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QA/QC of Subgrade and Embankment Construction: Technology Replacement and Updated Procedures

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16. Abstract

The Dynamic Cone Penetrometer (DCP) is a device that is used for the estimation of *in situ* compaction quality of constructed subgrades and embankments. It is a relatively inexpensive, light-weight and easy to use device that measures the dynamic penetration resistance of the compacted soil, from which an estimate of soil strength and stiffness characteristics can be made. Owing to its ease of use, many DOTs in the U.S. have employed the DCP in their compaction quality control procedures, and over the past few decades, extensive research has been carried out on the development of correlations between the results of the DCP test and the results of strength and stiffness tests performed on compacted soils (e.g., California bearing ratio, and resilient modulus)

The objectives of this research are to refine DCP-based quality assurance and quality control correlations for compaction quality control developed by previous research studies carried out at Purdue for the Indiana Department of Transportation, especially focusing on i) grouping of the soils based on their mechanical response to the DCP loading, and ii) limiting the *in situ* moisture range of the soils used for development of correlations within -2% of the optimum moisture content of the tested soil. The factors outlined above are studied, and in particular, soil grouping is examined critically. The AASHTO ('A-based') classification employed previously for classification of soils is replaced by a new classification criteria specifically developed for the DCP test. Soils are grouped into one of the two categories of coarse-grained or fine-grained soils on the basis of the size of the dominant particle in the soil. The criteria developed for the classification of soil into one of these two categories is based on index properties of the soil, such as the standard Proctor maximum dry density, optimum moisture content, plasticity index (PI) and fines content (percentage passing0.075 mm sieve size).

For the purpose of refinement of the QA/QC correlations, extensive field and laboratory tests (more than 750 DCP tests) were carried out on soils found in Indiana to add to the existing database of DCP test results. The database was then statistically analyzed for extraction of the representative DCP test value (number of DCP blows required for a specific depth of penetration into the compacted soil) for different types of soil.

Results show that the DCP test results for fine-grained soils have a good correlation with the PI, which is indicative of the clay content of the soil, while the DCP test results for coarse-grained soils have good correlations with the optimum moisture content of the soil, which is indicative of the targeted *in situ* density of the soil. Furthermore, a statistical analysis of the distribution of DCP blow counts in the field revealed that the mean of a minimum of 7 closely spaced tests is required to get a representative blow count of the compacted soil at a given location. More targeted testing is needed to assess the frequency of DCP testing required for larger areas.

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EXECUTIVE SUMMARY

QA/QC OF SUBGRADE AND EMBANKMENT CONSTRUCTION: TECHNOLOGY REPLACEMENT AND UPDATED PROCEDURES

Introduction

The dynamic cone penetrometer (DCP) is a simple tool for assessing the quality of the compaction at a jobsite. Not only is the DCP light and easy to transport and use, its use is also not limited by the availability of power. Once proper correlations are established, quality assurance and quality control (QA/QC) procedures are fairly easy to administer.

QA/QC tests for compacted soils have in the past been limited to density tests, which have certain shortcomings. Performance-based tests, such as the DCP, are used to ensure that compaction quality is achieved and that the compacted soil satisfies a certain minimum level of performance. Research on the development of various correlations for the DCP has been carried out over the past decade. Correlations have been developed between the DCP blow count and quality control parameters, such as California bearing ratio (CBR), resilient modulus (MR) and relative compaction. As a result, the DCP has gained more acceptance as a QA/QC testing device by the construction industry.

The DCP can be used in tandem with other means of quality control to ensure the construction of a well-built subgrade or embankment. However, as in the case with other tests, the statistical variability associated with the test results needs to be accounted for in the process of development of QA/QC correlations.

In this research study, in order to develop suitable QA/QC correlations for the DCP blow count values and the compaction quality of the subgrade, soils were classified into two main categories: coarse-grained (or sand-dominated soils) and fine-grained (or clay-dominated soils). For soils with clay contents greater than about 20%, the behavior of the soil is similar to that of clay. To account for all the governing factors that control the mechanical behavior of soil and its response to DCP loading, the decision as to which category a soil belongs—sand-dominated soil or clay-dominated soil—was based on the compaction characteristics, plasticity index (PI) and fines content of the soils.

Findings

The DCP blow counts for coarse-grained soils had a good correlation with the optimum moisture content (OMC) obtained from standard Proctor compaction tests performed in the field or in the laboratory. The blow counts for penetration of 0 to 12 inches decreased when the OMC of the soils (compacted to at least 95% relative compaction) increased.

The demarcation between manufactured and natural coarse-grained soils was also clear. Correlations were developed for manufactured coarse-grained soils with respect to both the OMC and the coefficient of uniformity.

The DCP blow counts for fine-grained soils, with fabric dominated by clay-size particles, had a very good correlation with the plasticity index (PI) of the soil. The PI is indicative of the clay content of soils and its value depends on the clay mineral or proportions of the clay minerals in the clay phase of the soil mass. Equations were developed for 0- to 6-inch and 6- to 12-inch penetration of the DCP.

The soil classification criteria and the accompanying DCP correlations developed in this research for quality control of compaction were found to be suitable to establish QA/QC procedures for compacted soil and received positive feedback from INDOT engineers.

Based on the statistical analysis of the DCP test data, it was determined that the mean of seven DCP tests performed at a given location provides a reasonable estimate of the actual mean of the compacted soil mass, assuming that the DCP test results follow a normal distribution. In addition to small-scale DCP testing, large-scale testing was also investigated, and a preliminary procedure for large-scale testing was proposed. Further work still needs to be carried out in the direction of establishment of the scale of fluctuation of compacted soils to further refine the preliminary large-scale testing procedure proposed in this research.

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1. INTRODUCTION

1.1 Background

Pavements typically consist of four main layers: (1) the surface course, which comes in direct contact with the vehicles and may be rigid or non-rigid, (2) the base course, which provides drainage and frost protection, (3) the subbase layer, which provides further load distribution and (4) the subgrade, which basically is soil in its natural state or modified to satisfy specified requirements (Christopher, Schwartz, & Bourdeau, 2006). Figure 1.1 shows a schematic of the cross section of the pavement layers. Out of the four main layers of a typical pavement, the subgrade is the most variable in composition and sensitive to changes in moisture conditions.

To ensure the construction of a sound and durable pavement, it is of paramount importance that the subgrade be capable of bearing the loads to be expected from construction activities and traffic during the lifetime of the pavement. While it is preferable to construct pavements on ground that is naturally stiff, homogeneously incompressible and impermeable, it is extremely unlikely to find such ideal conditions in the field. It is because of this fact that the first step in pavement construction is always some form of ground improvement.

Compaction is one of the most prevalent forms of ground improvement practiced in highway construction. Compaction is defined as the densification of soil by application of mechanical energy resulting in the removal of air voids from the soil matrix, or in simple terms, it is the process of making a soil uniformly dense using a wide spectrum of techniques.

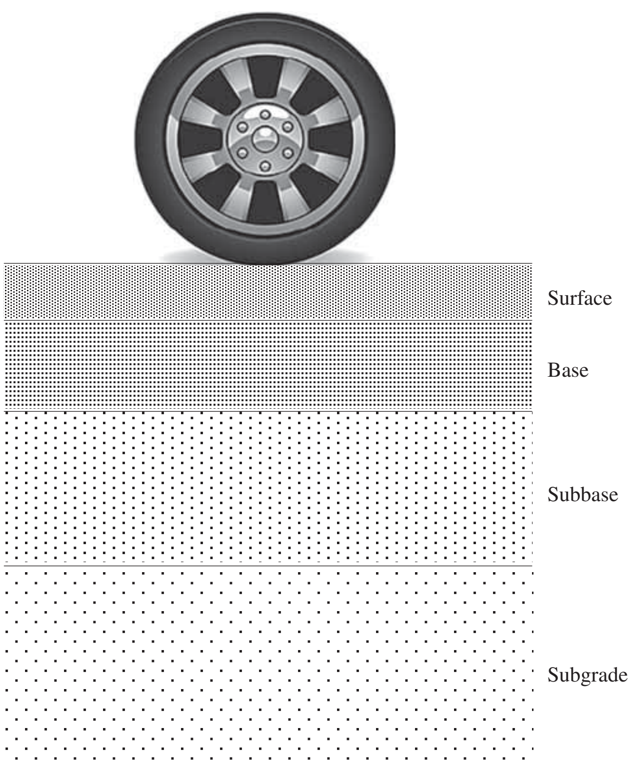

Figure 1.1 Schematic of pavement layers.

Historically, the process of compaction of soil was used in India and China, intentionally or unintentionally, in the construction of walls, levees and dams when the people constructing these earth structures trampled on the dumped soil and in the process densified it (Holtz, Kovacs, & Sheahan, 2010). However, it was only in the past century or so that intensive research was done on the compaction characteristics of different soils to better understand and control the process of compaction.

Compaction is necessary in subgrade construction because it increases soil density, homogenizes the soil, reduces the permeability of the soil and, in a broad sense, improves the mechanical response of the soil, making it more amenable to design considerations. In the past century or so, the process of compaction has improved significantly with use of specialized equipment that takes into account the different compaction characteristics of various types of soil.

Since the late 1880s, pavement made of asphalt or concrete have been in use in the U.S. (Christopher et al., 2006). In the late 1920s, the U.S. took interest in the construction of a diverse roadway network spanning the nation and, given the manpower and financial input involved in such large-scale projects, specifications and quality checks became imperative to the effort. To this end, quality assurance (QA) and quality control (QC) guidelines were developed and adopted by federal, state and private agencies responsible for the construction of these highways. Today, compaction QA/QC criteria form an integral part of the pavement construction process.

QA criteria, in the form of processes and end-product specifications are set and administrated by design engineers, focusing on the process of construction of the pavements. QC, in the form of testing specifications checked by the contractor, focus on the end product (the constructed subgrade) and the minimum requirements (also called *end-product specifications*) in terms of density, strength and stiffness that must be met by the compacted soil for it to be acceptable for construction of subsequent pavement layers.

With respect to the construction of subgrade, quality control is enforced by use of destructive or non-destructive tests that check whether the *end-product specifications* have been achieved (Holtz et al., 2010). These *end-product specifications* are often provided in terms of the *relative compaction* (RC) achieved in the field, which is defined as the ratio of the dry density of the subgrade, measured in the field using various techniques, to the maximum dry density established from laboratory tests performed on representative soil samples in accordance with ASTM D-698 and ASTM D-1557 or AASHTO T 99 and AASHTO T180:

$$RC(\%) = \frac{\text{Field dry density}}{\text{Maximum dry density}} \times 100 \tag{1.1}$$

Most state Department of Transportations (DOTs) in the U.S. prescribe to RC values in the range of 95–100% (Illinois Department of Transportation, 2004; Indiana Department of Transportation, 2012; Iowa Department of Transportation, 2012; Minnesota Department of Transportation 2014; New York Department of Transportation, 2008; South Dakota Department of Transportation, 2004). To enforce quality control criteria, *in situ* field density measurements are carried out by either destructive or non-destructive testing. Destructive tests methods, such as the sand cone test and balloon test, have been traditionally used to estimate the *in situ* soil density of the compacted subgrade, but are not preferred as they are time consuming and cause damage locally to the constructed subgrade. Nowadays, with the development of more sophisticated technology, subgrade quality control practices have moved towards non-destructive testing. By virtue of being accurate (Noureldin, Zhu, & Harris, 2005), less damaging to the subgrade and quicker than most destructive methods, many DOTs have started to transition from destructive to non-destructive methods for quality control testing, while retaining some destructive testing methods as a means of cross checking results from the non-destructive tests. Nuclear gauges (NG), time domain reflectometers (TDRs), falling-weight deflectometer (FWDs) and dynamic cone penetrometers (DCPs) are some of the most widely used devices for non-destructive testing of subgrade.

The focus of this research is the use of the DCP for quality control of subgrade construction. Figure 1.2 shows a schematic of the device of interest. The device has a falling weight (hammer) attached to the end of a shaft with a cone at the tip. The DCP probe penetrates the subgrade due to the impact (called a blow) of the falling weight on the anvil. Based on the resistance to penetration (resistance to penetration may be defined as the number of blows required for a specific depth of penetration or the penetration per unit blow) offered by the subgrade to the DCP probe, an estimation of the strength and stiffness characteristics of the

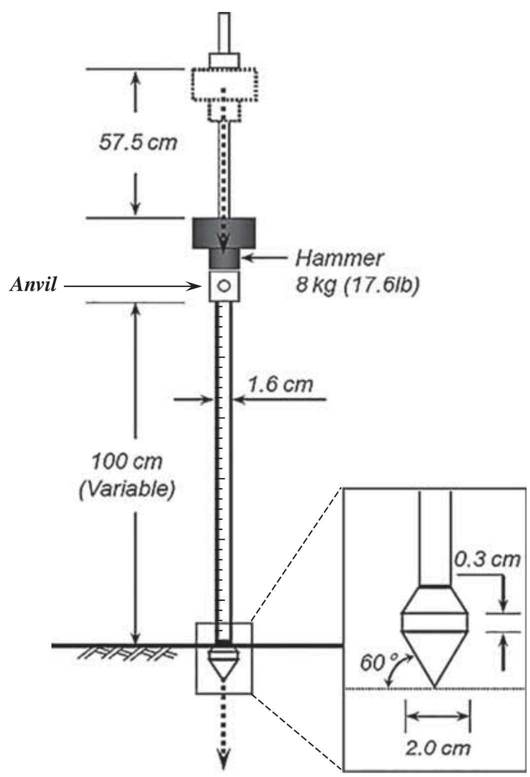

Figure 1.2 The dynamic cone penetrometer (DCP) (after Kim et al., 2010).

subgrade can be made based on previously established correlations. A more detailed description of the device and its specifications for use in subgrade construction quality control will be provided in subsequent chapters.

1.2 Problem Statement

The objective of this research was to develop QA/QC correlations for the DCP that will be applicable for all types of soils found in Indiana and to address some of the issues associated with soil grouping and moisture sensitivity in previously developed correlations by Kim, Prezzi, and Salgado (2010). As many other DOTs before it, when INDOT decided that a move towards use of the dynamic cone penetration test (DCPT) for compaction quality control was potentially in its interest, it funded preliminary projects (Kim et al. 2010; Luo, Salgado, & Altschaeffl, 1998; Salgado & Yoon, 2003) to assess the viability of using the DCPT for this purpose and to develop a methodology to do so reliably. Data was collected and organized from field and laboratory test results aiming at establishing the basis for the development of proper correlations between DCPT results and end-product specifications, as adopted by INDOT. It was understood that the same blow count implied different things depending on the soil the test was performed in and the state of the soil during field testing (compaction state and moisture content). INDOTs interest at the time was to use the AASHTO “A-based” soil classification system.

The data collected strongly indicated that reasonable correlations could be developed between the DCPT blow count required to satisfy INDOTs relative compaction criteria and the controlling soil properties that affect the mechanical response of the soil to the loading by the DCP (Kim et al., 2010). The most important aspects that needed further research were: (1) identification of the main soil groups that showed similar response to the impact loading from the DCP, (2) identification of an individual or a combination of controlling properties that govern the mechanical response of each soil group to loads applied by the DCP, and (3) quantification of the effect of the moisture content of the subgrade soil after compaction on the DCP blow count measured.

Although the criteria previously established by Kim et al. (2010) did remarkable work in addressing many of the issues associated with the establishment of DCPT blow-count criteria for compacted subgrade, further research was still needed on the three aspects outlined above. Collection of a wide-ranging set of field and laboratory test data was required to augment the data set collected in previous studies and in the development of a more reliable DCP-based quality control criteria. Therefore, based on the outcomes of the previous studies (Chen, Wang, & Bilyeu, 2001; Kim et al. 2010; Salgado & Yoon, 2003) the objectives of this research were defined as: (1) refinement of the correlations established by Kim et al. (2010), focusing on grouping of the soils based on their response to the DCP loading while limiting the *in situ* moisture range of the soils used for development of correlations to +1% to −2% of the optimum moisture content (OMC) of the soils tested and (2) assessment of the frequency of testing required in the field for proper compaction quality control and assurance. In particular, soil grouping was reviewed critically and the AASHTO classification presently employed for purpose of grouping of soils was revisited.

1.3 Organization of the Report

The report has been organized into 8 chapters. Chapter 2 focuses on the literature review, detailing the subgrade and embankment construction process, description of the DCP equipment and the previous correlations developed for its use in compaction quality control. Chapter 3 outlines the research approach and statistical analysis procedure developed for analysis of field DCP blow count criteria. Chapter 4 and 5 present the results of field and laboratory tests performed during the course of the study. Chapter 6 focuses on the development of QA/QC correlations for the DCP and describes the effect of fabric and moisture content on the mechanical response of the soil to DCP loads. Chapter 7 looks into the small scale and large scale variability associated with the DCP testing and frequency of testing required for quality control. Chapter 8 finally presents a summary of the results, conclusions and further research avenues.

2. LITERATURE REVIEW

This chapter focuses on compaction processes and procedures, detailing the need and importance of QA/QC for subgrade construction and the use of Dynamic Cone Penetrometer for QA/QC.

2.1 Subgrade and Embankment Construction

In most compaction projects in earthwork practice, a generic procedure leading up to the construction of compacted subgrade is followed (Holtz et al., 2010). A design problem, such as the construction of an embankment or a subgrade, is received from a client and subsequently a suitable fill material (usually available *in situ* soil) is identified on the basis of the required engineering properties (compressibility, hydraulic conductivity, sensitivity to frost and tendency of swelling and shrinking). If locally available fill material is deemed unsuitable for a compaction project, then the option of using a more suitable material from nearby location or use of chemically modified soil is explored. These decisions are affected by constraints related to time and economic factors associated with the project. Once a suitable fill material (natural or modified soil) is accepted, then based on the engineering properties of the fill material obtained from laboratory test results, a solution to the design problem is developed by consulting geotechnical engineers. This comprises the *design phase* of the project.

After completion of the design phase, engineers prepare the earthwork and compaction specifications, which are used by contractors during the *construction phase* of the project. The construction specifications are established via a two-way dialogue between the engineers and contractors involved in the project. These specifications comprise an important component of a successful compaction project. They specify the (1) *compaction targets to be achieved* in the field (e.g., the minimum relative compaction), (2) *equipment* to be used for compaction (compactor size and type), (3) *methods and procedures* to be followed (maximum lift thickness, frequency of compaction, acceptable range of compaction water content and number of compactor passes), and (4) *compaction quality control tests* (and their acceptable results) to be performed on the compacted soil to assess compaction quality.

Figure 2.1 presents an overview of a typical compaction project from the perspective of the design engineer. As shown in Figure 2.1, by the double arrows between the compaction specifications and contractor input, development of the construction specifications requires significant exchange of ideas and dialogue between the engineers and the contractors, often leading to beneficial modifications to construction and testing approaches that are adopted for a project. Generally, in the context of INDOTs work, freedom is given to the contractor to choose a suitable subgrade material and compact it to satisfy the specifications set by INDOT.

Specifications have not only to be developed but also followed for compaction projects to be successful. It is for this reason that specifications contain particulars regarding compaction quality control tests that need to be performed on the compacted soil.

The process of performing compaction quality control tests on the compacted soil to assess the state of construction and to enforce a standard of construction quality is termed as *quality assurance and quality control (QA/QC)*. If the tests are performed by engineers, serving as representatives of the client, the process is categorized as quality assurance (QA). On the other hand, if tests are performed by the contractor to

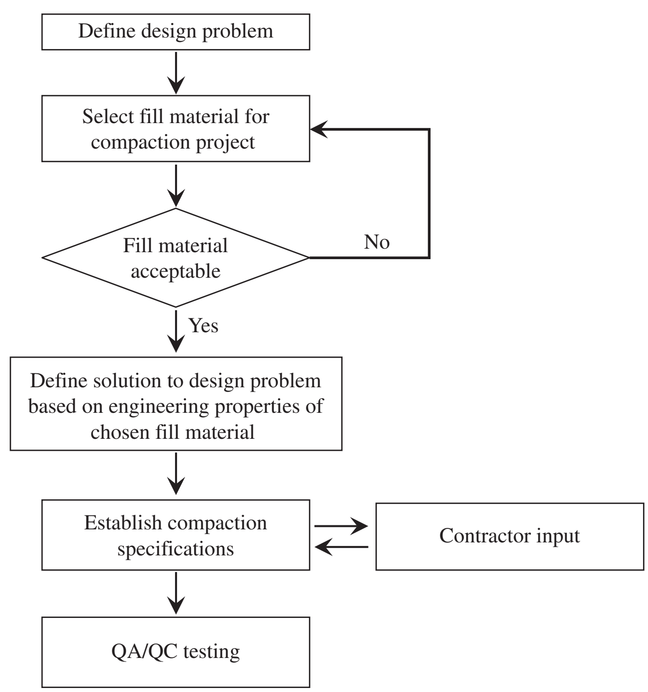

Figure 2.1 Overview of a typical compaction project.

check on the build quality of the compacted soil, the process is categorized as quality control (QC). QA/QC procedures form an essential part of the earthwork-construction process, especially in large-scale compaction projects. They allow for the streamlining of compaction processes and also insure the construction of properly compacted and well-built earth structures. Therefore, the main objectives of the QA/QC procedures are to: (1) ensure that the construction specifications are met and (2) the end product (constructed subgrade or embankment) is in accordance with the requirements of the client (which in most cases are state agencies). This demands that the construction specifications set by engineers be accurate, detailed and suitable for the construction project in question. The next section outlines the details of construction specifications.

2.1.1 Construction Specifications

Construction specifications that need to be prepared by the engineers can be broadly divided into three main categories. These are (1) *methods specifications*, (2) *end-product specifications* and (3) *performance-based specifications* (Transportation Research Board, 2009). Table 2.1 summarizes the specifications adopted by various state agencies in the U.S.

Methods specifications for subgrade and embankment projects are concerned with the compaction processes and materials. These specifications include type and weight of compactor, the range of the compaction water content, number of passes required to achieve the target relative compaction, the maximum allowed lift thickness and maximum size of particles of the fill material to be used in compaction. To establish *methods specifications*, knowledge of the engineering properties of the accepted fill material is essential. Often, test fills and test pads are constructed to determine *methods specifications*, to specify the lift thickness and number of compactor passes required to attain the desired relative compaction (Rodriguez, Del Castillo, & Sowers, 1988; Transportation Research Board, 2009).

End-product specifications for subgrade and embankment projects are concerned with the constructed subgrade and, for most DOTs, refer to the minimum value of relative compaction *RC* that must be achieved to deem the compacted subgrade soil fit for pavement construction. By achieving the minimum required value of RC specified for the compacted subgrade soil (due to a more uniform compaction of the subgrade soil), the possibility of occurrence of differential settlement is reduced, and, in general, the compacted subgrade soil should be able to safely sustain the applied traffic loads (Holtz et al., 2010; Transportation Research Board, 2009). For most roadway earthwork projects, a value of 95–100% *RC* is chosen as the end-product specification (Kim et al., 2010).

In addition to the *methods* and *end-product specifications*, *performance-based specifications* also form a fundamental part of construction specifications. These specifications describe the desired levels of fundamental engineering properties [e.g., resilient modulus (MR) and California bearing ratio (CBR)] which must be achieved by the compacted soil (Transportation Research Board, 2009).

State agencies that commission the construction of compacted embankments and subgrades have, over the past decade, started to realize the importance of establishing the performance characteristics of the compacted soils that would ensure the sustainable design of pavements (NCHRP, 2004). This is because, while the achievement of end-product specification of *RC* does give a measure of the expected performance of the compacted soil, quantification of the performance characteristics of the compacted soil is still necessary. Most state agencies dont yet explicitly include performance-based specifications in their construction specifications, but a shift can be observed towards their inclusion in light of research findings (Fleming, 1998; Livneh & Goldberg, 2001; Pinard, 1998).

Engineers today design pavements for optimum performance using the Mechanistic-Empirical Pavement Design Guide (MEPGD), which requires the quantification of the strength and stiffness characteristics of the compacted soil (NCHRP, 2004). As a result, QA/QC tests specifically targeting the estimation of strength and stiffness of the compacted soil, which are representative of the *performance-based specifications*, have gained significant importance in earthwork projects.

2.1.2 QA/QC Testing

The objective of the QA/QC tests is to provide the engineers and contractors with methods of judging whether the *end-product* and *performance-based specifications* have been met by the compacted soil. Therefore, QA/QC tests can be categorized in to two main types: (1) *density-based tests,* which are part of the end-product specifications (RC), and (2) *performance-based tests,* which are part of the performance specifications. The first category of tests focuses on the measurement of *in situ* soil density, providing an indirect measure of the performance of the compacted soil, while the second category of tests focuses on the explicit estimation of strength and stiffness characteristics of the compacted soil. A comprehensive review of the two types of QA/QC tests has been done by Kim et al. (2010). Table 2.2 provides the most prevalent tests employed for compaction QA/QC with a brief description of each test and a list of pros and cons.

In addition to the tests described in Table 2.2, there are many other tests, similar to the ones described by Holtz and Kovacs (1981), Kim et al. (2010), and Rodriguez et al. (1988) that are used to assess the performance characteristics of compacted soil. The next section reviews the Dynamic Cone Penetrometer, followed by a brief history of the device and its use as a performance-based QA/QC test.

2.2 Dynamic Cone Penetrometer

The dynamic cone penetrometer (DCP) is a simple and lightweight penetration device often used for the characterization of compacted subgrade soil. It was developed by A. J. Scala in 1956 to evaluate the properties of flexible pavement, by developing correlations with CBR results (Scala, 1956) of compacted soils. Due to the benefit of being an economic, light and simple test, it has proven over the years to be a useful device and has been adopted widely for subgrade compaction quality control.

TABLE 2.1
End-product and method specification of various state agencies

		End-Product Specification	Methods Specification
Agency	**Condition**	**RC Specification**	**Range of Compaction Water Content**
AASHTO (2003)	For subgrade with A-1, A-2-4, A-2-5 and A-3 soil (according to AASHTO classification)	RC ≥ 100%	OMC* ±2%
	All other cases	RC ≥ 95%	
Illinois (2004)	Embankment height < 1.5 feet	All lifts: RC ≥ 95%	For top 2 feet, compaction water content can be no more than 120% of OMC
	1.5 feet ≤ Embankment height ≤ 3 feet	First lift: RC ≥ 90% Consecutive lifts: RC ≥ 95%	In case of existence of adjacent structures, not more than 110% of OMC for top 2 feet
	Embankment height ≥ 3 feet	First 1/3 of embankment height: RC ≥ 90% Second 1/3 of embankment height: RC ≥ 93% Last 1/3 of embankment height: RC ≥ 95%	
Texas (2004)	PI ≤ 15%	RC ≥ 98%	Compaction water content should be above OMC
	15% < PI ≤ 35%	98% ≤ RC ≤ 102%	
	PI > 35%	95% ≤ RC ≤ 100%	
New York (2008)	Subgrade	RC ≥ 95%	Not specified, kept at contractors discretion
	Embankment	RC ≥ 90%	
Indiana (2012)	Subgrade	RC ≥ 100%	Within −3% below OMC for silts and loess soils
	Embankment	RC ≥ 95%	Otherwise within +1% above and −2% below the OMC
Iowa (2012)	Subgrade	RC ≥ 95%	Within −6% below OMC for subgrade construction
	Embankment	RC > 95%	
Missouri (2011)	Within top 18 inches of subgrade and/or within 100 feet of structures	RC ≥ 95%	"Near the OMC,* as deemed suitable by engineers to improve compaction conditions"
	Minimum acceptable except in cases outlined above	RC ≥ 90%	
Minnesota (2014)	Less than 3 feet below road core and/ or within 3 feet of a structure	RC ≥ 100%	65% to 102% of OMC
	All cases except above	RC ≥ 95%	65% to 115% of OMC
South Dakota (2004)	At top of berm slope	RC ≥ 97%	If OMC is < 15%, then OMC ±4%
	All cases except above	RC ≥ 95%	If OMC > 15%, then OMC −4% to OMC +6%
Wisconsin (2013)	Embankments less than 6 feet in height or within 200 feet of bridge abutments	RC ≥ 95%	Such that material should not undergo rutting
	Embankments over 6 feet in height: Material at depth greater than 6 feet	RC ≥ 90%	
	Embankments over 6 feet in height: Material at depth less than 6 feet	RC ≥ 95%*	
Virginia (2007)	Top 6 inches of all compacted soil	RC ≥ 100%	±20% of OMC
	Percentage retained on #4 sieve < 50%	RC ≥ 100%	
	Percentage retained on #4 sieve 51–60%	RC ≥ 95%	
	Percentage retained on #4 sieve 61–70%	RC ≥ 90%	
North Carolina (2013)	All embankments	RC ≥ 95%	Near OMC, to be determined by field technicians based on "reasonable effort of compaction"

* OMC: Optimum moisture content as estimated from standard Proctor tests following ASTM D 698.

TABLE 2.2
Density-based and performance-based tests for QA/QC

Test	Description	Pros	Cons
Density-Based Tests			
Sand Cone	Test used for estimation of *in situ* density of the soil by sand replacement method (ASTM D1556)	Simple Reasonably accurate results	Time consuming Sensitive to ground vibration Erroneous when large size particles present in soil
Nuclear Gauge	Device used for estimation of *in situ* soil density and water content by use of gamma radiation and high speed neutrons. Gamma radiations measure wet density and neutrons help in measurement of water content (ASTM D6938-10)	Quick and efficient Accurate results if calibrated properly	Radioactive core poses health issues Trained and certified personnel needed to operate device Expensive in comparison to other available methods
Performance-Based Tests			
California Bearing Ratio	Measure of mechanical strength of subgrade, defined as the ratio of pressure required to penetrate a soil sample through a set depth using a standard piston bar at a constant rate of penetration to the pressure required to achieve an equal penetration using same piston bar at the same penetration rate on a standard crushed rock material (ASTM D1883 – 07e2)	Widely used by state and federal pavement construction agencies	Unable to simulate the shear stresses that generate due to repeated traffic loading Possible to get same CBR values for two different soil specimens with different stress strain behavior
Resilient Modulus	Ratio of the deviator stress to the recoverable elastic strain under repeated loading (ASTM STP1437)	Simulates the response of soil to traffic loading conditions	Complex Time consuming Sample prepared in laboratory may not be representative of the field conditions
Dynamic Cone Penetration	Device that measures the resistance offered by soil to the penetration by a cone (of standard size) when loaded dynamically by means of a drop hammer (ASTM D6951M)	Fast, simple and easy to operate Inexpensive Results can be correlated with shear strength of subgrade or other design parameters (CBR, MR)	Results are significantly affected by *in situ* moisture conditions, especially for fine-grained soils Presents highly variable and unreliable results in soils with large gravel content
Falling-Weight Deflectometer	Device used to measure the *in situ* elastic modulus of compacted material. Comprises a falling mass and a displacement measuring sensor attached to center of bearing plate	Simplicity of operation	Issues with calculation of elastic modulus of the subgrade caused by nonlinear elastic and plastic deformation

2.2.1 The DCP Equipment

Figure 2.2 shows a schematic of the DCP in its current standardized form. The DCP has had a number of changes in its shape and dimensions over the years (Livneh & Ishai, 1987; Scala, 1956). The original DCP developed by A. J. Scala (1956) had a falling weight hammer of 9 kg (instead of the present 8 kg), a drop height of 508 mm (instead of the current 575 mm) and cone angle of 30 degrees (instead of the present 60 degrees). It was standardized to its current form for use in the U.S. in 2003 by ASTM standard D6951. The standardizations of the weight of the hammer and the dimensions of the device by ASTM were based on the dimensions and weights of the DCP used by the Transvaal Road Department, South Africa, as described by Kleyn (1975).

As can be seen in Figure 2.2, in its standardized form, the DCP consists of two connected 16-mm-diameter shafts. The lower shaft has an anvil on its upper end and a replaceable cone tip, with a 60 degree cone angle, on the lower end. The shaft itself has depth markings that indicate the distance from the base of the cone tip. The upper shaft has an 8 kg sliding hammer that is dropped from a height of 575 mm to cause impact on the anvil. The two shafts are connected just below the anvil by means of a sliding connection with a bolt washer and a clip pin. All parts of the device are made from stainless steel to prevent rusting.

To perform a DCP test ideally two operators are required. One records the measurements, while the other raises and drops the hammer. At the start of the test, the cone tip is placed on the compacted soil, ensuring that the shaft is vertical and "seated" by means of light tamping of

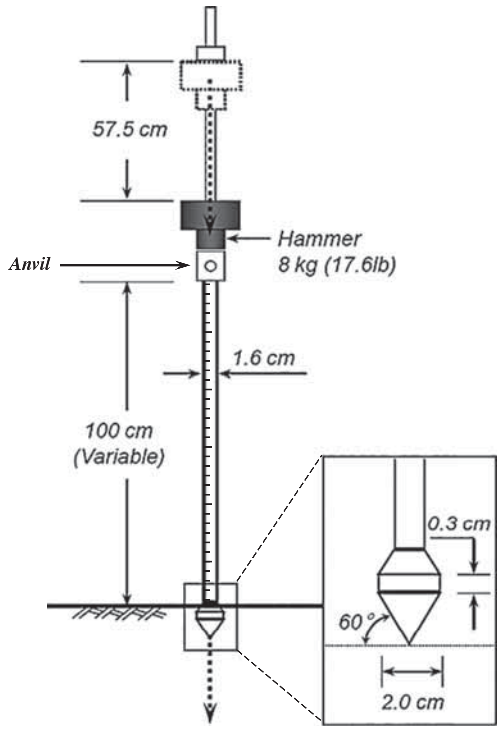

Figure 2.2 The dynamic cone penetrometer (DCP) (after Kim et al., 2010).

the hammer on the anvil until the cone tip is just inside the soil to be tested (seating is only necessary in case of clays). The 8 kg hammer is then raised to its full height of 575 mm and dropped on to the anvil, driving the cone into the soil. The hammer blows are repeated and the number of blows required to penetrate a specific depth of the compacted soil are recorded. Depth measurements are taken either from the scale etched on the lower shaft (shown in Figure 2.2) or using a reading device attached to the DCP (not shown in Figure 2.2). The results obtained from the test can then interpreted in terms of penetration ratio (PR) (penetration of the cone tip into the compacted soil per unit drop of hammer; in units of inches/blow or mm/blow) or in terms of number of blows required to penetrate a specific depth of the compacted soil (e.g., number of blows required for a penetration of 6 or 12 inches into the compacted soil). The choice of units depends on the nature and objective of the correlations to be developed.

2.2.2 Development of the DCP Correlations and Application in QA/QC

Since the inception of the DCP in 1956, substantial research has been done on interpretation of test results. Research studies (Amini, 2003; Ayers, Thompson, & Uzarski, 1989; Bester & Hallat, 1977; Gabr & Hopkins, 2000; Harison, 1987; Kleyn, 1975; Van Vuuren, 1969) have focused on the development and refinement of correlations between DCP test results, CBR values, and subgrade resilient modulus values of compacted soils. Table 2.3 and Table 2.4 highlight some of the DCP correlations developed by recent and widely accepted research studies.

The use of DCP QA/QC specifications, in addition to the regular moisture and density tests, is becoming more widespread all over the world (Livneh & Livneh, 2013). By virtue of being a relatively inexpensive, light and simple test, the DCP test has proved to be useful to state agencies (such as INDOT) that are actively involved in QA/QC testing of compacted soils used to construct embankments and subgrades. As pointed out by Luo et al. (1998), the small and lightweight design of the DCP allows it to be easily carried and used in remote areas and congested construction sites; this may not be possible with many of the other field testing equipment.

TABLE 2.3
Correlations between DCP test results and CBR (after Salgado & Yoon, 2003)

Property	**Reference**	**DCP Correlations**	**Remarks**
California Bearing Ratio	Kleyn (1975)	$\log(CBR) = 2.62 - 1.27 \times \log(PR)$	Equation developed from laboratory test results
	Harison (1987)	$\log(CBR) = 2.56 - 1.16 \times \log(PR)$	Equations based on results of laboratory tests performed on fine-grained soils
	Livneh, Ishai, and Livneh (1995)	$\log(CBR) = 2.46 - 1.12 \log \times (PR)$	Equation based on results of laboratory tests performed on fine-grained and coarse-grained soils
	Livneh, Livneh, and Ishai (2000)	$\log(CBR) = 2.20 - 0.72 \log \times (PR)^{1.5}$	Equation developed based on results of field tests performed on coarse- and fine-grained soils [reported to work well with coarse- and fine-grained soils
	Gabr and Hopkins (2000)	$\log(CBR) = 1.40 - 0.55 \times \log(PR)$	Equation developed based on field and laboratory testing on aggregate base coarse
	George et al. (2009)	$\log(CBR) = 1.675 - 0.7852 \log(PR)$	Equation developed based on results of field tests on lateritic subgrades

CBR: California bearing ratio.
PR: Penetration ratio = ratio of penetration of DCP probe tip to penetration depth (mm/blow).

TABLE 2.4
Correlations between DCP test results and resilient modulus (after Salgado & Yoon, 2003)

Property	Reference	DCP Correlations		Remarks
Subgrade Resilient Modulus	Chen, Hossain, and Latorella (1999)	$M_R = 338 \times (PR)^{-0.39}$	M_R: Resilient modulus of subgrade PR: Penetration ratio, ratio of penetration of DCP probe tip to penetration depth (mm/blow)	Resilient modulus was back-calculated from FWD results Good correlation for PR values between 10 and 60 mm/blow was observed
	Herath, Mohammad, Gaspard, Gudishala, and Abu-Farsakh (2005)	$M_R = 520.62 \times \left(\frac{1}{PR^{0.7362}}\right) + 0.40\left(\frac{\gamma_d}{w_c}\right) + 0.44PI$	M_R: Resilient modulus of subgrade PR: Penetration ratio, ratio of penetration of DCP probe tip to penetration depth (mm/blow) γ_d: *In situ* dry density (kN/m^3) w_c: Water content (%) PI: Plasticity index (%)	Resilient modulus was experimentally determined[%] in the laboratory using samples collected from the field
	Mohammad and Herath (2007)	$M_R = 165.5 \times \left(\frac{1}{PR^{1.147}}\right) + 0.0966 \times \left(\frac{\gamma_d}{w_c}\right)$	M_R: Resilient modulus of subgrade (MPa) PR: Penetration ratio, ratio of penetration of DCP probe tip to penetration depth (mm/blow) γ_d: *In situ* dry density (kN/m^3) w_c: Water content (%)	Correlation between DCP and M_R developed for fine-grained soils; laboratory testing[%] for M_R done on core samples collected from the field

Some research studies (Fleming, 1998; Livneh & Goldberg, 2001; Pinard, 1998) have stated that the density-based tests, which are currently the norm for QA/QC testing of compacted subgrade soils, are not sufficient to ensure compliance of the subgrade soil layers with performance requirements. Studies point out that while the current end-product specification of relative compaction (RC) is the accepted norm in many projects, such a specification may not produce the desired engineering properties (strength and stiffness) for the compacted soil in roadway service conditions, especially for coarse-grained soils, such as sands and silts, for which it is hard to establish well-defined, representative density relationships for the soils used in compaction due to slight variations in soil within the source borrow pit (Livneh & Livneh, 2013). It is therefore recommended to use DCP, along with regular moisture assessments and other soil stiffness tests, for *in situ* compaction characterization (Siekmeier, Young, & Beberg, 2000).

