POTENTIAL APPLICATIONS OF DYNAMIC AND STATIC CONE PENETROMETERS IN MDOT PAVEMENT DESIGN AND CONSTRUCTION

FINAL REPORT

by

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16. Abstract

This report discusses the current applications of the dynamic and static cone penetrometers in pavement design and construction. The dynamic cone penetrometer (DCP) is the most versatile rapid, in situ evaluation device currently available. Correlations to CBR, unconfined compressive strength, resilient modulus, and shear strengths, and its use in performance evaluation of pavement layers make it an attractive alternative to more expensive and time consuming procedures. Many useful correlations between the DCP penetration index and other material properties continue to be reported. Other possible applications of DCP such as its use in the quality control of compaction of fill are discussed. In addition, advantages and disadvantages of the penetrometer testing are reported. The static cone penetrometer has also several applications in such areas as the evaluation of resilient modulus of cohesive soils, estimation of CBR, and the determination of relative density of sands. An overview of current practices ae well as areas of possible future trends are reported.

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ABSTRACT

This report discusses the current applications of the dynamic and static cone penetrometers in pavement design and construction. The dynamic cone penetrometer (DCP) is the most versatile rapid, in situ evaluation device currently available. Correlations to CBR, unconfined compressive strength, resilient modulus, and shear strengths, and its use in performance evaluation of pavement layers make it an attractive alternative to more expensive and time consuming procedures. Many useful correlations between the DCP penetration index and other material properties continue to be reported. Other possible applications of DCP such as its use in the quality control of compaction of fill are discussed. In addition, advantages and disadvantages of the penetrometer testing are reported. The static cone penetrometer has also several applications in such areas as the evaluation of resilient modulus of cohesive soils, estimation of CBR, and the determination of relative density of sands. An overview of current practices as well as areas of possible future trends is reported.

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CHAPTER I

INTRODUCTION

In order to perform an effective and reliable pavement design, an accurate and representative material characterization technique is essential. Such a technique would be even more beneficial if it were simple, and could be performed rapidly. During construction, a quality control program and inspection is conducted to ensure that proper specifications are followed. After construction, performance evaluation of pavement structures is needed to characterize the nature of rehabilitation strategy.

The DCP, also known as the Scala penetrometer, was developed in 1956 in South Africa as an in situ pavement evaluation technique for evaluating pavement layer strength (Scala, 1956.) Since then, this device has been extensively used in South Africa, the United Kingdom, the United States, Australia, and many other countries, because of its portability, simplicity, cost effectiveness, and the ability to provide rapid measurement of in situ strength of pavement layers and subgrades. The DCP has also been proven to be useful during pavement design and quality control program.

The DCP has been intended to alleviate many of the deficiencies of systems that are manually pushed into soil or paving materials. The device is relatively simple in design and operation, and operator variability is reduced and thus correlations with strength parameters are more accurate. The DCP consists of a steel rod with a steel cone attached to one end driven into the pavement structure or subgrade using a sliding hammer. Material strength is measured by the penetration (usually in millimeters or inches) per hammer blow. The cone has an angle of 30 degrees with a diameter of 20 mm (0.79 in). The hammer is 8 kg (17.7 lb) with a drop height of 575 mm (22.6 in).

Researchers at the Engineering, Research, and Development Center, Waterways Experiment Station have proposed several modifications of the DCP so that it can be adapted for automation (Webster et al., 1992). In this case, the angle of the cone is 60 degrees and a there is an option to use a hammer mass of 4.6 Kg (10.2 lb) for weaker soils. The diameter of the cone is slightly larger than of the rod to ensure the resistance to penetration is exerted on the cone.

Operating the DCP can be physically arduous and the collection and analysis of the data time consuming. In an effort to automate the operation, data collection and analysis, an automated dynamic cone penetrometer (ADCP) has been suggested (e.g., Parker et al., 1988; Hammons, et, al., 1998). The ADCP, designed and constructed for quick set-up and simple operation, generally consists of a pneumatic system for raising the DCP weight, a vertical frame with wheels for carrying the DCP hammer lifting and release mechanism and the penetration rod. The ADCP lifts and drops the weight, records the number of blows and penetration, and extracts the rod after the completion of the test. In this approach, penetration is typically measured by a rotary encoder on the chain wheel and sent to a PC.

Some applications of the DCP include correlations to CBR, unconfined compressive strength, resilient modulus, and shear strengths, and its use in performance evaluation of pavement layers and quality control of compaction of fill.

