friction.

Cohesion is the primary variable contributing to the shear strength of a fine-grained soil. The angle of internal friction is negligible for fine-grained soils. Therefore, the shear strength is only dependent on the cohesion of a specific soil and does not change as a function of relative confinement.

This study shows that an increase in confining pressure in fine-grained soils does not significantly affect its DCP values. However, the DCP values are significantly affected by soil classification. In this respect, DCP values of in situ soils can be determined and meaningfully compared without having to determine the confining pressure. This study confirmed the findings by Kleyn (34) and) Harrison (39) that gradation and plasticity of the soil affect DCP values. No previous study on the direct effects of confining pressure was found in the literature.

5.1.4. Correlation Analysis of DCP Values With the Resilient Modulus

The M_r and the DCP values for 15 fine-grained soils at optimum moisture content and 8 fine-grained soils at optimum moisture content +20% states are presented in Tables 13 and 14. A graphical plot of these data are shown in Figures 24 and 25 for the two conditions. Correlation analysis with the M_r showed that the natural logarithm of DCP (Ln DCP) values rather than DCP values demonstrated a stronger linear association. Figure 26, which shows plots of M_r against Ln DCP, illustrates this. The output of the correlation analysis is shown in Appendix A, Figure 34. For soils at the optimum moisture content, the correlation coefficient of the Ln DCP values with M_r indicates a significant linear trend ($r = -0.6109$, $p = 0.0156$).

For soils at the optimum moisture content +20%, the mean DCP value was 6.4652 and the mean M_r value was 2924. The standard deviations were 1.4861 and 799.7283, respectively. There is no significant association between the DCP values and resilient modulus ($r = -0.0143$, $p = 0.9737$).

The DCP values of fine-grained soils in a near saturated state, which is analogous to the optimum moisture content +20% in this study, have questionable validity according to several studies (21, 31, 41). The presence of large amounts of soft clay significantly increases friction on the DCP rod. The increased friction results in the soils appearing to be significantly stronger than they actually are (41).

The resilient modulus determination at a near saturated state may also contribute to the poor correlation with the DCP values. Elliott et al. (44), in his study on resilient modulus testing, found that the M_r/moisture content relationship was not negatively linear for all values of moisture content. M_r values decreased rapidly upon slight increase in moisture content beyond a certain point. Many possible explanations can be offered for the poor correlation, but an in-depth evaluation is beyond the scope of this study.

This study shows that the Ln DCP/M_r correlation of fine-grained soils is significant at the optimum moisture content but no correlation between the DCP values and M_r is significant at the optimum moisture content +20%.

TABLE 13

CORRESPONDING RESILIENT MODULUS AND DCP VALUES FOR FINE-GRAINED SOILS AT OPTIMUM MOISTURE CONTENT

Soil Description	DCP Values (in./blow)	Resilient Modulus (psi)
Woodward	3.9	4151
Heiden2	3.2	5848
Landslide	4.6	3423
Bengal	5.1	2963
Clebit	5.0	2814
Quinlan	4.7	2639
Heiden1	4.7	4098
Durant1	2.7	4811
Carnasaw	3.9	4295
Dalhart	5.3	4022
Osage	9.5	2725
St. Paul	6.7	2278
Darnell	5.8	5005
Dougherty	3.8	4045
Durant2	5.4	4450

TABLE 14

CORRESPONDING RESILIENT MODULUS AND DCP VALUES FOR FINE-GRAINED SOILS AT OPTIMUM MOISTURE CONTENT +20%

Soil Description	DCP Values (in./blow)	Resilient Modulus (psi)
Woodward	6.1	3350
Heiden2	8.1	3175
Landslide	5.4	2287
Bengal	8.1	2684
Clebit	6.0	2184
Quinlan	5.5	2119
Heiden1	8.2	3300
Durant1	4.3	4485

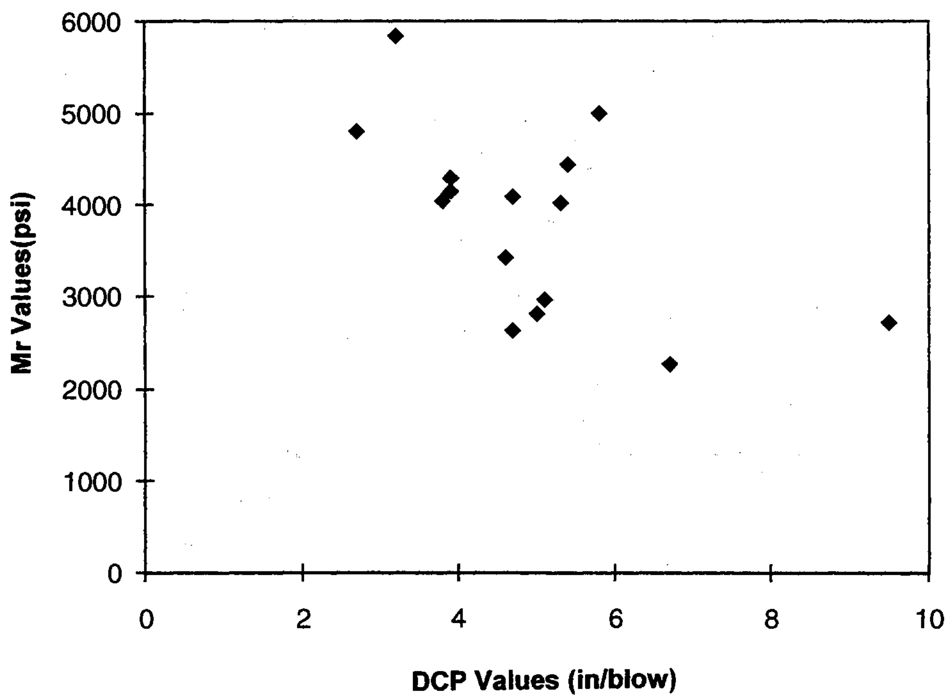


Figure 24. Plots of M_r Against DCP Values for Soils
at Optimum Moisture Content

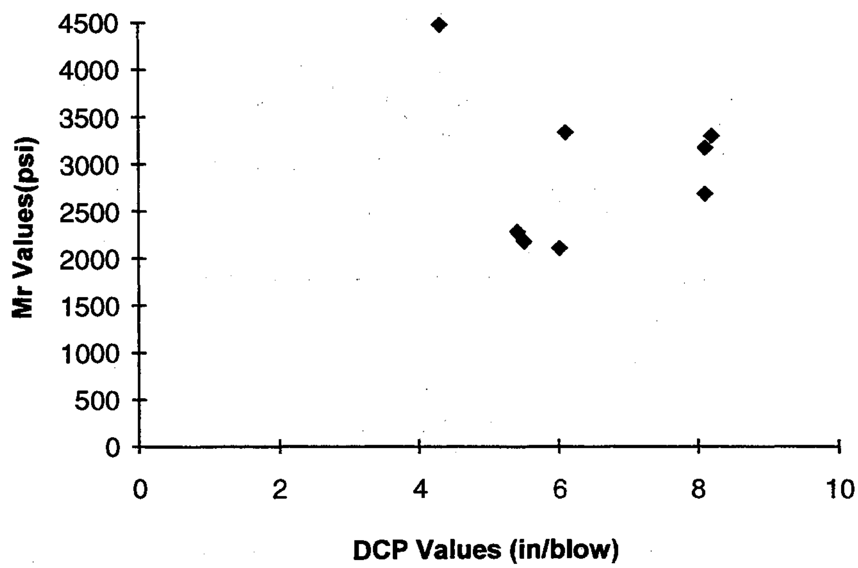


Figure 25. Plots of M_r Against DCP Values for Soils at Optimum Moisture Content +20%

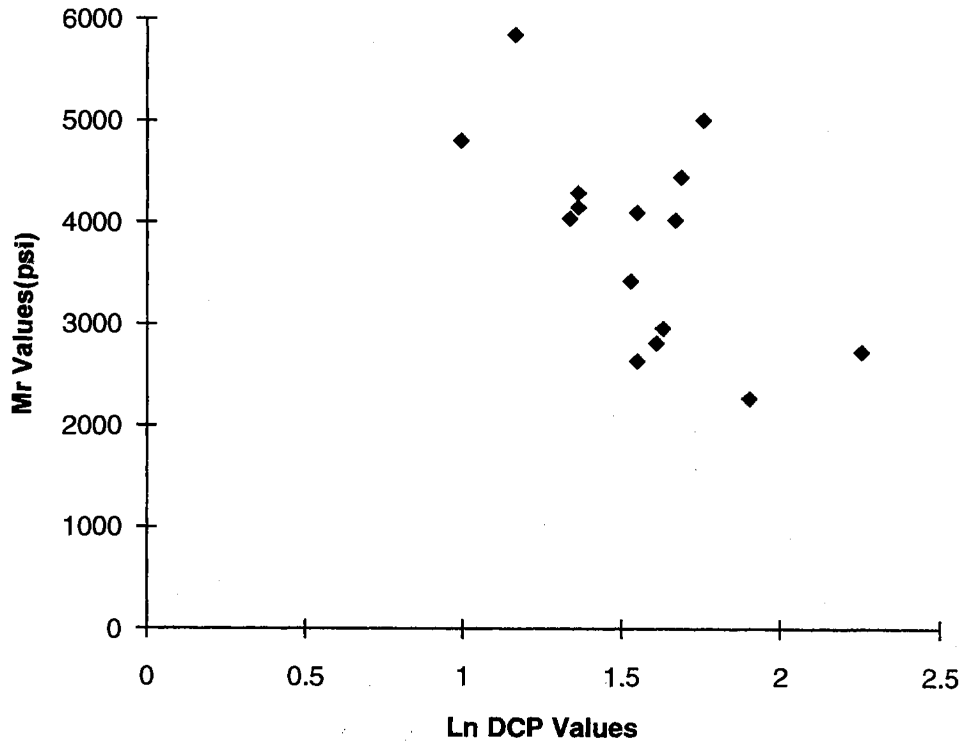


Figure 26. Graph of M_r Against Ln DCP

5.1.5. Simple Linear Regression to Predict M_r From DCP Values

Simple linear regression analysis involving M_r as the dependent variable and DCP value as the independent variable was carried out at the optimum moisture content. The outcome of the correlation analysis described in the previous section was utilized. The output of the linear regression analysis is shown in Appendix A, Figure 35.