Findings of similar studies, in the past decade, have motivated the research carried out by projects funded by state DOTs for the development of QA/QC correlations for the DCP (Amini, 2003; Burnham, 1997; Kim et al., 2010; Luo et al., 1998; Salgado & Yoon, 2003). In addition to state DOTs in the U.S., DCP research had previously also been carried out for the U.S. army by Webster, Grau, and Williams (1992), in which procedures for the use of DCP to estimate soil strength were presented. The DCP penetration index (number of DCP blows per mm of penetration) was correlated against the CBR strength value of the soil as a check for the operability of aircraft and military vehicles on un-surfaced soils. As can be seen from Table 2.3 and Table 2.4, most of the correlations for the DCP have been developed to relate the DCP test results (*i.e.*, the penetration ratio PR) to the CBR and soil resilient modulus test results. Based on field and laboratory testing, useful correlations between the penetration ratio of the DCP and the subgrade resilient modulus have also been developed (Gabr &

TABLE 2.5
Correlations developed by Kim et al. (2010)

Soil Type (AASHTO)	Correlations		Penetration Depth
A-1 and A-2 (coarse-grained soils, no gravel)	$NDCP_{(0''-to-12'')} = 4.0 \times \ln(C_u) + 2.6$	$NDCP$: number of blow counts required for specific depth of penetration Cu: coefficient of uniformity	Penetration from top of compacted soil to 12-inch depth
A-3 (fine sands, non-plastic)	$NDCP_{(0''-to-12'')} = 59 \times e^{[-0.12 \times w_{copt}]}$	$NDCP$: number of blow counts required for specific depth of penetration *wcopt*: OMC of compacted soil	Penetration from top of compacted soil to 12-inch depth
A-4 to A-7 (fine-grained soils)	$NDCP_{(0''-to-6'')} = 17 \times e^{[-0.07 \times PI \times (F40/100)]}$ $NDCP_{(6''-to-12'')} = 27 \times e^{[-0.08 \times PI \times (F40/100)]}$	$NDCP$: number of blow counts required for specific depth of penetration PI: Plasticity index of compacted soil (%) $F40$: Percentage passing #40 sieve	Penetration from surface to 6-inch depth Penetration from 6-inch to 12-inch depth from top of compacted soil

TABLE 2.6
Correlations developed by White et al. (2002)

Soil Performance Classification		Maximum Mean DCP Index (mm/blow)	Maximum Mean Change in DCP Index (mm/blow)
Cohesive (Percentage passing No. 200 sieve $>$ 36%)	Select	75	35
	Suitable	85	40
	Unsuitable	95	40
Intergrade (Percentage passing No. 200 sieve 16–35%)	Suitable	45	45
Cohesionless (Percentage passing No. 200 sieve $<$16%)	Select	35	35

Hopkins, 2000; George, Rao, & Shivashankar, 2009; Mohammad & Herath, 2007).

Salgado and Yoon (2003) developed the following correlation between the DCP penetration ratio PR and the *in situ* dry density of clayey sands:

$$\gamma_d = \left(10^{1.5} \times PR^{-0.14} \times \sqrt{\frac{\sigma'_v}{P_A}}\right)^{0.5} \times \gamma_w \qquad (2.1)$$

where γ_d is the dry unit weight of the soil, PR is the penetration ratio defined as the penetration per unit blow of the DCP, σ'_v is the effective vertical stress in the soil mass, PA is the atmospheric air pressure and γ_w is the unit weight of water. Similar results were also obtained by George et al. (2009) for lateritic subgrades. Livneh and Livneh (2013) suggest that such equations, owing to the considerable uncertainty associated with DCP tests results, should be used in tandem with conventional density tests, such as the nuclear gauge.

In addition to the studies highlighted in Table 2.3 and Table 2.4, DCP correlations have also been attempted specifically to address the QA/QC concerns of the state DOTs (Kim et al., 2010; Salgado & Yoon, 2003; White, Bergeson, & Jahren, 2002). The DOTs of Indiana, Minnesota, Iowa, Ohio, among others, have supported remarkable research on these topics. Table 2.5 and Table 2.6 provides the QA/QC correlations developed by Kim et al. (2010) for Indiana DOT and White et al. (2002) for Iowa DOT, respectively.

This approach of development of correlations specifically addressing the QA/QC concerns of the DOTs holds merit and has proven useful to field engineers who can quickly check the compliance of constructed subgrade with the RC specifications, without having to carry out numerous time consuming density tests. Furthermore, the DCP test results can also be used to estimate values of CBR and MR using the correlations available in the literature, as described in Table 2.3 and Table 2.4.

2.3 Summary

The compaction process comprises of many parts, from statement of design problem to choice of compaction material, establishment of compaction specifications and QA/QC testing to ensure achievement of adequate compaction state. It requires the active involvement of both the contractors and design engineers involved in the construction project. Each state DOT has its own specifications for compaction quality control, but all of them use a minimum limit on the relative compaction as the criteria to check for achievement of adequate compaction. Compaction QA/QC is carried out by either density based tests such as the sand cone and the nuclear gauge test or by performance based tests such as the DCP and FWD.

The density based tests measure the *in situ* density of the compacted soil, while the performance based tests measure the strength and stiffness characteristics of the compacted soil. Many DOTs have started to move towards the use of performance based tests for QA/QC, in the sphere of which, the DCP stands as one of the most commonly used performance based test in the U.S. for compaction quality control due to its ease of use and simple application process.

DCP was developed by A. J. Scala in 1950s and 1960s to evaluate the properties of flexible pavements by use of correlations between DCP and the CBR test results (Scala, 1956). Since then, the DCP has been used and developed by a number of state agencies inside and outside the U.S. to check the compaction quality of compacted soils. Correlations have been developed and refined between DCP test results and CBR, MR and *in situ* dry density of the compacted soil over the past few decades, and progress is still being made in the sphere of further development of these correlations.

3. RESEARCH APPROACH

This chapter describes the research approach and procedures followed for field and laboratory testing. It also presents the statistical approach adopted for extraction of representative DCP results from raw field DCP data.

3.1 Overview

The mechanical response of compacted soils to DCP tests is affected by soil type, density of the compacted soil, compaction water content and fabric of the compacted soil. Therefore, to develop DCP blow count correlations for use in subgrade compaction quality control, it was necessary first to develop a holistic methodology, taking into account all the relevant aspects that affect the mechanical response of compacted soil.

To assess the state of the compacted soil in the field and its response to the DCP impact loading, a large number of tests were performed on compacted embankments and subgrades in INDOT road construction sites across Indiana, as shown in Figure 3.1. Field tests were performed to obtain: (1) the *in situ* dry density of the compacted soil, (2) the water content of the soil at the time of testing and (3) the number of blows required for the DCP to penetrate the compacted subgrade to a specific depth (0 to 6 inches and 6 to 12 inches of penetration into the subgrade soil).

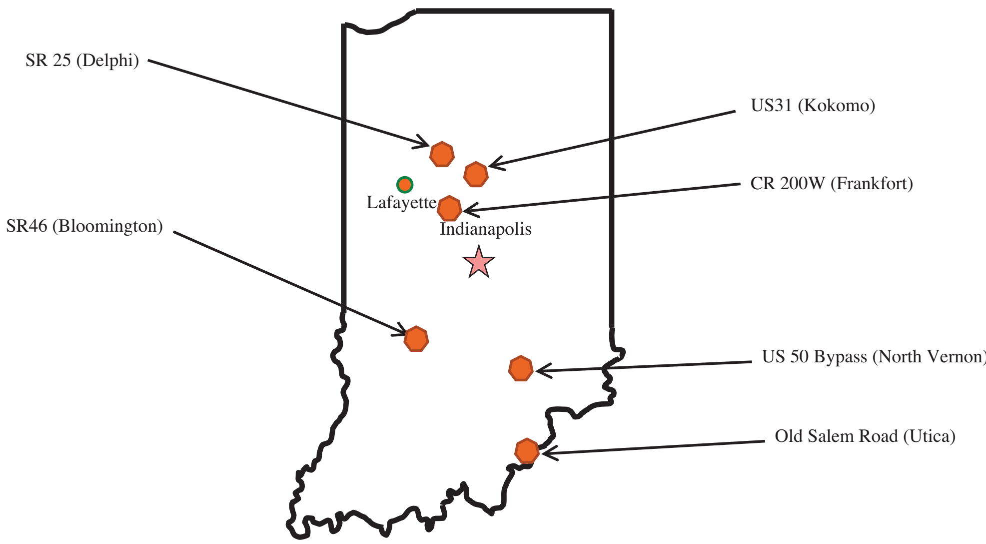

Figure 3.1 INDOT road construction sites where DCP tests were performed.

The Dynamic Cone Penetrometer Test (DCPT) database used in this research was augmented using the DCPT data from a previous JTRP study done at Purdue by Kim et al. (2010). The entire DCPT blow count data was statistically analyzed; correlations were investigated between the main index properties of the subgrade soils (maximum dry density, optimum moisture, plasticity index, etc.) and the DCP blow counts extracted from the statistical analysis of the raw DCP blow count data.

3.2 Field Testing Procedure

INDOT projects with subgrade/embankment construction underway during the duration of this research project were selected for collecting the data needed for this research. At each of the selected locations, ten DCPTs and one sand cone test (comprising one test set) were performed. The ten DCPTs were performed within the perimeter of a 1-meter-diameter circle. The sand cone test, which was performed to obtain the *in situ* density of the compacted subgrade, was performed at the center of the circle. Figure 3.2 shows a field image and a schematic representation of the spatial distribution of the DCPTs and sand cone tests, as performed in the field.

For each individual DCP test, the number of blows from the sliding hammer on the DCP required for 0 to 6 inches and 6 to 12 inches of penetration from the surface of the constructed subgrade/embankment were recorded in a field data sheet. Figure 3.3 shows an example of the field data sheet used for recording DCP test data.

After performing the field tests at each location, soil samples were collected for additional laboratory testing. The soil removed from the ground during the sand cone tests was kept in air-tight, zip-lock bags and used to obtain the *in situ* water content of the compacted subgrade at the time of testing. In addition, approximately 25 lbs. of soil was collected from within the 1-m-diameter circle for index testing.

Approximately 40 minutes were required to perform the field tests corresponding to each test set (ten DCPTs and one sand cone test). Four to eight test sets were performed in sequence depending on weather conditions and availability of free testing space on compacted subgrade. Figure 3.4 shows a typical test sequence corresponding to four data sets.

After all the field and laboratory tests were performed, the laboratory and field data was organized into a database according to soil type and index properties (the details of the soil grouping is described in chapter 4 and chapter 6) and

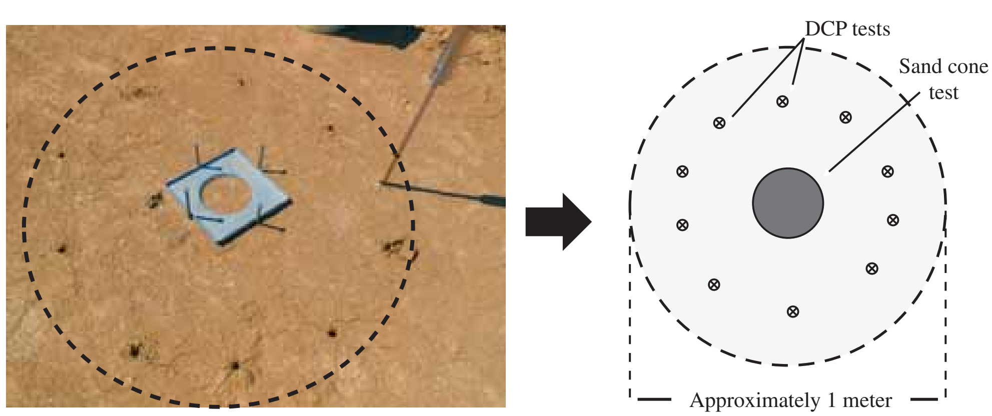

Figure 3.2 Spatial distribution of DCPTs and sand cone tests performed in one set in the field.

Date			
Location			
Set No.			
Remarks			
Test No.	No. of Blows for 0- to 6-inch Penetration	No. of Blows for 6- to 12-inch Penetration	No. of Blows for 0- to 12-inch Penetration
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			

Figure 3.3 Field data collection sheet.

then analyzed statistically. The procedure followed for the statistical analysis of the DCP data is explained in the next section.

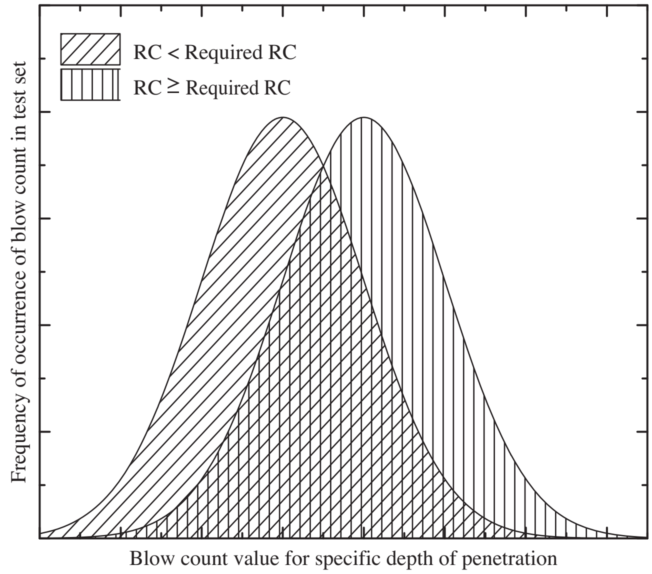

Figure 3.5 Distribution of DCP test results performed on similar soils compacted at different RCs.

3.3 Statistical Analysis Procedure

The objective of the DCPT data analysis was to obtain the blow count required to penetrate the constructed subgrade/embankment when a certain ground density, stated in terms of a relative compaction value, had been achieved. As is true with most field tests, the results obtained from the DCP tests performed on the field were scattered and no meaningful correlations could be developed from the raw data.

In order to obtain reasonable correlations between DCP test results and soil properties for compaction quality control, a logical method of processing the data was required. To fill such a need, the statistical procedure developed by Kim et al. (2010) was applied to the raw DCP data, with slight modifications, to obtain representative DCP blow count numbers corresponding to required relative compaction (RC = 95% or 100%) values for the compacted subgrade.

Figure 3.5 shows an idealized representative plot of frequency distribution of the results of 2 DCP test sets performed on the same type of soil compacted at different RCs. Plotted in Figure 3.6 is the frequency of occurrence of blow counts against the number of blows required for specific depth of penetration into the compacted soil. As can be seen in the Figure, the number of blows of the DCP required for specific depth of penetration into the compacted soil increases with the increase in RC. The mean of the frequency distribution increases with increase in RC, while the Coefficient of Variation (standard deviation per unit mean) doesnt change significantly. The magnitude of increase of the mean depends on the soil type.

The statistical procedure was developed by Kim et al. (2010) keeping in mind that the blow count for the QA/QC correlations corresponding to a particular type of soil should not only account for a certain percent of the test results obtained from soils compacted at the required RC, but also for an equivalent or higher percentage of the test results obtained from soils compacted at RC lower than the required RC. By this method, it was ensured that the blow counts obtained from tests performed on subgrades compacted at RC lower than the required RC were not ignored and also taken into consideration during the analysis.

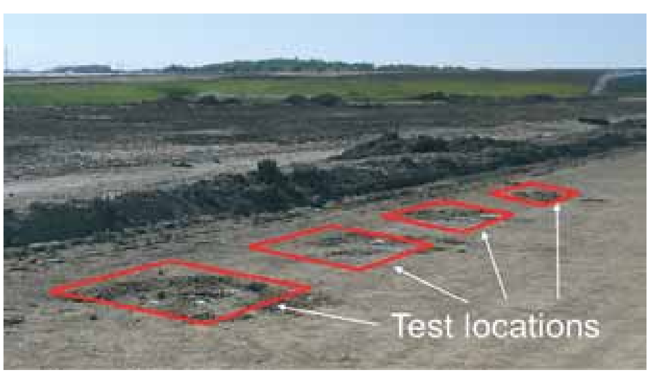

Figure 3.4 Typical test sequence corresponding to four data sets.

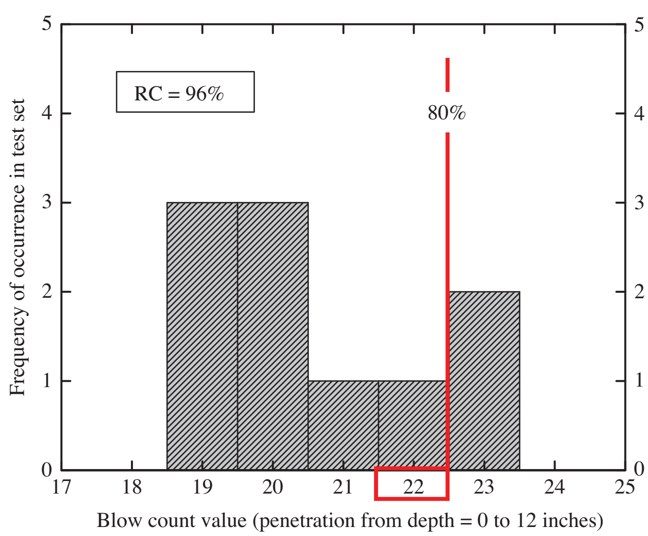

Figure 3.6 Frequency vs. DCP blow count for 0- to 12-inch penetration into compacted soil (DCP-C-A1-1-DE-1).

Based on the above concept, the statistical procedure developed to process the raw DCP data comprised the following steps:

1. The DCP blow counts, for specific depth of penetration (for fine-grained soils this depth was 0 to 6 and 6 to 12 inches while for coarse-grained soils this depth was 0 to 12 inches), were plotted against their frequency of occurrence for each data set. Figure 3.6 shows an example of a plot of frequency of occurrence of DCP blow count vs. blow count value for 0 to 12 inches of penetration of the DCP probe into a subgrade compacted to RC of 96%.
2. *Test sets were grouped* on the basis of the results from the index tests performed on the soil samples collected from the field. For soils sensitive to moisture change, only the DCP data associated with *in situ* water content at the OMC or within −2% of the OMC during the time of testing were considered in the statistical analysis of the data.
3. A *reference RC value of 95% or 100% was selected.*
4. For all test sets within a group, with RC values lower than the reference RC, the blow count value encompassing at least 90% of the values out of the 10 DCP tests (9 out of 10 DCPTs) was selected from the frequency histograms plotted in the first step. As an example, Figure 3.7 shows the frequency histogram of a test set performed on the same soils type as in Figure 3.6, but compacted to RC of 90%. A clear drop in the blow count values can be observed from the results obtained for tests performed on RC = 96%, as shown in Figure 3.6.
5. The highest blow count, out of the blow count values obtained in step 4 for each of the test sets within a group, was selected and termed Blow Count A. For example, in Figure 3.7, it can be seen that the blow count value encompassing at least 90% (9 out of the 10) of the DCP tests is 15 blows
6. Once *Blow Count A* was identified, the test set within the same group with an RC equal to the reference RC was chosen. For this test set, the DCP blow count value encompassing at least 80% of the DCP tests was selected and termed *Blow Count B*. Figure 3.6 serves as an example for a test set with RC equal to the reference RC of 96%. It can be seen in Figure 3.6, that the DCP blow count value encompassing at least 80% of the test results (8 out of 10) is equal to 22 blows for that particular test set.
7. In case of occurrence of multiple test sets with RC close to the chosen reference RC (or within +1% of the reference RC) within a soil group, step 6 was repeated for each of those test sets. The highest of all those DCP values was selected and termed *Blow Count B*. Significant difference was not seen in the data, with a value of 1 blow count observed to be the maximum difference between the blow counts of test sets at similar RCs.
8. *Blow Count A* and *Blow Count B* were then compared, and the higher of the two values was considered as the DCP blow count corresponding to the reference RC of the compacted subgrade soil. For the example used for illustration in Figure 3.6 and Figure 3.7, *Blow Count A* is equal to 15 (corresponding to RC = 90%, which is lower than the reference RC of 96%) and *Blow Count B* is equal to 22 (corresponding to the reference RC of 96%). The higher of the two, in this case *Blow Count B,* is chosen as the DCP blow count corresponding to the penetration depth of 12 inches for a reference RC of 96% for the soil type represented by the group.
9. The test procedure was then repeated for all test sets of each soil group.

Use of the outlined statistical method of selection of blow count ensured that none of the data associated with RC of lower than 95% was lost and that the DCP blow counts values obtained were representative of the state of the *in situ* soil. The use of blow count values corresponding to 80% and 90% (for calculation of Blow Count A and Blow Count B) increased the probability of choosing blow count values equal to or greater than the population mean. Moreover, by the use of this statistical procedure, the issue of scatter associated with the DCP field test results was addressed in a logical manner.

The above steps were used to extract the representative blow counts of all the test sets performed in the field. The frequency histograms for individual test sets can be found in Appendix A and Appendix B.

To supplement the data collected by researchers at Purdue, INDOT also conducted moisture sensitivity tests to ascertain the blow count variation observed in soils compacted at different moisture contents (above and below the OMC) but to the same relative compaction (95%). Data collected by INDOT is presented in chapter 6.

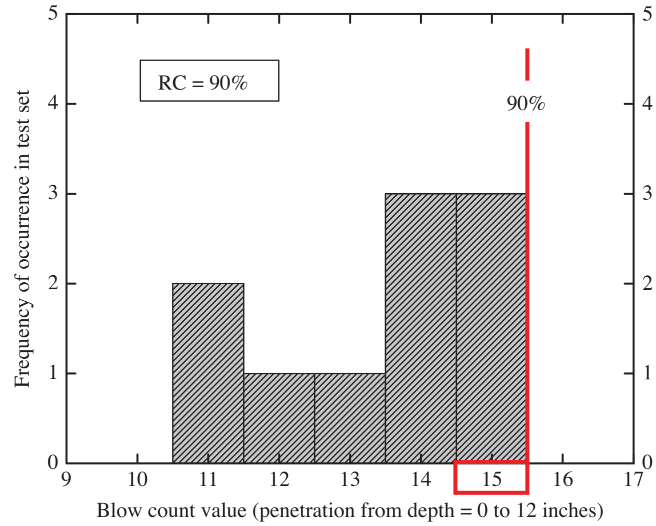

Figure 3.7 Frequency vs. DCP blow count for 0- to 12-inch penetration into compacted soil (DCP-C-A1-1-DE-4).

3.4 Summary

To develop DCP blow count correlations for use in subgrade compaction quality control, it is necessary to develop a holistic methodology, taking into account all the relevant aspects that affect the mechanical response of compacted soil. To assess the state of the compacted soil in the field and its response to the DCPs impact loading, a large number of tests (more than 800 DCP tests) were performed on compacted embankments and subgrades in INDOT road construction sites across Indiana to augment the existing database of DCP test results.

At each test site, multiple sets of tests were performed. Each test set comprised of 10 DCP tests, 1 sand cone test and 1 *in situ* water content measurement. The entire DCPT blow count data was statistically analyzed, and correlations were investigated between the main index properties of the compacted soils and the DCP blow counts extracted from the statistical analysis of the raw DCP blow count data.

The statistical process of analyzing the DCP data developed by Kim et al. (2010) was adopted in this study. This allowed for considerable reduction in scatter of the DCP test results and helped in the development of correlations between DCP test results and the soil index properties. The main steps of the statistical analysis procedure were:

1. Using the field data, plot histogram of the frequency of occurrence of a particular blow count (for a specific depth of penetration) against the value of the blow count for each test set.
2. For a given soil type, from the plotted frequency histograms, select the DCP blow count higher than 90% of the blow counts of all test sets with an RC value less than the reference RC (i.e., RC $<$ 95% or 100%) and refer to it as Blow Count A for that soil type.
3. For the same soil type, select the DCP blow count higher than 80% of the blow counts of all the test sets with RC equal to or greater than (within +1%) the reference RC (according to the RC specifications of INDOT, RC $>$ 95% or 100%) and refer to it as Blow Count B for that soil type.
4. Choose higher of the two blow counts, Blow Count A and Blow Count B, as the representative blow count of the soil type.

Use of the statistical method of selection of blow count ensured that none of the data associated with RC of lower than 95% was lost and that the DCP blow counts values obtained were representative of the state of the *in situ* soil. The use of blow count values corresponding to 80% and 90% (for calculation of Blow Count A and Blow Count B) increased the probability of choosing blow count values equal to or greater than the population mean.

4. LABORATORY TEST RESULTS

In addition to the data collected in previous research studies, more than 800 DCP tests were performed on constructed subgrade and embankments in 5 major INDOT road projects, within a single construction season. Soil samples were collected from the location of each test set to obtain the index properties of the soil. Laboratory tests performed on the 80 soil samples collected included: (1) grain size distribution (sieve and hydrometer analysis), (2) soil plasticity (liquid limit and plastic limit), (3) standard Proctor compaction test and (4) specific gravity. This chapter presents the laboratory test data for soils tested during the course of the project.

4.1 Testing Outline

To collect data for analysis of DCP blow counts a systematic approach was followed. Field testing was carried out in the summer and early Fall of 2013. Testing locations were identified where subgrade or embankments were being constructed by INDOT and sets of tests were performed to collect data as soon as the soil was compacted and space was made available for testing. Testing was carried out in 5 major project sites across the state of Indiana. Table 4.1 outlines the testing schedule with the date, location, roadwork project name and number of test sets that were performed on a particular day.

Based on the laboratory test results, it was observed that the index properties of the soil samples collected on the same day from the same test site did not vary much (in these cases, the soil was classified into a single soil type). Generally, the test sets performed within the same day were located 2–3 meters from each other.

4.2 Soil Grouping and Identification

Owing to the large number of laboratory tests performed, soils were identified by a system described in this section. Each of the 80 soils tested were first categorized as fine-grained or coarse-grained soil on the basis of their compaction characteristics, plasticity index and fines content of the soil, then grouped into one of the 7 main AASHTO soil types and further sub-grouped within the AASHTO classification on the basis of their compaction property of standard proctor OMC. The individual soils types were identified as follows:

$$\text{“}Test\text{”} - \text{“}Fine/Coarse\text{”} - \text{“}AASHTO\ classification\text{”} - \text{“}Subgroup\text{”}$$

where *Test* = DCP for all soils tested, *Fine/Coarse* = soil classification as fine or coarse-grained (F/C) based on its compaction characteristics and nature and content of fines in the soil; refer to Table 4.2 (details of theoretical reasoning behind classification criterion is presented in chapter 6) *AASHTO classification* = soil classification based on the grain size distribution and plasticity (soil was identified as one of the 7 AASHTO soil types; A-1 to A-7), and *Subgroup* = soil subgrouping according to compaction characteristics (soil was numbered from 1 onwards within the AASHTO classification).

Individual test sets performed in specific locations were identified by suffixing two more parts to the naming system.

$$\text{“}Test\text{”} - \text{“}Fine/Coarse\text{”} - \text{“}AASHTO\ classification\text{”} - \text{“}Location\ code\text{”} - \text{“}Set\ number\text{”}$$

where *Location code* indicates the location where the soil was tested, as described in Table 4.3, and *Set number* describes the set number of the soil tested in a given location.

TABLE 4.1
Field testing locations, dates and number of tests performed

Date	Location, Project	Number of Test Sets Performed
5/16/13	Delphi, SR25	1
5/17/13	Delphi, SR25	4
5/23/13	North Vernon, US50	4
5/24/13	Kokomo, SR31	8
5/30/13	Utica, Old Salem Road	6
6/04/13	Kokomo, SR31	8
6/19/13	North Vernon, US50	4
6/25/13	Kokomo, SR31	4
7/11/13	Kokomo, SR31	4
7/16/13	Delphi, SR25	7
7/29/13	North Vernon, US50	8
8/06/13	Kokomo, SR31	4
8/08/13	Delphi, SR25	4
8/13/13	Delphi, SR25	6
8/20/13	Bloomington, SR46	4
7/17/14	Frankfort, CR200W	4

TABLE 4.2
Soil grouping criteria

Soils Group (F or C)		OMC (%)	MDD pcf	MDD kN/m^3	PI (%)	Percentage Passing #200 Sieve (0.075 mm)
Coarse-grained (or sand dominated) (C)	Natural soils	< 12	> 120	> 18.9	< 5, Non plastic	≤ 25
	Manufactured soils	12–14	112–118	18–19	NP, fine sands	0–8
Transitional (or silt dominated)	Coarse (TC)	[12–15)	(110–120]	(17.3–18.9]	≤ 8–10	< 60
	Fine (TF)				≥ 8–10	≥ 60
Fine-grained (or clay dominated) (F)		≥ 15	≤ 110	≤ 17.3	≥ 8–10	≥ 60

For example, consider a soil which has fines content of about 70–80%, PI of 12–14%, standard Proctor OMC of 15% and Proctor maximum dry density of 18.6 kN/m^3 (116 pcf). Based on Table 4.2, it is a fine-grained soil (F), which according to AASHTO classification is an A6 type soil. Therefore, based on the above naming criteria, the soil will be identified as "DCP-F-A6". Now, assuming that it is the A6 type soil with the lowest OMC out of all the A6 soils tested, it will be given the lowest subgroup of 1, identifying it as "DCP-F-A6-1." Furthermore, individual test sets are numbered chronologically; for example, a soil tested in Kokomo (Location code KO according to Table 4.3) would be identified as "DCP-F-A6-1-KO" and any individual test sets performed on this soil in Kokomo is numbered starting from 1 ("DCP-F-A6-1-KO-1," "DCP-F-A6-1-KO-2," "DCP-F-A6-1-KO-3" and so on).

In this chapter, as we describe only the index properties of the soil, we will be identifying the soil just by the first four parts of the naming system, *i.e.*, for the above described soil, it would be "DCP-F-A6-1." To identify results from the individual DCP test sets, we will be including the last two parts in the naming system in the next chapter.

TABLE 4.3
Location codes

Location	Code
Delphi, SR 25	DE
Kokomo, SR31	KO
North Vernon, U.S. 50 Bypass	NV
Bloomington, SR 46	BL
Utica, Old Salem Road	UT
Frankfort, CR200W	FR

4.3 Combined Test Results

Based on the naming system described above, Table 4.4 and Table 4.5 show the main soil types along with the ranges of their index properties. Table 4.4 shows the range of results for the fine-grained soils; as can be observed, the soils range from low plasticity soils to medium plasticity soils, with PI ranging from 8% to 20%. Based on the colloidal activity (Pandian & Nagaraj, 1990; Skempton, 1953) of the clays found in the soils, it can be stated that the soils most likely have illite and kaolinite in the clay fraction (and possibly some calcium montmorillonite). Table 4.5 shows the results for the coarse-grained soils. The fines content of these soils (percentage passing No. 200 sieve) is less than 50%.

4.4 Grain Size Distribution

Figure 4.1, Figure 4.2, Figure 4.3 and Figure 4.4 show the representative grain-size distributions (GSD) of the soils tested during the course of this research. Figure 4.1 presents the grain size distribution of the soils tested in Kokomo, Figure 4.2 presents the representative grain size distribution of the soils tested in North Vernon, Figure 4.3 presents the representative grain size distribution of the soils tested in Utica, Frankfort and Bloomington and Figure 4.4 presents the representative grain size distribution of the soils tested in Delphi.

As can be seen from these figures, fine-grained soils are found predominantly in Bloomington, Utica, North Vernon and Kokomo, while coarse-grained soils are found in Delphi. Some tests performed in Kokomo were on structural backfills (mostly manufactured soils) and therefore have minimal fines content.

4.5 Compaction Test Results

Figure 4.5 and Figure 4.6 present the representative standard Proctor compaction curves for the soils tested in

TABLE 4.4
Combined test results for fine-grained soils

Soil ID	#200 Passing (%)	Clay (%)	PI (%)	Activity	Gs	MDD pcf	MDD kN/m^3	OMC (%)
DCP-F-**A4**-2	65–70	30	≃ 8	0.30	≃ 2.67	115–116	18.4–18.6	14–15
DCP-F-**A6**-1	70–80	20	12–14	0.65	≃ 2.67	113–116	18.1–18.6	14–15
DCP-F-**A6**-2	75–80	20	10–11	0.55	≃ 2.68	≃ 108	≃ 17.3	≃ 17
DCP-F-**A6**-3	85–93	25	12–19	0.52	≃ 2.68	104–106	16.5–17	19–20
DCP-F-**A7**-1	93–98	15	22–24	1.50	≃ 2.68	103–104	16.5–16.7	20–21

TABLE 4.5
Combined test results for coarse-grained soils

Soil ID	#200 Passing (%)	PI (%)	Gs	MDD		OMC (%)
				pcf	kN/m^3	
DCP-C-**A4**-1	40–45	4	≃ 2.65	≃ 132	≃ 21.1	≃ 8
DCP-C-**A1**-1	6–17	Non plastic	≃ 2.65	128–133	20.5–21.5	(8–10]
DCP-C-**A1**-2	10–20	0–4%	≃ 2.65	125–128	20.0–20.5	(10–11]
DCP-C-**A1**-3	8–15	Non plastic	≃ 2.65	121	19.0–19.4	(11–12]
DCP-C-**A3**-1	0–10	Non plastic	≃ 2.65	112–118	17.9–18.9	(12–13]
DCP-C-**A4**-2	50–60	4–5	≃ 2.65	116	18.5	13

accordance with ASTM D698. Figure 4.5 presents the compaction curves for the fine-grained soils, and Figure 4.6 presents the compaction curves for the coarse-grained soils.

4.6 Summary

More than 800 DCP tests were performed on constructed subgrade and embankments in five major INDOT roadwork projects for the purpose of collecting data for the development of correlations between DCP test results and the soil index properties. To collect data for the analysis of the DCP blow counts, a systematic approach was followed. Testing locations were identified where subgrade or embankments were being constructed by INDOT and sets of tests were performed to collect data as soon as the soil was compacted and space was made available for testing.

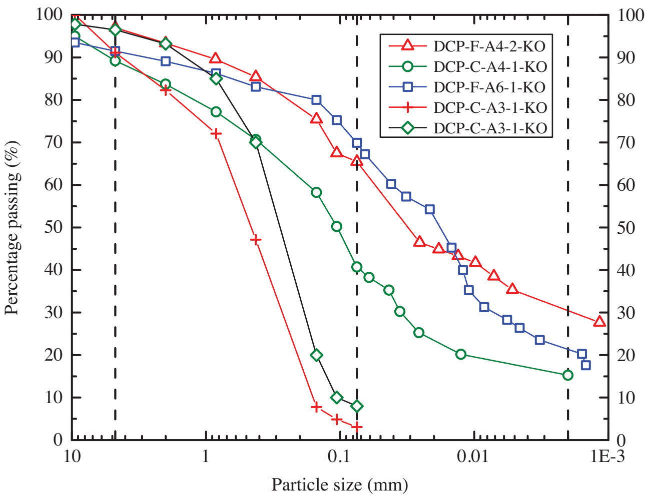

Figure 4.1 Grain size distribution for soils in Kokomo.

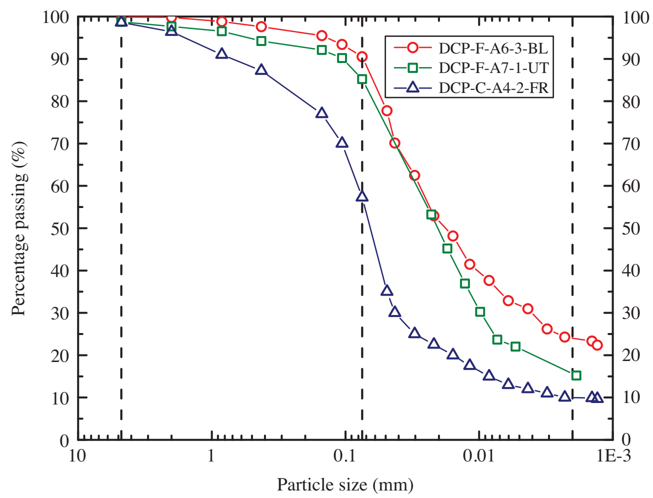

Figure 4.3 Grain-size distribution of soils in Bloomington, Utica and Frankfort.

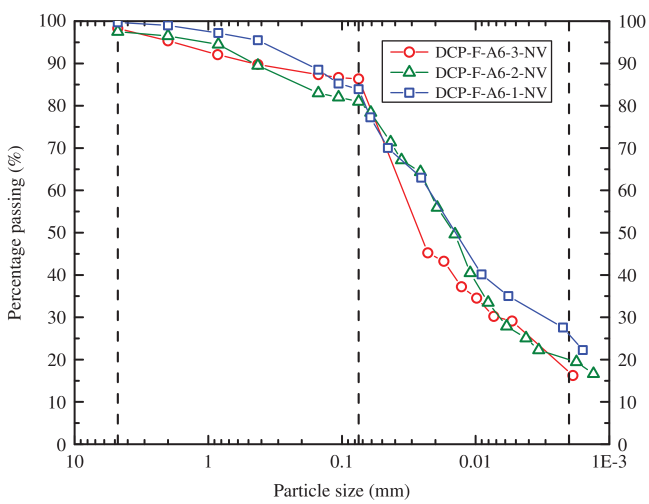

Figure 4.2 Grain-size distribution of soils in North Vernon.

