1-1-2005

Evaluation of the Dynamic Cone Penetrometer (DCP) and Geo-technical Remote Acquisition of Data System (G-RAD) for earthwork quality control testing for cohesive soils

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Evaluation of the Dynamic Cone Penetrometer (DCP) and Geo-technical Remote Acquisition of Data System (G-RAD) for earthwork quality control testing for cohesive soils

by

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A thesis submitted to the graduate faculty

in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Major: Civil Engineering (Geotechnical Engineering)

Program of Study Committee:
David J. White, Major Professor
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Iowa State University
Ames, Iowa
2005

Graduate College
Iowa State University

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ABSTRACT

Problem Statement: In 1998 Iowa State faculty, Dr. Kenneth Bergeson and Dr. David White conducted a study to evaluate the quality of Iowa's highway embankments. They concluded that the construction practices and embankment quality control were insufficient resulting in slope instability and uneven pavement surfaces. They later determined that existing tools and methods for construction quality control (specifically, the Dynamic Cone Penetrometer-DCP) needed to be adapted to more precisely and more efficiently evaluate engineering parameters of compacted embankment fills. DCP had not widely been used as a quality control tool in fine-grain soils that characterize most of Iowa's highways. They proposed two solutions: 1) adopting DCP to fine-grain materials in embankments; and 2) using Iowa State University (ISU)-developed Geotechnical Remote Acquisition of Data System (G-RAD) to collect and analyze data from the DCP.

Goal of the thesis project: Given the opportunity to improve embankment construction, it is important to test and document uses of new and existing tools and technologies. The goals of this thesis project are:

1) To demonstrate and document how DCP is used as a quality control tool in testing strength and uniformity of cohesive soils.
2) To demonstrate and document how G-RAD can be used to make DCP data collection and processing more effective.

This thesis clearly reviews the demonstration activities, presents the results of those activities, and documents a methodology for utilizing DCP in conjunction with G-RAD to measure and collect data to improve embankment construction.

Conclusions: Based on the data gathered, it has been established that the use of DCP in Iowa's cohesive soil embankments improves construction methods by providing data that ensures adequate soil strength is achieved during construction. Traditionally, in-situ measurement of soil strength has been time consuming and impractical. This project demonstrates that G-RAD in conjunction with DCP improves not only the quality of construction but the accuracy and efficiency of the quality control processes. Future use of the DCP and G-RAD system are recommended as a quality control tool for construction of cohesive soil embankments of Iowa.

CHAPTER 1 INTRODUCTION

1.1 Background

Highway embankment construction is the first step in constructing a quality highway. Often embankments are constructed from remolded materials, whose engineering properties are more difficult to predict than undisturbed materials. Once the embankment has been constructed, it forms the foundation upon which the highway is built.

In Iowa, the construction of highway embankments has traditionally relied on using the sheepsfoot roller walk out method specification where fill material is considered compact when the sheepsfoot penetrate less than a 1/4 of an inch. While the method is inexpensive and a fast way to show that the fill is compact, it is not for all soils a sufficient method to determine that adequate soil compaction has been achieved. In the case where the fill materials are wet of standard proctor optimum, the sheepsfoot roller will typically not "walk out". When the fill materials are dry, the roller walks out much faster because of the increased strength of the soil even at low compaction. Furthermore, there are no measurements from this method that can be used as input parameters used for the design of the highway pavement thickness.

These limitations lead engineers to re-evaluate this method for highway embankment construction quality control. As a result of the evaluation, engineering teams concluded that the embankment construction quality control was substandard. Construction problems were categorized in two technical areas: 1) slope stability; and 2) roughness and inconsistency of the pavement quality of highways shortly after construction. To further evaluate and address these issues, a team of collaborators was established to conduct *Highway Embankment*

Quality Studies. Results of the study have been published by Bergeson et al. (1998) and White et al. (1999, 2002) in conjunction with the Iowa Department of Transportation.

Studies by Bergeson et al. (1998) and White et al. (1999, 2002) revealed that the sheepsfoot walkout specification was an insufficient measure for quality control in most soil types. They concluded that more stringent and specific quality control tools were necessary to produce a quality embankment. Quality control measures that integrate data regarding moisture and density of the soils improve the quality of embankment construction but still fail to account for specific, precisely calculated engineering parameters that are important in ensuring embankment quality. For example, current methods for embankment construction fail to take into account strength or modulus parameters used in pavement design.

White et al. (1998) suggested the use of the Dynamic Cone Penetrometer (DCP) to measure strength and stability of embankments in addition to moisture content. The DCP is an ideal tool for measuring the in-situ strength because it is simple to use, inexpensive, and relies on standardized correlations to pavement design parameters, such as the California Bearing Ratio (CBR).

The instrument is easy to set up and operate; however, there are very few documented methods for quality control using the DCP in cohesive soils. Existing documentation and publications about DCP (including a recently published report from the Minnesota Department of transportation (2004)) focus on its use in evaluating and measuring engineering properties for quality control of granular materials only. The industry has yet to adopt DCP as a standardized tool for embankment construction as it relates to evaluating cohesive soils. Most Iowa soils are comprised of fine-grained cohesive materials, which makes adopting DCP for measuring fine grain materials a priority.

Industry's failure to utilize DCP in developing specifications for cohesive soils means that we fail to take advantage of two opportunities: 1) the opportunity to more adequately and accurately measure a variety of soil types and therefore construct more solid embankments; and 2) the opportunity to utilize a tool that could easily be adapted to electronically collect and process DCP data in the a simple, precise and efficient manner.

Iowa State University has developed a software tool for use on a pocket PC, the Geotechnical Remote Acquisition of Data (G-RAD). GRAD can be used to improve the efficiency of DCP data collection and analysis for quality control. In addition to collecting and processing DCP data, it can also be used for field moisture and density data entry along with other data such as lift thickness for ease of use and analysis. However, like DCP use cohesive soils, G-RAD has yet to be extensively tested in the field.

1.2 Goals and Objectives

Given the opportunity to improve embankment construction, it is important to test and document uses of new and existing tools and technologies. The goals of this project are:

1) To demonstrate and document how DCP is used as a quality control tool in testing strength and uniformity of cohesive materials.

2) To demonstrate and document how G-RAD can be used to make DCP data collection and processing more effective.

This thesis clearly reviews the demonstration activities, presents the results of those activities, and documents a methodology for utilizing DCP in conjunction with GRAD to measure and collect data to improve embankment construction.

1.3 Thesis Outline

The thesis is divided into six chapters:

Chapter I. Introduction

Chapter II. The history and correlations of the DCP

Chapter III. Instructions for using DCP for quality control including a discussion about different methods used as a basis for quality control testing for strength and uniformity

Chapter IV. Instructions for using G-RAD in conjunction with DCP testing

Chapter V. A description of field tests and presentation of DCP and GRAD field results, including the application of the quality control method using field data

Chapter VI. Conclusions and recommendations of the thesis

CAHPTER 2 LITERATURE REVIEW

2.1 Introduction

This chapter presents the rationale for the study and a review of the test device, the Dynamic Cone Penetrometer (DCP), its history and uses. It also provides an overview of work conducted using the device and correlations that have been made to various engineering properties of soil that demonstrates the DCP's application for use as a quality control device.

2.2 Description Of The Four-Phase Study Leading To This Thesis Project

This thesis describes phase IV of a four part study, *the Highway Embankment Quality Study*, performed for the Iowa Department of Transportation (Iowa DOT). Phases I, II, and III, described below, were performed by Bergeson and White who later published results of the study (Bergeson et al. (1998) and White et al. (1999, 2002)). Phase IV is ongoing and will be concluded in 2006. The primary focus of the Embankment Quality studies is the evaluation of highway embankment quality for the Iowa Department of Transportation.

Phase I: Embankment Quality Phase I research was initiated as a result of internal Iowa DOT studies that raised concerns about the quality of embankment construction. The results of the study identified problems with slope stability of large embankments and pavement performance (roughness) shortly after completion of construction. Phase I evaluated the quality of embankments being constructed utilizing the sheepsfoot walk out method specification. Overall, the evaluation demonstrated that quality in embankments construction is inconsistent.

Phase II: Phase II research incorporated field investigation and small pilot compaction studies to establish a method for improved field soil classification and to document industry-standardized construction practices. Observations from phase II demonstrated that

(1) sheepsfoot roller walk out, is not, for all soils, a reliable indicator of degree of compaction, adequate stability, or proper compaction moisture content;

(2) during fill placement, much of the fill material is typically very wet and compacted at high levels of saturation, which causes instability;

(3) compacted lift thickness was measured to vary from 177-560 mm (7-22 in) and roller passes averaged 4 to 5 passes;

(4) the Dynamic Cone Penetrometer (DCP) is a simple, inexpensive and adequate in-situ testing tool to evaluate in-place stability and uniformity. Recommendations were made to develop and pilot test new compaction and QC/QA guidelines.

Phase III: Phase III work consisted of developing and pilot-testing the Quality Management and Earthwork (QM-E) program on a full-scale project The pilot project tested primarily select soils and served as a tool to evaluate the feasibility of implementing a statewide Contractor QC and Iowa DOT QA program for earthwork grading. Results revealed that applying quality control measures that included classifying soils, determining moisture content, and testing for soil stability improved embankment quality for select soils. Corrective action was taken in cases where non-compliance was observed.

Phase IV: The primary objectives of the phase IV research are to

- demonstrate the QM-E program on two full scale projects in unsuitable soils,
- train and certify additional contractor and Iowa DOT field personnel for Grading Certification Level I,

- refine the QM-E program and generate an Iowa DOT developmental specification document for future statewide implementation and
- improve data collection, management, and report generation for QC/QA operations.

This thesis focuses on two aspects of the Phase IV Embankment Quality Research project. The two aspects are 1) the use of the dynamic cone penetrometer as a tool for quality control and 2) data collection and management for QC/QA.

2.3 Background

Applying quality control measures for earthwork construction is critical for insuring a consistent and quality product. Engineers consider several factors when implementing quality control measures. These factors commonly consider soil density or compaction as a key measure of quality control. Additional measures of embankment quality are strength, compressibility, and permeability.

The strength of an embankment, whether shear strength or compressive strength, directly correlates to the load bearing capacity of the embankment. The design of an embankment or a foundation is based on or limited by the load bearing capacity of the foundation soils.

Soil compressibility refers to how much the soil can be compressed when loads are placed on the soil. These could be cyclic loads, as in highway traffic, or dead loads, as in pavement placed on an embankment of the highway. In most instances, when the soil is compressed, the soil is said to have "settled;" However, engineers define "settlement" with less stringent criteria when referring to the settlement of an embankment or building pad because, as soils are loaded, they consolidate. In this instance, settlement must be equally distributed to prevent "differential settlement" which results in cracking building floors, sinking pavement and floors slanting in the direction of where the heaviest loads are placed. On highways, differential settlement causes ruts to appear on roads frequented by heavy traffic.

Soil permeability is defined according to how freely water flows through the subgrade. Additionally, if soil is susceptible to shrink and swell, soil permeability will play a significant role as increasing amounts of water permeates the soil and causes increased swelling of expansive soils. Furthermore, in areas with frost, heave susceptibility, the permeation of water into soil, will lead to frost heave in the winter months. This causes damage to the embankment and is ultimately seen on the pavements.

Malisch (1996) says adequate compaction avoids these problems related to the engineering properties mentioned above by increasing the load-bearing capacity, decreasing the water seepage and minimizing soil settlement. According to Hilf (1991), soil "compaction is the process by which a mass of soil, consisting of solid soil particles, air, and water, is reduced in volume by the momentary application of loads, such as rolling, tamping, or vibration. Compaction involves an expulsion of air without significantly changing the amount of water in the soil mass." The soil then retains the same amount of water in its uncompacted state as it does in the compacted state.

The most commonly used parameter for specifying correct compaction is density, (Selig, 1982). It is also the parameter used to determine the amount of compaction that has been achieved. Selig, (1982), suggests that this is primarily a consequence of historical tradition and convenience. Traditional studies suggest that increasing density also indicates an increase in other engineering measures, such as compaction, permeability, etc. The most commonly utilized field-density tests for structural fills are the sand-cone (ASTM D 1556), the rubber balloon method (ASTM D 2167), and the nuclear method (ASTM D 2922), (Schmidt, 1985).

Each method offers advantages and disadvantages. After the density has been measured, the measurements are compared to the predetermined maximum density and optimum moisture content. The maximum density and the optimum moisture content are determined using the ASTM D698-78 or the ASTM D 1557-78. The field test "passes" (or complies) if the measured density is at or above the specified relative compaction.

The field density test meets the requirements of compaction; however, it does not directly measure soil strength. While these tests demonstrate that some soils meet the density and moisture criteria, they do not ensure the soils meet adequate strength requirements, especially for strength. To ensure soil strength, the Dynamic Cone Penetrometer can be used.

2.4 The Dynamic Cone Penetrometer (DCP)

In the mid 1950s, A.J Scala developed the DCP to determine the California bearing ratio (CBR) of soil for the determination of pavement thickness. The CBR value is an indicator of the soil strength.

Scala's original model featured a 9.07 kg (20 lb) drop hammer falling a distance of 508 mm (20 in). The DCP's 15.875 mm (5/8 in) diameter rod calibrated in 5.08 cm (2 in) increments determined the penetration with a penetration distance of 762 mm (30 in) into the soil. The configuration used a 30° cone with 20 mm (0.79 in) diameter at its widest point.

D. J. Van Vuuren continued to develop the DCP through the late 1960s (Van Vuuren, 1969). His device was very similar to that developed by Scala, except that it featured a 10kg

(22 lb) hammer dropped 460 mm (18.1 in) with a 30° cone connected to a 16 mm (.63 in) diameter rod. This design penetrated to a depth of 1000 mm (39.4 inches).

In 1973 the South Africa's Transvaal Roads Department began to use the DCP as a rapid evaluation device for evaluation of existing roads. For their purposes, they changed the hammer weight to 8 kg (17.6 lb), the falling distance to 574 mm (22.6 in), and utilized two kinds of cones- the 30° and the 60° cones.

"The criterion for compaction control is usually in situ density, which in turn correlates with CBR. This accommodates for the difficulty inherent in obtaining representative CBR values..." (Van Vuuren 1969). Van Vuuren notes the several problems with this process:

- Conventional field CBR equipment costs hundreds of dollars and smaller municipalities can rarely afford such equipment. Furthermore, the limited amount of construction and design work fails to warrant such costs.
- Half a day or more is required to complete one in situ CBR test on various layers up to a depth of 1 m. This is necessary if one needs the complete picture of the strength variation with depth. If CBR at the surface is the sole measure determined, only a very shallow thickness is evaluated, which is likely insufficient for the purpose of quality control. Due to the time requirement and costs of thorough testing, CBR filed testing is not an ideal method for quality control.
- CBR equipment is cumbersome and transporting it presents a challenge if it is to be used in remote areas with low accessibility or where the load carrying capacity of the in situ soils is low.

In an effort to overcome the aforementioned difficulties, Van Vuuren investigated several instruments and determined that the DCP was the least expensive, simplest tool that most closely correlates with conventional CBR; DCP correlates within a range of CBR 1 to 50 which is wide enough to be useful.

Apart from DCP's application in obtaining CBR values, the DCP can be a useful tool for site investigation or reconnaissance expeditions. Van Vuuren elaborates on DCP's other uses, including its capacity to;

- reveal soft patches in compacted soils. A longer rod can be used for soundings deeper than 1m.
- estimate, with experience, the density of soil structures, such as earth fill and soil retaining walls, without disturbing them.
- be used in conjunction with a hand auger for quick terrain evaluation. Penetration readings are calculated alongside auger holes spaced at extremities of the area. The type of material can be ascertained from boreholes and the penetrometer can be used to probe the areas between the boreholes and interpret the soil over the whole area.
- function as a quality control instrument on compaction jobs. Lower layers can also be retested without disturbing the upper layers.

Agencies like the Minnesota DOT have suggested using the DCP for similar applications, as outlined by Van Vuuren. According to Burnham et al. (1993), the DCP can be used to identify weak spots. The weak spot will generate a high DCP index. Once the weak spot has been identified, the cause can be determined and the area reworked to improve its strength. Other applications for DCP referred to by Burnham et al. (1993) are;

- DCP's use in identifying high strength layers in pavement structures. The DCP was used to measure the relative strength of stabilized and unstabilized road layers.
- DCP's capacity to measure uniformity of a base material or a subgrade. DCP index of layers can be compared to see the uniformity of the areas and the uniformity between different locations.
- DCP's use in supplementing normal soil survey operations. DCP tests can be performed near thin wall sampler holes or through a drilled hole and the results compared with those obtained in the lab from field samples.
- DCP can also be used as a quality control tool during the backfill compaction of pavement edge drain trenches.

MNDOT has approved a specification for use of the DCP as a quality control tool in granular material during compaction of highway construction material. The method has been approved as an alternative to the specified density method. The specification requires the material to be compacted to achieve a DCPI of less than or equal to 10 mm/Blow for a layer defined as 75 mm (but can be increased to 150 mm if Vibratory roller is used). The frequency of the test is one in every $800m^3$.

Historically, engineering researchers and field specialists concur that the DCP has great potential for use as a quality control tool in earthwork construction. In his conclusions, Hassan (1996) noted that DCP's had excellent potential for use as a compaction control device. Despite earlier cautions by researchers, Hassan's report also features research demonstrating that the DCP is well suited for use in both granular and fine grained soils. Edil et al. (2004) also noted that the DCP can be used as a control tool by measuring the strength and stiffness of the soil. The in situ strength and stiffness properties of various materials can rapidly and directly monitored in their current state of density and moisture condition.

From their study in 1995, Bratt et al. concluded that the use of moisture and density as control parameters alone in earthwork construction does not always ensure soil stability, especially in moisture sensitive soils. They go on to say that stability, as measured by the DCP penetration rates, is more predictable by moisture content than by soil density, and that control of moisture content is therefore more critical for obtaining stability. Furthermore, Bratt et al. (1995) concluded that the DCP index necessary for achieving adequate stability (minimum 6-8 CBR) also indirectly indicates that moisture – density levels are acceptable.

Burnham and Johnson (1993) state that the DCP is an ideal tool for monitoring all aspects of pavement subgrade and base construction. The authors detail use of the DCP for verification of the compaction levels and uniformity and identification of problem areas that develop as a result of unavoidable soil conditions induced by rainy weather. The authors cite an example in which a stabilized section at an airport was checked with a DCP only to reveal that the upper 12 inches of the section had been stabilized but the yielding of the area, due to construction traffic, was actually caused by a soft layer 30 to 40 inches (76 to 102 cm) below the surface.

Other uses for the DCP noted without extensive explanation include determination of settlement potential and, to a limited extent, classification of soils being tested (Hassan, 1996). Nazzal (2003) writes of Huntley (1990), who suggested a tentative soil classification system based on penetration resistance, denoted as n, in blows per 100 mm as illustrated in Table 2.1 and Table 2.2. The tables must be used with extreme caution until further understanding of the skin friction on the upper drive rod is established.

Table 2. 1 Suggested classification for granular soils using DCP (Huntley, 1990)

Classification	n Value Range		
	Silt sand	Sand	Gravelly sand
Very Loose	< 1	< 1	< 3
Loose	1 - 2	2 - 3	3 - 7
Medium dense	3 - 7	4 - 10	8 - 20
Dense	8 - 11	11 - 17	21 - 33
Very Dense	> 11	> 17	> 33

Table 2. 2 Suggested classification for cohesive soils using DCP (Huntley, 1990)

Classification	n Value Range
Very soft	< 1
Soft	1 - 2
Firm	3 - 4
Stiff	5 - 8
Very stiff to Hard	> 8

Researchers recommend the use of the DCP for quality control because it is light, inexpensive, portable and versatile. Brat et al. (1995) list a number of practical benefits of the DCP in comparison to density gauge. Some of the DCP's benefits include the following;

- DCP costs up to one-tenth of the price of a nuclear density gauge.
- DCP requires little maintenance and regulation while there is periodic maintenance and regulation required for the nuclear gauge.
- DCP is simple to operate and efficient; it takes only minutes to train a technician to use the DCP and the device can complete five tests in the time it takes to complete one nuclear density gauge.
- The DCP is more versatile. The nuclear density gauge can be used to evaluate the top 203 mm (8 inches) of material, while the DCP can measure stability to a depth of 1m. This versatility allows the operator to investigate and determine the limits or source of surface instability.

It must be noted, however that the DCP is ideally used to supplement other quality control techniques - like the use of the nuclear density gauge. In doing so, the nuclear density gauge can be used to measure the density and moisture content of the material and, subsequently, the DCP can be used to measure the stability of the material (Sowers and Hedges, 1996).

2.5 Existing Correlations with DCP Penetration Index

There are several correlations used with the DCP. The common correlations are with California Bearing ratio (CBR), Unconfined Compressive Strength (UCS), Shear strength and Resilient Modulus.

2.5.1 DCPI Correlations With CBR

The most common correlation of the DCPI is to CBR. CBR is defined as the ratio of the resistance to penetration developed by a subgrade soil to that developed by a specimen of standard crushed-rock base material. CBR values are often used as input parameters for road and pavement design. Several studies have been performed to determine the correlation between DCPI and CBR, and a number of relationships have been documented. Several of the relationships used for the correlation of CBR to DCPI are in the form of the following equation:

$$\text{Log CBR} = A - B \log (\text{DCPI})$$

Where A and B are regression coefficients, A ranging from 2.438 to 2.60 and B ranging from 1.07 to 1.16. CBR is expressed as a percent and DCP is in mm/blow (Hassan 1996).

The variation of these equations is based on materials used in the study to develop the relationship. Webster et al. (1992) state that the best equation for use with most materials is:

$$\text{Log CBR} = 2.46 - 1.12 \text{ Log DCPI} \quad (2.1)$$

where CBR is the California Bearing Ratio in percent and DCPI is the penetration index in mm/blow. This is the equation that has been adopted by the ASTM D 6951. Livneh et al.

(1995) carried out field and laboratory tests to develop correlation between DCP and CBR. With data obtained from 56 points, an equation was developed which was later refined by adding more data points. The improved equation was based on 135 data point; however, according to Livneh et al., from a practical standpoint, the two equations yield almost identical results.

Other relations are presented in Table 2.3 below from publications by Ese et al. (1994), Salgado et al. (2003) and Amini (2003).

Table 2. 3 DCP-CBR Correlations

Correlation equation	Material tested	Reference
log (CBR) = 2.56 -1.16 log (DCP)	Granular and Cohesive	Livneh (1987)
log (CBR) = 2.55 -1.14 log (DCP)	Granular and Cohesive	Harison (1987)
log (CBR) = 2.45 -1.12 log (DCP)	Granular and Cohesive	Livneh et al (1992)
log (CBR) = 2.46 -1.12 log (DCP)	Various soil types	Webster et al. (1992)
log (CBR) = 2.62 -1.27 log (DCP)	Unknown	Kleyn (1975)
log (CBR) = 2.14 -1.04 log (DCP)	Granular and Cohesive	Livneh et al (1995)
log (CBR) = 2.44 -1.07 log (DCP)	Aggregate base course	Ese et al. (1995)
log (CBR) = 2.60 -1.07 log (DCP)	Aggregate base course and cc	NCDOT (pavement, 1998)
log (CBR) = 2.53 -1.14 log (DCP)	Piedmont residual soil	Coonse (1999)

ASTM specification D 6951 – 03 uses the following correlations to estimate CBR:

$$CBR = \frac{1}{0.002871(DCP)} \qquad \text{(CH soils)} \ldots\ldots\ldots\ldots\ldots\ldots \text{(2.2)}$$

$$CBR = \frac{1}{(0.017019(DCP))^2} \qquad \text{(CL soil for CBR <10)} \ldots\ldots\ldots\ldots \text{(2.3)}$$

$$CBR = \frac{292}{DCP^{1.12}} \qquad \text{(All other soils)} \ldots\ldots\ldots\ldots\ldots \text{(2.4)}$$

where CBR is in percent and DCP is the penetration index in mm/Blow. These are the correlations that will be used in this study. Instead of using one generalized equation for all soils types, this study relies on the application of those correlations that generate the best estimation of CBR from DCP.

2.5.2 Unconfined Compressive Strength

Another published correlation of the DCPI features unconfined compressive strength (UCS). Kleyn et al. (1983) published a graphical representation for the correlation of between UCS and DCPI. McElvaney and Djatnika (1991) published an equation for the correlation between UCS and DCPI based on laboratory studies. Their equation is

$$\text{Log UCS} = 3.21 - 0.809 \text{ Log DCPI} \quad \text{...} \quad (2.5)$$

where UCS is the unconfined compressive strength and DCPI is the penetration index in mm/blow. This equation assumes 99% confidence that the probability of underestimation will not exceed 15 percent.

White el al. (1999) performed studies that correlated the UCS to DCPI. The work was continued in phase IV of the Embankment Quality Project, where more soils were used to develop correlations. The information is presented in chapter 3 of this thesis.

2.5.3 Shear Strength

Laboratory results gained from studies by Ayers et al. (1989) provided predictive equations for the correlation between shear strength and DCPI for granular materials. The equation is of the form DS = A - B (DCPI), where DS is the Deviator stress at failure (shear strength) and A and B are regression coefficients. As shear strength of granular materials varies with confining pressure, the experiments were performed at different confining pressures. Equations were developed for the different confining pressures. The selection of the appropriate prediction equation requires an estimate of the confining pressure under field loading conditions; this was stated to require further investigation.

2.5.4 Modulus Correlations

Studies performed to relate resilient modulus (M_R) to DCPI relate it either through the CBR relation to DCPI then relate CBR to resilient modulus, or they relate resilient modulus directly to the DCPI. The AASHTO Guide for Design of Pavements has adopted the following equation for the relation between CBR and resilient Modulus,

M_R (MPA) = 10.34*CBR or M_R (psi)=1500*CBR.................................... (2.6)

With the use of this equation with the equation adopted by ASTM 6951 for CBR (equation 2.4 above), we find that DCPI correlation yields results that are very similar to those obtained from the Falling weight Deflectormeter (FWD) (Chen et al. (2001)). The equation combining equation 2.4 and 2.6 can be written as follows:

M_R (MPA) = 664.67 * DCPI $^{-0.7168}$ or M_R (ksi)= 96.468* DCPI$^{-0.7168}$................(2.6a)

Hassan (1996) developed a correlation between DCPI and M_R using the model

M_R = 7013.065 – 2040.783 * Ln DCPI.. (2.7)

where M_R is in psi and DCPI in in/blow. This correlation is only significant at optimum moisture content; it becomes insignificant at moisture content +/- 20% of optimum moisture content.

Elastic modulus correlation with DCPI has been determined by Chai et al. (1998) using CBR-DCP results and DCP tests to determine in situ subgrade using the equation

E(MN/m^2) = 17.6 (269/DCPI)$^{0.64}$.. (2.8)

where DCPI is in blows per 300 mm.

Jianzhou et al. (1999) discovered a strong correlation between DCPI and the FWD-Backcalculated moduli in the form

$E_{(back)}$ = 338 DCPI$^{-0.39}$... (2.9)

$E_{(back)}$ is back calculated subgrade modulus (MN/m^2).

2.6 Control Test Frequency

The frequency of quality control testing is as critical for a quality product as the quality control tests themselves. Trenter (2001) lists the following as some of the factors on which the control test frequency depends;

- The volume of fill placed and nature of the structure
- The uniformity of the fill, e.g. whether just one soil (or rock) type or several, and whether the material type(s) are uniform in themselves
- The outcome of the compaction trials, i.e. whether or not generally consistent control test results were achieved; the wider the spread of the results during the trials, the more tests should be performed during main works construction
- The progress of the main works compaction itself.

Trenter (2001) further explains that the size of the site can also affect the frequency. In the case of a small site, much maneuvering can disturb finished work, therefore, relatively more control testing will be warranted than for a larger site. In 1996 Trenter and Charles published guidelines that can be used for determining the minimum frequency of quality control testing in a graphical form. The frequency of testing for 1,000, 10,000 and 100,000 m^3 compacted material are given as 5, 3 and 2 tests respectively. The guidelines should be taken as preliminary as the frequency of quality control testing ultimately depends on the factors described above

2.7 Conclusion

There is a compelling need for constructing quality embankments for highways. The design and construction of the embankment is based on the strength and stiffness of the soil on which the pavement is placed. For design, studies have demonstrated that a subgrade with CBR of about 6-8 % is sufficient for highway pavement (Illinois DOT, 1982, Bratt et al 1995). The strength of the embankment is dependent upon the moisture content and the compaction of the embankment.

Frequency tests during the construction of the embankment are important to ensure that quality is achieved. The tests most commonly performed during construction are moisture tests and density tests. There is a need, however, to supplement these tests with a test that measures the strength and stability of the embankment. As the literature review has revealed, the DCP is a tool that can be used efficiently and effectively to achieve consistent results. It is inexpensive, versatile and easy to use. There are several correlations that can be used to obtain design parameters already in place on which quality control can be based.

CHAPTER 3: DCP INSTRUCTIONS FOR QUALITY CONTROL

This chapter presents the instructions for using the DCP as a quality control tool. Quality Control using the DCP has three main parts. The following narrative outlines instructions for each of the three sections of quality control.

Part I. Conduct the DCP tests in the field after the material has been placed. The inspector performs as many DCP tests to adequately represent the engineering properties of the whole volume of soil placed.

Part II. The second part is the data processing. This is were the data is collected and analyzed to give results in the form of strength parameters using correlations or just as DCPI and graphically in profiles and control charts. This can be done with a paper and pencil with a calculator or it can be sped up using data collection and analysis systems like G-RAD. The use of G-RAD is further explained in chapter 4.

Part III. Apply criterion that can be used to determine the quality of the placed materials. Correlation of DCP index to engineering parameters can be used as limits for Quality control.

3.1 PART I: Conducting DCP Testing

DCP Instructions

The following are instructions on how to operate the dynamic cone penetrometer (DCP) in its use as a quality control tool to measure the strength and uniformity of subgrade material in fine grained soils.

The aim for these instructions is to show the inspector how to assemble and operate the DCP and how to take and record readings from a DCP test.

The DCP is a field tool that can be used by field engineers or field technicians to inspect the material placement during embankment construction. Very minimum training is required to use the tool. The DCP test may require two operators. One person can operate the test but it may be uncomfortable. If two people perform the test, one person operates the DCP while the other reads and records the number of blows and the penetration of each or as many blows as are required.

The instructions will help the operator know how to assemble the DCP, perform a DCP test and take measurements. The results from this test will then be used in the second and third stages of the quality control process.

ORGANIZATION

a. *Description of the equipment*

   This section will briefly discuss the parts of the DCP and how to assemble the instrument

b. *Setup of the DCP*

   This section will show how to assemble the DCP

c. *Test procedure*

   This section will discuss how to perform a DCP test.

DESCRIPTION OF EQUIPMENT

The DCP and the test procedure used for the test are described in detail in ASTM standard D 6951- 03, Standard test method for use of the DCP in shallow pavement applications. Figure 3.1 shows an illustration of the DCP with all its parts. The device used in this study is manufactured by Kessler Soils Engineering Products, Incorporation. As shown in Figure 3.1, the DCP consists of the upper and the lower shafts with a diameter of 16 mm (5/8 in). The upper shaft has an 8 kg (17.6 lb) drop hammer with a 575 mm (22.6 in) drop height. The hammer can be converted to a 4.6 kg (10.1 lb) when the testing weaker material where the 8 kg (17.6 lb) would produce excessive penetration. The upper shaft is attached to the lower shaft through the anvil coupling. The lower shaft contains the anvil and a cone is attached at one end.

A permanent cone is used if the test is performed in a material from which retrieval of the instrument is not very strenuous. Disposable cones may be used for the ease of retrieving the instrument in the absence of an extraction jack. Both the permanent cone and disposable cone have a 60° angle and 20 mm (0.79 in) at the widest point. The shaft diameter is smaller than the diameter of the cone to ensure that the resistance is only exerted on the cone tip. A graduated drive rod or vertical scale is used to measure the penetration depth per number of blows.

SETUP OF THE DCP

Assemble the DCP as seen in the Figure 3.1. To assemble the instrument, slide the top rod through the hammer, (the top rod is the one with the handle), with the smaller part of the hammer at the bottom. Holding the top rod upside down, screw it into the coupler of the bottom rod. Tighten the coupler with a wrench, taking care not to strip the threads. If using the permanent cone tip, screw it into the bottom rod, otherwise, if using the disposable cone tips, attach the disposable cone tip attachment to the bottom rod. The attachment has an O-ring that holds the disposable tips. Ensure that all joints are securely tightened including the coupler assembly and the adapter for the disposable cone tip. Operating the DCP with loose joints will lead to damage of the equipment.

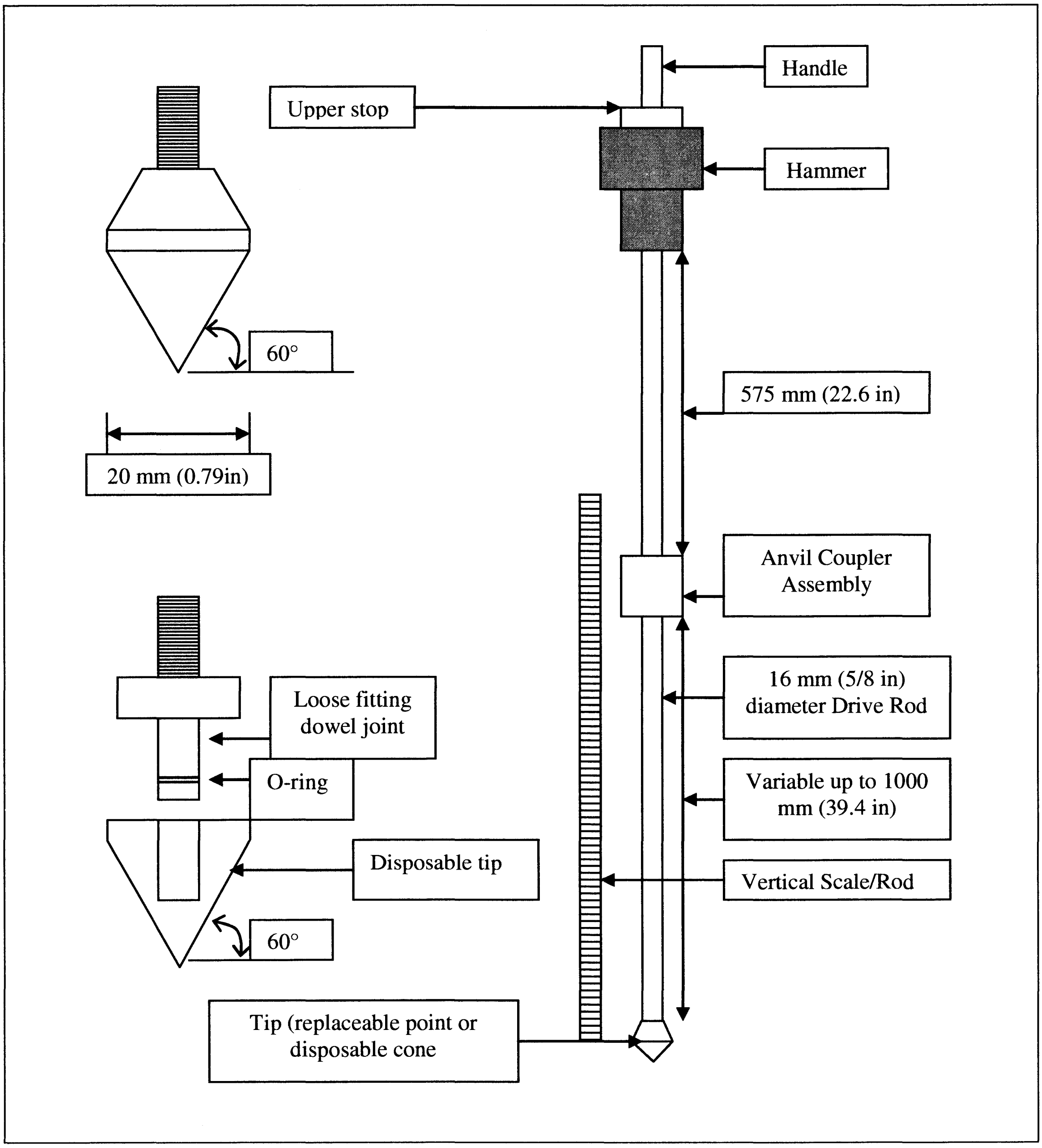

Figure 3. 1 Structure of the Dynamic Cone Penetromter

TEST PROCEDURE FOR CONDUCTING DCP TESTING

After the DCP has been assembled the operator is ready to perform the test. To Perform the DCP test, seat the DCP at the test location such that the top of the widest part of the tip is flush with the surface being tested as shown in Figure 3.2. Hold the DCP vertically and using a reference point on the DCP, note the initial reading on the vertical scale. The distance is measured to the nearest 1 mm (0.04 in). Holding the instrument vertically minimizes side friction so that the hammer delivers the full force to the lower rod during the test.

Lift the hammer, ensuring that the hammer only touches the handle at the top when raised, and does not raise the instrument (Figure 3.2). Allow the hammer to free fall, while maintaining the instrument in a vertical position. At this point, record the new reading using the same reference point as before. Continue lifting and dropping the hammer until the instrument has penetrated to the desired depth for the test. Care is needed in operating the DCP to prevent injury as there are pinch points on the instrument.

Table 3.1 shows the recommendations that ASTM standard D 6951 and Army Corps of Engineers have for how often readings are to be taken. When material is stiff, readings are taken less frequently, however when the material is soft, readings are taken after each blow. If the material is too soft, the 4.6 kg (10.1 lb) hammer can be used to get a better profile of the material. This will mostly likely be unnecessary as this will mean that the material will not pass the quality control.

Table 3. 1 Penetration Rate Recommendations

Penetration Rate		Record After
mm/blow	in/blow	
2.54 – 12.7	0.1 – 0.5	10 Blows
15.24 – 25.4	0.6 – 1.0	5 Blows
> 25.4	>1.0	Each Blow

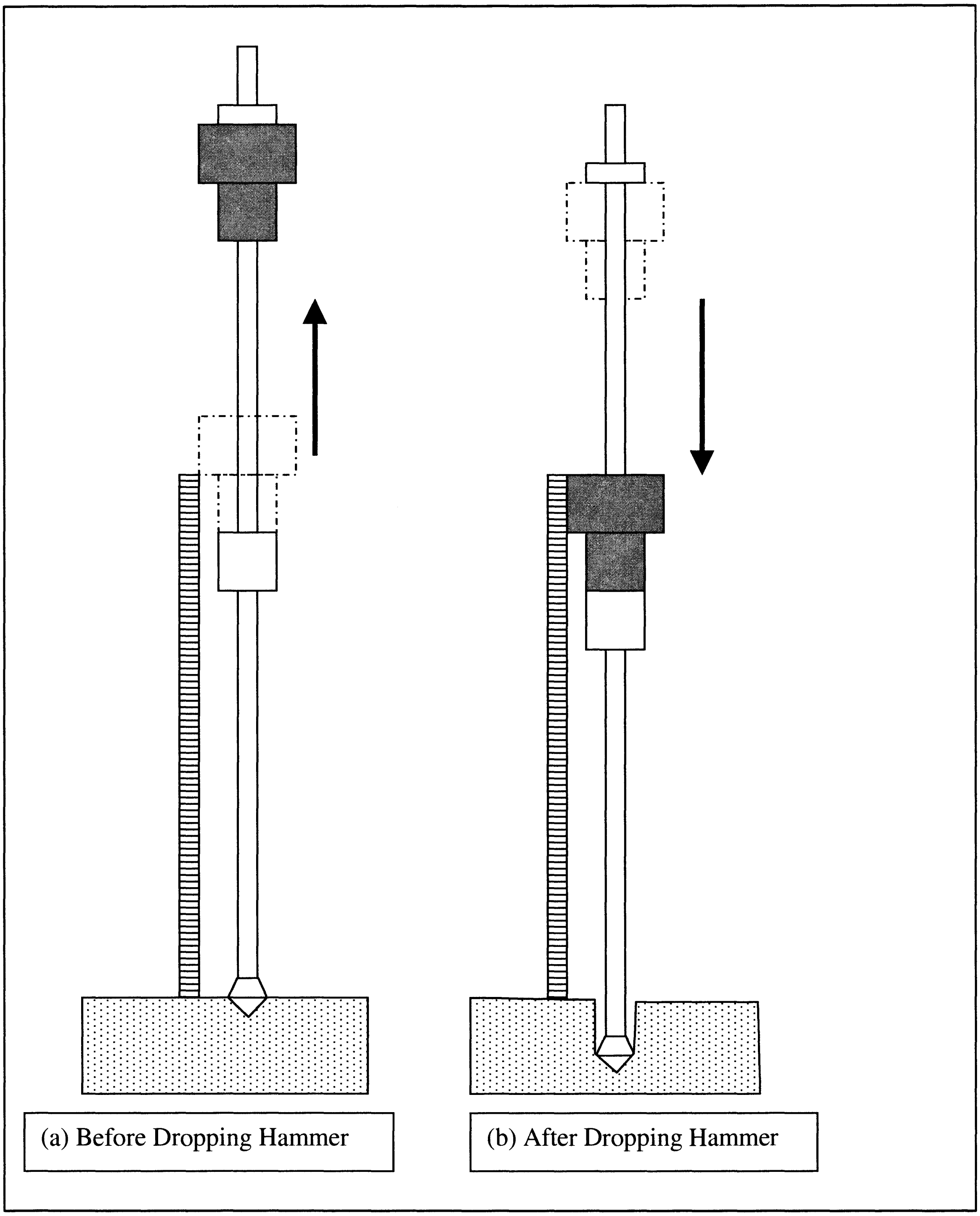

Figure 3. 2 Dynamic Cone Penetration Test

3.2 PART II: Data Collection and Analysis

DCP data can be collected by completing a form with the headings Number of Blows, Cumulative Penetration, Penetration Between Readings, Penetration per blow, Hammer Factor DCP index CBR and Moisture. Table 3.2 shows an example of a form that can be used for data collection and analysis.

Table 3. 2 Data collection Form

Number of Blows (1)	Cumulative Penetration (mm) (2)	Penetration Between Readings (mm) (3)	Penetration Per Blow(mm) (4)	Hammer Factor (5)	DCP index (mm/blow) (6)	CBR (%) (7)	Moisture (%) (8)
0	107	-	-	-	-	-	-
2	286	179	89.5	1	89.5	1.903	22.5
2	368	82	41	1	41.0	4.561	
3	524	156	52	1	52.0	3.495	
3	631	107	35.7	1	35.7	5.331	
2	740	109	54.5	1	54.5	3.316	
2	859	119	59.5	1	59.5	3.006	
1	953	94	94	1	94.0	1.801	
1	1005	52	52	1	52.0	3.495	

1) No. of hammer blows between test readings
2) (2) Cumulative cone penetration after each set of hammer blows starting from initial reading
3) Difference in cumulative penetration (2) at start and end of hammer blow set
4) (3) divided by (1)
5) Enter 1 for the 8 kg hammer and 2 for the 4.6 kg hammer
6) (4) multiplied by (5)
7) From CBR Versus DCP correlation using a chosen formula; for example

$$CBR = \frac{(292)}{(DCP)^{1.12}}$$

8) % Moisture content when available

The penetration rate, PR, penetration index or DCP penetration index, DCPI can then be calculated as the ratio of the depth penetrated per blow, mm (in)/blow for each segment. The DCPI for the whole depth can also be calculated using weighted averages using the following formula:

$$DCPI_{wt.avg} = \frac{1}{H}\sum_{i}^{N}[(DCPI_i).(z_i)] \quad (3.1)$$

Where N is the total number of DCPI recorded in a given penetration depth of interest, z is the penetration distance per blow set and H is the overall penetration depth of interest. In a study by Albright (2002) the weighted average method yields a narrower standard of deviation for the representative DCPI value and provides better correlations to other field tests than the arithmetic average method based on available field data. Table 3.3 shows sample calculation of DCPI of the soil.

