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Approved by: Michael Kreger 07/10/2014

Head of the Department Graduate Program Date

QUALITY ASSURANCE AND QUALITY CONTROL CORRELATIONS FOR THE DYNAMIC CONE PENETROMETER

A Thesis

Submitted to the Faculty

of

Purdue University

by

Eshan Ganju

In Partial Fulfillment of the

Requirements for the Degree

of

Master of Science in Civil Engineering

August 2014

Purdue University

West Lafayette, Indiana

"Scientific understanding proceeds by way of constructing and analyzing models of the segments or aspects of reality under study. The purpose of these models is not to give a mirror image of reality, not to include all its elements in their exact sizes and proportions, but rather to single out and make available for intensive investigation those elements which are most decisive."

- David Muir Wood

(Soil Behavior and Critical State Soil Mechanics)

ACKNOWLEDGEMENTS

The author would first like to thank his advisors, Professor Monica Prezzi and Professor Rodrigo Salgado for their guidance and support, technical and financial, during the course of the study. Special thanks are also due to Professor Tonglin Zhang for his advice on the statistical aspects of the research presented in this dissertation.

The author is also grateful to colleague Hobi Kim for helping with the DCP field tests and providing valuable insights that helped shape essential parts of the author's dissertation.

Last, but not the least, the author would like to acknowledge the financial support and help for laboratory and field testing received from the Indianan Department of Transportation (INDOT) via the Joint Transport Research Project (JTRP).

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ABSTRACT

Ganju, Eshan. M.S.C.E., Purdue University, August 2014. QA/QC Correlations for Dynamic Cone Penetrometer. Major Professor: Monica Prezzi.

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 passing 0.075 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.

CHAPTER 1. INTRODUCTION

1.1 Background

Pavement construction comprises formation of four main layers: i) the surface course, which comes in direct contact with the vehicles and may be rigid or non-rigid, ii) the base course, which provides drainage and frost protection, iii) the sub-base layer, which provides further load distribution and iv) the subgrade, which basically is soil in its natural state or modified to satisfy specified requirements (Christopher et al. 2006). Figure 1.1 shows a schematic of the cross section of the pavement layers. Out of the four layers of a typical pavement, the subgrade is the most variable in composition and sensitive to changes in moisture conditions.

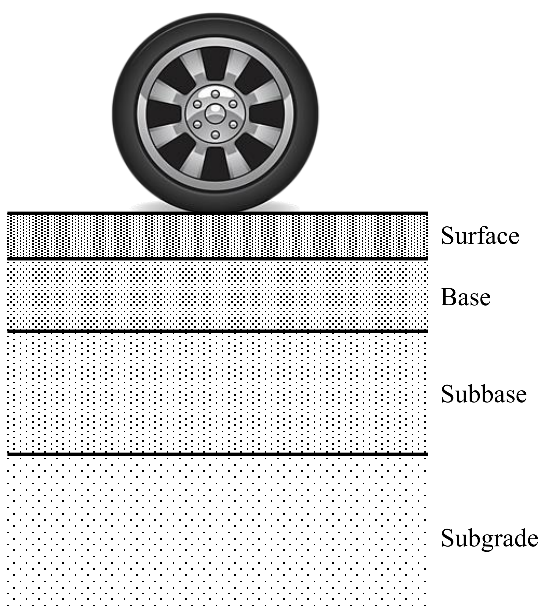


Figure 1.1 Schematic of pavement layers

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, the probability of having such ideal conditions in the field is practically non-existent. 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 denser using a wide spectrum of techniques. Historically, the process of compaction of soil was used in India and China (for example, the Great Wall of China was constructed in many sections using compacted soil), 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 1981). However, it was only in the past century or so that intensive research has been 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 1920's, the U.S. took interest in the construction of a diverse roadway network and, given the manpower and financial input involved in such large-scale projects, specifications and quality checks were 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 pavements. Today, compaction QA/QC criteria form an integral part of the pavement construction process. QA criteria, in the form of process specifications set by the design engineers, focus on the process of construction of the pavements. QC criteria on the other hand 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 regard to the construction of subgrade, QC is enforced by use of destructive or non-destructive tests that check whether the *end-product specifications* were achieved (Holtz & Kovacs 1981). 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 Department of Transportations (DOTs) in the U.S. prescribe to RC values in the range of 95-100% (South Dakota Department of Transportation 2004; Illinois Department of Transportation 2004; Iowa Department of Transportation 2012; New York Department of Transportation 2008; Minnesota Department of Transportation 2014; Indiana Department of Transportation 2012). To enforce QC 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 QC practices have moved towards non-destructive testing. By virtue of being accurate (Noureldin et al. 2005), less damaging to the subgrade and quicker than most destructive methods, most DOTs have started to transition from destructive to non-destructive methods for QC 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 (DCP's) are some of the most widely used devices for non-destructive testing of subgrade.

The focus of this research is the use of the Dynamic Cone Penetrometer (DCP) for QC of subgrade construction. Figure 1.2 shows a schematic of the device. 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 upon impact by the falling weight on the anvil. Based on the resistance (number of blows required for 6 or 12in penetration) offered by the subgrade to the penetration of the DCP probe, an estimation of the strength and stiffness characteristics of the subgrade can be made based on previously established correlations. A more detailed description of the device and its specifications for use in subgrade construction QC will be provided in subsequent chapters.

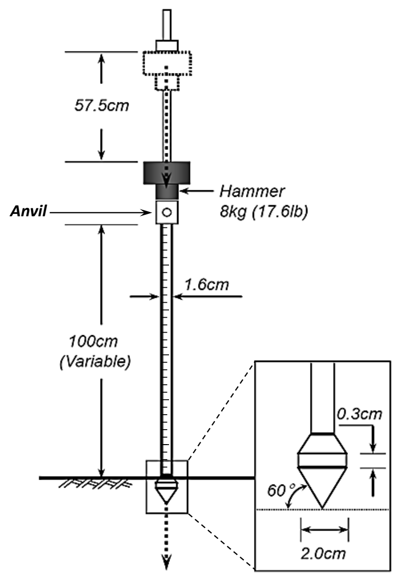


Figure 1.2 The dynamic cone penetrometer (DCP)

1.2 Problem statement

The objective of this research was to develop QA/QC correlations for the DCP that are 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 (Kim et al. 2011). As many other DOTs before it, when the funding agency, INDOT, decided that a move towards use of the Dynamic Cone Penetration Test (DCPT) for compaction QC was potentially in its interest, it funded preliminary projects (Luo et al. 1998; Salgado & Yoon 2003; Kim et al. 2011) 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 (moisture content). INDOT's 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 INDOT's 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. 2011). The most important aspects that needed further research were: i) identification of the main soil groups that showed similar response to the impact loading from the DCP, ii) 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 iii) the effect of variation in moisture content of the subgrade soil after compaction on the DCP blow count measured.

Although the criteria previously established (Kim et al. 2011) 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 QC criteria.

Based on the above points and results obtained from previous studies (Chen et al. 2001; Salgado & Yoon 2003; Kim et al. 2011) the objectives of this research were defined as *refinement of the correlations established in* (Kim et al. 2011), focusing on i) *grouping of the soils based on their 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 of the soils tested. The factors outlined above were studied, 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 thesis

The thesis has been separated into 7 chapters. Chapter 2 focuses on the literature review, detailing the subgrade and embankment construction process, description of the DCP equipment and the 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 variability associated with the DCP testing and frequency of testing required for quality control. Chapter 8 finally presents a summary of the results and conclusions.

CHAPTER 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 & Kovacs 1981). A design problem, such as 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 borrow-pits 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 geotechnical engineers. This comprises the *design phase* of the project.

After completion of the design phase, engineers also 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 i) *compaction targets to be achieved* in the field (e.g., the minimum relative compaction), ii) *equipment* to be used for compaction (compactor size and type), iii) *methods and procedures* to be followed (maximum lift thickness, acceptable range of compaction water content and number of compactor passes), and iv) *compaction quality control tests* (and their acceptable results) to be performed on the compacted soil to assess compaction quality.

Figure 2.1 presents a simplified 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.

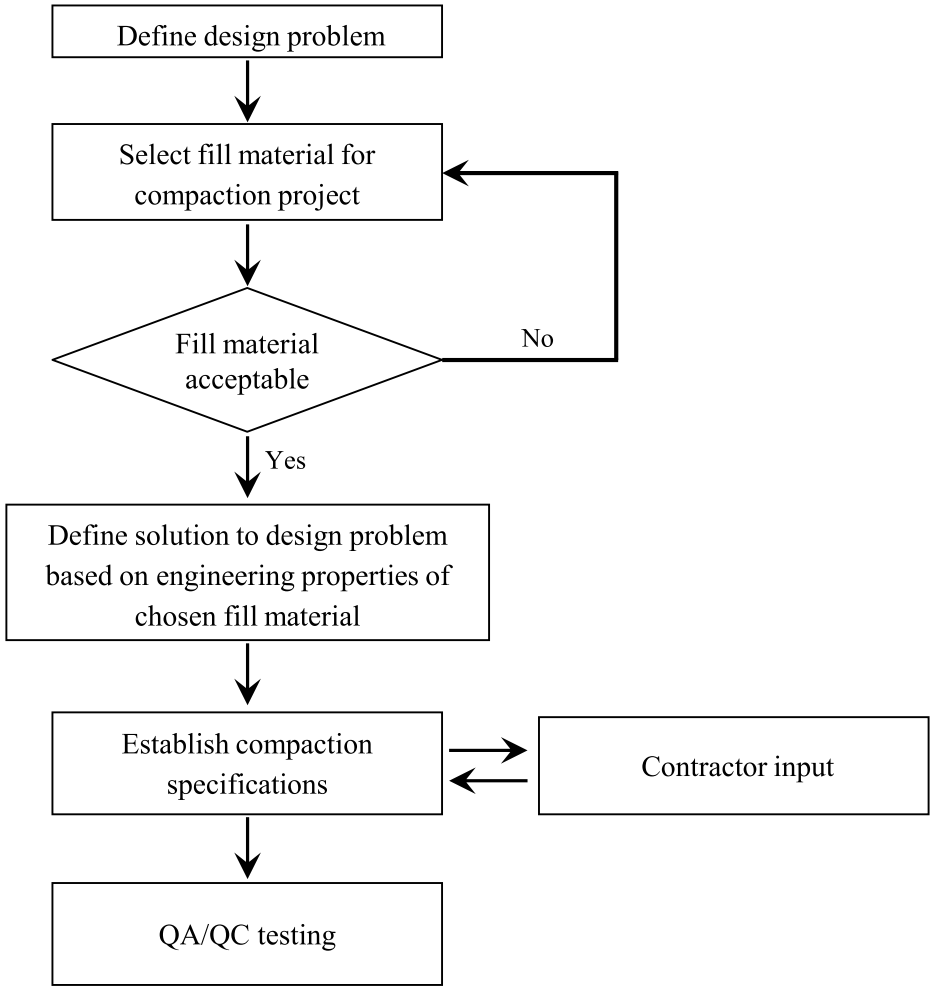


Figure 2.1 Overview of typical compaction project

Specifications have not only to be developed but also followed for compaction projects to be successful. It is for this reason that specifications contain specifics 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 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 built earth structures. Therefore, the main objectives of the QA/QC procedures are to i) ensure that the construction specifications set by consensus of engineers and contractors are met and ii) 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 i) *methods specifications*, ii) *end-product specifications* and iii) *performance-based specifications* (Transport Research Board 2005). 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 (Transport Research Board 2005; Rodriguez et al. 1988).