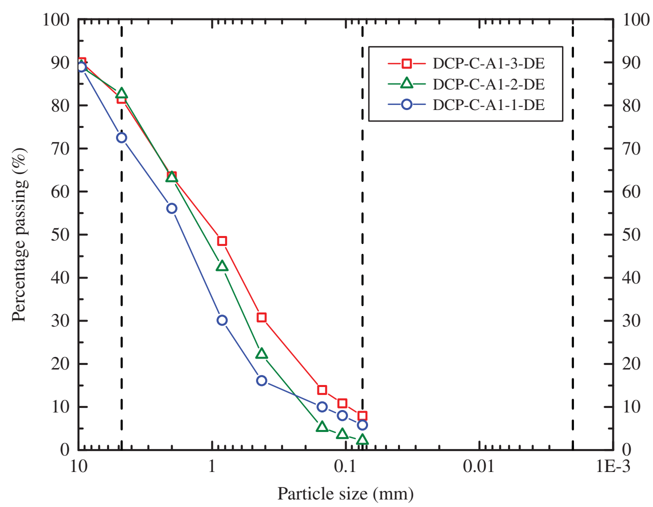

Figure 4.4 Grain-size distribution of soils in Delphi.

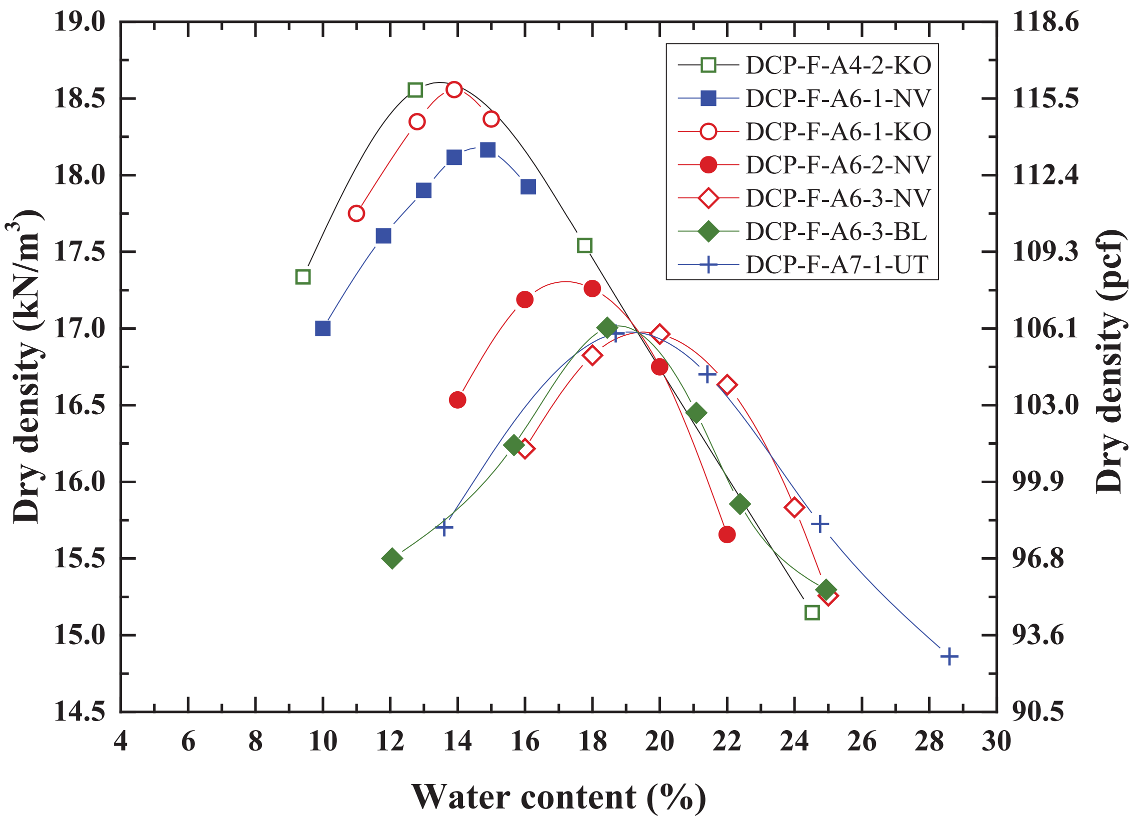

Figure 4.5 Compaction curves for fine-grained soils.

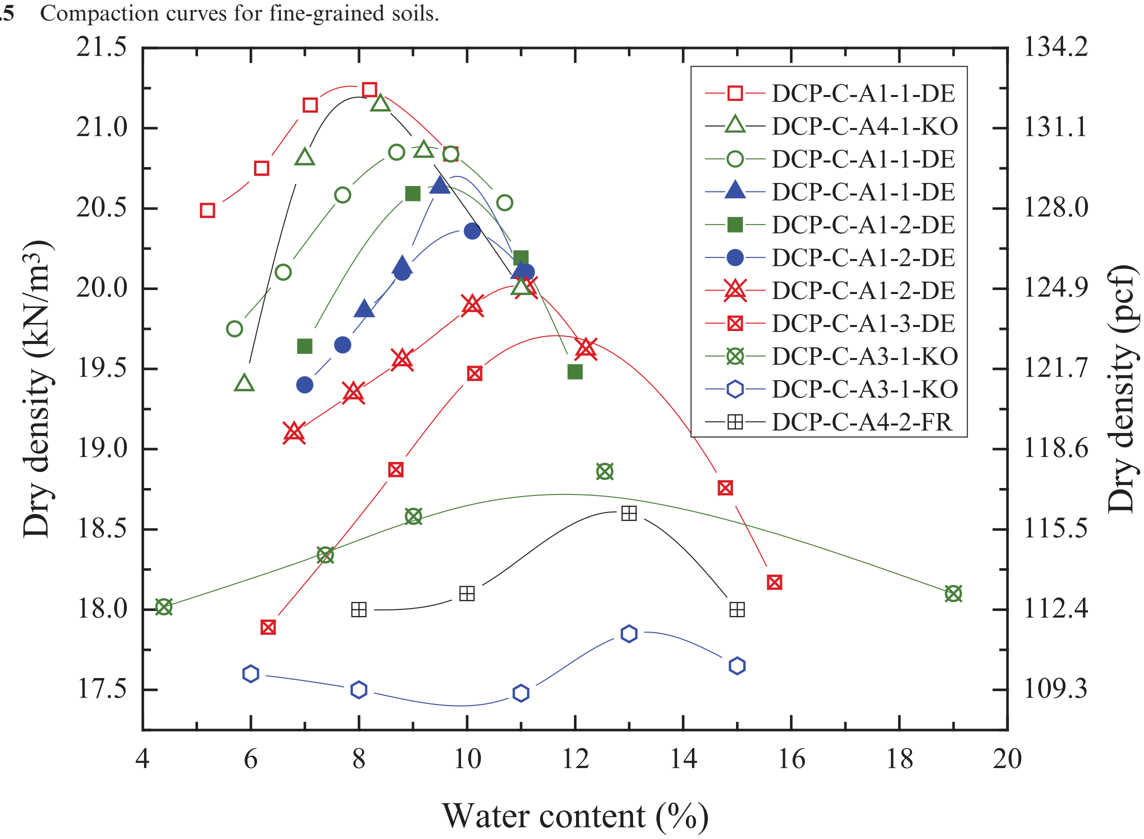

Figure 4.6 Compaction curves for coarse-grained soils.

Owing to the large number of laboratory tests that needed to be performed, soils were identified by a system of naming that allowed the soils to be identified according to their index properties. Soils from each of the 80 locations tested were first categorized as fine-grained soil or coarse-grained soil on the basis of their compaction characteristics, plasticity index and fines content, then grouped into one of the 7 main AASHTO soil types and further sub-grouped within the AASHTO classification on the basis of their standard Proctor OMC. Further subgrouping was done to account for location of testing and set number.

The fine-grained soils tested had PI values ranging from 8% to 24%, fines content above 65–70%, standard Proctor maximum dry density in the 16.5–18.5 kN/m^3 range and OMC in the 15–20% range. Based on their colloidal activity, which ranged from 0.3 to 1.5, it could be stated that the soils most likely had illite and kaolinite in the clay fraction (and possibly some calcium montmorillonite).

The coarse-grained soils, on the other hand, had fines content less than 45% and a PI of less than 4%. The standard Proctor maximum dry density for the coarse-grained soils ranged from 18 to 21 kN/m^3, and the OMC was observed to be in the range of 8–13%.

5. FIELD TEST RESULTS

The objective of this chapter is to present, in a concise manner, the results of DCP tests performed on constructed subgrades and embankments during the course of this study. The individual DCPT results for each test set can be found in Appendix A and Appendix B.

More than 800 DCP tests were performed in sets of 10 tests, thereby giving us a total of about 80 test sets. For each test set, along with the 10 DCP tests, a sand cone test and an *in situ* water content measurement were also performed to obtain the *in situ* dry and wet density of the soil (details of the field testing procedure can be found in chapter 3). Field sand cone tests, combined with the test results of laboratory compaction tests, gives us the RC value of the *in situ* soil during the DCP tests.

This chapter first presents the results obtained from tests performed in the field on fine-grained soil. After the results for fine-grained soils are presented, the chapter details

TABLE 5.1
Field test results for fine-grained soils

		MDD[1]				0 to 6 Inches		6 to 12 Inches				
Test set ID	**Field Test Date**	**pcf**	**kN/m3**	**OMC[2] (%)**	**PI (%)**	**80%**	**90%**	**80%**	**90%**	**RC (%)**	**wc (%)**	**Δwc (%)**
DCP-F-A4-2-KO-1	5/24/13	115	18.4	14	8	8	8	8	9	93.0	11.0	−3.0
DCP-F-A4-2-KO-2						10	10	9	10	94.0	13.5	−0.5
DCP-F-A4-2-KO-3						9	10	14	15	94.0	13.0	−1.0
DCP-F-A4-2-KO-4						10	11	15	15	95.0	13.0	−1.0
DCP-F-A4-2-KO-5						11	12	17	17	97.0	10.5	−2.5
DCP-F-A4-2-KO-6						10	10	15	16	95.0	13.0	−1.0
DCP-F-A4-2-KO-7						11	12	15	17	96.0	12.5	−1.5
DCP-F-A4-2-KO-8						10	11	15	15	95.5	13.5	−0.5
DCP-F-A6-1-KO-1	8/06/13	116	18.6	14	12	11	12	NA	NA	98.5	11.0	−3.0
DCP-F-A6-1-KO-2						15	18	NA	NA	98.5	10.5	−3.5
DCP-F-A6-1-KO-3						12	13	NA	NA	97.8	11.7	−2.3
DCP-F-A6-1-KO-4						13	13	NA	NA	95.5	10.9	−3.1
DCP-F-A6-1-NV-1	7/29/13	113	18.1	15	14	5	6	7	8	93.0	15.0	0.0
DCP-F-A6-1-NV-2						6	6	9	9	94.5	15.5	+0.5
DCP-F-A6-1-NV-3						7	7	11	13	95.0	14.0	−1.0
DCP-F-A6-1-NV-4						7	7	13	13	94.7	13.5	−1.5
DCP-F-A6-1-NV-5						9	9	12	13	96.5	13.5	−1.5
DCP-F-A6-1-NV-6						8	8	9	11	95.5	14.0	−1.0
DCP-F-A6-1-NV-7						8	9	10	10	96.5	13.0	−2.0
DCP-F-A6-1-NV-8						8	9	9	9	96.5	13.0	−2.0
DCP-F-A6-2-NV-1	6/19/13	108	17.3	17	11	7	8	9	11	90.0	13.0	−4.0
DCP-F-A6-2-NV-2						8	9	6	6	91.0	11.5	−5.5
DCP-F-A6-2-NV-3						7	8	11	11	89.0	14.0	−3.0
DCP-F-A6-2-NV-4						6	6	9	10	91.0	12.0	−5.0
DCP-F-A6-3-BL-1	8/20/13	105	16.8	19	19	8	8	8	10	95.0	18.7	−0.3
DCP-F-A6-3-BL-2						7	8	9	10	94.0	19.0	0.0
DCP-F-A6-3-BL-3						7	8	8	8	96.0	18.8	−0.2
DCP-F-A6-3-BL-4						7	8	8	8	96.0	18.5	−0.5
DCP-F-A6-3-NV-1	5/23/13	106	17.0	20	12	12	13	10	15	98.0	17.0	−3.0
DCP-F-A6-3-NV-2						22	23	13	14	95.0	16.0	−4.0
DCP-F-A6-3-NV-3						11	11	9	11	98.0	17.0	−3.0
DCP-F-A6-3-NV-4						9	10	6	7	93.0	20.0	0.0
DCP-F-A7-1-UT-1	5/30/13	104	16.7	20	24	7	8	12	12	96.0	19.0	−1.0
DCP-F-A7-1-UT-2						7	8	14	15	98.0	18.5	−1.5
DCP-F-A7-1-UT-3						7	7	12	14	95.0	18.5	−1.5
DCP-F-A7-1-UT-4						7	7	11	12	94.5	19.5	−0.5
DCP-F-A7-1-UT-5						6	7	16	17	95.5	19.5	−0.5
DCP-F-A7-1-UT-6						5	6	8	8	93.0	19.0	−1.0

[1] MDD: Maximum dry density obtained from the Standard Proctor Compaction Test (ASTM D698).

[2] OMC: Optimum moisture content obtained from the Standard Proctor Compaction Test (ASTM D698).

TABLE 5.2
Field test results for coarse-grained soils

Test set ID	Field Test Date	MDD[1]		OMC[2] (%)	0 to 12 Inches		RC (%)	wc (%)	Δwc (%)
		pcf	kN/m3		80%	90%			
DCP-C-A4-1-KO-1	6/25/13	132	21.1	8	21	23	95	4.9	−3.1
DCP-C-A4-1-KO-2					22	23	95	5.2	−2.8
DCP-C-A4-1-KO-3					27	28	96.5	4.5	−3.5
DCP-C-A4-1-KO-4					24	25	96.5	4.7	−3.3
DCP-C-A1-1-DE-1	7/16/13	133	21.3	8	22	23	96.5	6.5	−1.5
DCP-C-A1-1-DE-2					19[1]	20[1]	95.5	6.2	−1.8
DCP-C-A1-1-DE-3					23	23	95.5	6.3	−1.7
DCP-C-A1-1-DE-4					15	15	90.3	6.4	−1.6
DCP-C-A1-1-DE-5	8/08/13	130	20.8	9	35	36	101.5	6.5	−2.5
DCP-C-A1-1-DE-6					30	33	100.0	7.0	−2.0
DCP-C-A1-1-DE-7					31	31	101.6	6.8	−2.2
DCP-C-A1-1-DE-8					33	34	101.2	7.5	−1.5
DCP-C-A1-1-DE-9	7/16/13	128	20.5	9.5	13	14	91.0	5.0	−4.5
DCP-C-A1-1-DE-10					12	13	91.0	6.0	−3.5
DCP-C-A1-1-DE-11					15	15	93.5	5.8	−3.7
DCP-C-A1-2-DE-1	5/17/13	128	20.5	10.2	41	41	92.5	8.6	−1.4
DCP-C-A1-2-DE-2					10	11	90.0	8.5	−1.5
DCP-C-A1-2-DE-3					38	45	95.6	10.3	+0.3
DCP-C-A1-2-DE-4					50	51	95.6	8.4	−1.6
DCP-C-A1-2-DE-5	8/13/13	127	20.3	10.2	27[1]	28[1]	100.0	9.5	−0.5
DCP-C-A1-2-DE-6					16	16	93.5	9.8	−0.2
DCP-C-A1-2-DE-7					19	20	95.0	9.8	−0.2
DCP-C-A1-2-DE-8					21	21	97.5	9.7	−0.3
DCP-C-A1-2-DE-9	8/13/13	125	20.0	11	16[1]	17[1]	97.5	9.5	−1.5
DCP-C-A1-2-DE-10					18	18	100.0	9.6	−1.4
DCP-C-A1-3-DE-1	5/16/13	121	19.4	12	32	35	98.5	8.5	−3.5
DCP-C-A3-1-KO-1	6/04/13	118	18.9	12.5	9	10	98.0	10.0	−2.5
DCP-C-A3-1-KO-2					9	10	98.5	10.0	−2.5
DCP-C-A3-1-KO-3					10	11	100.1	9.0	−3.5
DCP-C-A3-1-KO-4					10	11	101.1	10.0	−2.5
DCP-C-A3-1-KO-5					11	13	102.5	10.0	−2.5
DCP-C-A3-1-KO-6					9	10	98.5	10.0	−2.5
DCP-C-A3-1-KO-7					11	11	102.0	10.0	−2.5
DCP-C-A3-1-KO-8	7/11/13	112	17.9	13	4	4	90.5	10.0	−3.0
DCP-C-A3-1-KO-9					5	5	91.2	10.0	−3.0
DCP-C-A3-1-KO-10					5	5	90.5	9.0	−4.0
DCP-C-A3-1-KO-11					5	5	90.6	10.0	−3.0
DCP-C-A4-2-FR-1	7/17/14	116	18.5	13	11	12	95	12.0	−1.0
DCP-C-A4-2-FR-2					10	11	95.5	11.9	−1.1
DCP-C-A4-2-FR-3					11	11	96	11.9	−1.1
DCP-C-A4-2-FR-4					10	10	94	12.1	−0.9

[1] MDD: Maximum dry density obtained from the Standard Proctor Compaction Test (ASTM D698).
[2] OMC: Optimum moisture content obtained from the Standard Proctor Compaction Test (ASTM D698).

the results obtained from field tests performed on coarse-grained soils.

5.1 Fine-Grained Soils

Out of the total of 80 test sets, 38 test sets were performed on fine-grained soils following the procedure outlined in chapter 3. DCP blow counts were recorded for 0- to 6-inch and 6- to 12-inch penetration of the DCP into the subgrade. Table 5.1 shows the results of the field tests performed on fine-grained soils. In Table 5.1, the test set ID is followed by the date of the field test, the compaction characteristics (maximum dry density and optimum water content), the plasticity index, the blow counts encompassing 80% and 90% of the respective test results (for 0- to 6-inch and 6- to 12-inch penetration) and the RC and water content of the *in situ* soils at the location of the test set. Also provided in Table 5.1 is the relative difference between the *in situ* water content and the OMC obtained from the laboratory tests performed on the collected samples. Test sets have been arranged in the increasing order of OMC.

5.2 Coarse-Grained Soils

A total of 41 test sets were performed on coarse-grained soils to obtain blow counts for 0- to 12-inch penetration of the DCP into the subgrade. The 0- to 12-inch penetration depth was chosen in accordance with the average lift thickness of the compacted soils of this type. The field data obtained was statistically analyzed to obtain blow counts associated with 80% and 90% of the DCP test results from each test set.

Table 5.2 shows the processed results of the field tests performed on coarse-grained soils in a similar fashion as the fine-grained soils. In Table 5.2, the blow counts encompassing 80% and 90% of the respective test results (for 0- to 12-inch penetration) are presented along with the RC and water content of the *in situ* soils at the location of the test set. Similar to the previous section, the relative difference between the *in situ* water content and the OMC obtained from laboratory tests performed on the collected samples for each test set are also provided in the Table 5.2, but the effect on *in situ* moisture is found to be markedly less than that on fine-grained soils. Test sets have been arranged in the increasing order of their OMC.

5.3 Summary

Out of the total of about 80 test sets, roughly 38 test sets were performed on fine-grained soils and 41 test sets were performed on coarse-grained soils. DCP blow counts were recorded for 0- to 6-inch and 6- to 12-inch penetration of the DCP into the subgrade, and sand cone tests were performed to obtain the *in situ* dry density at the location of each test set. Water content measurements were also taken at each location tested.

For the purpose of development of correlations between the DCP blow counts required for a specific depth of penetration of the DCP into the subgrade/embankment and the soil index properties, different depths were chosen for the two different soil types (fine-grained and coarse-grained). For fine-grained soils, the depth ranges chosen were 0 to 6 inches and 6 to 12 inches, while for coarse-grained soils, the depth chosen was 0 to 12 inches. The choice of depth was based on the average compacted lift thickness of the soils in the field. Table 5.1 and Table 5.2 present the blow counts associated with 80% and 90% of the test results for the fine-grained and coarse-grained soils, respectively. Data presented in the two tables and data from previous DCP field tests were used to develop the QA/QC correlations for the DCP.

6. DEVELOPMENT OF QA/QC CORRELATIONS

This chapter focuses on the development of QA/QC correlations for the DCP using the data collected during the course of this study and presented in chapter 4 and chapter 5.

6.1 Soil Response—Fabric and Moisture Content

To develop meaningful QA/QC correlations between the statistically extracted DCP blow counts and the index properties of the soil, it is first important to understand how different types of soils respond to the impact loads applied by the DCP, and then group them according to their mechanical response. The response of soil to loading is governed mainly by its fabric, moisture content and compaction density.

6.1.1 Soil Fabric

The response of soil to external loading depends on the nature of the interacting particles within the soils mass. In the simplest of classification system, soils can be considered to behave mostly like a sand, or mostly like a clay. In the field, it is highly improbable to find pure sands or pure clays. Therefore, to make an accurate judgment of the expected mechanical response of the soils, it is important to identify what type of behavior will dominate. This depends on the fabric of the soil.

As described by Mitchell and Soga (2005), the fabric is a broad term that describes the type and arrangement of particles that comprise the soil mass. Quantification of the fabric to explain the dominating influence of the clay phase was described by Mitchell (1976), details of which will be described in the coming sections.

According to Mitchell (1976), the behavior of a soil is significantly influenced by the proportion of the clay-size particles in the soil. If the proportion of the clay-sized particles in the soil mass is small enough such that the larger, sand-sized particles are on average in contact with each other, then the behavior of the soil will be dominated by the properties of the sand-sized particles (refer to Figure 6.1 (a)). While on the other hand, if the clay-sized particles reach a certain critical percentage by mass, such that the sand-size particles are surrounded by a layer of clay-size particles and are no more in contact on average, then the behavior of the soil will be governed by the properties of the clay-phase of the soil mass (refer to Figure 6.1 (b)).

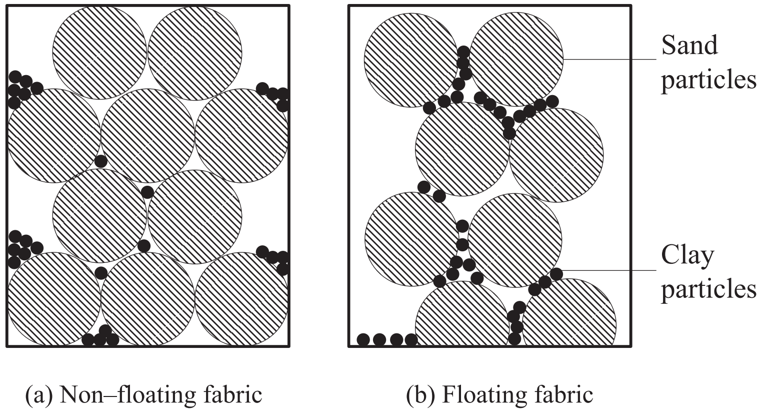

Figure 6.1 Soil fabric dominance: (a) non-floating fabric and (b) floating fabric.

Such a fabric is termed as a floating fabric, because the sand-size particles are seen as floating in a matrix of clay-size particles. It is interesting to note that the proportion by weight of clay-size particles required to reach such a condition is often significantly less than 50%, as we shall see next.

Research was carried out on the effect of non-plastic fines by Salgado, Bandini, and Karim (2000) and of plastic and non-plastic fines by Carraro, Prezzi, and Salgado (2009) on the mechanical response of sands; it suggests that non-plastic fines start to affect the behavior of soils at percentages as low as 20% by weight due to the development of a *floating fabric* in the soil. Therefore, a classification system developed to identify soil behavior as fine-grained or coarse-grained soil needs to consider the limit percentage of the dominant particle size and the nature of the particles contributing to the development of a floating fabric.

As shown in Figure 6.1, a floating fabric is one in which the larger particles get completely surrounded by the smaller particles and, as a result, the volume change and shear behavior of the soil is controlled by the interaction of the smaller particles.

Even though the sand particles may be the major soil phase by weight, the behavior of the soil is still dominated by the clay particles. Consider the derivation of the clay-size proportion required for the development of a floating fabric in a soil mass consisting of binary particle sizes (we assume the presence of only two particle sizes, clay and sand). In reality, the gradation may be more varied, but to simplify the analysis, we make such an assumption.

In order to have a floating fabric, the volume of void V_V in the coarse-grained phase will be filled by the volume of water V_W plus the volume of clay V_C:

$$V_V = V_W + V_C \tag{6.1}$$

The volume of voids V_V in the coarse-grained phase can be expressed as:

$$V_V = e_G V_{GS} \tag{6.2}$$

where e_G is the void ratio of the coarse-grained phase (intergranular void ratio) and V_{GS} is the volume of the coarse-grained phase.

Expressing the weight of the clay particles W_C and the coarse-grained particles W_G in terms of the total weight of solids W_s and the clay percentage C by weight of W_s, we get:

$$W_s = W_C + W_G \tag{6.3}$$

$$W_C = \frac{C}{100} W_s \tag{6.4}$$

$$W_G = \left(1 - \frac{C}{100}\right) W_s \tag{6.5}$$

From the definition of water content wc (%), the weight of water can be written as:

$$W_W = \frac{wc}{100} W_s \tag{6.6}$$

From the definition of G_s, the volume of clay V_C and the volume of coarse-grained phase V_G can be obtained:

$$G_s = \frac{W_s}{V_s \gamma_w} \tag{6.7}$$

For the clay phase:

$$G_{sC} = \frac{W_C}{V_C \gamma_w} \Rightarrow V_C = \frac{W_C}{G_{sC} \gamma_w} \tag{6.8}$$

where G_{sC} is the specific gravity of clay size particles and γ_w is the unit weight of water.

For the coarse-grained phase:

$$G_{sG} = \frac{W_G}{V_G \gamma_w} \Rightarrow V_G = \frac{W_G}{G_{sG} \gamma_w} \tag{6.9}$$

where G_{sG} is the specific gravity of sand size particles.

Substituting equation (6.4) in equation (6.8) and equation (6.5) in equation (6.9), results in:

$$V_C = \frac{W_s}{G_{sC} \gamma_w} \left(\frac{C}{100}\right) \tag{6.10}$$

$$V_G = \frac{W_s}{G_{sG} \gamma_w} \left(1 - \frac{C}{100}\right) \tag{6.11}$$

Substituting equation (6.11) in equation (6.2), we get:

$$V_V = e_G \frac{W_s}{G_{sG}\gamma_w}\left(1 - \frac{C}{100}\right) \quad (6.12)$$

Now, using the unit weight of water γ_w and equation (6.6), we can obtain the volume of water as:

$$V_W = wc\frac{W_s}{100\gamma_w} \quad (6.13)$$

Now, substituting equation (6.10), equation (6.12) and equation (6.13) into equation (6.1):

$$e_G \frac{W_s}{G_{sG}\gamma_w}\left(1 - \frac{C}{100}\right) = wc\frac{W_s}{100\gamma_w} + \left(\frac{W_s}{G_{sC}\gamma_w}\right)\frac{C}{100} \quad (6.14)$$

Solving for C, we obtain:

$$C = \left(\frac{100 \times e_G - G_{sG} \times wc}{G_{sG} + G_{sC} \times e_G}\right) \times G_{sC} \quad (6.15)$$

In order to estimate the clay content required for the development of a floating fabric in the soil mass, the inter-granular void ratio, the water content of the soil mass and the specific gravity of the clay and sand are required. It needs to be re-emphasized here that the derivation described above assumes that: (1) there are only two phases in the soil mass (solid and liquid) and (2) there are only two particle sizes in the soil mass. In natural soils, often there are other particle sizes, and in compacted soils the degree of saturation is not 1 (but generally lies in the range of 0.9–1.0). Therefore, the results obtained from the above equation should give us an estimate of the clay content required for the development of a floating fabric in the soil mass, but not an exact value.

Based on the previous discussions and the requirements of equation (6.15) for computation of clay content required for development of a floating fabric in the soil mass, we need to compute the *inter-granular void ratio*. To get the inter-granular void ratio, we need to have an idea of the *in situ* dry density of the soil mass, from where the *in situ* global void ratio *e* (Volume of voids/Volume of solids) can be calculated using:

$$e = \frac{G_s \times \gamma_w}{\gamma_d} - 1 \quad (6.16)$$

Using equation (6.16) and assuming a G_s value of 2.65, we can estimate the global void ratio. From here, we can have an estimate of e_G (inter-granular void ratio). But we still need the value of the *in situ* dry density γ_d corresponding to the point when the soil starts to develop a floating fabric.

To get an estimate for *in situ* dry density, consider the case of compacted soils. The *targeted* dry densities and *in situ* moisture content for compaction in the field are fairly clear and depend on the compaction tests performed on the soil samples in the laboratory prior to the field compaction process. Soils are compacted in the field at or near the OMC obtained from Proctor tests performed in the laboratory to achieve the maximum dry density or a certain percentage of it. The range of the targeted *in situ* density can be obtained from a family of compaction curves, as shown in Figure 6.2.

Given that a clear idea of the targeted *in situ* compaction densities is available, a reasonable approach to obtaining the *in situ* dry density of the soil mass as it starts to develop a floating fabric would be to observe the targeted *in situ* compaction densities of the soils which exhibit a change in the response from sand-dominated to clay-dominated behavior. In general, this transition occurs for maximum compaction density values in the 110–120 pcf range and for OMC values in the 12–15% range. These ranges of maximum dry density and optimum water content are associated with transitional soils, sandy loams and clayey silts (Holtz et al., 2010), *i.e.*, as soils transition from coarse-grained behavior to fine-grained behavior. Therefore, a glimpse of the answer

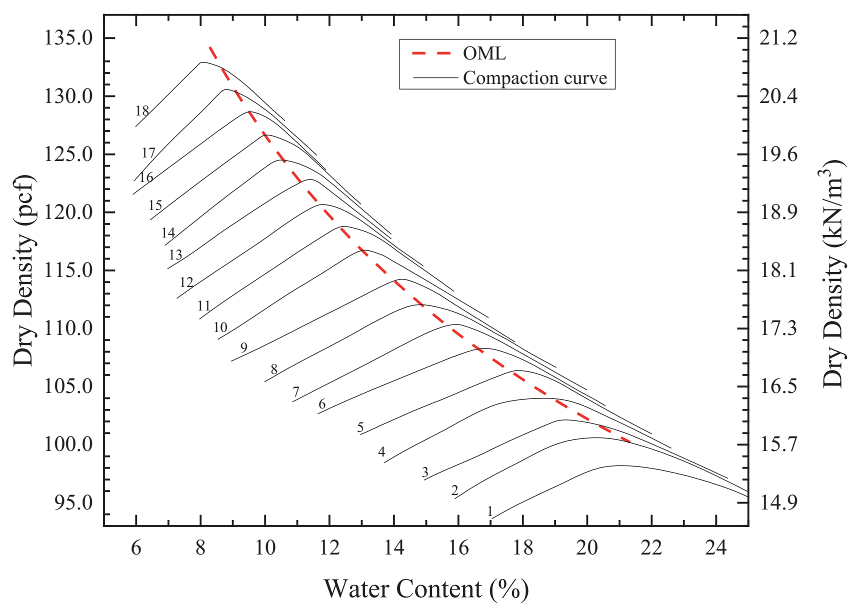

Figure 6.2 Family of standard Proctor compaction curves and optimum moisture line (OML) (from the INDOT family of curves; modified after Kim et al. (2010)).

to the question "What clay content is required for a floating fabric to develop?" can be acquired from here.

The global void ratio (volume of voids/volume of solids) of transitional soils for their *in situ* density and specific gravity is around 0.4–0.5 (based on their *in situ* dry density); therefore the inter-granular void ratio must be higher. For sands in their loosest state, the inter-granular void ratio is around 0.9 (maximum void ratio for sand in its loosest state). Based on the above values, we can reasonably assume that the inter-granular void ratio for compacted soils can be in the range of 0.5–0.6.

Knowing that the water content of the soil in its compacted state will be about 12–14% (compaction water content in the field for the soils of interest), the clay content required for the development of a floating fabric for various values of water content and inter-granular void ratios can be plotted, as shown in Figure 6.3.

From the above results of water content versus clay content required to develop a floating fabric for three different values of inter-granular void ratios, the clay content at which a floating fabric develops in the soil mass is about 15–20%. Therefore, as the clay content in compacted soils is greater than about 15–20%, the behavior of the soil changes from sand-like to clay-like. Similar percentage values were obtained by Salgado et al. (2000) from tests performed on mixtures of sands with non-plastic fines, indicating that a floating fabric starts to form in the soil mass when the percentage by weight of non-plastic fines reaches about 20%.

For the soils found in Indiana, the clay fraction generally consists of mixtures of illite, kaolinite and calcium montmorillonite according to data provided by INDOT and the colloidal activity (Pandian & Nagaraj, 1990; Skempton, 1953) of the soils that were tested for this research. In order to determine the accurate clay content of a soil compacted in the field, hydrometer tests would need to be performed. Generally, soils with fabric dominated by clays have OMC values above 15% and a maximum dry density in the range of 105–110 pcf (16.5–17.5 kN/m^3). It is recommended that for soils with OMC above 15%, hydrometer tests be performed to properly characterize their nature. Nevertheless, an approximate estimate of the clay content required for a soil to develop a floating fabric can be obtained from its plasticity index and clay mineral type.

Figure 6.4 shows a plot of PI vs. clay content for various values of activity together with the data obtained for the soils tested in this research (the locations within the state of Indiana at which the PIs and clay contents were determined can also be seen in Figure 3.1). Using Figure 6.4, an estimate of the PI of the soil at which it starts to develop a floating fabric can be made. Assuming a colloidal activity of 0.5 (the most common value in the state of Indiana) and knowing that 15% to 20% of clay can cause the soil to transition from sand-like to clay-like soil behavior, the PI corresponding to such a state is in the 8–10% range.

Since the PI is more easily evaluated in the field than the clay fraction of a soil, it can serve as an indicator of the transition from sand-dominated to clay-dominated behavior. Albeit convenient, the use of PI as an indicator must be taken with utmost caution as a change in the colloidal activity and, hence of the clay mineral type, can change the PI percentage associated with the transition. Further research in this direction can prove to be fruitful for practical applications for the DOTs.

6.1.2 Moisture Content

Once the behavior of a soil mass is identified as either fine-grained or coarse-grained, it becomes important to address the issue of the soil water content at time of DCP testing. For

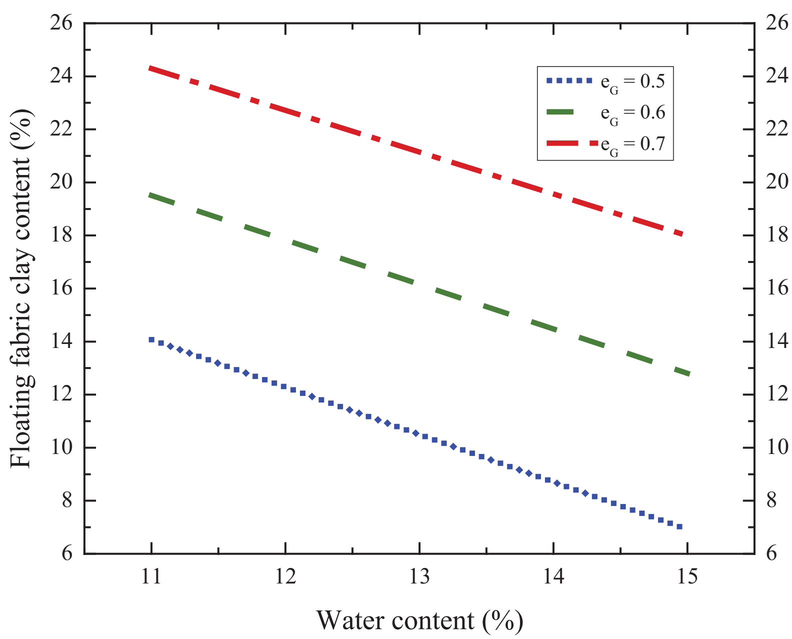

Figure 6.3 Clay content required for the development of floating fabric with respect to the water content and inter-granular void ratio of the soil.

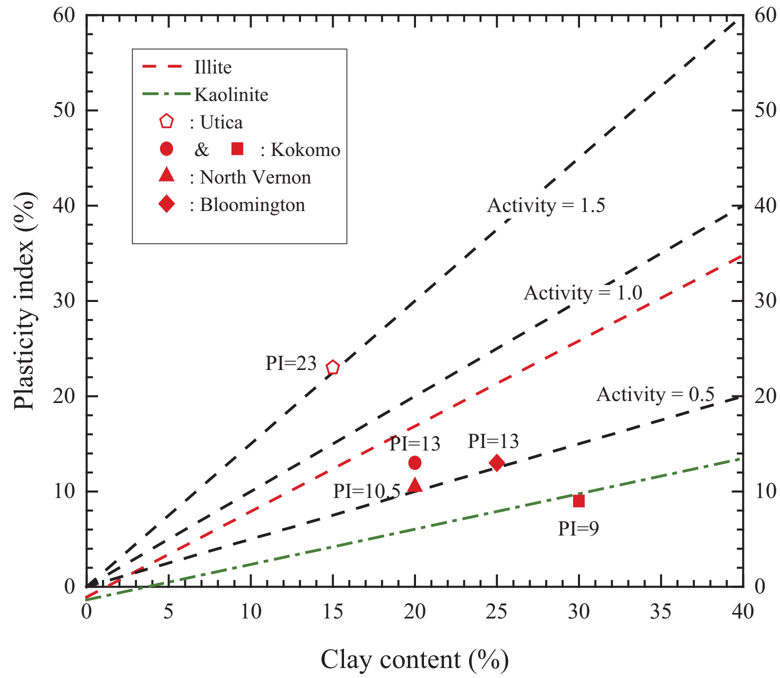

Figure 6.4 Plasticity index vs. clay content of soils in Indiana (after Skempton, 1953).

fine-grained soils, the soil state variable called matric suction (difference of pore air and pore water pressures) plays an important role in the mechanical response of the soil (Blight, 2013; Fredlund, Rahardjo, & Fredlund, 2012). As the degree of saturation and water content of the soil mass increase, matric suction decreases.