Table 3. 3 Sample calculations of DCP-I and UDCP-I

Entered Data		Calculated		change in	DCPI	UDCPI	DCPI.z	UDCPI.z
#Blows (1)	Depth (mm) (2)	#blows (3)	Depth(mm) (4)	Depth (z) (mm) (5)	(mm/blow) (6)	(mm/blow) (7)	(8)	(9)
0	107	0	0		0			
2	286	2	179	179	90		16021	
2	368	2	261	82	41	48.5	3362	3977
3	524	3	417	156	52	11.0	8112	1716
3	631	3	524	107	36	16.3	3816	1748
2	740	2	633	109	55	18.8	5941	2053
2	859	2	752	119	60	5.0	7081	595
1	953	1	846	94	94	34.5	8836	3243
1	1005	1	898	52	52	42.0	2704	2184
							MDCPI	MUDCPI
							62	17

1) No. of hammer blows between test readings
2) (2) Cumulative cone penetration after each set of hammer blows
3) Same as (1)
4) Subtracting the value of the initial reading in order to start from 0.

5) Difference in cumulative penetration (2) at start and end of hammer blow set

6) (5) divided by (1)

7) Difference in DCPI from one test to the next

8) (5) multiplied by (6)

9) (5) multiplied by (7)

10) MDCPI is sum (8) divided by sum of (5)

11) MUDCPI is sum (9) divided by sum of (5)

Data Collection

The process of data collection and processing can be performed with paper and pencil and a calculator but takes too long. As shown by the procedure mentioned above, the number of calculations involved presents a many of opportunities for errors by mistyping numbers into the calculator and rounding off numbers inappropriately.

To speed this process up, a desktop computer can be used with a spread sheet set up so that the only data entered is the number of blows and the penetration depth. The spread-sheet will then calculate the penetration index of each layer and calculate the weighted average of the whole depth penetrated. The spread sheets can also be set up to convert the DCPI to other engineering parameters using correlation equations.

Unfortunately, using a desktop will delay the quality control process. Providing a desktop computer at each jobsite is unreasonable so a different method of data analysis is required that will not only speed up the data collection process but also analyze the data and perform the correlation calculations needed, in a fast real time manner.

3.3 PART III: Applying control criterion

Basis for the control limits

Quality control limits for the DCP mentioned for use with G-RAD, can be based on the following methods. White et al. (1999) presented a recommendation of control limits for maximum DCP-I and the maximum change in DCP-I (maximum UDCP-I). The maximum DCP-I value can be based on three other parameters, the required k-value, slope stability and bearing capacity. The Uniformity can be based on two other methods; the first is using the area between the plot of the number of blows as a percent of the total number of blows required for a depth for the actual test and an ideal test, to calculate a uniformity number and the second is using the plot of the gradient on a plot of number of blows as a percent of the total number of blows required for a depth of an actual test and an ideal test. The basis for quality control limits are discussed below.

Phase II recommendations

In the final report of phase II for the Highway Embankment Quality project, White et al. (1999) recommended the maximum mean DCP-I and mean DCP-I change shown in Tables 3.4 and 3.5.

Table 3. 4 Maximum Mean DCP index

	Soil Performance Classifcation	Maximum Mean DCP Index (mm/blow)
Cohesive	Select	75
	Suitable	85
	Unsuitable	95
Intergrade	Suitable	45
Cohesionless	Suitable	35

Table 3. 5 Maximum Mean Change in DCP index

	Soil Performance Classifcation	Maximum Mean Change in DCP Index (mm/blow)
Cohesive	Select	35
	Suitable	40
	Unsuitable	40
Intergrade	Suitable	45
Cohesionless	Suitable	35

The numbers were proposed as the basis for the quality control using the DCP after the soil has been properly classified using the Soil Performance classification for the disposal of the soil. The limits shown in Table 3.4and 3.5 can be changed after DCP tests on test strips have been conducted strips have been conducted. The recommended frequency of testing DCP is one test for every 1000 m^3 compacted material and determination of soil performance classification will be once every 25000 m^3 or if there is a change in material as determined by the engineer.

The maximum mean DCP index is used to control the strength of the soil placed for the embankment construction. The maximum mean change is used as a measure of uniformity. If the soil is uniform, the mean change in DCP index will be small. However, if the embankment is not uniform, the change in DCP index will be large.

During Phase IV of the Embankment Quality project, research continued into how to determine the maximum mean DCP index and the uniformity of an embankment for use as limits in the quality control process. Unconfined Compressive Strength (UCS) correlation with DCP index has been studied by some authors (Kleyn, 1983, Bester and Hallat, 1977, and White et al. 1999), showing very good correlation. As part of the Phase IV research, two studies where used to study the correlation further, Sheldon bypass project and the Spangler Lab project sites. It was concluded from these two projects that the correlation between unconfined compressive strength and DCP index can be used to establish limits for the quality control purposes.

In the final report of phase II for the Highway Embankment Quality project, White et al. (1999) evaluated shear strength of soils by obtaining several Shelby tube samples of fill for UCS testing. The evaluation use a section, approximately 4 feet deep, compacted using rubber-tired rolling. Within the test section, fill materials, compaction effort, and lift thickness were uniform. Three-foot long Shelby tube sampling operations and full depth DCP index tests were performed. Shelby tubes were hydraulically pushed with a drill rig to obtain relatively undisturbed samples and transported to the laboratory where unconfined compressive strength (ASTM D 2166) tests were performed. DCP index tests, which were performed in-place adjacent to the Shelby tube sampling locations, were matched appropriately with corresponding Shelby tube depths and strength results.

Table 3. 6 Engineering properties of the soils in the DCP-UCS correlation

No.	LL	PI	Percent passing No. 200 sieve	AASHTO	Unconf. Compress. (lb/in^2)	In-situ Density (lb/ft^3)	In-situ Moisture content (%)	Percent Comp	Deviation from Optimum Moisture	DCP Index (mm/blow)
ST-la	68	52	96	A-7-6(55)	30.4	105.5	22.8	103.9	+2.8	36
b	61	45	96	A-7-6(47)	21.8	103.8	24.7	102.3	+4.7	70
ST-2a	64	47	96	A-7-6(49)	30	105.8	22.9	104.2	+ 2.9	73
b	62	46	96	A-7-6(48)	34.4	105.9	22.6	104.3	+2.6	34
ST-3a	62	47	96	A-7-6(49)	18.7	105.4	22.3	103.8	+2.3	100
b	69	53	96	A-7-6(56)	31.5	101.3	26.6	99.8	+6.6	69
c	62	46	96	A-7-6(48)	-	105.5	23.6	103.9	+3.6	41
ST-4a	69	52	96	A-7-6(55)	20.1	105.7	23.3	104.1	+3.3	110
b	62	46	96	A-7-6(48)	-	105	23.4	103.4	+3.4	67
c	65	48	96	A-7-6(51)	-	-	-	-	-	32
ST-5a	63	46	97	A-7-6(49)	19.2	104.7	23.5	103.2	+3.5	130
b	60	44	96	A-7-6(46)	33.9	106.4	23	104.8	+3.0	46
c	61	45	96	A-7-6(47)	-	106.8	22.7	105.2	+2.7	33
ST-6a	52	37	96	A-7-6(38)	28.2	108.4	21.6	106.8	+1.6	81
b	64	47	96	A-7-6(49)	29.8	104.7	24	103.2	+4.0	51
ST-7a	66	49	96	A-7-6(52)	28	103.1	24.5	101.6	+4.5	100
b	63	46	96	A-7-6(48)	28.8	110.7	17.3	109.1	-2.7	54
c	55	39	96	A-7-6(40)	28.2	106	24.7	104.4	+4.7	47
Proc.H	63	42	96	A-7-6(45)	-	101.5	20	-	-	-

Individual test results of moisture, density, strength, soil index properties, and DCP index are provided in Table 3.6. UCS values varied from 18.7 psi (2690 psf) to 33.9 psi (4880 psf) with DCP index values of 100 and 46 mm/blow, respectively. A strong relationship is depicted between unconfined compressive strength and DCP index, as shown in Figure 3.29.

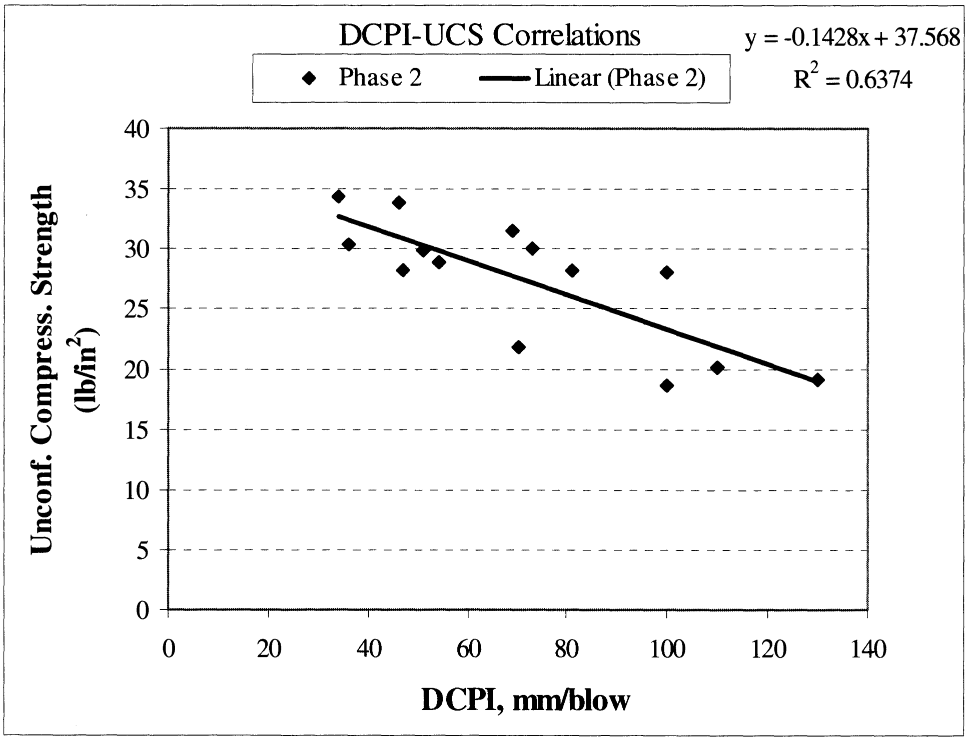

Figure 3. 3 DCP index versus UCS after White et al (1999)

As mentioned above, research work on the correlation between DCP index and UCS was continued in phase IV of the Highway embankment project. The following is a description of the two project sites that were used to study the correlation.

Project No.7: Highway 60 - Sheldon Bypass

The project was visited on July 7th, 2004. It is located on the Iowa State Highway 60, Sheldon bypass construction project southeast of the Sheldon in O'Brien County, Iowa. The aim of this site visit was to perform DCP testing and to collect Shelby tube samples of the soil for laboratory strength testing on a section of the embankment that was complete and had been left to consolidate.

Four test spots were selected for testing. DCP tests were performed at the test spots to a depth of about 800 mm. Shelby tubes were hydraulically pushed with a drill rig to obtain relatively undisturbed samples and transported to the laboratory where UCS (ASTM D 2166) tests were performed. Whenever possible, the samples were divided into three sections to get variation of strength from different depths. The strength was then compared with DCPI from the DCP tests at equivalent depths. Table 3.7 below shows the results of the DCP tests with the corresponding UCS values and the moisture content determined from the lab samples. Table 3.8 shows the DCP-I, UCS and the consistency of the soil. Figure 3.4 shows the plot of UCS and DCPI.

Table 3. 7 UCS and DCP-I with moisture content

Sample Number	Strength (psi)	DCPI (mm/blow)	Moisture content
6704-1-T	67.5	25.9	12.1
6704-1-B	62.5	34.1	15.1
6704-2-T	42.5	39.3	15.4
6704-2-B	32.4	46.6	15.4
6704-3-T	67.9	21.9	11.0
6704-3-M	52.5	29.7	14.5
6704-3-B	30.8	42.0	14.2
6704-6a-T	4.2	101.6	21.2
6704-6a-B	59.5	20.0	12.7
6704-6b-T	7.8	89.6	19.4
6704-6b-B	57.2	18.7	15.6

Table 3. 8 DCP-I and UCS from Sheldon site

Sample	strengh psi	DCP-I	Strength (KN/m^2)	strength lb/ft2	strength tons/ft2	Consistency
6704-1-T	67.5	27.1	463.40	9678.40	4.839201	Hard
6704-1-B	62.5	43.2	429.08	8961.48	4.480742	Hard
6704-2-T	42.5	40.2	291.77	6093.81	3.046904	Very stiff
6704-2-B	32.4	58.7	222.42	4645.30	2.32265	Very stiff
6704-3-T	67.9	23.2	466.12	9735.06	4.867528	Hard
6704-3-M	52.5	30.6	360.40	7527.11	3.763553	Very stiff
6704-3-B	30.8	47.1	211.43	4415.90	2.207951	Very stiff
6704-6a-T	4.2	144.4	28.83	602.17	0.301084	soft
6704-6a-B	59.5	20.0	408.45	8530.72	4.26536	Hard
6704-6b-T	7.8	170.7	53.41	1115.45	0.557723	Medium
6704-6b-B	57.2	22.0	392.66	8200.96	4.10048	Hard

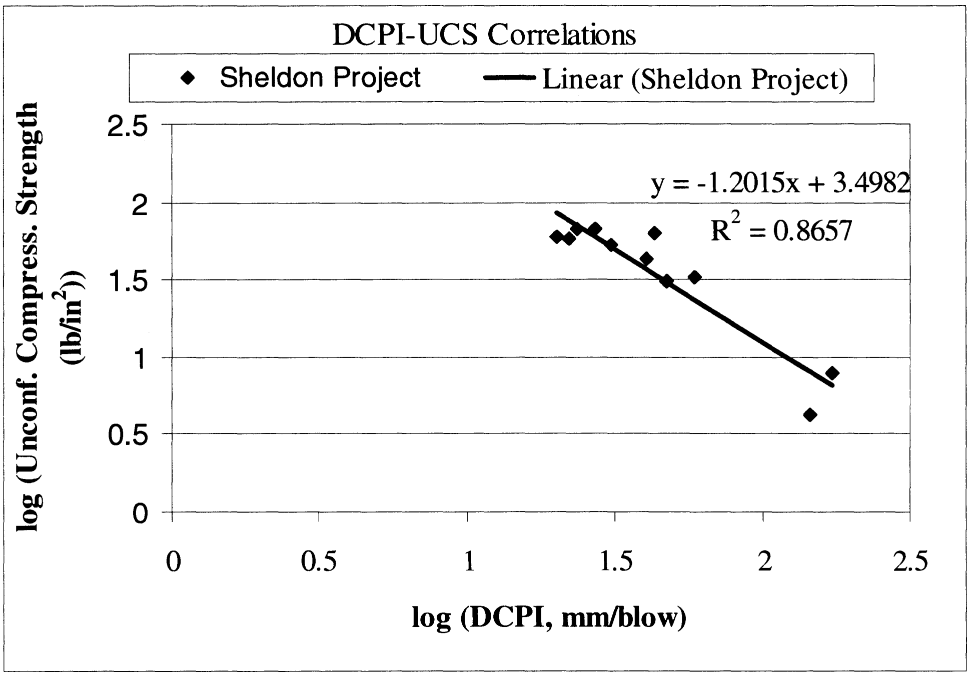

Figure 3. 4 Log strength Vs log DCPI

Figures 3.5 to figure 3.8 show DCP profiles of the DCP tests performed at the site. The CBR values range from 0.5 to 20.

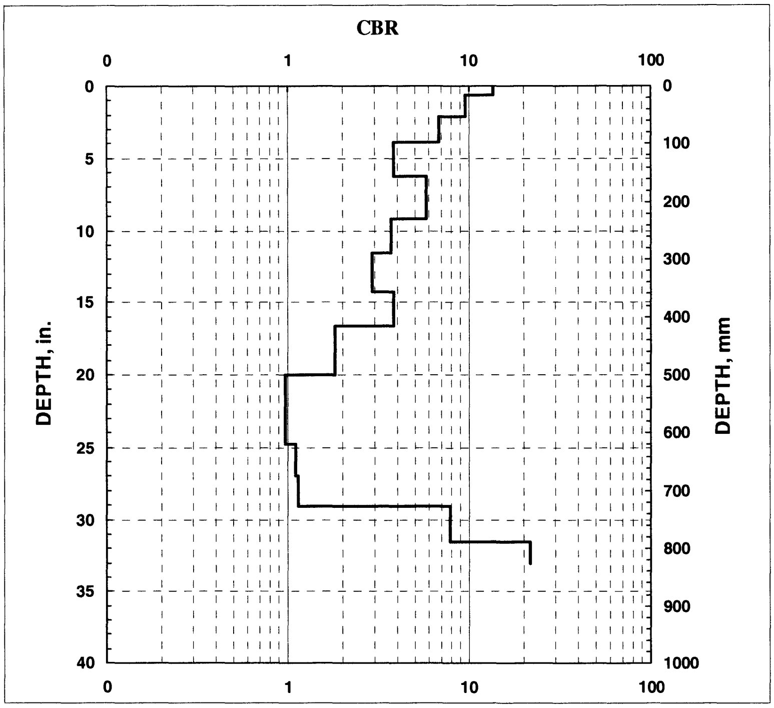

Figure 3. 5 DCP profile of test 6704-1

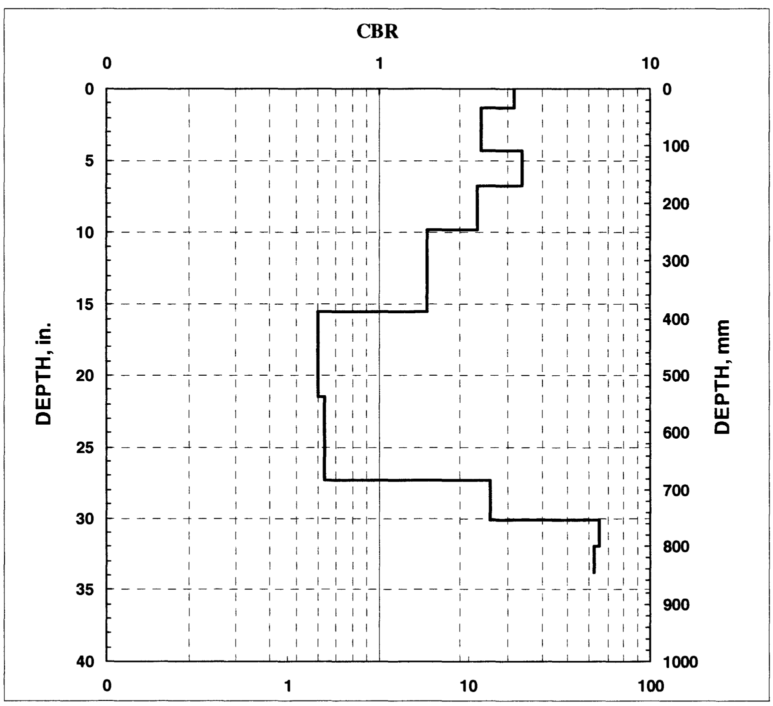

Figure 3. 6 DCP profile of test 6704-2

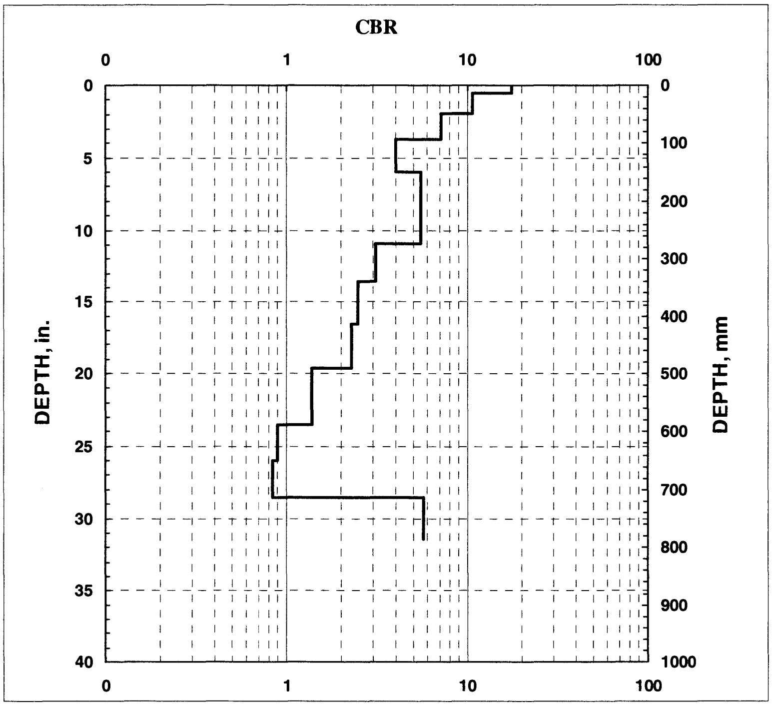

Figure 3. 7 DCP profile of test 6704-3

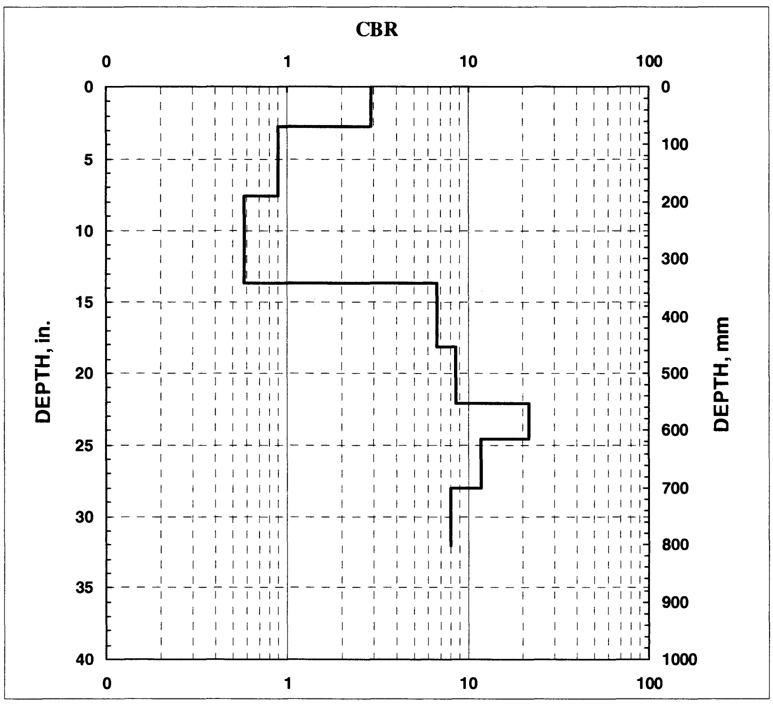

Figure 3. 8 DCP profile of test 6704-6

The DCP tests show the variation of the strength of the soil with depth. Figures 3.5 to 3.7 show that towards the top, were the soil was dry, the strength is relatively higher, decreasing toward the bottom were the soil has higher moisture content. Figure 3.8 shows higher strength toward the bottom. This test spot represented by Figure 3.8 was specifically selected for testing because it was much wetter than the surrounding areas. As is shown in the DCP profile, the top layer, which was wetter, was weaker than the bottom layer which was dryer. This test site illustrates two things. One is the moisture content is very important and secondly that the DCP can be used to get an idea of weaker spots on a sites which can then be investigated to determine the cause of the weakness.

Project No 8: Spangler Lab Project – Unconfined Compressive Strength with DCP

This project was part of research on stress-strain behavior of micro-piles in a different of soils. The soils used for this test were weathered shale, Loess, and Glacial till. The test site is located at the Spangler geotechnical lab field testing area in Ames Iowa. Wooden boxes, 600 mm by 600 mm by 600 mm, were filled with hand tamped soil. Seven meter long Micro-piles were then placed through the boxes into the preexisting ground. The Boxes were arranged in such a way as to form pairs. The reason for this was that the boxes would be pushed against each other to monitor the behavior of the micro-piles. Figure 3.9, 3.10, 3.11, and 3.12 show the preparation and final setup of the boxes. Figure 3.13 shows the arrangement of the boxes and the types of soil that were in the boxes.

Figure 3. 9 Site preparation

Figure 3. 10 Tamping the soil

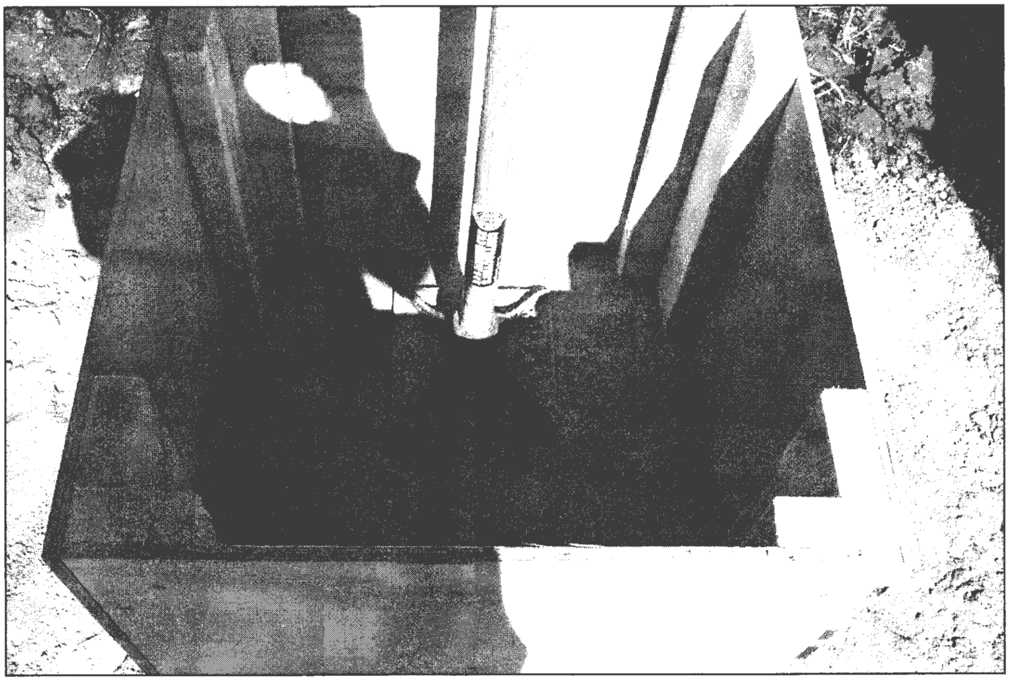

Figure 3. 11 Soil in the box

Figure 3. 12 Boxes protected from rain

After the testing the stress strain relations, DCP testing was performed in the boxes and then Shelby tube samples were taken from several of the boxes including some that did not have micro-piles in them.

After the samples were removed from the Shelby tubes, unconfined compressive strength tests were performed. Whenever possible, two or more divisions were made from the samples for the strength testing. Data of the unit weight and moisture content of the samples as obtained from the samples is shown in Table 3.9. The strength data from the tests with corresponding DCPI for the depths is also shown in Table 3.9. The individual profiles of the DCP tests are in Appendix B.

Table 3. 9 Spangler Project site DCPI with Unconfined Compressive Strength

Sample	strengh psi	DCP-I	Strength (KN/m^2)	strength lb/ft2	strength tons/ft2	Consistency
3A Top	16.7	87.8	114.64	2394.34	1.197168	Stiff
3A bottom1	15.8	85.1	108.46	2265.30	1.13265	stiff
3A Bottom2	13.2	85.1	90.61	1892.53	0.946265	Medium
3B Middle	16.2	70.0	111.21	2322.65	1.161325	stiff
3C Top	14.5	87.8	99.54	2078.91	1.039457	Stiff
4A top	3.5	253.3	24.03	501.81	0.250904	soft
4A Bottom1	4.1	178.5	28.15	587.83	0.293916	soft
6A Bottom1	23.5	65.4	161.32	3369.28	1.684638	stiff
6B Top	28	51.2	192.21	4014.46	2.007228	Very stiff

Table 3. 10 results of DCP testing from the Spangler project site

Test #	MDCPI	UMDCPI
1N	305	47
1S	848	0
2N	68	12
2S	77	14
3N	85	13
3S	86	12
4N	230	62
4S	236	34
5N	77	18
5S	77	15
6N	61	12
6S	52	7
7N	106	39
7S	80	16
8N	243	51
8S	215	55
9N	80	11
9S	60	10
10N	92	11
10S	104	20
11N	216	44
11S	133	44
12N	60	13
12S	63	9
14N	87	24
14S	76	17

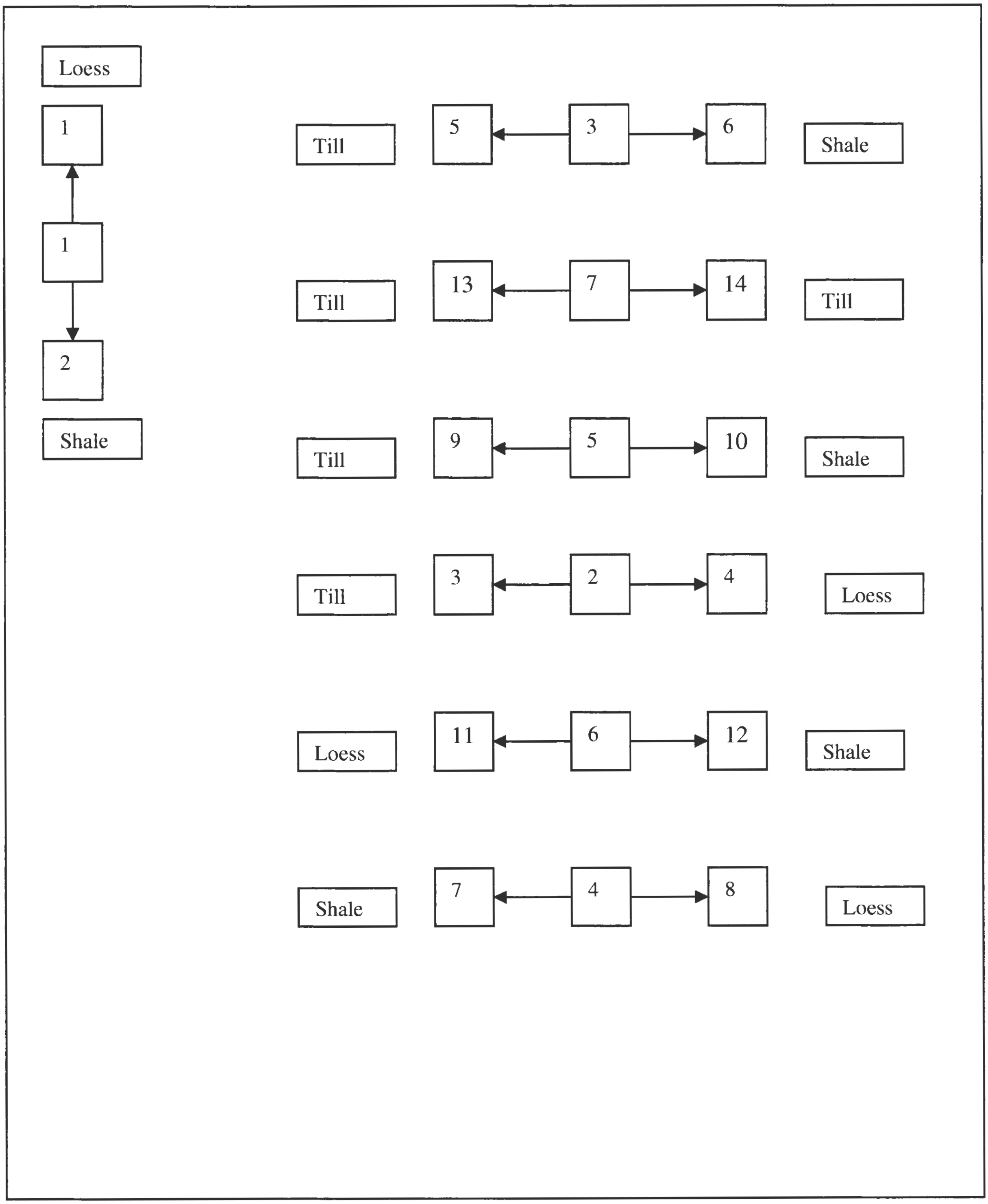

Figure 3. 13 Arrangement of test boxes and soils in each box

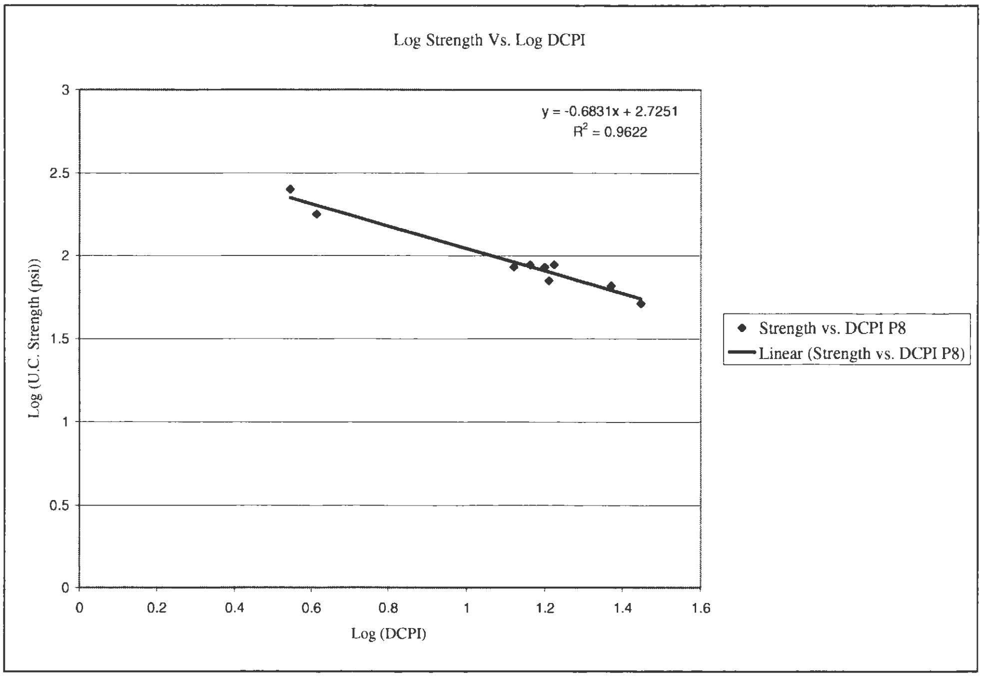

Figure 3. 14 Log (Unconfined Compressive Strength(psi)) vs. log (DCPI (mm/blow))

Figure 3.14 shows the plot of the relationship of unconfined compressive strength with DCP-I. The correlations for DCP-I and UCS from these studies can then be used to determine basis for the limits that can be used for DCP quality control testing.

The data from these two projects and the data from White et al (1999) was then plotted on a single plot to obtain a better correlation. Figure 3.15 shows the individual correlations from the three studies. Figure 3.16 shows the plot of the combination of data. The equation for the correlation is as follows;

$$\log DCP = -0.357(\log UCS)^2 + 0.732(\log UCS) + 1.966 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.2a)}$$

$$\log UCS = -0.727(LogDCP)^2 + 1.548(LogDCP) + 1.832 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.2b)}$$

Where UCS is the log UCS, (UCS in kN/m^2) and DCP is log DCP-I (DCP-I in mm/blow). Equation 3.2a converts UCS into DCP-I and equation 3.1b converts DCP-I to UCS. The correlations are based on 34 data points and have an R^2 value of 0.84. It should be noted that equations 3.2a and 3.2b are only valid for DCP values greater than 10 mm/blow.

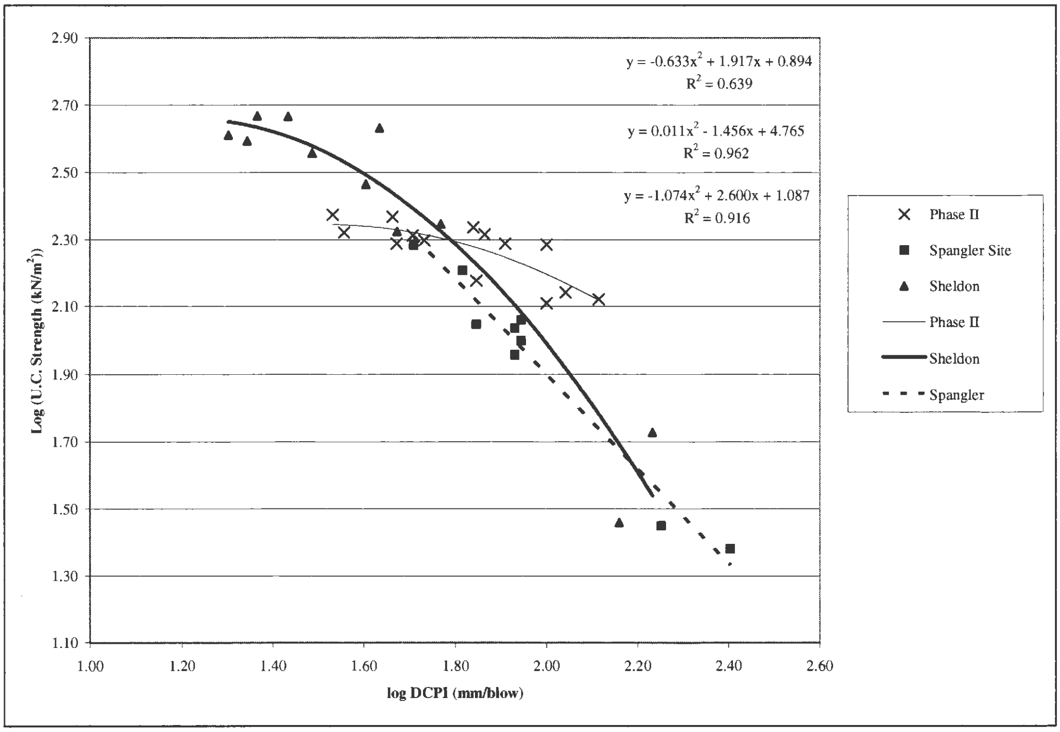

Figure 3. 15 Plot of individual correlations, from top to bottom, Phase II, Spangler and Sheldon

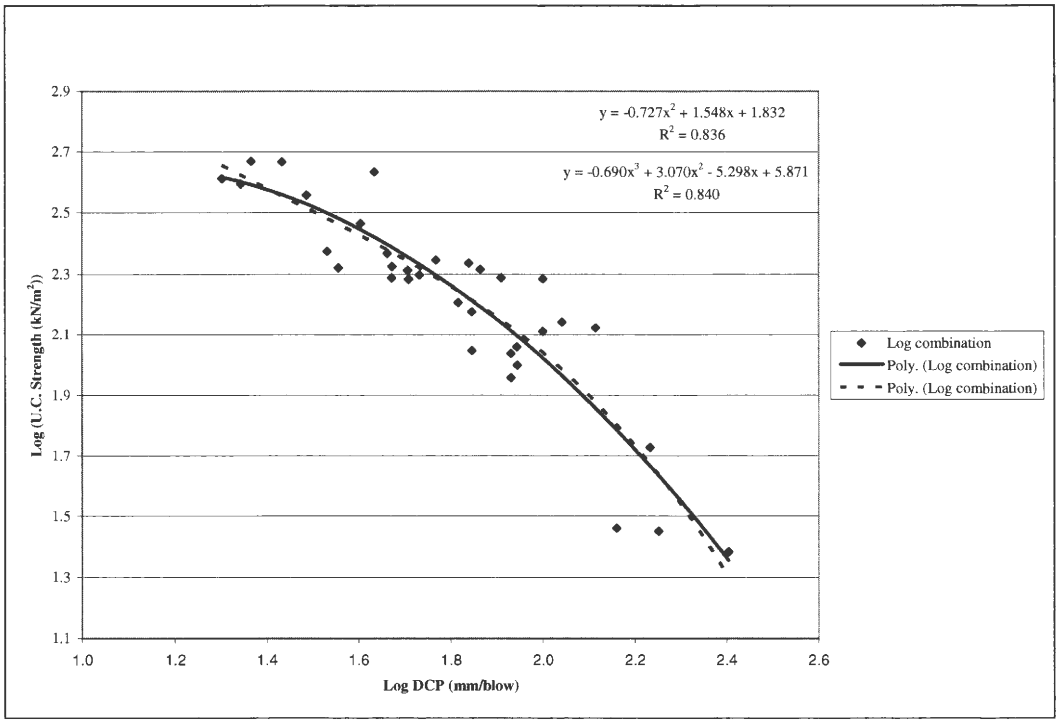

Figure 3. 16 Plot of combination correlation

Maximum DCPI

The maximum DCP-I used in quality control can be based on three parameters. The three parameters being, the pavement thickness design parameters modulus of subgrade reaction value (k-value) or the resilient modulus (M_r), the bearing capacity of the soil and the minimum strength of soil required to ensure slope stability.

Pavement thickness

Quality control can be based on measuring the modulus of subgrade reaction value (k-value) or the resilient modulus (M_r) during construction. The value measured in the field can be compared with compared with the value used in designing the pavement thickness. For instance, the Iowa Department of Transportation uses an M_r of 3000 psi for pavement design of highways. Using equation 2.6a in chapter 2, this gives a DCPI value of 126 mm/blow. For quality control, this value can be used for as the maximum value for the DCPI value during construction. This will ensure that the soil modulus on which the pavement thickness design was based on, is achieved in the field.

Bearing Capacity

The second method for basing the quality control parameters is the bearing capacity of the soil required to support the structure being constructed. In the construction of box culverts, the bearing capacity of the soil can be easily obtained using a DCP. The bearing capacity that is required will be known from the design. The DCP can be used as a tool to verify the value in the field during construction. Using equation 3.2a, the design bearing capacity, the minimum UCS, is converted from UCS to DCP-I. This DCP-I value will be the maximum DCP-I value. This is the number that is used for the quality control of the soil, for the embankment is constructed.

Slope Stability

For this method, a slope stability analysis can be performed to find the minimum strength required for the slope to be stable, given a factor of safety. For the undrained condition, the shear strength can be given by the following equation.

$$s = c_u = \frac{q_u}{2} \quad (3.3)$$

Where s is the shear strength, C_u is the undrained cohesion, σ is the normal load, ϕ is the friction angle of the soil and q_u is the unconfined compressive strength. Using the correlations for UCS to DCP-I from the studies discussed, the minimum strength required for the stability of the embankment can be used as the controlling limit for the maximum DCP-I value for quality control.

From exploratory studies for the highway embankment, a C_u value can be obtained. This value can then be used to perform stability analysis. After the factor of safety has been established based on this C_u value, the analysis is repeated, changing the shear strength, to obtain the strength that gives the minimum factor of safety, for example, a factor of safety of 1.5. This minimum strength can then be used as the minimum strength requirement for the embankment. This value can be converted to DCP-I using equation 3.2a. This DCP-I value can then be used as the maximum mean DCP-I for quality control.