End-product specifications for subgrade and embankment projects are concerned with the constructed subgrade and, for most DOT's, 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 & Kovacs 1981; Transport Research Board 2005). For most roadway earthwork projects, a value of 95%-100% RC is chosen as the end-product specification (Kim et al. 2011).

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 (M_R) and California Bearing Ratio (CBR)] which must be achieved by the compacted soil (Transport Research Board 2005). 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 (National Cooperative Highway Research Program (NCHRP) 2004).

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 don't yet explicitly include performance-based specifications in their construction specifications, but a shift can be observed towards their inclusion in light of research findings (Pinard 1998; Fleming 1998; Livneh & Goldberg 2001).

Engineers 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 (National Cooperative Highway Research Program (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.

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

Agency	Condition	End-product specification	Methods Specification
		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 $OMC^{\#}$ In case of existence of adjacent structures, not more than 110% of $OMC^{\#}$ for top 2 feet
	1.5 feet ≤ Embankment height ≤ 3 feet	First lift: RC ≥ 90%	
		Consecutive lifts: RC ≥ 95%	
	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 contractor's discretion
	Embankment	RC ≥ 90%	
Indiana (2012)	Subgrade	RC ≥ 100%	Within -3% below $OMC^{\#}$ for silts and loess soils Otherwise within +1% above and -2% below the OMC
	Embankment	RC ≥ 95%	
Iowa (2012)	Subgrade	RC ≥ 95%	Within -6% below OMC for subgrade construction
	Embankment	RC > 95%	

Table 2.1 continued

Agency	Condition		End-product specification: RC Specification	Methods specification: Range of compaction water content
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% If OMC$^{\#}$ >15%, then OMC$^{\#}$-4% to OMC$^{\#}$+6%
	All cases except above		RC ≥ 95%	
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%	
		Material at depth smaller 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"

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: i) *density-based tests,* which are part of the end-product specifications (RC), and ii) *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 can be found in (Kim et al. 2011). 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 in (Kim et al. 2011; Holtz & Kovacs 1981; 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.

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

Density-Based Tests			
Test	**Description**	**Pros**	**Cons**
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	• Cannot 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

Table 2.2 continued

Performance-Based Tests			
Test	**Description**	**Pros**	**Cons**
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, M_R)	• 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 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer (DCP) is a simple and light-weight 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.

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 (Scala 1956; Livneh & Ishai 1987). The original DCP developed by *A. J. Scala* (Scala 1956) had a falling weight hammer of 9kg (instead of the present 8kg), 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) (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. 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, two operators are needed. 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 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.

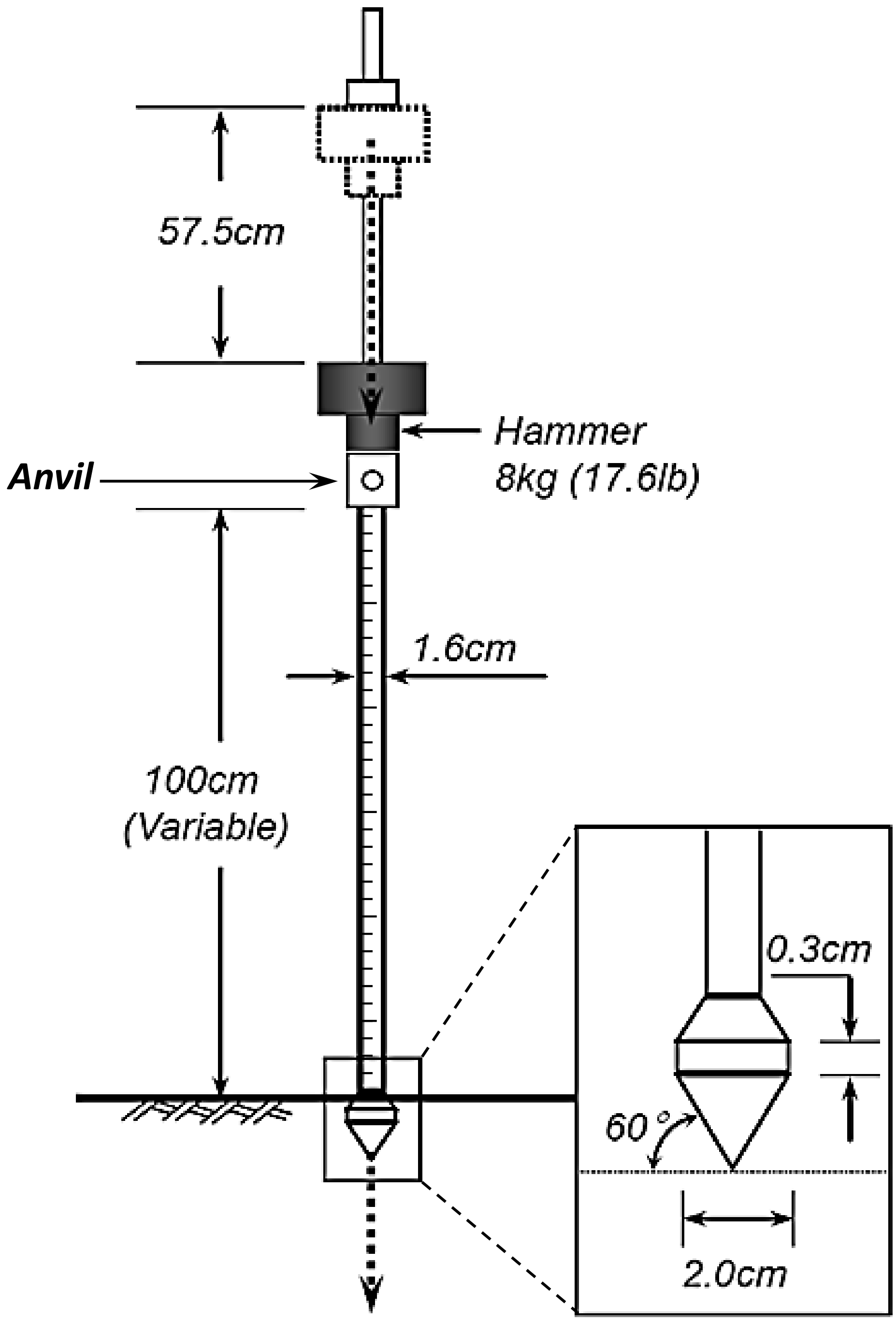


Figure 2.2 The Dynamic cone Penetrometer (After Kim et al. 2011)

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 (Van 1969; Kleyn 1975; Bester & Hallat 1977; Harison 1987; Ayers et al. 1989; Gabr & Hopkins 2000; Amini 2003) 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 frequent around the world (Livneh & Livneh 2013) . By virtue of being an economic, light and simple test, the DCP test has proved to be very useful to state agencies (such as INDOT) that are actively involved in the construction and QA/QC testing of compacted soil structures, such as 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 possible with many of the other field testing equipment.

Some research studies (Livneh & Goldberg 2001; Pinard 1998; Fleming 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 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 other soil stiffness tests for *in situ* compaction characterization (Siekmeier et al. 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 (Kim et al. 2011; Salgado & Yoon 2003; Amini 2003; Burnham 1997; Luo et al. 1998). In addition to state DOTs in the U.S., DCP research had previously also been carried out by the U.S. army (Webster et al. 1992), in which procedures for the use of DCP to estimate soil strength were presented. The DCP 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 Table2.3 and Table 2.4, most of the correlations for the DCP have been developed to relate the DCP test results (*Penetration Ratio*, *PR*) to the CBR and soil resilient modulus test results. Based on field and laboratory testing, useful correlations between penetration ratio (*PR*) for DCP and subgrade resilient modulus have also been developed and validated by (Gabr & Hopkins 2000; George et al. 2009; Mohammad & Herath 2007).

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

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

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, P_A 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 & 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 Table2.4, DCP correlations have also been attempted specifically to address the QA/QC concerns of the state DOTs (Kim et al. 2011; Salgado & Yoon 2003; White et al. 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. 2011) 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 M_R using the correlations available in the literature, as described in Table2.3 and Table 2.4

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

Property	Reference	DCP Correlations		Remarks
California Bearing Ratio	(Kleyn 1975)	$\log(\text{CBR}) = 2.62 - 1.27 \times \log(PR)$	• CBR: California bearing Ratio • *PR*: Penetration ratio, ratio of penetration of DCP probe tip to penetration depth (mm/blow)	Equation developed from laboratory test results
	(Harison 1987)	$\log(\text{CBR}) = 2.56 - 1.16 \times \log(PR)$		Equations based on results of laboratory tests performed on fine grained soils
	(Livneh et al. 1995)	$\log(\text{CBR}) = 2.46 - 1.12 \log \times (PR)$		Equation based on results of laboratory tests performed on fine-grained and coarse-grained soils
	(Livneh et al. 2000)	$\log(\text{CBR}) = 2.20 - 0.71 \times (\log(PR))^{1.5}$		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 & Hopkins 2000)	$\log(\text{CBR}) = 1.40 - 0.55 \times \log(PR)$		Equation developed based on field and laboratory testing on aggregate base coarse
	(George et al. 2009)	$\log(\text{CBR}) = 1.675 - 0.7852 \log(PR)$		Equation developed based on results of field tests on lateritic subgrades

Table 2.4 Correlations between DCP test results and Resilient Modulus, after (Salgado & Yoon 2003)

Property	**Reference**	**DCP Correlations**		**Remarks**
Subgrade Resilient Modulus	(Chen et al. 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 to 60 mm/blow was observed
	(Herath et al. 2005)	$M_r = 520.62 \times \left(\dfrac{1}{PR^{0.7362}}\right) + 0.40\left(\dfrac{\gamma_d}{wc}\right) + 0.44PI$	• 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) • *wc*: Water content (%) • *PI*: Plasticity index (%)	Resilient modulus was experimentally determined[%] in the laboratory using samples collected from the field
	(Mohammad & Herath 2007)	$M_r = 165.5 \times \left(\dfrac{1}{PR^{1.147}}\right) + 0.0966 \times \left(\dfrac{\gamma_d}{wc}\right)$	• 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) • *wc*: 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

Table 2.5 Correlations developed by (Kim et al. 2011)

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)	$NDCP_{(0"-to-12")} = 59 \times e^{[-0.12 \times wc_{opt}]}$	• *NDCP*: number of blow counts required for specific depth of penetration • wc_{opt}: 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*: 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
	$NDCP_{(6"-to-12")} = 27 \times e^{[-0.08 \times PI \times (F40/100)]}$		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

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 quality assurance and control 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. Most DOTs have started to move towards the use of performance based tests for quality assurance and control, in the sphere of which, the DCP stands as one of the most commonly used performance based test in the US for compaction quality control due to its ease of use and simple application process.