When the water content of a compacted soil is in a range that leads to an unsaturated state, the water forms thin films (contractile skin) at the air-water interface and pulls the particles together, resulting in increased confinement (attributed to the increase in the suction in the soil). Figure 6.5 shows an idealized schematic of the contractile skin in unsaturated soil. The magnitude of this increase in confinement depends on the soil type and is found to be more prominent in fine-grained (clay-dominated) soils, which have larger specific surface area, and thus a larger area for the contractile skin to pull at, as compared to coarse-grained (sand-dominated) soils.

As confinement changes, the strength of the soil also changes, which in turn causes a change in the response of the soil to the DCP impact loading. Therefore, in order to develop reliable QA/QC correlations, it is necessary to

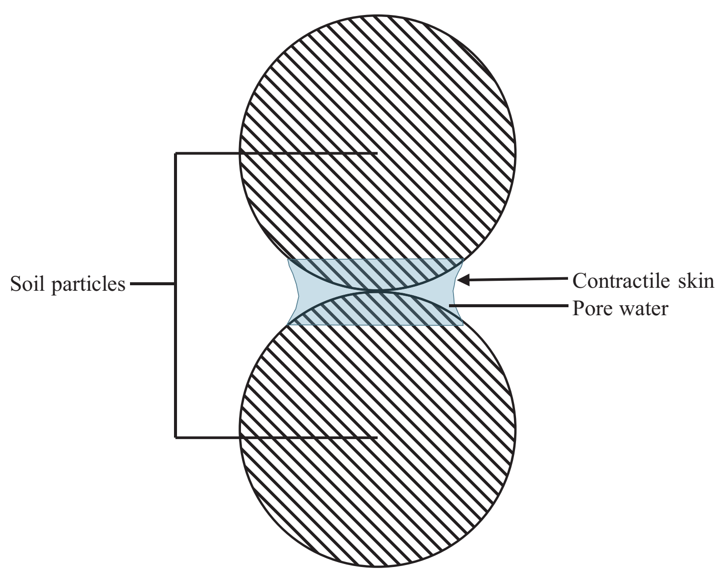

Figure 6.5 Formation of contractile skin in unsaturated soils.

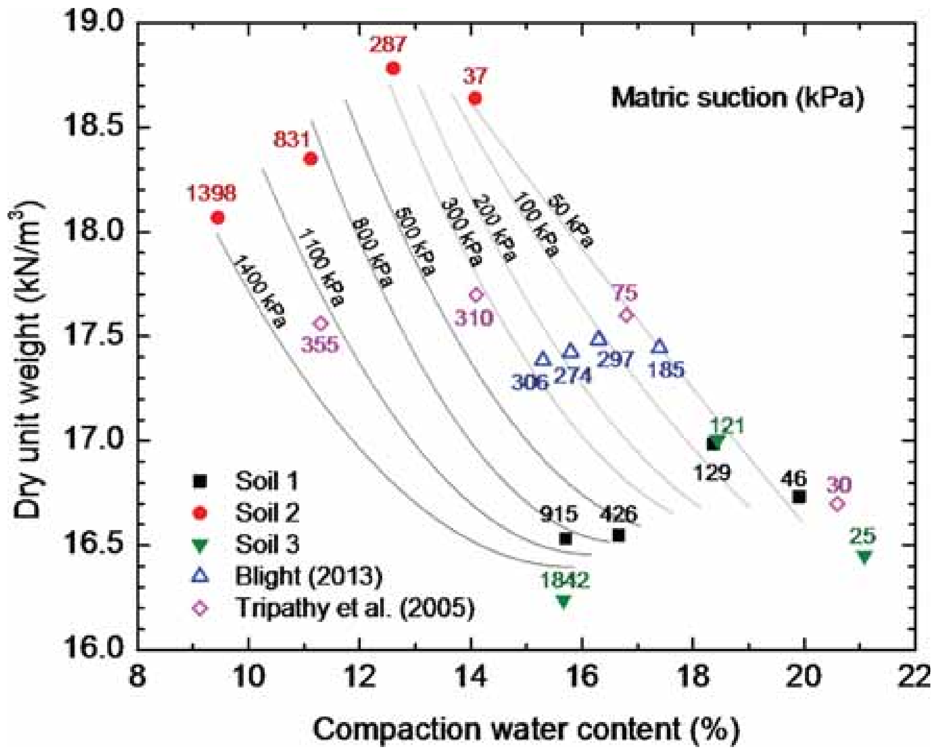

Figure 6.6 Variation of matric suction at different compaction states for fine-grained soils (Kim et al., 2014).

take into account the effect of matric suction on DCP test results (*i.e.*, the DCP blow count measured in a soil compacted in the field depends on its degree of saturation).

Figure 6.6 shows results of matric suction measurements carried out (using the filter paper technique) by Kim, Ganju, Tang, Prezzie, and Salgado (2014) at various compaction states of different soils, in the form of suction contours on the dry density-compaction water content space for fine-grained soils. The properties of the soils in Figure 6.6 are shown in Table 6.1 and Table 6.2.

From Figure 6.6 it can be seen that as we move towards the dry-side of optimum, matric suction increases and, as we move towards the wet-side of optimum, it decreases. An increase in matric suction causes the soil to become stiffer due to the increase in confinement experienced by the soil particles, it contributes to an increase in the blow counts obtained from DCP tests.

It is necessary to quantify this aspect of the variation in DCP test results with changes in *in situ* moisture. High blow counts may not necessarily mean achievement of proper compaction and exceedance of the target RC and,

TABLE 6.1
Index properties of soils tested by Kim et al. (2014)

		Plasticity (%)									
ID	**Sample Location in Indiana**	**PL**	**LL**	**PI**	**Silt (%)**	**Clay (%)**	**MDD (kN/m^3)**	**OMC (%)**	**Gs**	**USCS**	**AASHTO**
Soil 1	Utica	20.2	40.6	20.4	75	18	16.8	18.3	2.68	CL	A-7-6
Soil 2	Kokomo	14.2	27.3	13.1	30	33	18.7	12.8	2.67	CL	A-6
Soil 3	Bloomington	19.4	39.1	19.7	67	24	16.7	18.6	2.67	CL	A-6

TABLE 6.2
Index properties of soils from the literature (after Kim et al., 2014)

		Plasticity (%)							
Authors	**Soil Name**	**PL**	**LL**	**PI**	**Silt (%)**	**Clay (%)**	γ**dmax (kN/m^3)**	**wcopt (%)**	**USCS**
Blight (2013)	Clay residual from shale	16	38	22	NA	21	17.5	16.2	NA
Tripathy et al. (2005)	Mudstone residual soil	28	42	14	42	11	17.7	15	CL

conversely, low blow counts may not necessarily mean non-achievement of the target RC value. Soils may have been compacted adequately above optimum, but because of lower suction values, lower blow counts may result. Therefore, the *in situ* moisture at the time of DCP testing plays an important role in the interpretation of the DCP test results and is a necessary component of the interpretation of the DCP test results.

Soils whose blow counts are sensitive to moisture content changes are the most important ones. As described before, fine-grained soils are the most important ones in regard to moisture sensitivity. In this study, to ensure that lower blow counts were not recorded due to higher moisture in the field, test results obtained from the field were carefully assessed and only those that had *in situ* water content either at the OMC or within −2% of the OMC were used in the development of QA/QC correlations. Furthermore, to supplement the correlations proposed in this report, results obtained from moisture sensitivity tests carried out by INDOT were also analyzed and described later in this chapter.

6.2 Grouping of Soils

For the purpose of development of QA/QC correlations for the DCPT, taking into account the role of fabric and moisture sensitivity of the fine-grained (clay-dominated) soils on DCP blow counts, natural soils have been grouped into two major categories: coarse-grained or sand-dominated soils and fine-grained or clay-dominated soils.

Naturally occurring sand-dominated soils are the ones in which the fabric is dominated by sand-size particles. In general, these soils usually have standard Proctor maximum dry density of about 120–135 pcf (18–20 kN/m^3) and OMC in the 8–12% range. On the other hand, clay-dominated soils usually have standard Proctor maximum dry density of about 95–110 pcf (14–16.5 kN/m^3) and OMC in the 15–22% range.

Soils with compaction characteristics in between these two ranges are referred to as transitional soils (soils transitioning in behavior from coarse-grained to fine-grained) and, therefore, can be classified as either coarse-grained or fine-grained depending on whether a floating fabric develops in the soil mass.

Based on these considerations, soils with maximum standard Proctor dry density between 110–120 pcf (16.5–18 kN/m^3) and OMC between 12% and 15% are expected to behave as fine-grained when the PI of these soils is higher than 8–10%. Figure 6.7 shows the demarcation of the three ranges of behavior exhibited by the soils on the compaction plane: coarse-grained, fine-grained and transitional. Also shown are the compaction properties of the natural soil tested in this study.

Table 6.3 gives the complete criteria for classification of soils into coarse-grained or fine-grained. It is to be noted here that the transitional soils are re-classified as coarse-grained or fine-grained and the corresponding correlations for coarse- or fine-grained soils are used to obtain their blow count values. Note also that certain manufactured soils, such as those used by INDOT for backfill of bridges (referred to as granular soils in the INDOT jargon), are also categorized as coarse-grained soils, but their compaction characteristics fall below the optimum moisture line in the transitional zone. These “manufactured soils” have been shown to have good correlations between the DCP blow count and *Cu* (coefficient of uniformity).

Similar boundaries, as seen in Figure 6.7, have also been observed by White et al. (2002) who made the demarcation between fine and coarse-grained soils (termed by White et al. (2002) as “cohesive” and “non-cohesive soils”) using the percentage passing the No. 200 (75 μm) sieve and the plasticity index of the soils estimated from the percentage passing the No. 40 (425 μm) sieve.

According to White et al. (2002), soils with percentage passing the No. 200 sieve greater than 36% by dry weight were classified as “cohesive” or clay- dominated soils, and soils with percentage passing the No. 200 sieve less than 16% by dry

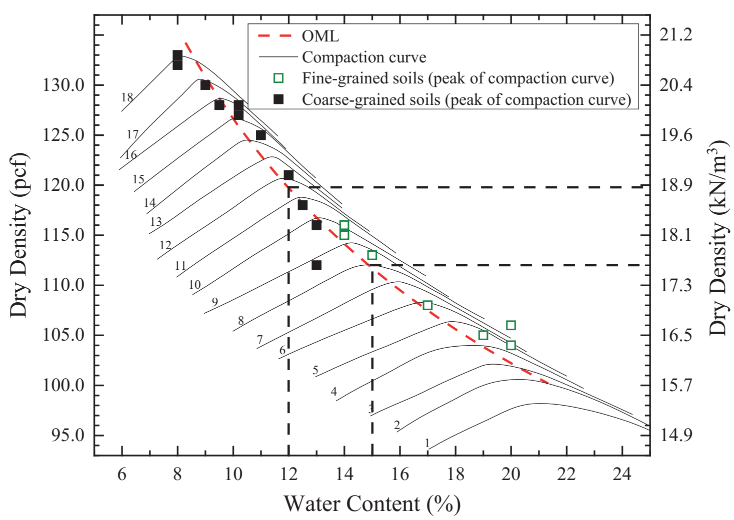

Figure 6.7 Soil behavior and compaction properties (INDOT family of curves).

TABLE 6.3
Soil grouping

Soils Group (C/F)		OMC (%)	MDD pcf	MDD kN/m3	PI (%)	Percentage Passing #200 Sieve (0.075 mm)
Sand-dominated soils	Natural soils	<12	>120	>18.9	Non plastic	<25
	Manufactured soils	12–14	112–118	18–19	NP, fine sands	0–8
Silt-dominated soils	Sand type	[12–15)	(110–120]	(17.3–18.9]	<9	<60
	Clay type				>9	>60
Clay-dominated soils		>15	<110	<17.3	>9	>60

Note: Silt-dominated soils are classified as either sand type or clay type soils for use of the DPC blow count correlations. For silt-dominated soils of the sand type, the correlations developed for coarse-grained (or sand dominated) soils should be used, while for those of the clay type, correlations developed for fine-grained (or clay dominated) soils should be used instead.

weight were classified as "cohesion-less" or sand-dominated soils. Furthermore, White et al. (2002) introduced a third soil classification called "intergrade", which represents the transition from clay-dominated to sand-dominated soils, with percentage passing the No. 200 sieve between 16% and 35%. In addition, if the intergrade soils had a liquid limit greater than 40% and a plasticity index greater than 10%, the "intergrade" soils were reclassified as "cohesive".

Based on the results of the tests performed on silty soils, it was observed that even when the percentage passing the No. 200 sieve was about 50–60%, the soil behavior was still not strongly dominated by the fines fraction and the DCP blow counts were more in line with the sand-like soils. This can probably be attributed to the nature of fines. Not all fines are created equal and the presence of non-plastic fines is different on the soils than that of plastic fines.

The compaction characteristics and PI seem to be better indicators of the expected behavior of the soil mass (assuming we know the type of clay mineral present in the soil from its activity) than only the percentage passing the No. 200 sieve. A PI of less than 8–10% (depending on the clay minerals that comprise the fines fraction of the soil) would indicate that the transitional soil is dominated mostly by non-plastic fine particles and will most probably tend to behave similar to sands. From the data, it was observed that the fines content of the soils with PI in this range was low.

It is necessary to point out here that the boundaries of the various index properties used for soil classification are subject to some variation due to variability associated with index testing. If a soil lies right on or very near a boundary, it becomes necessary to analyze the soil for its dominant fabric and make a suitable judgment accordingly.

6.3 Development of DCP Blow Count Correlations

With a system of classification of soil behavior in place, QA/QC correlations were developed for coarse-grained (sand-dominated soil, both manufactured and natural) and fine-grained (clay-dominated soil) soil groups. The following sections presents the developed DCP blow count correlations for compaction quality control.

6.3.1 Coarse-Grained Soils

For sand-dominated soils, correlations were developed for the blow counts required for 0- to 12-inch penetration of the DCP probe into the compacted soil mass. A penetration of 0 to 12 inches was selected based on the average lift thickness used during the compaction process in the field. For these types of soils, it was found that the DCP blow counts had very good correlations with the OMC of the soil.

A higher OMC implies a lower compaction density (see Figure 6.2), which in turn implies a lower DCP blow count for 0- to 12-inch penetration. Figure 6.8 shows the plot of the DCP blow count obtained from the statistical analysis of the field DCP data (for soils compacted to RC 95%) vs. the OMC of the soil obtained from Proctor tests performed in the laboratory. At each of the points plotted, a representative value of the *in situ* moisture content at the time of testing is plotted next to the data point as Δwc, which is equal to the difference between the OMC obtained from the standard Proctor test performed in the laboratory and the field moisture content at the time of DCP testing. A positive value indicates that the *in situ* water content was above the OMC at the time of DCP testing, and a negative value implies that the *in situ* water content was below the OMC at the time of DCP testing.

In Figure 6.8, we can see that the solid (red) line shows the blow counts for 0- to 12-inch penetration for natural soils with some fines content, and the dashed (green) line shows the 0- to 12-inch penetration blow counts for manufactured soils (which in the INDOT jargon is referred to as *granular soils*), which have virtually no fines content. The line for the natural soils, because of their higher density, plots above that for the manufactured soils even though they both have the same compaction water content. The Δwc may appear to be quite large, but since these are coarse-grained soils, the effect of *in situ* moisture is not significant on the DCP blow count as compared to the density of the soil. Provided that the targeted dry density is reached in the field, the blow counts are not significantly affected by a relatively small change of *in situ* moisture content at the time of testing.

The DCP correlations for the natural coarse-grained soils can also be presented in a different way, as shown in Figure 6.9, which includes the DCP blow counts associated with both 95% and 100% RC (appearing on the right and

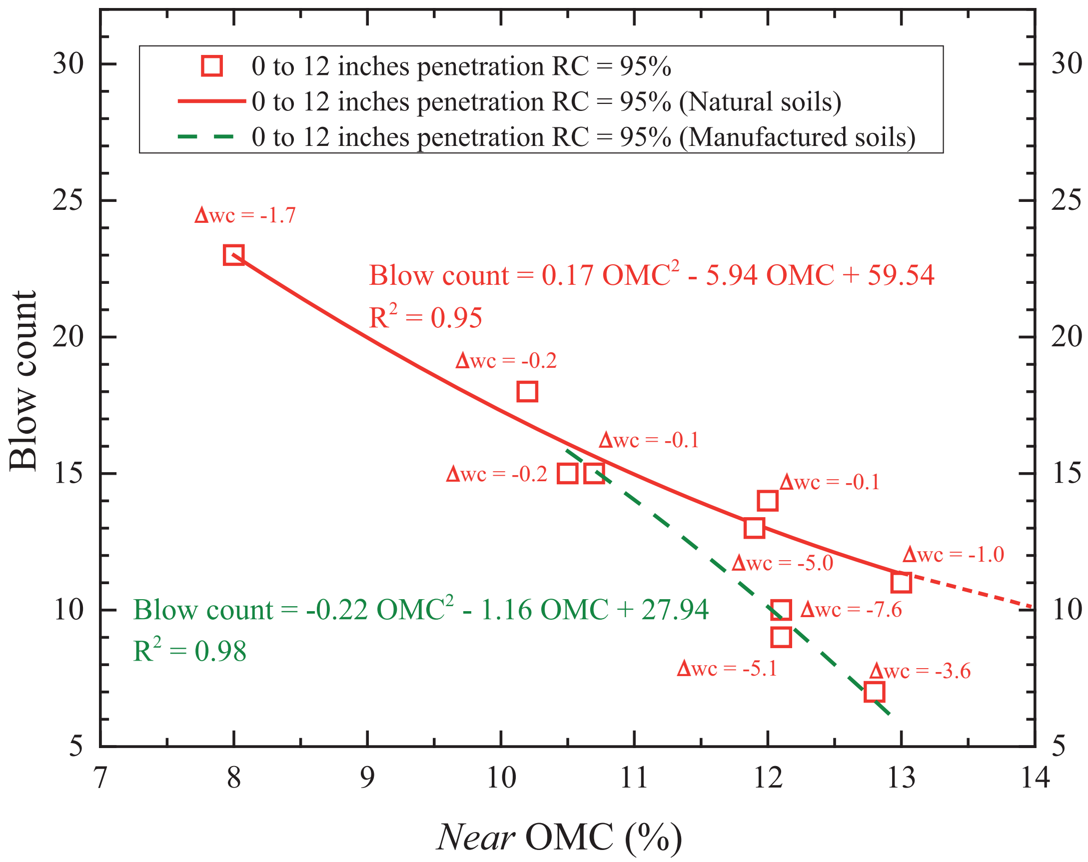

Figure 6.8 Blow count for 0- to 12-inch penetration into compacted soil vs. near OMC of compacted soil.

top axes). The optimum moisture line for coarse-grained soils (the solid line) is also shown in Figure 6.9. Such a chart can be used by field engineers to get a quick estimate of the target DCP blow count required for the compacted soil to achieve 95% or 100% relative compaction, knowing the laboratory compaction test results (maximum dry density and OMC) or the one point Proctor data from the field. Whenever possible, it is preferable to use Figure 6.8.

The DCP blow counts measured for manufactured sands (soils with almost no fines, and composed predominantly of sand-size particles, such as structural backfill and B-borrow sands) had a good correlation with the coefficient of uniformity ($C_u = D_{60}/D_{10}$), which can be easily obtained from a particle size distribution curve. Since these soils have a low percentage passing the No. 200 sieve (less than 10%), it is easier to estimate the C_u in the field (no need to perform hydrometer tests). As the coefficient of uniformity of the soil increases, a wider range of particle sizes are included in the soil mass, as a result of which, the soil can achieve a denser compacted state, and a greater DCP blow count. Figure 6.10 shows a plot of the blow counts for 0- to 12-inch penetration of the DCP, for soil compacted to 95% RC, vs. the C_u of the soil. For these soils, it was observed that the DCP blow counts for 100% RC were approximately 2 blows above those for 95% RC. Further testing may help in refining this blow count increase.

For INDOT, the manufactured soils are generally of 3 main types, No. 4, No. 30 and ½ inch (named according to INDOT jargon; Indiana Department of Transportation 2012). For these three soils, the C_u is generally in the range of 3–6, 2–3 and 6–8, respectively. Using the previously described criteria for manufactured coarse-grained soil and the C_u ranges, the recommended DCP blow counts for these three classes of soils are given in Table 6.4. The data presented is tabular form can be used by the engineers in the field to quickly assess the recommended blow counts.

6.3.2 Fine-Grained Soils

QA/QC correlations were developed for clay-like soil groups for blow counts required for 0- to 6-inch and 6- to 12-inch penetration of the DCP probe into the compacted soil. The penetration depths were chosen on the basis on the average lift thickness of the soils compacted in the field.

For soils with fabric dominated by the clay fraction, the DCP blow counts for 0–6 and 6–12 inch penetration of the DCP had a very good correlation with the plasticity index of the soil. Figure 6.11 shows the DCP blow count correlations

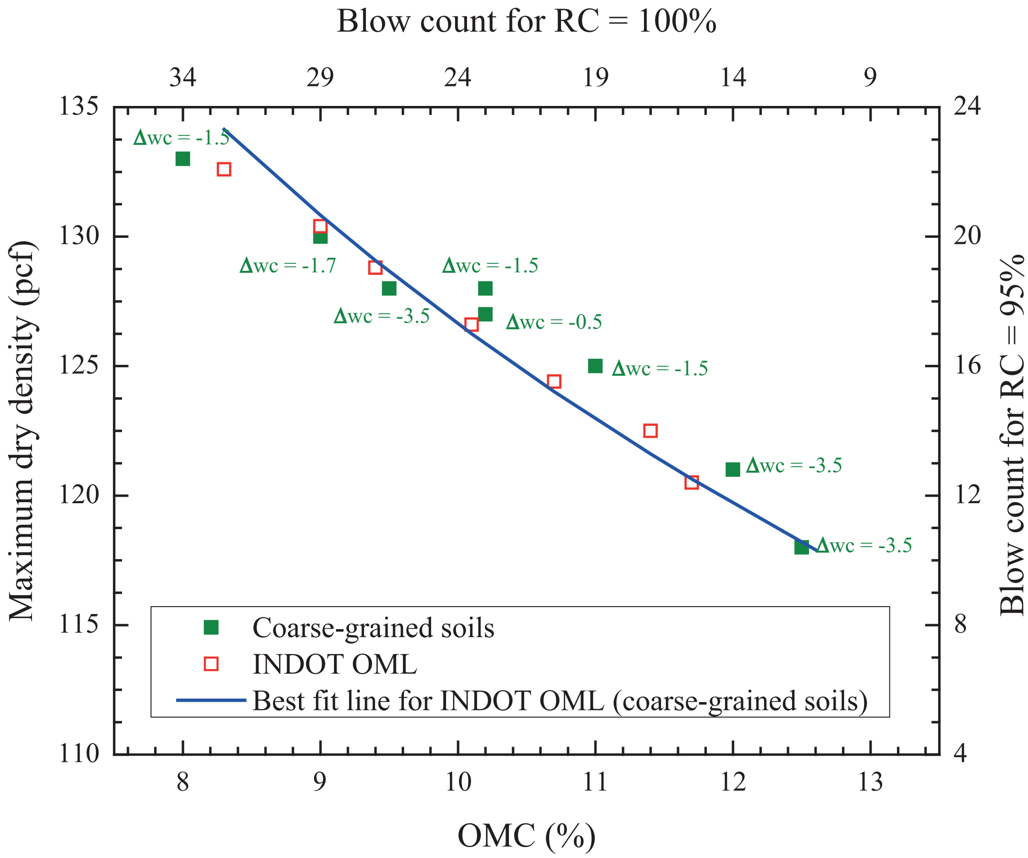

Figure 6.9 Four-way blow count chart for 100% and 95% RC.

together with the data collected during the course of this project. As can be seen in the Figure, the R^2 value is above 0.98. Also, the DCP blow counts seem to stabilize to approximately 7–9 blows for 0- to 6-inch penetration and to 10–12 blows for 6- to 12-inch penetration for PI values larger than 14%.

In Figure 6.11, Δwc represents the difference between the *in situ* water content measurement of the compacted soils and the OMC obtained from Proctor compaction tests ASTM D698. To ensure that the moisture controls were maintained, only test results with *in situ* water content within -2% of the OMC were used to develop the correlations presented in Figure 6.11.

The DCP blow count values should be either obtained from the chart shown in Figure 6.11 or calculated using the developed correlations. However, in the field it may be difficult to perform the Atterberg limits tests to determine the PI of the soil to be compacted, and often only the one-point Proctor tests are performed to get estimates of the OMC and maximum dry density of the soil. Therefore, for a quick and approximate range of DCP blow counts in terms of the OMC of the fine-grained *in situ* soil, we can suggest a blow count value for 0- to 6-inch penetration of about 8–10 blows for soils with OMC values between 15% and 18%. For soils with OMC in the range of 18–22%, a blow count value for 0- to 6-inch penetration of about 7–8 is suggested. These ranges are recommended for fine-grained soils with *in situ* moisture within the range of OMC -2%. It must be emphasized again that the most accurate DCP blow count value can be acquired from the plot in Figure 6.11 and the above mentioned values are just an approximation for the convenience of the site engineer.

In addition to the correlations developed by research done at Purdue, moisture sensitivity calculations were also carried out at INDOT to ascertain the variation in DCP blow counts caused by variation in *in situ* moisture content. As pointed out before, DCP test results should always be supplemented with *in situ* moisture data. DCP test data without accompanying moisture data can lead to misinterpretation of results.

In the tests performed by INDOT, soils were compacted at moisture contents of OMC -2%, OMC and OMC $+2\%$ in a CBR mold (with collar attached) using a falling-weight hammer (following standard Proctor test protocols). Once compacted, DCP test were performed in the CBR mold to obtain the blow counts required for 0- to 6-inch penetration for samples compacted at different moisture contents. The CBR mold itself has a diameter and depth of 6 inches, the addition of a collar allows for an increase in 3 inches in height. Therefore, the DCP tip is at a distance of 3 inches

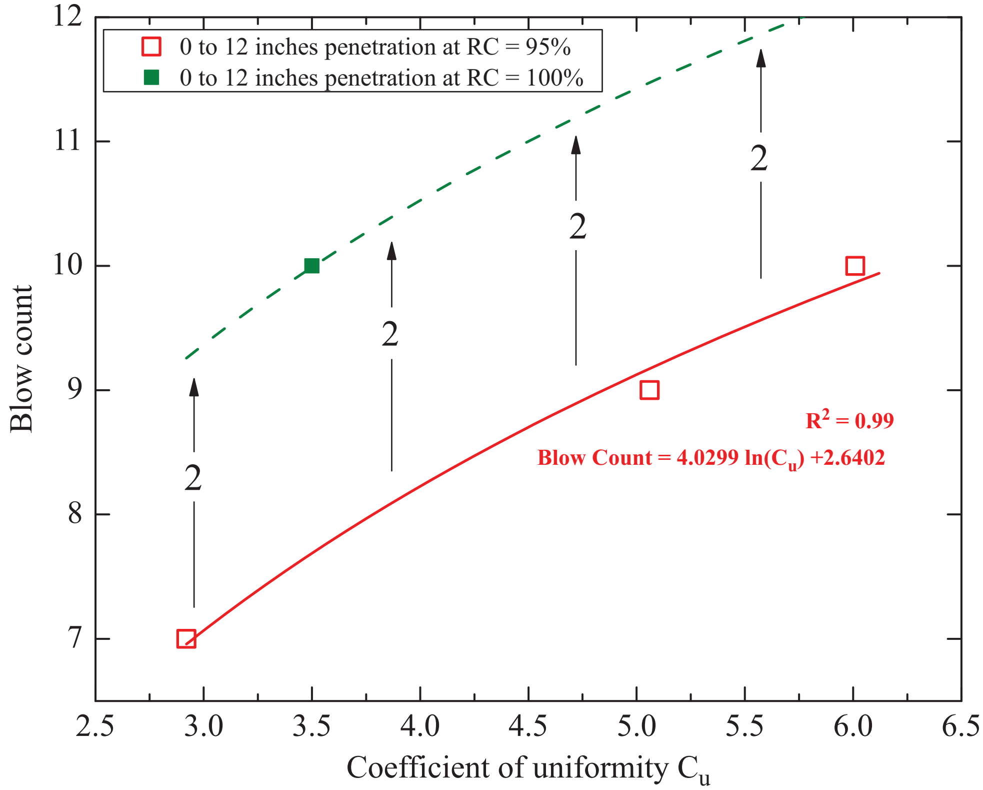

Figure 6.10 DCP blow count correlations for clean sands.

from the base and the wall of the mold. These measured blow counts results are thus affected by the boundaries of the CBR mold.

Being mindful of these limitations, the data obtained from the tests was analyzed and only the data from soils which were compacted at the desired density (RC = 95±2%) and at moisture content of ±0.75% of the target moisture content were used to assess soil moisture content sensitivity of DCP blow counts.

The index properties of the soils used by INDOT in the moisture sensitivity test are shown in Table 6.5. Figure 6.12 shows the plot of the results obtained from performing DCP tests on soils compacted at OMC −2%, OMC, and OMC +2% along with the correlations (between DCP blow count for 0- to 6-inch penetration into fine-grained soil and plasticity index of the soil) developed from the research carried out at Purdue in dashed lines. From Figure 6.12, an approximate idea about the variation in DCP blow count with change *in situ* compaction moisture can be obtained. For low PI (8–16%) values, the blow count variation with change in compaction moisture is more significant than for high (>16%) PI values.

In general, soils compacted at OMC +2% often have a low CBR value (approximately 4–5%; Webster et al., 1992), and even though the compaction specification of 95% relative compaction may have been achieved, the stiffness may not be high enough for the soils to be suitable for construction. Based on this, it is recommended that the compaction moisture content of fine-grained soils, whose stiffness is significantly affected by the moisture content at the time of compaction, be limited to a maximum of +1% above the OMC. The lower bound of OMC −2% can be maintained. The upper moisture limit of OMC +1% is set to ensure that the minimum required DCP bow count at any time is at least 7 DCP blows (corresponding to CBR values of approximately 8–9%) for 0 to 6 inches of penetration.

TABLE 6.4
Tabular form of recommended blow count values for manufactured coarse-grained soils (INDOT jargon = *granular*) commonly used by INDOT in earthwork projects

		0- to 12-Inch Blow Count	
Soil Type	C_u	**RC = 95%**	**RC = 100%**
No. 30	2–3	6–7*	+2**
No. 4	3–6	7–10*	+2**
½ inch	6–8	10–12*	+2**

*Note that we have test results from field testing of clean sands with *Cu* between 3 and 6, blow counts for *Cu* outside of this range have been extrapolated

**Based on the test results from the field, the blow counts for RC of 100%, in case of clean sands, was about +2 of the blow count for RC of 95%

6.3.3 Combined Correlations

Table 6.6 provides the DCP blow count correlations developed for coarse-grained soils and fine-grained soils, and Table 6.7 shows the suggested blow counts for the implementation of correlations for different types of soils

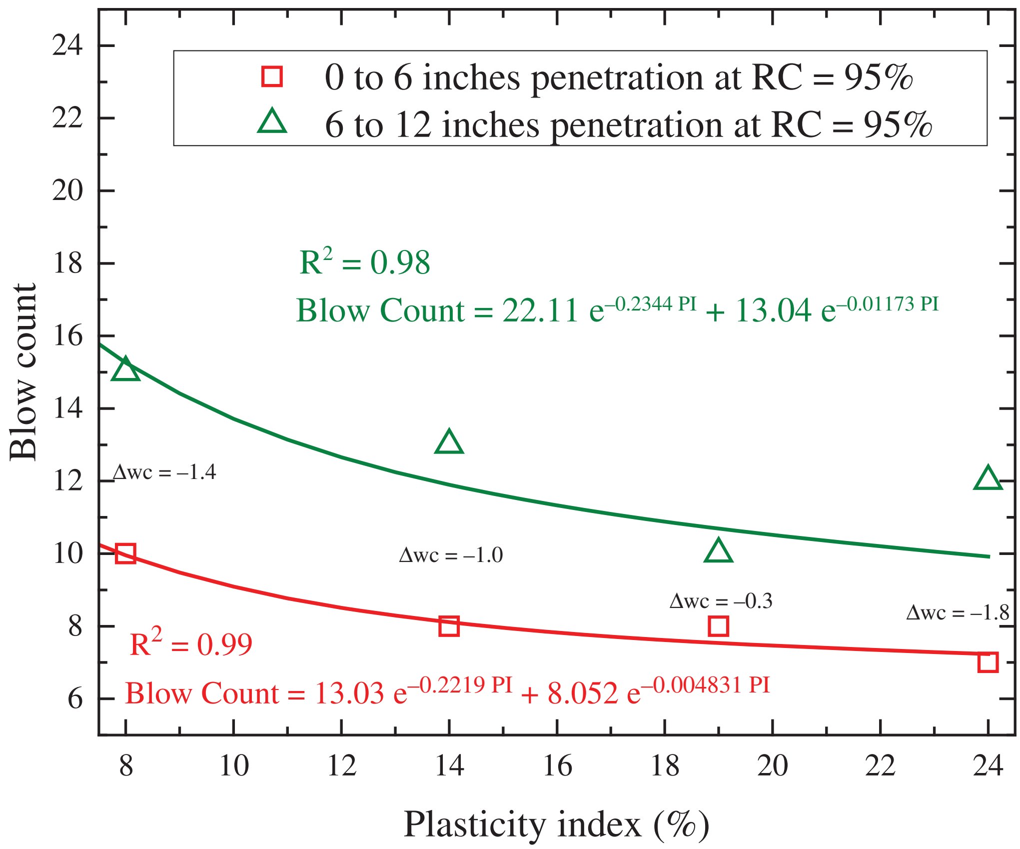

Figure 6.11 Blow count correlations for fine-grained soils.

TABLE 6.5
Index properties and corresponding blow counts for 0 to 6 inches of penetration for the soil used in the moisture sensitivity tests performed by INDOT

										DCP Blow Count at		
AASHTO	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**	**LL (%)**	**PL (%)**	**PI (%)**	**MMD (pcf)**	**OMC (%)**	**OMC −2**	**OMC +2**	**OMC**
A-6	6.2	7.3	57.2	29.3	36	22	14	110.2	17.3	9	3	5
A-7-6	15.5	9.5	35.8	39.2	57	23	34	99	21	6	3	5
A-6	4.9	8.8	53.8	32.6	40	21	19	104	18.5	7	3	6
A-4	0.3	3	74.2	22.5	39	29.4	9.6	98.4	21.9	9	4	7

in the field. Table 6.6 shows the DCP blow count correlations developed for coarse-grained (with separate correlations for natural and *manufactured soils*) and fine-grained soils (with separate correlations for 0- to 6-inch and 0- to 12-inch penetration). Also shown are the R^2 values of the correlations when fitted to field data and the range of applicability of the developed correlation in terms of the index properties of the soils.

6.4 Summary

To develop QA/QC correlations for the DCP, the soils tested were categorized into fine and coarse-grained soils on the basis on the dominant soil fabric. A criterion for classification of the soils into these two categories was established based on the compaction characteristics, plasticity index and fines content of the soils.

Soil behavior was found to depend strongly on the dominant soil particle size in the soil mass. An estimate of the percentage by weight of clay required for a soil to develop a floating fabric is needed in order to categorize soils properly. Simple volumetric calculations were performed to obtain the clay content above which soils would behave as fine-grained (clay-dominated) soils.

After proper soil classification, correlations for either coarse-grained or fine-grained soils were then used to obtain the DCP blow counts required for compaction quality control. It was observed that coarse-grained soils have a good correlation with the OMC obtained from the

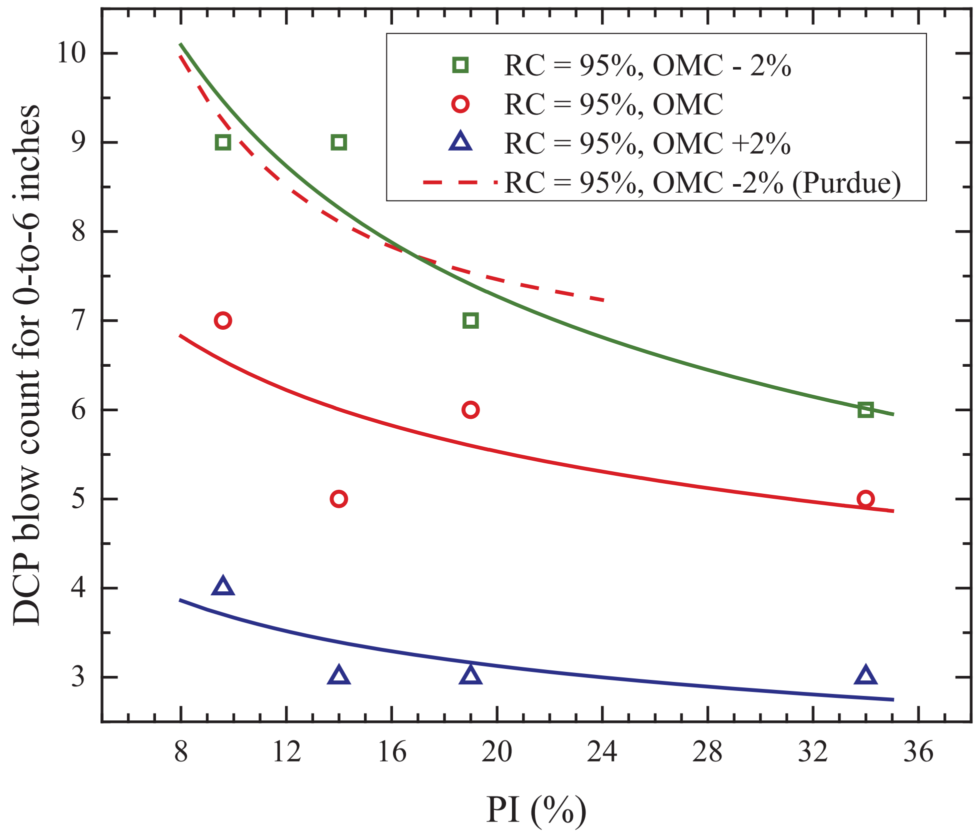

Figure 6.12 Moisture sensitivity of fine-grained soil compacted at RC = 95%.