Figure 3.17 and 3.18 show slope stability analysis assuming 0.6 m of aggregate base course and portland cement concrete pavement with a unit weight of 22.5 kN/m^3 (145 lb/ft^3) assumed for the layers together.

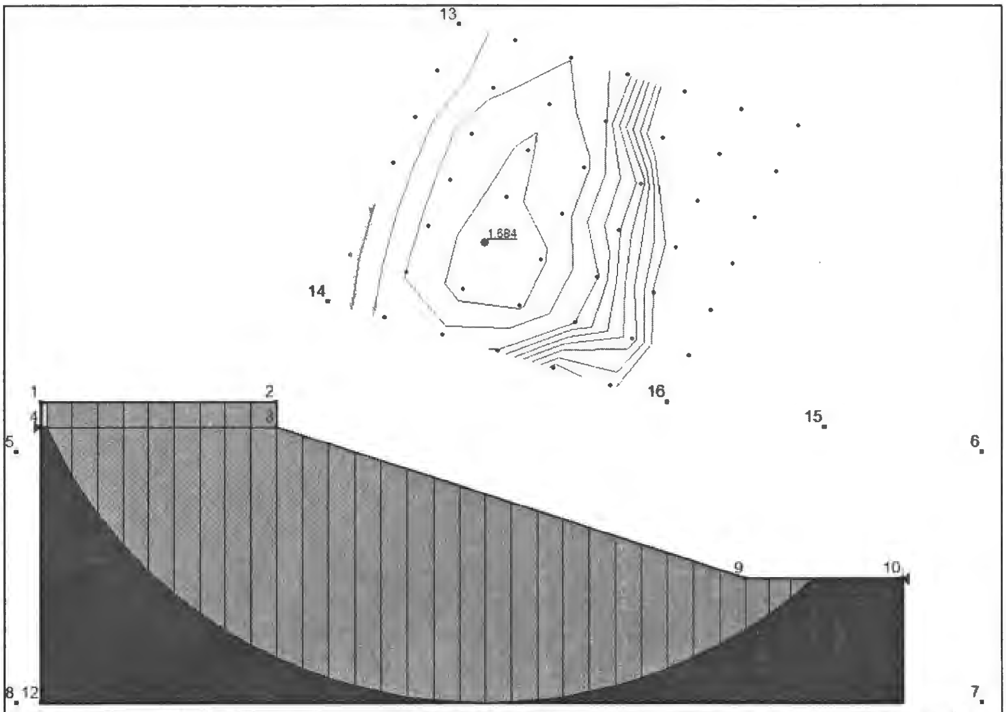

Figure 3. 17 Slope stability analysis using 15kN/m^3 unit weight and a C_u of 15 kN/m^{2}soil. FS is 1.684.

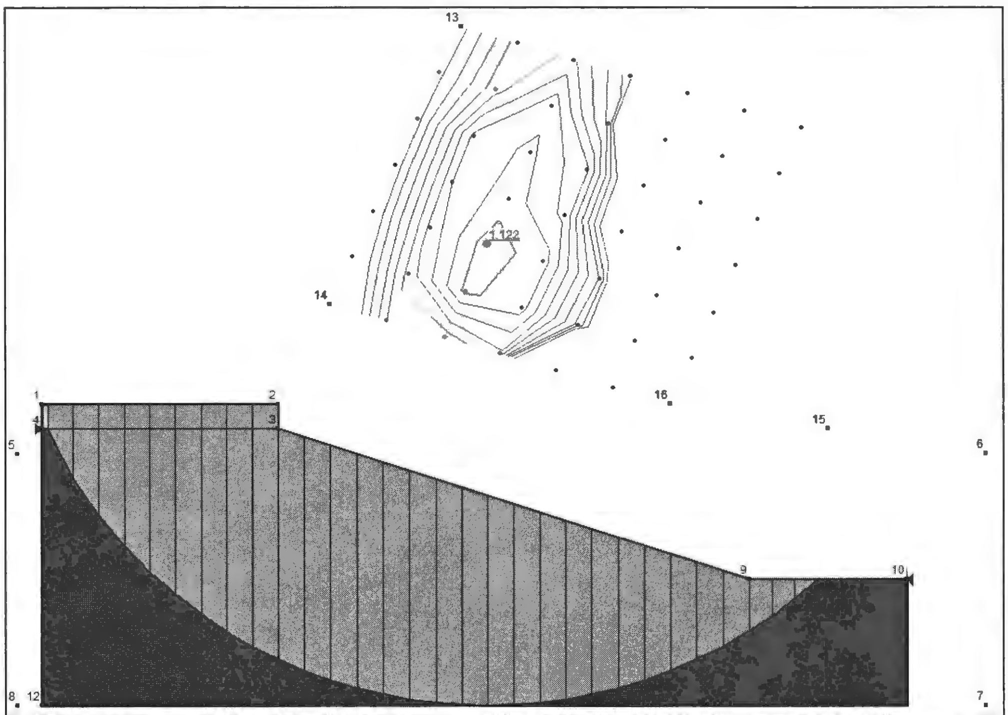

Figure 3. 18 Slope stability analysis using 15kN/m^3 unit weight and a C_u of 20 kN/m^{2}soil. FS is 1.122.

The slope stability analysis was perform using Bishop's method of slices, assuming the water table is well below the ground surface to influence the analysis. The cross section is 20 feet high, 18 feet wide at the top with a slope of 1 : 3 (vertical : horizontal).

Using equation 3.3, this gives a UCS value of 30 kN/m^2 for a factor of safety of 1.684. Using equation 3.2a gives a DCP value of 185 mm/blow. At the start of the construction, test strips can be used to refine the DCP-I value for quality control. The value can be reduced to a value for that location and one which would be achievable by the equipment available.

Table 3.11 shows slope stability analysis results using unit weights of 13, 15 and 18 kN/m^3 and cohesion values of 10, 15, 20 and 25 kN/m^2. The table also shows the corresponding DCP values.

Table 3. 11 Slope Stability analysis

Density (kN/m^3)	Cohesion (kN/m^2)	UCS (kN/m^2)	MDCP (mm/blow)	FS
13	10.0	20.00	206	1.330
13	15.0	30.00	185	1.995
13	20.0	40.00	167	2.659
13	25.0	50.00	151	3.324
15	10.0	20.00	206	1.122
15	15.0	30.00	185	1.684
15	20.0	40.00	167	2.394
15	25.0	50.00	151	2.993
18	10.0	20.00	206	1.041
18	15.0	30.00	185	1.562
18	20.0	40.00	167	2.083
18	25.0	50.00	151	2.604

Uniformity

1. A very simple method that can be used for uniformity quality control is a visual inspection of a DCP profile. If the DCP test profile has a lot of variance, the embankment is not very uniform, if the test shows limited variance, the embankment is uniform. It can be seen

from the following figures that some profiles are more uniform than others. Figures 3.19 and 3.20 can be considered less uniform than Figures 3.21 and 3.22.

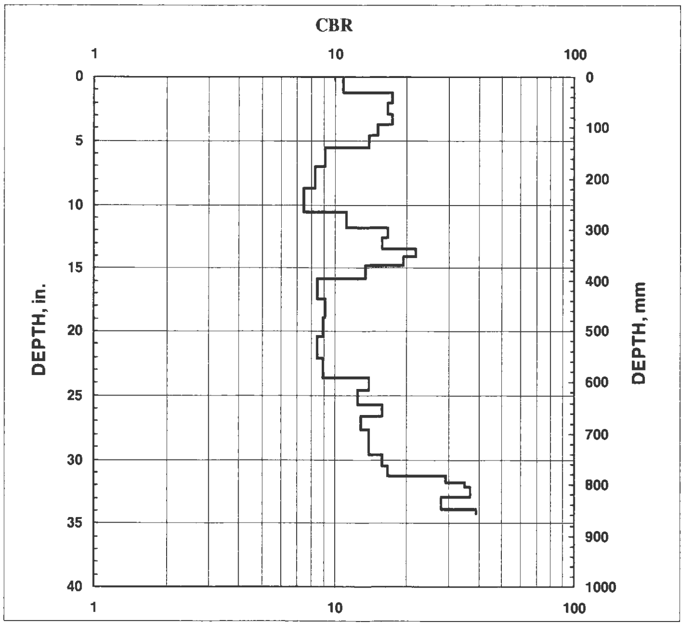

Figure 3. 19 Estimate of lift thickness from a DCP profile

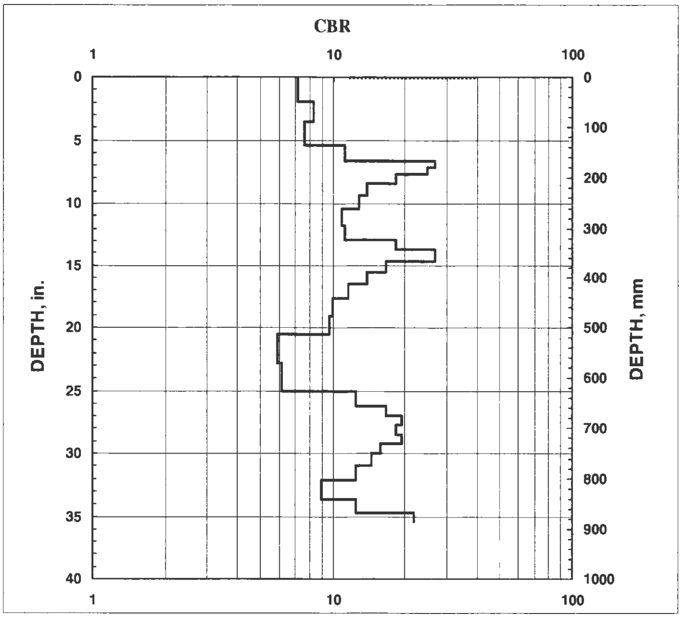

Figure 3. 20 Estimate of lift thickness from a DCP profile

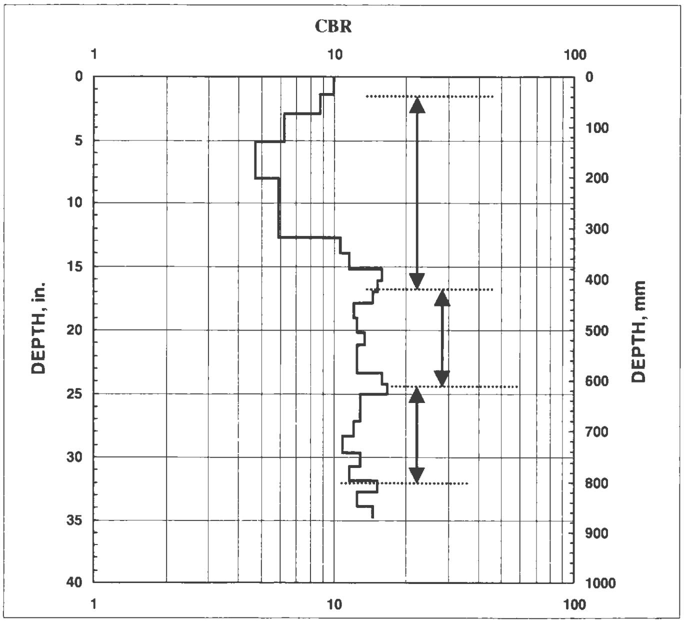

Figure 3. 21 Varying strength, stiffer towards the bottom

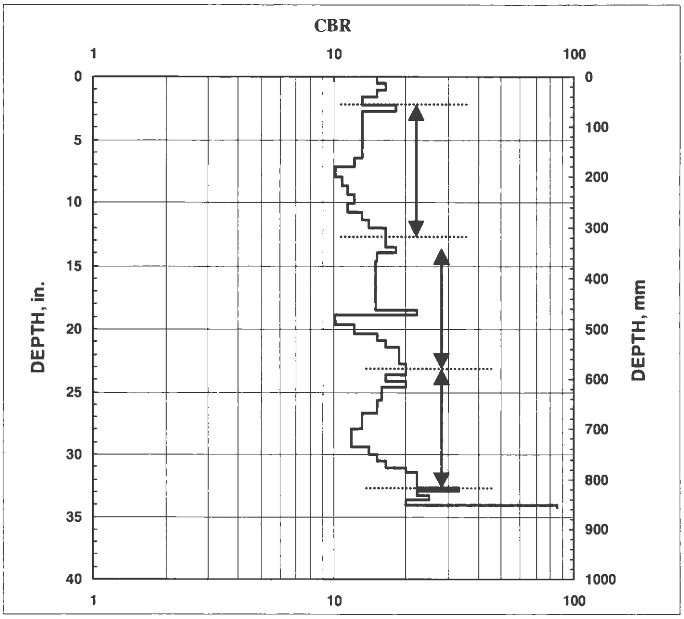

Figure 3. 22 Uniformity

Furthermore, the non uniformity, also known as the "Oreo cookie" effect, where a soft layer is sandwiched by two stiff layers, can be seen in the profiles, defining the lift thickness, which can be used to estimate the lift thickness.

2. Uniformity of the embankment tested can be assessed visually by making a plot of the number of blows as a percentage of the total required to penetrate to a depth against the depth. (Cumulative number of blows x 100 /total number of blows). Table A.1 in Appendix A shows an example of data used to produce the plot. An ideal plot can be added, one that is uniform throughout the profile. The plot is added by calculating the depth that would be penetrated using the cumulative number of blows from an actual DCP test as shown in table A.1 of Appendix A. Equations of the lines can be fitted to the lines, and then using integration, the area between the two lines can be calculated. The area calculated can then be normalized by dividing by the depth penetrated during the test. The number can be called the Uniformity number. The uniformity number of the test can be compare to one that has been established as a limit from test strips. Example plots are shown in Appendix A.

A uniform embankment will have small uniformity numbers while one that is not uniform will have large uniformity numbers. Test strips can be used to establish the uniformity number to be used for quality control. The control number can be established by obtaining an average uniformity number and then allowing one or two standard deviations for the maximum uniformity number or, a fixed maximum number can be established from relatively uniform test strips.

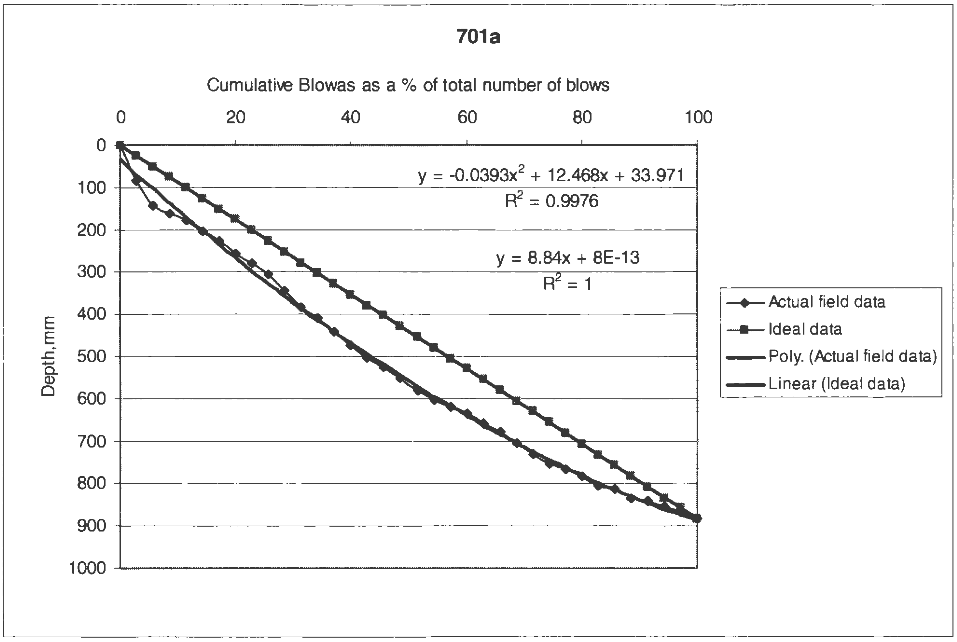

Figure 3. 23 area under the curve

The embankment will be accepted as uniform, if the uniformity number from the DCP tests is less than the established maximum number. If the Uniformity number is greater than the number established from the test strips, then the embankment layers are not uniform and corrective action is needed. Table 3.12 shows examples of DCP tests with the area and the uniformity numbers. The maximum uniformity number observed is 14.7. The average uniformity value from this data set is 6.64. The standard deviation is 3.82, therefore using the criteria of average plus one standard deviation; the maximum uniformity number will be 10.46. This would be the number that would be used in the quality control process for controlling uniformity.

Table 3. 12 Uniformity based on Area with uniformity numbers

Test	Area	Depth (mm)	Uniformity number
1A	1119.33	218	5.13
2A	1256.12	225	5.58
3A	2147.83	219	9.81
4A	1100.788	323	3.41
5A	1667.55	315	5.29
6A	2500	319	7.84
7A	1422.52	309	4.60
8A	3520.42	336	10.48
9A	576	221	2.61
10A	968.96	255	3.80
1H	2734	214	12.78
2H	3627.76	340	10.67
3H	2915.01	229	12.73
4H	3016.51	285	10.58
5H	3439.96	234	14.70
6H	1920.02	285	6.74
7H	60.731	338	0.18
8H	2708.74	293	9.24
9H	1264.2	253	5.00
10H	298.19	340	0.88
syd1	4887.074	910	5.37
syd2	10715.907	987	10.86
syd3	5646.6	898	6.29
syd4	3853.212	857	4.50
syd5	3686.867	904	4.08
syd6	2643.893	875	3.02
syd7	2723.02	892	3.05

3. Another way to assess uniformity is to plot the dy/dx of the actual test data and that of an ideal test. The x used here is the number of blows as a percentage of the total number of blows for the DCP test. The dy/dx is calculated as the change in y (depth) divided by the change in x. Making a plot of the dy/dx vs. depth of the actual test and the dy/dx of the ideal situation, gives the plot given in Figure 3.24. The plot can also be use to see increasing and decreasing stiffness of the soils being tested. If the slope is increasing, the soil is getting weaker and if the slope is decreasing, the soil is getting stiffer. This can be used to estimate layer changes and know whether the layer is stiffer of weaker.

Table 3.13 shows the mean differences of 15 tests. The mean of the average differences is 3.167. The standard deviation of the data set is 0.624. If the maximum mean difference is set to be the average from the test strips plus one standard deviation, taking this data set to be a test strip data set, then the maximum mean value of the average difference would be 3.79. This would be the number used to control the uniformity of the embankment

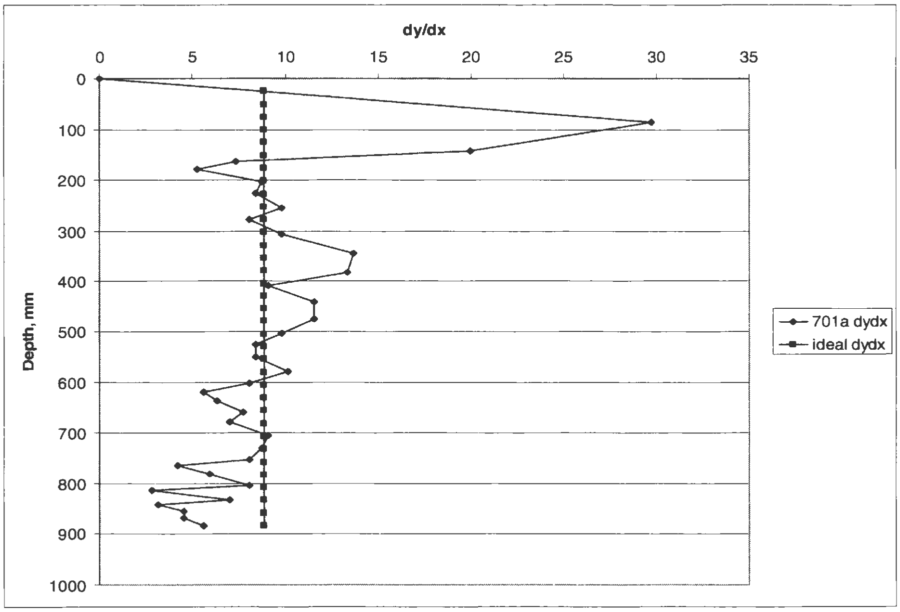

Figure 3. 24 Uniformity using dy/dx

Table 3. 13 Average differences of 15 tests

Test #	Mean difference
701a	2.938
701b	4.057
701c	3.611
701d	3.747
701e	3.281
701aa	4.425
701bb	2.812
701cc	2.970
701dd	2.720
701ee	3.148
701aaa	2.793
701bbb	2.907
701ccc	2.897
701ddd	2.690
701eee	2.505

CHAPTER 4: Geotechnical Remote Acquisition of Data System (G-RAD) as an optimal data collection and analysis tool

Researchers at Iowa state university have developed a PDA software that can be used improve and increase the efficiency of the DCP. It is called Geotechnical Remote Acquisition of Data System (G-RAD). G-RAD is a compilation of data collection and processing programs that can be placed on a pocket pc to use in field data collection and processing. G-RAD has also got supporting desktop spreadsheets that can be used at an office.

Figure 4. 1 G-RAD system with GPS attachment on a Dell Pocket PC

G-RAD consists of the following programs, G-RAD, a DCP data collection and processing program, G-Control, a program for collection of DCPI and UDCPI, Density, moisture, and lift thickness in the field. A GPS attachment to the pocket pc takes the program further by giving it capabilities to collect GPS coordinates for the field locations of the test sites. Figure 4.1 shows a pocket PC with the program with the GPS attachment. Another program which is part of the G-RAD, Area Calculator, takes advantage of the GPS and can be used to calculate the area of a section on a jobsite using the GPS coordinates and furthermore, given the lift thickness, it can calculate the volume of material placed and compacted.

The research team instrumental in developing the software included Dr. David White, Dr. Edward Jaselskis, Dr Russell Walters, Jianzhong zhang and Joels Malama. Jianzhong Zhang wrote the program code for G-RAD for the Pocket-PC. Joels developed the excel spreadsheets that were used to produce the program G-Control and also worked to trouble shoot the whole program.

G-RAD Instructions

These are the instructions of how to use G-RAD for data collection and analysis of DCP field data and other field engineering parameters measured for quality control.

The instructions will show the user how to use G-RAD to improve the efficiency of DCP in its use for quality control. The instructions will help the user enter the data into the programs on a pocket pc and analyze the data to give profiles and control charts that can be used in the field for quality control using the DCP. The intended users for G-RAD are the same skill level that uses the DCP.

The instructions will show the user how to start the programs on the hand held device needed for the data collection and analysis. The user will be instructed on how to enter data, store it, and view the analysis results in the form of data, profiles and control charts. The user will also be instructed on how to capture GPS data to record the location of the test.

The equipment needed for using G-RAD is the handheld PC with the software installed and a GPS attachment. When manipulating the data at a desk computer, G-RAD can be used on desktop computer spreadsheets.

The instructions presented below refer to G-RAD system installed on the Hewlett Packard (HP) 2215 model pocket pc. Other pocket pc devices may have variations in their operations. Refer to the owner's manual of the pocket pc for further instructions. The HP 2215 has four input methods for entering information into the device including; typing and writing. The instructions for using G-RAD will be presented using the keyboard input method to type information by tapping on the keyboard. To activate the keyboard, tap the keyboard symbol at the bottom right hand side of the screen. Tap the area you would like to write in and use the keyboard to type.

4.1 G-RAD

G-RAD is the program that is used to collect DCP data and analyzing the data, to give the DCP index and the correlated CBR values of each blow, DCP index and CBR values the of the entire depth penetrated. It also gives the profile of the depth penetrated. The profile displays the weighted average DCP index of the entire profile and it also gives the mean change in DCP index (UDCPI) of the profile. The UDCPI is used as an indication of the uniformity of the soil tested.

1. Starting the program

   To start the program, tap the “Start” menu and choose “Programs”. Tap “G-RAD” from the programs menu to select the program.

2. Setup of the programs

   a. Once the program is started, the screen shown in Figure 4.2 appears. Select the hammer type you are using where the 4.6 kg hammer is for soft materials, and the 8 kg hammer is for stiffer materials. Refer to ASTM D6951 note 3 for details on hammer selection

Figure 4. 2 G-RAD Start Screen

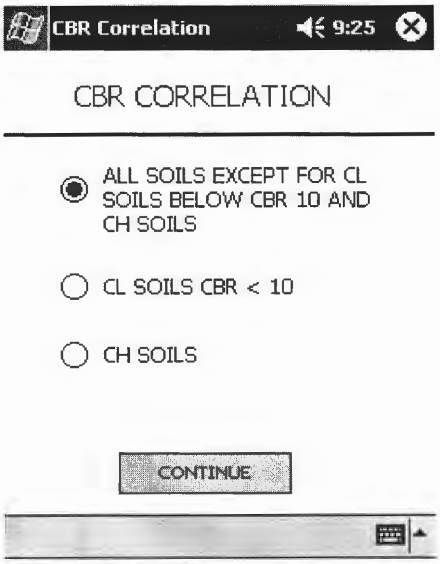

Figure 4. 3 CBR correlation screen

b. After the hammer type has been selected, tap "CONTINUE" and the screen in Figure 4.3 will appear. This screen is used to select the CBR correlation equation to be used. Refer to ASTM standard D6951-03 for details of soil type and the type of equation used for each soil.

c. After the CBR correlation has been chosen, tap "CONTINUE". The screen in Figure 4.4 will appear. At this point, if intending to use GPS, enable it now. If using G-RAD with G-Control do not enable GPS. To enable GPS, tap the "GPS" sub menu and select "Enable" from the menu. To disable GPS, tap the GPS submenu and select "Enable". A check mark appears when GPS is enabled.

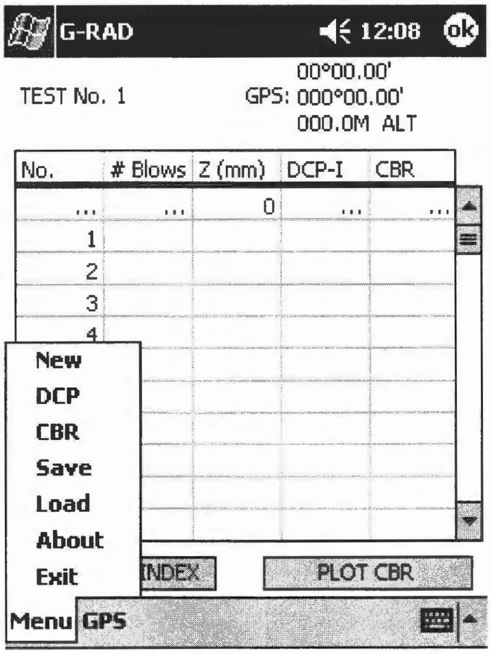

Figure 4. 4 Data entry screen

Data Collection

1. Data Entry and capture of GPS data

Once the keyboard has been activated, the user is ready to collect data. Tap the area next to the TEST No. to enter the id number of the test. Tap the box under the Z (mm) column to enter the initial reading. Tap the box in the #blows column to enter the number of blows and then tap the box next to it to enter the penetration depth. DCPI and its CBR correlation will appear in the DCP-I and CBR columns respectively. Continue to enter the data until the test is completed. If moisture content for the test location has been determined, enter it in the moisture content column. The GPS coordinates of the test location are captured once the data is saved. Figure 4.5 shows the data entry screen. Once data collection is complete, save data in a location of choice by tapping "Menu" and then tapping "Save".

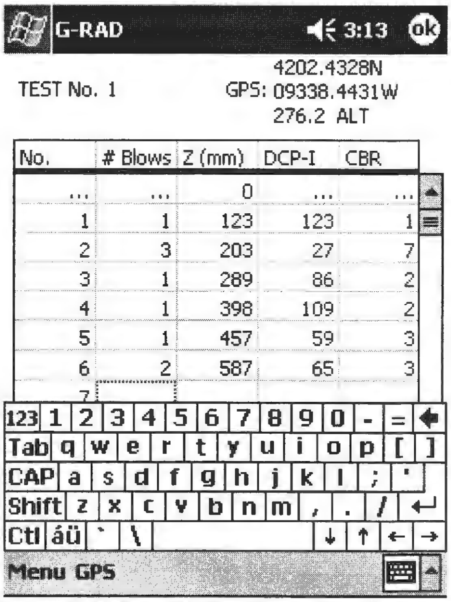

Figure 4. 5 Data Collection with Keyboard

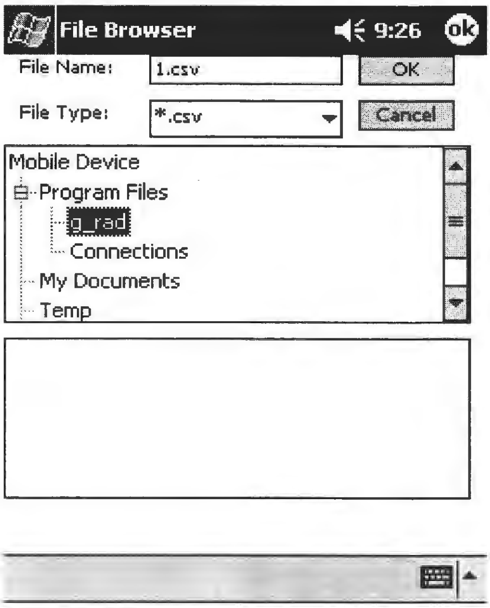

Figure 4. 6 Saving a file

Figure 4.6 shows the file save screen. Enter the file name and then tap "ok". To view a previous stored file, tap "Menu" and then "Load". Locate the folder in which the file is saved. Tap the file name to load the file.

2. Display of results

The DCP index and the corresponding CBR values for each penetration depth are shown automatically on the same screen. To view the profile of the test, tap "Menu" and then "DCP". The profile of the test will appear with the weighted average DCP index for the profile and the average change in DCP index displayed. Figure 4.7 shows an example DCP-I profile screen. The red line shows the average DCPI of the profile.

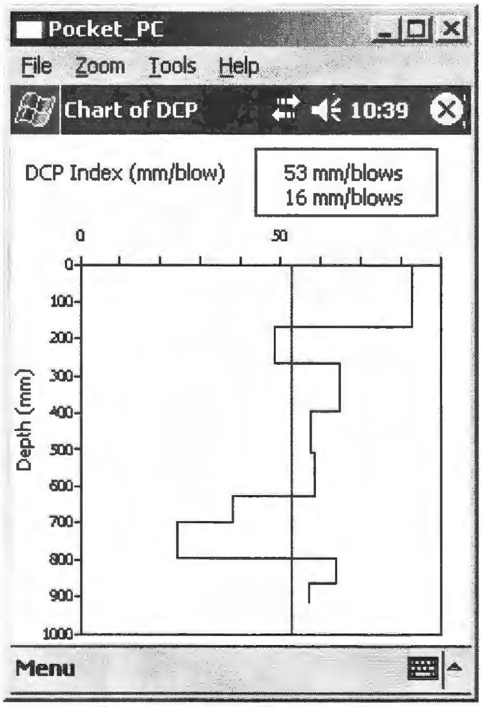

Figure 4. 7 DCP-I profile

To view the CBR profile, tap the Menu and then "CBR". Figure 4.8 shows the CBR profile with average CBR % and the average change in CBR. Alternatively, minimize the keyboard by tapping the keyboard symbol and then tap "PLOT DCP INDEX" to view the DCP index plot, and "PLOT CBR" for the CBR profile. When data collection is complete, tap "Menu" then select "Exit" or tap "OK" in the top right hand corner, save you work if you have not done so when prompted, otherwise tap "No" to exit.

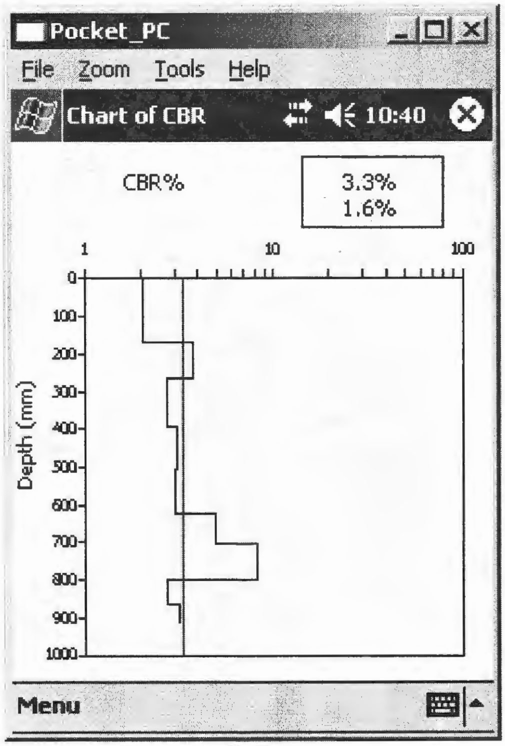

Figure 4. 8 CBR profile

4.2 G-CONTROL

G-Control is a program that can collect GPS coordinates, DCP index, UDCPI, moisture, density, and lift thickness for each test location. When a number of the tests results have been collected, control charts of each engineering parameter can be displayed to help the inspector make decisions for quality control. The data recorded can be saved for later viewing as well.

1. To start the program, tap the "Start" menu, from the "Start" menu, tap "Programs", and then from the program files, select the program "G-Control" by tapping on it. The first screen that comes up is the data capture and entry screen shown in figure 4.9.

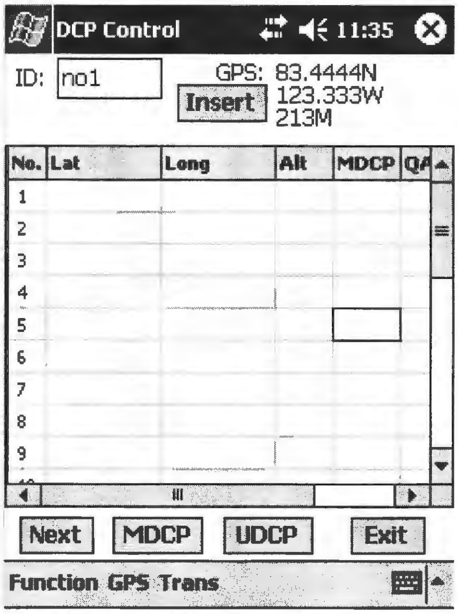

Figure 4. 9 G-Control screen

2. Once the program is started, tap the ID box to enter the name of the set of tests that are about to be collected. If using GPS to capture GPS coordinates fro the location, activate GPS, by tapping the "GPS" submenu and then tapping "Enable". To disable the GPS, tap "GPS" and then tap "Enable". A check mark next to "Enable" indicates that GPS is enabled.

3. Once the GPS has been activated, the program is ready for data collection. To insert GPS coordinates of the location being tested, tap the Row you want and then Tap "Insert" to insert the GPS coordinates for that location. Figure 4.10 shows the GPS coordinates entered in the rows. To enter the mean DCP index, UDCPI, moisture, density, lift thickness and the Quality Assurance (QA) values for each parameter, tap the respective box in the row then enter the data. The scroll bar at the bottom allows the user to reach the other parameters that may not be visible.

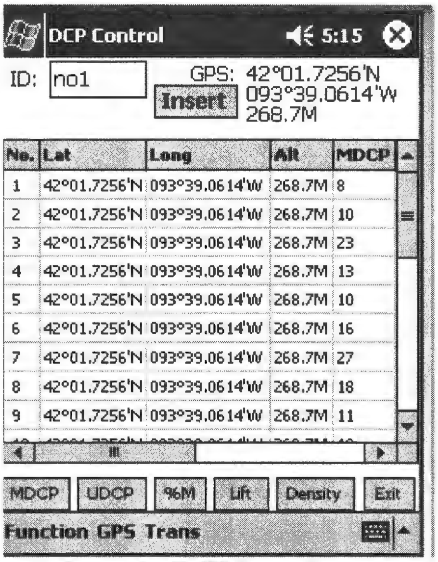

Figure 4. 10 GPS coordinates

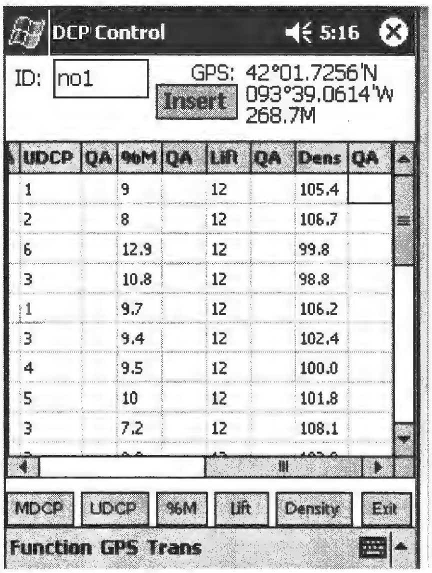

Figure 4. 11 G-Control screen continued

To save the data, tap "Function" and then tap "Save". Enter the file name and select the appropriate folder in which to store the data. Tap "ok" when finished, the file will be saved.

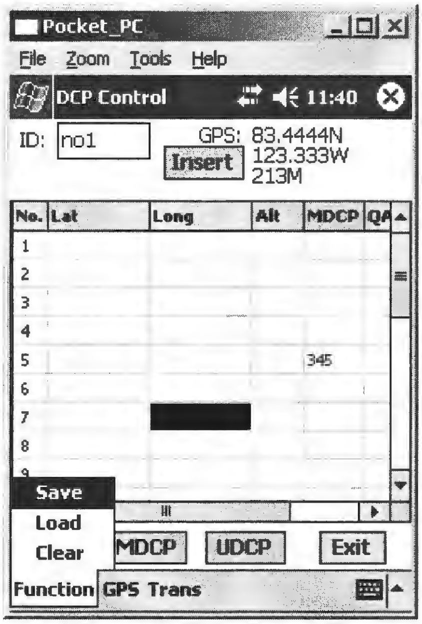

Figure 4. 12 Saving a File

Figure 4. 13 File name entry

4. To view the control charts of the parameters entered, tap the respective tab. Enter the requested control limits and tap okay, and then tap “Draw” to display the chart. For the mean DCP index control chart, enter the maximum DCP index for Unsuitable soils, Suitable soils and Select soils as defined by the Iowa DOT classification method (White and Bergeson, 2002). For the UDCP, enter the maximum average change for the Suitable

and Unsuitable soils and for the Select soils. For the moisture, enter the upper, Optimum and lower limits of moisture content. For the Density chart, enter the Maximum density and the Minimum percent (as a decimal, e.g. 0.95) compaction that is specified for the construction. Figure 4.14 shows the screen for entering the control limits for the MDCP for strength and stability and Figure 4.15 shows an example of the control chart for the MDCP. To view the data input screen, tap the "Back" button.

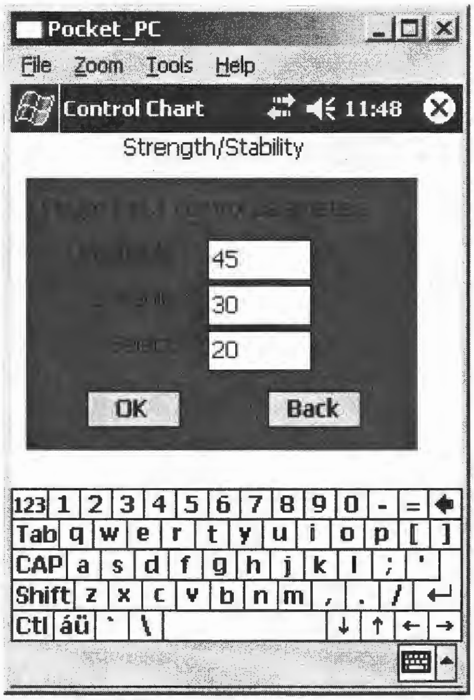

Figure 4. 14 Control limits entry for MDCP

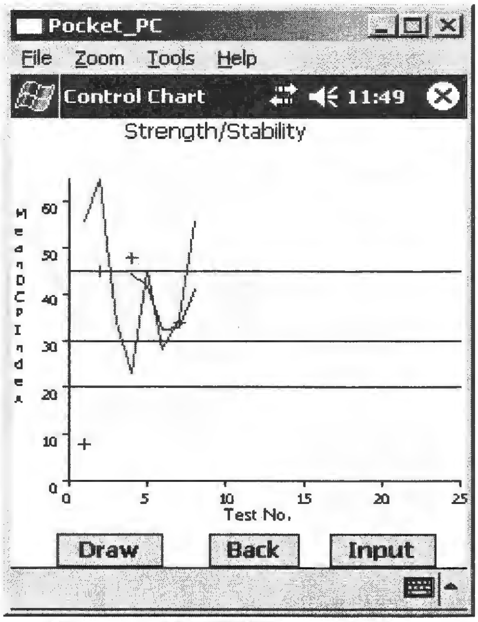

Figure 4. 15 MDCP control chart

5. To view a previous stored file, tap “Function” and then tap “Load”. Locate the folder in which the file is saved. Tap the file name to load the file. Figure 4.16 shows the file open screen.

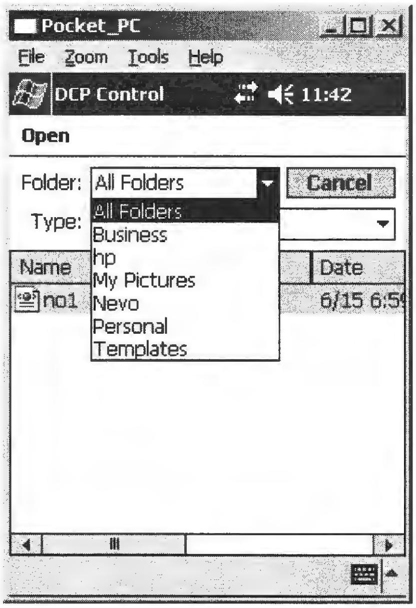

Figure 4. 16 Opening saved file

6. When Data collection is complete, Tap “Exit” at the bottom of the screen.

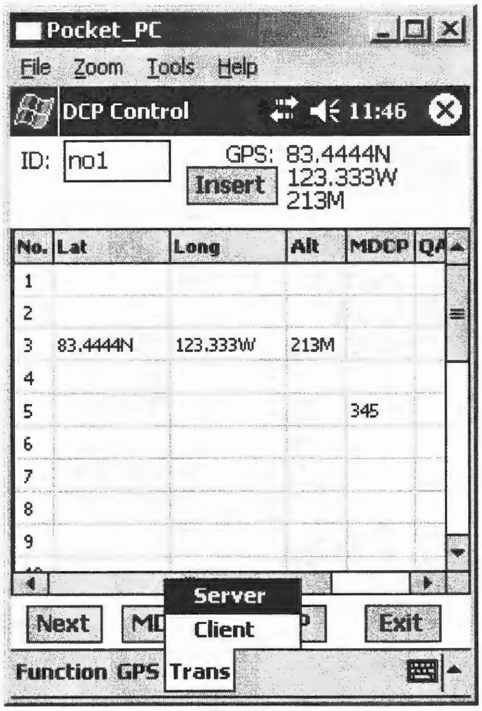

Figure 4. 17 Data transfer

7. Data can be transferred between two units or to a laptop computer equipped with Bluetooth technology. To transfer data between two PDA's, or to a laptop computer, Bluetooth must be activated and association between the two PDA's must be made (Refer to device instruction Manual for details on association). While G-Control is open tap "Trans" and then tap the name of the device to send data to. Once the Units recognize each other, data will be transferred. Save the received data. Figure 4.17 shows the data transfer screen.

4.3 AREA CALCULATOR

Area calculator is a program that uses GPS coordinates of corners of a given polygon taken in directional sequence, without crossing lines, to calculate the area of that polygon. To calculate an estimate of the volume of material moved, an average lift thickness can be added to calculate the volume. Figure 4.18 shows an example of a calculated area and Figure 4.19

shows the calculation of its volume with a lift thickness of 0.5m. This program is useful in estimating the number of tests that need to be performed based on the size of the area being tested and the volume of material being placed.