In addition to the DCP, the static cone penetration test (CPT) has been widely used to classify soil and measure its strength. It is a simple static device that enables soil parameters to be rapidly obtained. The CPT may be used in a variety of applications including the evaluation of resilient modulus, soil classification and the determination of soil properties and relative density, estimation of CBR, and applications in weakly cemented sands, dynamic compaction, embankment settlement, and embankment stability.

Many useful applications for both the DCP and CPT continue to be reported. This study reviews the various applications of the dynamic and static cone penetrometers, and attempts to recommend possible areas of needed research on the applicability of the penetrometers in MDOT design and construction schemes.

CHAPTER 2

DCP APPLICATIONS METHODOLOGIES

2.1 Introduction

The basic design of the DCP has been relatively unchanged since its inception in the 1950's. The mass of the falling weight has been altered several times. The cone tip has also undergone numerous revisions to its basic design. More recently, the automated dynamic cone penetrometer has been suggested to automate the operation, data collection and analysis procedures (e.g., Hammons, et al., 1998).

The development of DCP was in response to the need for a simple and rapid device for the characterization of subgrade soils (e.g., Melzer and Smoltczyk, 1982; McGrath, 1989; McGrath, et al., 1989; and Mitchell, 1988). The DCP, however, was not a widely accepted technique in the United States in the early 1980's (Ayers, 1990). In the last few years, some DOT's as well as other organizations have shown considerable interests in the use of the DCP for several reasons (e.g., De Beer and van der Merwe, 1991; Meier and Baladi, 1988; Newcomb, et al., 1994; Newcomb, et al., 1995; Parker, et, el., 1998; Truebe and Evans, 1995; Tumay, 1994; Burnham and Johnson, 1993; and White, et al., 2002). First, the DCP is adaptable to many types of evaluations. Second, there are no currently available rapid evaluation techniques. Third, the DCP testing is economical. In addition, many correlations that permit the estimations of various parameters as well as experience in the use of the DCP exist (e.g., Allersma, 1988; Bester and Hallat, 1977; Bukoski and Selig, 1981; Chen et al., 1999; Chen et al., 2001; and Chan and Armitage, 1997). A new standard test method, ASTM D6951, for use of the DCP in shallow pavement applications has been recently developed (ASTM, 2003). This chapter summarizes the current applications and practices of the DCP.

2.2 Relationships between DCP Penetration Resistance and California Bearing Ratio (CBR)

Extensive research has been performed to develop empirical relationships between DCP penetration resistance and CBR measurements (e.g., Kleyn, 1975; Harison, 1987; Livneh, 1987; Livneh and Ishai, 1988; Chua, 1988; Harison, 1983; Van Vuuren, 1969; Livneh, et. al., 1992; Livneh and Livneh, 1994; Ese et. al., 1994; and Coonse, 1999). Based on the results of past studies, many of the relationships between DCP and CBR have the following form:

$$\log(CBR) = a + b \log(DCPI) \qquad (1)$$

Where DCPI = DCP penetration resistance (mm/blow); a = constant that ranges from 2.44 to 2.60; and b = constant that ranges from –1.07 to –1.16.

A summary of some of these correlations is presented in Table 1.

Table 1. DCP-CBR Correlations

Correlation Equation	Material tested	Reference
log(CBR) = 2.56 – 1.16 log (DCPI)	Granular and cohesive	Livneh (1987)
log(CBR) = 2.55 – 1.14 log (DCPI)	Granular and cohesive	Harison (1987)
log(CBR) = 2.45 – 1.12 log (DCPI)	Granular and cohesive	Livneh et al. (1992)
log(CBR) = 2.46 – 1.12 log (DCPI)	Various soil types	Webster et al. (1992)
log(CBR) = 2.62 – 1.27 log (DCPI)	Unknown	Kleyn (1975)
log(CBR) = 2.44 – 1.07 log (DCPI)	Aggregate base course	Ese et al. (1995)
Log(CBR) = 2.60 – 1.07 log (DCPI)	Aggregate base course and cohesive	NCDOT (Pavement, 1998)
Log(CBR) = 2.53 – 1.14 log (DCPI)	Piedmont residual soil	Coonse (1999)

2.3 Relationships between DCP Penetration Resistance and Resilient Modulus

Several researchers have developed correlations between resilient modulus (M_R) and DCPI. Hassan (1996) indicated that the correlation of M_R with the DCPI is significant at optimum moisture content but insignificant at optimum moisture content $\pm$ 20%. He developed a simple regression model in the following form:

$$M_R \text{ (psi)} = 7013.065 - 2040.783 \ln(\text{DCPI}) \qquad (2)$$

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Where DCPI is in inches/blow.