TABLE 6.6
DCPT correlations

Soil Type		Correlations	R^2	Penetration Depth (inches)	Range of Applicability
Sand-dominated soils	Natural	Blow Count $= 0.17 \times OMC^2 - 5.94 \times OMC + 60$	0.95	0 to 12	8 < OMC% < 13
	Manufactured	Blow Count $= 4.03 \times \ln(C_u) + 2.64$	0.99	0 to 12	3.0 < Cu < 6.0
Clay-dominated soils		Blow Count $= 13.03 \times e^{-0.22 \times PI} + 8.05 \times e^{-0.005 \times PI}$	0.99	0 to 6	8 $\geq$ PI%
		Blow Count $= 22.11 \times e^{-0.23 \times PI} + 13.04 \times e^{-0.012 \times PI}$	0.98	6 to 12	

TABLE 6.7
Implementation plan for DCP correlations

Soil Type	Maximum Dry Density (pcf)	OMC (%)	PI (%)	DCP 0–12 Inches	DCP 0–6 Inches*	Suggested Compaction Moisture
Sand-dominated soils	≥130	8–10	NP	18–23	NA	[OMC −3%,OMC]
	120–130	10–12	NP	13–17	NA	[OMC −3%,OMC]
	115–120	12–14	NP	10–12	NA	[OMC −3%,OMC]
Clay-dominated soils	110–114	14–16	9–12	NA	9	[OMC −1%,OMC +2%]
	105–110	16–18	12–18	NA	8	[OMC −1%,OMC +2%]
	≤105	18–24	18–24	NA	7	OMC ±2%

* DCP blow counts correspond to *in situ* soil moisture at OMC −2%

standard Proctor compaction test performed in the field or in the laboratory. The DCP blow counts for penetration of 0 to 12 inches decreased when the OMC of the soils compacted to at least 95% RC increased. The demarcation between manufactured and natural coarse-grained soils was also made, and correlations were presented for manufactured coarse-grained soils with respect to both the OMC and *Cu*.

QA/QC correlations for fine-grained soils were developed as well. It was found that the DCP blow counts measured for the fine-grained soils with fabric dominated by clay-size particles showed a very good correlation with the plasticity index of the soil, which serves as a good indicator of the clay content of the soil mass for a given activity value. The *in situ* water content of the soil was within −2% of the OMC for all the data used in the development of the correlations for the fine-grained soils.

In addition to the tests performed by Purdue, moisture sensitivity tests were also performed by researchers at INDOT to assess the variation in DCP blow count with change in compaction moisture content. Laboratory tests done by INDOT revealed that for fine-grained soils, the DCP blow count varies significantly with moisture (3–5 blows for 1% moisture content change), especially for soils with PI in the range of 10% to 20%.

7. STATISTICAL ANALYSIS OF DCPT RESULTS

Knowledge of the DCP blow count criteria from the correlations is not enough for proper compaction quality control in the field. There are additional questions that need to be examined:

- How many tests (consisting of one test set) need to be performed at a given location to be reasonably sure that the average blow count obtained from the test results is representative of the soil compaction at that location (referred to as small-scale testing frequency)?
- At what separation distance must the test sets be performed along the length of the final constructed subgrade to properly assess compaction quality (referred to as large-scale testing frequency)?

To answer these questions, an understanding of the variability and distribution associated with the DCP test results are required.

7.1 *In Situ* DCP Test Results—Small-Scale Variability and Distribution

The DCP test results have a certain distribution associated with them (see blow count distributions at various sites shown in Appendices A and B). The data shows that at each location where a test set is performed, for which there is an associated RC value, multiple values of blow counts are obtained. This means that a singular DCP test performed at a given location will not necessarily give us the representative blow count of the compacted soil. Therefore, it becomes necessary to ascertain the number of tests needed to be performed at a given location to get a representative blow count at that location.

It was observed that individual test sets did not show a consistent distribution, and that many distributions could be fit to the field test data. This lack of fit to a specific distribution within a test set was expected in the sense that the individual test sets formed the samples of a population and, therefore, did not follow the same distribution as the population itself. To understand the distribution of the population, it was necessary to increase the sample size, *i.e.,* the number of DCP tests performed in a given location. This could be done by: (1) grouping test sets [joining together the DCP test results from test sets that had similar soil properties and *in situ* compaction conditions (RC and *in situ* water content)] and (2) performing grid testing [performing multiple DCP tests in one location (30 or more tests) to get a better sense of the distribution of the population].

These two approaches were carried out simultaneously and, by doing so, more clear patterns and trends started to become visible. The increased number of tests resulted in samples which were more indicative of the trends in the population of the DCP test results. Once the sample size was increased, it was necessary to follow a logical procedure to find out the actual distribution associated with the population. For this, distribution fitting had to be carried out on the DCP test data obtained from grouping of test sets and grid testing.

7.1.1 Distribution Fitting

Distribution fitting is defined as the procedure of selecting a statistical distribution that best fits to a data set generated from random testing or process. It allows us to analyze how the field test results vary and, therefore, helps us to deal with the risk and uncertainty involved in the test results.

The process of distribution fitting involves a number of steps. First, a number of trial distributions are fit to the data using the algorithms available in specialized software (e.g., *EasyFit*). After distributions have been fitted, it is necessary to determine how well the individual distributions fit the empirical test results. This can be achieved via the *goodness of fit* tests or by visually/graphically comparing the empirical test results and the theoretical (fitted) distributions. Based on the combination of the two (goodness of fit tests and visual comparison), the most valid model can be selected to describe the data.

7.1.1.1 Grouping of test sets. First, the data of the grouped test sets were analyzed. Soils were grouped according to their compaction characteristics and index properties and

TABLE 7.1
Soil group properties

Group		Test Set ID	RC (%)
Coarse-grained soils	A	DCP-C-A3-1-KO-1	98.0
		DCP-C-A3-1-KO-2	98.5
		DCP-C-A3-1-KO-6	98.5
	B	DCP-C-A3-1-KO-4	101.1
		DCP-C-A3-1-KO-5	102.5
		DCP-C-A3-1-KO-7	102.0
Fine-grained soils	C	DCP-F-A6-3-BL-1	95.0
		DCP-F-A6-3-BL-2	94.0
		DCP-F-A6-3-BL-3	96.0
		DCP-F-A6-3-BL-4	96.0
	D	DCP-F-A6-2-NV-1	90.0
		DCP-F-A6-2-NV-2	91.0
		DCP-F-A6-2-NV-3	89.0
		DCP-F-A6-2-NV-4	91.0

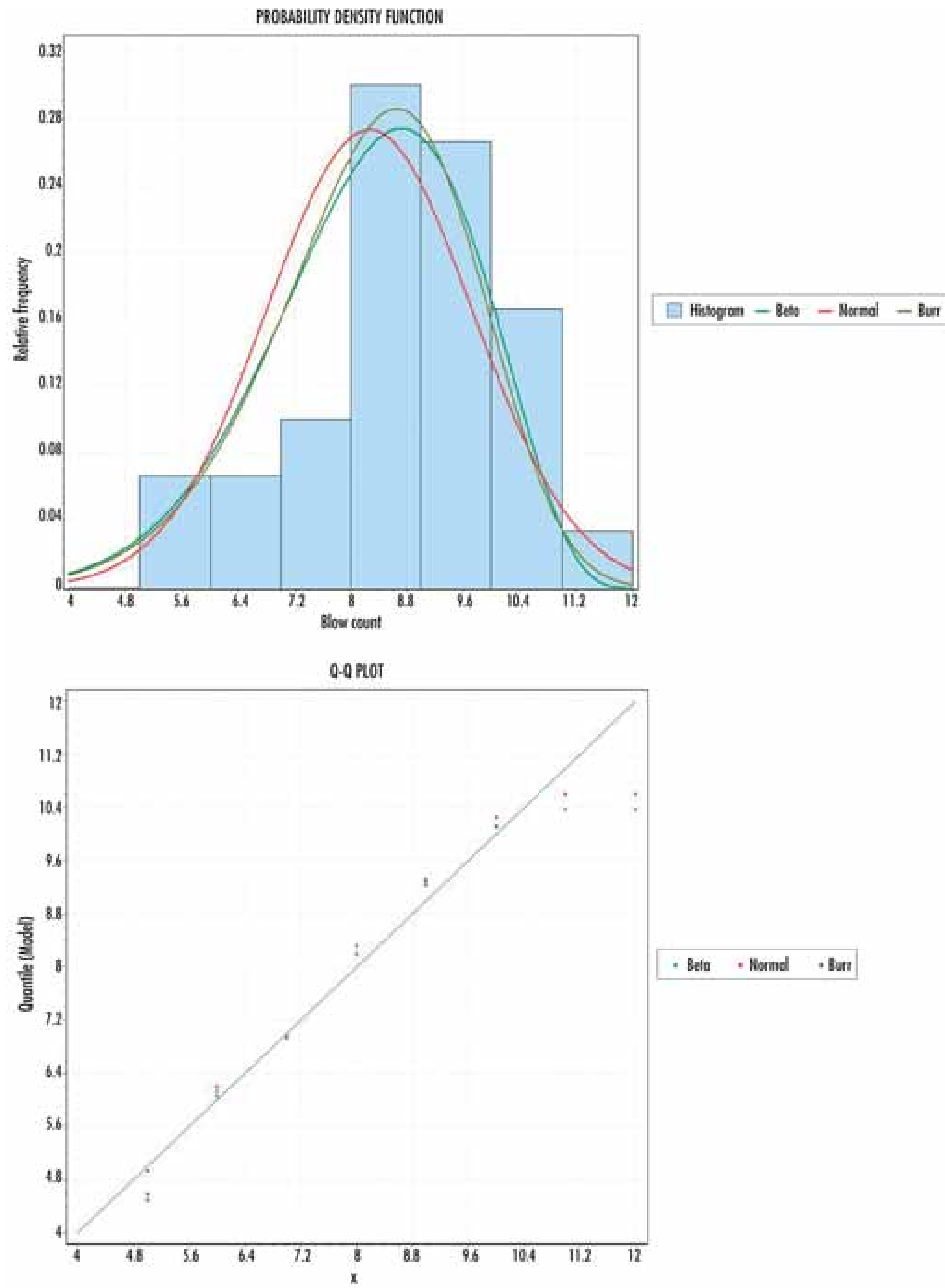

Figure 7.1 Group A: DCP-C-A3-KO-(1, 2, 6).

their *in situ* compaction condition. Table 7.1 shows the properties of the soils grouped for the purpose of the statistical analysis.

It was observed that the Normal, Beta and Burr distributions fit the test results reasonably well in most cases, both visually and in terms of the goodness of fit tests. Figure 7.1 and Figure 7.2 show the fitted distributions on grouped data for DCP tests performed in coarse-grained soils and the accompanying Quantile-Quantile (QQ) plots [a QQ plot is a plot of the quantiles of the empirical data and the fitted distribution; quantiles are points taken at regular intervals from the cumulative distribution function of a random variable (Devore, 2011)]. Figure 7.3 and Figure 7.4 show the fitted distributions on grouped data for DCP tests performed in fine-grained soils and the accompanying QQ plots. These figures show that while individually different distributions may fit the different data groups better, overall, the distributions were equally good with only minor differences in the ability to fit the data. This point is highlighted by the QQ plots.

In addition to the QQ plots, the Kolmogorov-Smirnov (KS) goodness of fit test was also performed on the distribution fittings, and it was found that the difference in the KS statistic for Normal and Beta distribution was not

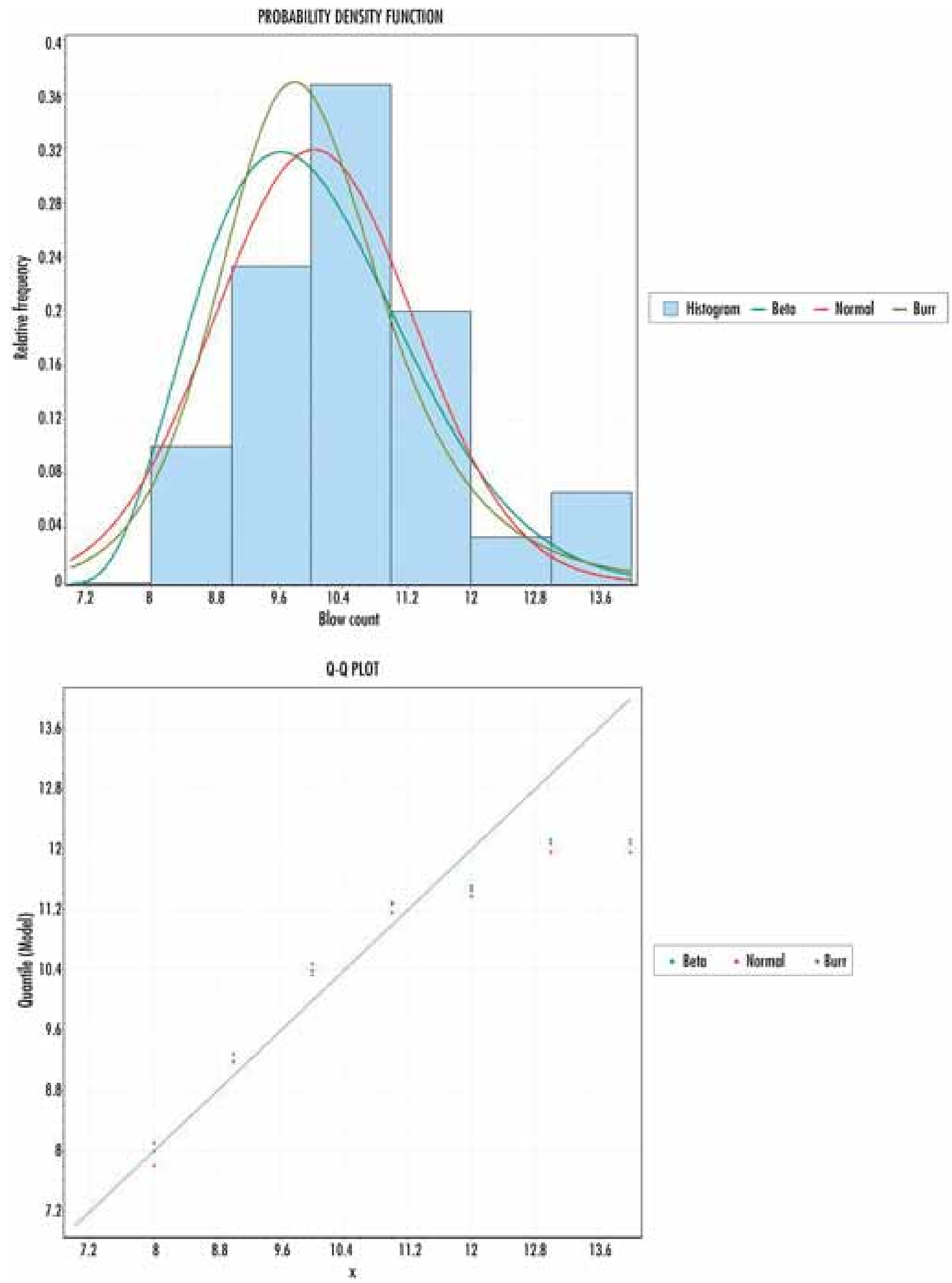

Figure 7.2 Group B: DCP-C-A3-1-KO-(4, 5, 7).

substantial, indicating that these different distributions fit the data equally well. Both the Normal and Beta distributions passed the KS goodness of fit test and were found to be suitable to describe the distribution of the data. Table 7.2 gives a comparison of the parameters of the Beta and Normal distributions and the KS statistics. The Beta distribution has the advantage of being versatile and thus fits the data slightly better (as can be seen from the KS statistics in Table 7.2). The normal distribution on the other hand has the advantage of having a fixed shape, which allows us to make certain predictions based on the average value of standard deviation of the DCP test data.

7.1.1.2 Grid testing. In addition to the grouping of the test sets, grid testing was also carried out to ascertain the distribution of the DCP blow counts at a given location. The objectives of the grid testing were to: (1) limit the area in which the DCP tests were performed and (2) increase the number of tests performed in a particular location. This ensured that the soils tested were in the same compaction condition and that the DCP test results were more representative of the population.

The grid testing was carried out in an area of 1.5 m by 1.5 m on compacted soil (soil type: DCP-F-A6-1). Within an area of 1.5 m by 1.5 m, 36 DCP tests were performed in a grid pattern (along 6 rows and 6 columns). Adjacent DCP

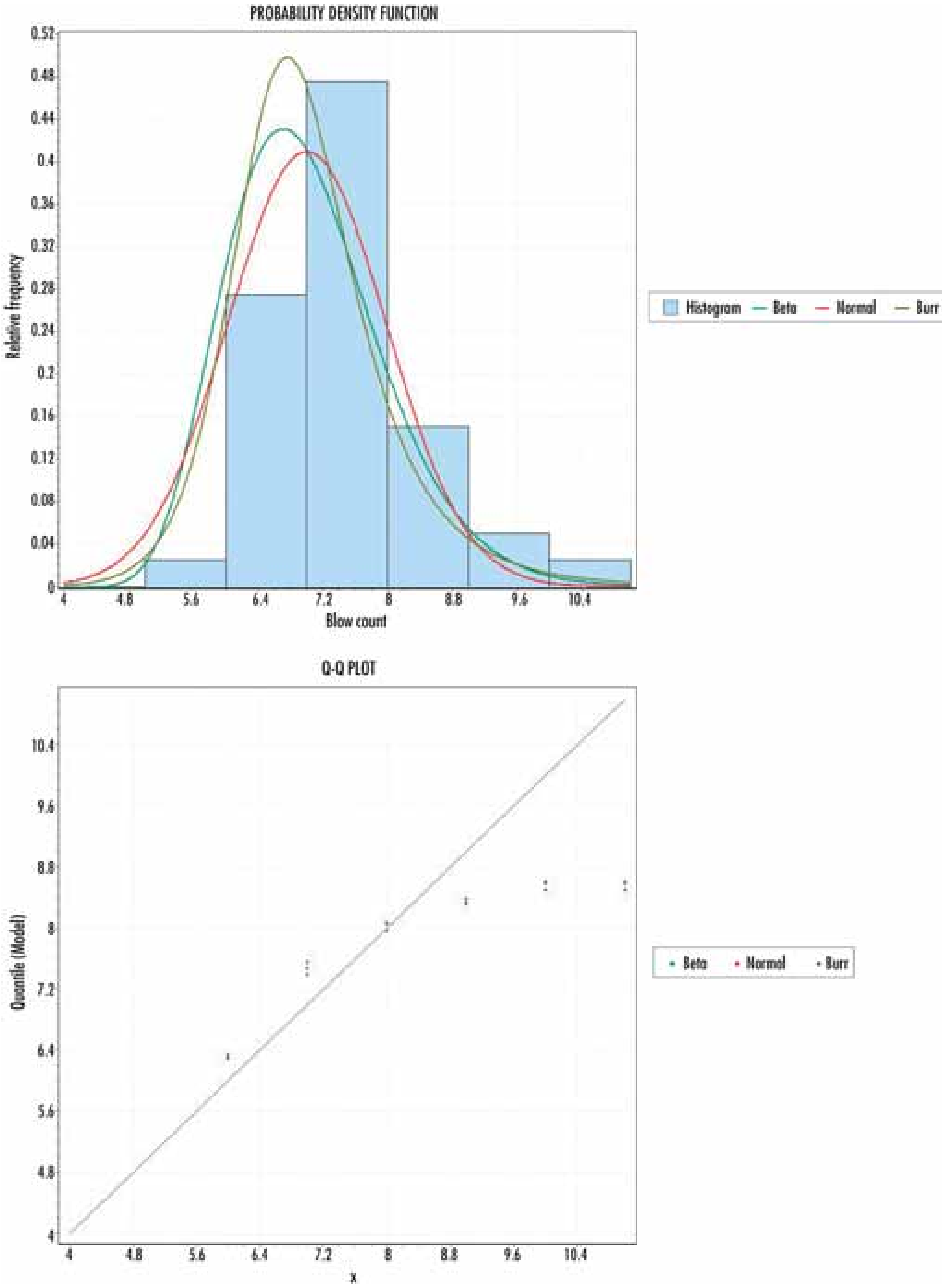

Figure 7.3 Group C: DCP-F-A6-3-BL-(1, 2, 3, 4).

tests were performed at a center-to-center spacing of at least 1 foot [$\approx$30 cm or approximately 15 DCP cone tip diameters (one cone diameter being 2 cm)]. This distance was chosen to reduce the probability of adjacent tests affecting each other. Figure 7.5 shows the schematic and the accompanying picture from the field of the grid testing.

To assess the *in situ* density of the compacted soil, four sand cone tests and multiple water content tests were also performed on the four edges of the grid, outside the testing area, just after the DCP tests had been performed. The RC values for all four locations were within range of 93%$\pm$1%, and the water content was within -2% of the OMC (= 15%).

The test results obtained from the DCP grid testing are shown in Figure 7.6; the numbers in the individual boxes represent the number of DCP blows required to penetrate 6 inches into the subgrade at a specific location. The DCP results obtained from the grid test performed in the field were analyzed, the frequency histogram was plotted and the most suitable frequency distribution was ascertained. Figure 7.7 shows the results from the distribution fitting on the DCP test results from the field grid testing and the QQ plots corresponding to Normal and Beta distributions. As can be seen from these plots, the Beta and Normal distribution fit the empirical DCP data well. QQ plots also

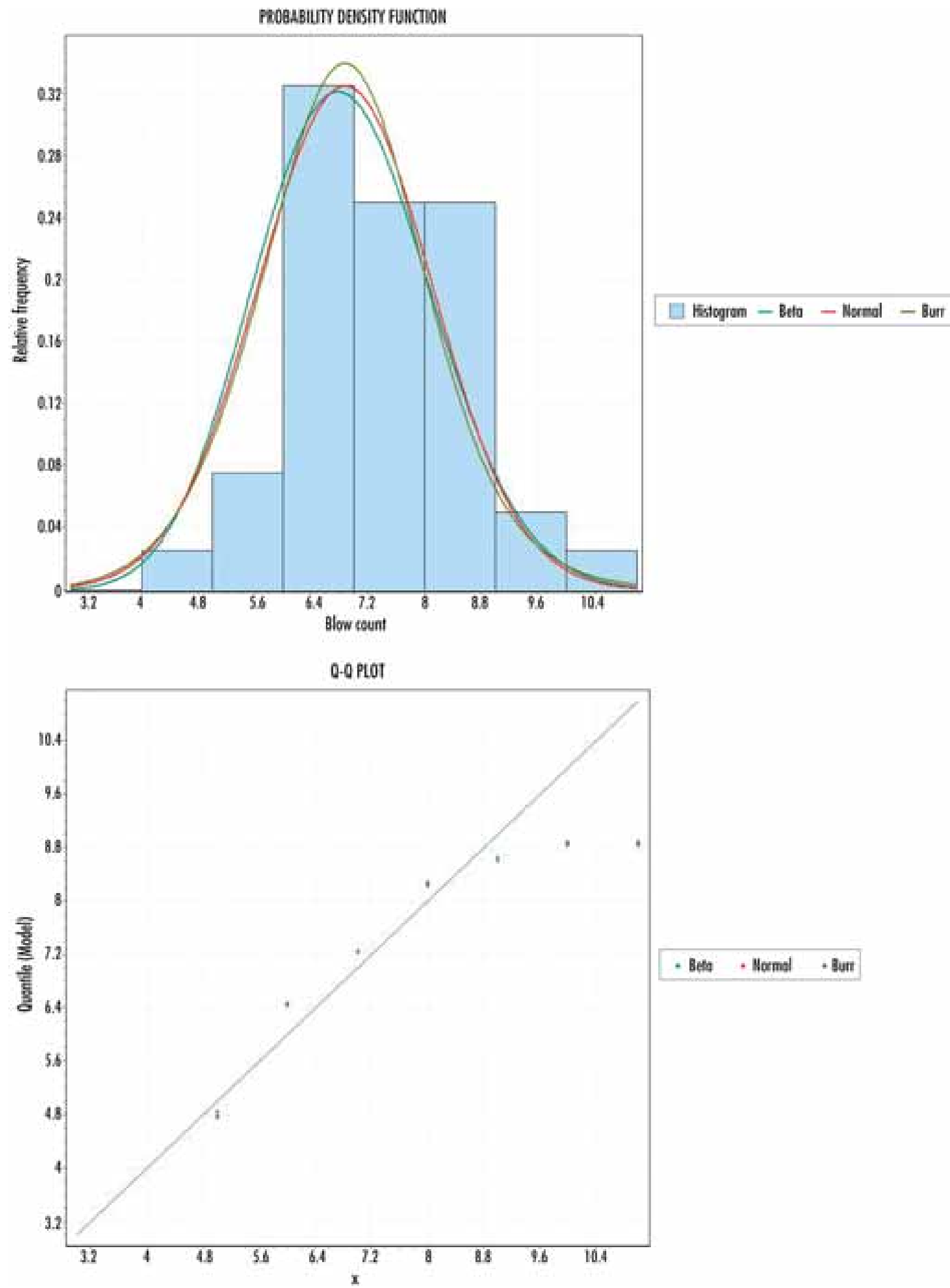

Figure 7.4 Group D: DCP-F-A6-2-NV-(1, 2, 3, 4).

TABLE 7.2
Comparison of parameters of Beta and Normal distributions

	Beta Distribution					Normal Distribution		
Test ID	**KS**	**a**	**B**	**L**	**U**	**KS**	**σ**	**μ**
DCP-C-A3-1-KO-(1,2,6)	0.1623	14.961	4.766	−3.45	12	0.1941	1.45	8.20
DCP-C-A3-1-KO-(4,5,7)	0.19418	4.1513	12.362	7	19	0.21063	1.25	10.03
DCP-F-A6-3-BL-(1,2,3,4)	0.24005	10.98	6.17E6	3.8	1.79E6	0.2750	0.97	7
DCP-F-A6-2-NV-(1,2,3,4)	0.18024	18.9	32.98	0.108	18.673	0.18683	1.23	6.88

KS: Kolmogorov-Smirnov statistic
a, b: shape parameters of Beta distribution
L, U: Lower and upper bound of Beta distribution
σ: standard deviation
μ: mean

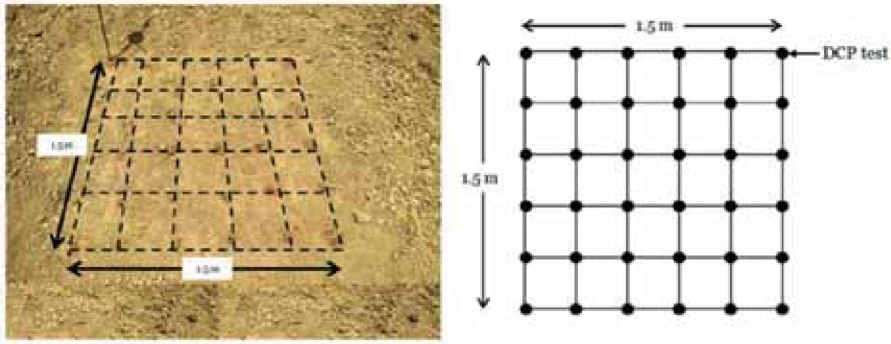

Figure 7.5 Grid testing in the field and schematic.

suggest a good fit for both of the distributions. Table 7.3 shows the distribution fitting parameters for the DCP grid testing.

The analysis of the data show that the Normal distribution is a good fit for the DCP blow count values obtained from tests performed at a given location. The grid testing allows for a better prediction of the distribution of the population. To gain an understanding of the distribution of the variance of the population, computer simulations were carried out. From the results of the grid testing, we can re-sample (with replacement) 100 tests and calculate the variance of the re-sampled sample. This process is repeated 1000 times to obtain the distribution of the sample variance and the standard deviation. This allows us to make calculations of confidence intervals, which can provide us with an answer to the question of how many tests need to be performed in a given location to obtain a representative blow count at that location.

For the simulations, the statistical software R was used. The blow count values obtained from the grid testing were used to carry out the simulation because these tests were performed in a small area and, therefore, had the highest probability of being representative of the distribution of the DCP test results. Table 7.4 gives the results of the

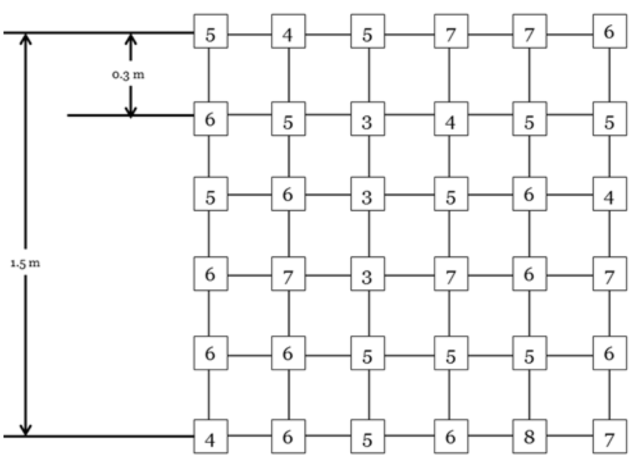

Figure 7.6 DCP blow count for 6-inch penetration at specific locations on the grid.

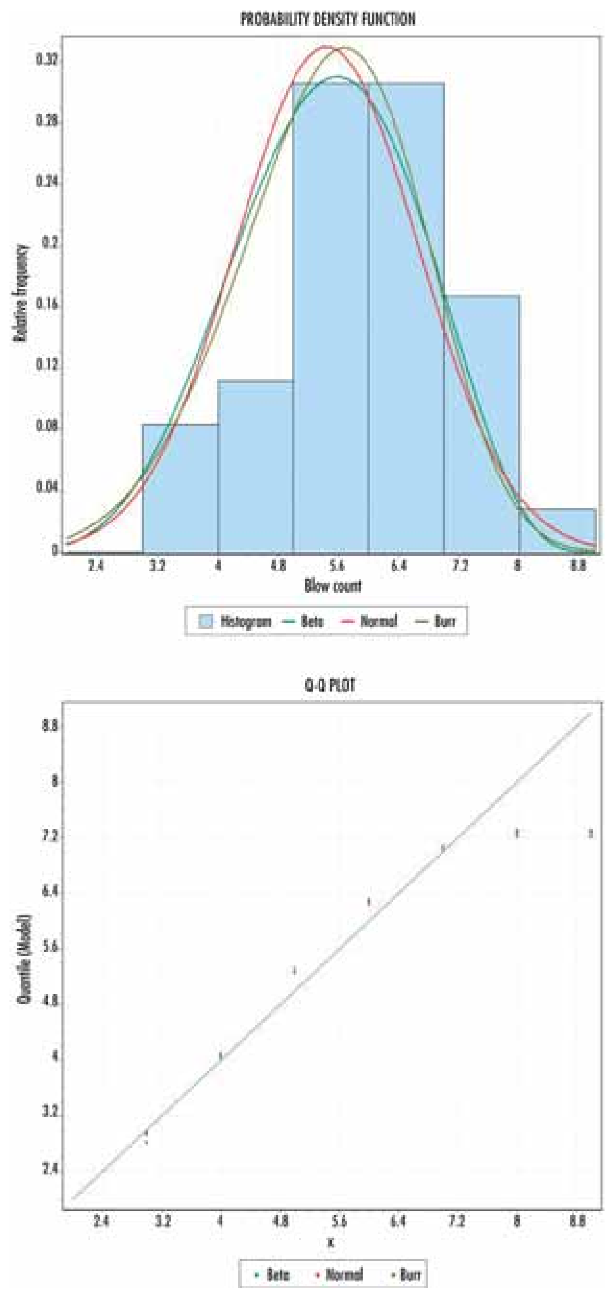

Figure 7.7 Probability density function and QQ plot for grid test results.

TABLE 7.3
Distribution fitting parameters for the field grid testing

	Beta Distribution					Normal Distribution		
Test ID	**KS**	**a**	**B**	**L**	**U**	**KS**	**σ**	**μ**
Grid	0.16	6.08	4.54	0.67	9.0	0.19	1.2	5.4

TABLE 7.4
Simulation results – percentile and corresponding variances

	Percentile (%)			
	25%	**50%**	**90%**	**98%**
Variance	1.2	1.5	1.7	1.9
Standard deviation	1.1	1.2	1.3	1.4

simulations performed. It presents various percentiles of DCP blow counts and the corresponding values of standard deviation and variance obtained from the simulations. From Table 7.4, it can be seen that to be reasonably conservative, we can take the standard deviation corresponding to the 90 percentile for our calculations.

7.2 Small-Scale Testing Frequency

The distribution of the population of DCP blow counts in compacted fine-grained soils can be reasonably assumed to be normally distributed. It was also seen from the re-sampling simulations that the population standard deviation, with the assumption of a normal distribution, can be taken as 1.3 DCP blow counts. Based on these observations, a better judgment can be made about the variability of the DCP test results and the number of tests that need to be performed at a given location to get a representative blow count for that location.

To solve this problem, consider a normally distributed population for the DCP test results. If a sample of *n* tests is performed on a compacted soil, the DCP blow count at that location will have a certain distribution and a mean. Now, if a number *n* of sets of tests are performed, assuming that the population distribution of DCP tests is normal, then it can be reasonably assumed that the mean of all those *n* tests will be normally distributed around the actual mean of the soil with a standard deviation equal to σ/n, where σ represents the actual mean of the DCP blow count of the population (Devore, 2012). Therefore, assuming that the population is normally distributed and knowing (1) the standard deviation of the population and (2) the number of tests performed, a confidence interval can be developed.

Our interest though is the question of how many DCP tests should be performed in one location to be reasonably confident that the mean of the sample tested is near the actual mean of the population, *i.e.*, a narrow confidence interval. This can be achieved by restricting the length of the confidence interval and the degree of confidence of the confidence interval developed. The length of the confidence interval *CI* is given by:

$$CI\ length = 2 \times z_{\alpha/2} \times \frac{\sigma}{\sqrt{n}} \qquad (7.1)$$

where $z_{\alpha/2}$ represents the confidence level (z-statistic), σ is the standard deviation of the population and n is the number of tests performed. Depending on the confidence level we wish to achieve, $z\alpha/2$ can be equal to 1.28 for 80% *CI*, 1.645 for 90% *CI* and 1.96 for 95% *CI*. The σ is assumed to be 1.3, based on the results of the simulations performed earlier.

Using equation (7.1), the number *n* of tests required to be confident of the fact that 95% of the time the mean of the population will be within ± 1 DCP blow count (a confidence interval of 2) of the mean of the sample can be obtained. Assuming a standard deviation of 1.3 and a confidence interval length of 2, from equation (7.1) the sample size is equal to 7. Therefore, the mean of 7 DCP tests performed within a reasonably small area (area of approximately 1.5 m by 1.5 m) should give us a good estimate of the mean of the population.

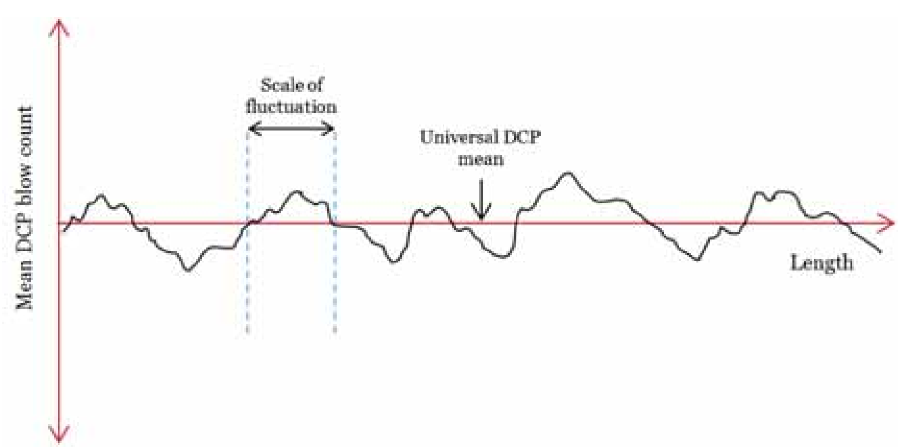

Figure 7.8 Scale of fluctuation.

7.3 Large-Scale Testing Frequency

Small-scale testing can only give us an estimate of the required local DCP blow count. To ascertain the compaction quality across the length of the compacted embankment or subgrade, we need to perform DCP tests along one direction. For this purpose, testing protocols need to be established that are based on a rational treatment of the problem.

The spatial variation of the properties of soil in the field can be modeled like a random field (Griffiths & Fenton, 1997). The DCP blow counts and the RC values for the soil are the random variables, which have a certain distribution in space.

From the small-scale testing presented in the previous sections, we have been able to ascertain the distribution of the DCP test results in a given location (locally). Along with local variations, we also have spatial variation of the DCP blow counts.

The objective of the large-scale testing is to find out the variation in the DCP blow count values with reasonable resolution such that field engineers can decide how often to perform DCP tests to assess compaction quality at the jobsite along the length of the compacted structure. For this, an understanding of the scale of fluctuation of DCP test results for a compacted soil is needed.

The scale of fluctuation can be considered as the length over which the properties of interest are significantly correlated. Consider, as shown in Figure 7.8, the variation of the mean of the DCP blow counts across the compacted soil. The length over which the compacted soil has a good correlation between the RC values can be considered as the scale of fluctuation. Now, to properly characterize the variation, it is necessary to have multiple tests performed within a scale of fluctuation. Which leads to the question, what is the scale of fluctuation of DCP test results for compacted soils?

The literature suggests that the scale of fluctuation of natural deposits of sandy and clayey soils for the CPT and SPT tests ranges from 10 meters to 80 meters (Phoon & Kulhawy, 1999). Therefore, conservatively the separation distance between consecutive test sets should be within the scale of fluctuation of 10–80 m to properly characterize the variability in natural soil. It should be noted here that the values of scale of fluctuation quoted from literature are for natural deposits. It is expected that the scale of fluctuation of compacted soils be somewhat larger than that of natural deposits, as the entire objective of the process of compaction is to make the soil homogeneous.

Another point to be noted here is that DCP blow counts vary significantly even within a small area of testing, as was seen in the grid testing described in the previous sections. Therefore, to assess the large-scale variation in DCP blow counts along the length of compaction, we need to take the average of seven tests (in accordance with the results of the small-scale variability of DCP blow counts) in a given location as the local representative blow count value. Using data collected in this manner, then we can assess the variation in the average blow counts over longitudinal distances along the length of compaction.