1. To access this program, from the "Start" menu, tap "Programs", and then from the program files, select the program "Area Calculator" by tapping on it.
2. Once the program is started, enable GPS from the "GPS menu". Enter a name in the ID box or leave the default if the name is not important. The GPS coordinates will change from 0's to actual coordinates when the GPS is ready.

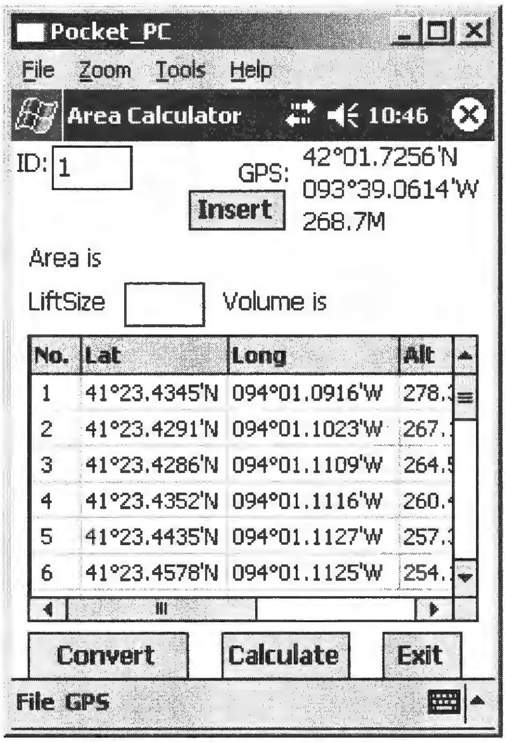

Figure 4. 18 Area Calculator

3. The program calculates the area by using the coordinates of the corners of the shape for which the area is being calculated. To obtain the coordinates the user walks around the perimeter of an imaginary polygon of the location stopping to capture the GPS coordinates of the corners of the polygon. To capture the GPS coordinates, tap a row, and

then tap “Insert” to capture the coordinates of that location. In the column labeled SP#, choose the correct state plane for the area (Iowa North is 1401 and Iowa south is 1402). Then tap “CONVERT” to convert the GPS coordinates to state plane (X-Y) coordinates. Continue to the next location and repeat the GPS capture and convert sequence as explained above. It is important to ensure that the coordinates are captured in sequence either a clockwise of anticlockwise, without crossing over. Random collection of GPS coordinates will give false and inaccurate area calculations.

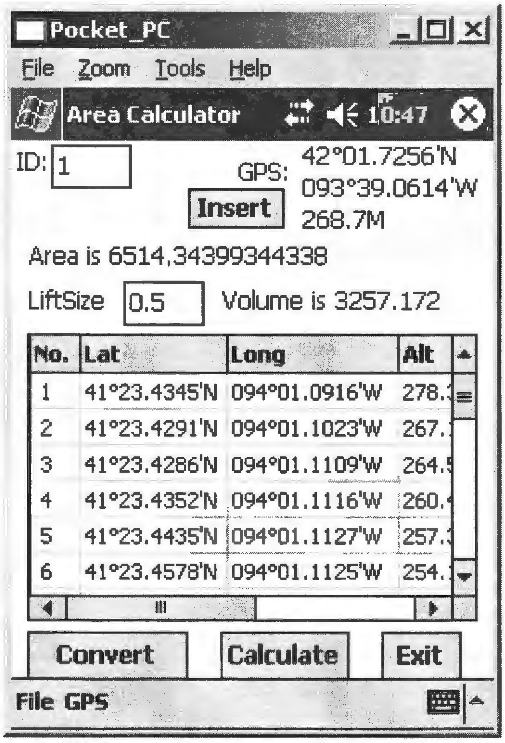

Figure 4. 19 Area calculator with volume calculation

4. Once data collection is complete, and the coordinates have been converted, tap “Calculate” for the Area to be calculated. To obtain the volume, enter lift size and then tap “Calculate”. The units of the area and volume calculated are square meters and cubic meters respectively.

5. To save the data, tap “File” then tap “Save” and then enter file name and select folder. Tap “OK” when done. To open a saved file, tap “File”, then tap “Load”. Locate the folder where the file is located and then tap the saved file to open it.
6. To exit out of the program, tap the Exit button

4.4 G-RAD SPREADSHEETS

In addition to the control charts produced on the pocket pc, regular pc version of G-Control was developed. Using a spread sheet program, for example Microsoft Excel, test data can be entered and control charts produced. This is a tool that can be used for quality control from the office. The spread sheets produce charts for DCP data, moisture data, Density data, and lift thickness data.

DCP DATA

The data entered into this spreadsheet is the GPS coordinates where available, the mean DCP data and the mean change in DCP from each test point. A moving average of the mean DCP data is then calculated. Control parameters are the maximum DCPI values for a required minimum strength required and the maximum change in mean DCP values to control the uniformity.

Figure 4.20 shows the spreadsheet for data entry of the mean DCP and the Control limits for mean DCP and for the change in mean DCP.   Clicking on the buttons labeled Strength and Uniformity produces control charts shown in figures 4.21 and 4.22. The charts produced can be used as visual aids in the decision making process for quality control.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
1																
2		Strength		Uniformity		QC	Four Point	QA				QC	Four Point	QA		
3	Soil					Mean	Moving Avg					Mean	Moving Avg		Suitable/	
4		Location	Latitude	Longitude	Elevation	dcp index	dcp index		Unsuitable	Suitable	select	change	change		Unsuitable	Select
5		0							95	85	75				40	35
6		1				17.2			95	85	75	6.5			40	35
7		2				13.9			95	85	75	8.8			40	35
8		3				28.0			95	85	75	5.8			40	35
9		4				29.3	22.1		95	85	75	5.3	6.6	5.8	40	35
10		5				28.4	24.9		95	85	75	3.9	5.9		40	35
11		6				19.8	26.4	24.0	95	85	75	3.5	4.6		40	35
12		7				12.4	22.5		95	85	75	2.9	3.9		40	35
13		8				13.0	18.4		95	85	75	3.9	3.5		40	35
14		9				13.7	14.8		95	85	75	2.2	3.1		40	35
15		10				12.9	13.0		95	85	75	3.7	3.2		40	35
16		11				11.6	12.8	4.0	95	85	75	2.9	3.2	5.2	40	35
17		12				14.5	13.2		95	85	75	4.3	3.3		40	35
18		13				13.9	13.2	11.0	95	85	75	2.9	3.5		40	35
19		14				15.0	13.8		95	85	75	3.4	3.4		40	35
20		15				15.2	14.7		95	85	75	3.7	3.6		40	35
21		16				14.0	14.5		95	85	75	2.3	3.1		40	35
22		17				12.1	14.1		95	85	75	1.7	2.8		40	35
23		18				12.4	13.4		95	85	75	2.0	2.4		40	35
24		19				11.5	12.5		95	85	75	2.5	2.1		40	35
25		20				13.3	12.3	48.0	95	85	75	3.3	2.4	5.6	40	35
26		21				14.0	12.8		95	85	75	2.7	2.6		40	35
27		22				15.4	13.6		95	85	75	5.7	3.5		40	35
28		23				14.3	14.3		95	85	75	6.8	4.6		40	35
29		24				14.1	14.5		95	85	75	2.0	4.3		40	35
30		25				28.3	18.0		95	85	75	12.1	6.6		40	35
31		28							95	85	75				40	35

Strength / Uniformity / Strength&uniformity / Sheet2 / Sheet3

Figure 4. 20 Data entry for Strength and Uniformity

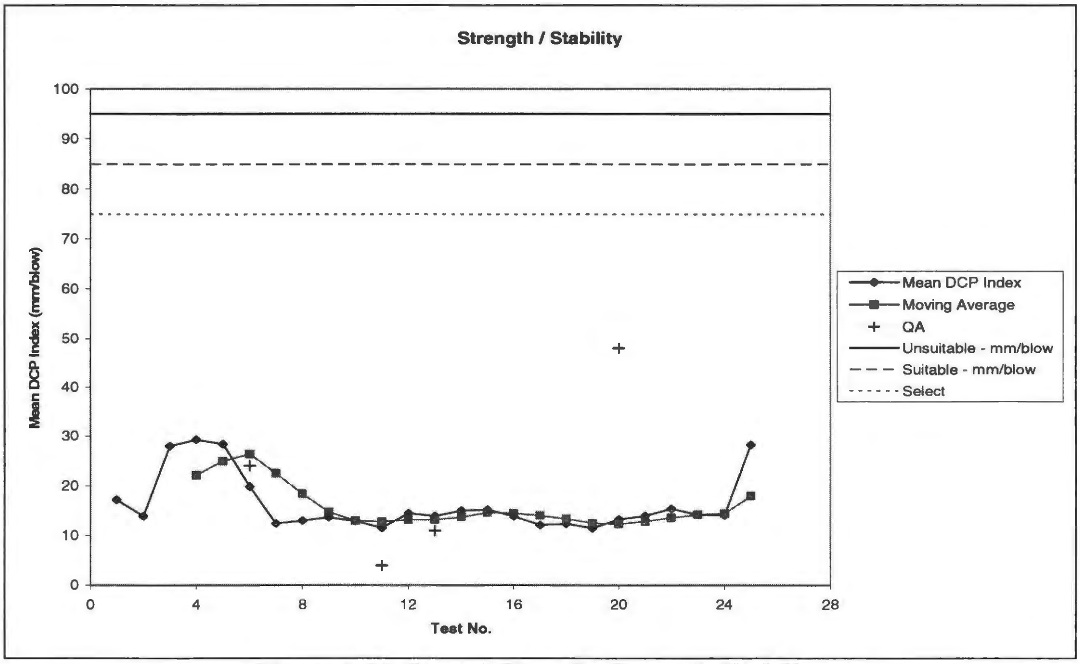

Figure 4. 21 Control Chart for Strength /Stability

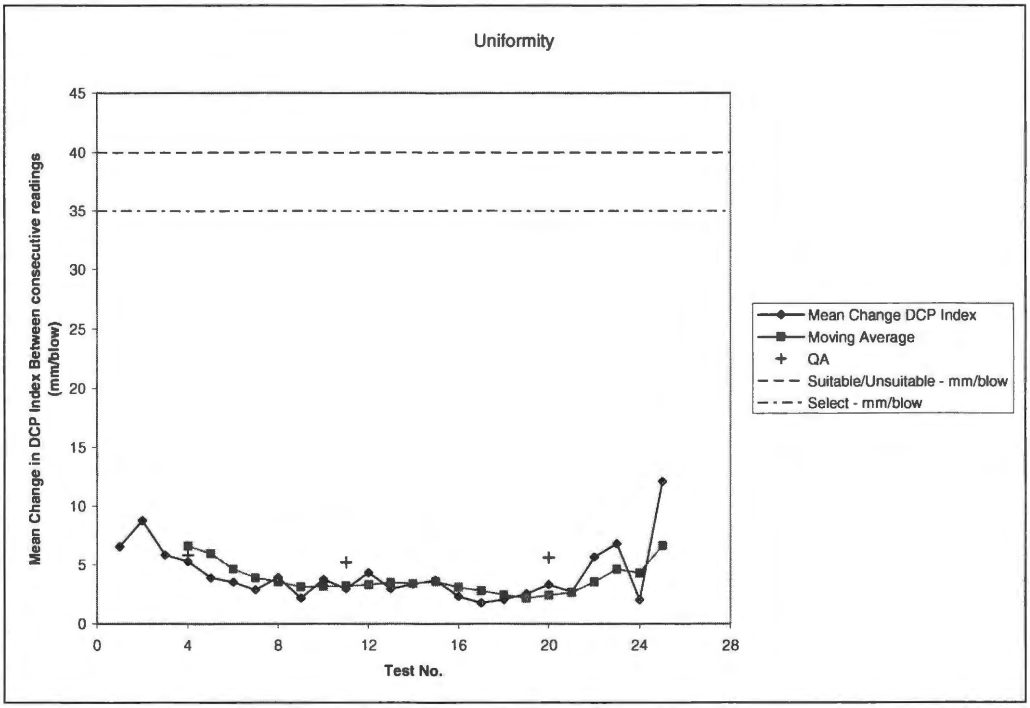

Figure 4. 22 Control Chart for uniformity

	A	B	C	D	E	F	G	H	I	J
1										
2					-	2	+	3		
3			QC	Four Point		Lower		Upper		
4				Moving Avg		Control	Optimum	Control		
5		Location	% M	%m	QA	Limit	%M	Limit		
6		0				14.3	16.3	19.3		Moisture Control
7		1	16.00			14.3	16.3	19.3		
8		2	17.40			14.3	16.3	19.3		
9		3	16.05			14.3	16.3	19.3		
10		4	16.05	16.38	17.55	14.3	16.3	19.3		
11		5	17.55	16.76		14.3	16.3	19.3		
12		6	16.10	16.44		14.3	16.3	19.3		
13		7	15.40	16.28		14.3	16.3	19.3		
14		8	14.80	15.96		14.3	16.3	19.3		
15		9	14.85	15.29		14.3	16.3	19.3		
16		10	13.40	14.61		14.3	16.3	19.3		
17		11	13.90	14.24		14.3	16.3	19.3		
18		12	15.55	14.43	18.50	14.3	16.3	19.3		
19		13	15.70	14.64		14.3	16.3	19.3		
20		14	16.40	15.39		14.3	16.3	19.3		
21		15	15.55	15.80		14.3	16.3	19.3		
22		16	15.50	15.79		14.3	16.3	19.3		
23		17	13.65	15.28		14.3	16.3	19.3		
24		18	13.65	14.59		14.3	16.3	19.3		
25		19	16.20	14.75		14.3	16.3	19.3		
26		20	15.85	14.84	14.10	14.3	16.3	19.3		
27		21	16.95	15.66		14.3	16.3	19.3		
28		22	14.85	15.96		14.3	16.3	19.3		
29		23	16.05	15.93		14.3	16.3	19.3		
30		24	13.90	15.44		14.3	16.3	19.3		
31		25	14.80	14.90		14.3	16.3	19.3		
32		26				14.3	16.3	19.3		

Sheet2 / Sheet3

Figure 4. 23 Data entry for moisture control

MOISTURE DATA

Data entered for this spreadsheet is the moisture content from each test point. From this data, a four point moving average is calculated. Control limits for the moisture content are then entered. Figure 4.23 shows the data entry spreadsheet with the control limits for the moisture content. Clicking the button labeled moisture control produces the control chart shown in Figure 4.24.

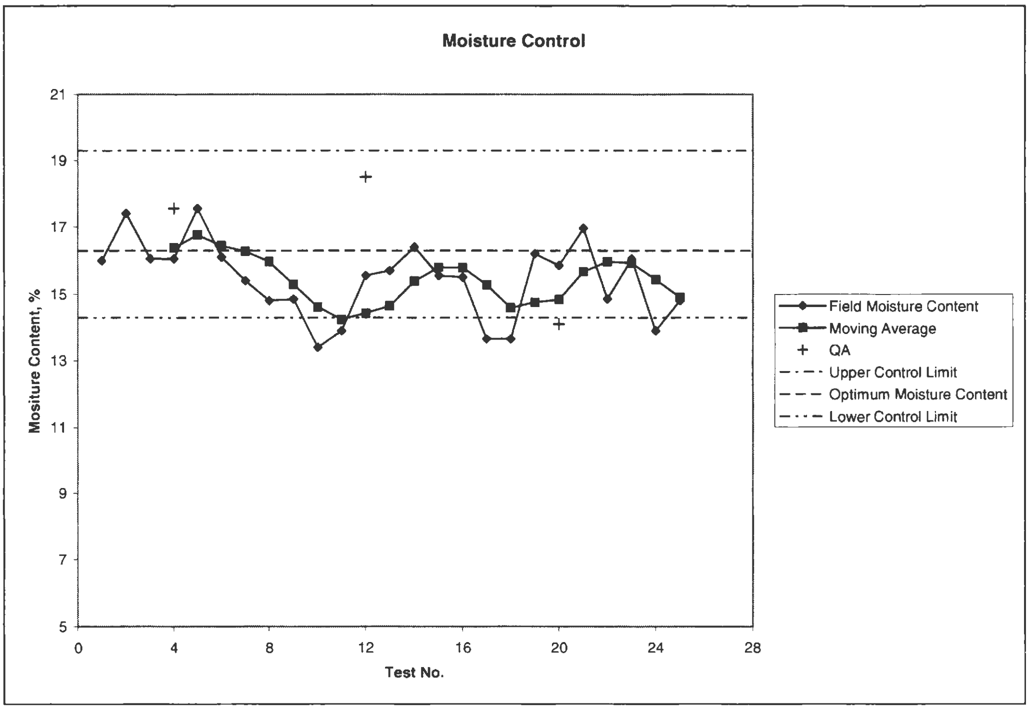

Figure 4. 24 Control chart for moisture content

DENSITY DATA

Data entered for this spreadsheet is the density from each test point. From this data, a four point moving average is calculated. Control limits entered for the density are; the maximum density from the proctor test for the soil tested and the minimum relative compaction required in percent. Figure 4.25 shows the data entry spreadsheet with the control limits for the density. Clicking the button labeled density control produces the control chart shown in Figure 4.26.

Control Rel. Compaction= 95 %

Maximum density = 112 lb/ft³

Density Control

Location	QC Dry Density lb/ft³	Four Point Moving Avg lb/ft³	QA	Minimum Density
0				106.4
1	103.0			106.4
2	104.0			106.4
3	107.0			106.4
4	105.0	104.8	107.5	106.4
5	108.0	106.0		106.4
6	105.6	106.4		106.4
7	106.8	106.4		106.4
8	105.9	106.6		106.4
9	102.3	105.2		106.4
10	107.0	105.5		106.4
11	110.5	106.4		106.4
12	111.2	107.8	108.5	106.4
13	106.8	108.9		106.4
14	109.0	109.4		106.4
15	106.8	108.5		106.4
16	104.0	106.7		106.4
17	106.5	106.6		106.4
18	110.5	107.0		106.4
19	105.0	106.5		106.4
20	105.9	107.0	104.1	106.4
21	107.0	107.1		106.4
22	104.9	105.7		106.4
23	106.1	106.0		106.4
24	110.9	107.2		106.4

Chart2 input Sheet3

Figure 4. 25 Data entry for Density Control

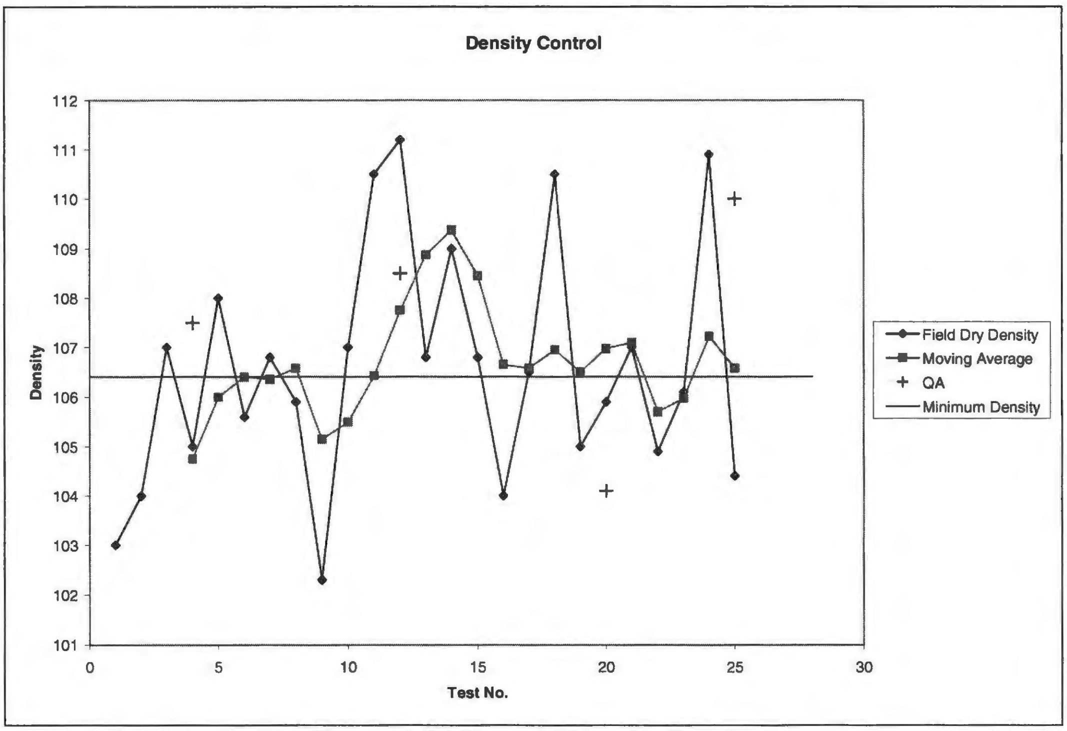

Figure 4. 26 Control Chart for Density

LIFT THICKNESS DATA

Data entered for this spreadsheet is the lift thickness from each test point. From this data, a four point moving average is calculated. Figure 4.27 shows the data entry spreadsheet for the lift thickness. Clicking the button labeled lift thickness produces the control chart shown in Figure 4.28.

LiftThickness

Location	Lift (cm)	4-point Moving Avg	QA
0			
1	15.20		
2	20.30		
3	22.80		
4	12.70	17.75	12.75
5	15.20	17.75	
6	12.70	15.85	
7	10.20	12.70	
8	15.20	13.33	
9	12.20	12.58	
10	12.70	12.58	
11	30.50	17.65	
12	22.80	19.55	20.30
13	10.10	19.03	
14	12.70	19.03	
15	15.20	15.20	
16	25.40	15.85	
17	20.30	18.40	15.20
18	20.30	20.30	
19	15.20	20.30	
20	22.80	19.65	
21	12.70	17.75	
22	20.30	17.75	17.50
23	10.10	16.48	
24	20.30	15.85	
25	17.80	17.13	

LiftThickness / Sheet1 / Sheet2 / Sheet3

Figure 4. 27 Lift thickness Entry screen

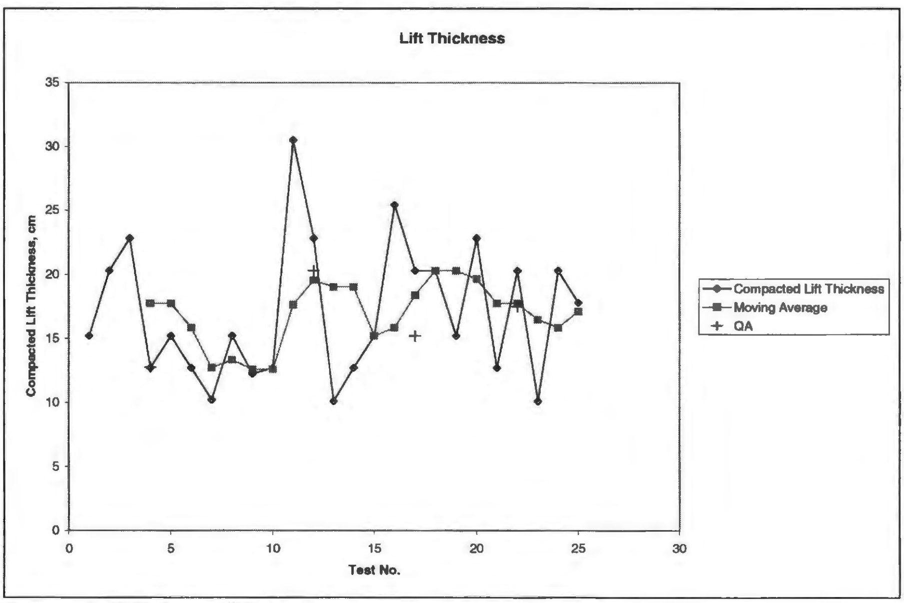

Figure 4. 28 Plot of lift thickness

CHAPTER 5: FIELD OBSERVATIONS

5.1 Introduction

To conduct field observations, seven highway construction projects around Iowa where visited; these sites were typically characterized by unsuitable soils. Field visits took place between June 2003 and August 2004. Additional testing was performed at two Caterpillar equipment demonstration projects. Tests performed at project sites included: moisture-density tests, DCP tests, Clegg Impact hammer tests, and Geogauge tests. Material from the sites was also collected for laboratory testing. Laboratory tests performed included Standard Proctor tests (ASTM 698) to define optimum moisture content and maximum dry unit weight, soil classification tests, which included percent finer than No. 200 sieve and Atterberg limit tests.

Figure 5. 1 G-RAD system with GPS attachment

Figure 5.1 shows the newly developed Geotechnical Remote Acquisition of Data (G-RAD) system on a Dell® PDA with a Pharos GPS unit attached. During the field site visits G-RAD was incorporated to collect field data and GPS coordinates at field test locations. G-

RAD is a spread-sheet system on a hand held computer with a Global Positioning System (GPS) attachment. Using the G-RAD system, data needs to be recorded only once, and files can be easily transferred between PDA's or to an email account. Chapter 4 provided a detailed description of G-RAD, including instructions for how to use G-RAD.

Table 5.1 summarizes the soil index properties from the referenced project sites. The table documents the soil classification using the Iowa DOT classification system for soil placement during construction as established by White and Bergeson (2002). The classification system organizes soils into categories; select, suitable and unsuitable, based on the soil's physical properties.

Table 5. 1 Summary of soil Index properties

Project No.	Material Number	LL	PL	PI	F_{200}	AASHTO	USCS	EPC
1	P1-A	27.0	16.22	10.78	54.36	A-6	CL	Select
	P1-B	29.3	13.33	15.97	55.66	A-6	CL	Select
2	P2-A	34.7	18.42	16.28	90.99	A-6	CL	Select
3	P3-A	39.9	24.24	15.66	97.25	A-6	CL	Suitable
	P3-B	68.8	21.4	47.40	88.31	A-7-6	CH	Unsuitable
4	P4-A	69.6	25.85	43.75	91.60	A-7-6	CH	Unsuitable
5	P5-A	33.6	25.09	8.51	99.26	A-4	CL	Unsuitable
6	P6-A	38.8	23.5	15.3	94.78	A-6	CL	Suitable
7	P7-A	34.3	14.6	19.7	60.10	A-6	CL	Select
8	P8-A	29	23	6	97.1	A-4	ML	Unsuitable
	P8-B	24	15	9	52.3	A-4	CL	Suitable
	P8-C	35	24	11	90.1	A-6	CL	Unsuitable
9	P9-A	29	16	13	68.9	A-6	CL	Suitable
10	P10-A	42	32	10	98.4	A-5	CL	Unsuitable
	P10-B	49	30	19	97.2	A-7-5	CL	Unsuitable

Quality control methods, discussed in chapters 3, are applied in this chapter. The quality control methods are applied and compared on the results of field tests from projects where DCP testing was performed.

5.2 Project No. 1: Highway 34 - Batavia By-pass

Field tests were conducted at this site on June 18th, 2003. The site was part of the Batavia bypass construction on highway 34 in Jefferson county Iowa. Our objective at the site was to observe construction and conduct some tests using nuclear density gauge. Unit weight and moisture content of the material was obtained at seven spots including the borough pits. Material for laboratory testing was also collected from the site. Table 5.2 below notes the results of these tests. Figures 5.2 and 5.3 shows proctor curves for the material obtained from field with the field results included on the plot.

Table 5. 2 Field Data from project 1

	Field Data		Lab results		Comparison	
Test point	Unit Weight (kN/m^3)	Water Content % M	Max Density (kN/m^3)	Water Content % M	%compaction	%M range
18-Jun	17.8	14.6	18.7	11.9	95.1	2.7
18-Jun	17.5	18.8	18.7	11.9	93.7	6.9
18-Jun	18.1	14.6	18.8	10.8	96.1	3.8
18-Jun	18.3	14.6	18.8	10.8	97.1	3.8

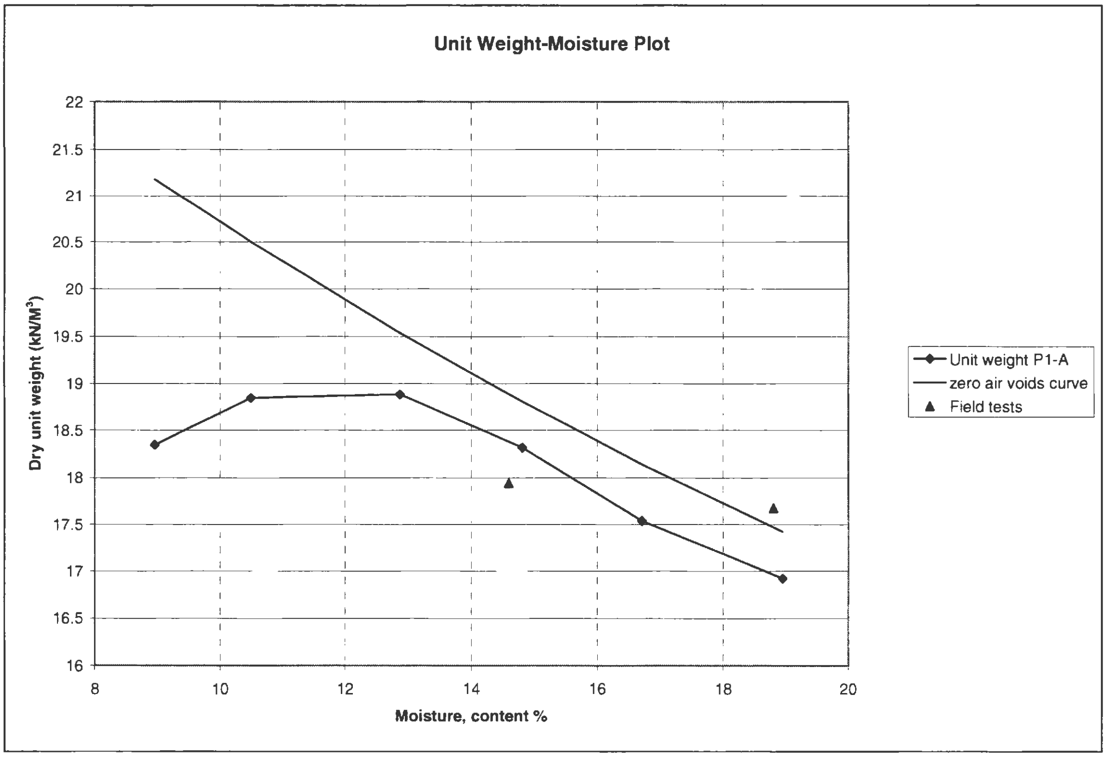

Figure 5. 2 Proctor P1-A from highway 34

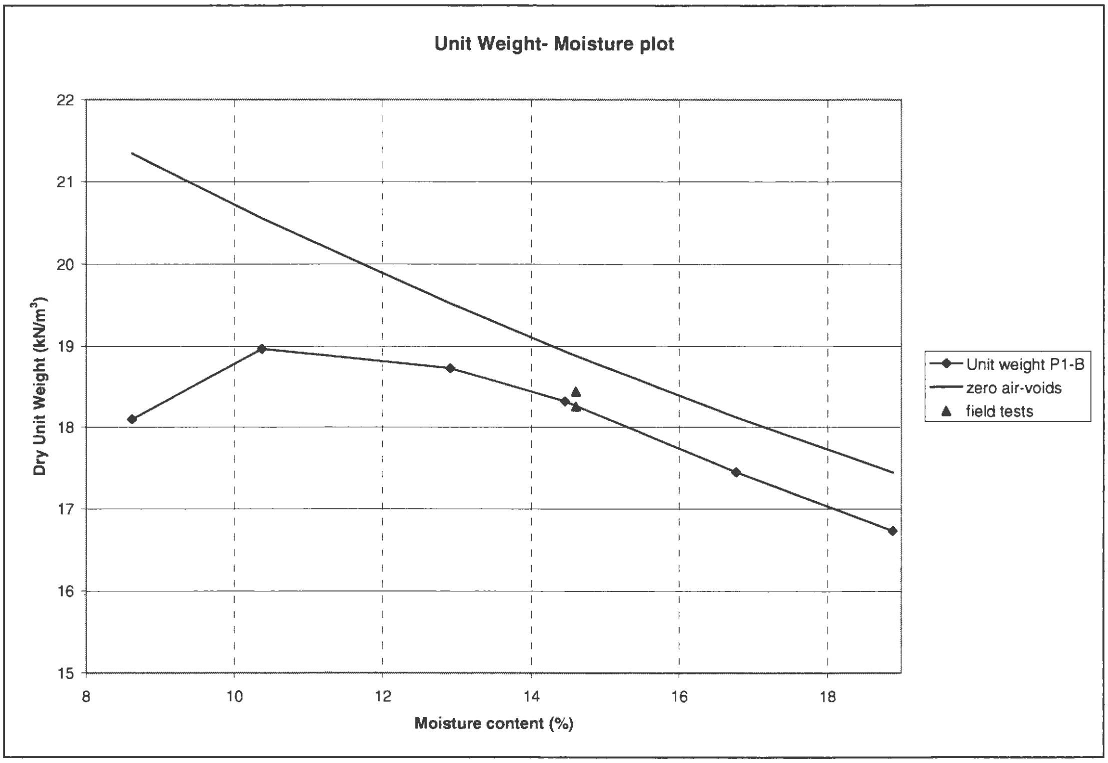

Figure 5. 3 Proctor P1- B from Highway 34

At the engineered borough pit, the site operated two excavators: a John Deere 450C LC and a Hitachi Ex 450 LC. The soil was hauled by Volvo A40 trucks and Caterpillar D400D trucks. Construction engineers operated a Caterpillar 140G grader and a Caterpillar D7H Bulldozer. For compaction, a sheepsfoot roller was used, pulled by a 7110 international tractor, as shown in Figure 5.4.

Figure 5. 4 Tractor pulled sheep's foot roller

The recommended lift size is 203.2 mm (8 in), with one roller pass per 25.4 mm (1 in); however, from observation, the lift thickness varied from 304.8 mm (12 in) to about 508 mm (20 in). The number of passes was inconsistent and varied from 4 to 25 passes. As indicated by the moisture density plots in Figures 5.2 and 5.3, all of the field tests fall on the wet side of optimum. The average relative compaction and moisture content at this project was 95.5% and 4.3 % above optimum, respectively.

5.3 Project No. 2: Highway 218 - South of Mt Pleasant

Field tests were conducted at this site on June 18th, 2003. The project was part of the expansion of highway 218 south of Mt. Pleasant in Henry County, Iowa. Our objective on this project was to continue the observation construction practices.

At this project site, the soil was hauled from the borough sites by Caterpillar scrapers. A bulldozer was used to level the fill material before a tractor, pulling a sheepsfoot roller, was used to compact the soil.

Table 5. 3 Field Data from Project 2

	Field Data		Lab results		Comparison	
Test point	Unit Weight (kN/m^3)	Water Content % M	Max Density (kN/m^3)	Water Content % M	%compaction	%M range
P2a	14.7	26.4	16.8	17.0	87.1	9.4
P2b	15.4	22.1	16.8	17.0	91.4	5.1
P2c	15.4	21.5	16.8	17.0	91.3	4.5

Three randomly selected spots were targeted for performing the tests. Performed at the site were moisture and density tests using a nuclear density gauge. Representative samples of soil were obtained for laboratory testing. Tests results are reported in Table 5.3. Figure 5.5 documents the moisture density relation as indicated by the results of the field tests.

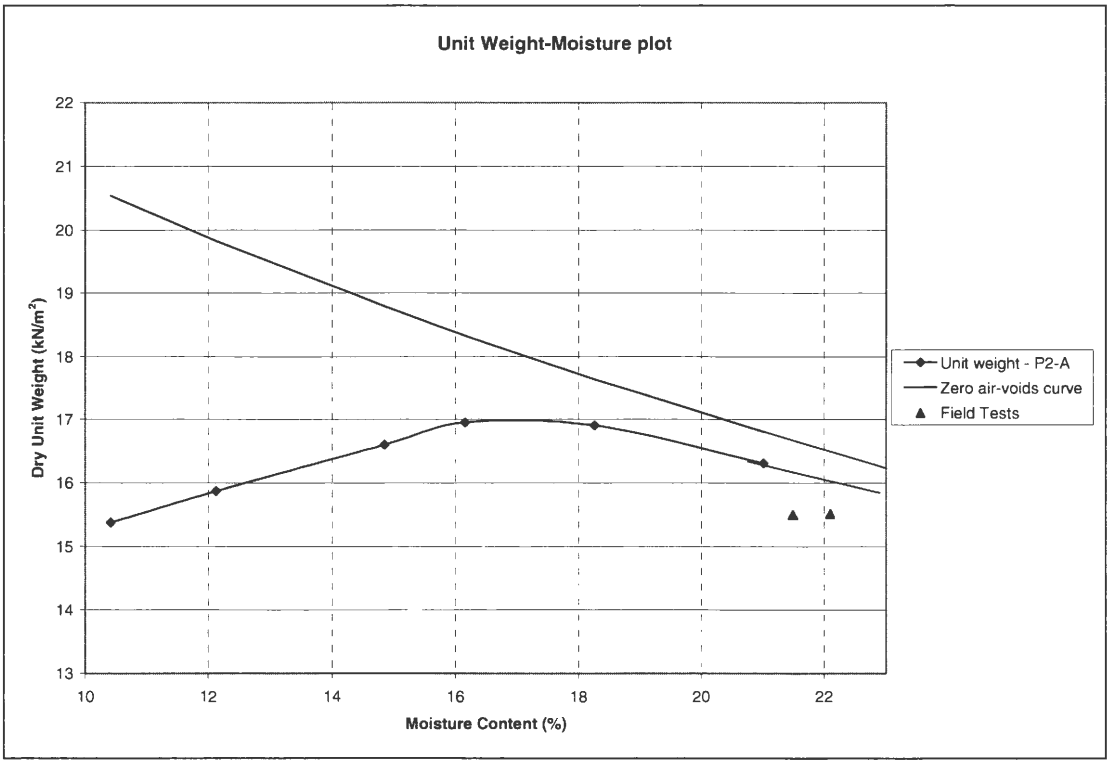

Figure 5. 5 Proctor P2 -A from Highway 218

The soil was specified to be compacted to roller walk out. Roller walk out was not accomplished, and the roller operator was instructed to move to a different site. As a result, there was inconsistency in the roller passes. Figure 5.5, documenting the field results, indicate that the soil was placed wet of the optimum moisture content. The soil index properties are listed in Table 5.1. The average relative compaction and moisture content at this project was 89.9% and 6.3 % above optimum, respectively.

5.4 Project No. 3: Highway 34 - West of Fairfield

This project was visited on June 25, 2003 and July 2, 2003. The project is part of the expansion project of Highway 34. The section tested is west of Fairfield in Jefferson County, Iowa. Our objective at this project was to observe construction practices and to perform tests, including the dynamic cone penetrometer (DCP) to measure quality control of the construction.

Five random spots were targeted for testing. Density testing and moisture content testing were performed using a nuclear gauge, and DCP testing was performed at the spots as well. Two sets of tests were performed at the five spots; each test set was conducted after the roller operator finished rolling the strip, before the next lift was placed. Representative samples of the material were collected for laboratory testing.

The soil was hauled by Scrapers. The site operated a Caterpillar D7H Bulldozer. The sheepsfoot roller was pulled by a 7110 International tractor.

Figure 5. 6 Performing DCP a test

Figure 5.6 shows DCP testing in progress and Figure 5.7 shows a profile of the test. Figure 5.8 shows the Proctor curve with field tests performed on site at various locations. DCP test results are available in Table 5.4.

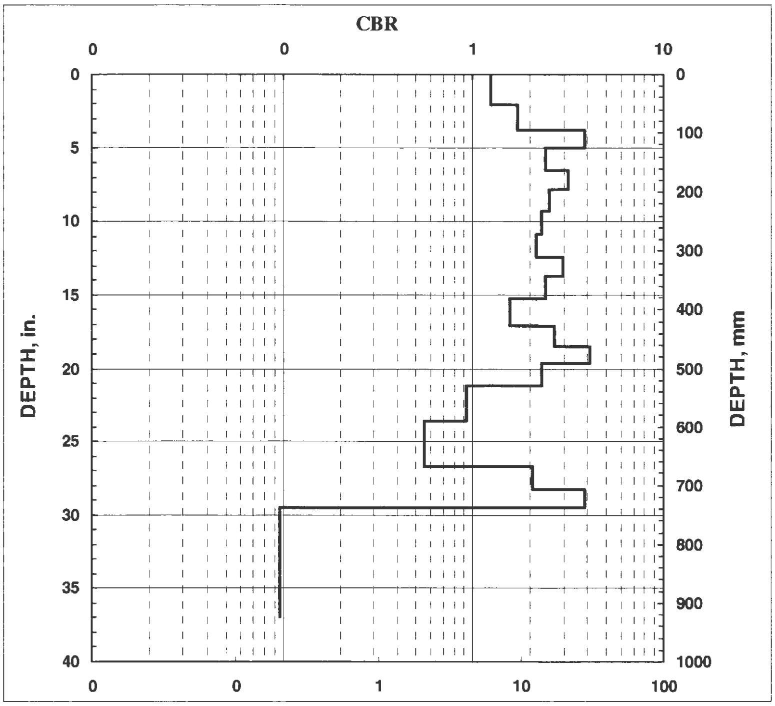

Figure 5. 7 DCP profile from Project 3

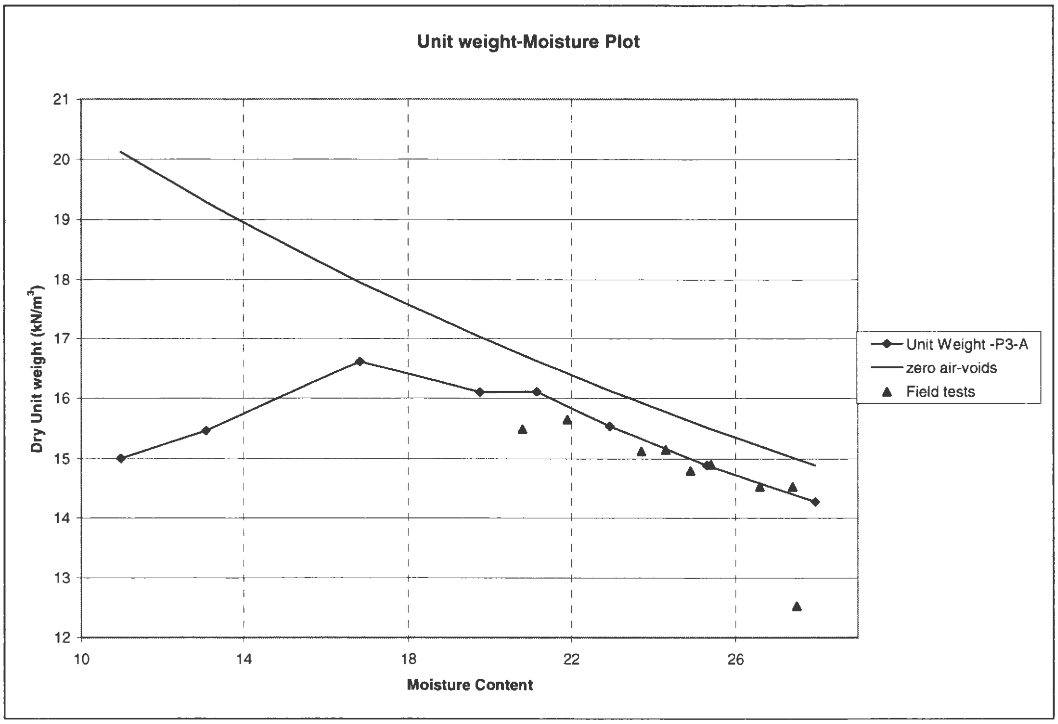

Figure 5. 8 Proctor P3-A from Highway 34

Table 5. 4 Field Data from Project 3

Test point	Field Data: Unit Weight (kN/m^3)	Field Data: Water Content % M	Lab results: Max Density (kN/m^3)	Lab results: Water Content % M	Comparison: %compaction	Comparison: %M range	DCP Data: MDCPI (mm/blow)	DCP Data: UMDCPI (mm/blow)
25-Jun	15.6	21.90	16.6	17.1	94.2	4.8	74	41.0
25-Jun	14.9	25.40	16.6	17.1	89.7	8.3		
25-Jun	15.1	23.70	16.6	17.1	91.0	6.6		
25-Jun	15.2	24.30	16.6	17.1	91.2	7.2		

The recommended lift size was 203 mm (8 in) with one roller pass per 25.4 mm (1 in); however, from observation, the lift thickness varied from 304.8 mm (12 in) to about 500.8 mm (20 in). It was also noted that most of the field tests were on the wet side of the optimum moisture content.