Chai and Roslie (1998) used the results of CBR-DCP relationships and the DCP tests to determine in situ subgrade modulus in the following form:

$$E(\text{MN/m}^2) = 17.6\ (269/\text{DCP})^{0.64}$$

(3)

Where DCP = blows/300mm penetration.

They also developed a relationship between the backcalculated modulus and the DCP value in the following form:

$$E(_{\text{back}}) = 2224\ \text{DCP}^{-0.996} \qquad (4)$$

Where $E_{(\text{back})}$ = Backcalculated subgrade modulus (MN/m^2)

Jianzhou et al. (1999) found that there was a strong relationship between DCPI and the FWD-backcalculated moduli in the following form:

$$E(_{\text{back}}) = 338\ \text{DCPI}^{-0.39} \qquad (5)$$

George and Uddin (2000) developed relationships between M_R and DCPI as a function of moisture content, liquid limit, and density. Due to the MDOT requirements for being able to correlate M_R in real time, they also provided simpler one-to-one relationships between DCPI and $M_{R.}$ For fine-grained soils the following relationship was developed:

$$M_R = 532.1\ DCPI^{-0.492} \tag{6}$$

The relationship for coarse-grained soils is of the following form:

$$M_R = 235.3\ DCPI^{-0.475} \tag{7}$$

2.4 Application of DCP in Unconfined Compressive Strength Evaluation of Lime-Stabilized Subgrade

McElvaney and Djatnika (1991), based on laboratory studies, have concluded that DCPI values can be correlated to the unconfined compressive strength (UCS) of soil-lime mixtures. They considered both individual and combined soil types in their analysis. They have also concluded that the inclusion of data on mixtures from material with zero lime content has negligible effects on the correlation equations, indicating that the correlation is mainly a function of strength and not the way in which strength is achieved. This observation was valid only for lower range of strain values. For the combined data, three relationships, with each model permitting estimated unconfined compressive strength to a predetermined reliability level, were developed. Their first relationship was a 'best-fit' or 50 percent line, which implies that there is a 50 percent probability that the value of UCS determined from the measured DCPI value using the regression equation will underestimate the 'real' value. They also developed relationships such that with different degrees of confidence (96 and 99 percent), the probability of underestimation is reduced to 15 percent. These relationships are summarized below:

log UCS = 3.56 – 0.807 log (DCPI); 50% probability of underestimation (8)

log UCS = 3.29 – 0.809 log (DCPI); 95% confident that probability of underestimation will not exceed 15 percent (9)

log UCS = 3.21 – 0.809 log (DCPI); 99% confident that probability of underestimation will not exceed 15 percent (10) where UCS = unconfined compressive strength (KPa).

In addition, the DCP, through its correlations with CBR, has been used to characterize stabilized bases and subgrades in isolated projects, but no consistent methodologies have been proposed (Little, et al., 1995). The DCP has also been used to verify Falling Weight Deflectometer (FWD) measurements and, consequently, moduli backcalculation derived from FWD deflection data for stabilized bases and subbases.

2.5 Relationships between DCP Penetration Resistance and Shear Strength of Cohesionless Materials

Based on the results of laboratory studies, Ayers, et al. (1989) provided predictive equations for various confining pressures in the following form:

$$DS = A - B(DCPI) \tag{11}$$

Where DS = shear strength, and A and B are regression coefficients.

2.6 Application of DCP in Quality Control of Compaction

2.6.1 Application of DCP Testing For Cohesive and Select Backfill Materials

Historically, the compaction levels of pavement subgrade and base layers have been determined by means of in-place density testing. In an effort to determine whether there is a reasonable correlation between the DCPI and in-place compaction density of cohesive and select backfill materials, some testing has been recently performed on these materials to determine if such a correlation exits. Most results of DCP testing have indicated too much variability in DCP results to practically apply a correlation (Burnham, 1997).