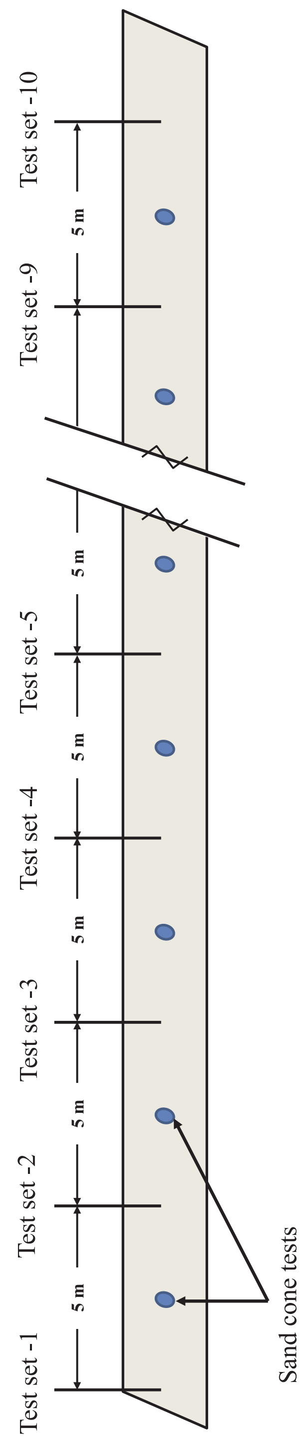

Figure 7.9 Spatial arrangement of 10 DCP test sets and sand cone test sets.

TABLE 7.5
DCP test results from variability study

Test Set	Average DCP Blow Count (0- to 12-inch penetration)	RC (%) Before[1]	RC (%) After[2]	Δwc
1	11	NA	96	−0.5
2	13	96	97	−0.6
3	13	97	95	−0.5
4	12	95	93	−0.2
5	12	93	95	−0.5
6	11	95	96	−0.5
7	11	96	93	−0.1
8	11	93	93	−0.1
9	12	93	96	−0.0
10	12	96	NA	−0.5

[1] Before = RC value is reported for SC test done between current and preceding test set

[2] After = RC value is reported for SC test done between current and proceeding test set

To assess the variability in the average DCP blow counts and to obtain a value for the scale of fluctuation for the compacted soils, DCP testing was carried out at an INDOT jobsite in Lebanon, IN. The soil type there is DCP-C-A4-2 (silt-dominated soil). Consecutive test sets (comprising of seven DCP tests each) were carried out at a separation of 5 meters. Ten of these test sets were carried out over a length of 50 meters of compacted subgrade. In addition to these DCP test sets, sand cone tests were also carried out between the consecutive tests sets (at a separation of 5 m). Moisture measurements were also taken at the location of each test set. Figure 7.9 shows the spatial arrangement of the DCP test sets and sand cone test sets performed in the field. Table 7.5 shows the results obtained from the field tests.

The test results indicate that the scale of fluctuation is about 15 m according to Vanmarckes (1977) simplified method of estimation of scale of fluctuation. Figure 7.10 shows the variation of DCP blow count along the compacted length, according to the testing sequence shown in Figure 7.9.

As it can be seen from the data presented in Table 7.5, even within the tested length of 50 m, not much variation is observed in the DCP blow count. To better quantify the scale of fluctuation, and to assess its variation with change in compaction practices, further field testing and more refined analysis will definitely be required.

Based on the obtained results, the following preliminary procedure is put forward for quality control of compacted soil:

1. Perform one test set, comprised of seven DCP tests, on one corner of the compacted subgrade;
2. Perform a second test set (comprised of seven tests) at a distance of 15 meters (distance A) from the first test set along the length of the compacted subgrade
3. Perform the next test set at distance of 15 meters (distance A) from the previous test set if the difference between the means of the previous two test sets is less than or equal to 1 DCP blow count, otherwise perform the test set at distance of 10 meters from the previous test set (distance B).
4. Repeat step 3 until the end of the length of the compacted structure.

In case of manufactured sands, the distances A and B can be reduced, in accordance with the recommendations of the site engineer, due to the higher variability associated with their compaction.

If the moisture of the compacted soils is on the wet-side of the OMC, a drop in the blow counts values will be observed and, on the other hand, if the moisture of the compacted soils is on the dry side of the OMC, an increase in the blow count will be observed. The moisture content of the soil at the time of compaction gives an indication of the expected blow counts. Therefore, moisture content measurements need to be taken with greater frequency. We have already seen in Figure 6.12 the variation in the blow counts for different compaction moisture contents.

The DCP blow count is significantly affected by the moisture conditions of the soil at the time of compac-

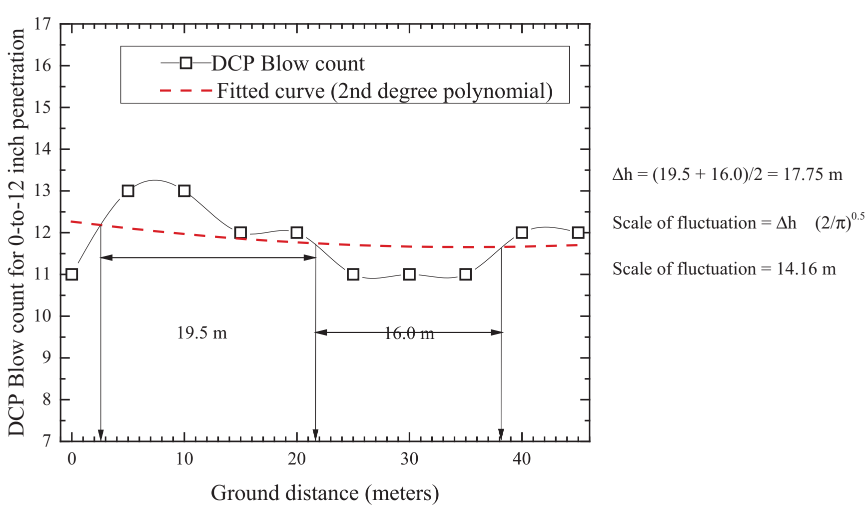

Figure 7.10 Vanmarcke (1977) estimation of scale of fluctuation.

tion. During compacting, it should be ensured that the moisture content of the soil is within the specification of -2 to $+1\%$ of the OMC. From Table 7.5, we can see that the specified compaction density is adequately achieved if the moisture content is within the specifications set by INDOT.

To have a better control over compaction quality, it is suggested, and agreed to by the INDOT engineers, that the number of moisture content measurements performed per day for each compaction lift be increased from the current prescribed specification of one moisture measurement per day to a minimum of one moisture measurement at every 4 hours.

In reference to field testing for small projects (less than 2000 cubic yards), it is recommended by INDOT engineers that three tests be performed at any given location to obtain an estimate of the representative blow count of the *in situ* compacted soil. The average of the three should be equal to or greater than the recommended blow count obtained from the criteria outlined in the previous chapter. Also, no individual test should have a blow count that is one or more blow counts below that obtained from the criteria recommended in this report.

When the moisture content is 1% above the OMC of the soil being tested, an average blow count value of one less than the value obtained from the criteria recommended may be considered acceptable. Also, no individual test in this case should have a blow count that is two or more blow counts below that obtained from the criteria recommended in this report.

7.4 Summary

Based on the statistical analysis of the DCP test data, it could be reasonably stated that the mean of seven tests per location can provide a reasonable estimate of the actual mean of the compacted soil mass, assuming that the DCP test results follow a normal distribution. In addition to local testing, large-scale testing was also investigated, and a preliminary procedure for large-scale testing was suggested:

1. Perform one test set, comprised of seven DCP tests, on one corner of the compacted subgrade;
2. Perform a second test set (comprised of seven tests) at a distance of 15 meters (distance A) from the first test set along the length of the compacted subgrade
3. Perform the next test set at distance of 15 meters (distance A) from the previous test set if the difference between the means of the previous two test sets is less than or equal to 1 DCP blow count, otherwise perform the test set at distance of 10 meters from the previous test set (distance B).
4. Repeat step 3 until the end of the length of the compacted structure.

Further work still needs to be carried out in the direction of establishment of the scale of fluctuation of compacted soils to further refine the large-scale testing procedure proposed.

8. SUMMARY AND CONCLUSIONS

The DCP is a simple tool for assessing the quality of the compaction at a jobsite. Not only it is light and easy to transport and perform, its use is also not limited by the availability of power, and once proper correlations are established, the QA/QC procedures are fairly easy to administer.

Quality control and assurance tests for compacted soils have in the past been limited to density tests, which have certain shortcomings associated with their application. Performance-based tests, such as the DCP, among others, are used to ensure that compaction quality is achieved and that the compacted soil satisfy a certain minimum level of performance. Correlations have been developed between the DCP blow count and quality control parameters, such as CBR, resilient modulus and relative compaction.

The DCP has been steadily gaining popularity as a QA/QC device that can be used in tandem with other means of quality control to ensure the construction of a well-built subgrade or embankment. However, the statistical variability associated with the test results needs to be accounted for in the process of development of QA/QC correlations.

In this research study, in order to develop suitable QA/QC correlations for the DCP, soils were classified into two categories (coarse-grained soils and fine-grained soils). In general, for clay contents above about 20%, the behavior is similar to that of clay. To account for all the factors that control the mechanical behavior of soil and its response to DCP loading, the decision as to which category a given soil belongs – sand-dominated soil or clay-dominated soil – was based on the compaction characteristics, plasticity index and fines content. The developed soil classification criteria, although practical and simple, still needs further research, especially for silt-dominated soils.

It was observed that the DCP blow counts for coarse-grained soils had a good correlation with the OMC obtained from standard Proctor compaction tests. The blow counts for 0 to 12 inches of penetration decreased when the OMC of the soils compacted to at least 95% increased. Correlations were developed for manufactured coarse-grained soils with respect to the OMC and the coefficient of uniformity.

It was further found that the DCP blow counts for fine-grained soils, with their fabric dominated by clay-size particles, had a very good correlation with the plasticity index of the soil. The PI of soils is indicative of the clay content, depending on the clay mineral or proportions of the clay minerals in the clay phase of the soil mass.

The soil categorization criteria and the accompanying DCP correlations for quality control of compaction were found to be suitable to establish QA/QC procedures for compacted soil and received positive feedback from INDOT engineers. Table 6.7 provides the blow counts suggested by the criteria developed in this research for the different soil classifications.

Based on the statistical analysis of the DCP test data, it could be reasonably stated that the mean of seven tests per location can provide a reasonable estimate of the actual mean of the compacted soil mass, assuming that the DCP test results follow a normal distribution. In addition to local testing, large-scale testing was also investigated, and a preliminary procedure for large-scale testing was proposed. Further work still needs to be carried out in the direction of establishment of the scale of fluctuation of compacted soils to further refine the large-scale testing procedure proposed.

Given that current INDOT procedures call for very limited number of DCP tests to be performed, it is suggested that at least for small projects, a minimum of three DCP tests be performed for every 2,000 cubic yards of compacted soils. The average of the three DCPTs performed at each location should not be smaller than the DCP blow count obtained from the criteria proposed in this research minus one blow count (note that this is applicable to DCP tests performed at the OMC).

Finally, to have a better control over compaction quality, it is suggested that the number of moisture content measurements performed per day for each compaction lift be increased from the current prescribed specification of one moisture measurement per day to a minimum of one moisture content measurement at every four hours. The better characterization of soil moisture content will likely result in better quality control of the compacted subgrade soil. In addition, it is also recommended that the compaction moisture of fine-grained soils, whose stiffness is significantly affected by moisture content, be limited to a maximum of +1% above the OMC. The lower bound of OMC −2% can be maintained as is. This moisture content limit is set to ensure that the minimum DCP blow count at any time is at least seven DCP blows for 0 to 6 inches of penetration.

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APPENDICES

APPENDIX A: FINE-GRAINED SOIL HISTOGRAMS

DCPT histograms for test sets performed on fine-grained soils are presented in the same order as in Table 5.1. For each test set, the histogram for 0- to 6-inch penetration is presented first followed by the histogram for 6- to 12-inch penetration.

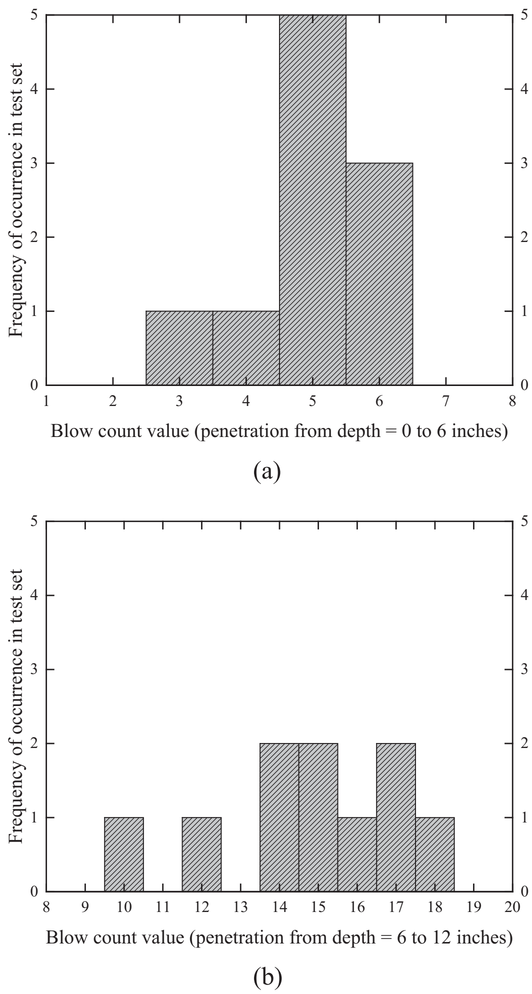


Figure A.1 Frequency vs. DCP blow count for DCP-F-A4-1-KO-1: (a) 0 to 6 inches and (b) 6 to 12 inches.

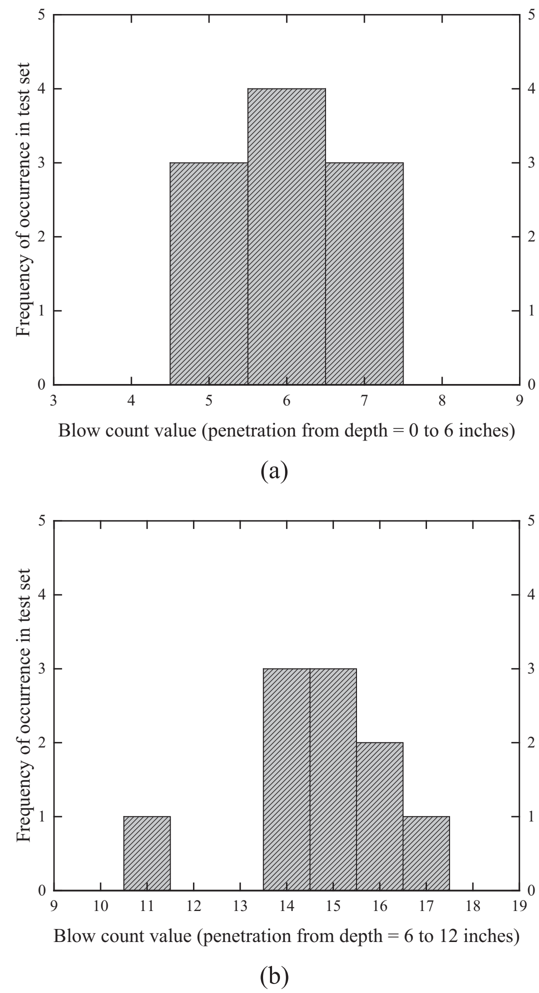
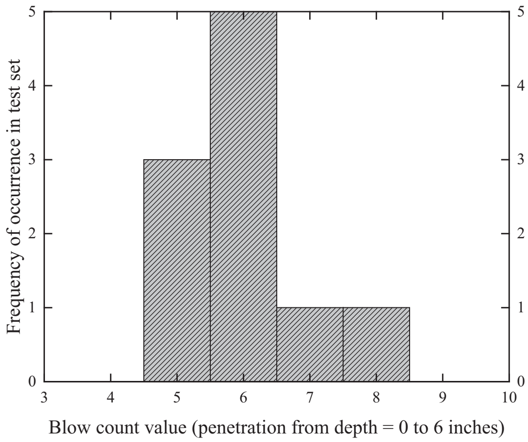
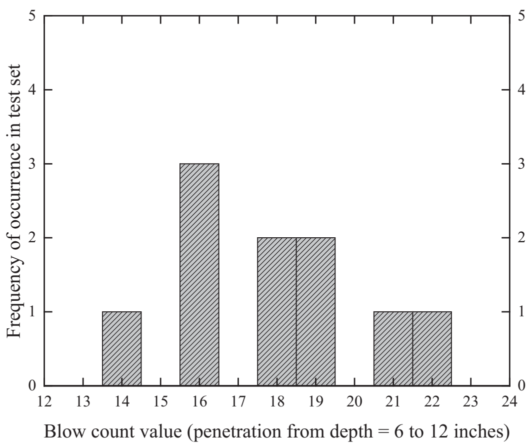


Figure A.2 Frequency vs. DCP blow count for DCP-F-A4-1-KO-2: (a) 0 to 6 inches and (b) 6 to 12 inches.

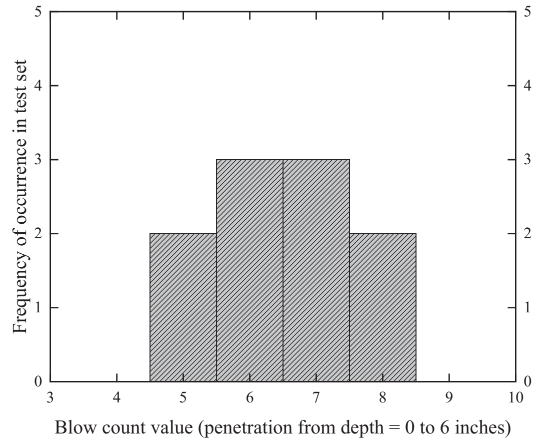


(a)

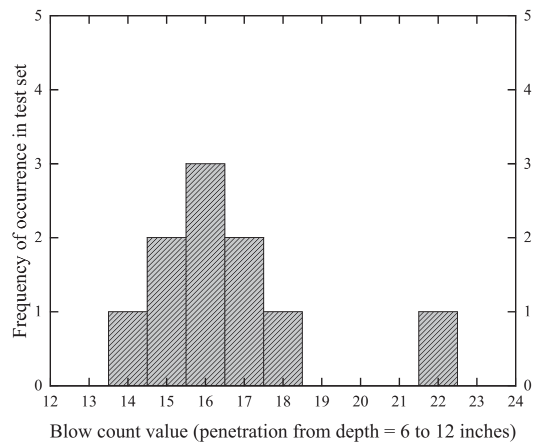


(b)

Figure A.3 Frequency vs. DCP blow count for DCP-F-A4-1-KO-3: (a) 0 to 6 inches and (b) 6 to 12 inches.



(a)



(b)

Figure A.4 Frequency vs. DCP blow count for DCP-F-A4-1-KO-4: (a) 0 to 6 inches and (b) 6 to 12 inches.

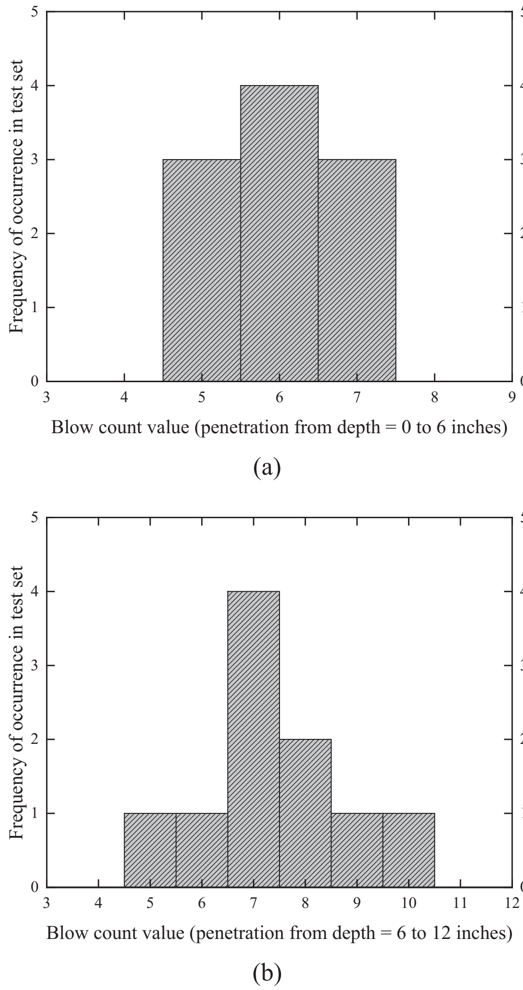


Figure A.5 Frequency vs. DCP blow count for DCP-F-A4-2-KO-1: (a) 0 to 6 inches and (b) 6 to 12 inches.

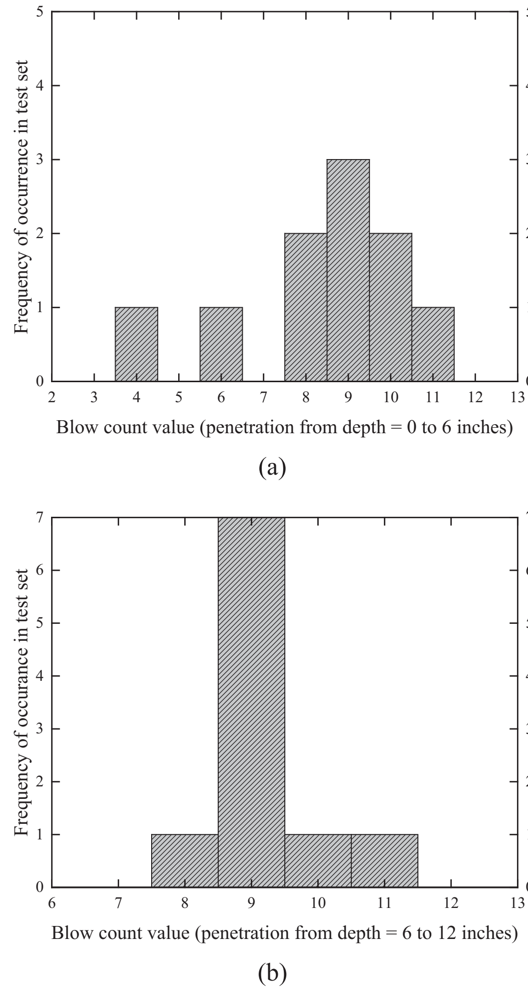


Figure A.6 Frequency vs. DCP blow count for DCP-F-A4-2-KO-2: (a) 0 to 6 inches and (b) 6 to 12 inches.

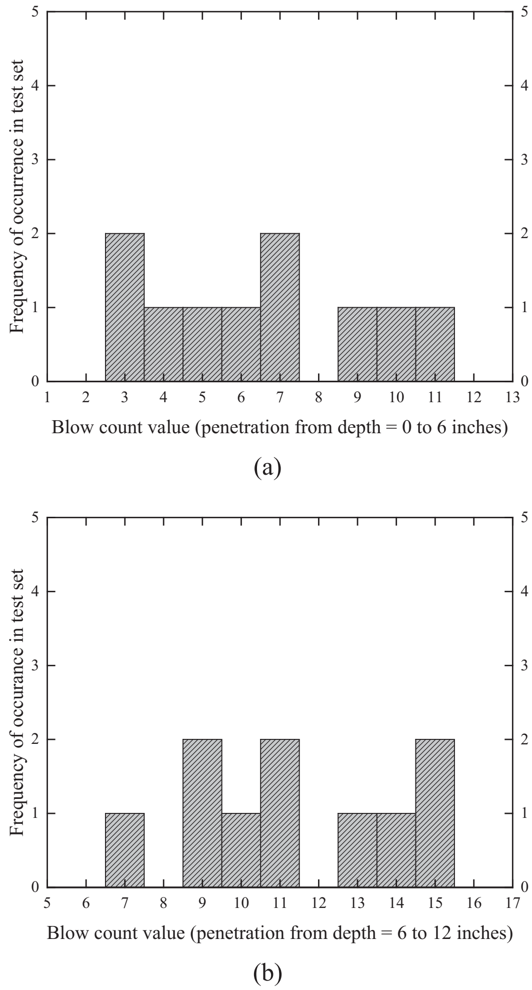


Figure A.7 Frequency vs. DCP blow count for DCP-F-A4-2-KO-3: (a) 0 to 6 inches and (b) 6 to 12 inches.

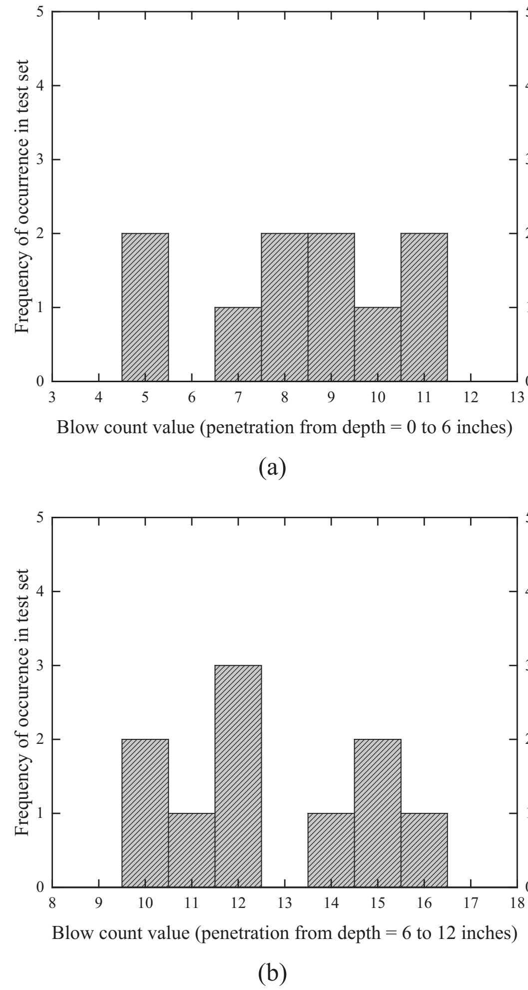
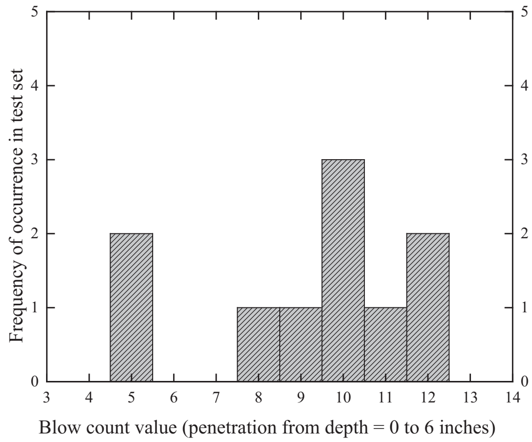
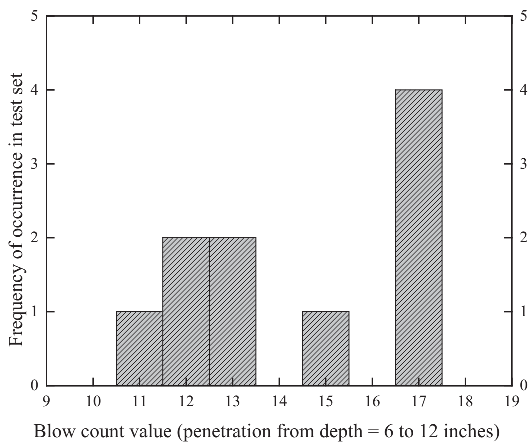


Figure A.8 Frequency vs. DCP blow count for DCP-F-A4-2-KO-4: (a) 0 to 6 inches and (b) 6 to 12 inches.

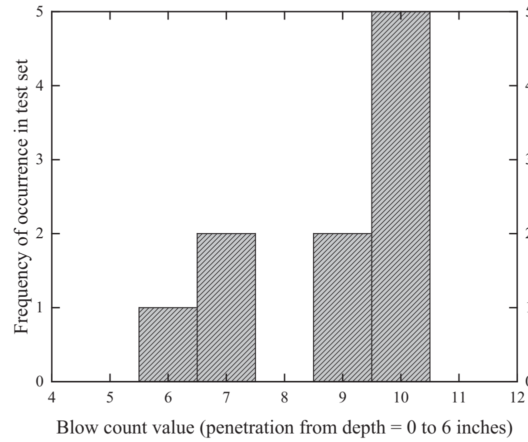


(a)

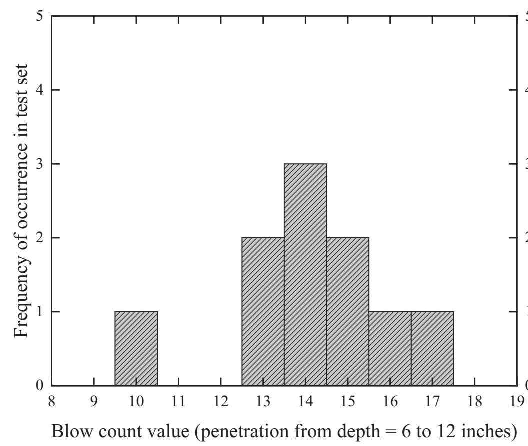


(b)

Figure A.9 Frequency vs. DCP blow count for DCP-F-A4-2-KO-5: (a) 0 to 6 inches and (b) 6 to 12 inches.

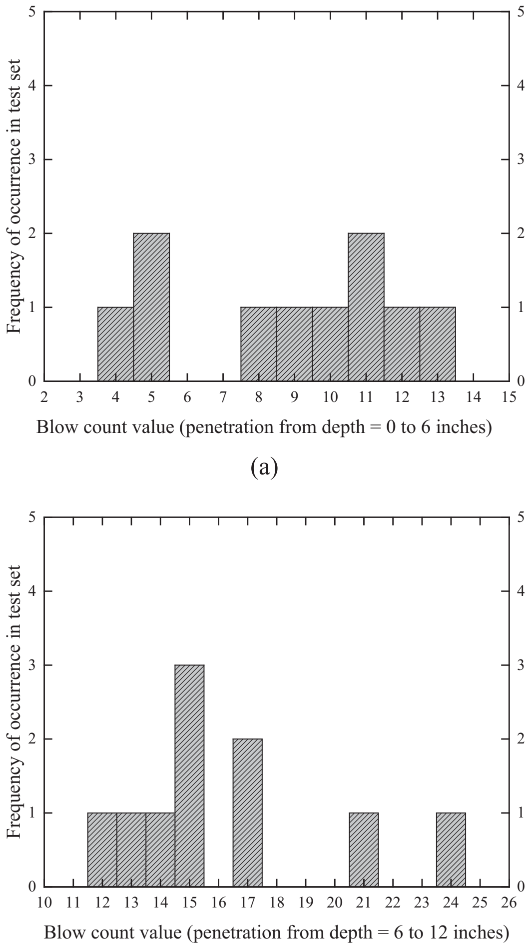


(a)



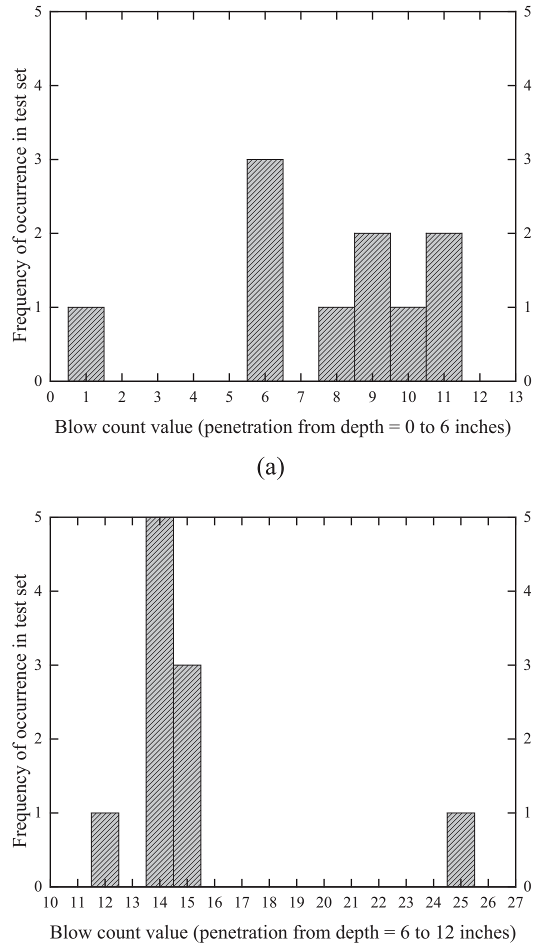
(b)

Figure A.10 Frequency vs. DCP blow count for DCP-F-A4-2-KO-6: (a) 0 to 6 inches and (b) 6 to 12 inches.



(b)

Figure A.11 Frequency vs. DCP blow count for DCP-F-A4-2-KO-7: (a) 0 to 6 inches and (b) 6 to 12 inches.



(b)

Figure A.12 Frequency vs. DCP blow count for DCP-F-A4-2-KO-8: (a) 0 to 6 inches and (b) 6 to 12 inches.

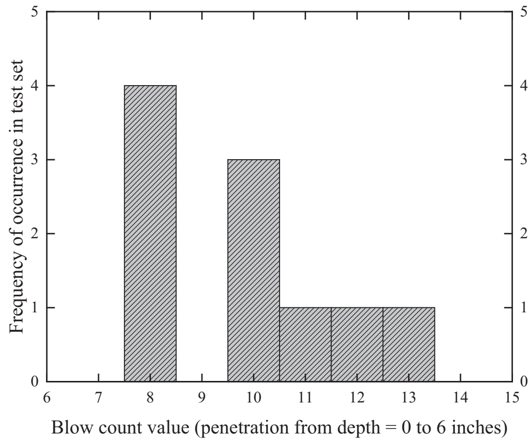


Figure A.13 Frequency vs. DCP blow count for DCP-F-A6-1-KO-1: 0 to 6 inches.

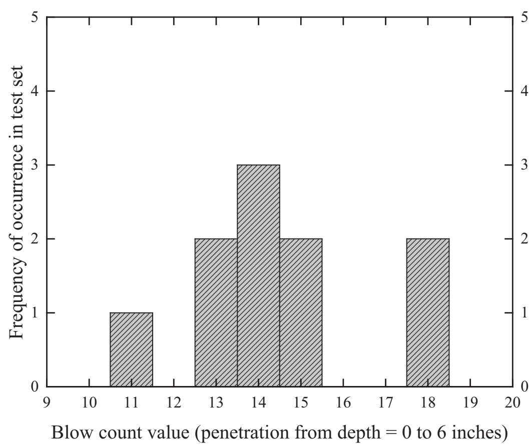


Figure A.14 Frequency vs. DCP blow count for DCP-F-A6-1-KO-2: 0 to 6 inches.

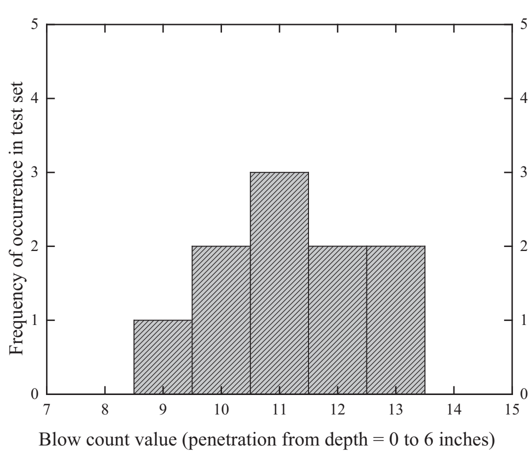


Figure A.15 Frequency vs. DCP blow count for DCP-F-A6-1-KO-3: 0 to 6 inches.

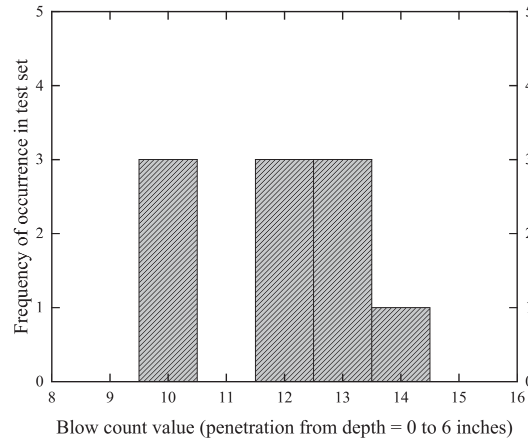


Figure A.16 Frequency vs. DCP blow count for DCP-F-A6-1-KO-4: 0 to 6 inches.