DCP was developed by A. J. Scala in 1950's and 1960's to evaluate the properties of flexible pavements by use of correlations between DCP and the CBR test results. Since then, the DCP has been used and developed by a number of state agencies inside and outside the US to check the compaction quality of compacted soils. Correlations have been developed and refined between DCP test results and CBR, M_R 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.

CHAPTER 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 the compacted soils to DCP tests is affected by soil type, density of the compacted soil, compaction water content and dominant 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's 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 i) the *in situ* dry density of the compacted soil, ii) the water content of the soil at the time of testing and iii) 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). Following the field tests, soil samples were collected from each of the locations where the DCP tests had been performed.

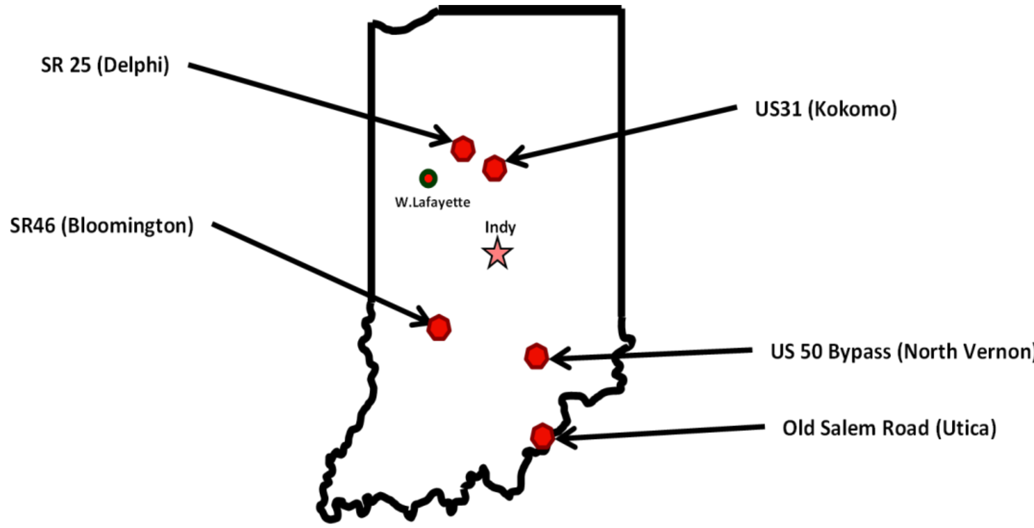


Figure 3.1 INDOT road construction sites were DCP tests were performed

The Dynamic Cone Penetrometer Test (DCPT) database used in this research was augmented using the DCPT data from a previous study (Kim et al. 2011). The entire DCPT blow count data was statistically analyzed; correlations were investigated between the main index properties of the subgrade soils 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 DCP test, the number of DCP blows 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. Table 3.1 shows an example of the filled data sheet used for recording DCP test data in the field.

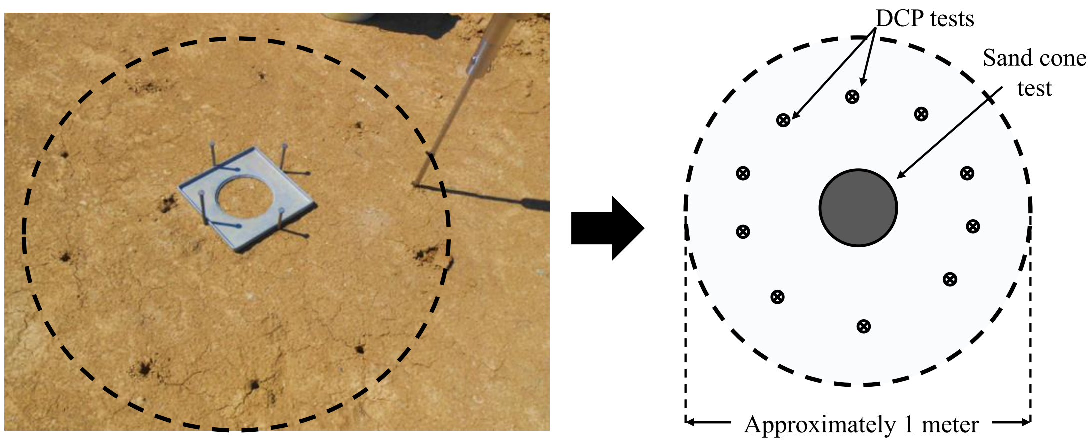


Figure 3.2 Spatial distribution of DCPTs and sand cone tests performed in one set in the field

Table 3.1 Field data collection sheet

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

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 3.3 shows a typical test sequence corresponding to four data sets.

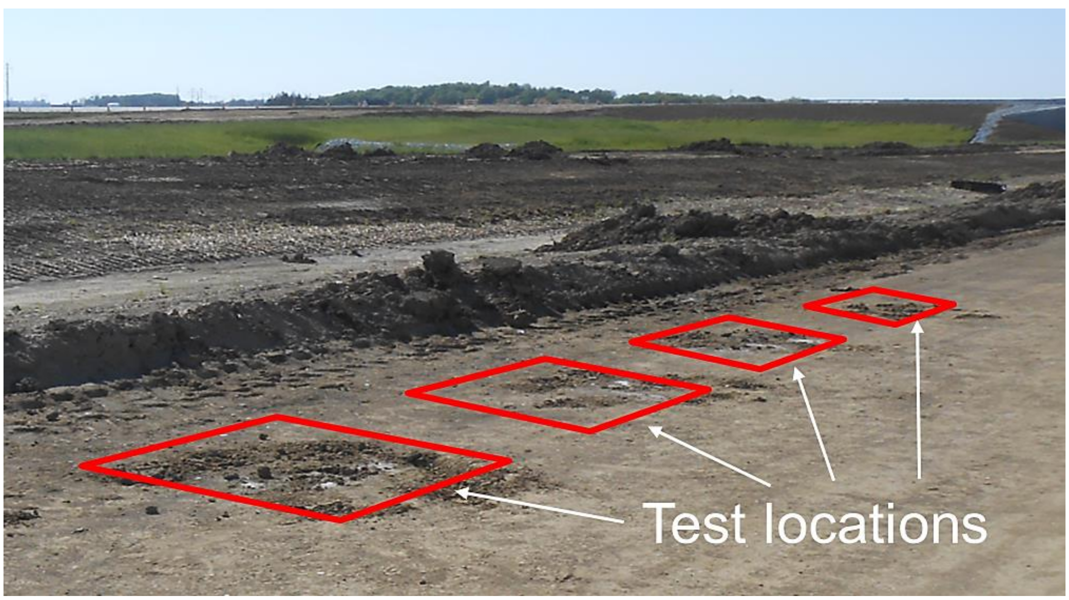


Figure 3.3 Typical test sequence corresponding to four data sets

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 will be described in chapter 4 and chapter 6) and then analyzed statistically. The procedure followed for the statistical analysis of the DCP data is explained in the next section.

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. 2011) 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 3.4 shows an idealized representative plot of results of 2 DCP test sets performed on the same type of soil compacted at different RCs. Plotted in Figure 3.4 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 standard deviation doesn’t change significantly. The magnitude of increase of the mean depends on the soil type.

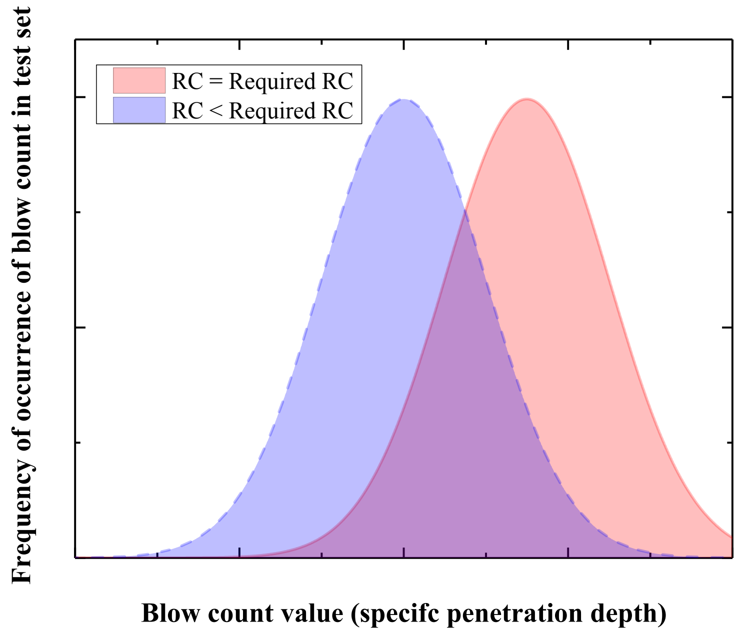


Figure 3.4 Distribution of DCP test results performed on similar soils compacted at different RCs

The statistical procedure was developed by (Kim et al. 2011) 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 done on subgrades compacted at RC lower than the required RC were not ignored and also taken into consideration during the analysis. 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 3.5 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 3.6 shows the frequency histogram of a test set performed on the same soils type as in Figure 3.5, 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 3.5.
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 3.6, 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 3.5 serves as an example for a test set with RC equal to the reference RC of 96%. It can be seen in Figure 3.5, 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 3.5 and Figure 3.6, *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. The extracted Blow Counts A and Blow Count B for individual test sets can be found in Table 5.1 and Table 5.2, for fine grained and coarse grained soils respectively.