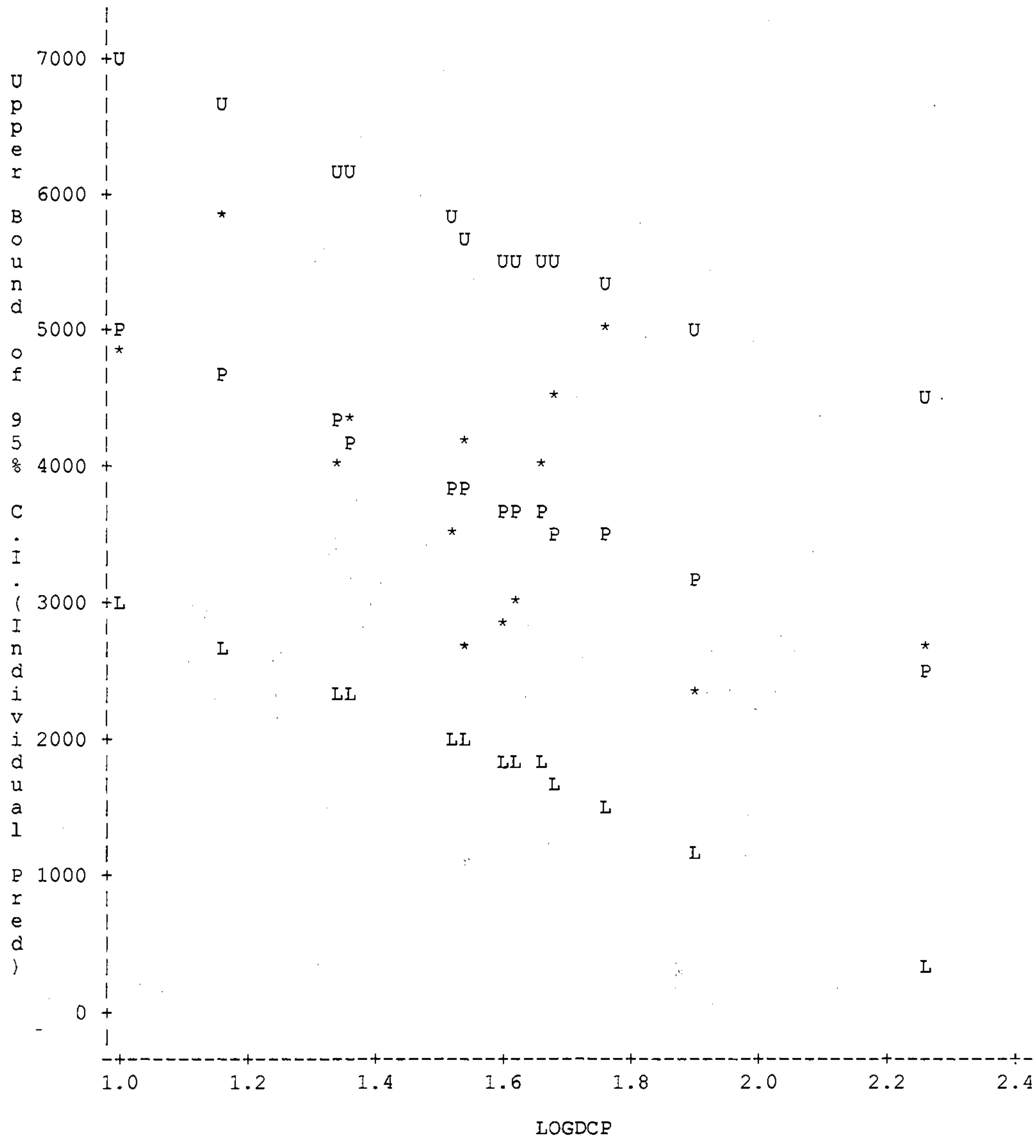
For soils tested at the optimum moisture content, the following relationship was developed and found to be better than other simple linear models:

$$M_r = 7013.065 - 2040.783 \text{ Ln DCP}$$

The observed significance level of the slope was significant ($p = 0.0156$). The coefficient of determination, R^2, was found to be 0.3732.

Although the R^2 value was low, an analysis of the data plots with the prediction equation in conjunction with the lower and upper bounds of the 95% confidence limit showed that these plots are reasonable as shown in Figure 27. Since most observations

```
Plot of U95*LOGDCP.  Symbol used is 'U'.
Plot of L95*LOGDCP.  Symbol used is 'L'.
Plot of P*LOGDCP.    Symbol used is 'P'.
Plot of MR*LOGDCP.   Symbol used is '*'.
```



NOTE: 7 obs hidden.

Figure 27. Prediction Intervals of Developed Relationship

were within the root mean square distance from the predicted linear regression equation, the model was deemed appropriate.

There is a practical design application for this relationship. Based on a specific design M_r value, the corresponding lower and upper bounds of DCP values can be determined by inverse regression techniques (46). For a specified resilient modulus value, the engineer will be able to estimate the corresponding maximum DCP value the soil should not exceed. For example, consider a specification of a minimum resilient modulus of 5000 psi. Referring to the developed relationship in Figure 27, if a straight line was to be produced horizontally from 5000 psi on the Y-axis until it reached the upper bound line, and projected vertically downward to the X-axis, it would provide the limiting value of the natural logarithm of the DCP value for the minimum resilient modulus value. In this case, a value of 1.95 was obtained for the natural logarithm of the DCP (DCP = 7.03). Field verification with the DCP device can be carried out to ensure that the maximum DCP values are not exceeded, implying minimum M_r values are met.

For soils tested at the optimum moisture content +20%, after considering various transformation possibilities, no linear relationship was found. DCP values alone were inadequate to explain the dependent variable, the resilient modulus.

The contrasting significance of the relationship between the resilient modulus and the DCP values of fine-grained soils at the two moisture states suggests that in using DCP values to predict M_r values, knowledge of the moisture state of the soil is very important. A relationship may be developed only for a particular moisture state and should not be generalized for all conditions. The usefulness of the DCP device in predicting M_r in fine-grained soils may be handicapped by the need to determine the moisture state of the soil tested. The use of the speedy moisturemeter device may be useful to overcome this situation.

5.2 Granular Materials

5.2.1 Effect of Maximum Aggregate Size and Confining Pressure on DCP Values

The DCP penetration data are shown in Appendix B, Tables 25 through 27. The DCP values for the six aggregate samples tested at 6, 12, and 18 psi confining pressure are summarized in Table 15. The values were generally lower than in fine-grained soils. Graphs showing DCP values against confining pressure and DCP values against maximum aggregate size for the three confining pressures are shown in Figures 28 and 29, respectively. Figure 30 graphically shows DCP values against the coefficient of uniformity at the three confining pressures for the various materials tested. The results of the analysis to evaluate the effect of maximum aggregate size and confining pressure on DCP values are presented in Appendix B, Figure 42.

TABLE 15

DCP VALUES OF GRANULAR MATERIALS AT VARIOUS CONFINING PRESSURES

Material Source	Maximum Agg. Size (in.)	Coefficient of Uniformity (Cu)	DCP Values 6	12	18
OSU Sand	0.25	6.3	1.7	0.9	0.3
OSU 3/8	0.38	19.8	0.5	0.2	0.1
OSU 3/4	0.75	18.8	0.6	0.4	0.1
ODOT Sand	0.25	3.3	4.1	2.3	0.8
ODOT 3/8	0.38	12.6	0.7	0.3	0.2
ODOT 3/4	0.75	12.4	1.0	0.6	0.4

Confining pressure in psi.

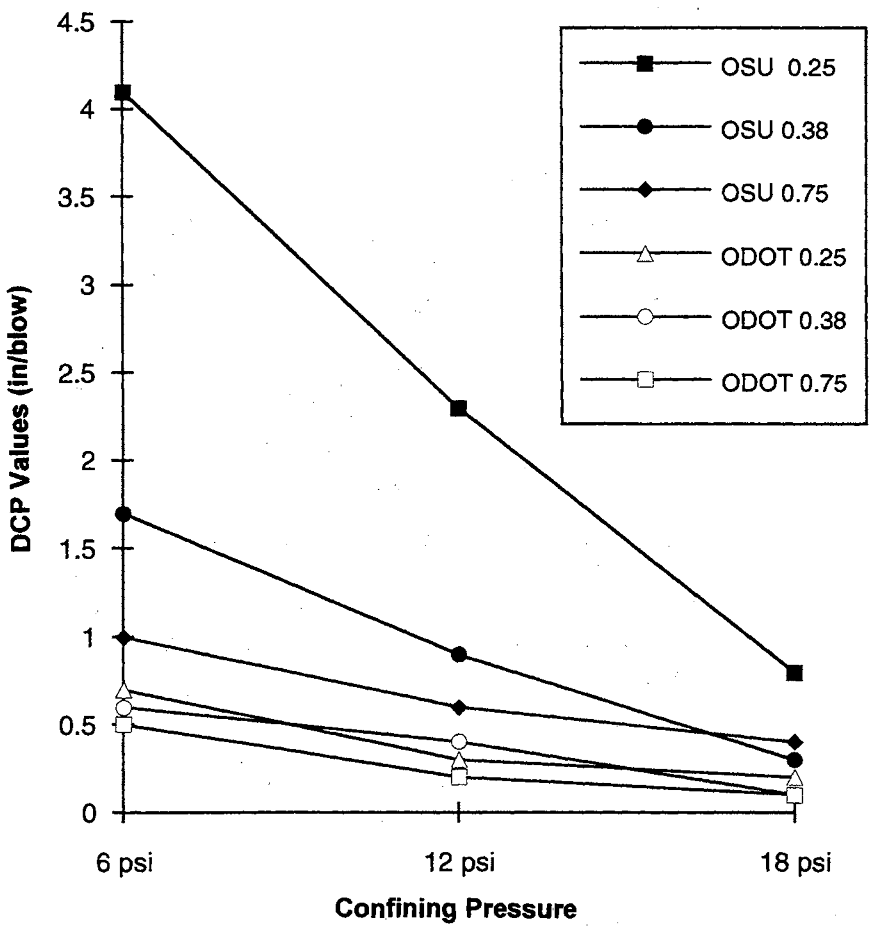
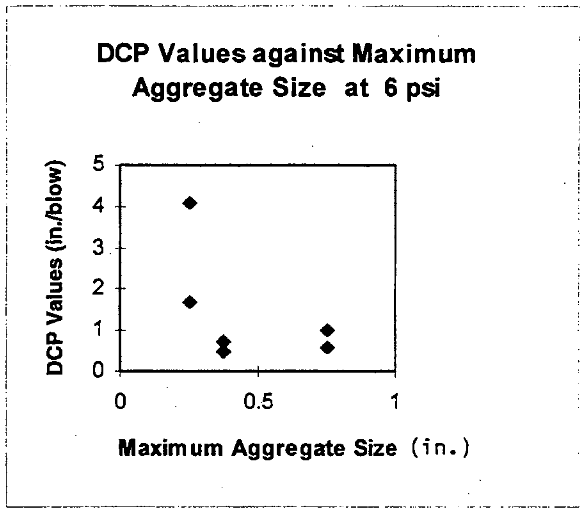
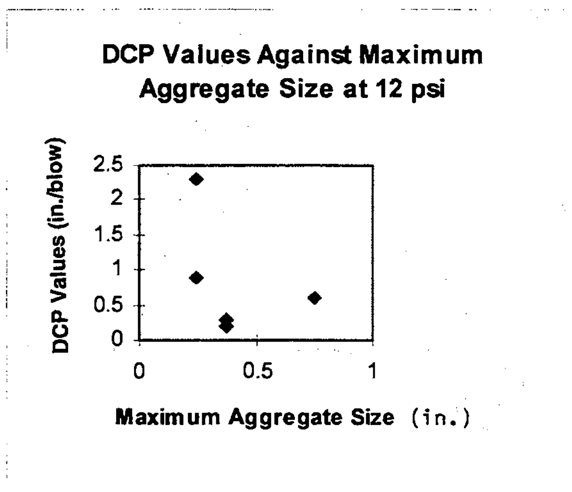


Figure 28. Graph of DCP Values Against Confining Pressure





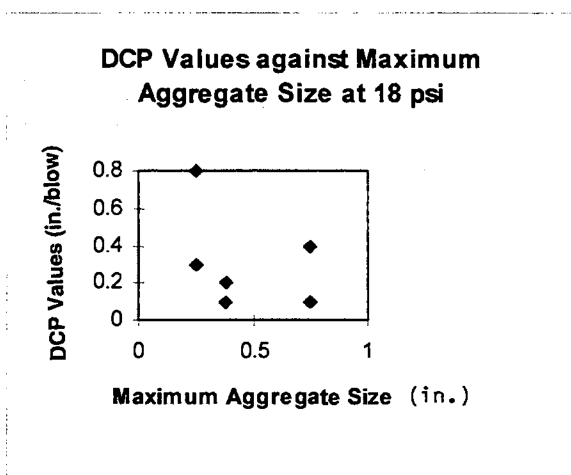


Figure 29. Graphs of DCP Values Against Maximum Aggregate Size

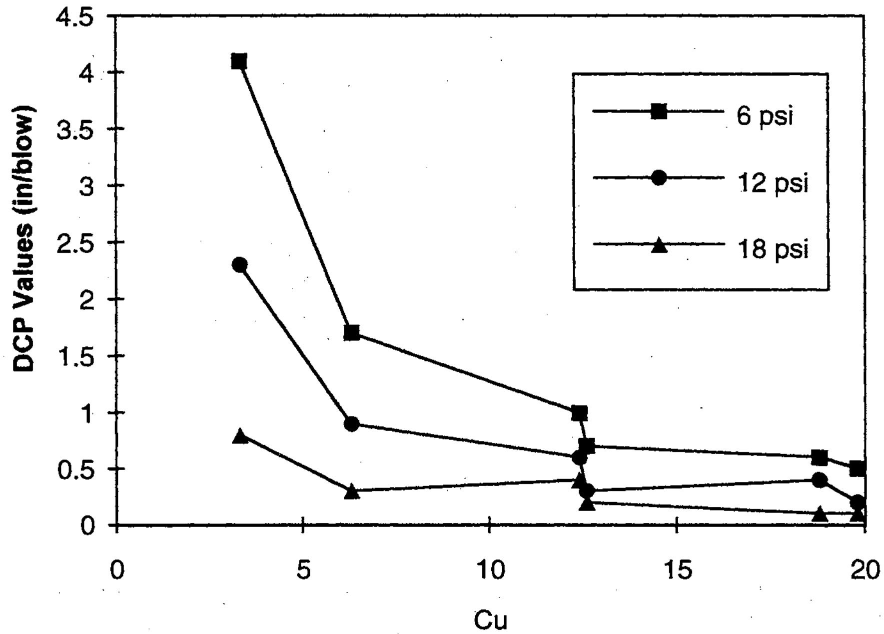


Figure 30. Graph of DCP Values Against Coefficient of Uniformity

There is no significant interaction between confining pressure and maximum aggregate size in determining DCP values ($p = 0.0731$). Maximum aggregate size is also not a significant factor in affecting DCP values ($p = 0.1892$). However, confining pressure is significant in determining DCP values ($p = 0.0075$). The trend analysis contrast of this split plot, which analyzes the effects of confining pressure on the DCP values, showed a significant linear trend ($p = 0.0026$), as shown in the output in Appendix B, Figure 42. The "lack of fit" analysis was not significant ($p = 0.6556$). That is, the linear trend adequately describes the relationship as shown in Figure 28.