As previously mentioned, the project was revisited on July 2, 2003. The site featured the same equipment from the first visit. Figure 5.9 shows the Proctor of the second material that was collected on the second trip. The soil falls on the dry side of the optimum moisture content the soil.

Table 5. 5 Field Data from Second visit of project 3

Test point	Field Data: Unit Weight (kN/m^3)	Field Data: Water Content % M	Lab results: Max Density (kN/m^3)	Lab results: Water Content % M	Comparison: %compaction	Comparison: %M range	DCP Data: MDCPI (mm/blow)	DCP Data: UMDCPI (mm/blow)
702a	14.4	29.40	14.6	25.3	98.2	4.1	33	6
702b	15.4	24.10	14.6	25.3	105.3	-1.2	47	16
702c	15.5	23.00	14.6	25.3	106.1	-2.3	67	18
702d	15.6	22.90	16.6	17.1	93.7	5.8	40	7
702e	14.2	23.50	16.6	17.1	85.8	6.4	43	8
702aa	15.5	22.50	14.6	25.3	105.9	-2.8	46	16
702bb	15.1	23.40	14.6	25.3	102.9	-1.9	42	9
702cc	15.5	23.40	14.6	25.3	105.8	-1.9	34	9
702dd	15.7	21.90	14.6	25.3	107.4	-3.4	38	7
702ee	15.8	22.10	14.6	25.3	108.1	-3.2	33	7

The average relative compaction and moisture content at this project was 90.9% and 6.5 % above optimum, respectively with a mean DCP index of 71 (CBR of 0.6) for the June 25th soil and 105% and -1.6 % below optimum, respectively, with a mean DCP index range of 43 to 67 (CBR range of 0.8 to 15) for the July 2nd soil. Table 5.5 documents the field results from the tests. Table 5.6 identifies results when applying quality control criteria.

Table 5. 6 Quality Control Comparison, project 3

Soil type	Unsuitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			95	**40**	**132**	**185**	**32**
702a	33	6	Pass	Pass	Pass	Pass	Fail
702b	47	16	Pass	Pass	Pass	Pass	Fail
702c	67	18	Pass	Pass	Pass	Pass	Fail
702d	40	7	Pass	Pass	Pass	Pass	Fail
702e	43	8	Pass	Pass	Pass	Pass	Fail
702aa	46	16	Pass	Pass	Pass	Pass	Fail
702bb	42	9	Pass	Pass	Pass	Pass	Fail
702cc	34	9	Pass	Pass	Pass	Pass	Fail
702dd	38	7	Pass	Pass	Pass	Pass	Fail
702ee	33	7	Pass	Pass	Pass	Pass	Fail

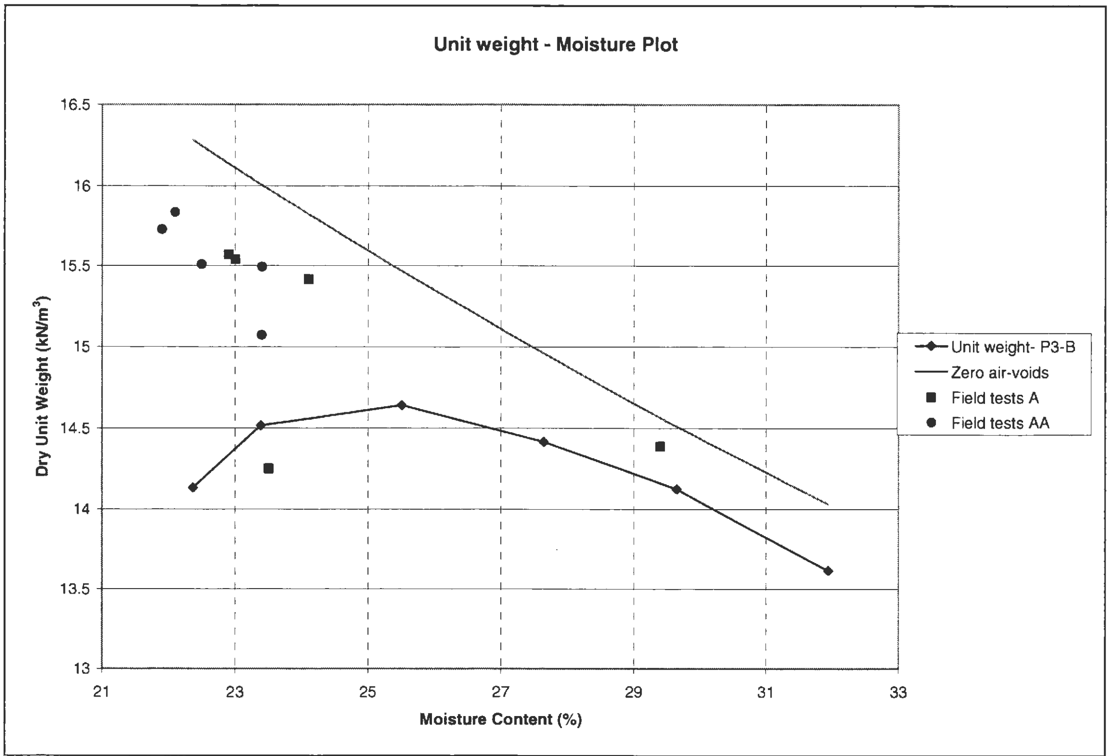

Figure 5. 9 Proctor P3-B from Highway 34

The Atterberg test results of the soil collected from this project suggest that the soil has swell potential. To check the swelling potential of the soil, swell potential tests were conducted at the Iowa State University engineering laboratory. The swell potential testing was performed on sample P3-B at 20.4, 20.9, 23.4, 24.4, 25.9, and 27.6 % moisture content. The tests were performed in accordance with ASTM D 4829, Standard Test Method for

Expansion Index of Soils. Figure 5.10 documents the swell potential results plotted with standard Proctor data. According to the ASTM 4829, as shown in Table 5.7, the swell potential of this soil is medium at worst and very low at best.

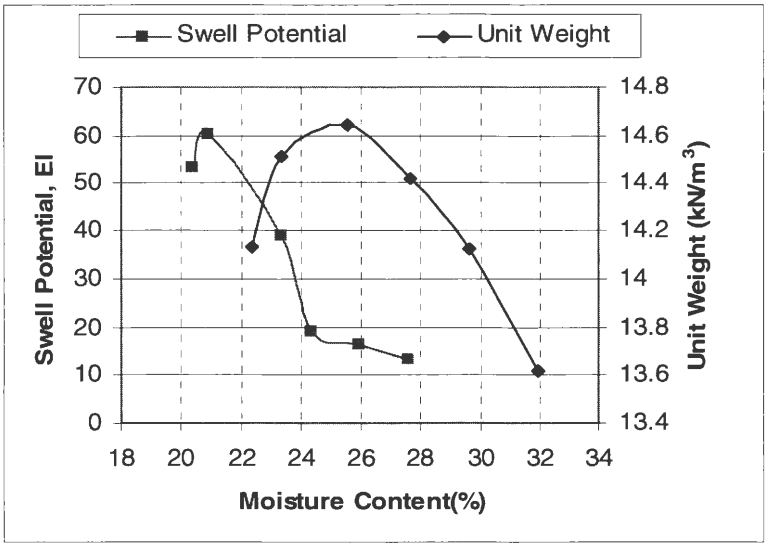

Figure 5. 10 Swell Potential of the soil P3-B

Table 5. 7 Expansion index

Expansion Index, EI	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

5.6 Project No. 4: Highway 218 - South of Mt. Pleasant by Salem Road

The project was visited on July 1 2003. The project is part of the expansion of highway 218 at Salem road south of Mt. Pleasant, in Henry County, Iowa. The objective of the project was to observe the construction practices and to perform field tests.

Five random spots were selected for testing at the site. Density testing and moisture content testing were performed using a nuclear gauge, and DCP testing was performed at the spots as well. Three sets of tests were performed at the five spots; each test set was completed after the roller operator finished rolling the strip, before the next lift was placed. Representative samples of the material were collected for laboratory testing. The results of the tests are documented in Table 5.8. Table 5.9 presents the quality control application on the DCPI data. Figure 5.11 documents a Proctor of the material taken from the site with field tests that were performed at different lifts.

Table 5. 8 Field data from Project 4

	Field Data		Lab results		Comparison		DCP Data	
Test point	Unit Weight (kN/m^3)	Water Content % M	Max Density (kN/m^3)	Water Content % M	%compaction	%M range	MDCPI (mm/blow)	UMDCPI (mm/blow)
701a	14.4	28.70	15.1	24.0	95.2	4.7	33	7
701b	14.4	24.20	15.1	24.0	95.4	0.2	45	10
701c	16.3	20.50	15.1	24.0	108.1	-3.5	23	4
701d	14.6	28.20	15.1	24.0	96.5	4.2	33	5
701e	15.4	24.80	15.1	24.0	101.7	0.8	31	4
701aa	14.1	26.60	15.1	24.0	93.7	2.6	33	5
701bb	14.0	28.10	15.1	24.0	92.8	4.1	29	5
701cc	15.1	25.60	15.1	24.0	99.8	1.6	38	8
701dd	14.4	22.10	15.1	24.0	95.7	-1.9	39	9
701ee	13.9	27.40	15.1	24.0	92.1	3.4	28	7
701aaa	15.3	23.60	15.1	24.0	101.4	-0.4	34	6
701bbb	15.7	21.60	15.1	24.0	103.8	-2.4	30	5
701ccc	16.1	19.40	15.1	24.0	106.5	-4.6	30	5
701ddd	16.4	20.50	15.1	24.0	108.6	-3.5	44	10
701eee	15.5	20.80	15.1	24.0	102.4	-3.2	27	4

Table 5. 9 Quality Control comparison, Project 4

Soil type	Unsuitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			95	**40**	**132**	**185**	**32**
701a	33	7	Pass	Pass	Pass	Pass	Fail
701b	45	10	Pass	Pass	Pass	Pass	Fail
701c	23	4	Pass	Pass	Pass	Pass	Pass
701d	33	5	Pass	Pass	Pass	Pass	Fail
701e	31	4	Pass	Pass	Pass	Pass	Pass
701aa	33	5	Pass	Pass	Pass	Pass	Fail
701bb	29	5	Pass	Pass	Pass	Pass	Pass
701cc	38	8	Pass	Pass	Pass	Pass	Fail
701dd	39	9	Pass	Pass	Pass	Pass	Fail
701ee	28	7	Pass	Pass	Pass	Pass	Pass
701aaa	34	6	Pass	Pass	Pass	Pass	Fail
701bbb	30	5	Pass	Pass	Pass	Pass	Pass
701ccc	30	5	Pass	Pass	Pass	Pass	Pass
701ddd	44	10	Pass	Pass	Pass	Pass	Fail
701eee	27	4	Pass	Pass	Pass	Pass	Pass

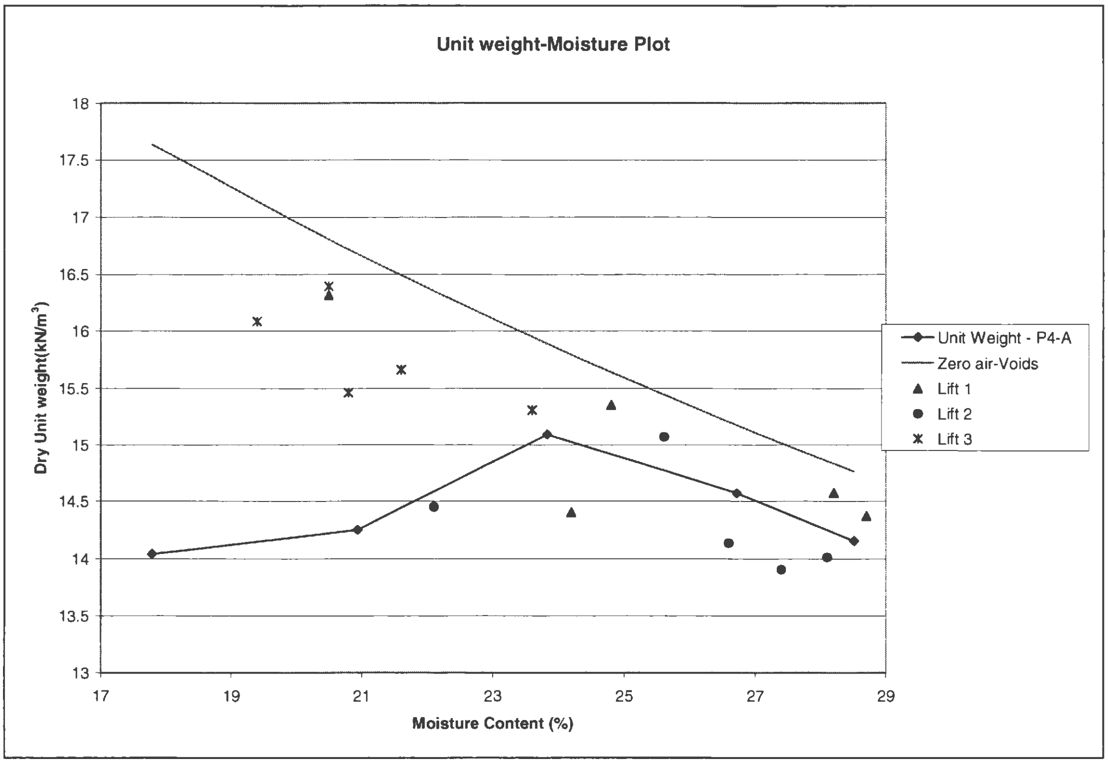

Figure 5. 11 Proctor P4-A from Highway 218

Figure 5. 12 Scraper hauling soil

Figure 5. 13 Tractor pulled sheepsfoot roller

The material was hauled by Caterpillar scrapers. A bulldozer leveled the material before it was compacted by a tractor pulled roller. Figure 5.12 and Figure 5.13 feature the equipment that was used on site.

Strength testing was performed on sample P4-A at 18.6, 20.1, 22, 23.6, 25.7, 26.3, and 31.8 % moisture content. The tests were performed in accordance with ASTM D 2166, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. The samples used were molded using the ISU 2"x 2" method. Figure 5.14 documents the strength moisture relationship with the unit weight.

This project also used the roller walk out specification; therefore neither moisture content nor density were evaluated using measures for quality control. Observations from the results reveal a scatter of moisture content and density, ranging from 19% 29% and 93% to 106% relative compaction.

The variation of moisture content caused differential settlement thereby resulting in rutting of pavements. Strength tests performed on this soils revealed high soaked strength close to the optimum moisture content; however, strength decreased sharply as the moisture content increased.

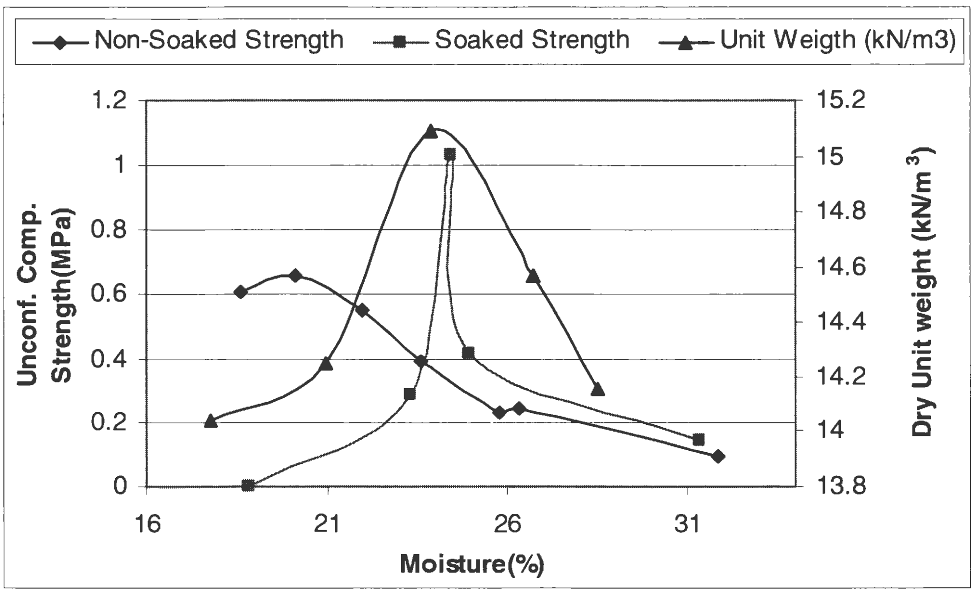

Figure 5. 14 Strength of the soil P4-A with Density

Figure 5.14 plots the variation of strength (soaked and non-soaked) as it relates to moisture content. It should be noted that at 18.08%, upon being introduced to water, the sample collapsed, whereas the samples with highest strength from the non-soaked strength tests were characterized by the same moisture content. Similarly, material placed dry of optimum was characterized by high strength but ultimately lost the strength when it was saturated.

5.7 Project No. 5: Exit Ramp of Highway 275 at I-29

The project was visited on two days, July 21st 2003 and July 22nd 2003. The project featured the construction of the embankment for the exit ramp of highway 275 at Interstate 29 in Council Bluffs, Iowa. My role in this project was to observe construction practices and to perform field tests after the material was compacted.

The tests that were performed at the site were moisture and density tests using a nuclear gauge and DCP testing. Testing was performed on two days. On the first day, three lifts were tested with six test points on the first two lifts and two tests on the third lift. On the second day, two lifts were tests with six tests spots on the first and five on the second lift. Representative samples of the soil were taken for laboratory testing. The results of the field tests are documented in Table 5.10. Figure 5.15 below shows the Proctor curve of the soil with data points from the field tests.

The recommended lift thickness was 203 mm (8 in) and one roller pass per one inch of lift thickness. The lift thickness was not measured, nor was the roller pattern followed. The quality control method used on this project was roller walk out. Tables 5.11and 5.12 presents the quality control application on the DCPI data.

Table 5. 10 Field data from project 5

Test point	Field Data: Unit Weight (kN/m^3)	Field Data: Water Content % M	Lab results: Max Density (kN/m^3)	Lab results: Water Content % M	Comparison: %compaction	Comparison: %M range	DCP Data: MDCPI (mm/blow)	DCP Data: UMDCPI (mm/blow)
721a	17.2	16.00	16.5	18.2	104.5	-2.2	19	7
721b	15.7	17.40	16.5	18.2	95.6	-0.8	17	7
721c	16.7	16.05	16.5	18.2	101.3	-2.2	16	5
721d	16.5	16.05	16.5	18.2	100.1	-2.2	17	5
721e	14.7	17.55	16.5	18.2	89.1	-0.6	16	4
721f	16.4	16.10	16.5	18.2	99.9	-2.1	21	3
721aa	16.6	15.40	16.5	18.2	101.0	-2.8	13	2
721bb	16.9	14.80	16.5	18.2	102.8	-3.4	14	4
721cc	17.5	14.85	16.5	18.2	106.1	-3.4	14	2
721dd	17.3	13.40	16.5	18.2	105.1	-4.8	13	3
721ee	18.0	13.90	16.5	18.2	109.2	-4.3	12	3
721ff	17.0	15.55	16.5	18.2	103.2	-2.7	15	3
721aaa	16.2	15.70	16.5	18.2	98.4	-2.5	14	2
721bbb	16.2	16.40	16.5	18.2	98.5	-1.8	17	3
722a	16.4	15.55	16.5	18.2	99.7	-2.7	15	3
722b	16.7	15.50	16.5	18.2	101.5	-2.7	14	2
722c	16.4	13.65	16.5	18.2	99.5	-4.6	12	2
722d	16.7	13.65	16.5	18.2	101.3	-4.6	12	2
722e	17.3	16.20	16.5	18.2	105.2	-2.0	12	3
722f	16.8	15.85	16.5	18.2	102.3	-2.4	13	3
722aa	15.7	16.95	16.5	18.2	95.6	-1.3	14	2
722bb	16.5	14.85	16.5	18.2	100.4	-3.4	18	6
722cc	16.8	16.05	16.5	18.2	101.8	-2.2	15	4
721dd	17.7	11.90	16.5	18.2	107.3	-6.3	15	2
722ee	17.1	14.80	16.5	18.2	104.0	-3.4	16	5

Table 5. 11 Quality Control comparison, Project 5

Soil type	Unsuitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI **95**	UDCPI **40**	M_R **132**	Stability **185**	CBR 6% **32**
721a	19	7	Pass	Pass	Pass	Pass	Pass
721b	17	7	Pass	Pass	Pass	Pass	Pass
721c	16	5	Pass	Pass	Pass	Pass	Pass
721d	17	5	Pass	Pass	Pass	Pass	Pass
721e	16	4	Pass	Pass	Pass	Pass	Pass
721f	21	3	Pass	Pass	Pass	Pass	Pass
721aa	13	2	Pass	Pass	Pass	Pass	Pass
721bb	14	4	Pass	Pass	Pass	Pass	Pass
721cc	14	2	Pass	Pass	Pass	Pass	Pass
721dd	13	3	Pass	Pass	Pass	Pass	Pass
721ee	12	3	Pass	Pass	Pass	Pass	Pass
721ff	15	3	Pass	Pass	Pass	Pass	Pass
721aaa	14	2	Pass	Pass	Pass	Pass	Pass
721bbb	17	3	Pass	Pass	Pass	Pass	Pass

Table 5. 12 Quality Control comparison, Project 5

Soil type	Unsuitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			95	**40**	**132**	**185**	**32**
722a	15	3	Pass	Pass	Pass	Pass	Pass
722b	14	2	Pass	Pass	Pass	Pass	Pass
722c	12	2	Pass	Pass	Pass	Pass	Pass
722d	12	2	Pass	Pass	Pass	Pass	Pass
722e	12	3	Pass	Pass	Pass	Pass	Pass
722f	13	3	Pass	Pass	Pass	Pass	Pass
722aa	14	2	Pass	Pass	Pass	Pass	Pass
722bb	18	6	Pass	Pass	Pass	Pass	Pass
722cc	15	4	Pass	Pass	Pass	Pass	Pass
721dd	15	2	Pass	Pass	Pass	Pass	Pass
722ee	16	5	Pass	Pass	Pass	Pass	Pass

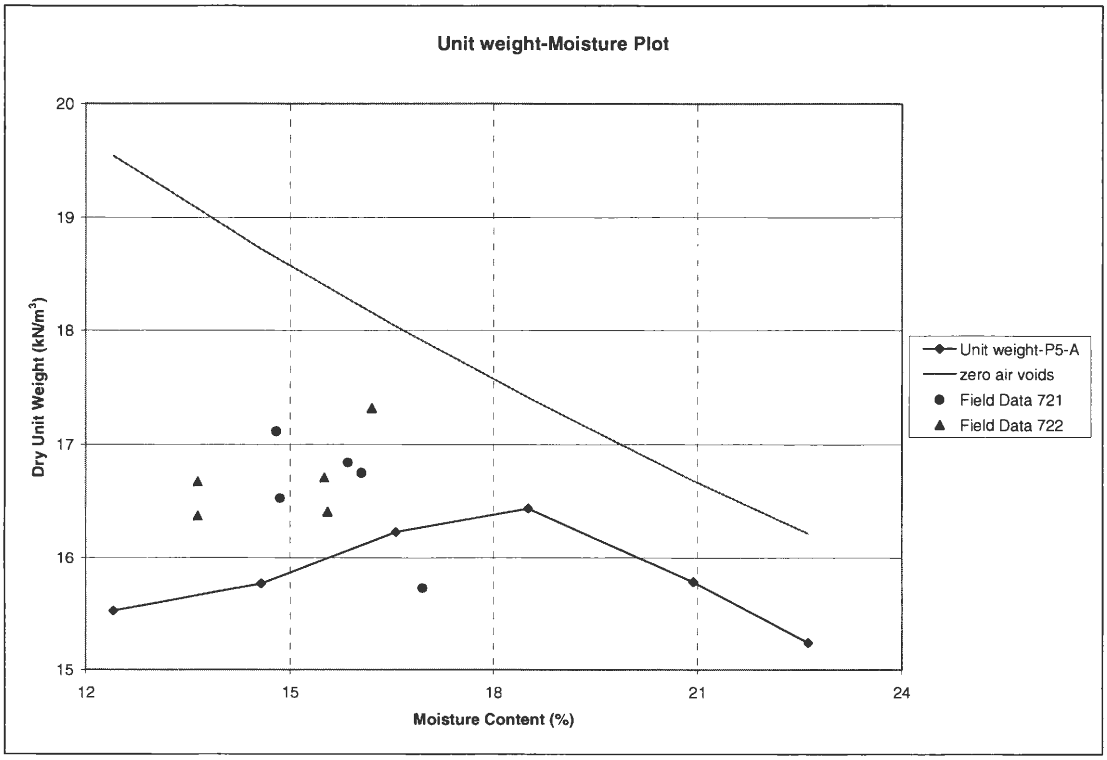

Figure 5. 15 Proctor P5-A from the Ramp at Highway 275 and I-29

Some of the material used on the embankment was hauled by a scraper, a Caterpillar 627 (Figure 5.16a), from a stock pile while the rest of the material was hauled from a loess borough site by side dump trucks. A D4C dozer was used to level the material before two

(a)

(b)

(c)

Figure 5. 16 Equipment used on the project (a) Scraper (b) Compactor (c) Tractor pulled roller

compactors rolled over the material. One of the compactors was the Caterpillar 816B (Figure 5.16b), and the other was a tractor-pulled sheep's foot roller (Figure 5.16c).

The quality control for this project was based on roller walk out. Measurements of moisture and density of the site revealed that the soil was dry of the optimum moisture content, while the relative compaction ranged from 95 to 109% with 1 point at 89%. Lift thickness was observed from DCP profiles ranging from 150 to 300mm (6 to 12in), whereas the specification was 203 mm (8 in) loose material. From the profiles, it can also be observed that the Oreo cookie effect was present in the layers.

The material in this area is loess. With introduction of either heavy loads or moisture, loess is susceptible to collapse. In this case, both these situations are expected. Loads from the pavement and traffic will be placed on the soil. The pavement cuts off the route for water to evaporate, so the soil will eventually be saturated. Collapse potential tests were performed to measure the susceptibility of the loess to collapse under pressure of loads and saturation.

Collapse potential testing was performed on sample P5-A in accordance with Single Oedometer method. The method is as follows;

1. Place an undisturbed soil sample in an oedometer and maintain the in-situ moisture content.
2. Apply a seating load of 100 lb/ft^2 and zero the dial gage
3. Increase the vertical stress in increments, allowing soil to consolidate with each increment. Normally the load may be changed when the rate of consolidation becomes less than 0.1% per hour. Continue this process until the vertical stress is equal to, or slightly higher than, that which will occur in the field.

4. Inundate the soil sample and monitor the resulting hydrocompression. This is the potential hydrocollapse strain e_w, for this over-burden stress.

Once the hydroconsolidation has ceased, apply an additional stress increment and allow the soil to consolidate.

Figure 5.17 plots the collapse potential in relation to moisture content with standard Proctor data. Instead of using the in-situ moisture content, the tests used moisture contents of 9.9, 12.1, 14.8, 18.2, 19.1, and 22.3 %.

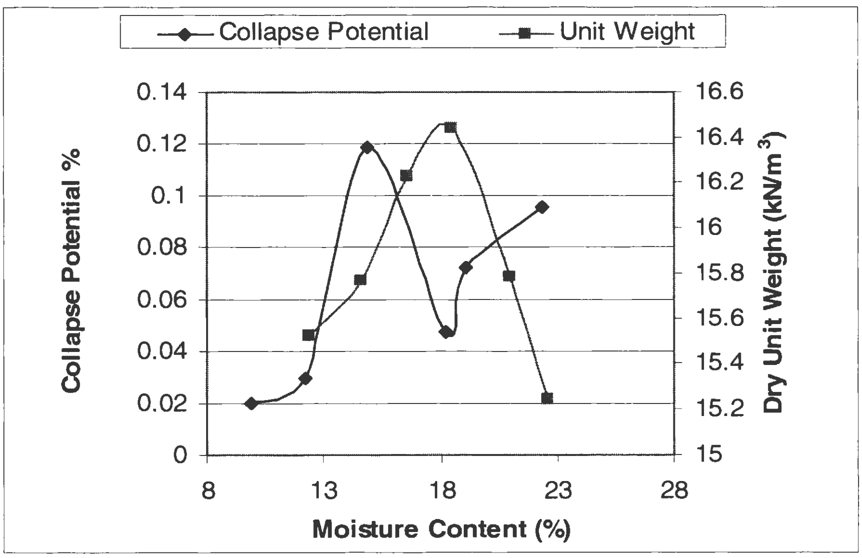

Figure 5. 17 Collapse Potential of Soil P5-A with unit weight Plot

Table 5. 13 Collapse Potential

Degree of Collapse	Collapse Potential %
None	0
Slight	0.1 - 2.0
Moderate	2.1 - 6.0
Moderately Severe	6.1 - 10
Severe	> 10

The field vertical stress was calculated by taking into account the 68.95 kPa (10 psi) for the pavement and traffic on the ramp and calculating the soil stress at 3m (10 ft). The sum of these factors—pavement and traffic plus the weight of the soil—will be used as the field vertical stress. This gives a field vertical stress of approximately 117.2 kPa (17psi). As indicated by Table 5.13, the results from the single oedometer test reveal that, at most, the potential for collapse was minimal.

5.8 Project No. 6: Highway IA 2 - Sydney Bypass

This project site was visited on June 1st 2004. The project is part of the Iowa highway 2 Sydney bypass east of Sydney in Fremont County, Iowa. The aim of the site visit was to perform several in-situ tests including moisture tests, density tests, and DCP tests. The moisture and density tests were performed using a nuclear gauge.

Fifteen spots were randomly selected for testing. Testing was performed after the roller operator finished compacting soil in each layer, before the site was ready for the next layer of soil to be placed. The results of the field tests are documented in Table 5.14.

Table 5. 14 Field Data form Project 6

No.	Station	Dry Density kN/m^3	%Comp.	%M	MDCPI (mm/blow)	UMDCPI (mm/blow)
1	41+ 00, 10'RT	14.2	**86.9**	22.5	58	20
2	41+ 25, 10'RT	15.9	**97.7**	20.7	43	12
3	41+ 50, 10'RT	15.4	**94.4**	22.09	48	11
4	41+ 75, 10'RT	16.3	**99.7**	17.9	42	9
5	42+ 00, 10'RT	15.6	**95.5**	24.9	44	10
6	42+ 25, 10'RT	16.2	**99.1**	19.2	61	24
7	42+ 25, 20'RT	15.9	**97.2**	20.2	56	14
8	42+ 00, 20'RT	15.3	**93.9**	23.1	67	9
9	41+ 75, 10'RT	15.0	**91.7**	23.3	72	13
10	41+ 50, 10'RT	15.5	**95.0**	20	58	13
11	41+ 25, 10'RT	15.8	**97.0**	18.4	139	39
12	41+ 00, 10'RT	14.9	**91.5**	24.8	64	17
13	40+ 75, 10'RT	15.5	**94.8**	22.5	62	17
14	40+ 50, 10'RT	15.1	**92.7**	23.4	68	15
15	40+ 25, 10'RT	15.3	**94.0**	23.8	51	12

The area tested was a fill area with material transported by dump truck from a cut area several hundreds of meters away. A scraper was used to level the material once it was dumped, and then a tractor-pulled sheepsfoot roller compacted the material.

Material samples from the project site were collected for lab tests that included unit weight-moisture relationship, plasticity index, and sieve analysis. The plasticity index and the sieve analysis were performed for soil classification. Figure 5.18 plots the unit weight-moisture relationship. The maximum unit weight is 16.32 kN/m^3 given at a moisture content of 18.5 %. This stage of the project did not incorporate measures to monitor moisture or density control. Relative compaction ranged from 86.9% to 99.7%.

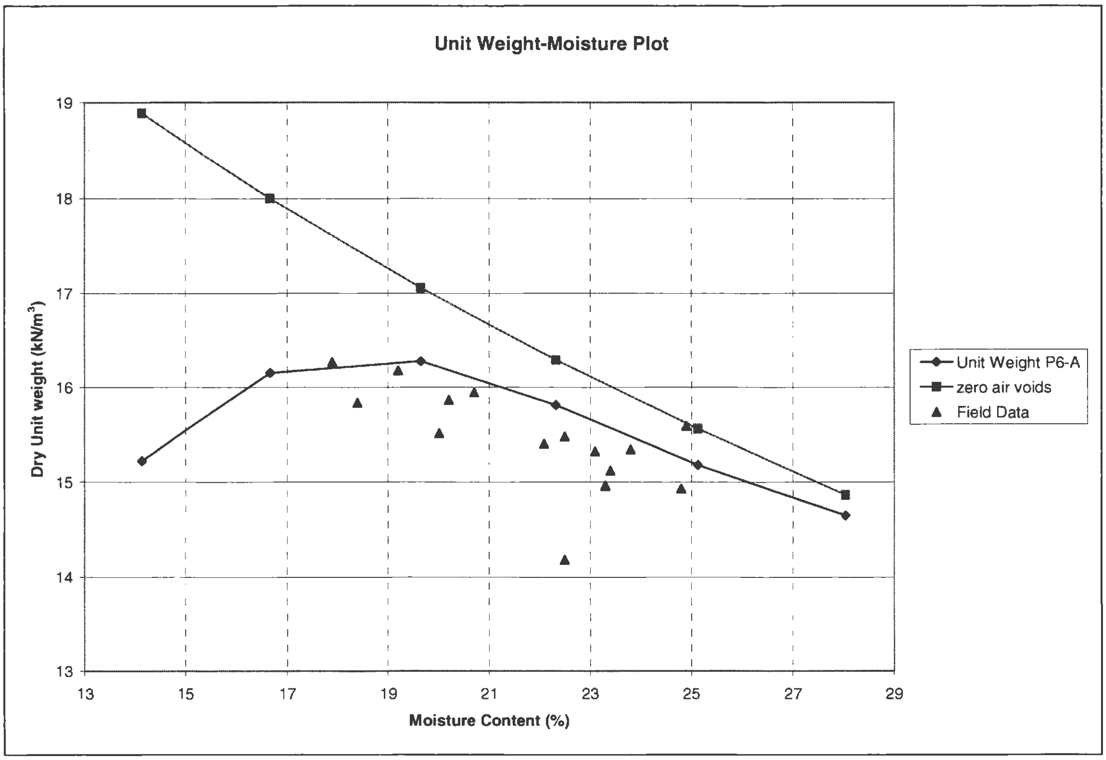

Figure 5. 18 Unit weight-Moisture P6-A

The lift thickness, as estimated from the DCP plots, ranged from about 200 mm to 430 mm. The CBR values ranged from 6 to 10. Appendix B documents field DCP plots. Table 5.15 presents quality control application on the field data.

Table 5. 15 Quality Control comparison Project 6

Soil type	Suitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			85	**40**	**132**	**185**	**32**
syd1	58	20	Pass	Pass	Pass	Pass	Fail
syd2	43	12	Pass	Pass	Pass	Pass	Fail
syd3	48	11	Pass	Pass	Pass	Pass	Fail
syd4	42	9	Pass	Pass	Pass	Pass	Fail
syd5	44	10	Pass	Pass	Pass	Pass	Fail
syd6	61	24	Pass	Pass	Pass	Pass	Fail
syd7	56	14	Pass	Pass	Pass	Pass	Fail
syd8	67	9	Pass	Pass	Pass	Pass	Fail
syd9	72	13	Pass	Pass	Pass	Pass	Fail
syd10	58	13	Pass	Pass	Pass	Pass	Fail
syd11	139	39	Fail	Pass	Fail	Pass	Fail
syd12	64	17	Pass	Pass	Pass	Pass	Fail
syd13	62	17	Pass	Pass	Pass	Pass	Fail
syd14	68	15	Pass	Pass	Pass	Pass	Fail
syd15	51	12	Pass	Pass	Pass	Pass	Fail

5.9 Project No. 9: CAT Edwards Facility

The project was visited between the 24th to the 26th of March 2004. The testing was part of a pilot study for caterpillar at the Edwards indoor facility near Peoria, IL.

Eight test strips, identified as A through H, were constructed and tested. Construction operations consisted of the following steps: (1) aerate/till existing soil with an RR350; (2) moisture condition soil with water truck; (3) remix with one to two additional passes of the RR350; (4) blade to level surface; (5) compact with 6 to 10 passes of the CAT CP-533E roller. The test strips varied in loose lift thickness and water content.

The soil type was relatively uniform and of glacial origin. Figures 5.19 and 5.20 shows plots of the unit weight moisture relationship with the data from field tests. A standard Proctor test indicates that optimum water content is around 12% to 13%, and the maximum unit weight of the soil is 18.5 kN/m^3.

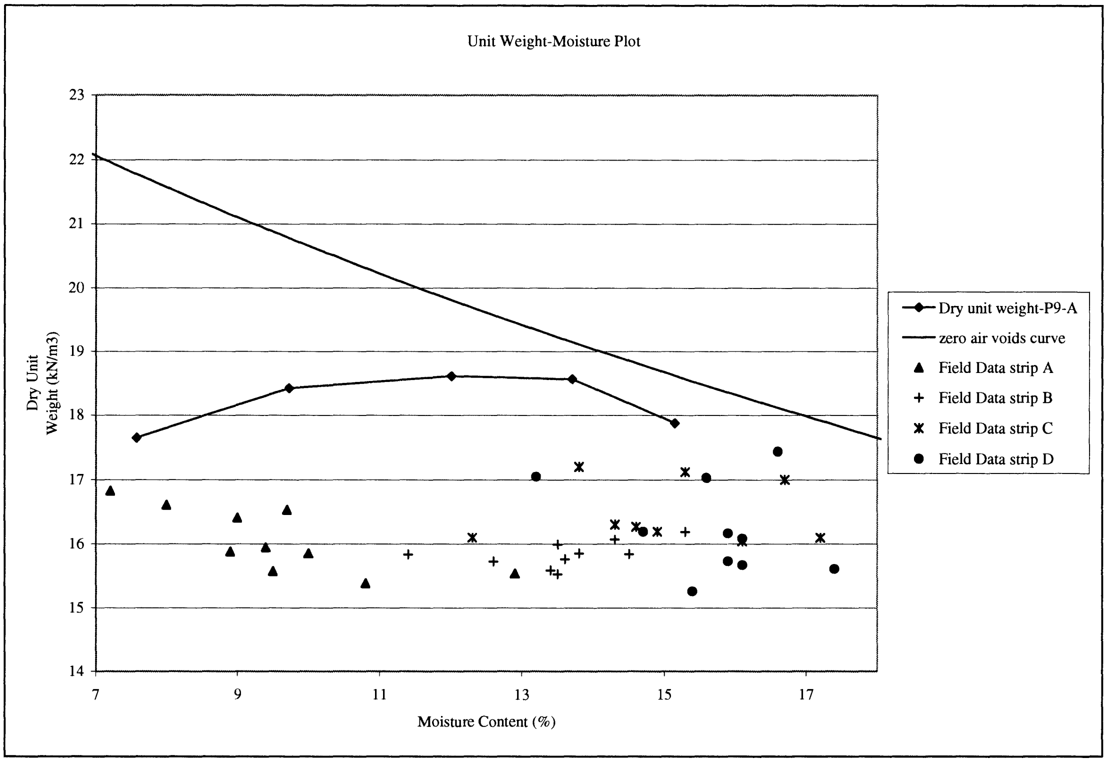

Figure 5. 19 Unit weight for the soils from Edwards Facility with field results of strips A-D

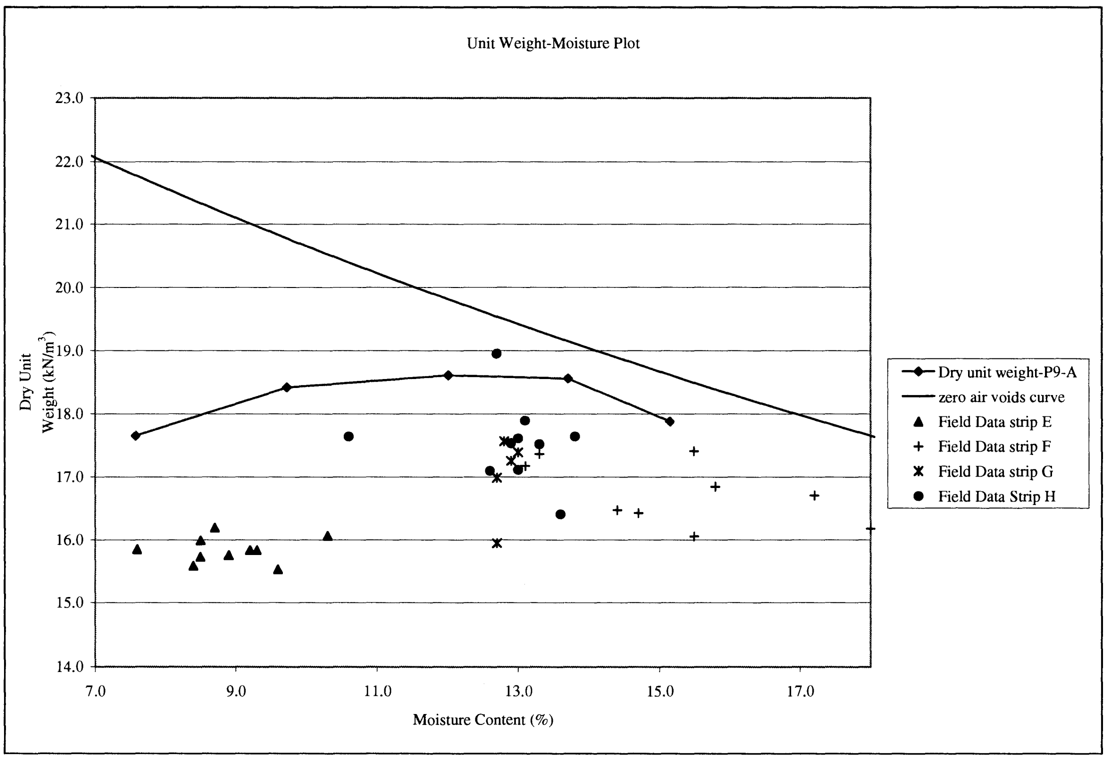

Figure 5. 20 Unit weight for the soils from Edwards Facility with field results of strips E-H

Test strips A through D were compacted first. Compaction was achieved with 6 roller passes. Loose lift thicknesses for these test strips were approximately 300 mm (12 in) for A and 400 mm (16 in) for B through D. Based on nuclear tests, the average moisture content increased from A to D as follows: 9.5%, 12.2%, 15.4%, and 17.3%, respectively.