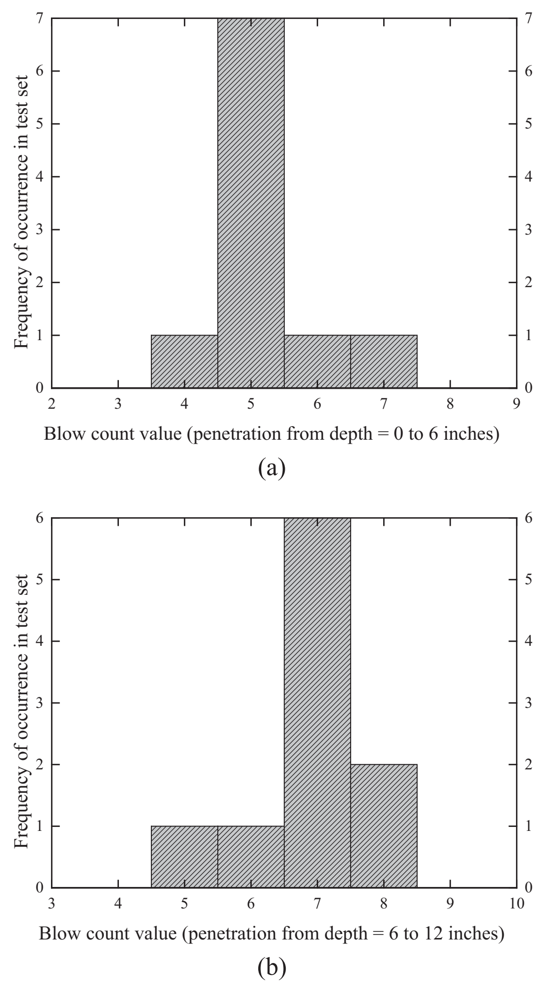
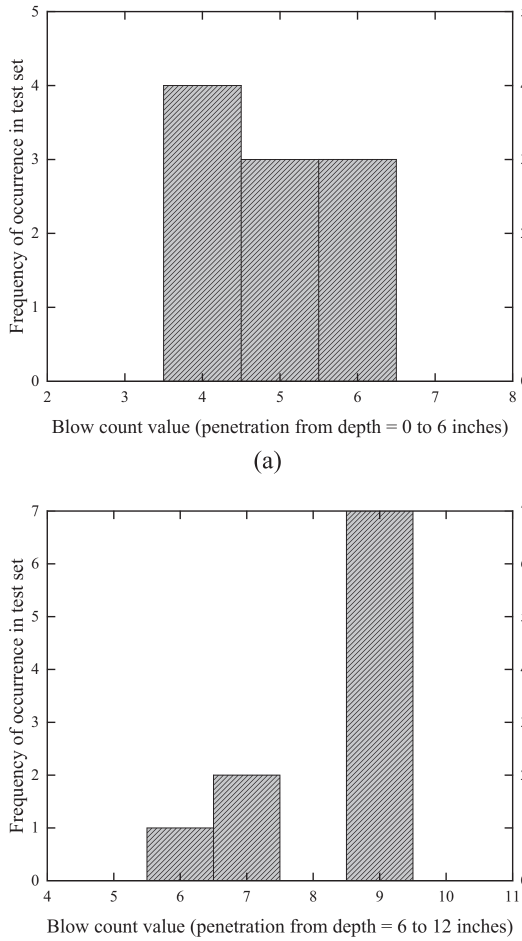
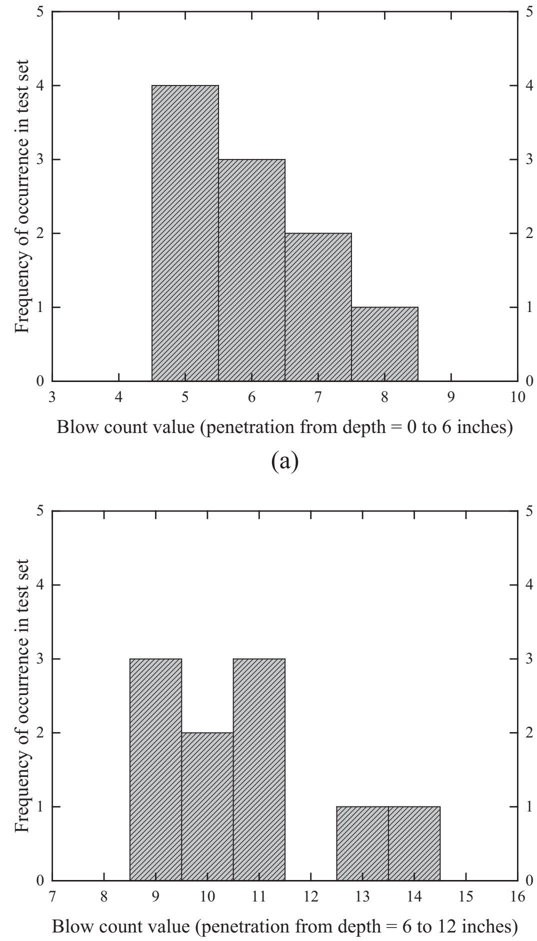


Figure A.17 Frequency vs. DCP blow count for DCP-F-A6-1-NV-1: (a) 0 to 6 inches and (b) 6 to 12 inches.



(a)
(b)

Figure A.18 Frequency vs. DCP blow count for DCP-F-A6-1-NV-2: (a) 0 to 6 inches and (b) 6 to 12 inches.



(a)
(b)

Figure A.19 Frequency vs. DCP blow count for DCP-F-A6-1-NV-3: (a) 0 to 6 inches and (b) 6 to 12 inches.

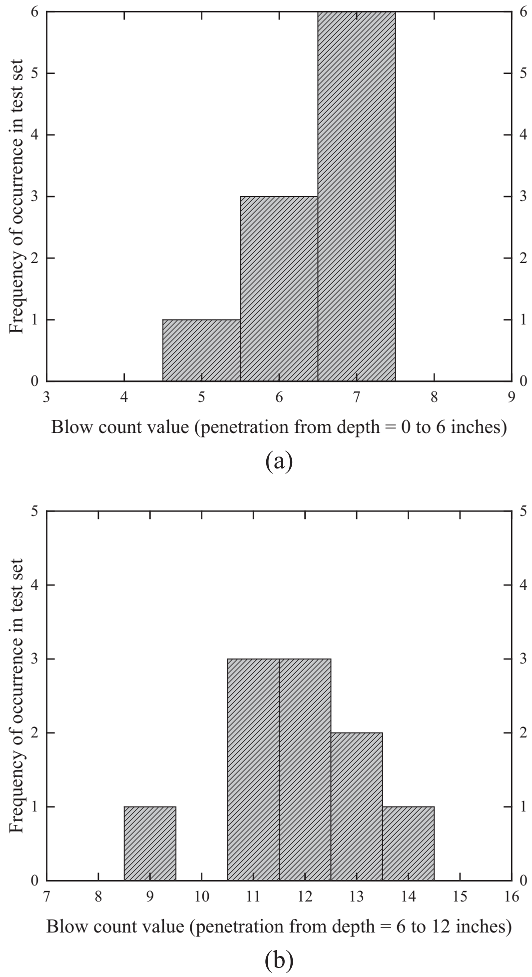


Figure A.20 Frequency vs. DCP blow count for DCP-F-A6-1-NV-4: (a) 0 to 6 inches and (b) 6 to 12 inches.

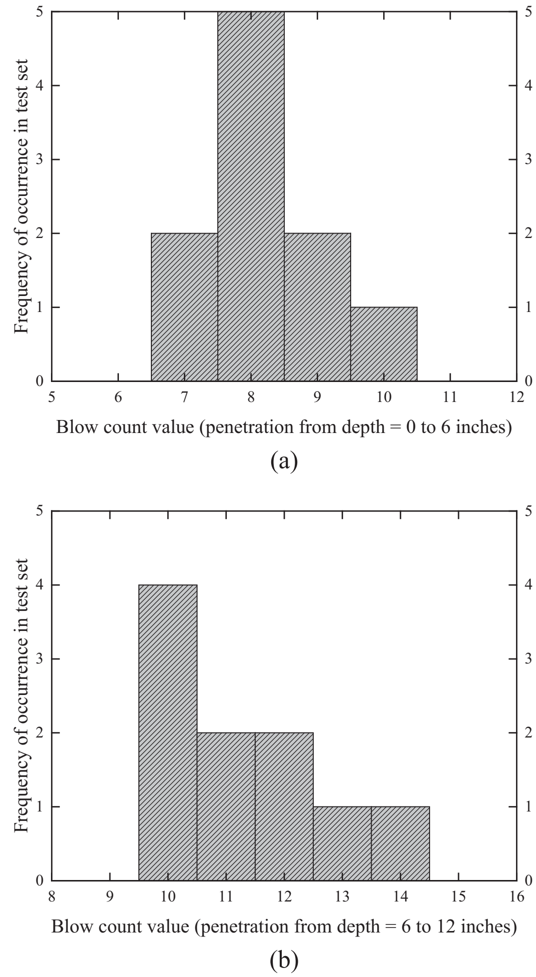
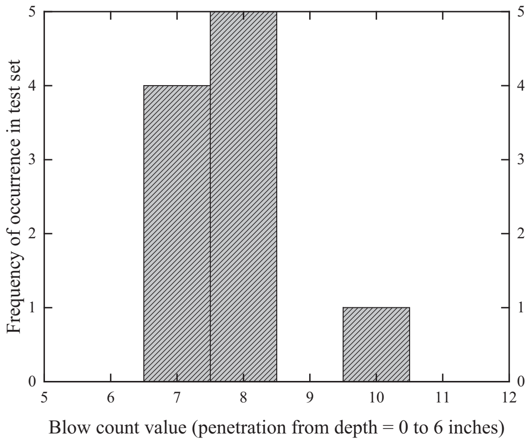
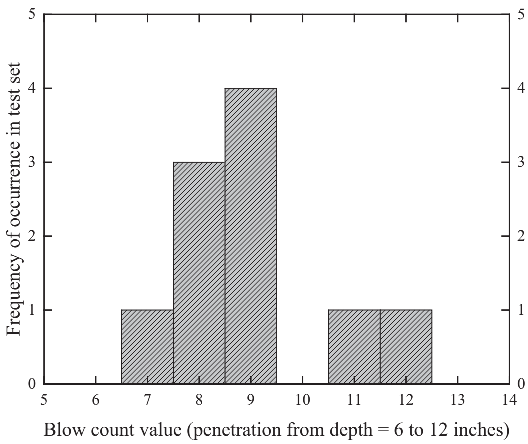


Figure A.21 Frequency vs. DCP blow count for DCP-F-A6-1-NV-5: (a) 0 to 6 inches and (b) 6 to 12 inches.

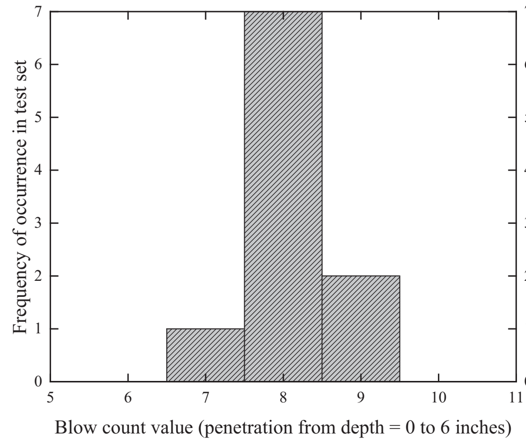


(a)

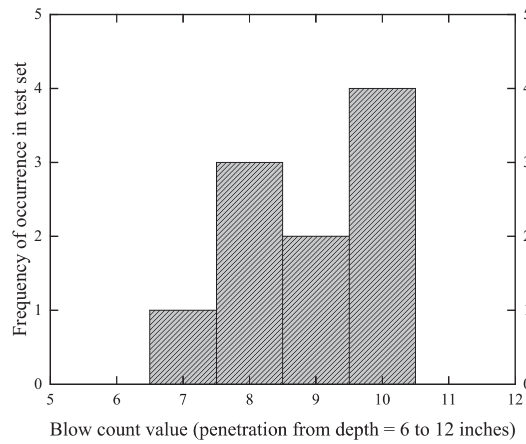


(b)

Figure A.22 Frequency vs. DCP blow count for DCP-F-A6-1-NV-6: (a) 0 to 6 inches and (b) 6 to 12 inches.



(a)



(b)

Figure A.23 Frequency vs. DCP blow count for DCP-F-A6-1-NV-7: (a) 0 to 6 inches and (b) 6 to 12 inches.

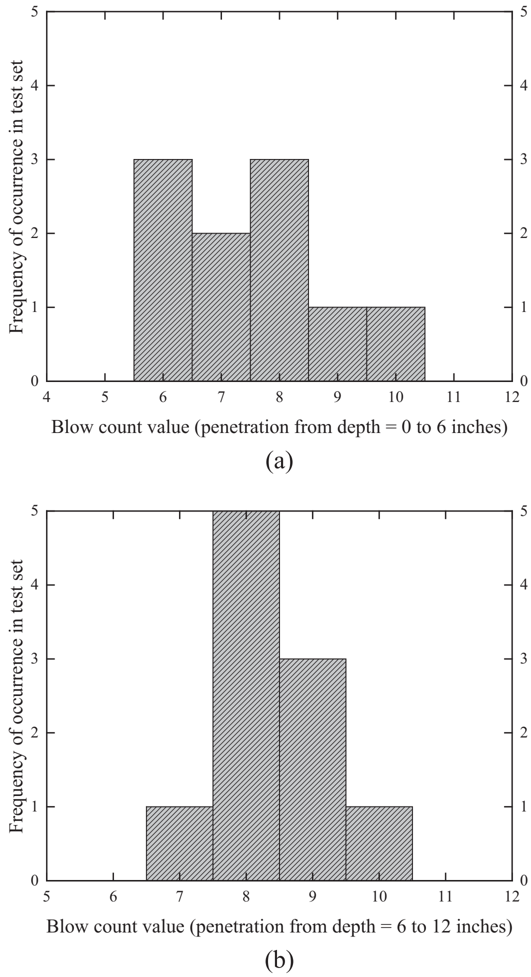


Figure A.24 Frequency vs. DCP blow count for DCP-F-A6-1-NV-8: (a) 0 to 6 inches and (b) 6 to 12 inches.

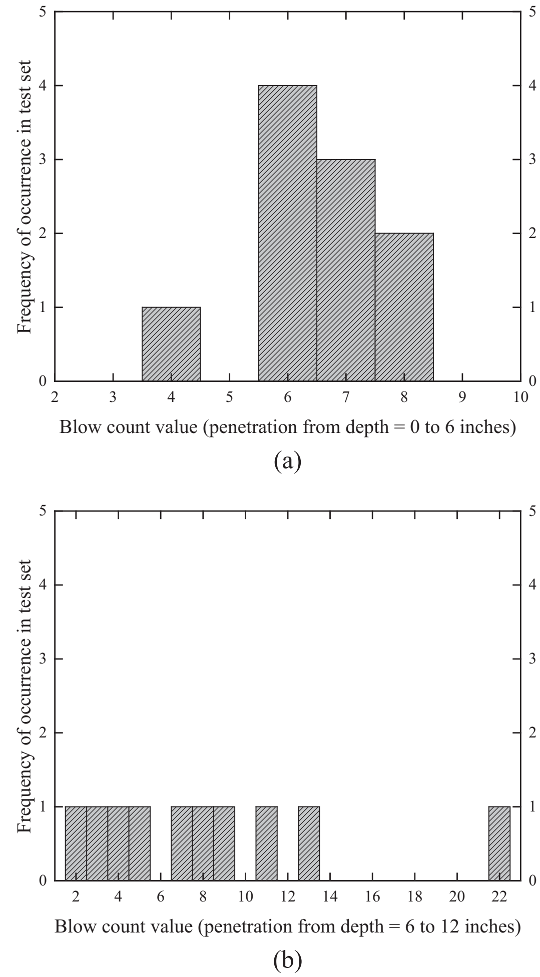


Figure A.25 Frequency vs. DCP blow count for DCP-F-A6-2-NV-1: (a) 0 to 6 inches and (b) 6 to 12 inches.

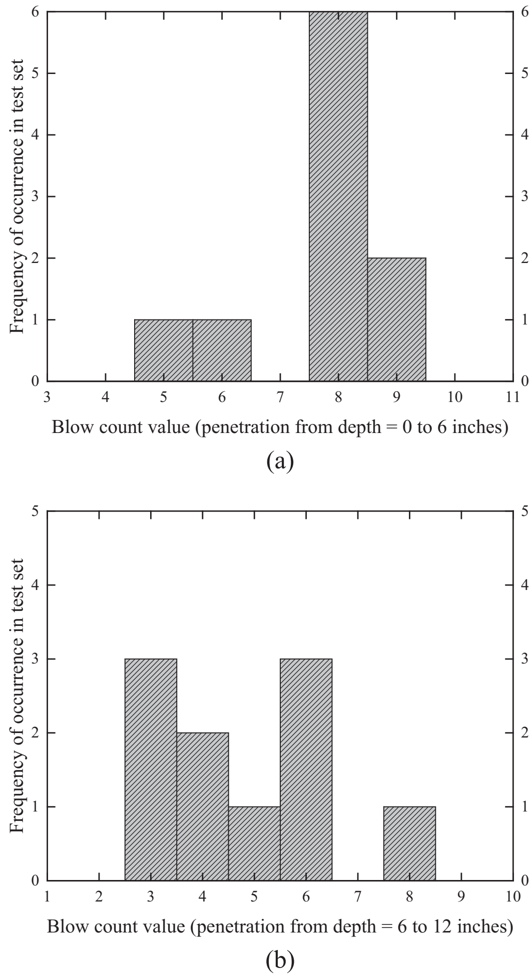


Figure A.26 Frequency vs. DCP blow count for DCP-F-A6-2-NV-2: (a) 0 to 6 inches and (b) 6 to 12 inches.

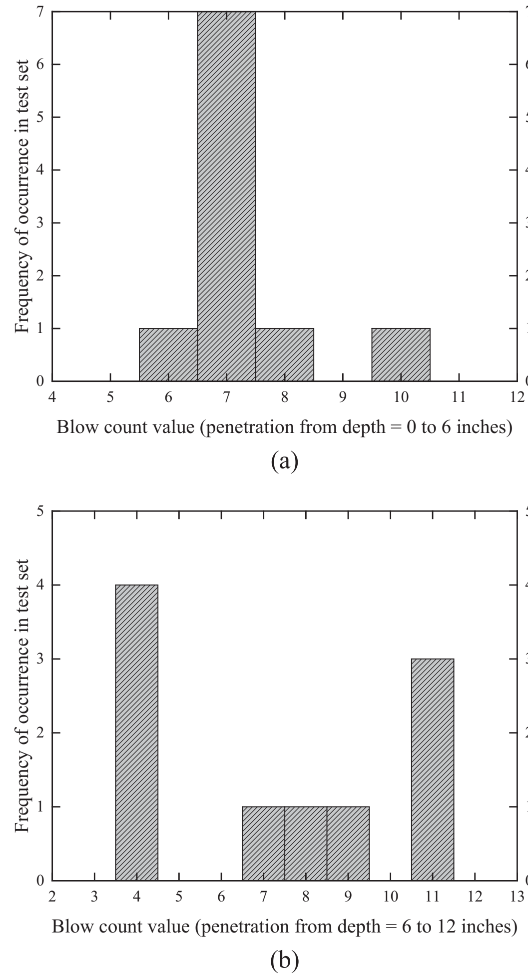


Figure A.27 Frequency vs. DCP blow count for DCP-F-A6-2-NV-3: (a) 0 to 6 inches and (b) 6 to 12 inches.

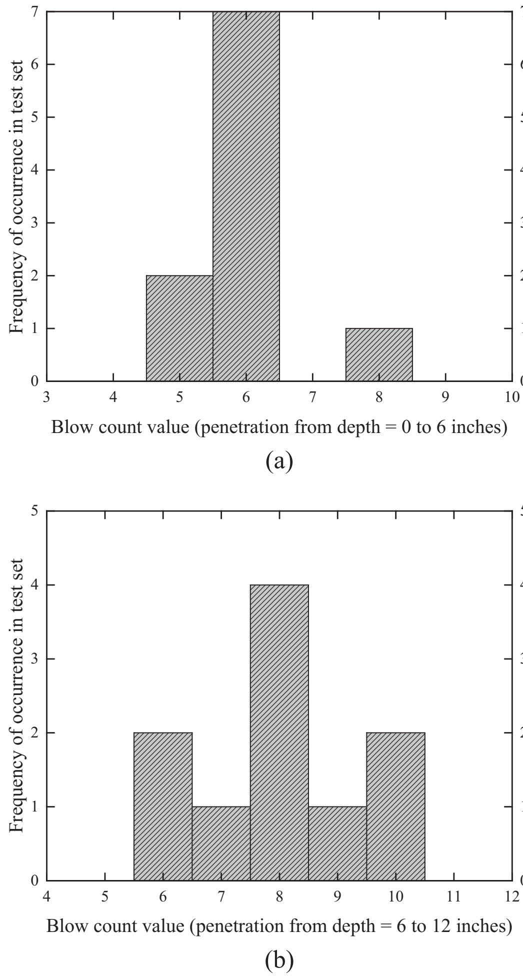


Figure A.28 Frequency vs. DCP blow count for DCP-F-A6-2-NV-4: (a) 0 to 6 inches and (b) 6 to 12 inches.

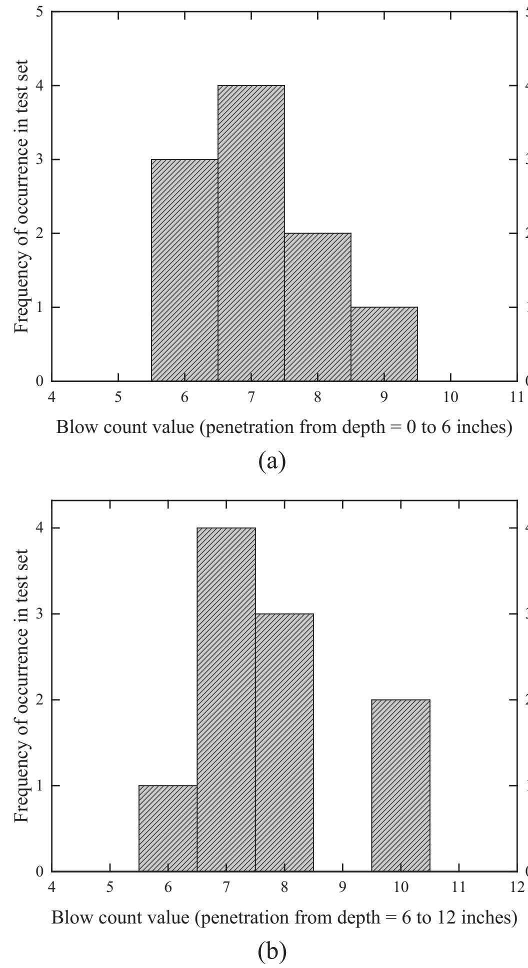
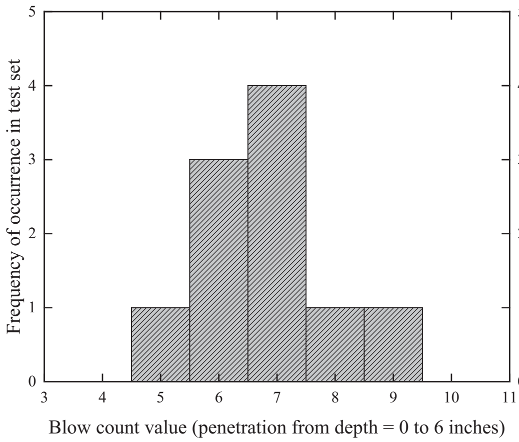
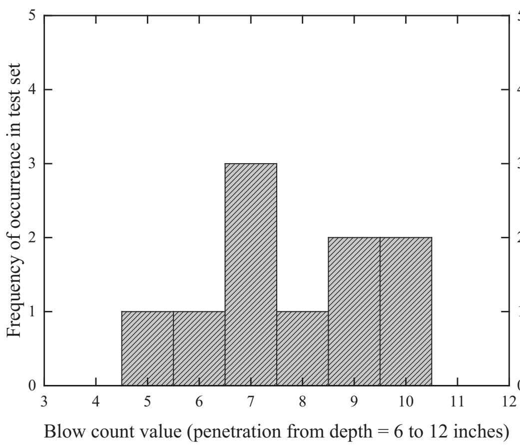


Figure A.29 Frequency vs. DCP blow count for DCP-F-A6-3-BL-1: (a) 0 to 6 inches and (b) 6 to 12 inches.

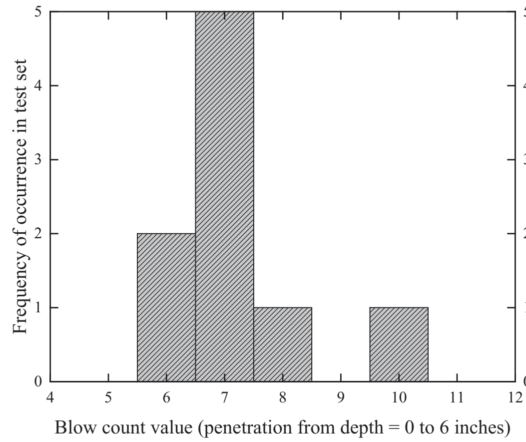


(a)

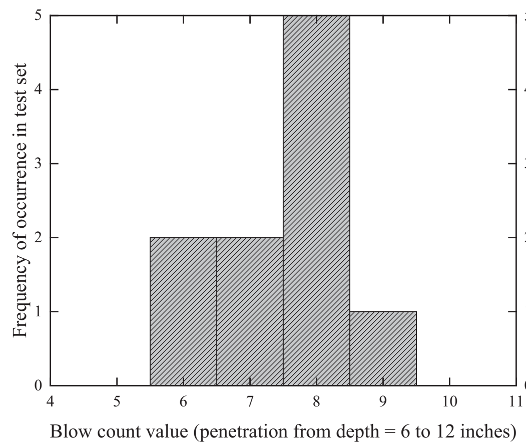


(b)

Figure A.30 Frequency vs. DCP blow count for DCP-F-A6-3-BL-2: (a) 0 to 6 inches and (b) 6 to 12 inches.



(a)



(b)

Figure A.31 Frequency vs. DCP blow count for DCP-F-A6-3-BL-3: (a) 0 to 6 inches and (b) 6 to 12 inches.

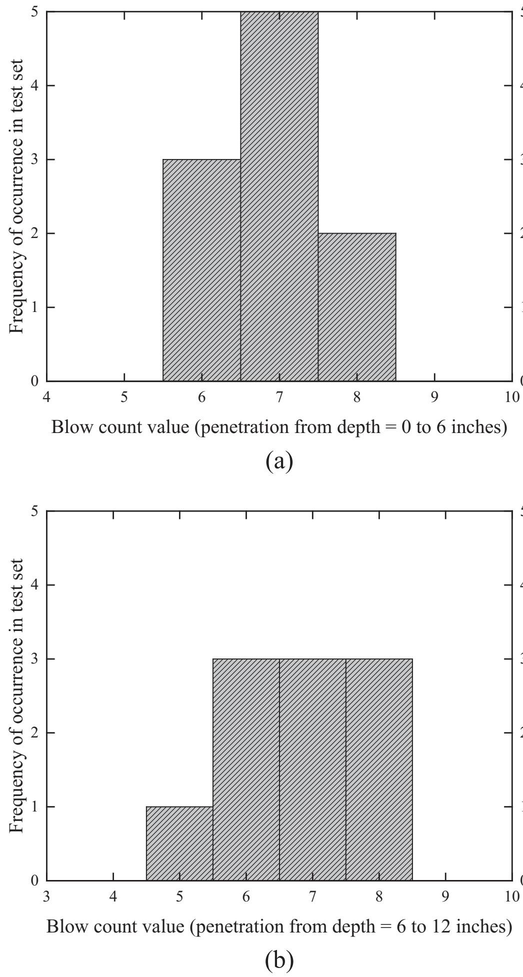


Figure A.32 Frequency vs. DCP blow count for DCP-F-A6-3-BL-4: (a) 0 to 6 inches and (b) 6 to 12 inches.

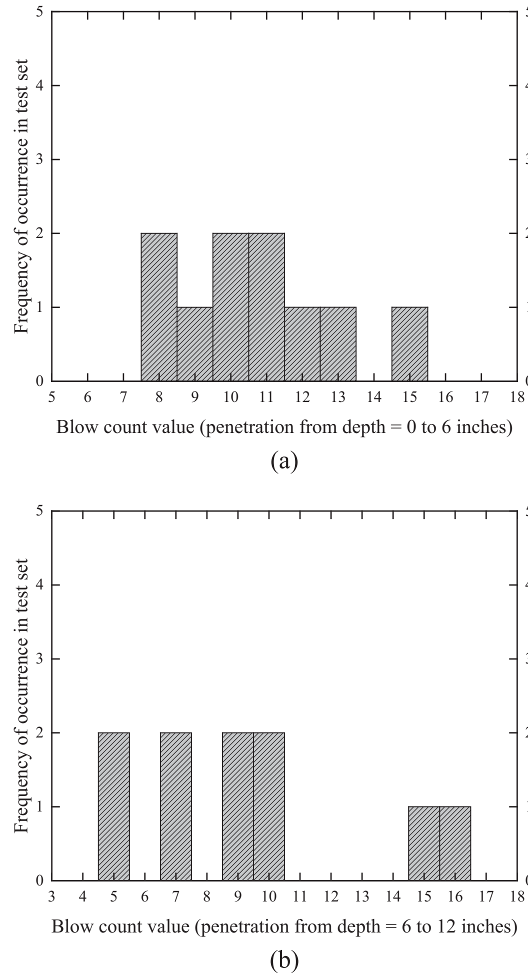
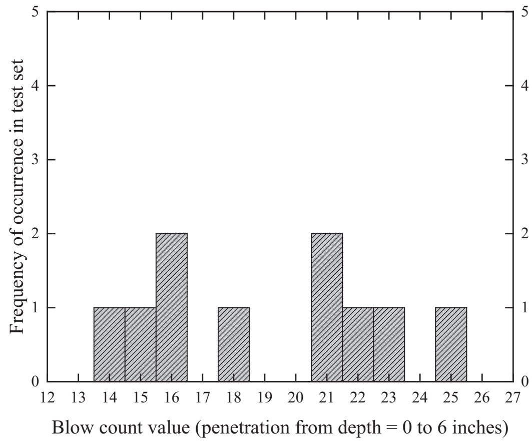
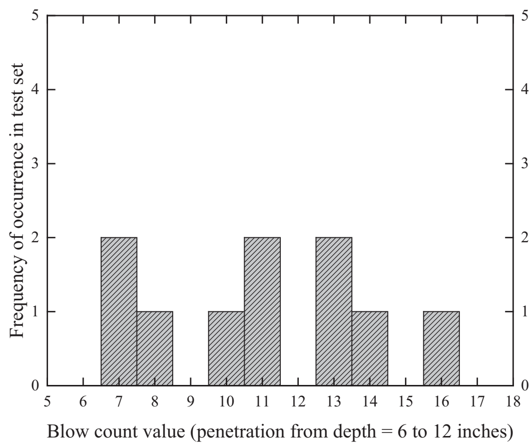


Figure A.33 Frequency vs. DCP blow count for DCP-F-A6-3-NV-1: (a) 0 to 6 inches and (b) 6 to 12 inches.

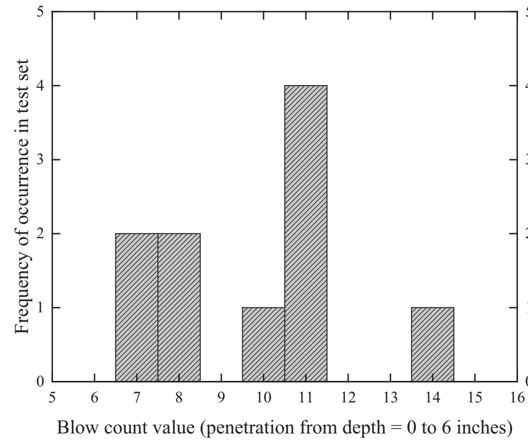


(a)

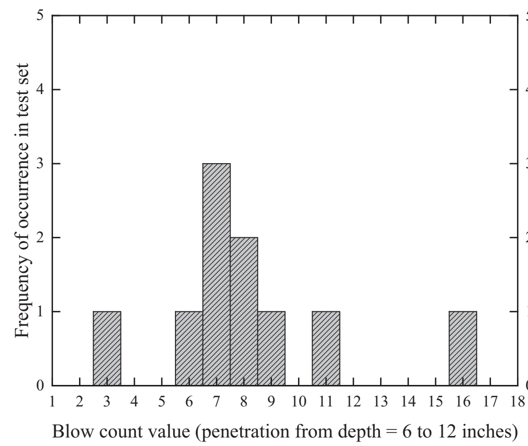


(b)

Figure A.34 Frequency vs. DCP blow count for DCP-F-A6-3-NV-2: (a) 0 to 6 inches and (b) 6 to 12 inches.

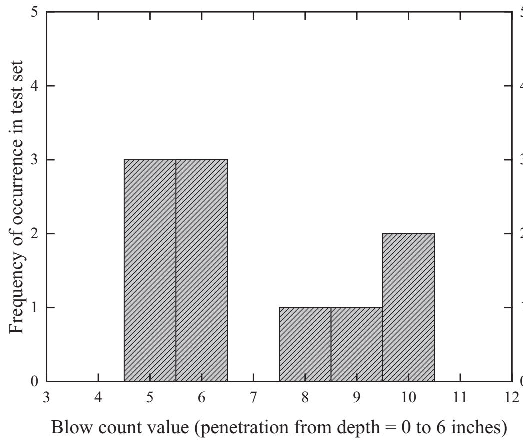


(a)

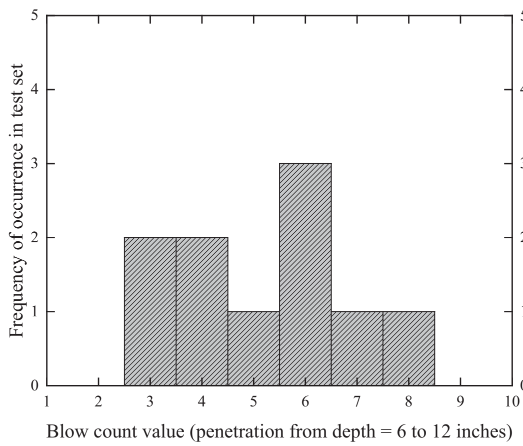


(b)

Figure A.35 Frequency vs. DCP blow count for DCP-F-A6-3-NV-3: (a) 0 to 6 inches and (b) 6 to 12 inches.

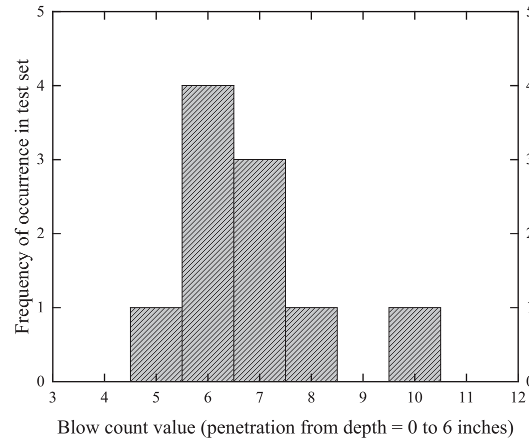


(a)

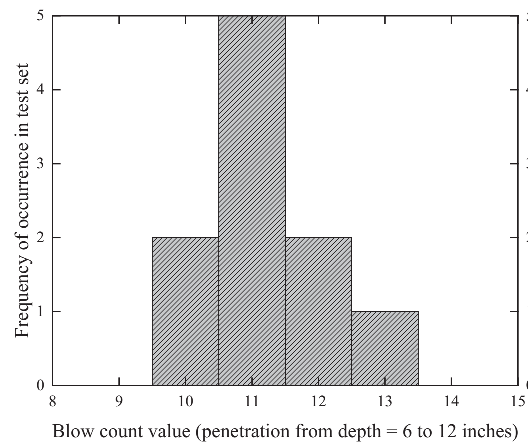


(b)

Figure A.36 Frequency vs. DCP blow count for DCP-F-A6-3-NV-4: (a) 0 to 6 inches and (b) 6 to 12 inches.



(a)



(b)

Figure A.37 Frequency vs. DCP blow count for DCP-F-A7-1-UT-1: (a) 0 to 6 inches and (b) 6 to 12 inches.

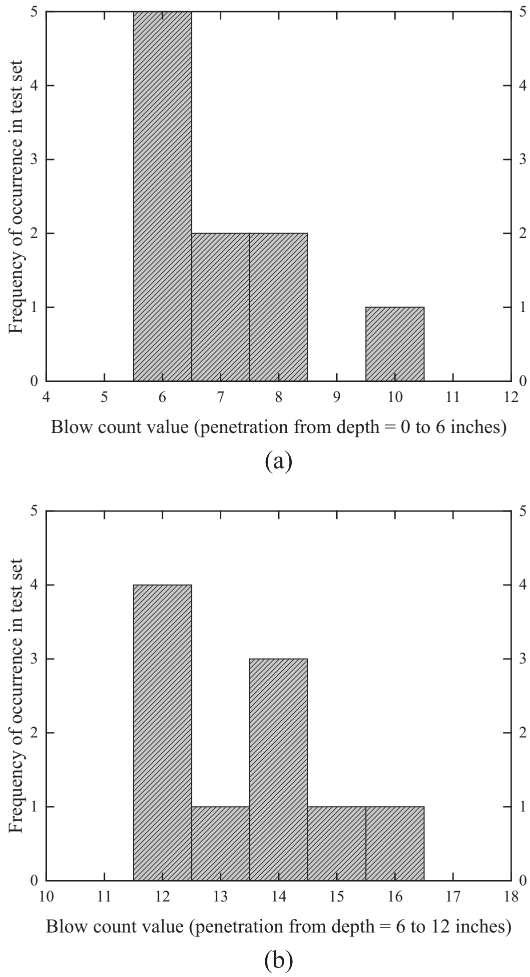


Figure A.38 Frequency vs. DCP blow count for DCP-F-A7-1-UT-2: (a) 0 to 6 inches and (b) 6 to 12 inches.

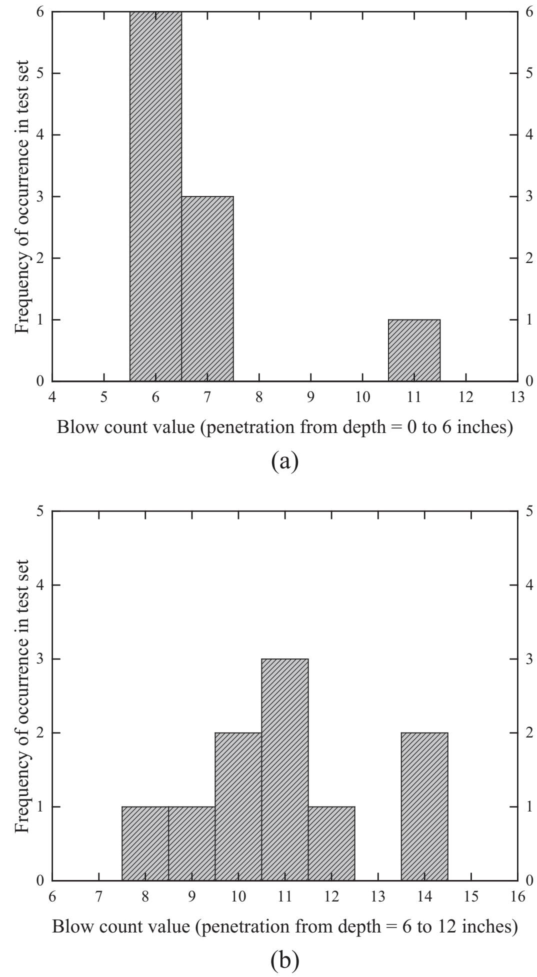


Figure A.39 Frequency vs. DCP blow count for DCP-F-A7-1-UT-3: (a) 0 to 6 inches and (b) 6 to 12 inches.