Confining pressure levels are significant for granular materials unlike in fine-grained soils. Findings by Ayers (14) regarding the insignificant effect of maximum aggregate size on DCP values of granular materials were thus confirmed. No studies regarding the confining pressure effect on DCP values was found in the literature.

5.2.7 Correlation Analysis of Maximum Aggregate Size and Coefficient of Uniformity With DCP Values at Three Confining Pressure Levels

The Spearman Correlation Coefficient for DCP values with the maximum aggregate size at the three levels of confining pressure was not significant [(6 psi, $r = -0.5976$, $p = 0.2103$); (12 psi, $r = -0.1213$, $p = 0.8190$); and (18 psi, $r = -0.4244$, $p = 0.4016$)]. The correlation analysis is summarized in Table 16. The findings from the analysis showed that there is evidence to indicate that no significant linear association exists between the maximum aggregate size and the DCP values for the three levels of confining pressure.

The coefficient of uniformity is a measure of the grain size distribution in a material. The correlation analysis for DCP values with the coefficient of uniformity using Spearman's Correlation Coefficient showed they were all significant at the three levels of confinement (6 psi, $r = 1.000$, $p = 0.0001$; 12 psi, $r = -0.8117$, $p = 0.0499$; and 18 psi, $r = -0.9276$, $p = 0.0077$). The summary of this analysis is shown in Table 16. Figure 30 suggests, in all the cases, there is a curvilinear relationship. A quadratic trend relating DCP value to Cu can be observed. There is evidence to show there is a significant negative linear association between the coefficient of uniformity and the DCP values for the three levels of confining pressure.

Generally, materials with a higher coefficient of uniformity have better gradation that leads to higher density. For a specified confining pressure, as the coefficient of uniformity increases, the DCP values linearly decrease. The analysis confirmed findings by Ayers (14) and Kleyn (34) that gradation affects DCP values in granular materials.

TABLE 16

CORRELATION ANALYSIS OF DCP VALUES WITH MAXIMUM AGGREGATE SIZE AND CU

Confining Pressure	Correlation Coefficient	Significance Level
DCP Vs Maximum Aggregate Size		
6	-0.5967	0.2103
12	-0.1213	0.8190
18	-0.4244	0.4016
DCP Vs Coefficient of Uniformity		
6	-1.0000	0.0001
12	-0.8117	0.0499
18	-0.9276	0.0077

CHAPTER VI

CONCLUSIONS

6.1 Summary

The main focus of this study was to conduct a series of tests on fine-grained soil samples and granular materials to determine the effects of various factors on Dynamic Cone Penetrometer (DCP) values. Other penetration testing procedures were also discussed. The existing pavement materials characterization methods and pavement design procedures were broadly reviewed.

The effect of an increase in moisture content on DCP values was analyzed. DCP values were analyzed to study the effect of an increase in density from 100% to 110% maximum dry density. Tests to evaluate the effects of confining pressure and AASHTO soil classification on DCP values were performed at 0, 15, and 30 psi on A-6 and A-7 soils. An attempt was made to establish a relationship between the resilient modulus and DCP values for the two moisture states. DCP values of six granular materials were determined at a relative confinement of 6, 12, and 18 psi through the use of a modified triaxial cell. Two sources of materials providing three categories of the granular materials (sand, 3/8- and 3/4-inch maximum aggregate size) were studied. The effects of maximum aggregate size, coefficient of uniformity, and confining pressure were determined.

6.2 Conclusions

The major conclusions drawn from this study can be summarized as follows:

1. Penetration testing methods offer a viable alternative to more expensive and time consuming procedures in characterizing fine-grained soils and granular materials. These methods should be considered in expediting the pavement design process providing meaningful correlation to established pavement design parameters such as CBR and

resilient modulus. More research is required if these methods are to be used directly in any major pavement design procedure.

2. The Dynamic Cone Penetrometer (DCP) is one of the most useful among the penetration testing methods for pavement design and evaluation. The DCP has been extensively correlated with CBR, unconfined compressive strength, and shear strength. However, the DCP is used as a direct parameter only in low volume road design methods in South Africa, Israel, and Australia.

3. The DCP values for fine-grained soils are influenced by the following:
   a. an increase in moisture content significantly decreases DCP values
   b. DCP values are related to AASHTO soil classification of fine-grained soils
   c. an increase in dry density significantly decreases DCP values
   d. an increase in confining pressure does not significantly influence DCP values.

4. The DCP values of granular materials are influenced by the following :
   a. confining pressure is a significant factor in determining DCP values
   b. maximum aggregate size is relatively insignificant in determining DCP values
   c. the coefficient of uniformity has a significant negative correlation to DCP values.

5. For fine-grained soils, the correlation of M_r values with the DCP is significant at optimum moisture content but insignificant at optimum moisture content +20%. (Note: optimum moisture content +20% means 1.2 times the optimum moisture content.) However, DCP values alone cannot be confidently used to predict M_r values for design in fine-grained soils.

6. In DCP testing, confining pressure is insignificant for fine-grained soils but significant for granular materials. Granular materials of higher coefficient of uniformity are affected less by confining pressure than materials of lower coefficient of uniformity.

6.3 Uses of DCP

The possible uses of the DCP devices are:

1. As a design input parameter through correlations with either CBR and resilient modulus.

2. As a direct design input parameter in low volume road pavement.

3. As an evaluation method to determine existing pavement support condition.

4. As a compaction control device.

5. To verify the backcalculated resilient modulus from the Falling Weight Deflectometer test.

6. As a method to be used for preliminary design in pavement design proper.

6.4 Recommendations

1. Future studies to refine the relationship between resilient modulus and DCP values for fine-grained soils should include a greater number of samples covering additional AASHTO classification categories. The correlation study should include samples at optimum moisture content +5, +10, and +15 %.

2. The sample size should be increased so that more DCP values for fine-grained soils at higher moisture contents can be obtained.

3. The study of the effects of AASHTO soil classification should include more soil types and involve more number of samples for each category.

4. A detailed study of the effect of the coefficient of uniformity and maximum aggregate size and their interaction effects on the DCP values of various types of granular materials should be undertaken.

5. Correlation studies between DCP values and M_r values for granular materials at various confining pressure should be carried out.

BIBLIOGRAPHY

(1) Oglesby, C. H. and Hicks, R. G. *Highway Engineering*. John Wiley & Sons, 1982.

(2) Yoder, E. J. and Witczak, M. W. *Principles of Pavement Design*. Wiley-Interscience, 1975

(3) Shahin, M. Y. *Pavement Management for Airports, Roads, and Parking Lots*. Chapman and Hall, 1994.

(4) Haas, R. and Hudson, W. R. *Pavement Management System*. McGraw-Hill Book Company, 1978.

(5) Transportation Research Board. "Pavement Structure Design Practices." *NCHRP Synthesis* 189, 1993.

(6) Transportation Research Board. "Compaction of Difficult Soils and Resilient Modulus Testing." *Transportation Research Record No. 1462*. National Academy Press, 1994.

(7) AASHTO. *Guide for Design of Pavement Structures*. American Association of State Highway Officials,1986.

(8) The Asphalt Institute. "Thickness Design—Asphalt Pavement for Highways and Streets." Manual Series No. 1, September 1981.

(9) Portland Cement Association. "Thickness Design for Concrete Highway and Street Pavements," 1984.

(10) Ullidtz, P. and Coetzee, N. F. "Analytical Procedures in Nondestructive Testing Pavement Evaluation." *Transportation Research Record No. 1482*. National Academy Press, 1995.

(11) Carter, B. J. "Chemical and Clay Mineralogical Characterization of Key Oklahoma Soils Selected for Resilient Modulus Determination." Oklahoma State University, September 1993.

(12) "Manual on Subsurface Investigations." American Association of State Highway Officials, 1988.

(13) "Geophysical Exploration." *Engineer Manual*. U.S. Army Corps of Engineers, May 1979.

(14) Ayers, M. E. "Rapid Shear Strength Evaluation of In Situ Granular Materials Utilizing the Dynamic Cone Penetrometer." Ph.D. dissertation. University of Illinois, 1990.

(15) ERES Consultants. " Techniques for Pavement Rehabilitation." Training Course—Participant's Notebook. Champaign, IL, 1987.

(16) Melzer, K. J. and Smoltczyk, U. "Dynamic Penetration Testing—State of the Art Report." *Proceedings, Second European Symposium on Penetration Testing.* Amsterdam, May 24-27, 1982, pp. 191-202.

(17) Schmertmann, J. H. "Penetration Testing in the U.S.A." *Proceedings, European Symposium on Penetration Testing.* Stockholm, June 5-7, 1974, pp. 217-219.

(18) Decourt, L. et al. "SPT: International Reference Test Procedure." *Proceedings, First International Conference on Penetration Testing.* Orlando, FL, March 20-24, 1988, pp. 3-26.

(19) Livneh, M. and Ishai, I. "The Relationship Between In Situ CBR Test and the Various Penetration Tests." *Proceedings, First International Conference on Penetration Testing.* Orlando, FL, March 20-24, 1988, pp. 445-452.

(20) Bukoski, R. F. and Selig, E. T. "Cone Penetration Testing and Experience." Geotechnical Eng. Division at the ASCE National Convention. St. Louis, MO, 1981, pp. 228-236.

(21) Sangelerat, G. *The Penetrometer and Soil Exploration.* Elsevier, 1972.

(22) Mitchell, J. M. "New Developments in Penetration Tests and Equipment." *Proceedings, First International Symposium on Penetration Testing.* Orlando, FL, March 20-24, 1988, pp. 245-262.

(23) Babu-Tweneboah, K. et al. "CPT and DMT Testing of Highway Pavements in Florida." *Proceedings, First International Symposium on Penetration Testing.* Orlando, FL, March 20-24, 1988, pp. 627-634.

(24) Meigh, A. C. "Cone Penetration Testing—Methods and Interpretation." Ciria, 1987.

(25) DeBeer, E. E. et al. "CPT: International Reference Test Procedure." *Proceedings, First International Symposium On Penetration Testing.* Orlando, FL, March 20-24, 1988, pp. 27-52.

(26) Lutenegger, A. J. "Current Status of the Marchetti Dilatometer." *Proceedings, First International Conference on Penetration Testing.* Orlando, FL, March 20-24, 1988, pp. 137-143.

(27) Mair, R. J. and Wood, D. M. "Pressuremeter Testing—Methods and Interpretation." Ciria, 1987.