Test strip E was compacted in forward and reverse directions with a total of ten passes (five forward and five reverse). Loose lift thickness averaged about 250 mm (10 in) and moisture content was about 8.9%. Test strips F and G were also compacted in forward and reverse directions. Loose lift thickness averaged about 660 to 710 mm (26 to 28 in). The average moisture contents for F and G were about 15.6% and 12.8%, respectively. Test strip H was compacted with only ten forward passes and had loose lift of about 300 mm (12 in) and water content near optimum at about 12.9%.

To evaluate changes in soil properties as they relate to compaction, five to ten test points were randomly identified within each test strip, and various measurements were taken: density, water content, strength (DCP), and stiffness (Clegg impact hammer).

A summary of the mean DCPI, moisture content, density, loose lift thickness and Clegg impact values from the measurements is documented in Tables 5.16 through 5.23.

Table 5. 16 Field Data for strip A from Project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1A	9	16.4	17.2	12	304.8	6
2A	8	16.6	16.7	17	304.8	6
3A	12.9	15.5	10.5	37	304.8	6
4A	10.8	15.4	15.4	20	304.8	6
5A	9.7	16.5	14.9	15	304.8	6
6A	9.4	15.9	13.7	28	304.8	6
7A	9.5	15.6	11.8	28	304.8	6
8A	10	15.9	10.6	32	304.8	6
9A	7.2	16.8	11.7	19	304.8	6
10A	8.9	15.9	7.5	31	304.8	6

Table 5. 17 Field Data for Strip B from Project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1B	14.3	16.1	10.5	45	406.4	6
2B	13.8	15.9	7.1	52	406.4	6
3B	13.4	15.6	6.8	50	406.4	6
4B	14.5	15.8	7.5	49	406.4	6
5B	13.6	15.8	11.5	44	406.4	6
6B	13.5	16.0	9.1	50	406.4	6
7B	15.3	16.2	8.1	48	406.4	6
8B	12.6	15.7	11.2	43	406.4	6
9B	13.5	15.5	8.3	49	406.4	6
10B	11.4	15.8	9.7	39	406.4	6

Table 5. 18 Field Data for Strip C from Project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1C	13.8	17.2	8.3	53	406.4	6
2C	17.2	16.1	3.7	116	406.4	6
3C	19.2	15.8	3.5	116	406.4	6
4C	14.6	16.3	7	66	406.4	6
5C	16.1	16.0	5.3	90	406.4	6
6C	16.7	17.0	5.3	91	406.4	6
7C	15.3	17.1	6.8	63	406.4	6
8C	14.3	16.3	5.7	90	406.4	6
9C	14.9	16.2	4.4	68	406.4	6
10C	12.3	16.1	7.3	51	406.4	6

Table 5. 19 Field Data for Strip D from Project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1D	13.2	17.1	6.5	34	406.4	6
2D	15.6	17.0	6.4	55	406.4	6
3D	16.1	16.1	3.5	93	406.4	6
4D	15.4	15.3	4.3	71	406.4	6
5D	17.4	15.6	4.6	92	406.4	6
6D	14.7	16.2	6.4	87	406.4	6
7D	15.9	16.2	5.3	76	406.4	6
8D	16.6	17.4	4.8	100	406.4	6
9D	15.9	15.7	4.7	130	406.4	6
10D	16.1	15.7	4.7	73	406.4	6

Table 5. 20 Field Data for Strip E from Project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1E	10.3	16.3	19.8	12	254	10
2E	7.6	16.6	19	18	254	10
3E	8.4	16.5	22.7	10	254	10
4E	9.3	15.5	13.4	19	254	10
5E	8.9	16.8	21.7	12	254	10
6E	8.5	16.5	14.5	16	254	10
7E	8.7	16.6	24.7	26	254	10
8E	8.5	16.3	16.6	17	254	10
9E	9.6	16.6	19.1	16	254	10
10E	9.2	16.8	31.3	9	254	10

Table 5. 21 Field Data for Strip E from Project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1F	18.4	15.7	6.1	59	660-710	10
2F	18	16.2	6.9	60	660-710	10
3F	15.8	16.8	7.5	69	660-710	10
4F	14.4	16.5	8	49	660-710	10
5F	13.3	17.4	7.1	57	660-710	10
6F	17.2	16.7	4.6	95	660-710	10
7F	15.5	17.4	5.7	56	660-710	10
8F	13.1	17.2	7.5	47	660-710	10

Table 5. 22 Field Data for Strip G from Project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1G	12.8	17.6	10.4	47	660.4	10
2G	12.7	17.0	12.4	41	660.4	10
3G	12.7	15.9	9.2	41	660.4	10
4G	12.9	17.3	12.9	38	660.4	10
5G	13	17.4	13.1	38	660.4	10

Table 5. 23 Field Data for Strip H from project 9

Test Point	Water content (%)	Unit Weight (kN/m^3)	Clegg Impact Value	Mean DCPI (mm/ blow)	Loose lift (mm)	Number of Passes
1H	12.7	18.9	11.3	25	304.8	10
2H	13	17.6	11.5	28	304.8	10
3H	13.6	16.4	10.5	22	304.8	10
4H	12.9	17.5	11.7	28	304.8	10
5H	13.8	17.6	11.7	36	304.8	10
6H	13.3	17.5	13.2	24	304.8	10
7H	10.6	17.6	16.4	17	304.8	10
8H	13.1	17.9	11.8	34	304.8	10
9H	13	17.1	14.9	20	304.8	10
10H	12.6	17.1	17.3	17	304.8	10

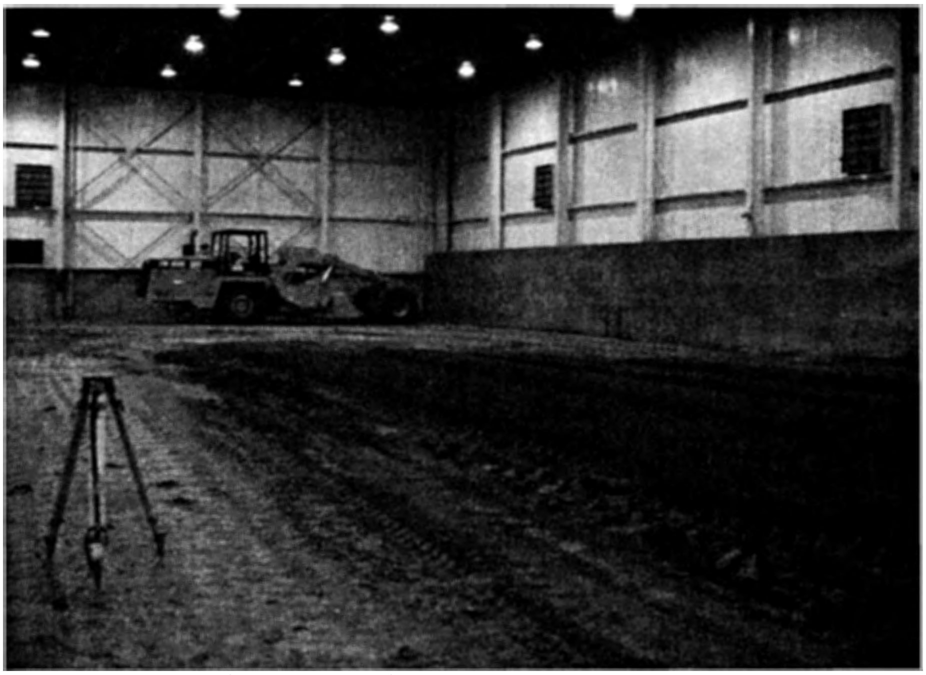

Figure 5. 21Test strips F-H after tilling

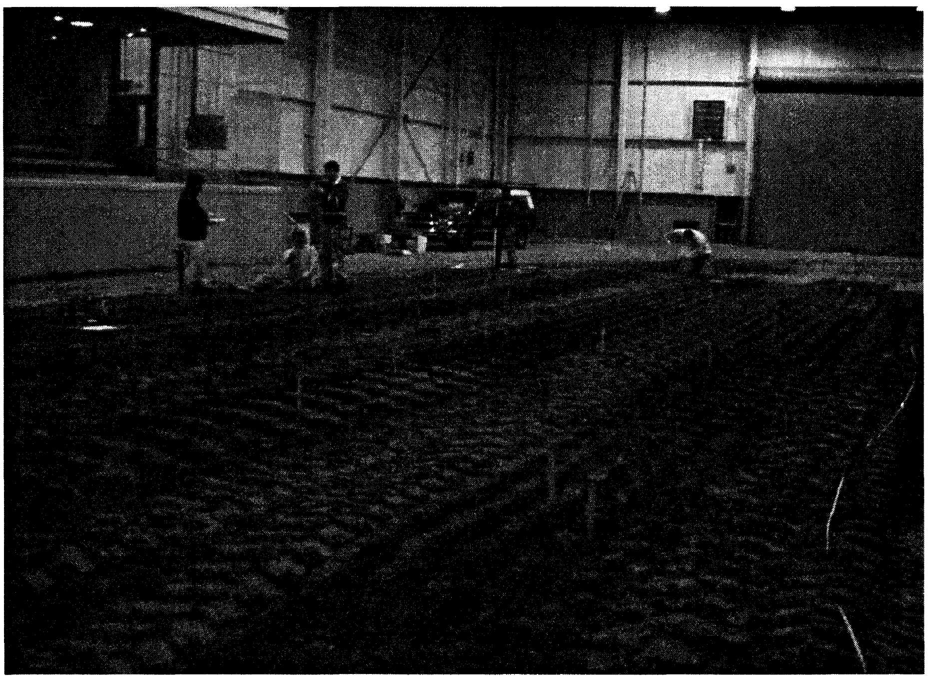

Figure 5. 22 Test strips A-D after compaction

To develop strength versus depth profiles, Dynamic Cone Penetrometer (DCP) tests were performed at all test points. Table 5.24 summarizes the average moisture content, average loose lift thickness, average compacted lift thickness and the average DCPI results for each test strip. The summary of mean change in DCPI is presented in Table 5.25. Individual DCP test results are provided in Appendix B. Compacted lift thicknesses were obtained from the DCP plots by observing the change in the DCP index profile with depth. The mean DCP index was calculated by averaging the index values in the uppermost lift and ignoring results from the underlying layer.

Table 5. 24 Summary of Average DCPI from Strips A-H

Test#/Strip	A	B	C	D	E	F	G	H
Average *w*%	9.54	12.21	15.44	17.25	8.90	15.59	12.82	12.87
Average Loose lift(mm)	305	406	406	406	254	660-710	660	305
Average Compacted Lift(mm)	256.54	248.92	254	256.54	187.96	452.12	538.48	162.56
Test point	Mean DCP index mm/blow							
1	14	46	50	37	12	62	39	19
2	17	50	93	51	16	10	38	22
3	41	47	98	103	10	52	39	19
4	20	42	62	79	16	64	35	22
5	15	42	85	89	13	50	35	27
6	26	43	82	83	16	51		22
7	27	42	58	72	21	84		16
8	29	42	86	83	16	55		25
9	18	52	64	122	15	47		17
10	29	40	58	76	10	44		18
Average	24	45	74	79	14	52	37	21

Table 5. 25 Summary of change in mean DCPI from strip A-H

Test#/Strip	A	B	C	D	E	F	G	H
Test point	Mean Change in DCP index mm/blow							
1	1	8	9	10	3	6	4	5
2	2	9	9	11	3	6	5	3
3	7	11	10	27	2	8	2	4
4	4	6	13	30	2	6	5	4
5	1	9	26	14	3	8	3	4
6	3	7	13	14	2	18		3
7	5	7	14	12	4	8		4
8	5	9	12	65	2	7		3
9	3	13	15	8	1	7		4
10	2	7	20	20	3	8		3
Average	3	9	14	21	2	8	4	4

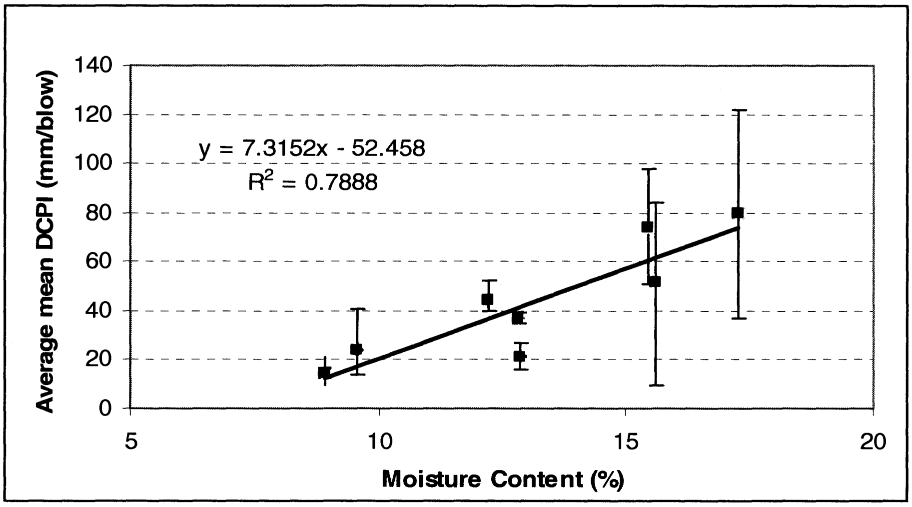

Figure 5. 23 Variation of Avg. MDCP with Moisture

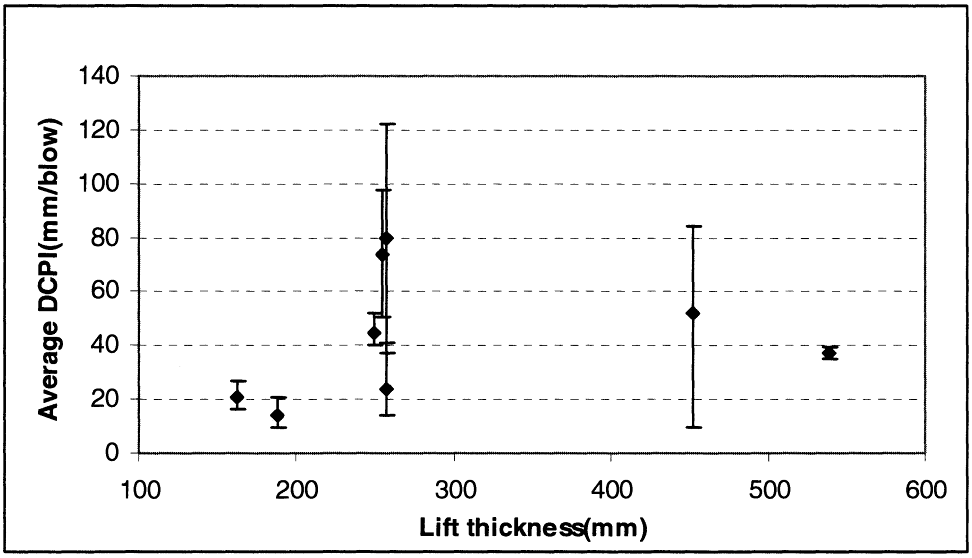

Figure 5. 24 Influence of lift thickness on mean DCPI

Stiffness – CIV Clegg Impact Values (CIV) are empirically related to CBR and soil stiffness parameters (i.e. modulus of subgrade reaction) and can simulate penetration of a roller pad/foot. Figure 5.25 indicates that the CIV increases as the water content decreases.

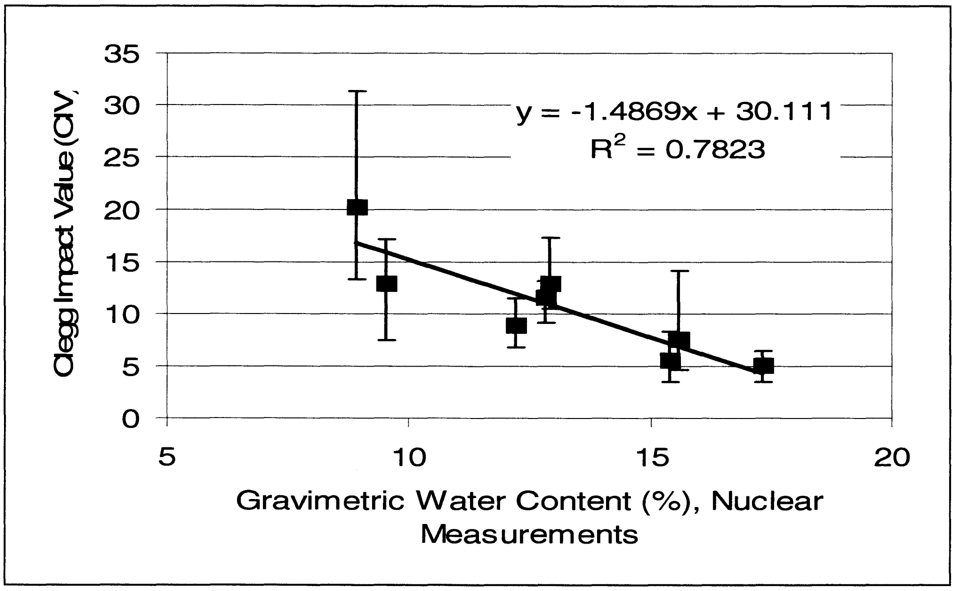

Figure 5. 25 Variation Clegg Impact Values with Moisture Content

Table 5. 26 Quality Control comparison Project 9

Soil type	Suitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			85	**40**	**132**	**185**	**32**
1A	14	1	Pass	Pass	Pass	Pass	Pass
2A	17	2	Pass	Pass	Pass	Pass	Pass
3A	41	7	Pass	Pass	Pass	Pass	Fail
4A	20	4	Pass	Pass	Pass	Pass	Pass
5A	15	1	Pass	Pass	Pass	Pass	Pass
6A	26	3	Pass	Pass	Pass	Pass	Pass
7A	27	5	Pass	Pass	Pass	Pass	Pass
8A	29	5	Pass	Pass	Pass	Pass	Pass
9A	18	3	Pass	Pass	Pass	Pass	Pass
10A	29	2	Pass	Pass	Pass	Pass	Pass
1B	46	8	Pass	Pass	Pass	Pass	Fail
2B	50	9	Pass	Pass	Pass	Pass	Fail
3B	47	11	Pass	Pass	Pass	Pass	Fail
4B	42	6	Pass	Pass	Pass	Pass	Fail
5B	42	9	Pass	Pass	Pass	Pass	Fail
6B	43	7	Pass	Pass	Pass	Pass	Fail
7B	42	7	Pass	Pass	Pass	Pass	Fail
8B	42	9	Pass	Pass	Pass	Pass	Fail
9B	52	13	Pass	Pass	Pass	Pass	Fail
10B	40	7	Pass	Pass	Pass	Pass	Fail
1C	50	9	Pass	Pass	Pass	Pass	Fail
2C	93	9	Fail	Pass	Pass	Pass	Fail
3C	98	10	Fail	Pass	Pass	Pass	Fail
4C	62	13	Pass	Pass	Pass	Pass	Fail
5C	85	26	Pass	Pass	Pass	Pass	Fail
6C	82	13	Pass	Pass	Pass	Pass	Fail
7C	58	14	Pass	Pass	Pass	Pass	Fail
8C	86	12	Fail	Pass	Pass	Pass	Fail
9C	64	15	Pass	Pass	Pass	Pass	Fail
10C	58	20	Pass	Pass	Pass	Pass	Fail
1D	37	10	Pass	Pass	Pass	Pass	Fail
2D	51	11	Pass	Pass	Pass	Pass	Fail
3D	103	27	Fail	Pass	Pass	Pass	Fail
4D	79	30	Pass	Pass	Pass	Pass	Fail
5D	89	14	Fail	Pass	Pass	Pass	Fail
6D	83	14	Pass	Pass	Pass	Pass	Fail
7D	72	12	Pass	Pass	Pass	Pass	Fail
8D	83	65	Pass	Fail	Pass	Pass	Fail
9D	122	8	Fail	Pass	Pass	Pass	Fail
10D	76	20	Pass	Pass	Pass	Pass	Fail

Table 5. 27 Quality Control comparison Project 9

Soil type	Suitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			85	**40**	**132**	**185**	**32**
1E	12	3	Pass	Pass	Pass	Pass	Pass
2E	16	3	Pass	Pass	Pass	Pass	Pass
3E	10	2	Pass	Pass	Pass	Pass	Pass
4E	16	2	Pass	Pass	Pass	Pass	Pass
5E	13	3	Pass	Pass	Pass	Pass	Pass
6E	16	2	Pass	Pass	Pass	Pass	Pass
7E	21	4	Pass	Pass	Pass	Pass	Pass
8E	16	2	Pass	Pass	Pass	Pass	Pass
9E	15	1	Pass	Pass	Pass	Pass	Pass
10E	10	3	Pass	Pass	Pass	Pass	Pass
1F	62	6	Pass	Pass	Pass	Pass	Fail
2F	52	6	Pass	Pass	Pass	Pass	Fail
3F	64	8	Pass	Pass	Pass	Pass	Fail
4F	50	6	Pass	Pass	Pass	Pass	Fail
5F	51	8	Pass	Pass	Pass	Pass	Fail
6F	84	18	Pass	Pass	Pass	Pass	Fail
7F	55	8	Pass	Pass	Pass	Pass	Fail
8F	47	7	Pass	Pass	Pass	Pass	Fail
9F	44	7	Pass	Pass	Pass	Pass	Fail
10F	43	8	Pass	Pass	Pass	Pass	Fail
1G	39	4	Pass	Pass	Pass	Pass	Fail
2G	38	5	Pass	Pass	Pass	Pass	Fail
3G	39	2	Pass	Pass	Pass	Pass	Fail
4G	35	5	Pass	Pass	Pass	Pass	Fail
5G	35	3	Pass	Pass	Pass	Pass	Fail
1H	19	5	Pass	Pass	Pass	Pass	Pass
2H	22	3	Pass	Pass	Pass	Pass	Pass
3H	19	4	Pass	Pass	Pass	Pass	Pass
4H	22	4	Pass	Pass	Pass	Pass	Pass
5H	27	3	Pass	Pass	Pass	Pass	Pass
6H	22	4	Pass	Pass	Pass	Pass	Pass
7H	16	3	Pass	Pass	Pass	Pass	Pass
8H	25	4	Pass	Pass	Pass	Pass	Pass
9H	17	3	Pass	Pass	Pass	Pass	Pass
10H	18	4	Pass	Pass	Pass	Pass	Pass

5.10 Project No. 10: CAT West Des Moines IA

The testing for this project took place at a construction site in West Des Moines Iowa. This project incorporated a Caterpillar compaction equipment demonstration. The testing at this site was performed on the 26th and the 28th of July in 2004. The test site had been tilled and then compacted with a CAT CP-533E sheepsfoot roller. For the testing performed on

July 26th, a strip was selected in the fill area, and five randomly select spots were tested after each of the four passes of the sheepsfoot roller. On July 28th, testing was performed on a strip after one, two, three, four, six and eight passes. A second strip was tested on the 28th after two, four and six passes.

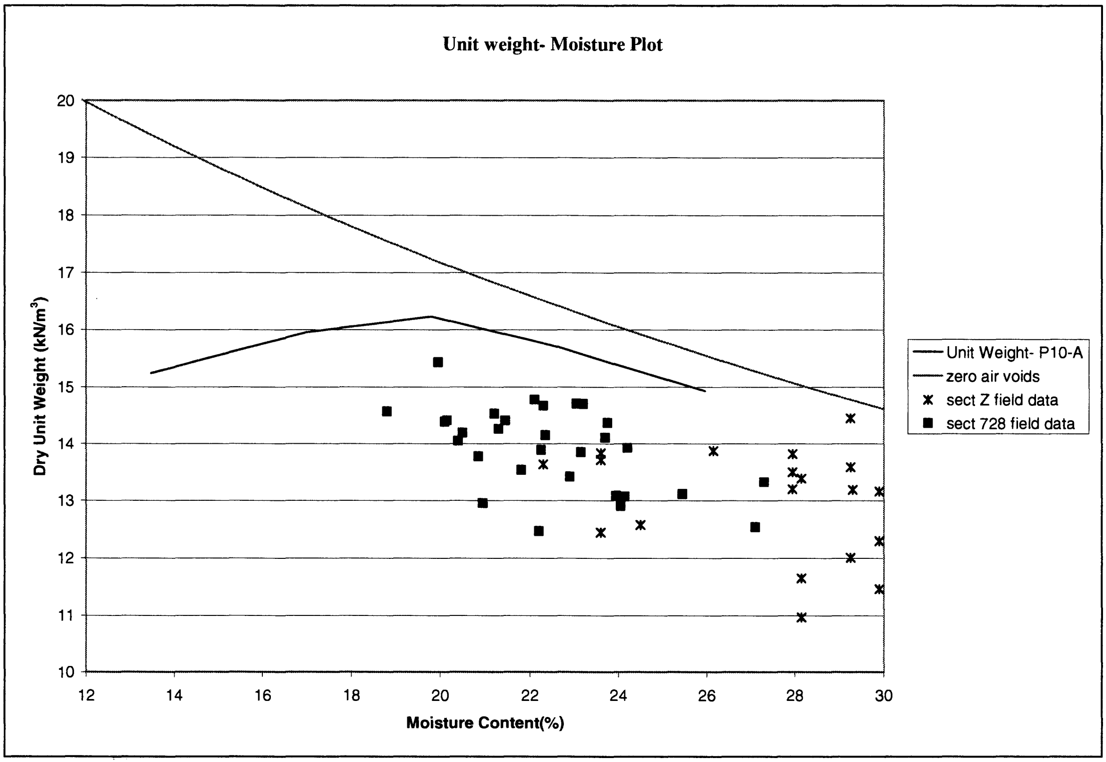

Figure 5. 26 Unit Weight Moisture plot P10-A

The tests performed on the soil included DCP tests, nuclear gauge testing (moisture and density), and Clegg impact. Figure 5.26 plots the moisture density relation of the soil on site for the material tests labeled Z and 728. Figure 5.27 plots moisture density relation of the soil labeled GS. Table 5.28 to 5.31 presents field results of the different tests perform at the site. The tables include the Mean DCP, moisture and density values. Estimates of the lift thickness are also included. The loose thickness of the first strip was about 400 mm (16 in), the CBR values from DCP tests ranged from 0.6 20 for Section Z, 0.5 to 15 for section 728 and 1.5 to 30 for section GS.

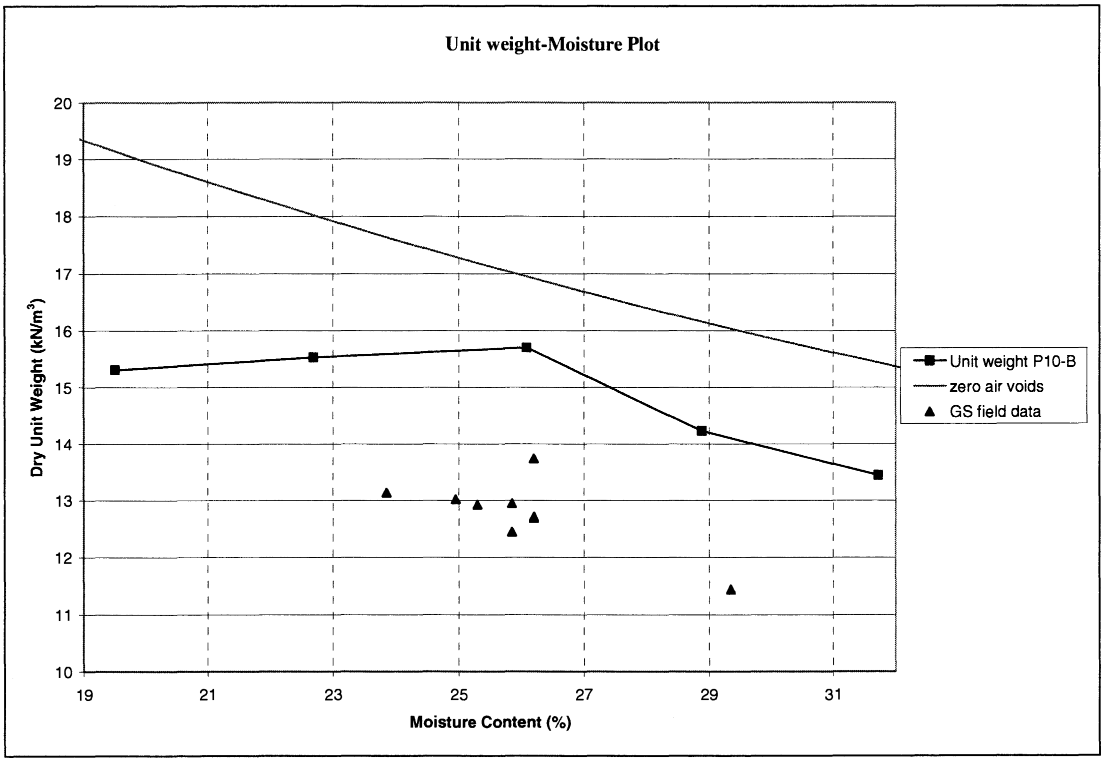

Figure 5. 27 Unit weight-moisture plot of P10-B

Table 5. 28 Field Data for Section Z from Project 10

	Point	Moisture Content(%)	Unit Weight	Clegg	Mean DCPI	Loose lift (mm)
Pass 1	Z1	28.0	13.8	3.4	162	406.4
	Z2	29.9	11.5	1.9	85	
	Z3	29.3	12.0	2.9	114	
	Z4	28.2	11.0	3.0	103	
	Z5	23.6	3.7	3.2	63	
Pass 2	Z1	32.0	12.9	3.8	57	
	Z2	29.3	13.2	3.4	232	
	Z3	26.2	13.9	3.1	90	
	Z4	24.5	12.6	4.1	81	
	Z5	22.3	13.6	4.2	91	
Pass 3	Z1	28.0	13.2	1.0	65	
	Z2	29.9	13.2	2.5	72	
	Z3	29.3	13.6	2.5	87	
	Z4	28.2	11.6	3.7	99	
	Z5	23.6	12.4	3.7	61	
Pass 4	Z1	28.0	13.5	2.0	88	
	Z2	29.9	12.3	3.1	56	
	Z3	29.3	14.5	3.2	93	
	Z4	28.2	13.4	4.5	75	
	Z5	23.6	13.8	4.1	56	

Table 5. 29 Field Data for Section GS from Project 10

	Point	Moisture Content(%)	Unit Weight	Clegg	Mean DCPI	Loose lift
Pass 2	GSNV1	29.4	11.5	4.9	50	381
	GSNV2	25.9	12.5	4.7	58	
	GSNV3	23.9	13.3	4.7	74	
Pass 4	GSNV1	26.2	12.8	5.6	38	
	GSNV2	25.9	12.8	6.6	74	
	GSNV3	25.3	13.9	6.9	56	
Pass 6	GSNV1	25.0	13.1	6.1	51	
	GSNV2	25.5	13.0	7.2	70	
	GSNV3	26.8	13.1	6	49	

Table 5. 30 Field Data for section 728 from project 10

	Point	Moisture Content(%)	Unit Weight	Clegg	Mean DCPI	Loose lift
Pass 1	728a	24.2	13.8	1.9	175	355.6-460
	728b	21.0	13.1	3.6	112	
	728c	24.0	13.1	3.1	136	
	728d	21.8	12.5	2.9	98	
	728e	20.9	13.3	2.5	240	
Pass 2	728a	20.5	13.9	3.7	116	
	728b	24.1	13.0	3.1	146	
	728c	22.9	13.1	3.4	247	
	728d	22.2	13.5	2.5	155	
	728e	20.2	13.8	2.7	132	
Pass 3	728a	18.8	14.2	4.1	91	
	728b	22.3	12.9	3.8	102	
	728c	23.7	13.4	2.8	143	
	728d	20.4	12.5	2.7	120	
	728e	21.2	14.4	4.7	144	
Pass 4	728a	23.1	14.6	5.7	86	
	728b	22.3	13.9	2.4	119	
	728c	22.4	14.1	4.1	159	
	728d	20.1	14.1	2.8	95	
	728e	21.5	14.5	3.8	127	
Pass 6	728a	20.0	14.7	3.7	99	
	728b	23.2	14.7	2.4	183	
	728c	23.8	14.2	4.7	97	
	728d	21.3	14.4	5.1	171	
	728e	22.1	14.4	2.9	93	
Pass 8	728a	20.0	15.4	3.7	94	
	728b	23.2	14.7	5.5	122	
	728c	23.8	14.4	3.3	131	
	728d	21.3	14.3	3.8	183	
	728e	22.1	14.8	2.5	206	

Table 5. 31 Quality Control comparison Section Z

Soil type	Unsuitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			95	**40**	**132**	**185**	**32**
Z1a	162	18	Fail	Pass	Fail	Pass	Fail
Z2a	85	19	Pass	Pass	Pass	Pass	Fail
Z3a	114	19	Fail	Pass	Pass	Pass	Fail
Z4a	103	51	Fail	Fail	Pass	Pass	Fail
Z5a	63	16	Pass	Pass	Pass	Pass	Fail
Z1aa	57	12	Pass	Pass	Pass	Pass	Fail
Z2aa	232	33	Fail	Pass	Fail	Fail	Fail
Z3aa	90	17	Pass	Pass	Pass	Pass	Fail
Z4aa	81	18	Pass	Pass	Pass	Pass	Fail
Z5aa	91	20	Pass	Pass	Pass	Pass	Fail
Z1aaa	65	13	Pass	Pass	Pass	Pass	Fail
Z2aaa	72	15	Pass	Pass	Pass	Pass	Fail
Z3aaa	87	16	Pass	Pass	Pass	Pass	Fail
Z4aaa	99	21	Fail	Pass	Pass	Pass	Fail
Z5aaa	61	9	Pass	Pass	Pass	Pass	Fail
Z1aaaa	88	68	Pass	Fail	Pass	Pass	Fail
Z2aaaa	56	44	Pass	Fail	Pass	Pass	Fail
Z3aaaa	93	19	Pass	Pass	Pass	Pass	Fail
Z4aaaa	75	16	Pass	Pass	Pass	Pass	Fail
Z5aaaa	56	10	Pass	Pass	Pass	Pass	Fail

Table 5. 32 Quality Control comparison Section GS

Soil type	Unsuitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			95	**40**	**132**	**185**	**32**
P2GS1	50	14	Pass	Pass	Pass	Pass	Fail
P2GS2	58	14	Pass	Pass	Pass	Pass	Fail
P2GS3	74	19	Pass	Pass	Pass	Pass	Fail
P4GS1	38	17	Pass	Pass	Pass	Pass	Fail
P4GS2	74	9	Pass	Pass	Pass	Pass	Fail
P4GS3	56	13	Pass	Pass	Pass	Pass	Fail
P6GS1	51	13	Pass	Pass	Pass	Pass	Fail
P6GS2	70	21	Pass	Pass	Pass	Pass	Fail
P6GS3	49	6	Pass	Pass	Pass	Pass	Fail
V2GS1	71	7	Pass	Pass	Pass	Pass	Fail
V2GS1W	53	7	Pass	Pass	Pass	Pass	Fail
V2GS2	37	4	Pass	Pass	Pass	Pass	Fail
V2GS2W	57	20	Pass	Pass	Pass	Pass	Fail
V2GS3	63	17	Pass	Pass	Pass	Pass	Fail
V4GS1	60	12	Pass	Pass	Pass	Pass	Fail
V4GS1W	46	7	Pass	Pass	Pass	Pass	Fail
V4GS2	44	7	Pass	Pass	Pass	Pass	Fail
V4GS2W	52	13	Pass	Pass	Pass	Pass	Fail
V4GS3	52	10	Pass	Pass	Pass	Pass	Fail
V8GS1	51	10	Pass	Pass	Pass	Pass	Fail
V8GS1W	44	2	Pass	Pass	Pass	Pass	Fail
V8GS2	42	5	Pass	Pass	Pass	Pass	Fail
V8GS2W	59	21	Pass	Pass	Pass	Pass	Fail
V8GS3	46	7	Pass	Pass	Pass	Pass	Fail

Table 5. 33 Quality Control comparison Section 728

Soil type	Unsuitable		Phase II		Max DCPI		
Test point	MDCPI (mm/blow)	UMDCPI (mm/blow)	MDCPI	UDCPI	M_R	Stability	CBR 6%
			95	**40**	**132**	**185**	**32**
P1728a	175	20	Fail	Pass	Fail	Pass	Fail
P1728b	112	62	Fail	Fail	Pass	Pass	Fail
P1728c	136	36	Fail	Pass	Fail	Pass	Fail
P1728d	98	35	Fail	Pass	Pass	Pass	Fail
P1728e	240	26	Fail	Pass	Fail	Fail	Fail
P2728a	116	29	Fail	Pass	Pass	Pass	Fail
P2728b	146	25	Fail	Pass	Fail	Pass	Fail
P2728c	247	0	Fail	Pass	Fail	Fail	Fail
P2728d	155	14	Fail	Pass	Fail	Pass	Fail
P2728e	132	38	Fail	Pass	Pass	Pass	Fail
P3728a	91	25	Pass	Pass	Pass	Pass	Fail
P3728b	102	21	Fail	Pass	Pass	Pass	Fail
P3728c	143	49	Fail	Fail	Fail	Pass	Fail
P3728d	120	31	Fail	Pass	Pass	Pass	Fail
P3728e	144	112	Fail	Fail	Fail	Pass	Fail
P4728a	86	29	Pass	Pass	Pass	Pass	Fail
P4728b	119	55	Fail	Fail	Pass	Pass	Fail
P4728c	159	20	Fail	Pass	Fail	Pass	Fail
P4728d	95	64	Pass	Fail	Pass	Pass	Fail
P4728e	127	69	Fail	Fail	Pass	Pass	Fail
P6728a	99	35	Fail	Pass	Pass	Pass	Fail
P6728b	183	101	Fail	Fail	Fail	Pass	Fail
P6728c	97	44	Fail	Fail	Pass	Pass	Fail
P6728d	171	22	Fail	Pass	Fail	Pass	Fail
P6728e	93	61	Pass	Fail	Pass	Pass	Fail
P8728a	94	40	Pass	Fail	Pass	Pass	Fail
P8728b	122	22	Fail	Pass	Pass	Pass	Fail
P8728c	131	45	Fail	Fail	Pass	Pass	Fail
P8728d	183	21	Fail	Pass	Fail	Pass	Fail
P8728e	206	22	Fail	Pass	Fail	Fail	Fail

5.11 Project No. 11: Wells Fargo West Des Moines

The site was visited on the 17th and the 18th of August, 2004. The site is located on the same construction site as project No 10 (above). Geopier® Foundation System was installed for a section of the building pad. The testing described here targeted areas surrounding a test pier used for other testing, including finding the influence zone of the pier. The DCP was one of the evaluation instruments.

The planned testing was to use DCP, Geogauge, and Clegg Impact hammer for testing points around the pier. Load tests were also performed adjacent to the pier underneath a horizontal beam which was being used as a reaction beam in the load test. Clegg and Geogauge tests were performed at these spots as well. The DCP was not used because of the limited head room underneath the horizontal beam would not accommodate the equipment.

The first 27 test spots were tested on the first day, while the remaining 17 were tested on the second day. There was no need to perform lab testing on the soil from the site as the soil was the same as that from project No 10.

The map of the test spots is presented in Figure 5.28. The tests points were arranged in rings around the pier at 1) distances of 61 cm (24 in), 76 cm (30 in), and 122 cm (48 mm) for eight lines from the pier and 2) distances of 61 cm(24 in), 76 cm(30 in), 91 cm (36 in), 122 cm(48 in), 152 cm(60 in), 213 cm(60 in) and 274 cm(108 in) for two lines from the pier. Table 5.35 shows a summary of the mean DCP, Clegg impact values and the values from the Geogauge measured at the site.

The DCP testing performed at this site was useful to demonstrate the variation of DCP results with distance. Figure 5.29 presents the results of the all the testing and how these results varied with distance. Figure 5.30 demonstrates specifically how DCP results vary with distance from the pier. The graphed data is scattered with distance, accounting for a relatively small difference in DCP values. This data demonstrates that the DCP is repeatable and can also be used to determine testing frequency for quality control.