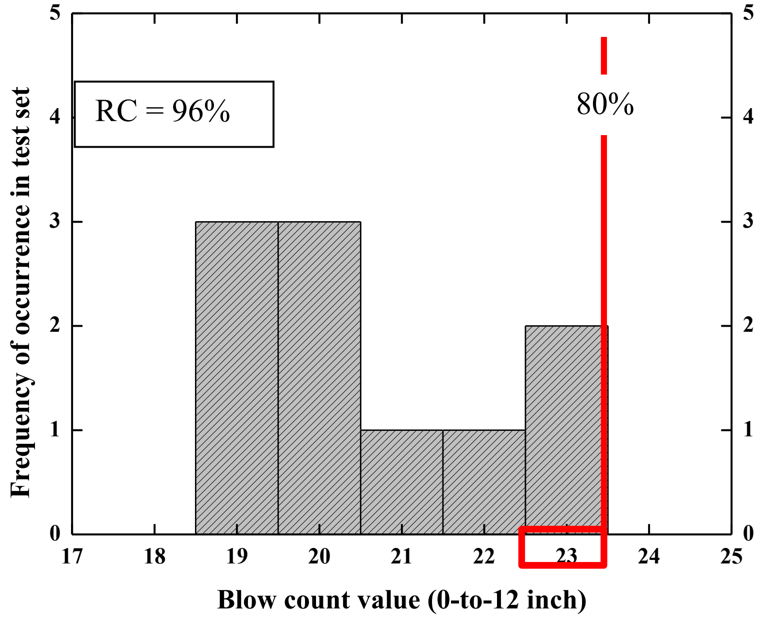


Figure 3.5 Frequency vs DCP blow count for 0-to-12 inches penetration into compacted soil (DCP-C-A1-1-DE-1)

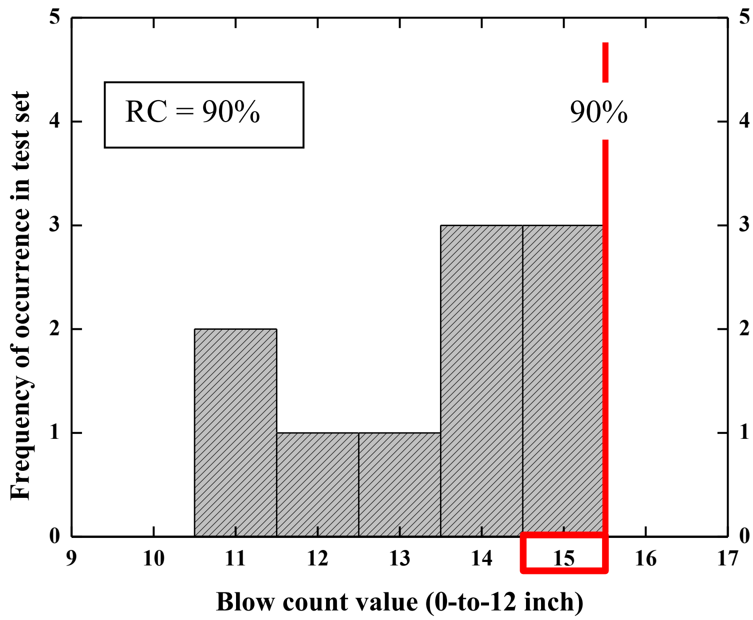


Figure 3.6 Frequency vs DCP blow count for 0-to-12 inches penetration into compacted soil (DCP-C-A1-1-DE-4)

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 DCP's impact loading, a large number of tests (more than 750 DCP tests) were performed on compacted embankments and subgrades in INDOT road construction sites across Indiana to augment the exisitng 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. 2011) 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.

CHAPTER 4. LABORATORY TEST RESULTS

In addition to the data collected in previous research studies, more than 750 DCP tests were performed on constructed subgrade and embankments in 5 major INDOT road-work projects. Soil samples were collected from the location of each test set to obtain the index properties of the soil. Laboratory tests performed on the 76 soil samples collected included: i) grain size distribution (sieve and hydrometer analysis), ii) soil plasticity (liquid limit and plastic limit), iii) standard Proctor compaction test and iv) 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.

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

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 76 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 Optimum moisture content. The individual soils types were identified as follows:

"*Test*"-"*Fine/Coarse*"- "*AASHTO classification*"- "*Subgroup*"

where, *Test* = Dynamic Cone Penetration (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.

"*Test*"-"*Fine/Coarse*"- "*AASHTO classification*"- "*Location code*"- "*Set number*"

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.

For example, consider a soil which has fines content of about 70-80%, PI of 12-14%, standard proctor optimum moisture content of 15% and proctor maximum dry density of 18.6kN/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 4.2 Soil grouping criteria

Soils group (F or C)		OMC (%)	MDD		PI (%)	Percentage passing #200 sieve (0.075 mm)
			pcf	kN/m^3		
Coarse grained (C)		<12	> 120	> 18.9	< 5, Non plastic	≤ 20
Transitional	Coarse (TC)	[12-15)	(110-120]	(17.3-18.9]	≤ 8-10	< 60
	Fine (TF)				≥ 8-10	≥ 60
Fine grained (F)		≥ 15	≤ 110	≤ 17.3	≥ 5-10	≥ 60

Table 4.3 Location codes

Location	Code
Delphi, SR 25	DE
Kokomo, SR31	KO
North Vernon, US 50 Bypass	NV
Bloomington, SR 46	BL
Utica, Old Salem Road	UT

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 (Skempton 1953; Pandian & Nagaraj 1990) 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 than50%.

Table 4.4 Combined test results for fine-grained soils

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

Table 4.5 Combined test results of coarse-grained soils

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

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 and Bloomington and Figure 4.4 presents the representative grain size distribution of the soils tested in Delphi.