(28) Briaud, J. L. et al. "Using a Pressuremeter for Pavement Design and Evaluation." *Proceedings, International Symposium on Bearing Capacity of Roads and Airfields.* Norway, June 23-25, 1982, pp. 484-492.

(29) Clough, G. W, Briaud, J. L., and Hughes, J. M. O. "The development of Pressuremeter Testing." *Proceedings, Third International Symposium on Pressure-meters.* Oxford University, England, April 1990, pp. 25-45.

(30) Cosentino, P. J. and Chen, Y. "Correlating Resilient Modulus from Pressuremeter Tests to Laboratory CBR Tests." *Transportation Research Record No. 1309.* Geotechnical Engineering, 1991, pp. 56-65.

(31) Steffanoff, G. et al. "Dynamic Probing—International Reference Test Procedure." *Proceedings, First International Symposium on Penetration Testing.* Orlando, FL, March 20-24, 1988, pp. 53-70.

(32) Scala, A. J. "Simple Methods of Flexible Pavement Design Using Cone Penetrometers." *New Zealand Engineering*, Vol. 11, No. 2, February 15, 1956.

(33) Van Vuuren, D. J. "Rapid Determination of CBR With the Portable Dynamic Cone Penetrometer." *The Rhodesian Engineer*, Paper No. 105, September, 1969.

(34) Kleyn, E. G. and Savage, P. F. "The Application of the Pavement DCP to Determine the Bearing Properties and Performance of Road Pavements." *International Symposium on Bearing Capacity of Roads and Airfields*. Trodheim, Norway, June 1982.

(35) Kleyn, E. G., Van Heerden, M. J. J., and Rossouw, A. J. "An Investigation to Determine the Structural Capacity and Rehabilitation Utilization of a Road Pavement Using the Pavement Dynamic Cone Penetrometer." *International Symposium on Bearing Capacity of Roads and Airfields*. Trodheim, Norway, June 1982.

(36) Meier, R. W. and Baladi, G.Y *Cone Index Based Estimates of Soil Strength: Theory and User's Guide for Computer Code CIBESS*. Technical Report No. SL-88-11. Waterways Experiment Station, Vicksburg, MS, March 1988.

(37) Freitag, D. R. "Penetration Tests for Soil Measurements." *Transactions of the ASCE*, Vol. 11, No. 6, 1968.

(38) Allersma, H. G. B. "Optical Analysis of Stress and Strain Around the Tip of a Penetrating Probe." *Proceedings, First International Symposium on Penetration Testing*. Orlando, FL, March 20-24, 1988, pp. 615-620.

(39) Harrison, J. A. "Correlation Between CBR and DCP Strength Measurements of Soils." *Proceedings, Instituition of Civil Engineers*, Part 2, December 1983.

(40) Chua, K. M. "Determination of CBR and Elastic Modulus of Soils Using a Portable Pavement Dynamic Cone Penetrometer." *Proceedings, First International Symposium on Penetration Testing*. Orlando, FL, March 20-24, 1988, pp. 407-414.

(41) McGrath, P. G. et al. "Development of Dynamic Cone Penetration Testing in Ireland." *Proceedings, Twelfth International Conference on Soil Mechanics and Foundation Engineering*. Rio De Janeiro, August 13-18, 1989, pp. 271-276.

(42) Bester, M. D. and Hallat, L. "Dynamic Cone Penetrometer." University of Pretoria, Pretoria, 1977.

(43) Holtz, R. D and Kovacs, W. D. *An Introduction to Geotechnical Engineering*. Prentice Hall, Inc., 1981.

(44) Lambe, T. W. and Whitman, R. V. *Soil Mechanics*. John Wiley & Sons, 1979.

(45) Elliott, R. P. and Thornton, S. I. "Resilient Modulus and AASHTO Pavement Design." *Transportation Research Record No. 1196*. National Academy Press, 1989.

(46) Draper, N. R. and Smith, H. *Applied Regression Analysis*. 2nd Edition. John Wiley & Sons, 1981.

APPENDIX A

SOIL INFORMATION, TEST RESULTS, AND SAS OUTPUT FOR FINE-GRAINED SOILS

TABLE 17

PHYSICAL AND CHEMICAL PROPERTIES OF FINE-GRAINED SOILS (11)*

	Soil Series	LL	PL	<2mm S	<2mm Si	<2mm C	% OM	PCF SD	% MM	PCF MD	PSI Rm	mm +20	MD +20	Rm +20	Rm-Rm+20	% OC	Fe	% CCE	CEC	pH	EC	EA
1	Westsum	51	33	11	45	44	19.5	103.8	20.1	103.4	4152	23.7	100.1	3304	848	0.3	-	22.0	20.1	8.1	.5	0
2	Renfrow	52	33	22	42	36	20.2	105.6	21.5	103.0	4700	24.7	97.3	3833	867	0.3	-	0	23.1	7.3	.3	1.0
3	Renthin	55	35	23	37	40	23.3	97.8	24.3	96.8	3922	28.0	90.9	3374	548	0.7	-	0	21.6	6.7	.1	5.8
4	Stephenville	28	12	62	16	22	15.0	112.1	16.0	110.8	3777	18.4	106.0	3052	725	0.4	-	0	11.2	5.2	0	5.6
5	Kirkland	61	43	11	38	51	23.4	96.8	23.7	96.3	4225	27.5	92.7	3555	670	0.1	-	TR	33.0	7.8	2.0	0
6	Zanies	33	16	54	17	29	16.5	105.8	17.6	107.0	5509	19.7	103.9	3069	2440	0.6	-	0	13.3	5.7	0	4.1
7	Dalhart	31	17	58	14	28	18.0	103.8	18.8	105.5	4022	21.4	102.5	2851	1171	0.4	.2	1.9	15.3	7.3	.3	0
8	St. Paul	41	25	26	37	37	20.4	98.1	23.2	97.4	2776	25.0	95.6	2538	238	0.6	.1	3.5	14.1	8.1	.3	0
9	Woodward	NP	NP	52	31	17	14.2	109.2	14.8	110.5	4151	17.8	107.6	3350	801	0.4	0	7.4	10.6	7.4	.3	0
10	Pratt	NP	NP	90	3	7	12.0	110.0	-	-	-	-	-	-	-	0.2	0	0	8.0	6.9	.1	2.6
11	Dougherty	27	12	70	2	28	14.5	108.6	15.3	108.3	4045	17.0	107.5	3849	196	0.2	.2	0	13.4	6.9	.1	2.9
12	Darnell	NP	NP	84	9	7	13.4	104.4	14.2	103.5	5005	15.0	103.0	3193	1812	0.2	.1	0	4.2	7.0	.2	1.3
13	Clebit	25	5	52	34	14	13.4	111.0	14.1	111.9	2814	15.9	109.7	2184	630	0.6	.2	0	15.2	6.8	.2	7.6
14	Bengal	59	27	16	38	46	23.6	96.4	25.4	95.7	2963	28.3	93.8	2684	279	0.5	.9	0	18.7	6.8	.1	15.1
15	Durant 1	63	41	15	40	45	22.4	96.5	23.4	97.6	4811	26.1	93.7	4485	326	0.6	.3	1.2	14.8	7.7	.1	0
16	Durant 2	59	37	15	41	44	21.9	99.9	22.3	100.4	4450	25.6	93.9	3161	1286	0.1	.3	1.9	19.9	8.4	.8	0
17	Heiden 1	64	40	17	35	48	23.0	99.0	23.1	97.0	4098	26.6	93.2	3300	798	0.9	.5	8.0	33.9	8.4	.6	0
18	Heiden 2	63	40	9	38	53	20.5	101.8	21.8	102.1	5848	24.9	98.4	3175	2673	0.6	.4	12.5	19.5	8.2	2.0	0
19	Quinlan	31	9	32	44	24	16.0	107.0	17.0	108.0	2639	19.6	101.3	2119	520	0.7	.2	15.4	5.6	7.4	.4	0
20	Richfield	56	38	6	48	46	23.0	94.9	24.4	94.6	4601	27.2	90.6	3551	1050	0.8	.1	3.1	27.5	7.9	.7	0
21	Dennis	50	28	20	38	42	22.0	99.2	23.8	98.0	1964	26.6	91.8	1346	618	0.3	.3	0	29.4	6.9	.5	9.2
22	Osage	73	46	3	45	52	27.8	89.5	29.8	87.7	2725	34.4	83.3	1492	1233	0.9	.3	0	50.7	7.5	.8	13.5
23	Clarksville	55	28	26	37	37	25.4	89.7	24.8	91.7	4194	29.3	85.3	2613	1581	0.3	.3	0	57.7	6.8	.1	14.1
24	Carnasaw	82	44	4	22	74	32.3	86.6	33.8	85.9	4295	39.2	79.4	1745	2550	0.2	.3	0	46.2	6.3	.1	38.1
25	Landslide	41	15	33	30	37	17.8	108.1	18.6	107.0	3423	21.2	102.0	2287	1136	0.2	.3	0	19.5	6.4	0.6	17.2

*LL—liquid limit; PL—plastic limit; NP—nonplastic; S—sand; Si—silt; C—clay; OM—optimum moisture; SD—standard density; MM—molded moisture; MD—molded density; Rm—resilient modulus; OC—organic carbon; Fe—extractable Fe_2O_3 (%); CCE—calcium carbonate equivalence; CEC—cation exchange capacity (1 cmol (+) per kg soil); EC—electrical conductivity (1 ds per m); EA—extractable acidity (1 cmol (+) per kg soil); Sm—smectite (formerly montmorillonite); MI—mica (illite); KK—kaolinite; QZ—quartz; HE—hemitite; GE—geothite; V—vermiculite; CL—chlorite; SM-MI; denotes interstratified mineral; (-) denotes analysis not required; TR—trace.

TABLE 18

DCP TEST DATA: SAMPLES AT OPTIMUM MOISTURE CONTENT TESTED AT 3 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Soil Description—Woodward			
0	15.6	0	
1	18.8	3.2	
2	22.7	3.9	
3	26.0	3.3	3.9
2. Soil Description—Heiden2			
0	14.7	0	
1	19.5	4.8	
2	22.7	3.2	
3	26.0	3.3	3.2
3. Soil Description—Landslide			
0	14.2	0	
1	18.3	4.1	
2	22.9	4.6	4.6
4. Soil Description—Bengal			
0	14.8	0	
1	18.3	3.5	
2	23.4	5.1	5.1
5. Soil Description—Clebit			
0	14.5	0	
1	19.9	5.4	
2	24.7	4.8	5.0
6. Soil Description—Quinlan			
0	13.9	0	
1	18.7	4.8	
2	23.4	4.7	4.7
7. Soil Description—Heiden 1			
0	14.1	0	
1	18.2	4.1	
2	22.9	4.7	4.7
8. Soil Description—Durant 1			
0	14.5	0	
1	17.5	3.0	
2	20.4	2.9	
3	23.0	2.6	2.7

TABLE 18 (CONTINUED)

Blow	Reading	Penetration Rate	DCP Value
9. Soil Description—Carnasaw			
0	14.3	0	
1	18.9	4.6	
2	22.8	3.9	3.9
10. Soil Description—Dalhart			
0	14.2	0	
1	19.9	5.7	
2	25.0	5.1	5.3
11. Soil Description—Osage			
0	13.7	0	
1	23.2	9.5	9.5
12. Soil Description—St.Paul			
0	13.9	0	
1	20.6	6.7	6.7
13. Soil Description—Darnell			
0	14.3	0	
1	19.7	5.4	
2	25.7	6.0	5.8
14. Soil Description—Dougherty			
0	14.1	0	
1	18.6	4.5	
2	22.4	3.8	3.8
15. Soil Description—Durant 2			
0	13.4	0	
1	18.8	5.4	5.4

TABLE 19
DCP TEST DATA: SAMPLES AT OPTIMUM MOISTURE CONTENT +20% (TESTED AT 3 PSI)