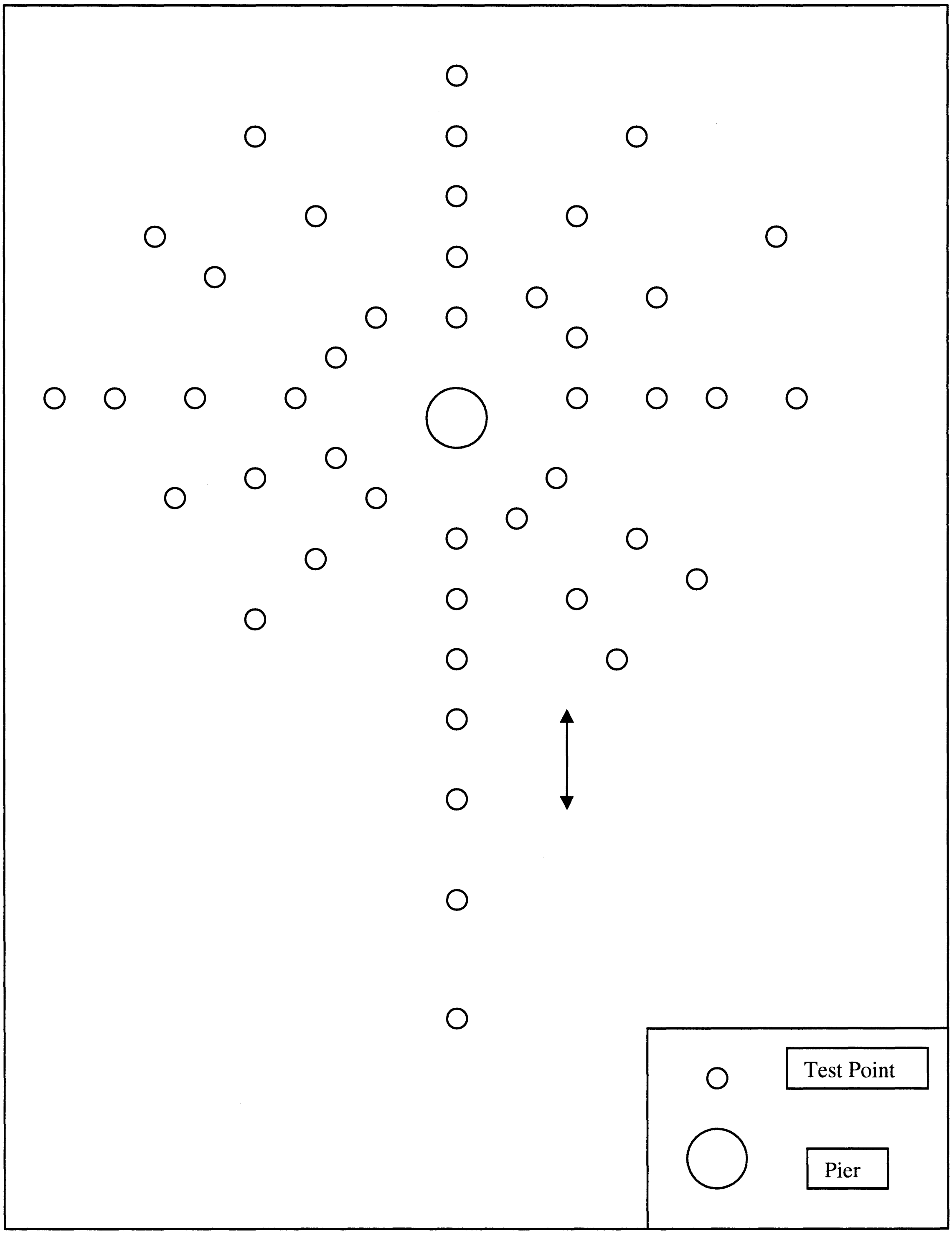

Figure 5. 28 Arrangement of test points around pier

Table 5. 34 Results of the field tests from Project 11

Test #	Distance from pier (mm)	MDCPI	UMDCPI	Geoguage		Clegg stiffness
				y-modulus Mpa	stiffness MN/m	
11	610	70	18	54.3	6.26	10.4
12	762	80	21	53.15	6.13	8.9
13	915	66	9	48.36	5.57	9.1
14	1219	64	9	50.62	5.83	10.1
15	1524	64	15	54.46	6.28	7.5
16	2134	55	5	56.35	6.5	7.3
17	2744	57	8	65.61	7.56	8.5
18	610	66	11			
19	762	59	11	62.2	7.17	8.4
20	610	66	11	52.25	6.02	9.5
21	726	72	14	58.92	6.79	9
22	610	50	8	53.24	6.14	9.7
23	762	61	11	61.68	7.1	9.2
24	610	61	11	47.51	5.48	8.8
25	762	53	9	75.73	8.73	10.7
26	1219	71	28	53.41	6.16	9.5
27	1219	54	5	72.58	8.37	10.1
28	1219	65	11	63.03	7.27	8.8
29	1219	57	7	47.84	5.51	8.9
30	610	52	5	61.46	7.08	5.9
31	762	74	10	68.88	7.94	5.8
32	1067	78	39	70.49	8.13	6.1
33	610	57	4	60	6.92	5.3
34	762	67	8	78.63	9.06	6.8
35	1067	71	13	71.61	8.25	6.2
36	762	61	8	76.79	8.85	7.1
37	1067	62	9	60.6	6.99	6.6
38	1372	68	6	55.63	6.41	5.8
39	1981	63	7	48.31	5.57	5.6
40	2591	68	6	75.15	8.66	8
41	610	65	19	42.2	4.86	6.6
42	762	71	13	60.16	6.93	6.5
43	1067	72	22	63.8	7.35	6.1
44	610	64	17	53.55	6.17	6.5
45	762	59	4	60.67	6.99	6
46	1067	59	5	55.73	6.42	6.7

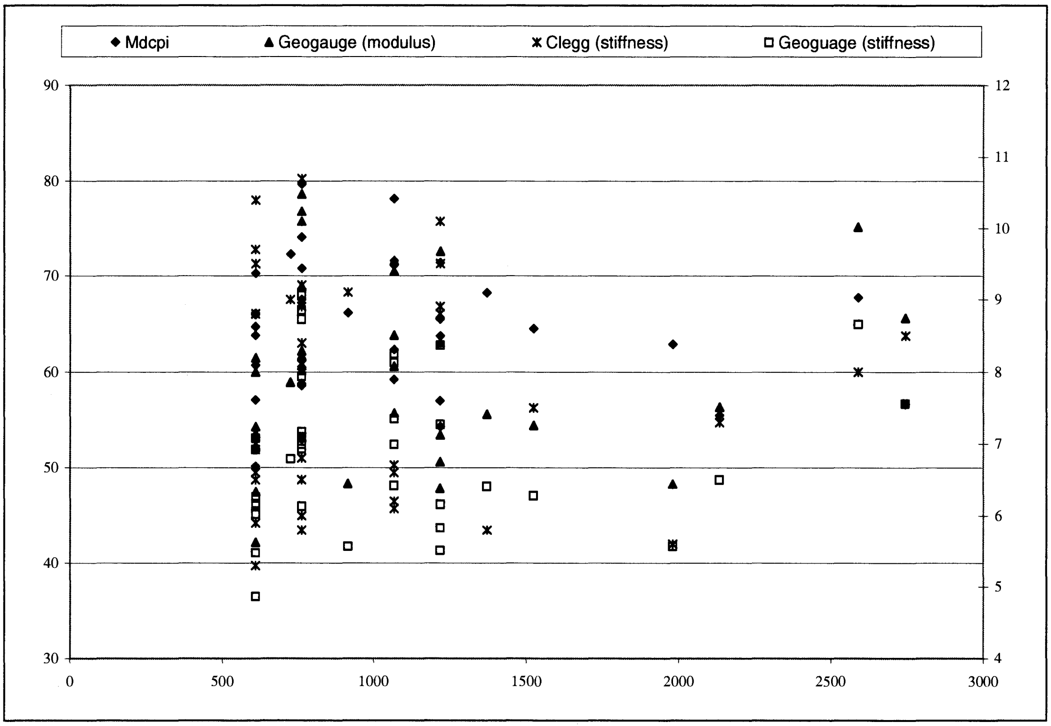

Figure 5. 29

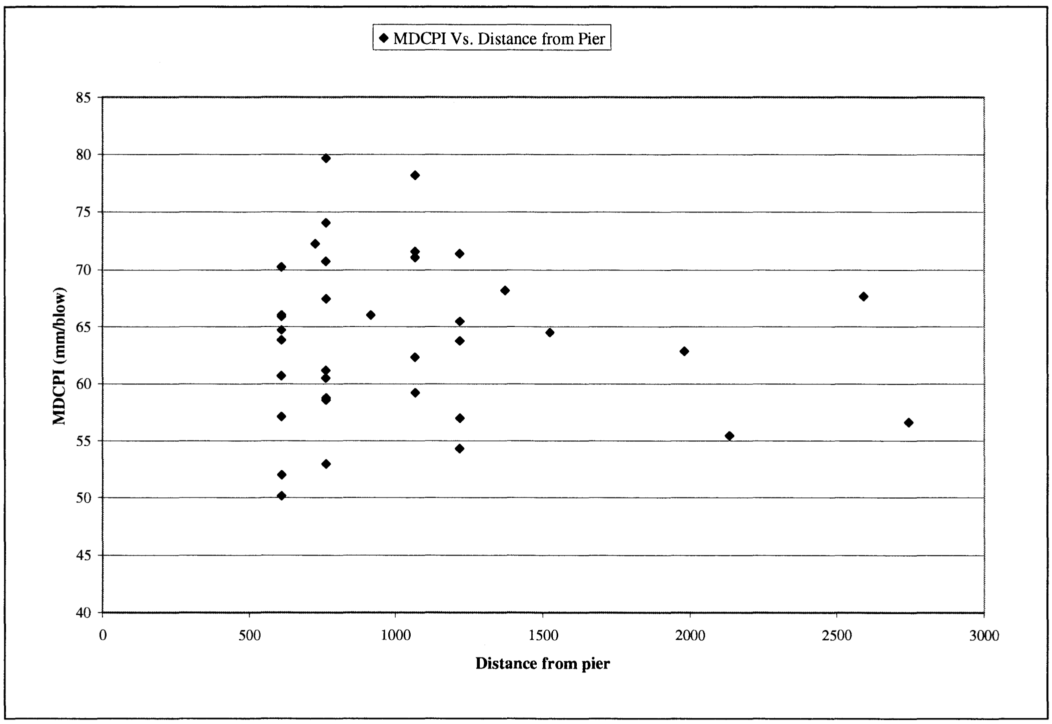

Figure 5. 30

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Introduction

The two main objectives of this thesis are:

1.To demonstrate and document how the DCP can be used as a quality control tool in testing strength and uniformity of fine grain materials and

2.To demonstrate and document how G-RAD can be used to make DCP data collection and processing more effective.

A literature review revealed that several engineering parameters can be correlated to the DCP Index. The engineering properties include, unconfined compressive strength which can be used in bearing capacity of a soil, California bearing ratio, which can be used as an indicator of the load bearing capacity of a geomaterial and modulus, both resilient modulus (M_R) and Modulus of substrate reaction (k). The modulus values and the CBR value are often used input parameters for pavement thickness design.

To use the DCP as a quality control tool, the design parameters are correlated to the DCP index. The design value is then the target value for the construction. The DCP index equivalent to the required engineering parameter is the limit used for quality control during construction. In this approach, quality control is achieved by indirect methods, measuring moisture and density of the soils, but also direct methods which target the very engineering parameters that are used for design of the highway materials and embankment soils.

In addition to measuring the engineering parameters, the DCP can be used to verify uniformity of the fill material placed. Ensuring that the fill is uniform will minimize the potential of the pavements rutting if soft layers in the embankment go unchecked, which could also lead to slope stability problems.

To demonstrate the quality control using the DCP, several construction sites were visited on which various other field tests where performed. The field tests included density and moisture testing using the nuclear density gauge and modulus and stiffness using the Geogauge and Clegg impact hammer.

Instructions for how to use G-RAD to improve the efficiency of the DCP are given in chapter 4. During some of the field site visits, data collection and processing was performed using G-RAD. Using G-RAD, the quality control results are instantaneous. The graphical views of the control charts simplify the decision making for the user.

6.2 Conclusions

Based on the data gathered, it has been established that the use of DCP in Iowa's fine-grain embankments would improve construction quality by providing data that ensures the of precise design paraments such as: slope stability, modulus of subgrade reaction, soil strength are measured during construction. Traditionally, measurement of these parameters has been time consuming and imprecise. This project demonstrates that G-RAD in conjunction with DCP improves not only the quality of construction but the accuracy and efficiency of the construction process.

6.3 Recommendations

Even though the quality control criterion used shows that all the sites had mostly passing results, further refining is needed for the criterion to be fully effective. Test strips should be used to establish the quality control limits that will be used on the projects.

There are only a few studies that have performed to correlate DCP index to soil Modulus or unconfined compressive strength. Further research should be performed to improve the correlations. Once the correlations are improved, quality control can be based on the pavement thickness design parameters, the modulus values and slope stability parameters, unconfined compressive strength.

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ACKNOWLEDGEMENTS

I owe my gratitude to all who afforded me help and support during the process of writing this thesis. I would like to especially thank my Major Professor, Dr. David White for the guidance, inspiration, and encouragement given throughout my graduate studies at Iowa State University. I would like to thank the Iowa Highway Research Board for funding the research. I would also like to thank Dr. Edward Jaselskis, Dr. Russell Walters, and Jianzhong Zhang for the work they contributed on the development of G-RAD. I would also like to thank my wife, Vita and the rest of my family for the encouragement throughout my studies. It would have been really hard to perform all the field and lab work without my fellow students. A special mention to Isaac Drew for coming out with me to job sites across Iowa to perform DCP testing--thank you guys. Lastly, I would like to express gratitude to my "home" church in Ames, Iowa, The Body of Christ Christian Worship Center; I really appreciate the love and support given to me during the writing of this thesis.

APPENDIX A

Uniformity Table and Plots

Table A. 1

Test #	701a	Calculated			Cum. blows as % of total blows	Ideal Depth	Actual dydx	ideal dydx	Difference
Entered Data		#blows	Depth, mm	cum # of blows					
#Blows	Depth ,mm								
0	115	0	0	0	0.00	0.0	0		
1	200	1	85	1	2.86	25.3	29.75	8.84	20.9100
1	257	1	142	2	5.71	50.5	19.95	8.84	11.1100
1	278	1	163	3	8.57	75.8	7.35	8.84	1.4900
1	293	1	178	4	11.43	101.0	5.25	8.84	3.5900
1	318	1	203	5	14.29	126.3	8.75	8.84	0.0900
1	342	1	227	6	17.14	151.5	8.4	8.84	0.4400
1	370	1	255	7	20.00	176.8	9.8	8.84	0.9600
1	393	1	278	8	22.86	202.1	8.05	8.84	0.7900
1	421	1	306	9	25.71	227.3	9.8	8.84	0.9600
1	460	1	345	10	28.57	252.6	13.65	8.84	4.8100
1	498	1	383	11	31.43	277.8	13.3	8.84	4.4600
1	524	1	409	12	34.29	303.1	9.1	8.84	0.2600
1	557	1	442	13	37.14	328.3	11.55	8.84	2.7100
1	590	1	475	14	40.00	353.6	11.55	8.84	2.7100
1	618	1	503	15	42.86	378.9	9.8	8.84	0.9600
1	642	1	527	16	45.71	404.1	8.4	8.84	0.4400
1	666	1	551	17	48.57	429.4	8.4	8.84	0.4400
1	695	1	580	18	51.43	454.6	10.15	8.84	1.3100
1	718	1	603	19	54.29	479.9	8.05	8.84	0.7900
1	734	1	619	20	57.14	505.1	5.6	8.84	3.2400
1	752	1	637	21	60.00	530.4	6.3	8.84	2.5400
1	774	1	659	22	62.86	555.7	7.7	8.84	1.1400
1	794	1	679	23	65.71	580.9	7	8.84	1.8400
1	820	1	705	24	68.57	606.2	9.1	8.84	0.2600
1	845	1	730	25	71.43	631.4	8.75	8.84	0.0900
1	868	1	753	26	74.29	656.7	8.05	8.84	0.7900
1	880	1	765	27	77.14	681.9	4.2	8.84	4.6400
1	897	1	782	28	80.00	707.2	5.95	8.84	2.8900
1	920	1	805	29	82.86	732.5	8.05	8.84	0.7900
1	928	1	813	30	85.71	757.7	2.8	8.84	6.0400
1	948	1	833	31	88.57	783.0	7	8.84	1.8400
1	957	1	842	32	91.43	808.2	3.15	8.84	5.6900
1	970	1	855	33	94.29	833.5	4.55	8.84	4.2900
1	983	1	868	34	97.14	858.7	4.55	8.84	4.2900
1	999	1	884	35	100.00	884.0	5.6	8.84	3.2400

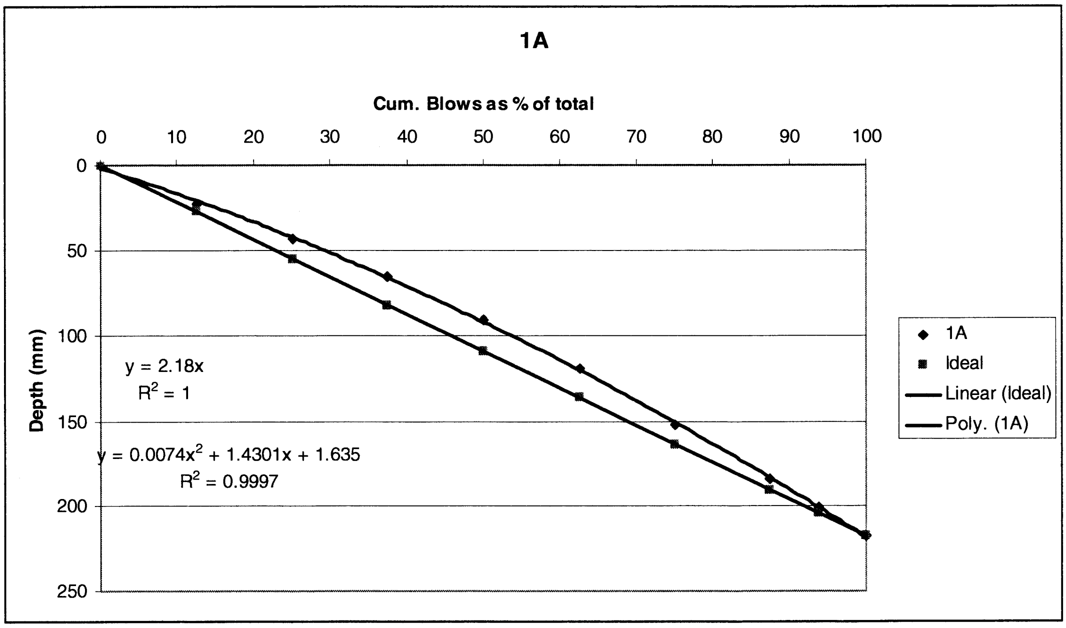
Notes:

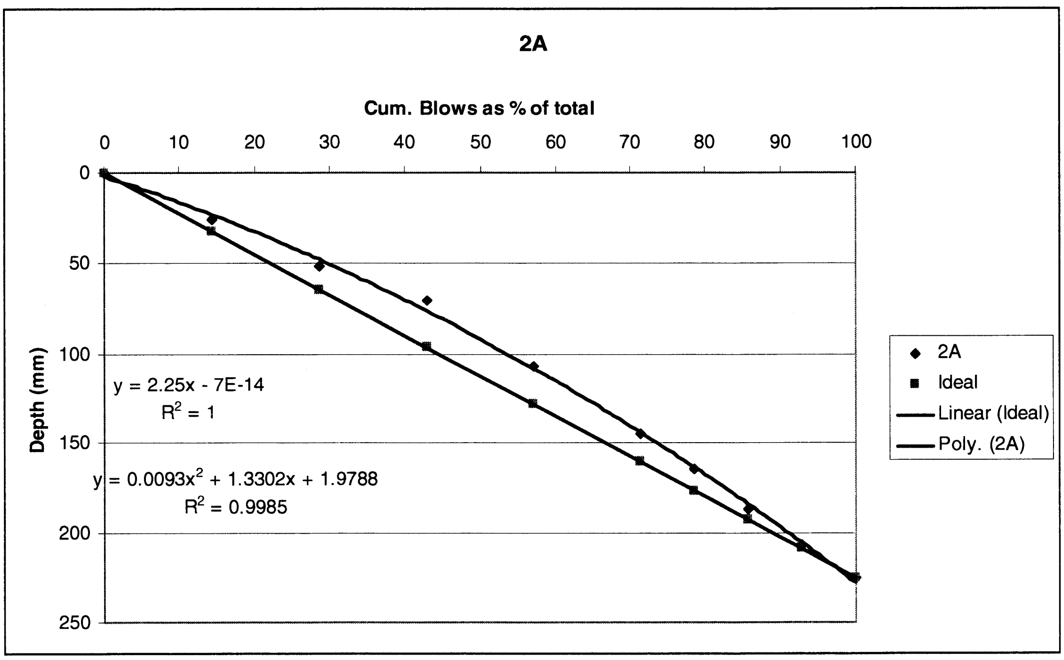
1.Column 7 (ideal depth) = Column 6 * (total depth penetrated/100)

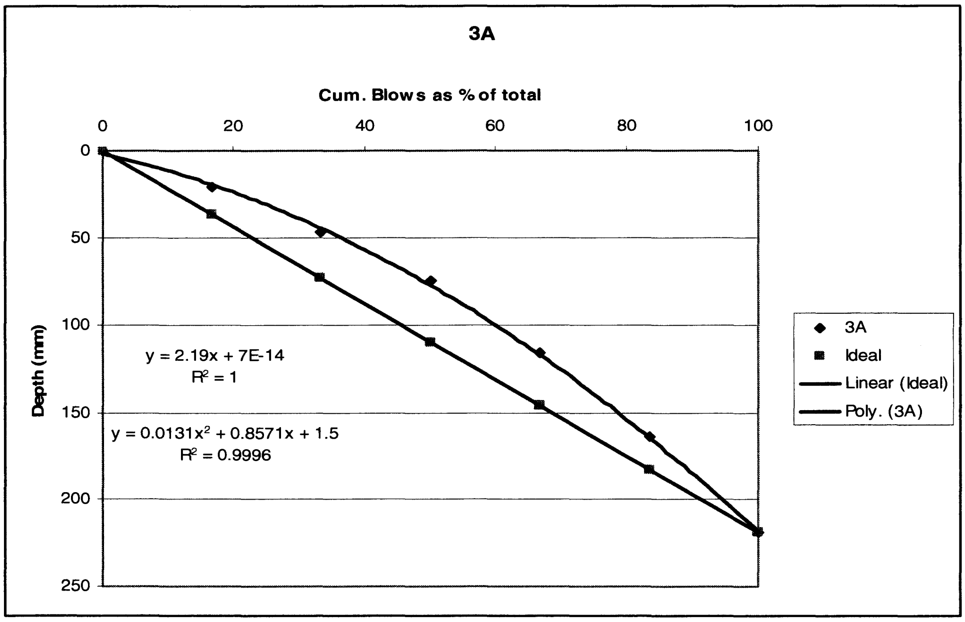
2.Column 8 Actual slope = Change in Depth / Change in Cumulative blows as a % of total blows

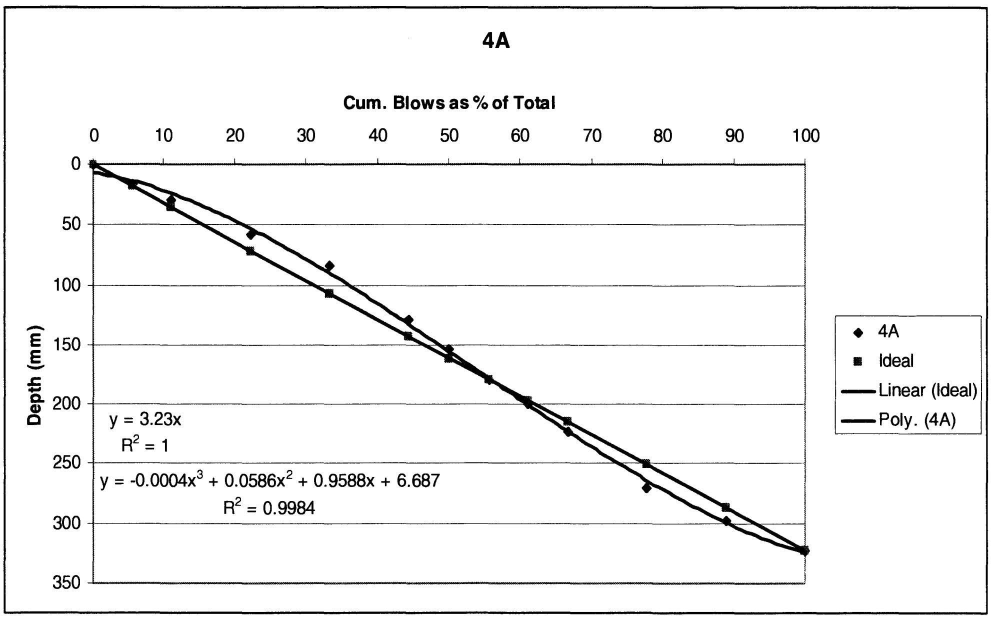
3. Column 9 ideal slope = change in ideal Depth/ change in cumulative blows as a % of total blows

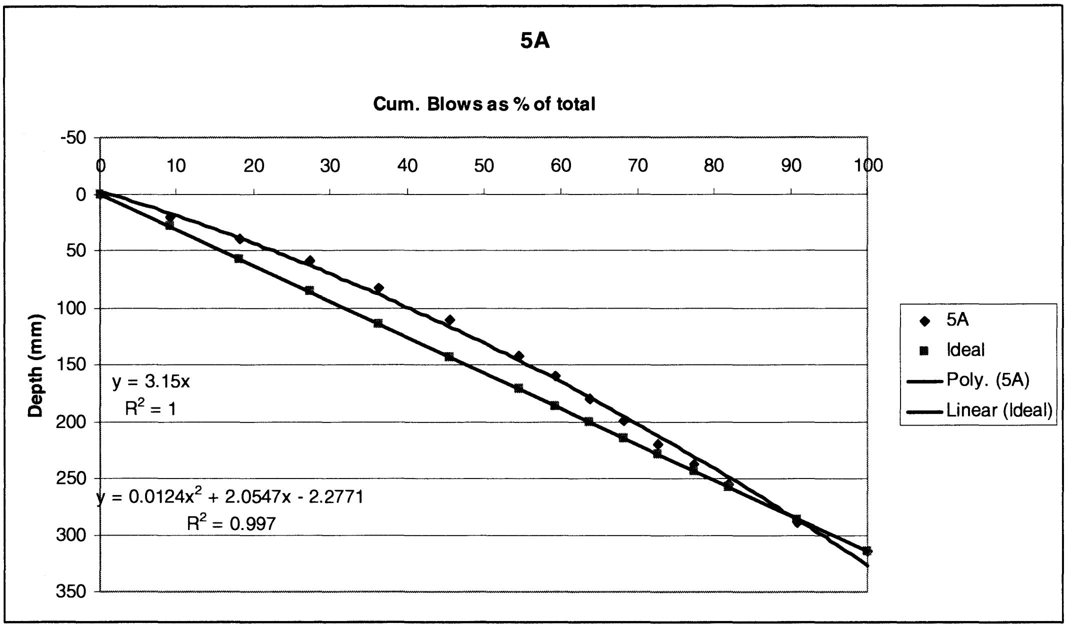
4. difference in the ideal and actual slope slopes

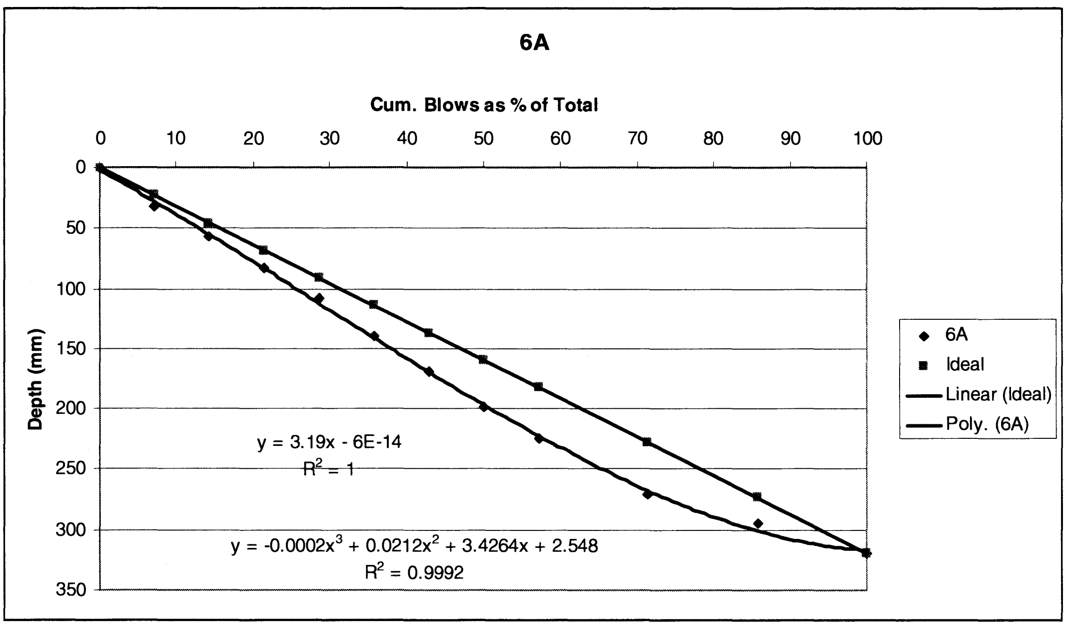

1A
Cum. Blows as % of total
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
Depth (mm)
y = 2.18x
$R^2 = 1$
$y = 0.0074x^2 + 1.4301x + 1.635$
$R^2 = 0.9997$
1A
Ideal
Linear (Ideal)
Poly. (1A)

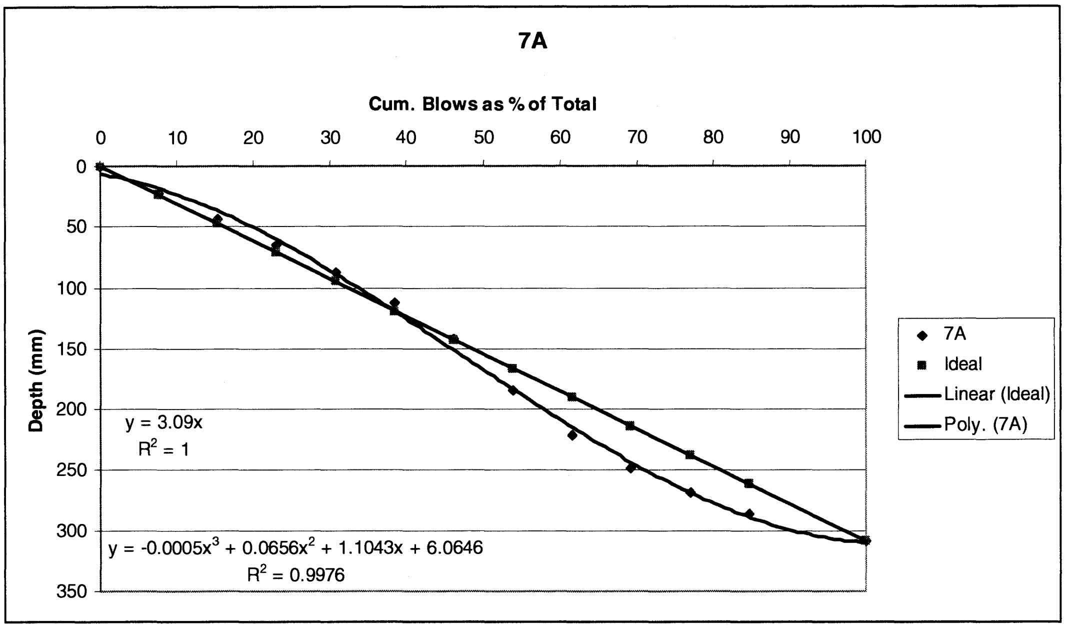

2A
Cum. Blows as % of total
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
Depth (mm)
y = 2.25x - 7E-14
$R^2 = 1$
$y = 0.0093x^2 + 1.3302x + 1.9788$
$R^2 = 0.9985$
2A
Ideal
Linear (Ideal)
Poly. (2A)

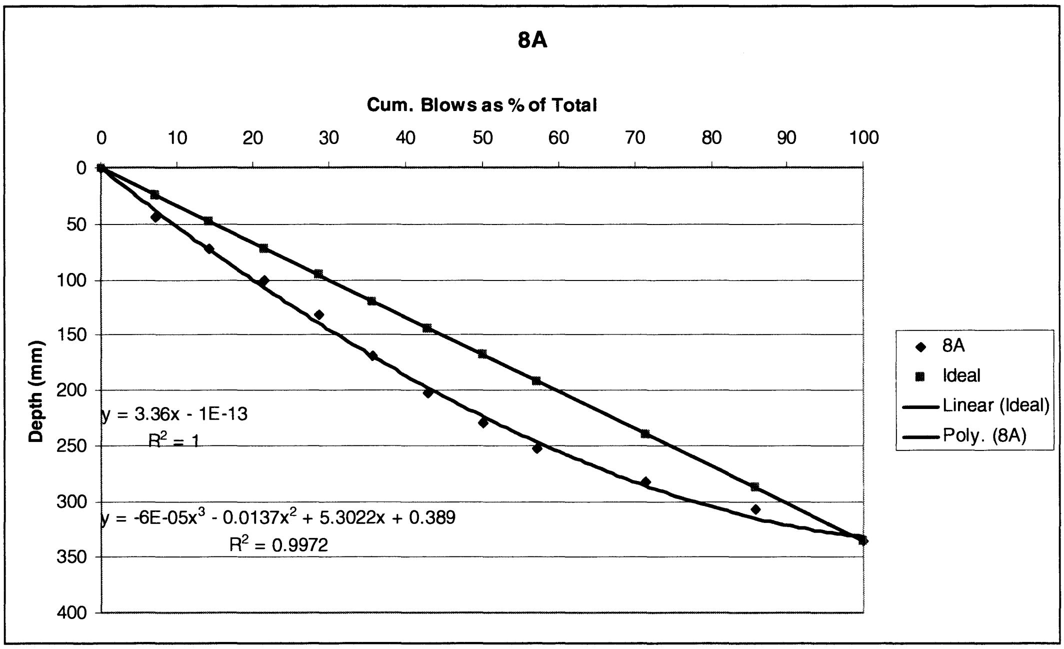

3A
Cum. Blows as % of total
0
20
40
60
80
100
0
50
100
150
200
250
Depth (mm)
y = 2.19x + 7E-14
R² = 1
y = 0.0131x² + 0.8571x + 1.5
R² = 0.9996
3A
Ideal
Linear (Ideal)
Poly. (3A)

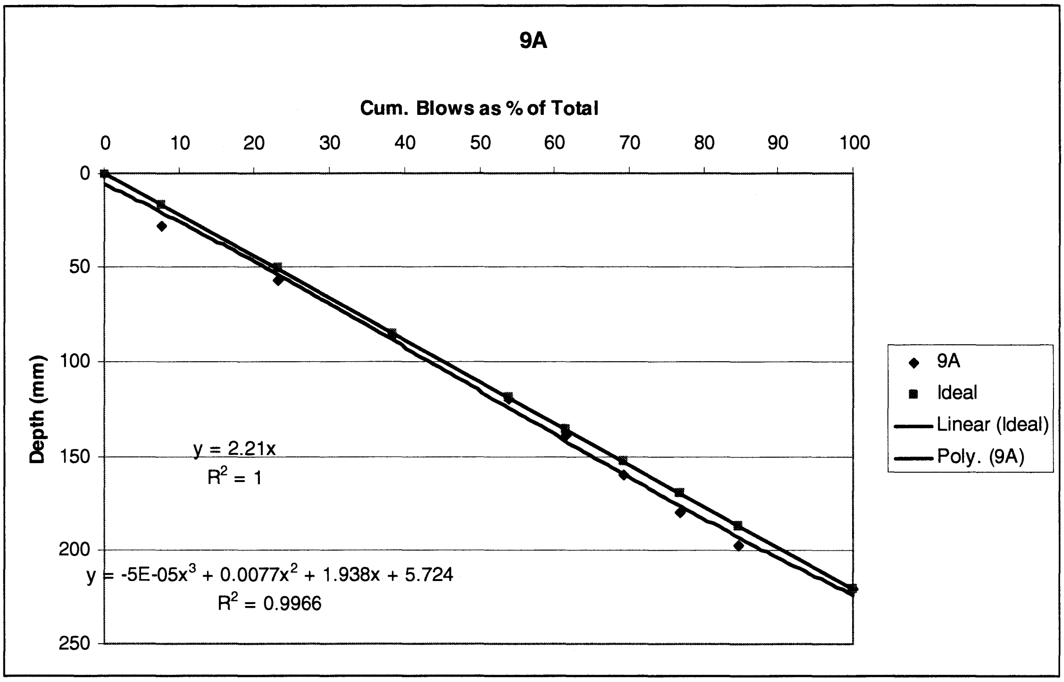

4A
Cum. Blows as % of Total
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
300
350
Depth (mm)
y = 3.23x
R² = 1
y = -0.0004x³ + 0.0586x² + 0.9588x + 6.687
R² = 0.9984
4A
Ideal
Linear (Ideal)
Poly. (4A)

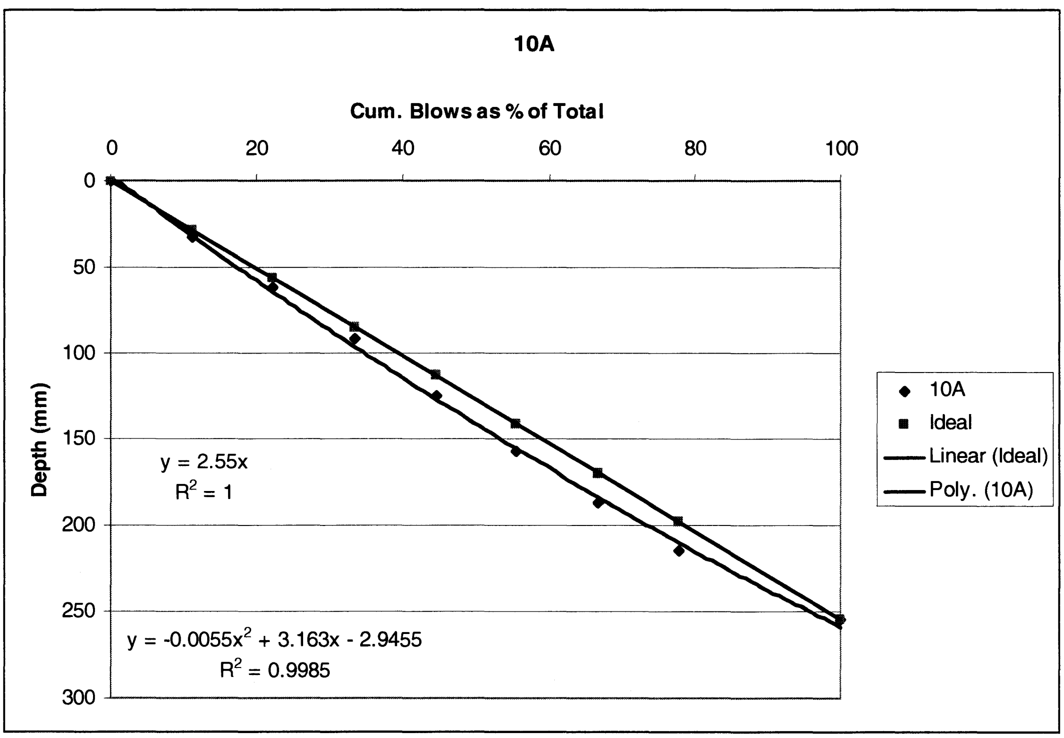

5A
Cum. Blows as % of total
Depth (mm)
y = 3.15x
R² = 1
y = 0.0124x² + 2.0547x - 2.2771
R² = 0.997
5A
Ideal
Poly. (5A)
Linear (Ideal)

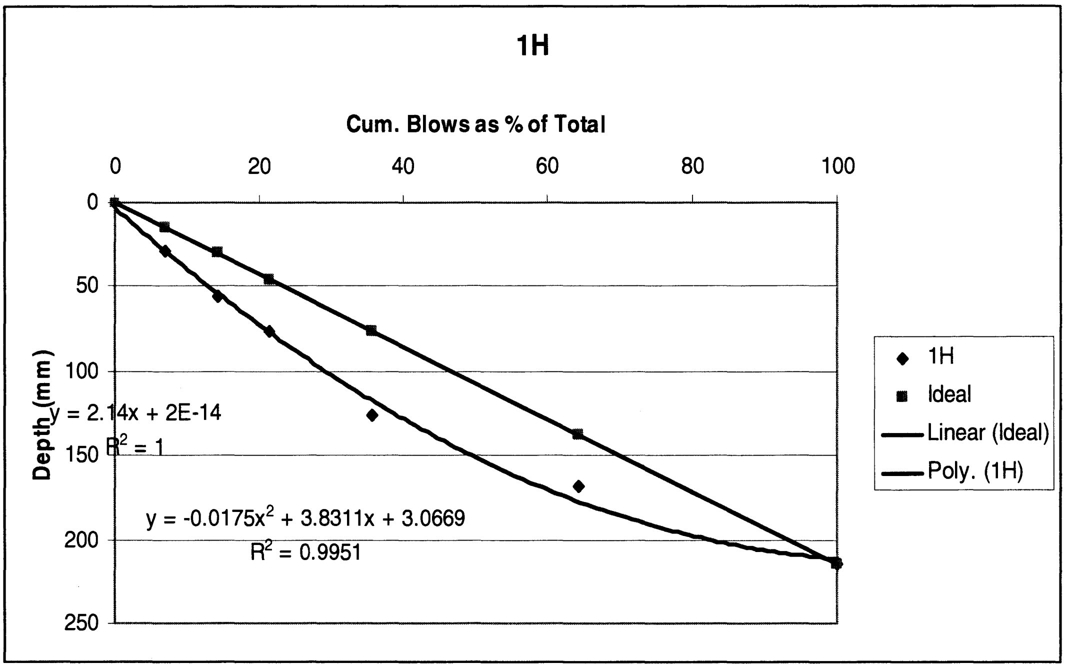

6A
Cum. Blows as % of Total
Depth (mm)
y = 3.19x - 6E-14
R² = 1
y = -0.0002x³ + 0.0212x² + 3.4264x + 2.548
R² = 0.9992
6A
Ideal
Linear (Ideal)
Poly. (6A)

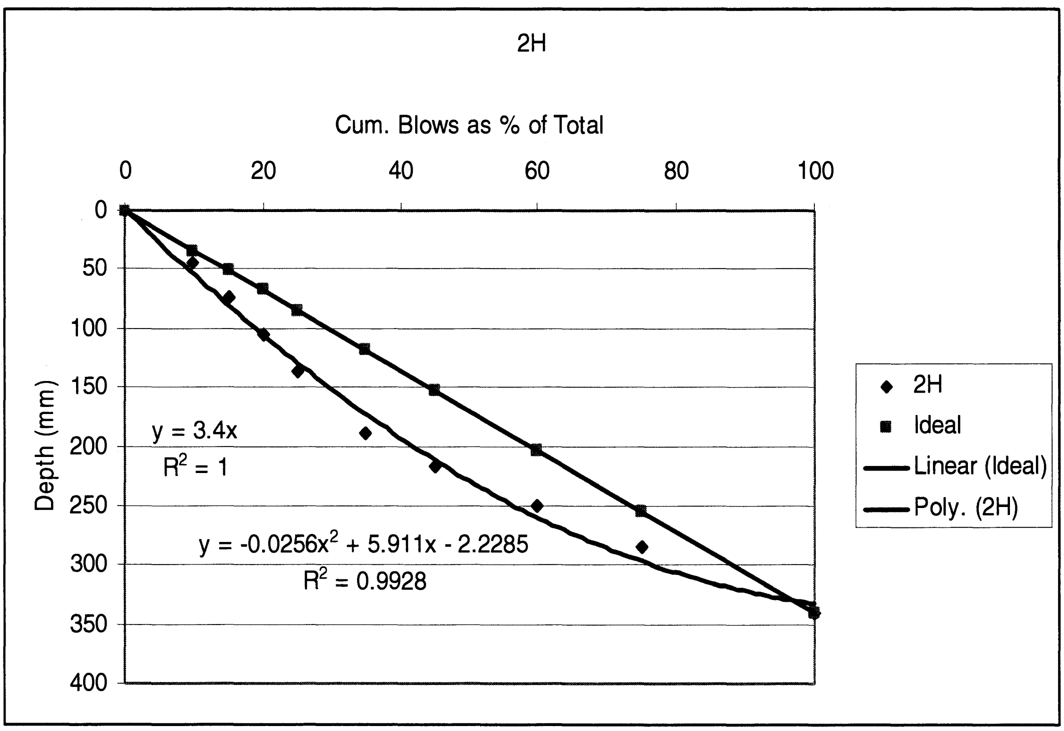

7A
Cum. Blows as % of Total
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
300
350
Depth (mm)
y = 3.09x
R² = 1
y = -0.0005x³ + 0.0656x² + 1.1043x + 6.0646
R² = 0.9976
7A
Ideal
Linear (Ideal)
Poly. (7A)