Siekmeier et al. (1999), as part of the Minnesota Department of Transportation study, investigated the correlation between DCP results and compaction of soils consisting of mixture of clayey and silty sand fill. They first correlated DCPI to the CBR. CBR was then related to the modulus using published relationships. They examined the relations between the modulus and percent compaction. It was concluded that a good correlation did not exist between the DCP results and percent compaction, partly because a typical range of soil mixtures at the site was not truly uniform.

2.6.2 Application in Quality Control of Granular Base Layer Compaction

The Minnesota Department of Transportation suggests this application to reduce testing time and effort while providing more consistent quality control of base layer compaction (Burnham, 1977). Using this procedure, immediately after the compaction of each layer of granular base material, DCP tests are conducted to insure that the DCPI is less than 19 mm per blow (0.75 inches per blow). The DCPI limiting value is valid for all freshly compacted base materials. The DCPI dramatically decreases as the materials "set-up time" increases and under traffic loading. Using this method, the DCP testing will only indicate those adequately compacted base layers that "pass". Test failure, however, must be confirmed by other methods such as the nuclear gauge or the sand cone density method.

Based on general agreement between the DCPI and percent compaction, the Minnesota Department of Transportation has revised the limiting penetration rate to the following (Siekmeier et al., 1998):

a) 15 mm/blow in the upper 75 mm (3.0 in);
b) 10 mm/blow at depths between 75 and 150 mm (3 and 6 in); and
c) 5 mm/blow at depths below 150 mm (6 in).

They concluded that the penetration rate is a function of moisture content, set-up time, and construction traffic, and that accurate and repeatable tests depend on seating the cone tip properly and beginning the test consistently. They recommended the following: a) the test be performed consistently and not more than one day after compaction while the base material is still damp; b) the construction traffic be distributed uniformly by requiring haul trucks to vary their path; and c) at least two dynamic cone penetrometer tests be conducted at selected sites within each 800 cubic meters of constructed base course. They proposed a Penetration Index Method (Trial Mn/DOT Specifications 2211.3C4) which described a step-by-step procedure for determining the "pass' and "fail" tests (Siekmeier, et al. 1998). These specifications are summarized in the Appendix.

Siekmeier et al. (1999), as part of the Minnesota Department of Transportation study, studied the correlation between DCP results and compaction of soils consisting of sand and gravel mixture with less than 10-percent fine. They first correlated DCPI to the CBR. CBR was then correlated to the modulus using published relationships. They examined the relations between the modulus and percent compaction. It was concluded a good correlation existed between the DCP results and percent compaction.

2.6.3 Application for Granular Materials Around Utilities

Many transportation agencies use granular soils as backfill and embedment materials in the installation of underground utility structures, including the thermoplastic pipe used in gravity flow applications. The granular backfill relies on proper compaction to achieve adequate strength and stiffness and to ensure satisfactory pipe performance. The commonly used standard proctor test cannot be used because it does not provide a well-defined moisture-density relationship. In addition, this approach requires density measurements on each lift of the compacted fill for the entire length of the pipe. Recent studies indicate that DCP blow count profiles provide a basis for comparison between compaction equipment, level of compaction energy, and materials. But, it should be noted that these data alone do not reveal what level of compaction must be achieved with each type of backfill material in order to achieve the specific performance criteria. The results have also indicated that the DCPI values are very sensitive to the depth of measurements (Jayawickrama, et al., 2000).

2.6.4 Application During Backfill Compaction of Pavement Drain Trenches

The Minnesota Department of Transportation has indicated that the DCP testing is reliable and effective in improving the compaction of these trenches. Using this procedure, immediately after installation of the pavement edge drainpipe and fine filter granular backfill material, DCP testing is conducted to insure that the DCPI is less than 75 mm per blow (3 inches per blow). In this approach, each 150 mm (6 inches) of compacted backfill material is tested for compliance (Burnham, 1997).

2.7 Application of DCP in Performance Evaluation of Pavement Layers

Performance evaluation of pavement layers is needed on a regular basis in order to categorize the implementation of rehabilitation measures (e.g., Kleyn, et al., 1982). The Minnesota Department of Transportation, based on the analysis of Mn/Road DCP testing, has recommended the following limiting values for DCPI during a rehabilitation study (Burnham, 1997):

a) Silty/Clayey material: DCPI less than 25 mm/blow (1.0 in/blow);
b) Select granular material: DCPI less than 7 mm/blow (0.28 in/blow); and
c) Mn/Road Class 3 special gradation requirements: DCPI less than 5 mm/blow (0.2 in/blow)

The above values are based on the assumption that adequate confinement exists near the testing surface. In the event that higher values than the above mentioned limiting values are encountered, additional testing methods are needed. It should be noted that the above values are independent of the moisture content. Moisture content can cause large variability in DCP test results. Nevertheless, a limiting value was recognized.