Blow	Reading	Penetration Rate	DCP Value
1. Soil Description—Woodword			
0	14.4	0	
1	20.5	6.1	6.1
2. Soil Description—Heiden 2			
0	14.5	0	
1	22.6	8.1	8.1
3. Soil Description—Landslide			
0	13.6	0	
1	18.8	5.2	
2	24.3	5.5	5.4
4. Soil Description—Bengal			
0	14.1	0	
1	22.2	8.1	8.1
5. Soil Description—Clebit			
0	14.4	0	
1	19.6	5.2	
2	26.0	6.4	6.0
6. Soil Description—Quinlan			
0	13.8	0	
1	19.1	5.3	
2	24.7	5.6	5.5
7. Soil Description—Heiden 1			
0	13.9	0	
1	22.1	8.2	8.2
8. Soil Description—Durant 1			
0	14.4	0	
1	19.1	4.7	
2	23.4	4.3	4.3

TABLE 20

DCP TEST DATA: SAMPLES AT MAXIMUM
DRY DENSITY TESTED AT 0 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Soil Description—Clebit			
0	14.1	0	
1	18.5	4.4	
2	23.5	5.0	5.0
2. Soil Description—Durant 1			
0	14.6	0	
1	17.8	3.2	
2	20.9	3.1	
3	23.7	2.8	2.9
3. Soil Description—Heiden 2			
0	14.2	0	
1	18.3	4.1	
2	23.8	5.5	5.5
4. Soil Description—Carnasaw			
0	14.3	0	
1	18.9	4.6	
2	23.1	4.2	4.2
5. Soil Description—Dalhart			
0	14.1	0	
1	19.7	5.6	
2	25.0	5.3	5.4
6. Soil Description—Bengal			
0	13.8	0	
1	18.5	4.7	
2	23.6	5.1	5.1
7. Soil Description—Heiden 1			
0	14.6	0	
1	19.2	4.6	
2	24.0	4.8	4.8

TABLE 21

DCP TEST DATA: SAMPLES AT 110% MAXIMUM DRY DENSITY AND TESTED AT 0 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Soil Description—Clebit			
0	14.1	0	
1	17.0	2.9	
2	19.8	2.8	
3	22.0	2.2	
4	24.1	2.1	2.4
2. Soil Description—Durant 1			
0	14.1	0	
1	16.4	2.3	
2	18.7	2.3	
3	20.9	2.2	
4	22.8	1.9	2.1
3. Soil Description—Heiden 2			
0	13.8	0	
1	17.2	3.4	
2	20.4	3.2	
3	23.0	2.6	3.0
4. Soil Description—Carnasaw			
0	14.1	0	
1	17.7	3.6	
2	20.1	2.4	
3	22.2	2.1	2.3
5. Soil Description—Dalhart			
0	14.4	0	
1	18.6	4.2	
2	22.1	3.5	
3	25.5	3.4	3.5
6. Soil Description—Bengal			
0	14.1	0	
1	18.0	3.9	
2	21.3	3.3	
3	24.5	3.2	3.3
7. Soil Description—Heiden 1			
0	14.1	0	
1	17.4	3.3	
2	20.6	3.2	
3	22.9	2.3	2.6

TABLE 22

DCP TEST DATA: SAMPLES AT OPTIMUM MOISTURE CONTENT TESTED AT 0 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Soil Description—Landslide			
0	14.4	0	
1	18.9	4.5	
2	23.3	4.4	4.4
2. Soil Description—Quinlan			
0	14.1	0	
1	19.1	5.0	
2	23.5	4.4	4.6

TABLE 23

DCP TEST DATA: SAMPLES AT OPTIMUM MOISTURE CONTENT TESTED AT 15 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Soil Description—Clebit			
0	14.9	0	
1	19.3	4.4	
2	24.0	4.7	4.7
2. Soil Description—Landslide			
0	14.5	0	
1	19.2	4.7	
2	23.4	4.2	4.2
3. Soil Description—Carnasaw			
0	13.8	0	
1	17.2	3.4	
2	19.8	2.6	
3	21.8	2.0	2.4
4. Soil Description—Dalhart			
0	14.1	0	
1	18.9	4.8	
2	23.8	4.9	4.9
5. Soil Description—Quinlan			
0	14.4	0	
1	19.0	4.6	
2	23.4	4.4	4.4
6. Soil Description—Durant 1			
0	14.8	0	
1	18.3	3.5	
2	21.2	2.9	
3	23.8	2.6	2.8

TABLE 24

DCP TEST DATA: SAMPLES AT OPTIMUM MOISTURE CONTENT TESTED AT 30 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Soil Description—Clebit			
0	14.3	0	
1	19.1	4.8	
2	24.0	4.9	4.9
2. Soil Description—Landslide			
0	14.9	0	
1	19.4	4.5	
2	23.4	4.0	4.0
3. Soil Description—Carnasaw			
0	14.6	0	
1	17.8	3.2	
2	20.1	2.3	
3	22.5	2.4	
4	24.8	2.3	2.4
4. Soil Description—Dalhart			
0	14.1	0	
1	18.6	4.5	
2	23.2	4.6	4.6
5. Soil Description—Quinlan			
0	14.7	0	
1	18.9	4.2	
2	22.8	3.9	
3	26.4	3.6	3.9
6. Soil Description—Durant 1			
0	14.8	0	
1	18.0	3.2	
2	20.4	2.4	
3	22.6	2.2	
4	24.7	2.1	2.3

Univariate Procedure

Variable = Diff

Moments

N	8	Sum Wgts	8
Mean	-2.225	Sum	-17.8
Std Dev	1.480106	Variance	2.190714
Skewness	-0.81124	Kurtosis	-0.26656
Uss	54.94	Css	15.335
Cv	-66.5216	Std Mean	0.523297
T:Mean = 0	-4.25189	Pr > |T|	0.0038
Num ^= 0	8	Num > 0	0
M (Sign)	-4	Pr >= |M|	0.0078
Sgn Rank	-18	Pr >= |S|	0.0078

Quantiles (Def = 5)

100% Max	-0.80	99%	-0.8
75% Q3	-0.90	95%	-0.8
50% Med	-1.90	90%	-0.8
25% Q1	-3.25	10%	-4.9
0% Min	-4.90	5%	-4.9
		1%	-4.9
Range	4.10		
Q3-Q1	2.35		
Mode	-4.90		

Extremes

Lowest	Obs	Highest	Obs
-4.9	(7)	-2.2	(8)
-3.5	(5)	-1.6	(4)
-3.0	(1)	-1.0	(3)
-2.2	(8)	-0.8	(6)
-1.6	(4)	-0.8	(2)

Figure 31. SAS Output to Evaluate Increase in Moisture Content on DCP Values

Univariate Procedure

Variable=Diff

Moments

N	7	Sum Wgts	7
Mean	1.957143	Sum	13.7
Std Dev	0.596817	Variance	0.35619
Skewness	-1.22239	Kurtosis	2.157381
Uss	28.95	Css	2.137143
Cv	30.4943	Std Mean	0.225576
T:Mean = 0	8.676217	Pr > |T|	0.0001
Num ^= 0	7	Num > 0	7
M (Sign)	3.5	Pr > = |M|	0.0156
Sgn Rank	14	Pr > = |S|	0.0156

Quantiles (Def = 5)

100% Max	2.6	99%	2.6
75% Q3	2.5	95%	2.6
50% Med	1.9	90%	2.6
25% Q1	1.8	10%	0.8
0% Min	0.8	5%	0.8
		1%	0.8
Range	1.8		
Q3-Q1	0.7		
Mode	1.9		

Extremes

Lowest	Obs	Highest	Obs
0.8	(2)	1.9	(4)
1.8	(6)	1.9	(5)
1.9	(5)	2.2	(7)
1.9	(4)	2.5	(3)
2.2	(7)	2.6	(1)

Figure 32. SAS Output to Evaluate Increase in Density on DCP Values

General Linear Models Procedure

Class	Level	Information
Class	Level	Values
Type	2	A6 A7
Con	3	0 15 30

Number of Observations in Data Set = 18

General Linear Models Procedure
Dependent Variable: Dcp

Source	Df	Sum of Squares	Mean Square	F Value	Pr > F
Model	5	10.95333333	2.19066667	4.45	0.0159
Error	12	5.90666667	0.49222222		
Corrected Total	17	16.86000000			
R^2	C.V.	Root Mse	DCP Mean		
	0.649664	17.53964	0.7015855	4.0000000	

Source	Df	Type I SS	Mean Square	F Value	Pr > F
Type	1	9.10222222	9.10222222	18.49	0.0010
Con	2	1.70333333	0.85166667	1.73	0.2186
Type*Con	2	0.14777778	0.07388889	0.15	0.8622

Source	Df	Type III SS	Mean Square	F Value	Pr > F
Type	1	9.10222222	9.10222222	18.49	0.0010
Con	2	1.70333333	0.85166667	1.73	0.2186
Type*Con	2	0.14777778	0.07388889	0.15	0.8622

T Tests (Lsd) for Variable: DCP

Note: This test controls the type 1 comparisonwise error, not the experimentwise error rate.

Alpha = 0.05 Df = 12 Mse = 0.492222
Critical Value of T = 2.18
Least Significant Difference = 0.7206

Means with the same letter are not significantly different.

T Grouping	Mean	N	Type
A	4.7111	9	A6
B	3.2889	9	A7

Figure 33. SAS Output to Evaluate Effects of AASHTO Soil Classification and Confining Pressure on DCP Values

Correlation Analysis

2 "var" Variables: Log DCP Mr

Simple Statistics

Variable	N	Mean	Std Dev	Sum	Minimum	Maximum
LogDCP	15	1.55591	0.30221	23.33858	0.99325	2.25129
Mr	15	3838	1010	57567	2278	5848

Pearson Correlation Coefficients/Prob > |R| Under Ho: Rho = 0/N = 15

	LogDCP	Mr
LogDCP	1.00000	-0.61087
	0.0	0.01560
Mr	-0.61087	1.00000
	0.01560	0.0

Figure 34. SAS Output of Correlation Analysis of Mr and Ln DCP Values

Model: Model 1

Dependent Variable: Mr

Analysis of Variance

Source	Df	Sum of Squares	Mean Square	F Value	Prob > F
Model	1	4613072.1444	4613072.14440	6.210	0.0270
Error	13	9657708.2556	472900.63505		
C Total	14	14270780.4000			

Root Mse	861.91684	R^2	0.3233
Dep Mean	3837.80000	Adj R^2	0.2712
C.V.	22.45862		

Parameter Estimates

Variable	Df	Parameter Estimate	Standard Error	T for H0: Parameter = 0	Prob > |T|
Intercep	1	5593.049523	738.70336822	7.571	0.0001
DCP	1	-354.357239	142.20391412	-2.492	0.0270

Model: Model 2

Dependent Variable: Mr

Analysis of Variance

Source	Df	Sum of Squares	Mean Square	F Value	Prob > F
Model	1	5325246.7802	5325246.78020	7.739	0.0156
Error	13	8945533.6198	688117.97076		
C Total	14	14270780.4000			