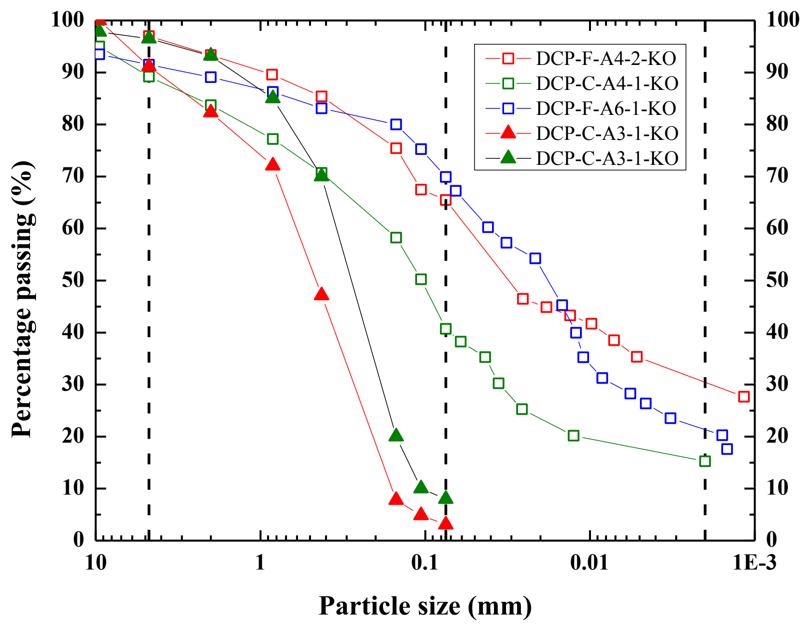


Figure 4.1 Grain-size distribution of soils in Kokomo

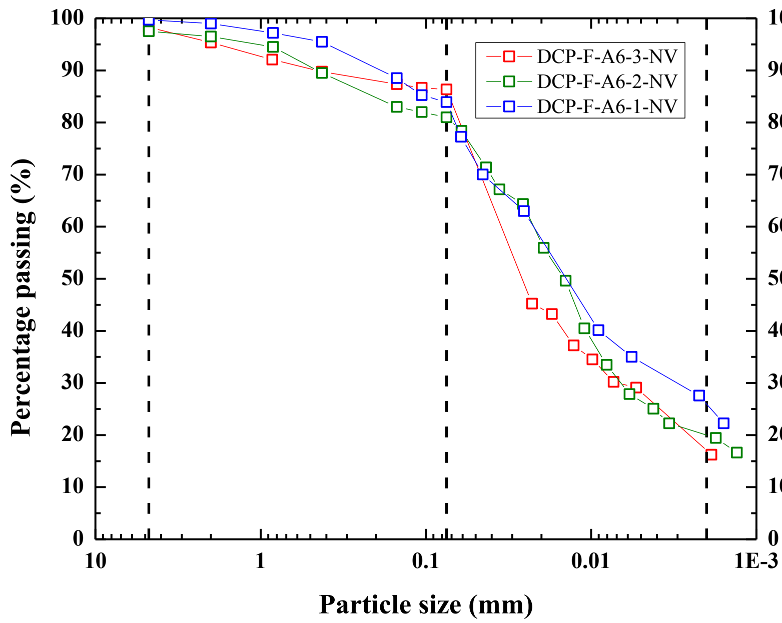


Figure 4.2 Grain-size distribution of soils in North Vernon

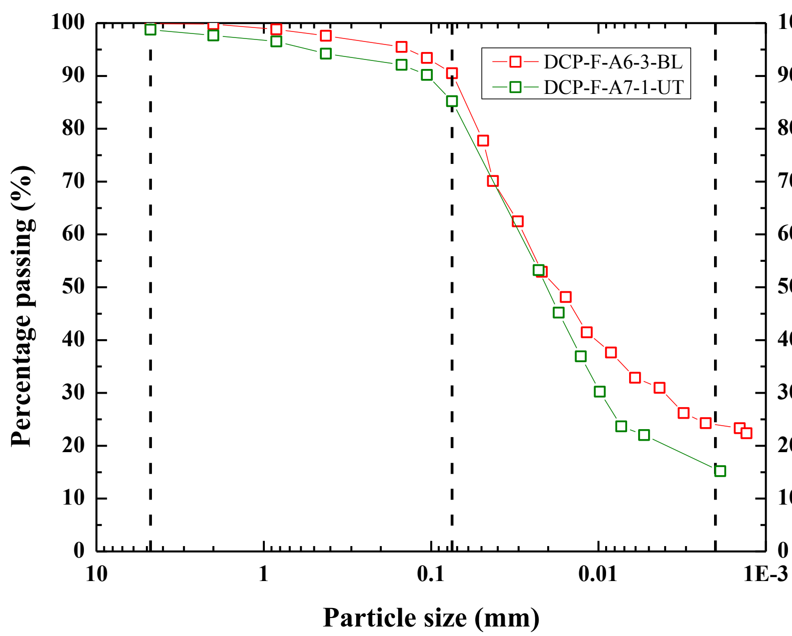


Figure 4.3 Grain-size distribution of soils in Bloomington and Utica

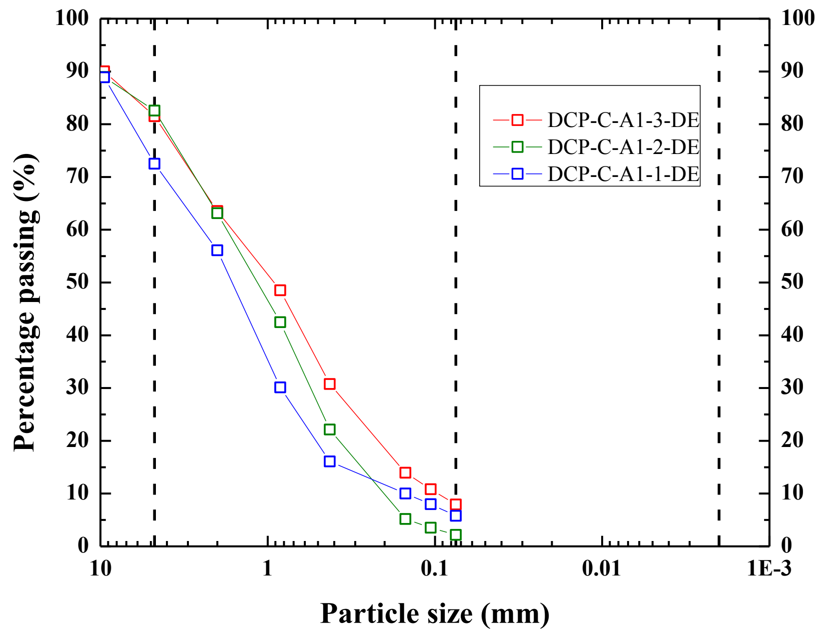


Figure 4.4 Grain-size distribution of soils in Delphi

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 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 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.

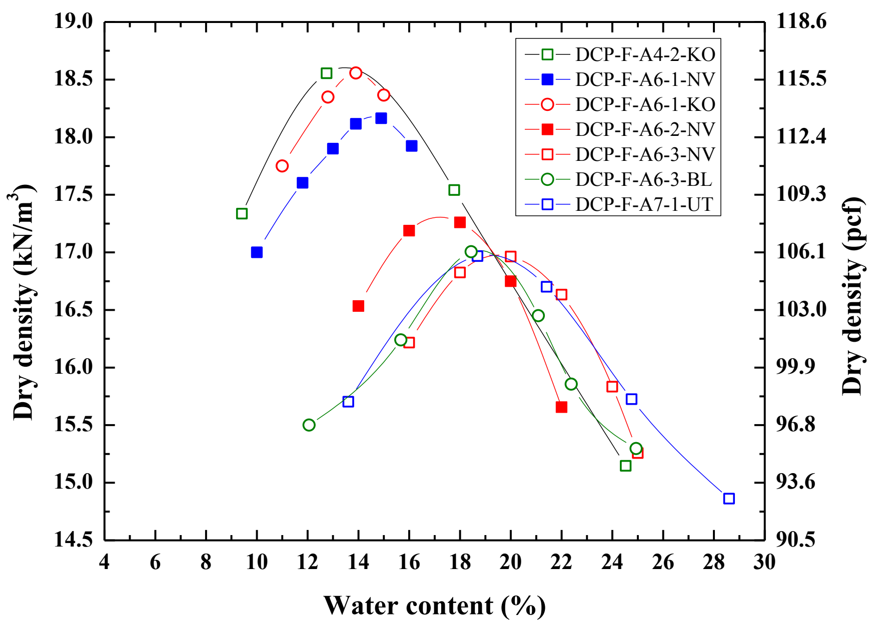


Figure 4.5 Compaction curves for fine-grained soils

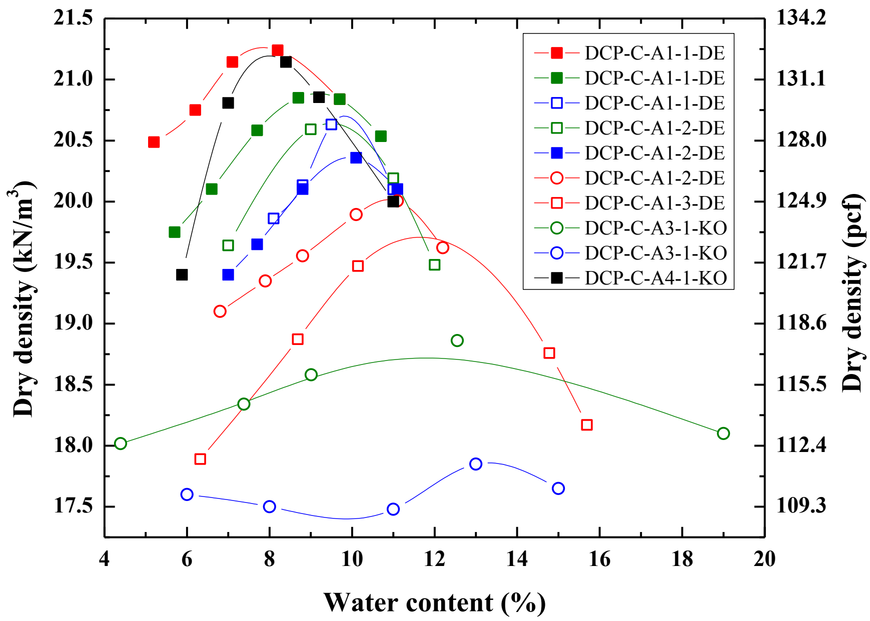


Figure 4.6 Compaction curves for coarse-grained soils

4.6 Summary

More than 750 DCP tests were performed on constructed subgrade and embankments in 5 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.

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. Each of the 75 soils 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 optimum moisture content. 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 20%, fines content above 65-70%, standard Proctor maximum dry density in the 16.5-18.5 kN/m^3 range and optimum moisture content 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 optimum moisture content was observed to be in the range of 8-13%.

CHAPTER 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. more than 750 DCP tests were performed in sets of 10 tests, thereby giving us a total of about 75 test sets. For each test set, along with the 10 DCP tests, a sand cone test and an *in situ* water content measurement was 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). This, 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 the results obtained from field tests performed on coarse-grained soils.

5.1 Fine-grained soils

Out of the total of 75 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 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 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 the 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 their OMC.

Table 5.1 Field test results fine-grained soils

Test set ID	**Field test date**	**MDD[#]**		**OMC[*] (%)**	**PI (%)**	**0-to-6 inch**		**6-to-12 inch**		**RC (%)**	**wc (%)**	**Δwc[*] (%)**
		pcf	**kN/m²**			**80%**	**90%**	**80%**	**90%**			
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

[#] MDD: Maximum dry density obtained from the Standard Proctor Compaction Test ASTM D698

[*] OMC: Optimum moisture content obtained from the Standard Proctor Compaction Test ASTM D698

Table 5.1 Field test results fine-grained soils

Test set ID	Field test date	MDD[#]		OMC[*] (%)	PI (%)	0-to-6 inch		6-to-12 inch		RC (%)	wc (%)	Δwc[*] (%)
		pcf	kN/m^2			80%	90%	80%	90%			
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

[#] MDD: Maximum dry density obtained from the Standard Proctor Compaction Test ASTM D698

[*] OMC: Optimum moisture content obtained from the Standard Proctor Compaction Test ASTM D698

5.2 Coarse-grained soil

A total of 37 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 the table, 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.

Table 5.2 Field test results for coarse-grained soils

Test set ID	Field test date	MDD#		OMC* (%)	0-to-12 inch		RC (%)	wc (%)	Δwc (%)
		pcf	kN/m²		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#	20#	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#	28#	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#	17#	97.5	9.5	-1.5
DCP-C-A1-2-DE-10					18	18	100.0	9.6	-1.4

MDD: Maximum dry density obtained from the Standard Proctor Compaction Test ASTM D698
* 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[#]		OMC[*] (%)	0-to-12 inch		RC (%)	wc (%)	Δwc (%)
		pcf	kN/m^2		80%	90%			
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

[#] MDD: Maximum dry density obtained from the Standard Proctor Compaction Test ASTM D698

[*] OMC: Optimum moisture content obtained from the Standard Proctor Compaction Test ASTM D698

5.3 Summary

Out of the total of 75 test sets, 38 test sets were performed on fine-grained soils and 37 test sets were performed on coarse-grained soils. DCP blow counts were recorded for 0-to-6 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 with the DCP blow counts required for specific depth of penetration of the DCP into the subgrade/embankment, different depth were chosen for the two different soils. 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.

CHAPTER 6. DEVELOPMENT OF QA/QC CORRELATIONS AND FIELD APPLICATION

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

To develop meaningful QA/QC correlations between the statistically extracted DCP blow counts and soil index properties, 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 by three main factors: i) fabric and ii) moisture content and iii) 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 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 & 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-size particles in the soil mass is small enough such that the larger, sand-size particles are on average in contact with each other, then the behavior of the soil will be dominated by the properties of the sand-size, particles (refer to Figure 6.1 (a)). While on the other hand, if the clay-size 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)). 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 much less than 50%, as we shall next.