Gabr et al. (2000) proposed a model by which the DCP data are utilized to evaluate the pavement distress state. They proposed a model to predict the distress level of pavement layers using penetration resistance of the subgrade and aggregate base course (ABC) layers based on coupled contribution of the subgrade and the ABC materials. They provided a step-by-step procedure, based on the correlation of the DCPI with CBR, by which the DCP data can be used to evaluate the pavement distress state for categorizing the need for rehabilitation measures. Although their pavement stress model was specific in this study regarding the type of the ABC material tested, the framework of the procedure can be used at other sites.

2.8 Application of DCP to Obtain Layer Thickness

DCP can also be used effectively to determine the soil layer thickness from the changing slope of the depth versus the profile of the accumulated blows. Livneh (1987) showed that the layer thickness obtained from DCP tests correspond reasonably well to the thickness obtained from the test pits. It was concluded that the DCP test is a reliable alternative for project evaluation.

2.9 Complementing FWD During Backcalculation

It has been shown that the DCP is very useful when the moduli backcalculated from falling weight deflectometer (FWD) data are in question, such as when the asphalt concrete is less than 76 mm (3 inches) or when shallow bedrock is present (e.g., Little et al., 1995). These two situations often cause a misinterpretation of FWD data. The DCP can be readily applied in these two situations to increase the accuracy of the stiffness measurement. In addition, it is not possible to conduct a FWD test directly on weak subgrade or base layers because of the large deflections that can exceed the equipment's calibration limit.

2. 10 Factors Affecting DCP Results

2.10.1 Material Effects

Several investigators have studied the influence of several factors on the DCPI. Kleyn and Savage (1982) indicated that moisture content, gradation, density, and plasticity were important material properties influencing the DCPI. Hassan (1996) performed a study on the effects of several variables on the DCPI. He concluded that for fine-grained soils, moisture contents, soil classification, dry density and confining pressures influence the DCPI. For coarse-grained soils, coefficient of uniformity and confining pressures were important variables.

2.10.2 Vertical Confinement Effect

Livneh, et al. (1995) performed a comprehensive study of the vertical confinement effect on dynamic cone penetrometer strength values in pavement and subgrade evaluations. The results have shown that there is no vertical confinement effect by rigid pavement structure or by upper cohesive layers on the DCP values of lower cohesive subgrade layers. In addition, their findings have indicated that no vertical confinement effect exists by the upper granular layer on the DCP values of the cohesive subgrade beneath them. There is, however, vertical confinement effect by the upper asphaltic layers in the DCP values of the granular pavement layers. These confinement effects usually result a decrease in the DCP values. Any difference between the confined and unconfined values in the rigid structure or in the case of granular materials is due to the friction developed in the DCP rod by tilted penetration or by a collapse of the granular material on the rod surface during penetration.

2.10.3 Side Friction Effect

Because the DCP device is not completely vertical while penetrating through the soil, the penetration resistance would be apparently higher due to side friction. This apparent higher resistance may also be caused when penetrating in a collapsible granular material. This effect is usually small in cohesive soils. Livneh (2000) suggested the use of a correction factor to correct the DCP/CBR values for the side friction effect.

CHAPTER 3

APPLICATIONS OF STATIC CONE PENETROMETER

3.1 Introduction

The cone penetration test (CPT) or static penetration test, originally called the Dutch cone penetration test, is a versatile sounding method than can be used to estimate soil properties and determine the type of materials. This is also known as the static penetration test. The cone penetration measures the cone resistance (q_c) and the frictional resistance (f_c). Generally, there are two types of penetrometers. For mechanical penetrometers, the tip is connected to an inner set of rods. The tip is first advanced about 40 mm (1.6 in) giving the cone resistance. With further testing, the tip engages the friction sleeve. For electric penetrometers, the tip is attached to a string of steel rods. The tip is pushed into the ground at the rate of 20 mm/sec (0.79 in/sec). Wires from transducers are threaded through the center of the rods and continuously give the cone and side resistance (Das, 1999). The mechanical type is less expensive, but the electric penetrometer has been adapted for automatic data acquisition, and almost immediate data reduction.