Root Mse	829.52876	R^2	0.3732
Dep Mean	3837.80000	Adj R^2	0.3249
C.V.	21.61469		

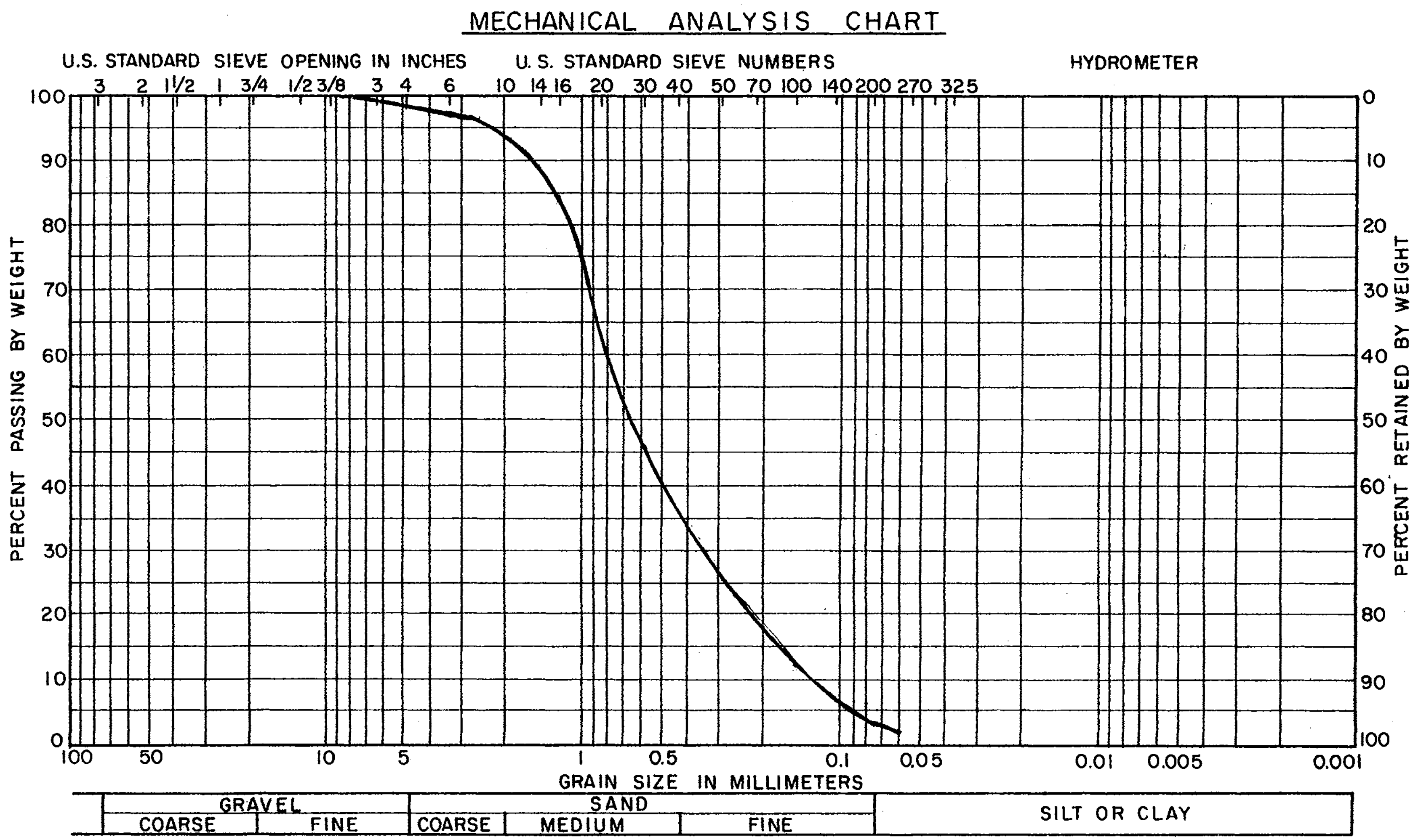
Parameter Estimates

Variable	Df	Parameter Estimate	Standard Error	T for H0: Parameter = 0	Prob > |T|
Intercep	1	7013.065037	1161.3314141	6.039	0.0001
LogDCP	1	-2040.782964	733.59844327	-2.782	0.0156

Figure 35. SAS Output of Linear Regression Analysis of Mr and Ln DCP Values

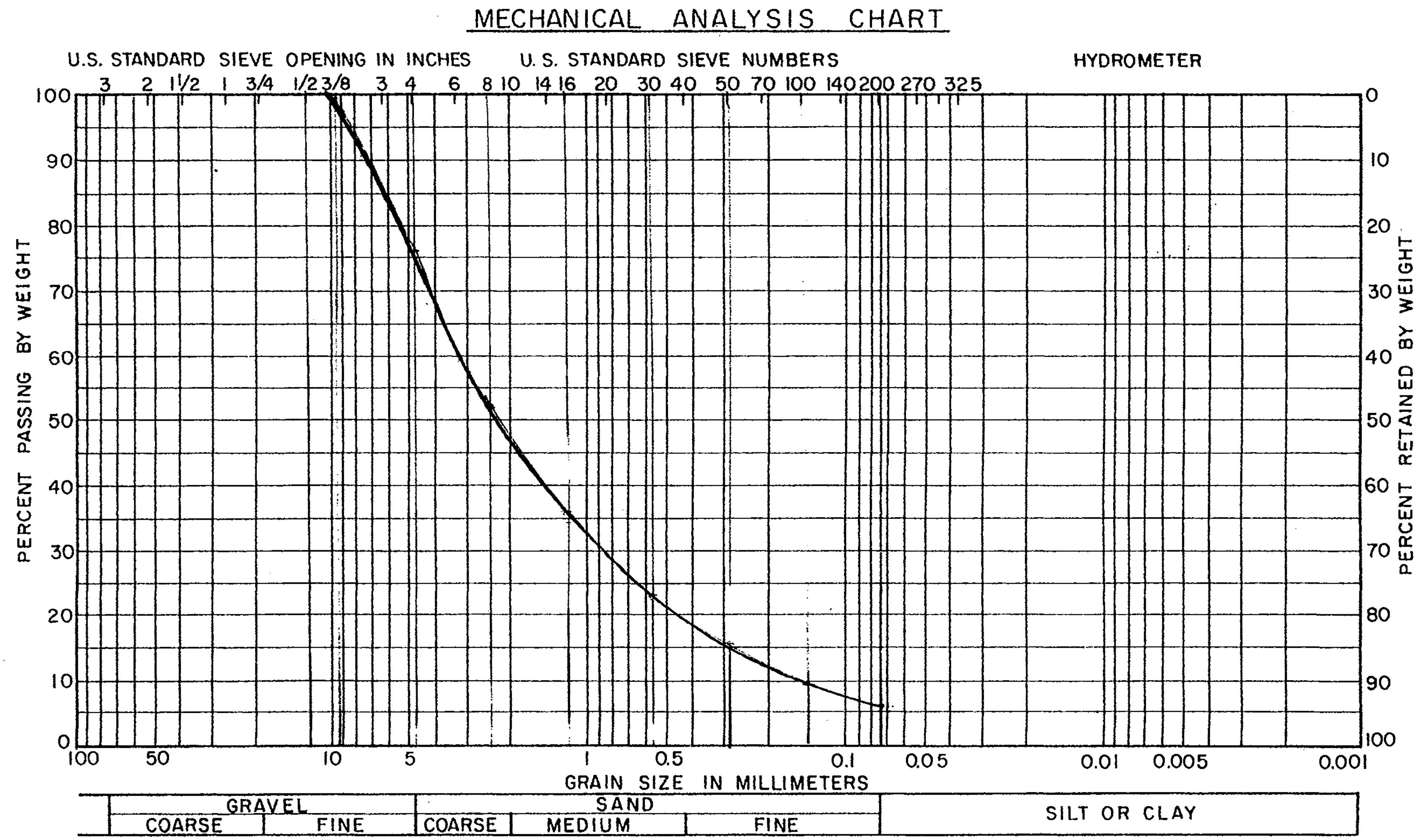
APPENDIX B

SOIL INFORMATION, TEST RESULTS AND SAS OUTPUT FOR GRANULAR MATERIALS



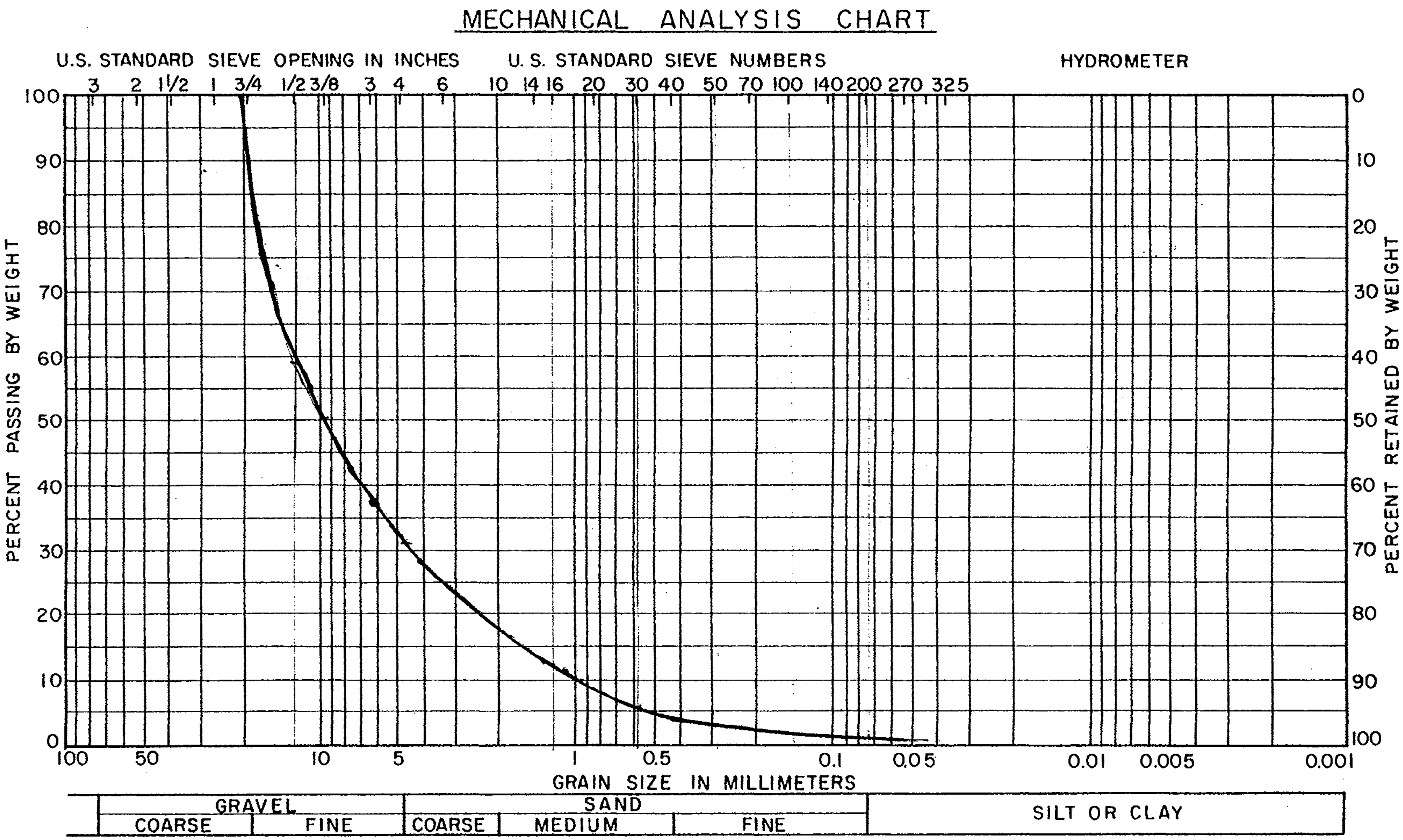
Coefficient of Uniformity (Cu) = 6.3   Dry Density (Compacted) = 115.1 lbs/cu ft