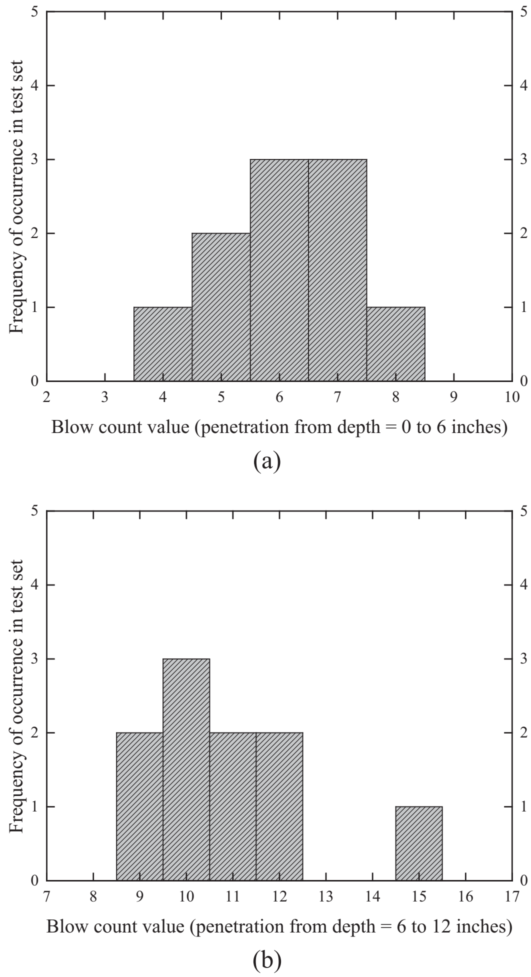


Figure A.40 Frequency vs. DCP blow count for DCP-F-A7-1-UT-4: (a) 0 to 6 inches and (b) 6 to 12 inches.

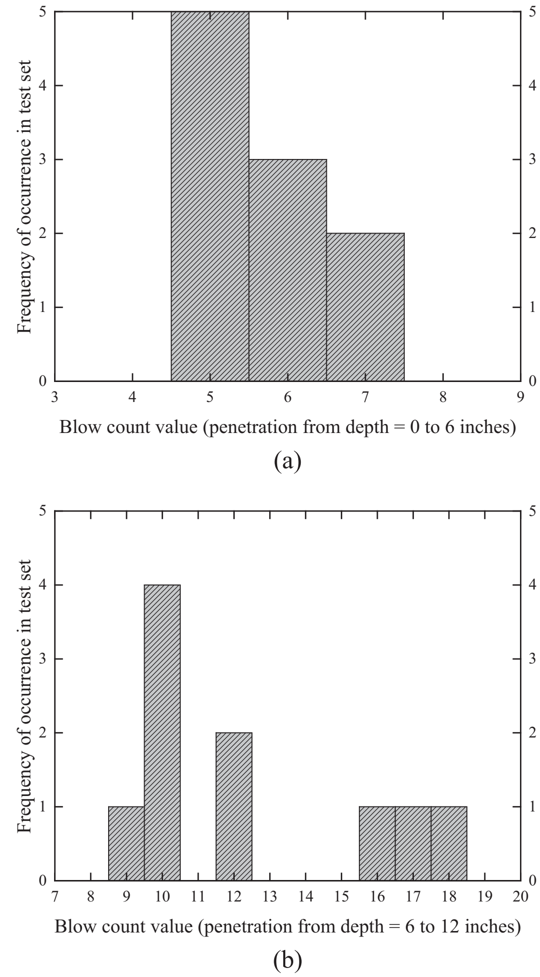
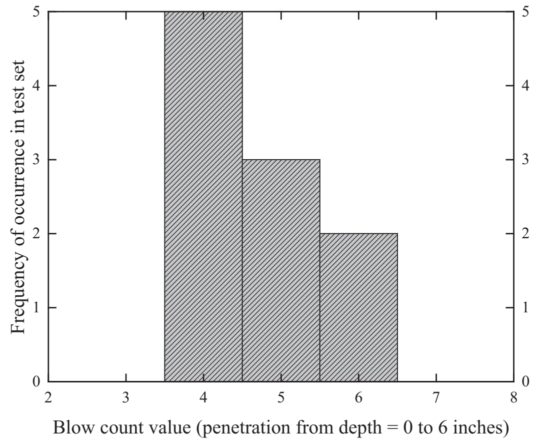
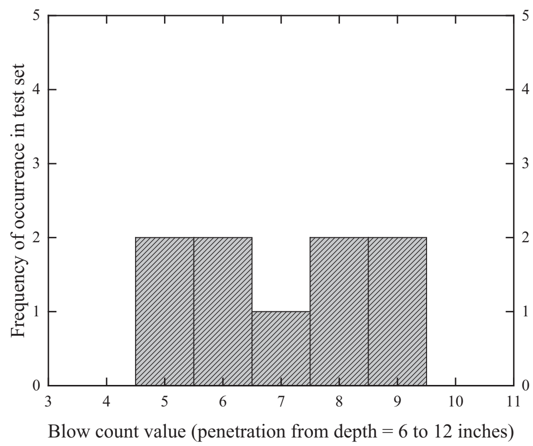


Figure A.41 Frequency vs. DCP blow count for DCP-F-A7-1-UT-5: (a) 0 to 6 inches and (b) 6 to 12 inches.



(a)



(b)

Figure A.42 Frequency vs. DCP blow count for DCP-F-A7-1-UT-6: (a) 0 to 6 inches and (b) 6 to 12 inches.

APPENDIX B: COARSE-GRAINED SOIL HISTOGRAMS

DCPT histograms for 0- to 12-inch penetration for test sets performed on coarse-grained soils are presented in this section in the same order as that in Table 5.2.

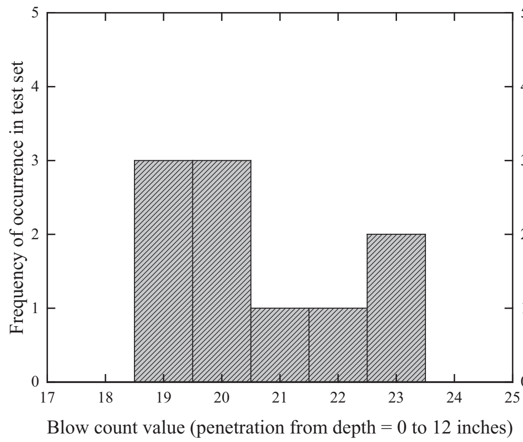


Figure B.1 Frequency vs. DCP blow count for DCP-C-A1-1-DE-1 (0- to 12-inch penetration).

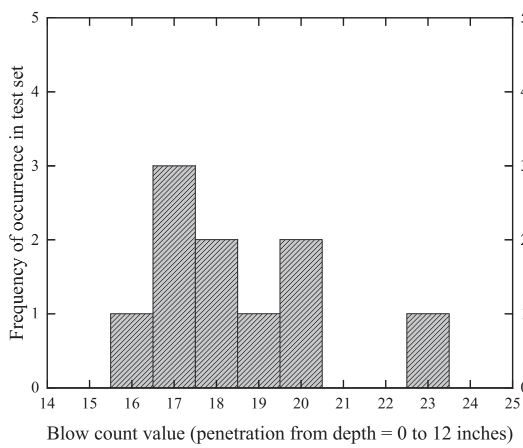


Figure B.2 Frequency vs. DCP blow count for DCP-C-A1-1-DE-2 (0- to 12-inch penetration).

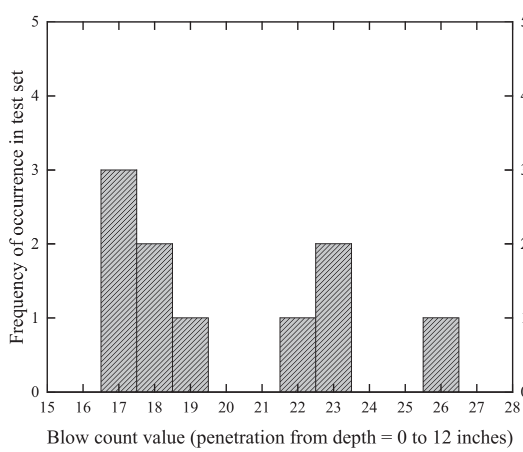


Figure B.3 Frequency vs. DCP blow count for DCP-C-A1-1-DE-3 (0- to 12-inch penetration).

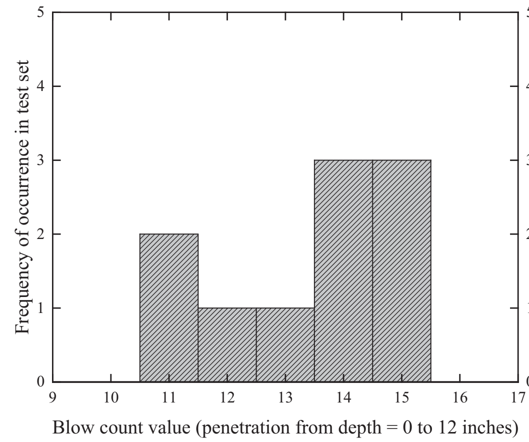


Figure B.4 Frequency vs. DCP blow count for DCP-C-A1-1-DE-4 (0- to 12-inch penetration).

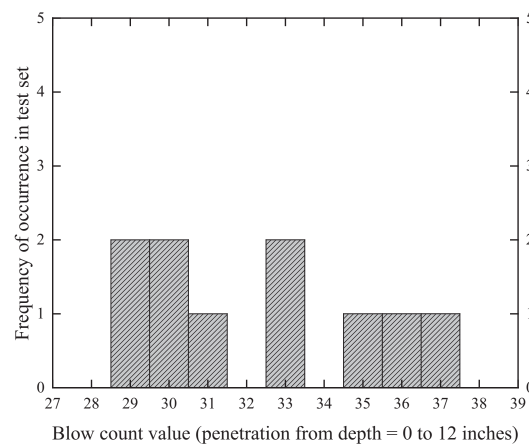


Figure B.5 Frequency vs. DCP blow count for DCP-C-A1-1-DE-5 (0- to 12-inch penetration).

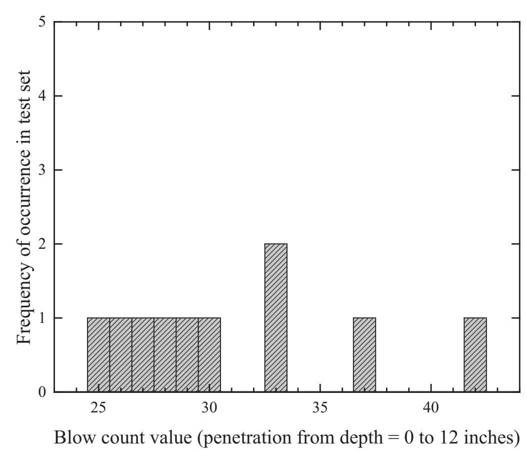


Figure B.6 Frequency vs. DCP blow count for DCP-C-A1-1-DE-6 (0- to 12-inch penetration).

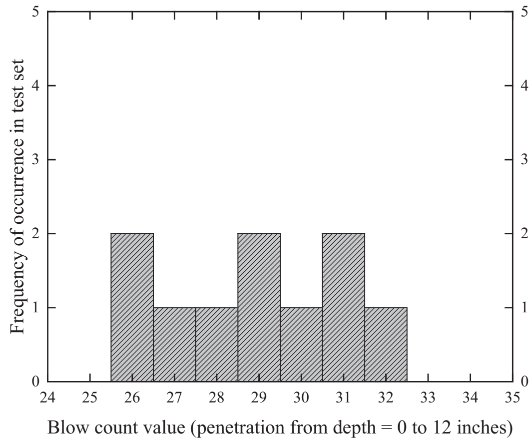


Figure B.7 Frequency vs. DCP blow count for DCP-C-A1-1-DE-7 (0- to 12-inch penetration).

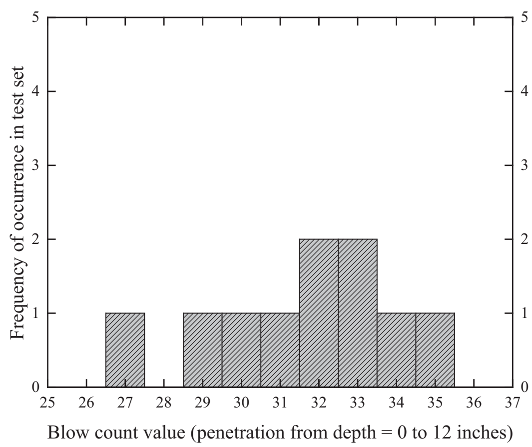


Figure B.8 Frequency vs. DCP blow count for DCP-C-A1-1-DE-8 (0- to 12-inch penetration).

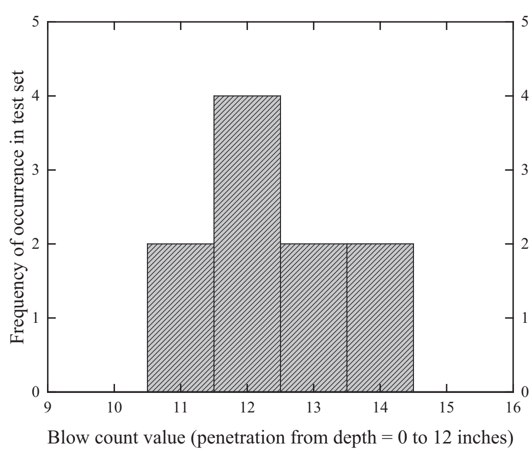


Figure B.9 Frequency vs. DCP blow count for DCP-C-A1-1-DE-9 (0- to 12-inch penetration).

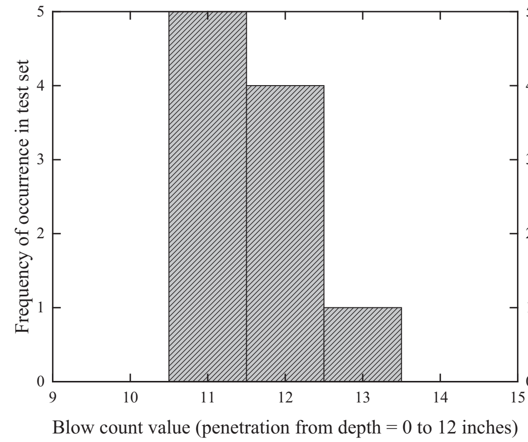


Figure B.10 Frequency vs. DCP blow count for DCP-C-A1-1-DE-10 (0- to 12-inch penetration).

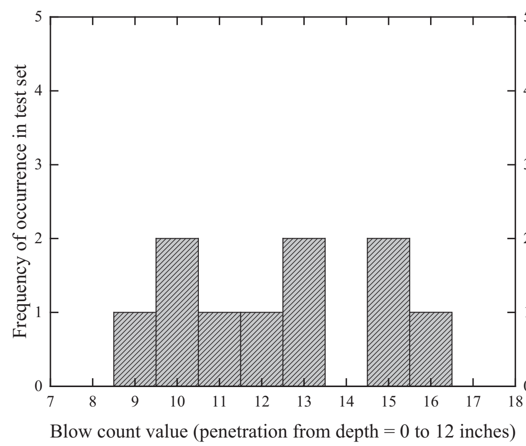


Figure B.11 Frequency vs. DCP blow count for DCP-C-A1-1-DE-11 (0- to 12-inch penetration).

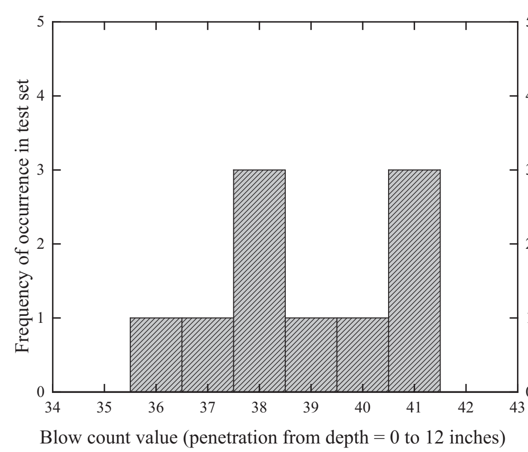


Figure B.12 Frequency vs. DCP blow count for DCP-C-A1-2-DE-1 (0- to 12-inch penetration).

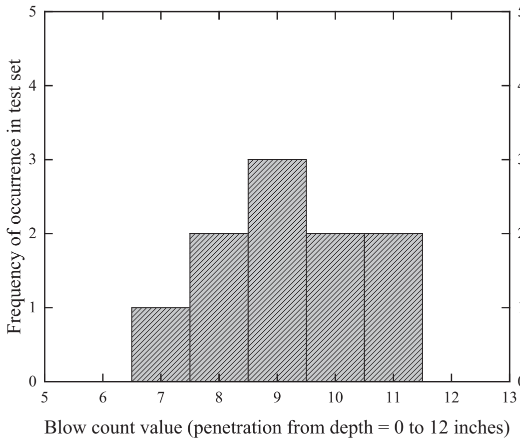


Figure B.13 Frequency vs. DCP blow count for DCP-C-A1-2-DE-2 (0- to 12-inch penetration).

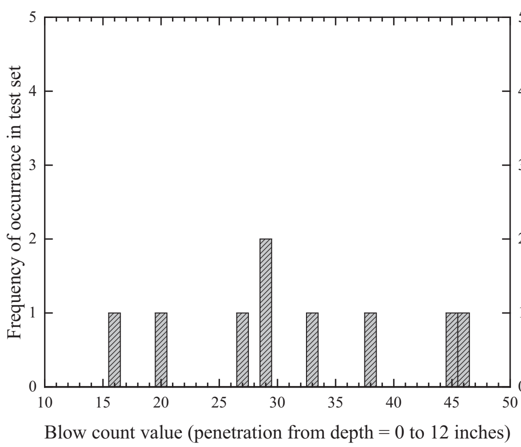


Figure B.14 Frequency vs. DCP blow count for DCP-C-A1-2-DE-3 (0- to 12-inch penetration).

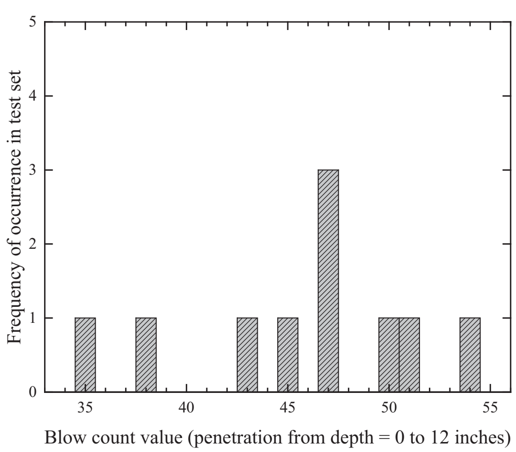


Figure B.15 Frequency vs. DCP blow count for DCP-C-A1-2-DE-4 (0- to 12-inch penetration).

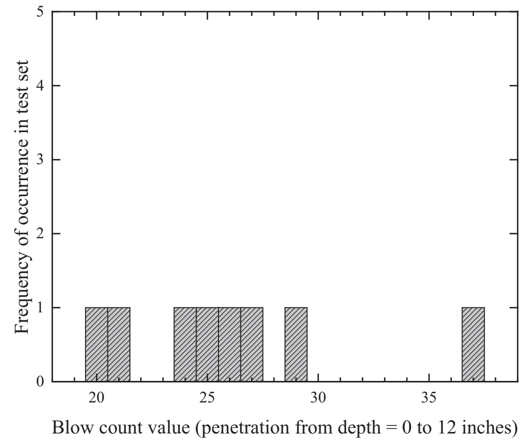


Figure B.16 Frequency vs. DCP blow count for DCP-C-A1-2-DE-5 (0- to 12-inch penetration).

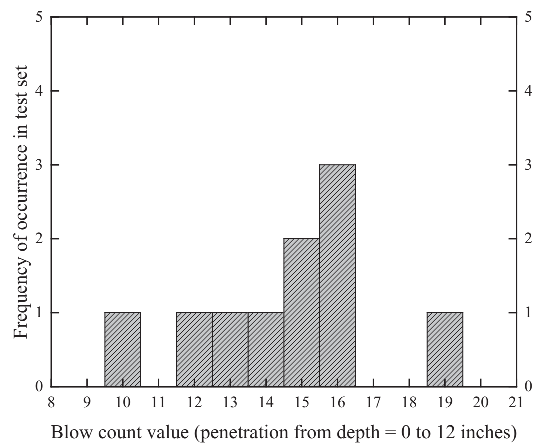


Figure B.17 Frequency vs. DCP blow count for DCP-C-A1-2-DE-6 (0- to 12-inch penetration).

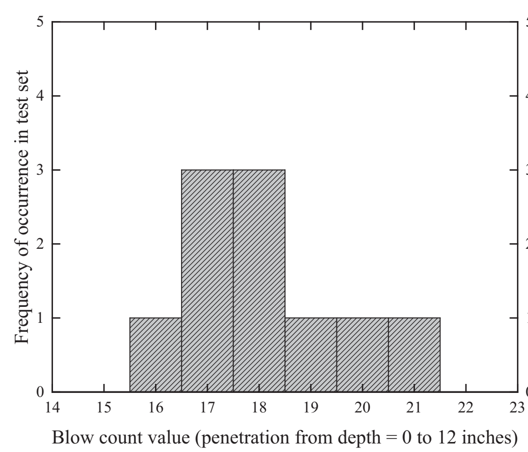


Figure B.18 Frequency vs. DCP blow count for DCP-C-A1-2-DE-7 (0- to 12-inch penetration).

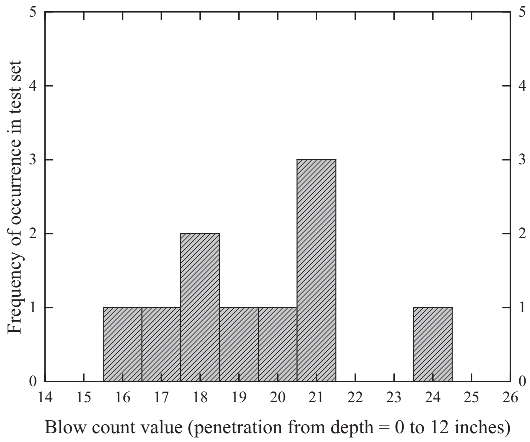


Figure B.19 Frequency vs. DCP blow count for DCP-C-A1-2-DE-8 (0- to 12-inch penetration).

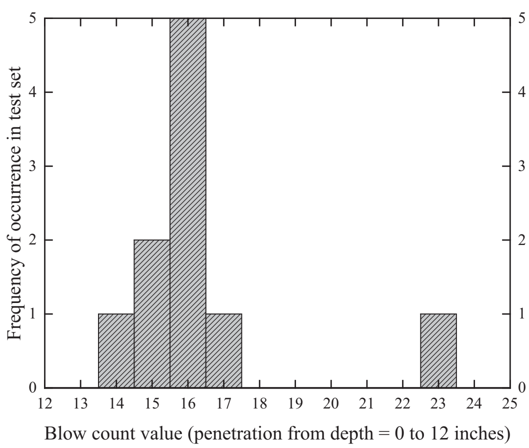


Figure B.20 Frequency vs. DCP blow count for DCP-C-A1-2-DE-9 (0- to 12-inch penetration).

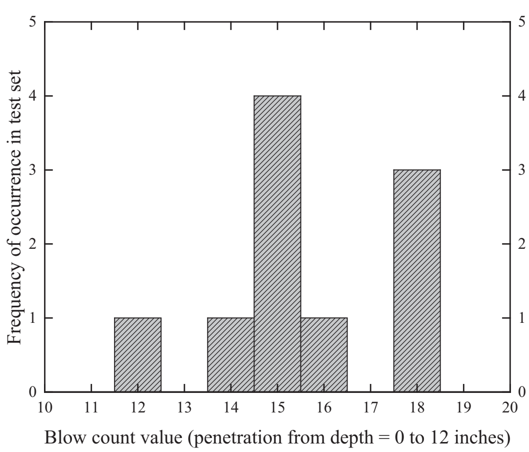


Figure B.21 Frequency vs. DCP blow count for DCP-C-A1-2-DE-10 (0- to 12-inch penetration).

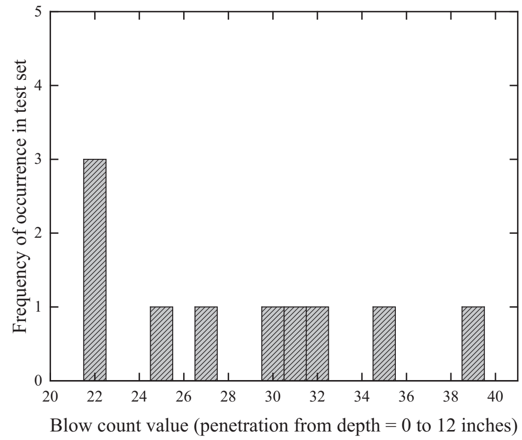


Figure B.22 Frequency vs. DCP blow count for DCP-C-A1-3-DE-1 (0- to 12-inch penetration).

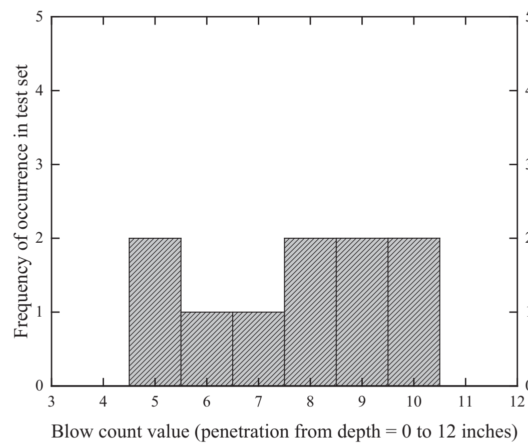


Figure B.23 Frequency vs. DCP blow count for DCP-C-A1-3-KO-1 (0- to 12-inch penetration).

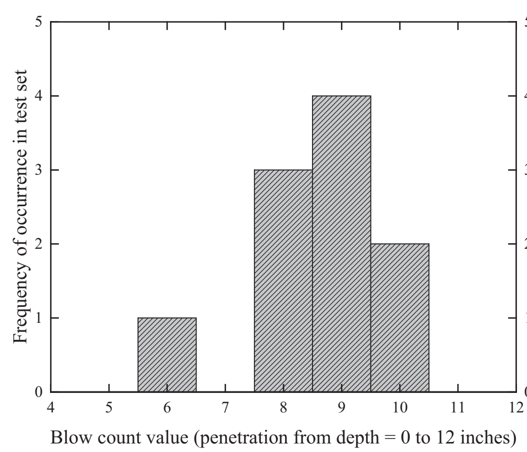


Figure B.24 Frequency vs. DCP blow count for DCP-C-A1-3-KO-2 (0- to 12-inch penetration).

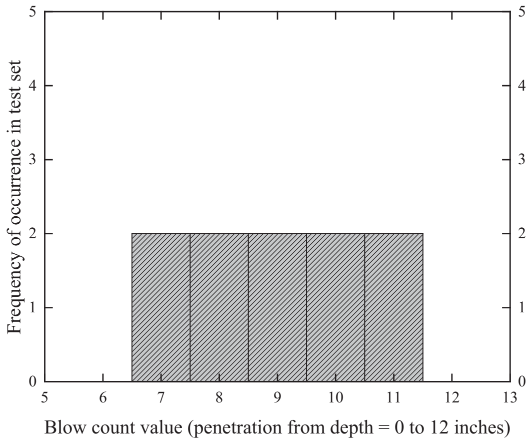


Figure B.25 Frequency vs. DCP blow count for DCP-C-A1-3-KO-3 (0- to 12-inch penetration).

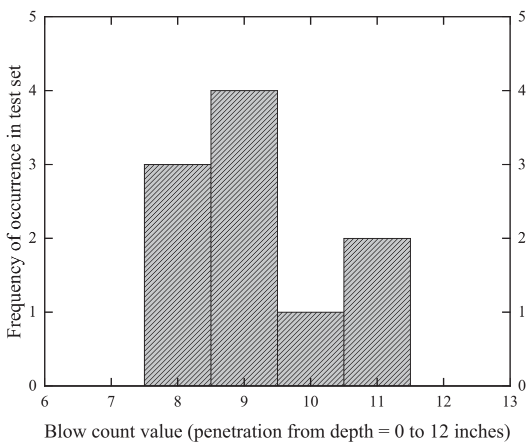


Figure B.26 Frequency vs. DCP blow count for DCP-C-A1-3-KO-4 (0- to 12-inch penetration).

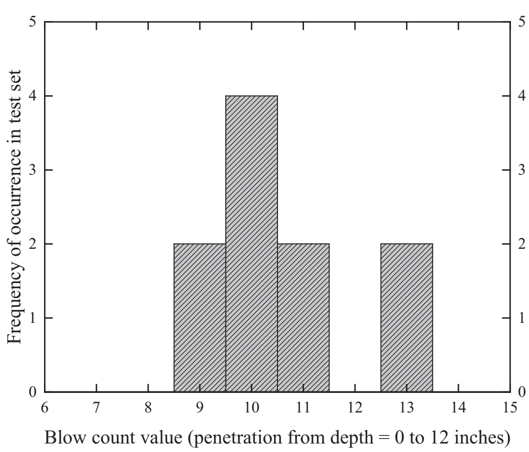


Figure B.27 Frequency vs. DCP blow count for DCP-C-A1-3-KO-5 (0- to 12-inch penetration).

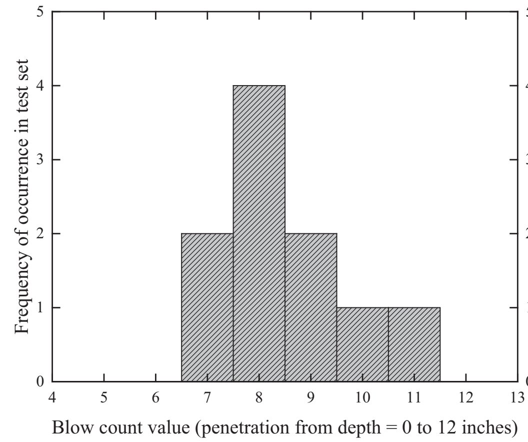


Figure B.28 Frequency vs. DCP blow count for DCP-C-A1-3-KO-6 (0- to 12-inch penetration).

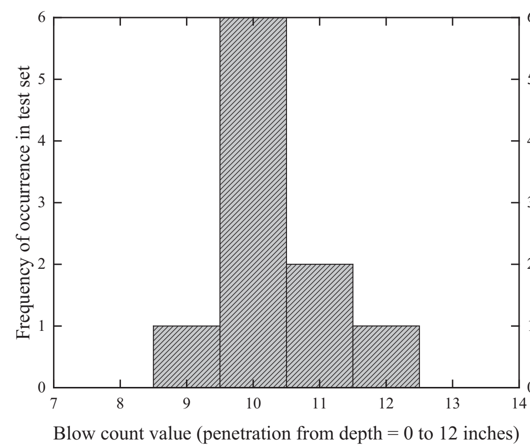


Figure B.29 Frequency vs. DCP blow count for DCP-C-A1-3-KO-7 (0- to 12-inch penetration).

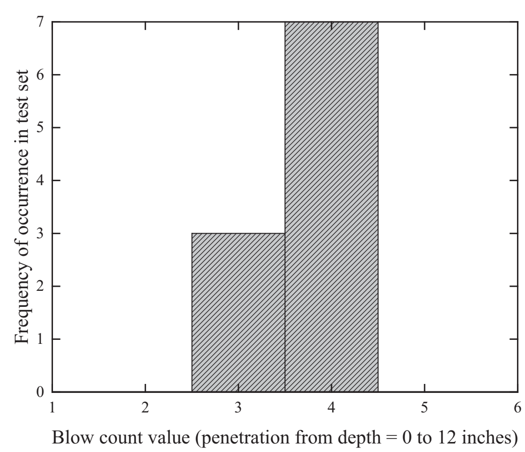


Figure B.30 Frequency vs. DCP blow count for DCP-C-A1-3-KO-8 (0- to 12-inch penetration).

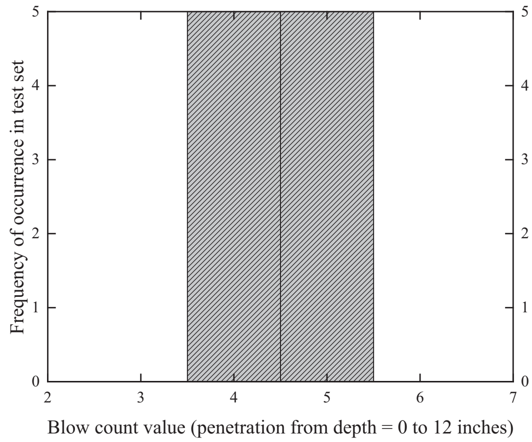


Figure B.31 Frequency vs. DCP blow count for DCP-C-A1-3-KO-9 (0- to 12-inch penetration).

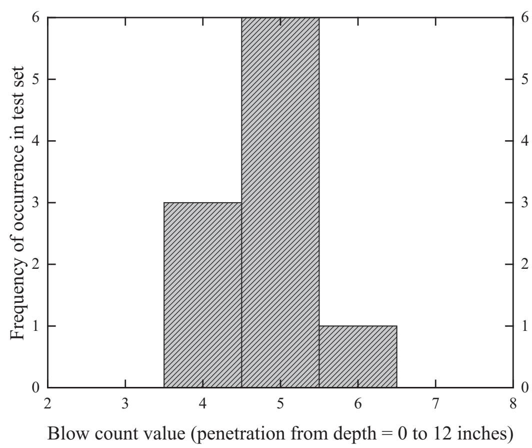


Figure B.32 Frequency vs. DCP blow count for DCP-C-A1-3-KO-10 (0- to 12-inch penetration).

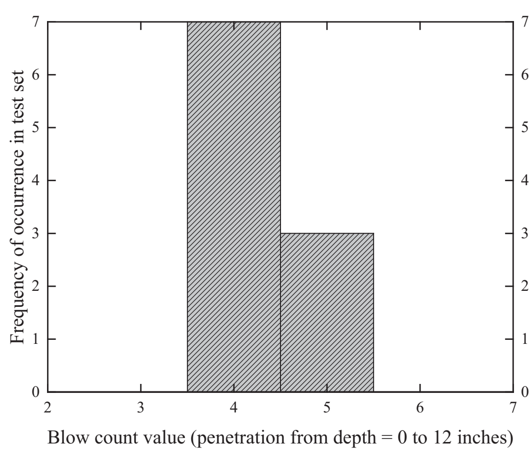


Figure B.33 Frequency vs. DCP blow count for DCP-C-A1-3-KO-11 (0- to 12-inch penetration).

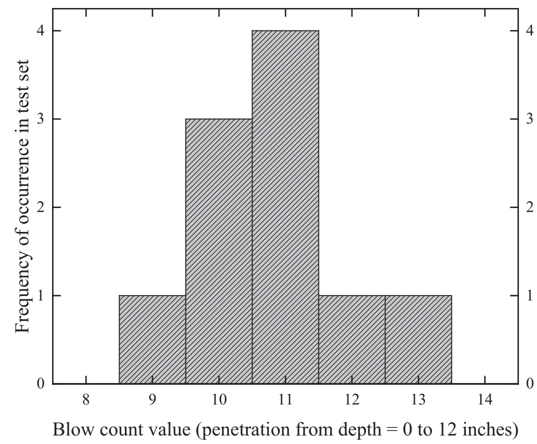


Figure B.34 Frequency vs. DCP blow count for DCP-C-A4-2-FR-1 (0- to 12-inch penetration).

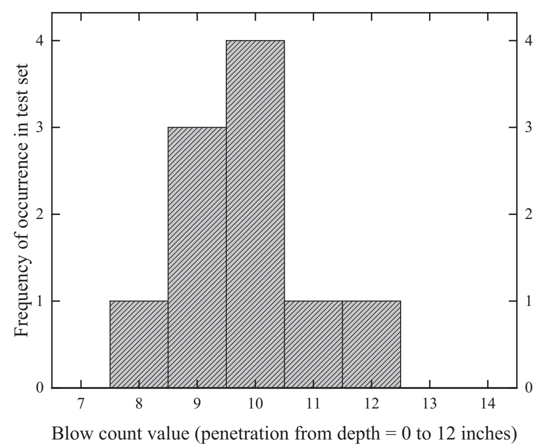


Figure B.35 Frequency vs. DCP blow count for DCP-C-A4-2-FR-2 (0- to 12-inch penetration).

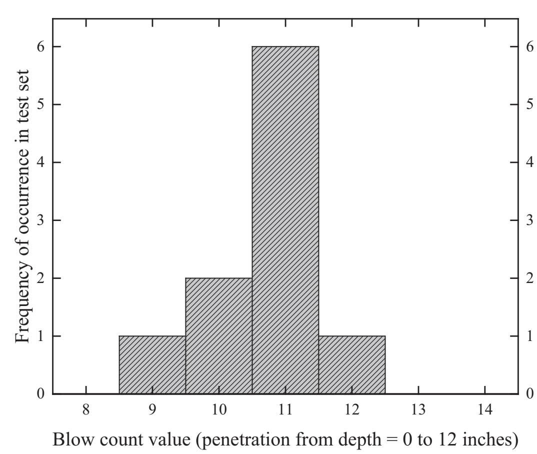


Figure B.36 Frequency vs. DCP blow count for DCP-C-A4-2-FR-3 (0- to 12-inch penetration).

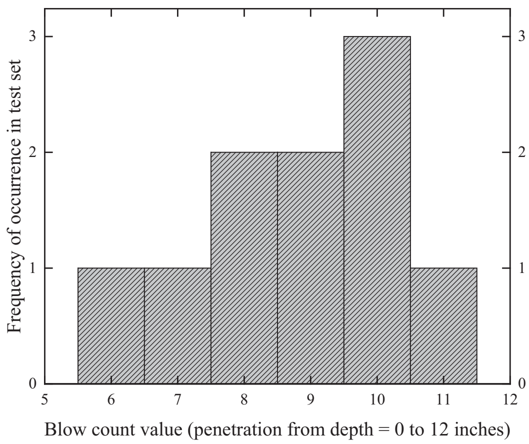


Figure B.37 Frequency vs. DCP blow count for DCP-C-A4-2-FR-4 (0- to 12-inch penetration).

About the Joint Transportation Research Program (JTRP)

On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,500 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

Free online access to all reports is provided through a unique collaboration between JTRP and Purdue Libraries. These are available at: http://docs.lib.purdue.edu/jtrp

Further information about JTRP and its current research program is available at: http://www.purdue.edu/jtrp

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