The CPT is a rapid, repeatable, reliable, and inexpensive procedure. However, it may not be used through coarse and very strong soils. In addition, no sample is recovered for visual inspection.

The CPT may be used in a variety of applications including the evaluation of resilient modulus, soil classification and the determination of soil properties and relative density, estimation of CBR, and applications in weakly cemented sands, dynamic compaction, embankment settlement, and embankment stability. Some of these applications are summarized below.

3.2 Evaluation of Resilient Modulus of Cohesive Subgrade Soil

Mohammad et al. (2000) has reported a preliminary study for the evaluation of resilient modulus of subgrade soils using the static cone penetration test. Two different types of Louisiana subgrade cohesive soils, a silty clay from a manmade embankment and a natural deposit of heavy clay, were selected for field and laboratory investigations. A preliminary relationship in the following form was developed:

$$M_R = a\, q_c^{\,n} + bf_c + cw + dp_d + e \qquad (12)$$

Where M_R = resilient modulus; q_c = cone resistance; f_c = frictional resistance; w = moisture content; p_d = dry density; n = an integer (1, 2, or 3); and a, b, c, d, and e are constants determined from the regression analysis.

It should be noted that a wider range of soil types and more tests are required before a more general model can be developed.

The cone penetration resistance has also been correlated to the equivalent modulus of elasticity, E, of soils by various investigators. Schmertmann (1970) gave a simple correlation for sand as

$$E = 2q_c \qquad (13)$$

Trofimemkov (1974) gave the following correlations for the stress-strain modulus in sand and clay:

$$E = 3q_c \quad \text{(for sands)} \qquad (14)$$

$$E = 7q_c \quad \text{(for clays)} \qquad (15)$$

3.3 Application in Weakly Cemented Sands

Naturally cemented deposits are very common throughout many parts of the world. These deposits are often characterized by their ability to withstand steep natural slopes. Some studies have indicated that the static cone penetrometer can locate sand layers with very low cementation. Correlations have also been developed between the cone resistance and the strength parameters for very weakly cemented sands (e.g., Puppala, et al., 1995; and Day, 1996). Their results have also demonstrated the significance of incorporating the effect of any cementation in estimating the strength parameters of sands.

3.4 Application in Soil Classification, and Estimation of Soil Properties and Relative Density

3.4.1 Soil Classification

One of the main applications of the static cone penetrometer is for stratigraphic profiling. There is considerable experience related to the identification and classification of the soil using the cone penetration test (e.g., Douglas and Olsen, 1981; Olsen and Farr, 1986; and Roberston, 1990).

Robertson (1990) reported a soil classification system based on CPT using normalized cone penetration test results with pore pressure measurements. Using normalized parameters and the available extensive CPT database, new charts were developed to represent a three-dimensional classification system. Factors such as changes in stress history, in situ stresses, sensitivity, stiffness, macrofabric, and void ratio were included in the development of the charts.

3.4.2 Soil Parameters

The cone resistance has been correlated to soil friction angle of granular soils and also to the consistency of cohesive soils (e.g., DeMello, 1971). Robertson, et al. (1982) correlated the cone resistance to the mean grain size (D_{50}) of the soil, which covered a wide range of soil types.

3.4.3 Relative Density

Several investigators including Schmertmann (1978), Villet and Mitchell (1981), Baldi et al. (1982, 1986), Robertson and Campanella (1983), Jamiolkowski et al. (1985, 1988), Puppala et al. (1995), and Juang et al. (1996) have developed correlations for the relative density (D_r) as a function of q_c for sandy soils. These relationships are also functions of vertical effective stress. A more rational theory for the correlations, which can be used for general conditions, has been developed by Salgado et al. (1997). The reader may refer to these papers for further information.

CHAPTER 4

SUMMARY AND RECOMMENDATIONS

4.1 Summary

The main focus of this study was to review the various applications of the dynamic and static cone penetrometers, and to suggest possible areas of needed research on the applicability of the penetrometers in MDOT design and construction schemes. The DCP is an extremely versatile evaluation tool in pavement design and construction. Some of the main advantages of the DCP are:

1. DCP has a wide variety of applications including estimations of CBR, resilient modulus, unconfined compressive strength, and shear strengths, as well as its use in performance evaluation of the pavement layers. Other potential application of the DCP includes its use in the quality control of granular base layer compaction.
2. The DCP is rapid and economical.
3. The DCP evaluations may be conducted and the results analyzed by personnel with limited training.