Figure 36. Information and Gradation Chart of OSU Sand



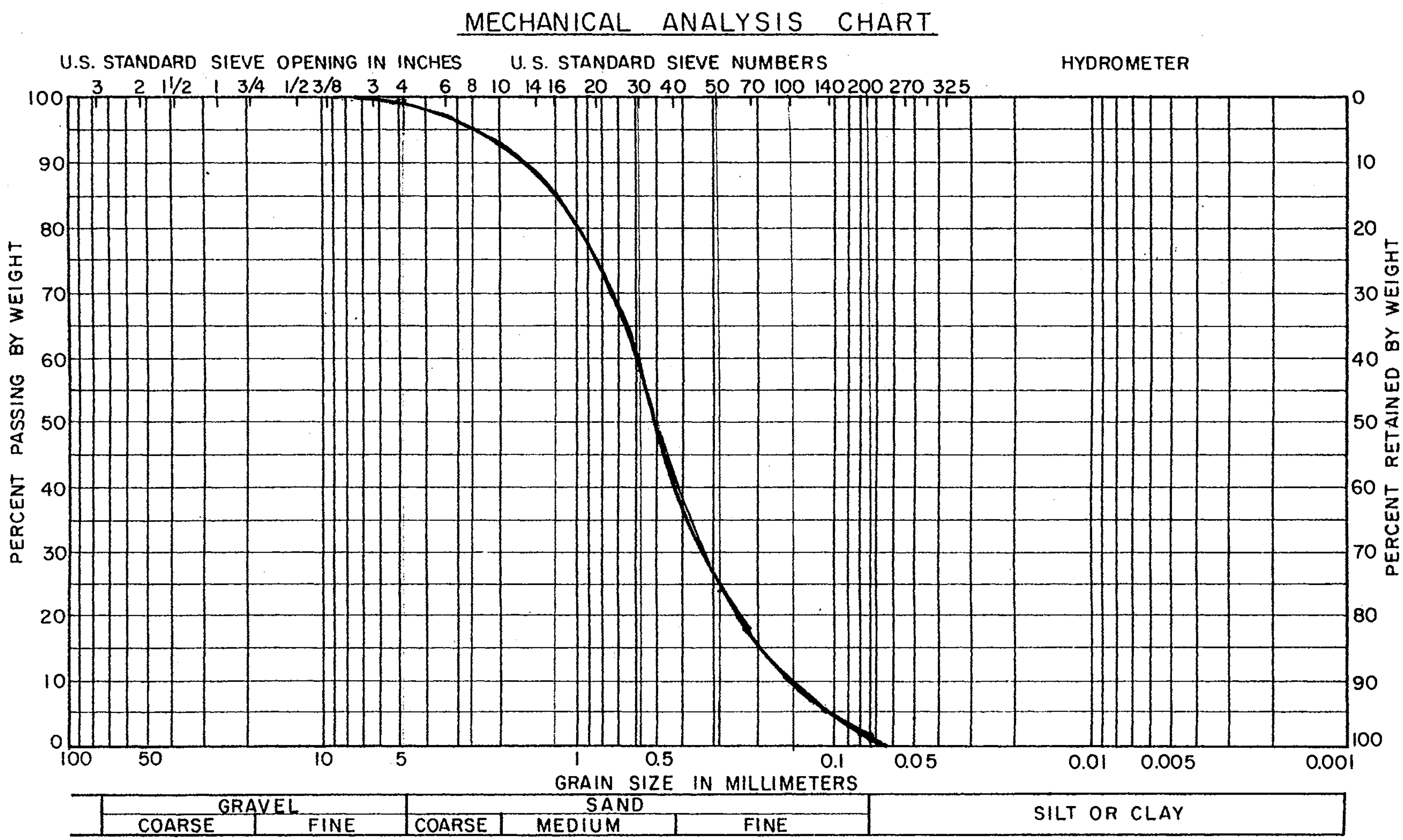
Coefficient of Uniformity (Cu) = 19.8 Dry Density (Compacted) = 130.6 lbs/cu ft

Figure 37. Information and Gradation Chart of OSU 3/8-in. Maximum Aggregate Size Material



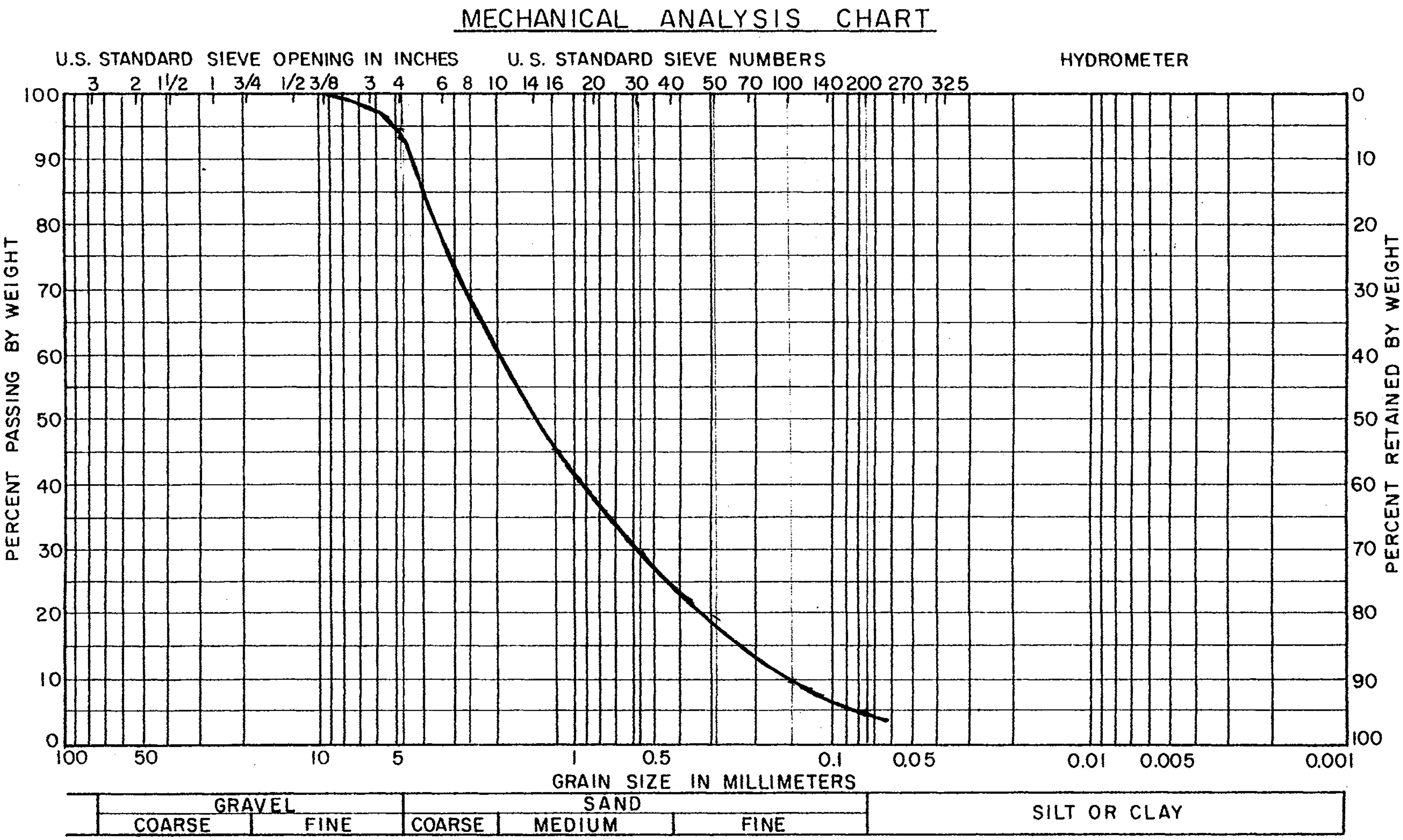
Coefficient of Uniformity (Cu) = 18.8 Dry Density (Compacted) = 123.3 lbs/cu ft

Figure 38. Information and Gradation Chart of OSU 3/4-in. Maximum Aggregate Size Material



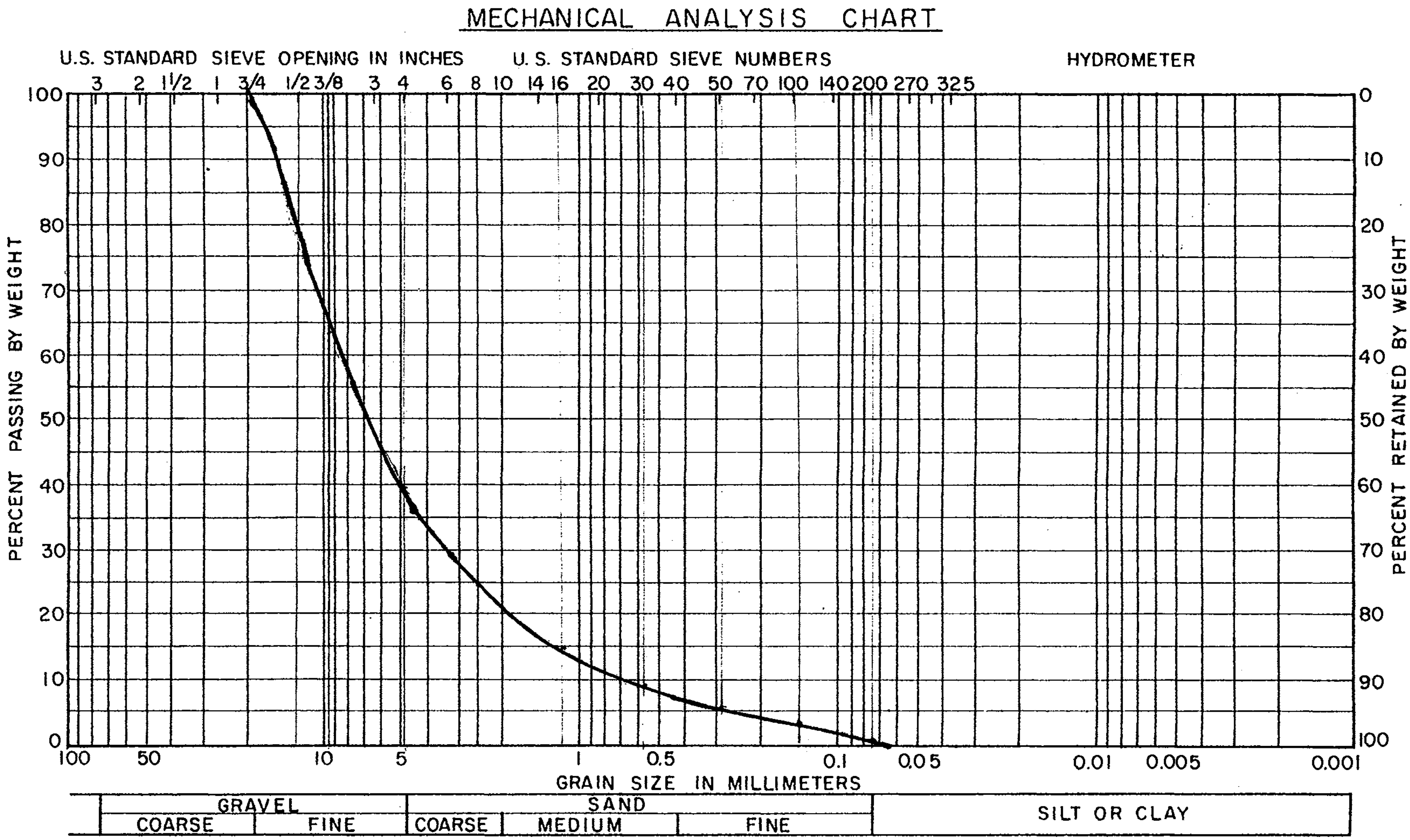
Coefficient of Uniformity (Cu) = 3.3 Dry Density (Compacted) = 120.2 lbs/cu ft

Figure 39. Information and Gradation Chart of ODOT Sand



Coefficient of Uniformity (Cu) = 12.6 Dry Density (Compacted) = 130.6 lbs/cu ft

Figure 40. Information and Gradation Chart of ODOT 3/8-in. Maximum Aggregate Size Material



Coefficient of Uniformity (Cu) = 12.4 Dry Density (Compacted) = 132.2 lbs/cu ft

Figure 41. Information and Gradation Chart of ODOT 3/4-in. Maximum Aggregate Size Material

TABLE 25

DCP TEST DATA: SAMPLES TESTED AT 6 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Material Description—OSU Sand			
0	14.1	0	
1	15.3	1.2	
2	16.6	1.3	
3	18.4	1.8	
4	20.1	1.7	
5	21.8	1.7	1.7
2. Material Description—ODOT Sand			
0	14.1	0	
1	18.4	4.3	
2	22.5	4.1	4.1
3. Material Description—OSU 3/8-in. Maximum Aggregate Size			
0	16.1	0	
1	16.6	0.5	
2	17.0	0.4	
3	17.5	0.5	
4	17.9	0.4	
5	18.4	0.5	
6	18.9	0.5	
7	19.3	0.4	
8	19.8	0.5	
9	20.2	0.4	
10	20.7	0.5	
11	21.2	0.5	
12	21.7	0.5	
13	22.2	0.5	0.5
4. Material Description—ODOT 3/8-in. Maximum Aggregate Size			
0	15.9	0	
1	16.8	0.9	
2	17.5	0.7	
3	18.2	0.7	
4	19.0	0.8	
5	19.8	0.8	
6	20.6	0.8	
7	21.3	0.7	
8	22.0	0.7	
9	22.7	0.7	
10	23.4	0.7	
11	24.0	0.6	
12	24.5	0.5	0.7