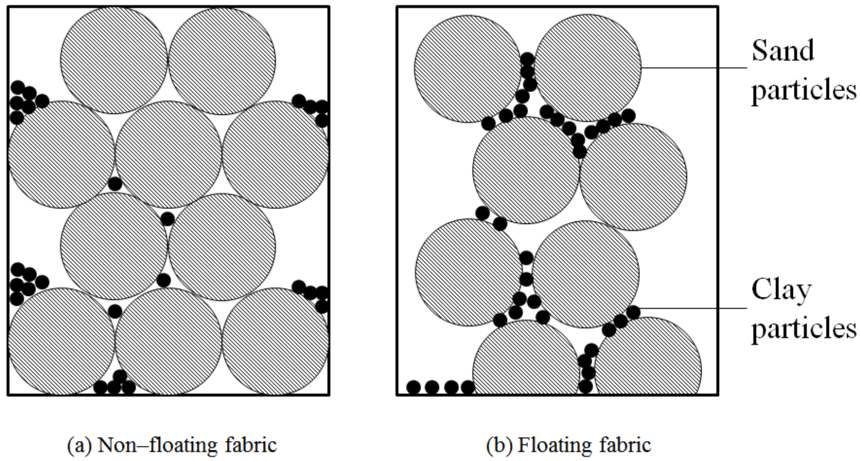


Figure 6.1 Floating fabric

Research was carried out on the effect of non-plastic fines (Salgado et al. 2000) and of plastic and non-plastic fines (Carraro et al. 2009) on the mechanical response of sands and 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 me more varied, but to simplify the analysis, we make such an assumption.

In order to have a floating fabric, the void volume in the coarse-grained phase V_V 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 in the coarse-grained phase V_V 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 (inter-granular 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 *Gs*, the volume of clay V_C and the volume of coarse-grained phase V_G can be obtained:

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

For the clay phase:

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

For the coarse-grained phase:

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

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

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

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

Substituting equation 6.11 in equation 6.2, we get:

$$V_V = e_G \frac{W_s}{Gs_G \gamma_w}\left(1 - \frac{C}{100}\right) \tag{6.12}$$

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

$$V_W = wc \frac{W_s}{100\gamma_w} \tag{6.13}$$

Now, substituting 6.10, 6.12 and 6.13 in to 6.1:

$$e_G \frac{W_s}{Gs_G \gamma_w}\left(1 - \frac{C}{100}\right) = wc \frac{W_s}{100\gamma_w} + \left(\frac{W_s}{Gs_C \gamma_w}\right)\frac{C}{100} \tag{6.14}$$

Solving for C, we obtain:

$$C = \left(\frac{100 \times e_G - Gs_G \times wc}{Gs_G + Gs_C \times e_G} \right) \times Gs_C \tag{6.15}$$

In order to estimate the clay content required for the development of a floating fabric in the soil mass, the intergranular 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: i) there are only two phases in the soil mass, ii) only two particle sizes exist in the soil mass and iii) the degree of saturation is equal to 1. In natural soils, often there are other particle sizes, and in compacted soils the degree of saturation is not 1, but lies generally 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, to get the inter-granular void ratio of the soil mass, we need to have an idea of the *in situ* dry density of the soil mass. The *in situ* global void ratio e (=Volume of voids / Volume of solids) can be calculated using:

$$e = \frac{Gs \times \gamma_w}{\gamma_d} - 1 \tag{6.16}$$

Using (6.16) and assuming a *Gs* value of 2.65, we can estimate the global void ratio, from which we can then estimate e_G (intergranular 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 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 Optimum Water Content (OWC) 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

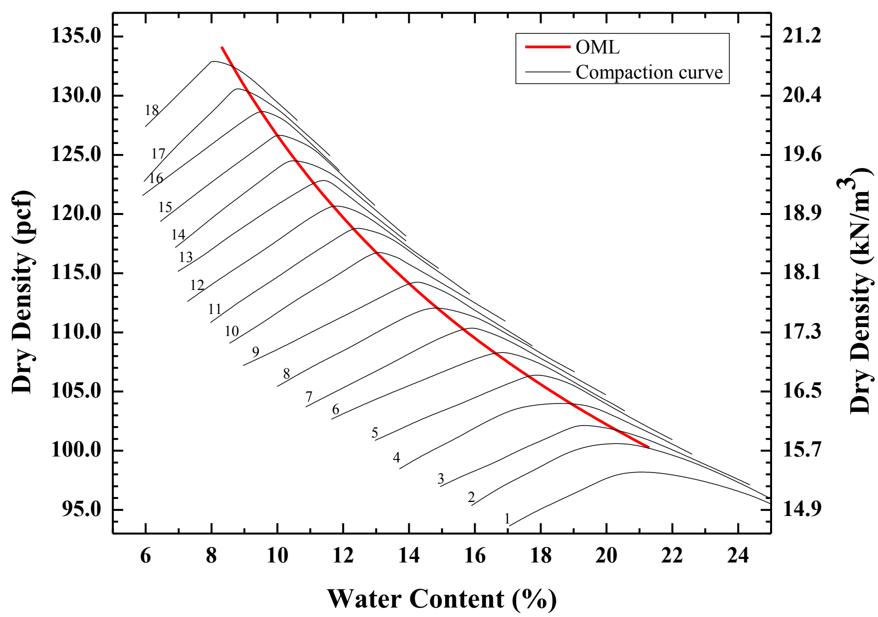


Figure 6.2 Family of standard Proctor compaction curves (INDOT family of curves modified after Kim et al 2010)

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 start to develop a floating fabric would be to observe the *in situ* compaction densities at which the soil response to DCP loading starts to change. A review of the literature (Holtz & Kovacs

1981; Rodriguez et al. 1988) and personal communications between INDOT engineers indicates that this range of compaction density is associated with soils having maximum dry density around 110 to 120 pcf and OMC of about 12 to 15 %. This range of maximum dry density and optimum water content is associated with transitional soils, sandy loams and clayey silts, i.e., as soils transition from coarse-grained (sand like) behavior to fine-grained (clay like) behavior. Therefore, a glimpse of the answer to the question "What clay content does a floating fabric 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; therefore the intergranular void ratio must be higher. For sands in their loosest state, the void ratio is around 0.9. Based on the above values, we can reasonably assume that the intergranular 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 intergranular void ratios can be plotted, as shown in Figure 6.3 below.

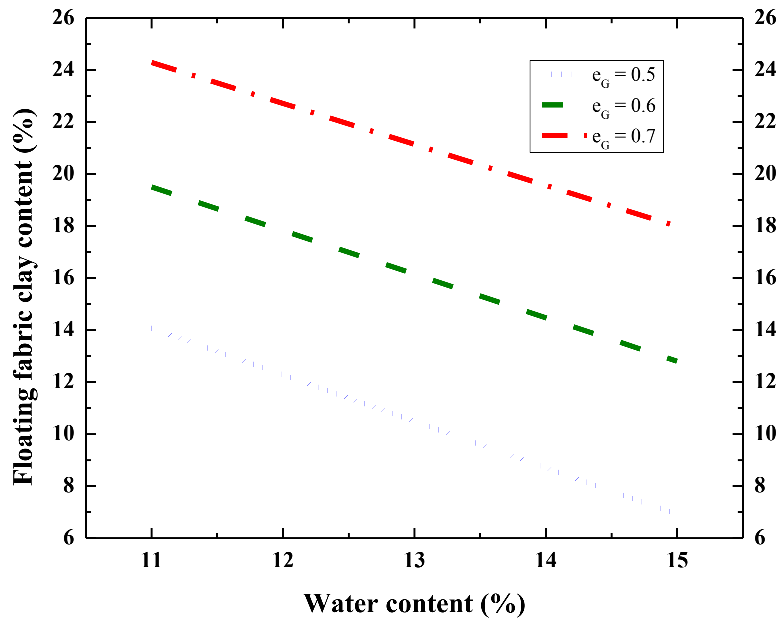


Figure 6.3 Clay content required for the development of floating fabric with respect to the water content and intergranular void ratio of the soil.

From the above results of water content and intergranular void ratio, it can be seen that for the soils transitioning from sand-like to clay-like behavior, the clay content at which a floating fabric can develop in the soil mass is about 15-20%. Therefore, as the clay content in the compacted soils goes beyond 15-20%, the behavior of the soil changes from sand-like to clay-like. Similar percentage values were obtained from test performed on mixtures of sands with non-plastic fines, indicating that a floating fabric stars to form in the soil mass when the percentage by weight of non-plastic fines reaches about 20% (Salgado et al. 2000).

In order to determine the clay content of a soil compacted in the field, hydrometer tests would need to be performed (this is not typically done by DOTs). Nonetheless, a rough estimate of the clay content required for a soil to develop a floating fabric can be obtained from its PI. 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 of the soils that were tested for this research (Skempton 1953; Pandian & Nagaraj 1990). Figure 6.4 shows a plot of PI vs. clay content for various values of activity together with the data obtained for the soils considered in this research (the locations in the state of Indiana at which the PIs and clay contents were determined are also shown in Figure 6.4). 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 and knowing that 15% to 20% of clay can cause the soil to change behavior from sand-like to clay-like soils, 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-like to clay-like behavior.

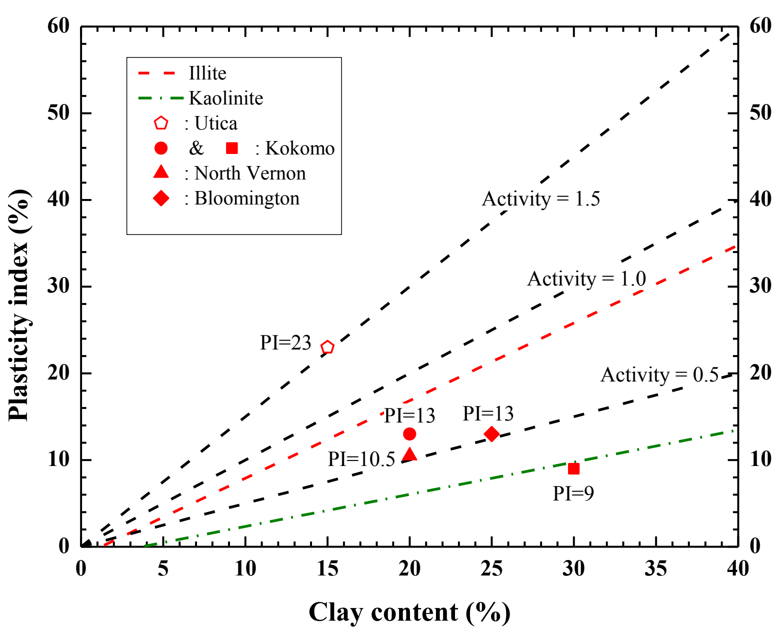


Figure 6.4 Plasticity index vs. clay content of soils in Indiana (After Skempton 1953)

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 fine-grained soils, the state parameter called matric suction (difference of pore air and pore water pressures) plays an important role in the mechanical response of the soil (Fredlund & Rahardjo 1993; Blight 2013). As the degree of saturation and water content of the soil mass increases, the 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. 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-like) 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-like) soils.