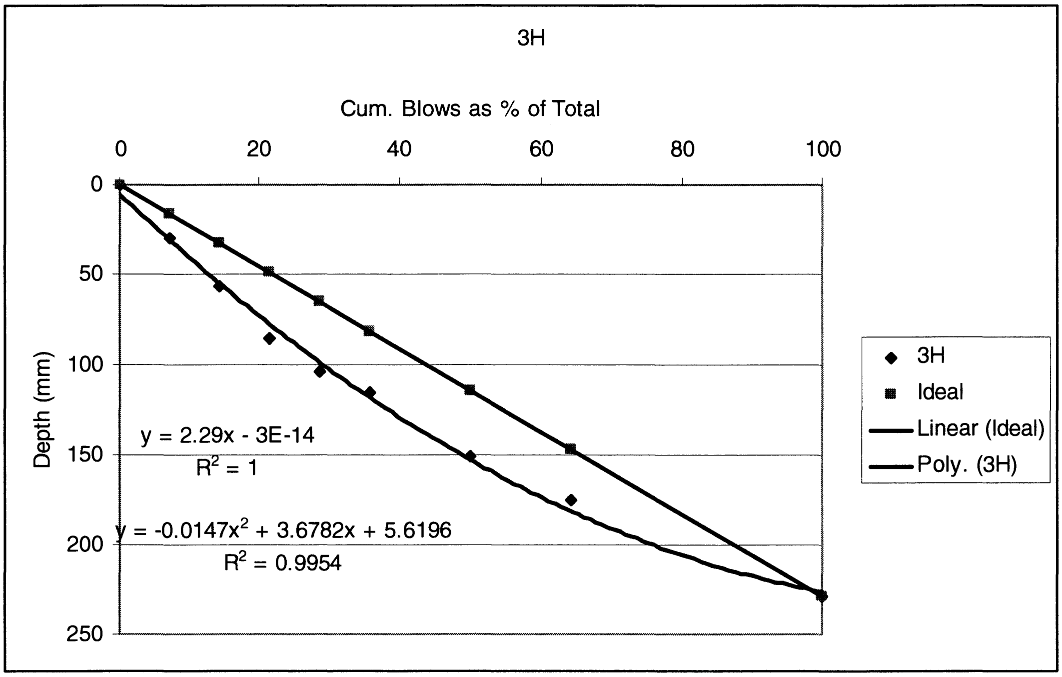

8A
Cum. Blows as % of Total
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
300
350
400
Depth (mm)
y = 3.36x - 1E-13
R² = 1
y = -6E-05x³ - 0.0137x² + 5.3022x + 0.389
R² = 0.9972
8A
Ideal
Linear (Ideal)
Poly. (8A)

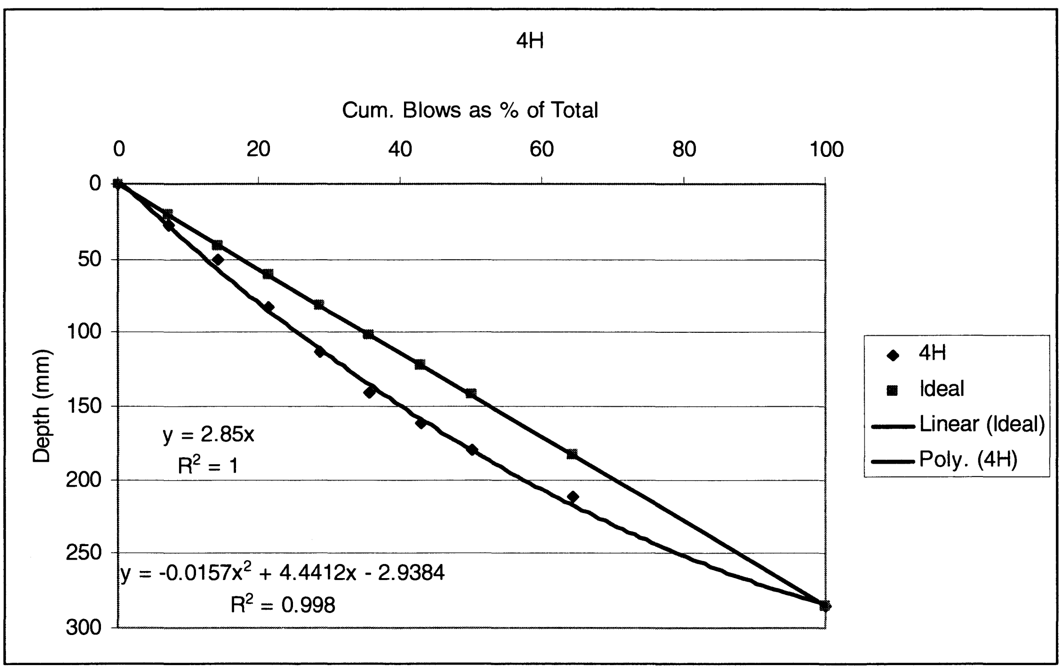

9A
Cum. Blows as % of Total
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
250
Depth (mm)
y = 2.21x
R2 = 1
y = -5E-05x3 + 0.0077x2 + 1.938x + 5.724
R2 = 0.9966
9A
Ideal
Linear (Ideal)
Poly. (9A)

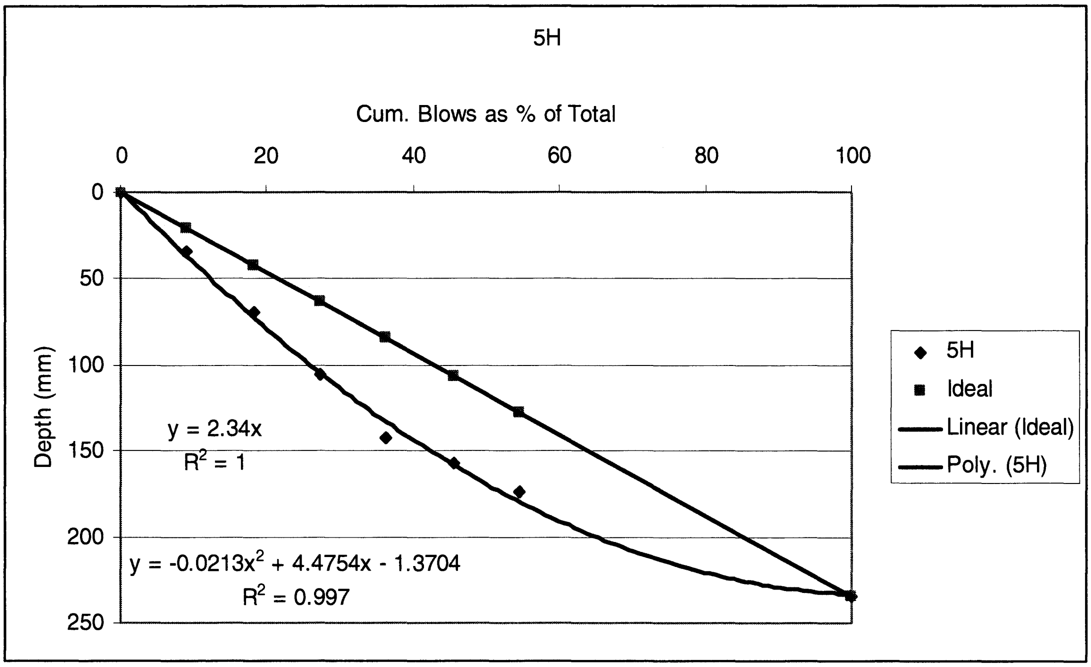

10A
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
300
Depth (mm)
y = 2.55x
R2 = 1
y = -0.0055x2 + 3.163x - 2.9455
R2 = 0.9985
10A
Ideal
Linear (Ideal)
Poly. (10A)

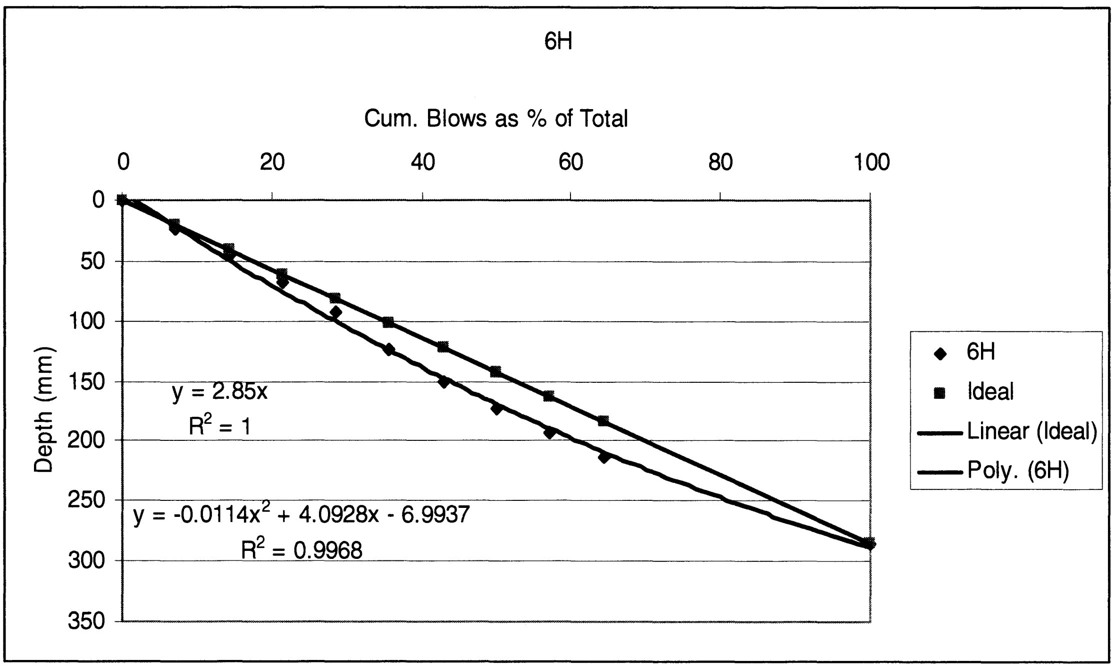

1H
Cum. Blows as % of Total
0
20
40
60
80
100
Depth (mm)
0
50
100
150
200
250
y = 2.14x + 2E-14
R2 = 1
y = -0.0175x2 + 3.8311x + 3.0669
R2 = 0.9951
1H
Ideal
Linear (Ideal)
Poly. (1H)

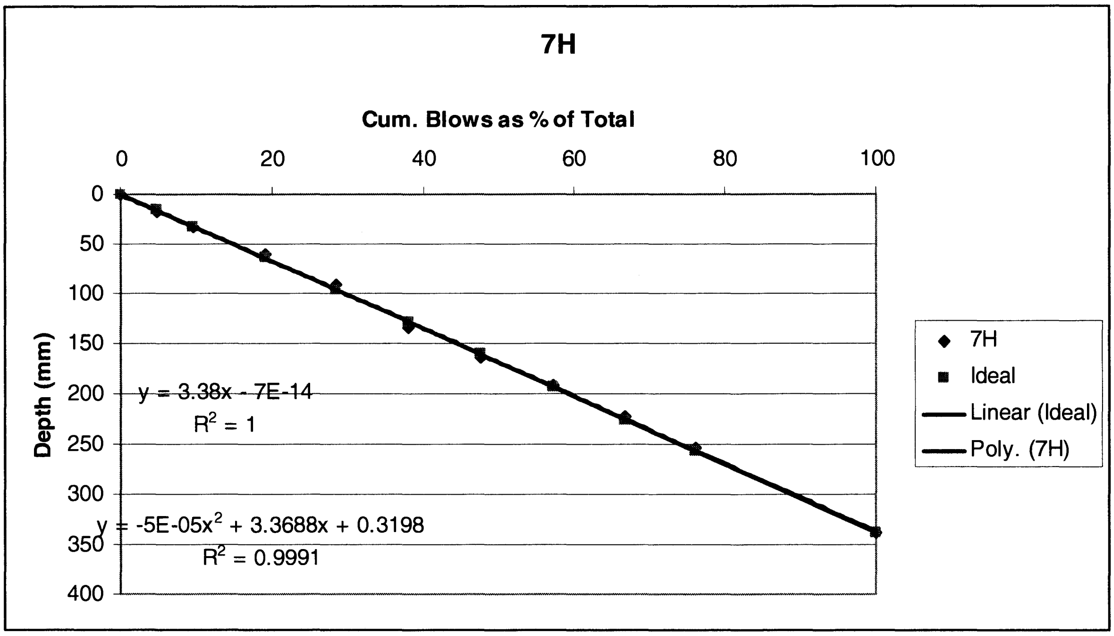

2H
Cum. Blows as % of Total
0
20
40
60
80
100
Depth (mm)
0
50
100
150
200
250
300
350
400
y = 3.4x
R2 = 1
y = -0.0256x2 + 5.911x - 2.2285
R2 = 0.9928
2H
Ideal
Linear (Ideal)
Poly. (2H)

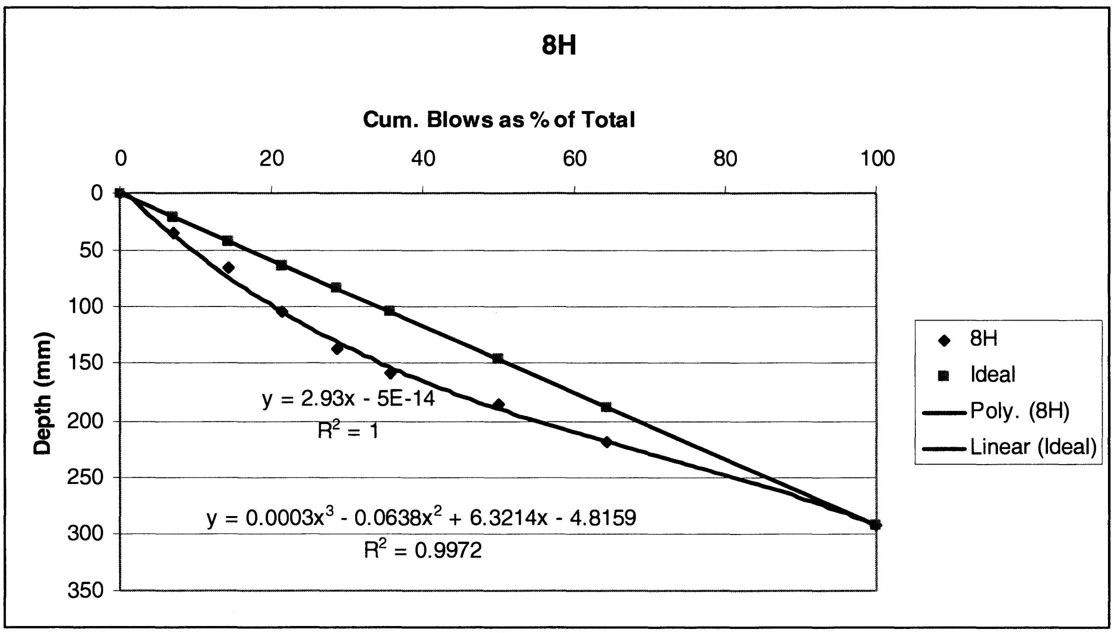

3H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
Depth (mm)
y = 2.29x - 3E-14
R2 = 1
y = -0.0147x2 + 3.6782x + 5.6196
R2 = 0.9954
3H
Ideal
Linear (Ideal)
Poly. (3H)

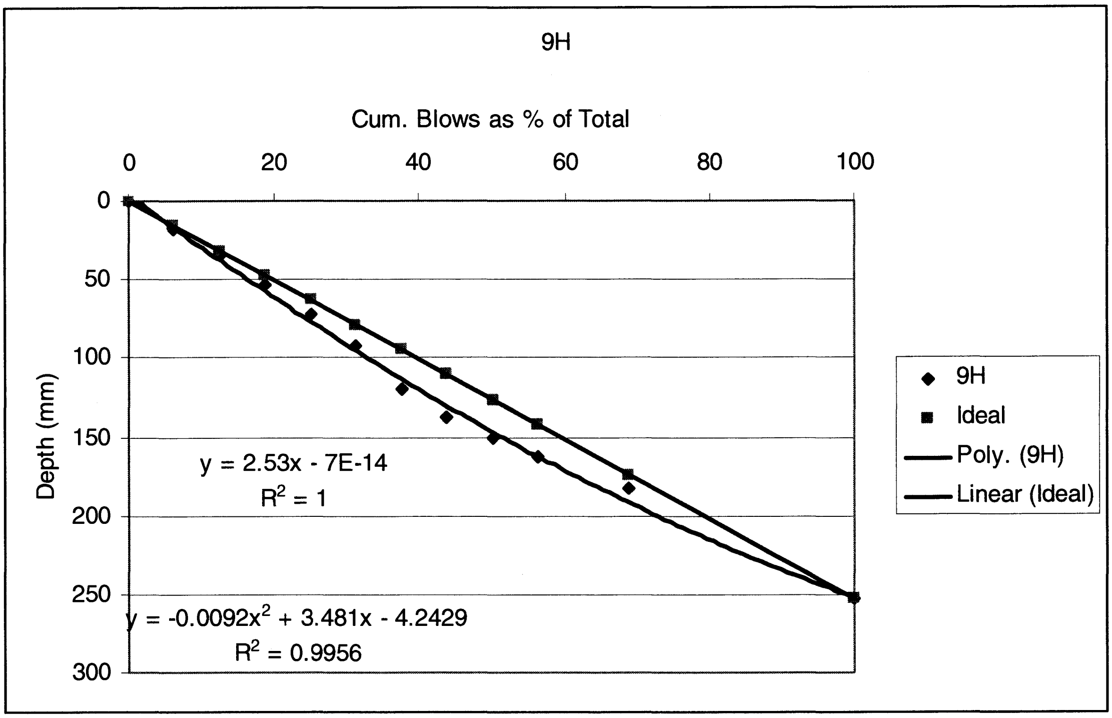

4H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
300
Depth (mm)
y = 2.85x
R2 = 1
y = -0.0157x2 + 4.4412x - 2.9384
R2 = 0.998
4H
Ideal
Linear (Ideal)
Poly. (4H)

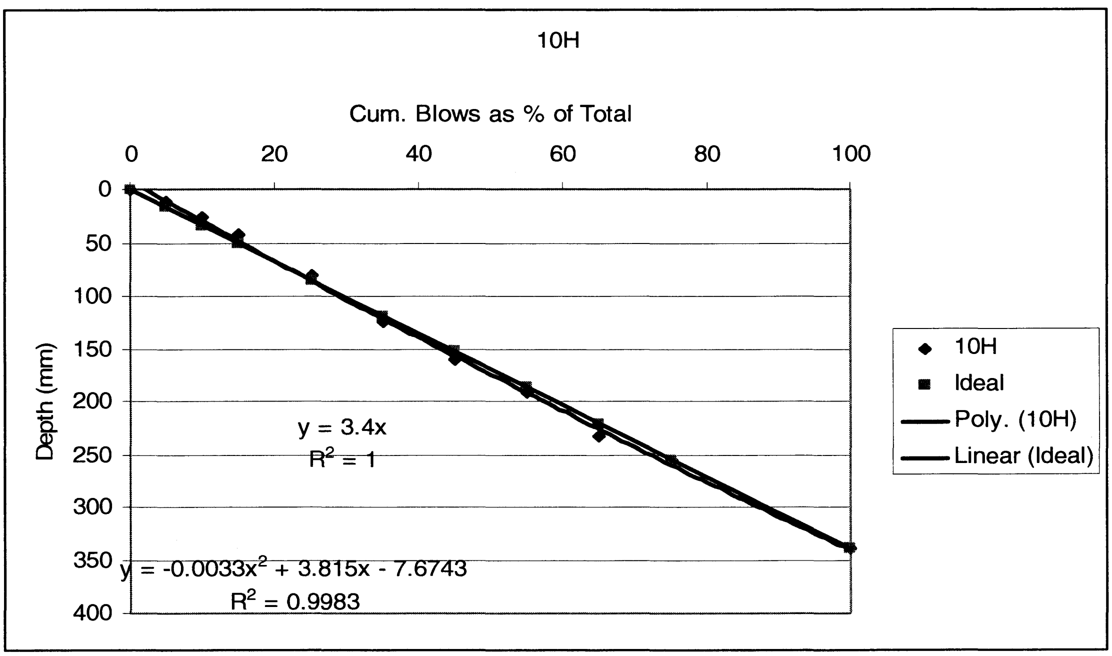

5H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
Depth (mm)
y = 2.34x
R^2 = 1
y = -0.0213x^2 + 4.4754x - 1.3704
R^2 = 0.997
5H
Ideal
Linear (Ideal)
Poly. (5H)

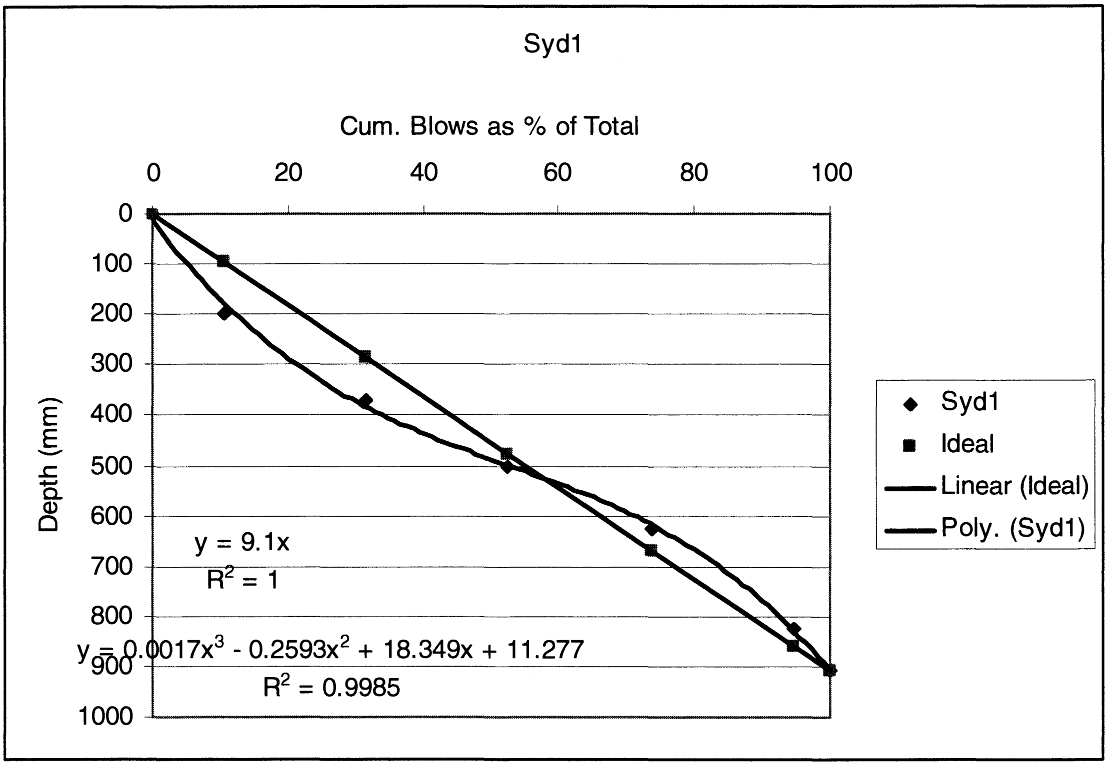

6H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
300
350
Depth (mm)
y = 2.85x
R^2 = 1
y = -0.0114x^2 + 4.0928x - 6.9937
R^2 = 0.9968
6H
Ideal
Linear (Ideal)
Poly. (6H)

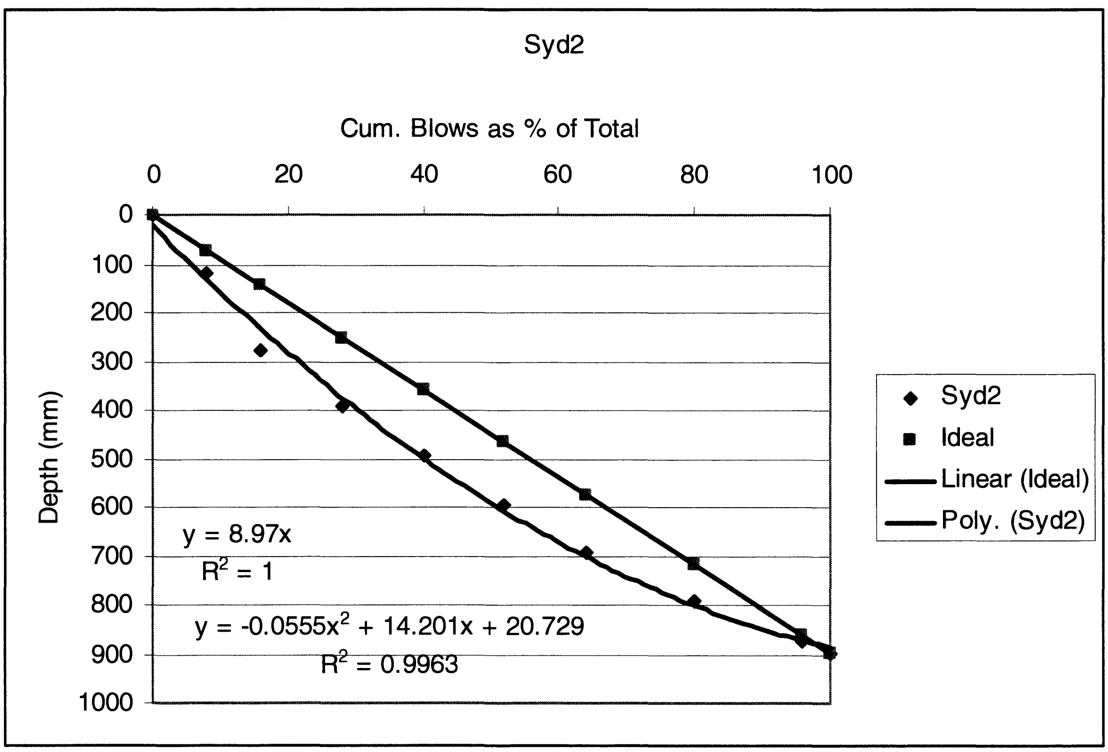

7H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
300
350
400
Depth (mm)
y = 3.38x - 7E-14
R² = 1
y = -5E-05x² + 3.3688x + 0.3198
R² = 0.9991
7H
Ideal
Linear (Ideal)
Poly. (7H)

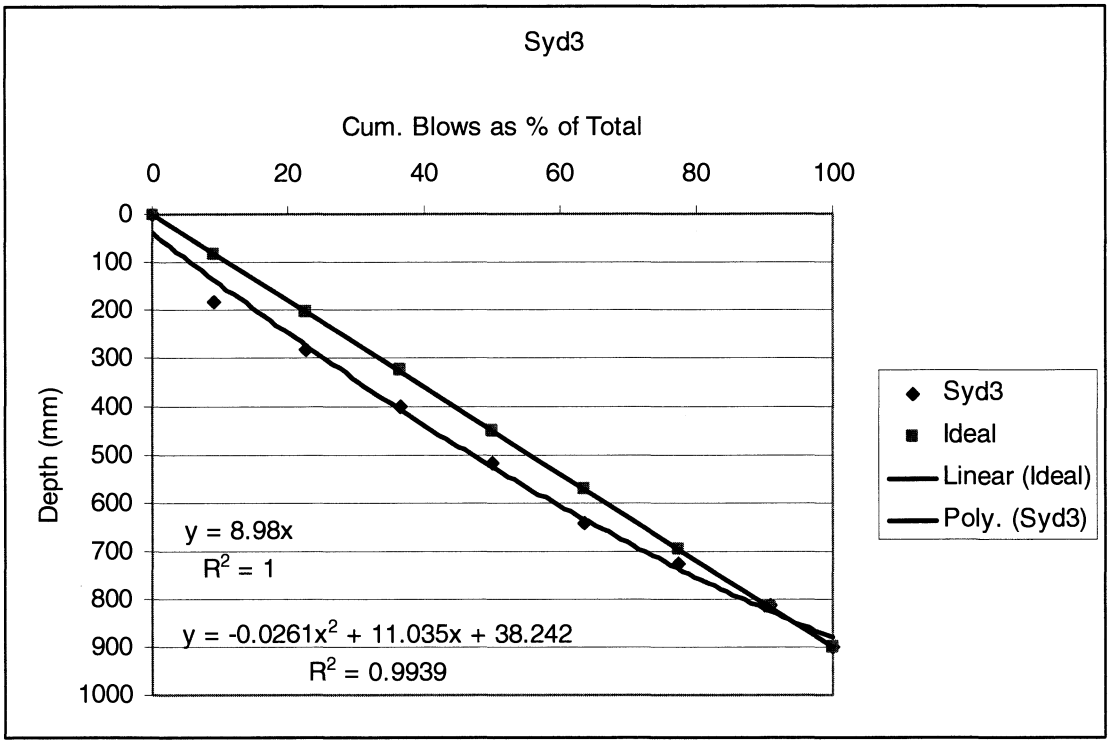

8H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
300
350
Depth (mm)
y = 2.93x - 5E-14
R² = 1
y = 0.0003x³ - 0.0638x² + 6.3214x - 4.8159
R² = 0.9972
8H
Ideal
Poly. (8H)
Linear (Ideal)

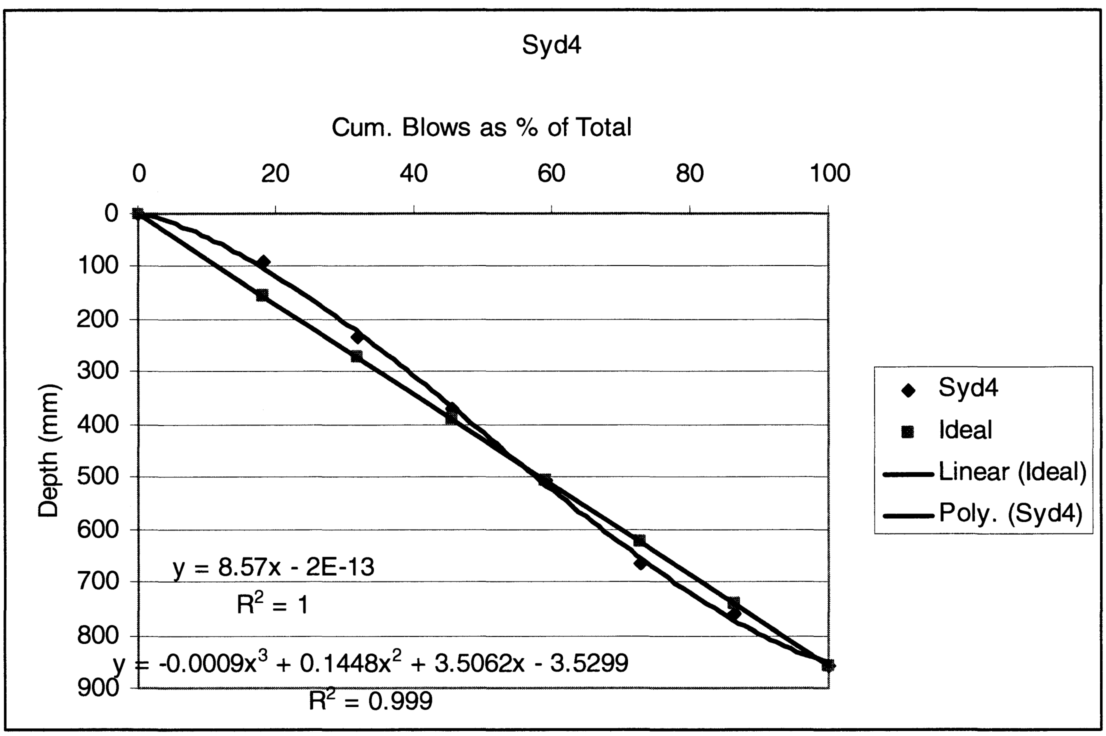

9H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
300
Depth (mm)
y = 2.53x - 7E-14
R² = 1
y = -0.0092x² + 3.481x - 4.2429
R² = 0.9956
9H
Ideal
Poly. (9H)
Linear (Ideal)

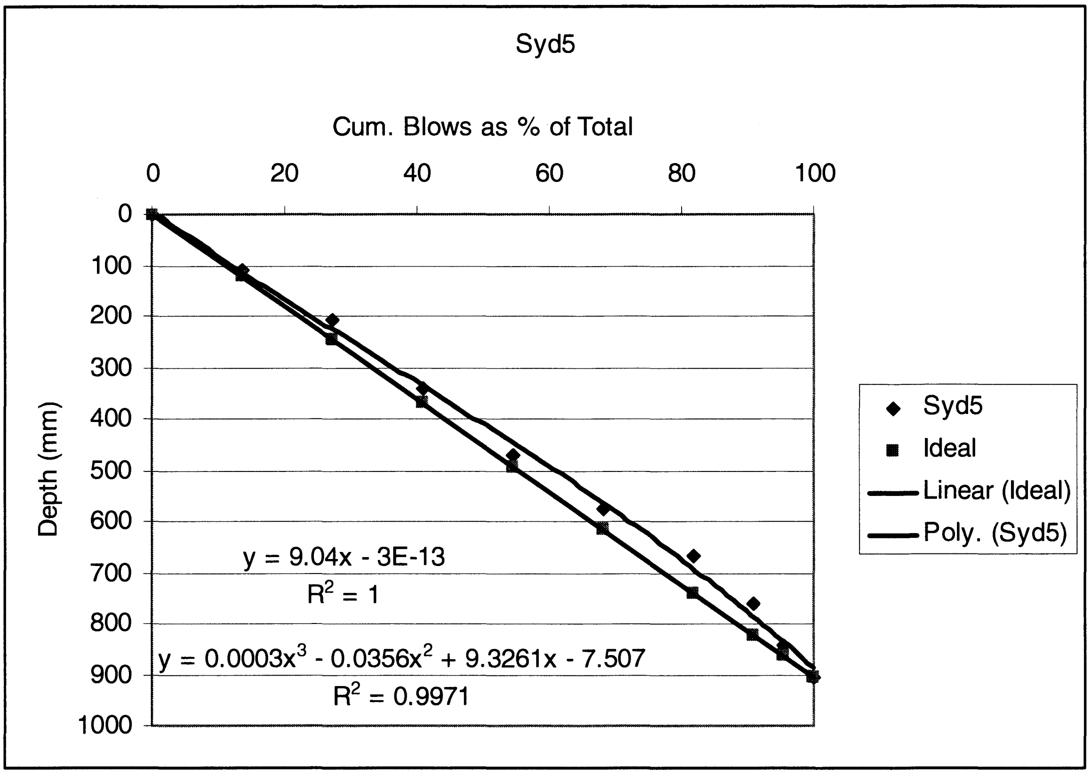

10H
Cum. Blows as % of Total
0
20
40
60
80
100
0
50
100
150
200
250
300
350
400
Depth (mm)
y = 3.4x
R² = 1
y = -0.0033x² + 3.815x - 7.6743
R² = 0.9983
10H
Ideal
Poly. (10H)
Linear (Ideal)

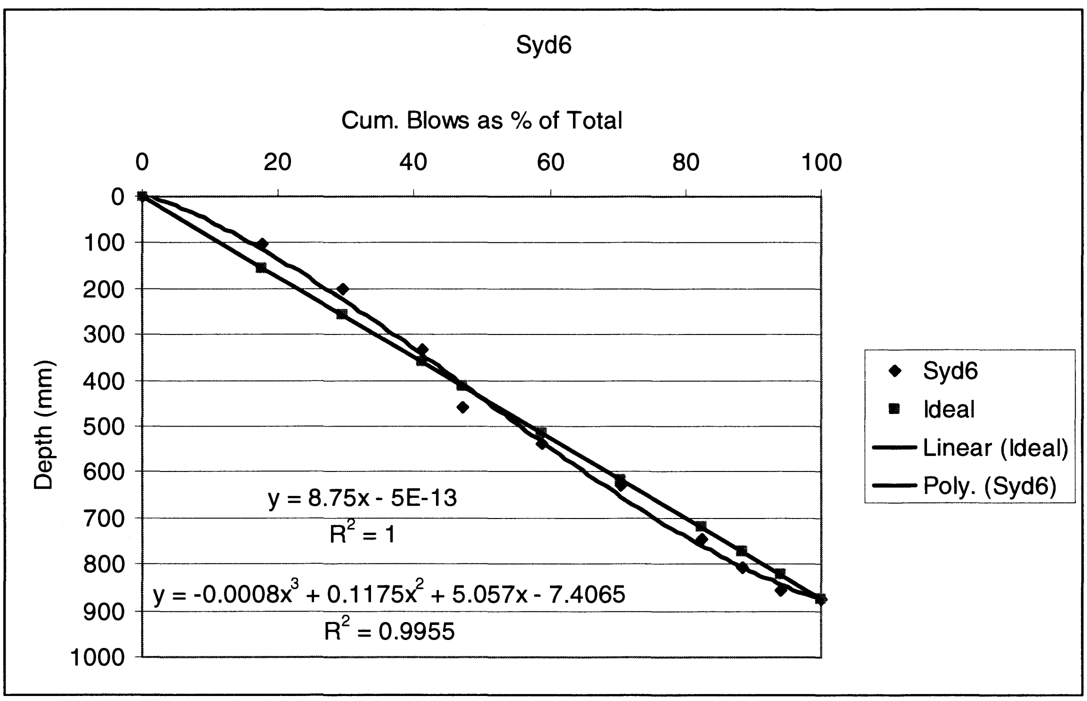

Syd1
Cum. Blows as % of Total
0
20
40
60
80
100
0
100
200
300
400
500
600
700
800
900
1000
Depth (mm)
y = 9.1x
R^2 = 1
y = 0.0017x^3 - 0.2593x^2 + 18.349x + 11.277
R^2 = 0.9985
Syd1
Ideal
Linear (Ideal)
Poly. (Syd1)

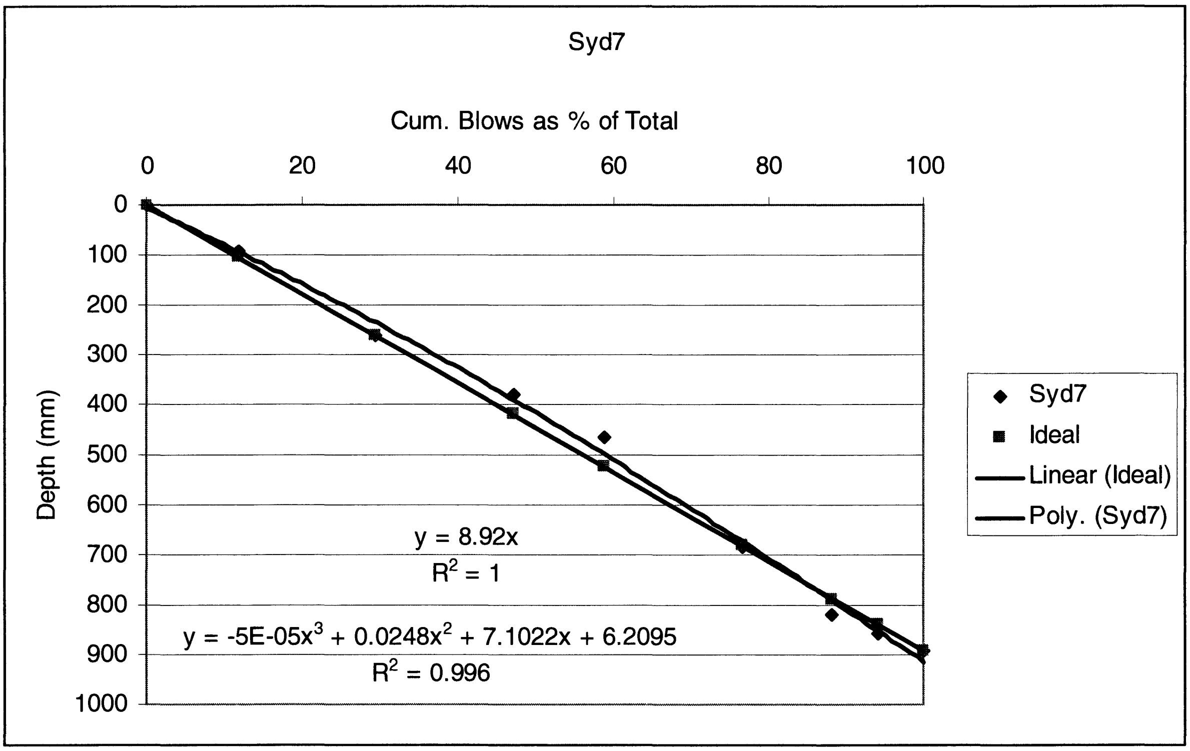

Syd2
Cum. Blows as % of Total
0
20
40
60
80
100
0
100
200
300
400
500
600
700
800
900
1000
Depth (mm)
y = 8.97x
R^2 = 1
y = -0.0555x^2 + 14.201x + 20.729
R^2 = 0.9963
Syd2
Ideal
Linear (Ideal)
Poly. (Syd2)


Syd3
Cum. Blows as % of Total
0
20
40
60
80
100
0
100
200
300
400
500
600
700
800
900
1000
Depth (mm)
y = 8.98x
R^2 = 1
y = -0.0261x^2 + 11.035x + 38.242
R^2 = 0.9939
Syd3
Ideal
Linear (Ideal)
Poly. (Syd3)


Syd4
Cum. Blows as % of Total
0
20
40
60
80
100
0
100
200
300
400
500
600
700
800
900
Depth (mm)
y = 8.57x - 2E-13
R^2 = 1
y = -0.0009x^3 + 0.1448x^2 + 3.5062x - 3.5299
R^2 = 0.999
Syd4
Ideal
Linear (Ideal)
Poly. (Syd4)


Syd5
Cum. Blows as % of Total
0
20
40
60
80
100
0
100
200
300
400
500
600
700
800
900
1000
Depth (mm)
y = 9.04x - 3E-13
R² = 1
y = 0.0003x³ - 0.0356x² + 9.3261x - 7.507
R² = 0.9971
Syd5
Ideal
Linear (Ideal)
Poly. (Syd5)


Syd6
Cum. Blows as % of Total
0
20
40
60
80
100
0
100
200
300
400
500
600
700
800
900
1000
Depth (mm)
y = 8.75x - 5E-13
R² = 1
y = -0.0008x³ + 0.1175x² + 5.057x - 7.4065
R² = 0.9955
Syd6
Ideal
Linear (Ideal)
Poly. (Syd6)



Syd7
Cum. Blows as % of Total
0
20
40
60
80
100
0
100
200
300
400
500
600
700
800
900
1000
Depth (mm)
y = 8.92x
R^2 = 1
y = -5E-05x^3 + 0.0248x^2 + 7.1022x + 6.2095
R^2 = 0.996
Syd7
Ideal
Linear (Ideal)
Poly. (Syd7)

APPENDIX B

Plots of Field DCP Tests

Minimum Requirements to View the DCP Plots	
Software	Microsoft Word
Computer/Processor	Computer with Pentium 133 megahertz (MHz) or higher processor; Pentium III recommended
Memory	RAM requirements depend on the operating system used: ▪ **Windows 98, or Windows 98 Second Edition** 24 MB of RAM plus an additional 8 MB of RAM for Word ▪ **Windows Me, or Microsoft Windows NT®** 32 MB of RAM plus an additional 8 MB of RAM for Word ▪ **Windows 2000 Professional** 64 MB of RAM plus an additional 8 MB of RAM for Word ▪ **Windows XP Professional, or Windows XP Home Edition** 128 MB of RAM plus an additional 8 MB of RAM for Word
Hard Disk	Hard disk space requirements will vary depending on configuration; custom installation choices may require more or less. Listed below is the minimum hard disk requirement for Word: ▪ 150 MB of available hard disk space An additional 115 MB is required on the hard disk where the operating system is installed. Users without Windows XP, Windows 2000, Windows Me, or Office 2000 Service Release 1 (SR-1) require an extra 50 MB of hard disk space for System Files Update.
Operating System	Windows 98, Windows 98 Second Edition, Windows Millennium Edition (Windows Me), Windows NT 4.0 with Service Pack 6 (SP6) or later,* Windows 2000, or Windows XP or later.
Drive	CD-ROM drive
Display	Super VGA (800 × 600) or higher-resolution monitor with 256 colors
Peripherals	Microsoft Mouse, Microsoft IntelliMouse®, or compatible pointing device