TABLE 25 (CONTINUED)

Blow	Reading	Penetration Rate	DCP Value
5. Material Description—OSU 3/4-in. Maximum Aggregate Size			
0	16.1	0	
1	16.8	0.7	
2	17.5	0.7	
3	18.2	0.7	
4	18.9	0.7	
5	19.5	0.6	
6	20.1	0.6	
7	20.7	0.6	
8	21.3	0.6	
9	21.9	0.6	
10	22.5	0.6	
11	23.0	0.5	
12	23.5	0.5	
13	23.9	0.4	0.6
6. Material Description—ODOT 3/4-in. Maximum Aggregate Size			
0	16.4	0	
1	17.7	1.3	
2	19.0	1.3	
3	20.2	1.2	
4	21.3	1.1	
5	22.3	1.0	
6	23.2	0.9	1.0

TABLE 26

DCP TEST DATA: SAMPLES TESTED AT 12 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Material Description—OSU Sand			
0	15.0	0	
1	15.8	0.8	
2	16.7	0.9	
3	17.6	0.9	
4	18.4	0.8	
5	19.1	0.7	
6	20.0	0.9	
7	20.9	0.9	
8	21.7	0.8	
9	22.4	0.7	
10	23.0	0.6	
11	23.7	0.7	
12	24.3	0.6	
13	24.9	0.6	
14	25.5	0.6	
15	26.1	0.6	0.9
2. Material Description—ODOT Sand			
0	15.4	0	
1	18.0	2.6	
2	20.2	2.2	
3	22.5	2.3	
4	24.6	2.1	2.3
3. Material Description—OSU 3/8-in. Maximum Aggregate Size			
0	16.1	0	
1	16.5	0.4	
2	16.8	0.3	
3	17.0	0.2	
4	17.2	0.2	
5	17.4	0.2	
6	17.6	0.2	
7	17.8	0.2	
8	18.0	0.2	
9	18.2	0.2	
10	18.4	0.2	
11	18.6	0.2	
12	18.8	0.2	
13	19.0	0.2	
14	19.2	0.2	
15	19.4	0.2	
16	19.6	0.2	
17	19.8	0.2	
18	20.0	0.2	

TABLE 26 (CONTINUED)

Blow	Reading	Penetration Rate	DCP Value
19	20.2	0.2	
20	20.4	0.2	
21	20.6	0.2	
22	20.8	0.2	0.2
4. Material Description—ODOT 3/8-in. Maximum Aggregate Size			
0	15.9	0	
1	16.6	0.7	
2	17.0	0.4	
3	17.4	0.4	
4	17.7	0.3	
5	18.0	0.3	
6	18.3	0.3	
7	18.6	0.3	
8	18.9	0.3	
9	19.2	0.3	
10	19.5	0.3	
11	19.8	0.3	
12	20.1	0.3	
13	20.4	0.3	
14	20.7	0.3	
15	21.0	0.3	
16	21.3	0.3	
17	21.6	0.3	
18	21.9	0.3	
19	22.2	0.3	
20	22.5	0.3	
21	22.8	0.3	
22	23.1	0.3	
23	23.4	0.3	
24	23.6	0.2	
25	23.8	0.2	
26	24.0	0.2	
27	24.2	0.2	0.3
5. Material Description—OSU 3/4-in. Maximum Aggregate Size			
0	16.1	0	
1	16.6	0.5	
2	17.1	0.5	
3	17.5	0.4	
4	18.0	0.5	
5	18.4	0.4	
6	18.9	0.5	
7	19.3	0.4	
8	19.8	0.5	
9	20.3	0.5	
10	20.7	0.4	

TABLE 26 (CONTINUED)

Blow	Reading	Penetration Rate	DCP Value
11	21.1	0.4	
12	21.5	0.4	
13	21.9	0.4	
14	22.3	0.4	
15	22.7	0.4	
16	23.1	0.4	
17	23.5	0.4	
18	23.8	0.3	
19	24.1	0.3	
20	24.4	0.3	
21	24.6	0.2	
22	24.8	0.2	
23	25.0	0.2	
24	25.1	0.1	
25	25.2	0.1	
26	25.3	0.1	
27	25.4	0.1	
28	25.5	0.1	0.4
6. Material Description—ODOT 3/4-in. Maximum Aggregate Size			
0	16.0	0	
1	17.3	1.3	
2	18.5	1.2	
3	19.6	1.1	
4	20.4	0.8	
5	21.0	0.6	
6	21.6	0.6	
7	22.2	0.6	
8	22.7	0.5	
9	23.1	0.4	
10	23.6	0.5	
11	24.0	0.4	
12	24.4	0.4	0.6

TABLE 27

DCP TEST DATA: SAMPLES TESTED AT 18 PSI

Blow	Reading	Penetration Rate	DCP Value
1. Material Description—OSU Sand			
0	14.1	0	
1	14.7	0.6	
2	15.3	0.6	
3	15.8	0.5	
4	16.3	0.5	
5	16.8	0.5	
6	17.3	0.5	
7	17.7	0.4	
8	18.1	0.4	
9	18.5	0.4	
10	18.9	0.4	
11	19.2	0.3	
12	19.5	0.3	
13	19.8	0.3	
14	20.1	0.3	
15	20.4	0.3	
16	20.7	0.3	
17	21.0	0.3	
18	21.3	0.3	0.3
2. Material Description—ODOT Sand			
0	15.4	0	
1	16.3	0.9	
2	17.1	0.8	
3	18.0	0.9	
4	18.9	0.9	
5	19.8	0.9	
6	20.6	0.8	
7	21.4	0.8	
8	22.2	0.8	
9	23.0	0.8	
10	23.7	0.7	
11	24.4	0.7	0.8
3. Material Description—OSU 3/8-in. Maximum Aggregate Size			
0	15.6	0	
1	15.8	0.2	
2	16.0	0.2	
3	16.1	0.1	
4	16.3	0.2	
5	16.4	0.1	
6	16.6	0.2	
7	16.7	0.1	
8	16.9	0.2	

TABLE 27 (CONTINUED)

Blow	Reading	Penetration Rate	DCP Value
9	17.0	0.1	
10	17.2	0.2	
11	17.3	0.1	
12	17.4	0.1	
13	17.5	0.1	
14	17.6	0.1	
15	17.7	0.1	0.1
4. Material Description—ODOT 3/8-in. Maximum Aggregate Size			
0	15.9	0	
1	16.4	0.5	
2	16.9	0.5	
3	17.4	0.5	
4	17.8	0.4	
5	18.2	0.4	
6	18.6	0.4	
7	19.0	0.4	
8	19.4	0.4	
9	19.8	0.4	
10	20.1	0.3	
11	20.4	0.3	
12	20.7	0.3	
13	20.9	0.2	
14	21.1	0.2	
15	21.3	0.2	
16	21.5	0.2	
17	21.7	0.2	
18	21.9	0.2	
19	22.1	0.2	0.2
5. Material Description—OSU 3/4-in. Maximum Aggregate Size			
0	14.7	0	
1	15.3	0.6	
2	15.7	0.4	
3	16.1	0.4	
4	16.4	0.3	
5	16.7	0.3	
6	17.0	0.3	
7	17.3	0.3	
8	17.6	0.3	
9	17.9	0.3	
10	18.2	0.3	
11	18.4	0.2	
12	18.6	0.2	
13	18.8	0.2	
14	18.9	0.1	
15	19.0	0.1	

TABLE 27 (CONTINUED)

Blow	Reading	Penetration Rate	DCP Value
16	19.1	0.1	
17	19.2	0.1	
18	19.3	0.1	
19	19.4	0.1	
20	19.5	0.1	
21	19.6	0.1	
22	19.7	0.1	
23	19.8	0.1	
24	19.9	0.1	
25	20.0	0.1	
26	20.1	0.1	
27	20.2	0.1	
28	20.3	0.1	0.1
6. Material Description—ODOT 3/4-in. Maximum Aggregate Size			
0	16.6	0	
1	17.1	0.5	
2	17.5	0.4	
3	18.0	0.5	
4	18.4	0.4	
5	18.8	0.4	
6	19.3	0.5	
7	19.6	0.3	
8	20.0	0.4	
9	20.4	0.4	
10	20.8	0.4	
11	21.2	0.4	
12	21.7	0.5	
12	22.1	0.4	
13	22.5	0.4	
14	22.9	0.4	
15	23.3	0.4	
16	23.6	0.3	
17	24.0	0.4	
18	24.4	0.4	
19	24.8	0.4	
20	25.1	0.3	
21	25.5	0.4	
22	26.0	0.5	
23	26.4	0.4	
24	26.8	0.4	
25	27.1	0.3	0.4

General Linear Models Procedure

Class	Levels	Values
Loc	2	ODOT OSU
Size	3	0.25 0.38 0.75
Con	3	6 12 18

Number of Observations in Data Set = 18

Dependent Variable: DCP

Source	Df	Sum of Squares	Mean Square	F Value	Pr > F
Model	11	15.7477777800	1.43161616	9.37	0.0062
Error	6	0.9166666700	0.15277778		
Corrected Total	17	16.66444444			

R-Square	C.V.	Root Mse	DCP Mean
0.944993	46.28700	0.3908680	0.8444444

Source	Df	Type I SS	Mean Square	F Value	Pr > F
Loc	1	1.74222222	1.74222222	11.40	0.0149
Size	2	6.43444444	3.21722222	21.06	0.0019
Loc*Size	2	1.50111111	0.75055556	4.91	0.0545
Con	2	3.77444444	1.88722222	12.35	0.0075
Size*Con	4	2.29555556	0.57388889	3.76	0.0731

Source	Df	Type III SS	Mean Square	F Value	Pr > F
Loc	1	1.74222222	1.74222222	11.40	0.0149
Size	2	6.43444444	3.21722222	21.06	0.0019
Loc*Size	2	1.50111111	0.75055556	4.91	0.0545
Con	2	3.77444444	1.88722222	12.35	0.0075
Size*Con	4	2.29555556	0.57388889	3.76	0.0731

Contrast	Df	Contrast SS	Mean Square	F Value	Pr > F
Linear	1	3.74083333	3.74083333	24.49	0.0026
Lof	1	0.03361111	0.03361111	0.22	0.6556

Tests of hypotheses using Type III MS for Loc*Size as an error term

Source	Df	Type III SS	Mean Square	F Value	Pr > F
Size	2	6.43444444	3.21722222	4.29	0.1892

Figure 42. SAS Output to Evaluate Effects of Maximum Aggregate Size and Confining Pressure and Contrast Analysis of Confining Pressure With DCP Values

2

VITA

Azmi Bin Hassan

Candidate for the Degree of

Doctor of Philosophy

Thesis: THE EFFECTS OF MATERIAL PARAMETERS ON DYNAMIC CONE PENETROMETER RESULTS FOR FINE-GRAINED SOILS AND GRANULAR MATERIALS

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Kota Bharu, Kelantan, Malaysia, January 26, 1954, the son of Hassan Bin Ahmad and Meriam Hj. Awang.

Education: Studied at Sultan Yahya Petra, Kuala Krai, for primary school, and Malay College Kuala Kangsar for secondary school education; received the Bachelor of Engineering (Honors) degree from the University of Wales Institute of Science and Technology in 1980; received the Master of Science degree from Oklahoma State University in 1987; completed requirements for the Doctor of Philosophy degree at Oklahoma State University in December, 1996.

Professional Experience: Civil Engineer with the Public Works Department, Malaysia, July 1980-July 1982; Water Resource Engineer, August 1982-March 1984; District Engineer, Tanah, Merah, April 1984-July 1985; Highway Maintenance Engineer, August 1985-August 1986; District Engineer, Pasir Mas, January 1988-December 1993; Highway Engineer (design and maintenance).