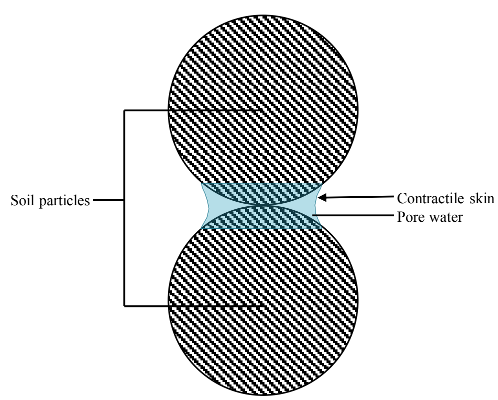


Figure 6.5 Formation of contractile skin in unsaturated soils

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 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 filter paper method) 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.

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

ID	Sample location in Indiana	Plasticity Index, %			Silt %	Clay %	MDD kN/m^3	OMC%	G_s	USCS	AASHTO
		PL	LL	PI							
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

Authors	Soil name	Plasticity Index, %			Silt %	Clay %	γ_{dmax}, kN/m^3	wc_{opt},%	USCS
		PL	LL	PI					
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

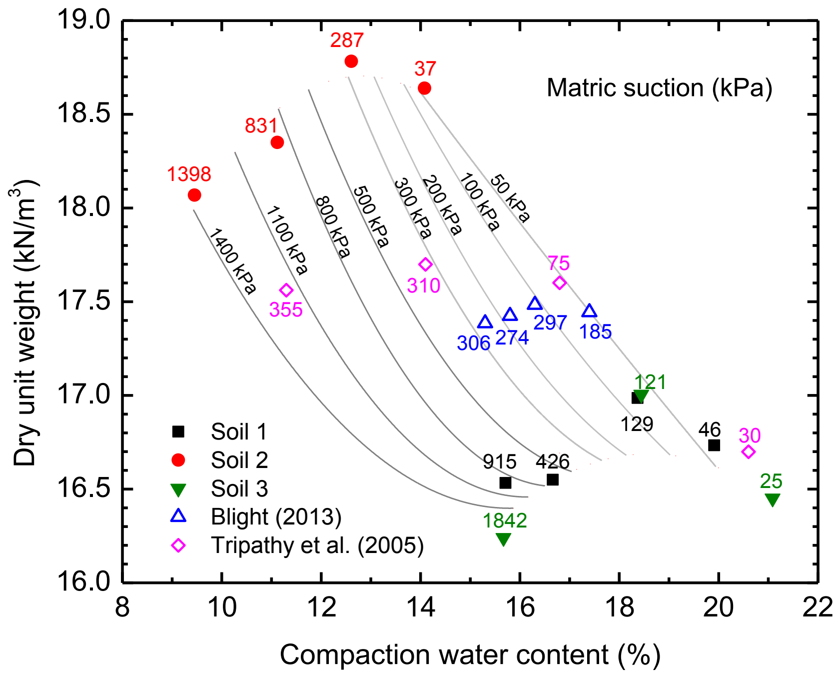


Figure 6.6 Variation of matric suction at different compaction states for fine-grained soils (Kim et al. 2014)

The soils whose blow counts are sensitive to moisture content variation are the most important ones. As described before, fine-grained soils are the most important ones in regard to moisture sensitivity. To ensure that lower blow counts were not recorded due to higher moisture in the field, the 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.

6.2 Grouping of soils

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

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

The 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 the discussion above, soils with maximum standard Proctor dry density between 110-120 pcf (16.5-18 kN/m^3) and optimum moisture content 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, 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 developed are used to obtain their blow count values. Note also that certain manufactured soils, such as those used by INDOT as backfill for bridges, are also categorized as coarse-grained soils, but their compaction characteristics fall below the optimum moisture line in the transitional zone.

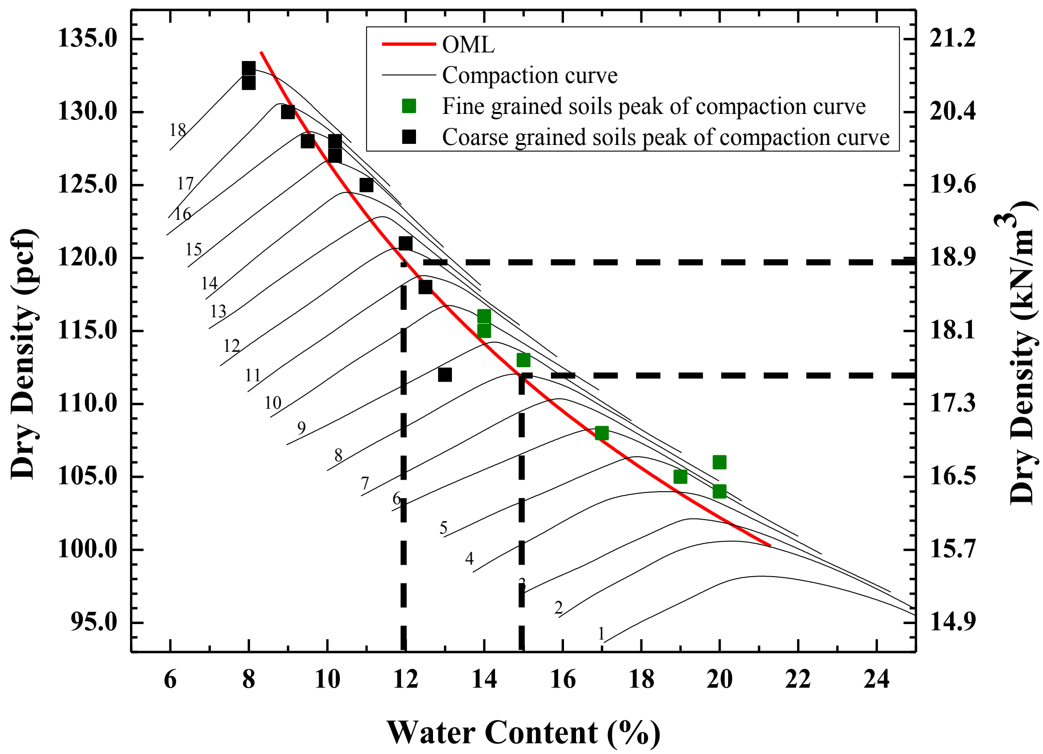


Figure 6.7 Soil behavior and compaction properties (INDOT family of curves)

Similar boundaries based on PI, as seen in Table 6.3, have also been observed by White et al. (2002) who made the demarcation between fine and coarse-grained soils (termed by these authors as "cohesive" and "non-cohesive soils") using the percentage passing the No. 200 (75µm) sieve and the plasticity index of the soils estimated from the percentage passing the No. 40 (425µm) sieve. 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 weight were classified as "cohesionless" 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 have a liquid limit greater than 40% and a plasticity index greater than 10%, the “intergrade” soil is 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.

Therefore, 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 comprises 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 could also be 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 judgment accordingly.

Table 6.3 Soil grouping

Soils group (C/F)		OMC (%)	MDD		PI	Percentage passing #200 sieve (0.075 mm)
			Pcf	kN/m^3		
Sand-like or coarse-grained (C) soil		<12	> 120	> 18.9	Non plastic	$\leq$ 25
Transitional (classified as sand-like or clay-like soils)	Transitional sand-like (TC)# soil	[12-15)	(110-120]	(17.3-18.9]	$\leq$ 8-10	< 60
	Transitional clay-like (TF)# soil				$\geq$ 8-10	$\geq$ 60
Clay-like or fine-grained (F) soil		$\geq$ 15	$\leq$ 110	$\leq$ 17.3	$\geq$ 5-10	$\geq$ 60

#Note: Transitional soils are classified as either sand-like (TC) or clay-like (TF) soils for use of the DPC blow count correlations. For TC soils the correlations developed for coarse grained soils should be used and for TF soils correlations developed for fine grained soils should be used.

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-like soil) and fine-grained (clay-like soil) soil groups. The following sections presents the developed DCP blow count correlations for compaction quality control.

6.3.1 Coarse-grained soils (sand-like soils)

For sand-like 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 inch 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 optimum moisture content of the soil.

A higher optimum moisture content implies a lower compaction density, 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 optimum moisture content of the soil obtained from Proctor tests performed in the laboratory. At each of the points plotted, the *in situ* moisture content at the time of testing is plotted next to the data point as Δwc, with a positive value implying that the *in situ* water content was above the OMC at the time of DCP testing, and a negative value implying 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 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. As long as the targeted dry density is reached in the field, the blow counts are not significantly affected by a small variation of *in situ* moisture content at the time of testing.

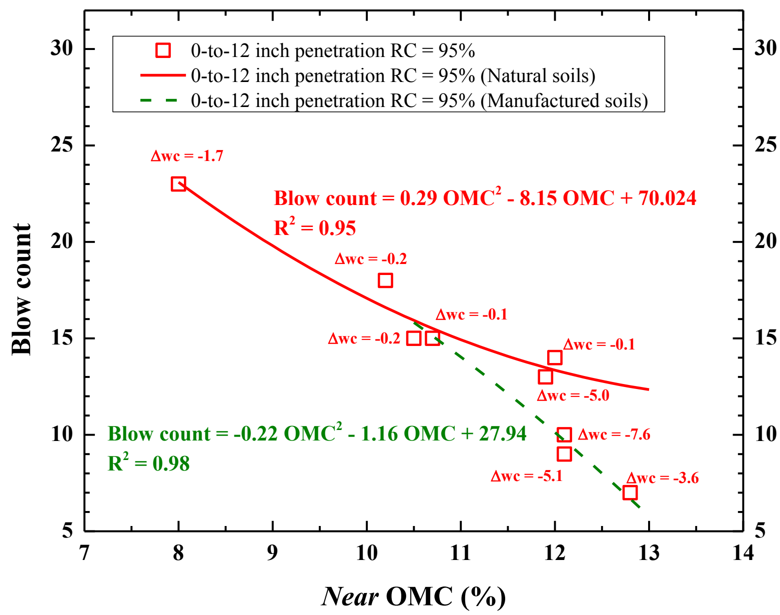


Figure 6.8 Blow count for 0-to-12 inch penetration into compacted soil vs. near optimum moisture content of compacted soil

The results 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 top axes). The optimum moisture line for coarse-grained soils (the solid blue line) is also shown in this figure. 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 optimum moisture content) or the one point proctor data from the field.