Some of the primary disadvantages of the DCP include:

1. High variability exists particularly in the case of large, well-graded granular materials.
2. The use of DCP for materials with a maximum aggregate size of larger than 2 inches is questionable.
3. Some of the existing strength relationships are only applicable to certain material types and conditions, and not to all cases.

The static cone penetrometer is a rapid, repeatable, reliable, and inexpensive procedure, which may be used in a variety of applications including the evaluation of resilient modulus of cohesive soils, estimation of CBR, and the determination of relative density of sands. Its advantages and disadvantages have been described earlier in Chapter 3.

4.2 Recommendations for Future Research

There are several areas that require further research. Additional studies are required to expand the use of the DCP for general design and construction purposes, and to examine the applicability of the current methodologies to various situations. Some possible recommendations are listed below:

<u>1. Application of DCP in Quality Control Compaction of Lime-Fly Ash Stabilized Granular Materials, Embankments, Soil Cement, Aggregate Base Soils, and the Field Density of Subgrades</u> Prior limited studies have indicated the DCP may be used during quality control program of granular base compaction. It is recommended that this application be examined for lime-fly ash (LFA) stabilized granular materials. Particular attention needs to be directed towards developing a limiting DCP criteria for successful compaction or failure, and the influence of various factors including moisture content, set-up time, climate factor, and construction traffic on the DCP values. The research should focus on investigating whether there are consistent DCP values with limited variability. The ultimate goal is to develop a Penetration Index Method for LFA including a step-by-step procedure for determining the “pass’ and “fail” tests. Such procedures may then be implemented in MDOT quality control program.

In addition, research is needed to determine the percent compaction of embankment fill, aggregate base soils, and soil-cement, and field density of soil subgrades. The main objective is to determine if consistent and reliable DCP values are achievable. In these cases, it is essential to use uniform materials to minimize the variability of the DCP values. Particular attention needs to be directed towards obtaining the limiting DCPI values for these materials, as well as investigating the influence of various factors such as moisture content and material type on the DCPI.

2. Evaluation of Unconfined Compressive Strength of Lime or Fly Ash Stabilized Subgrades Limited laboratory results have shown that the DCP values may be correlated to the unconfined compressive strength of soil-lime mixtures for limited range of strength values. Applicability of the DCP for various circumstances including higher strength values, and different soil types and parameters remains to be seen. A comprehensive testing program may need to be conducted to develop correlations with the unconfined compressive strength for various soil types encountered in Mississippi. Particular attention needs to be directed towards developing simple relations, and identifying possible limitations of the correlations, and the influence of various soil types and parameters.

3. Application of DCP in Quality Control of Select Backfill Compaction Prior attempts have shown that this application has not been successful, partly because the backfill material has not been uniform. Research is needed to investigate the feasibility of this application and to develop possible limiting criteria (if any) for uniform select materials, such as uniform silty sands. In this approach, the focus is to possibly develop correlations between DCPI and percent compaction. Particular attention needs to be

directed towards determining whether there are consistent DCP values with limited variability. The effect of several factors including moisture content and set-up time on the DCP values and correlations will need to be evaluated.

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APPENDIX

Minnesota Department of Transportation DCP Test Procedure for Compaction Quality Control of Granular Base Materials

Appendix – Minnesota Department of Transportation DCP Test Procedure for Compaction Quality Control of Granular Base Materials (Siekmeier, et al., 1998)

1. Locate a level, undisturbed area.
2. Place the DCP device on the base aggregate test site. To seat the DCP cone tip properly, carefully raise the sliding weighted hammer until it meets the handle, then release the hammer under its own weight. If the seating process causes initial penetration exceeding 20 mm, relocate the test to a site at least 300 mm from the previous test location and reseat the cone. If the second test site fails the above criteria, compaction is not acceptable and the area being tested must be recompacted.
3. Record the penetration measurement after seating using the graduated rule on the DCP.
4. Carefully raise the hammer until it meets the handle, then release the hammer under its own weight. Repeat this process two more times for a total of three times when testing a lift of 75 mm or less. Repeat this process four more times for a total of five times when testing a lift of between 75 and 150 mm.
5. Record the final penetration measurement from the graduated rule on the DCP.
6. Subtract the measurement in step 3 from the measurement in step 5 and then divide the difference of the measurements by the number of blows required for testing. If the resulting value is 15 mm/blow or less, the test passes.