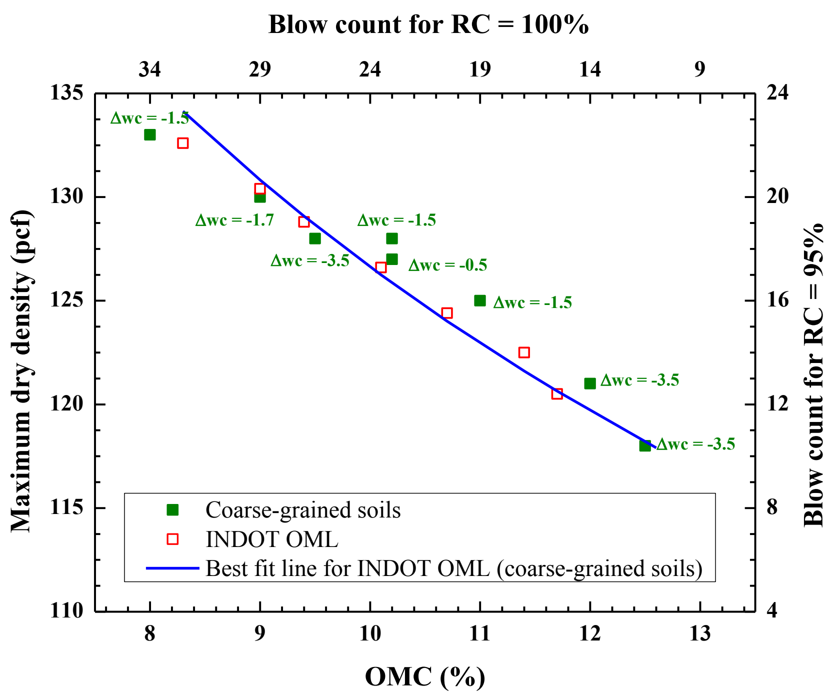


Figure 6.9 Four-way blow count chart for 100% and 95% RC

It was furthermore observed that 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%), 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.

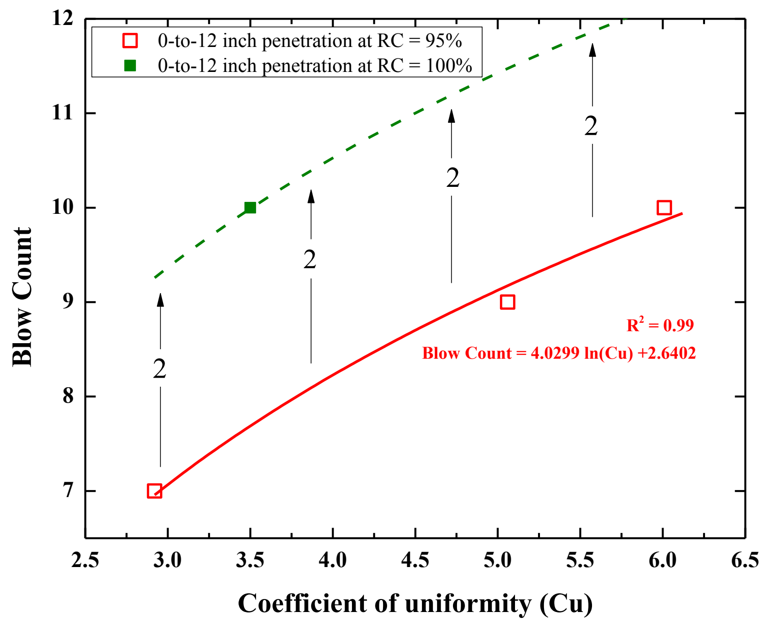


Figure 6.10 DCP blow count correlations for clean sands

6.3.2 Fine-grained soils (clay-like soils)

QA/QC correlations were developed for clay-like soil groups for blow counts required for 0-6 and 6-12 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.

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

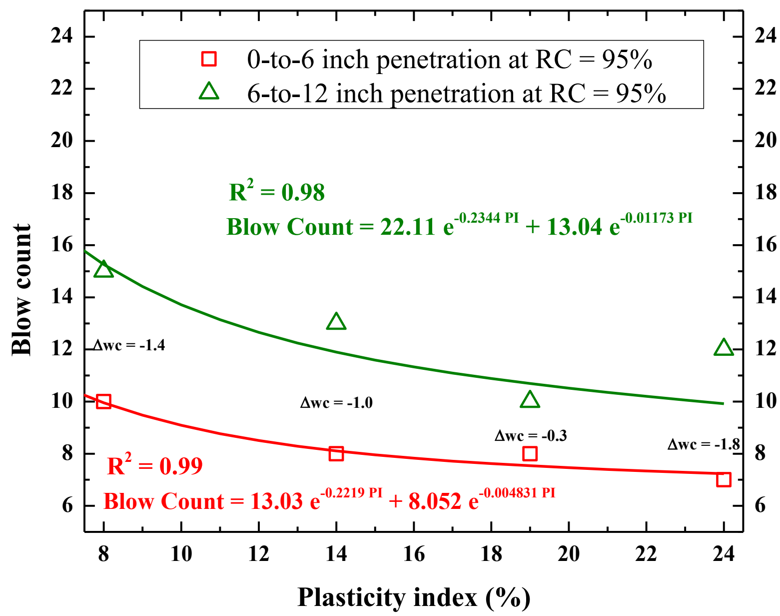


Figure 6.11 Blow count correlations for fine-grained soils

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.

6.3.3 Combined correlations

Table 6.4 provides the DCP blow count correlations developed for coarse-grained soils and fine-grained soils.

Table 6.4 DCPT correlations

Soil type		Correlations	R^2	Penetration Depth (inches)	Range of applicability
Coarse- grained soils	Natural	$\text{Blow Count} = 0.29 \times OMC^2 - 8.15 \times OMC + 70$	0.95	0-to-12	8<OMC%<13
	Manufactured	$\text{Blow Count} = 4.03 \times \ln(Cu) + 2.64$	0.99	0-to-12	3.0<Cu<6.0
Fine-grained soils		$\text{Blow Count} = 13.03 \times e^{-0.23 \times PI} + 8.05 \times \mathrm{e}^{-0.005 \times PI}$	0.99	0-to-6	5 < PI%
		$\text{Blow Count} = 22.11 \times e^{-0.23 \times PI} + 13.04 \times e^{-0.012 \times PI}$	0.98	6-to-12	

6.4 Summary

To develop QA/QC correlations for the DCP, the soils tested were categorized in to 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. In mixed soils, such as those found in nature, an estimate of the percentage by weight 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-like) soils.

After proper soil classification, correlations for either coarse-grained or fine-grained soils can then be 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 optimum moisture content obtained from the 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 OMC and C_u.

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 could be used as an indication 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. All correlations developed are presented in Table 6.4.

CHAPTER 7. FIELD APPLICATIONS

Knowledge of the DCP blow count criteria from the correlations is not enough for proper compaction quality control in to the field. It is necessary to know i) how many tests (comprising one test set) need to be performed in the field at a given location to be reasonably sure that the average blow count is representative of the soil compaction in that location (referred to as small-scale testing frequency) and ii) how many test sets need to be performed along the length of the final constructed subgrade to assess the compaction quality and associated variability (referred to as large-scale testing frequency). This needs an understanding of the variability and distribution associated with the DCP test results.

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 locations shown in Appendix A and B). This implies that at each location where a test set was performed, for which there is an associated RC value, multiple values of blow counts can be obtained. This means that 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 the 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 variability in DCP blow counts 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. 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 (i) 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 (ii) 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 performing 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 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 (for 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.

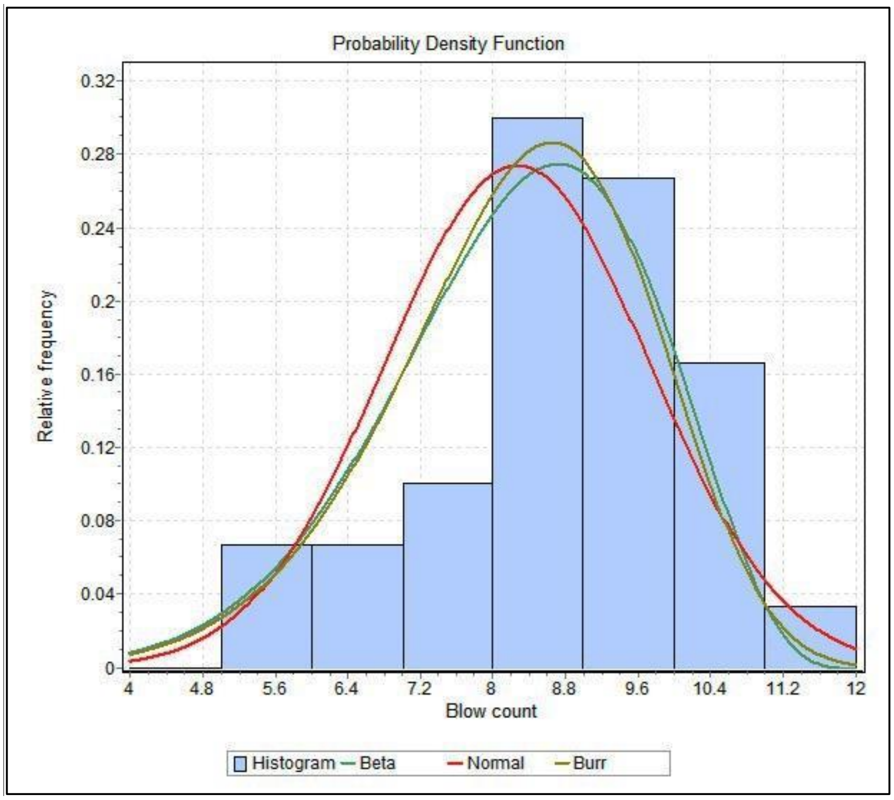
7.1.1.1 Grouping of test sets

Firstly, the data of the grouped test sets were analyzed. Soils were grouped according to their compaction characteristics and index properties and their *in situ* compaction condition. Table 7.1 shows the properties of the soils grouped together for the purpose of the statistical analysis.

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

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 6.12 and Figure 6.13 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 6.14 and Figure 6.15 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 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 6.5 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 Table6.5). 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.



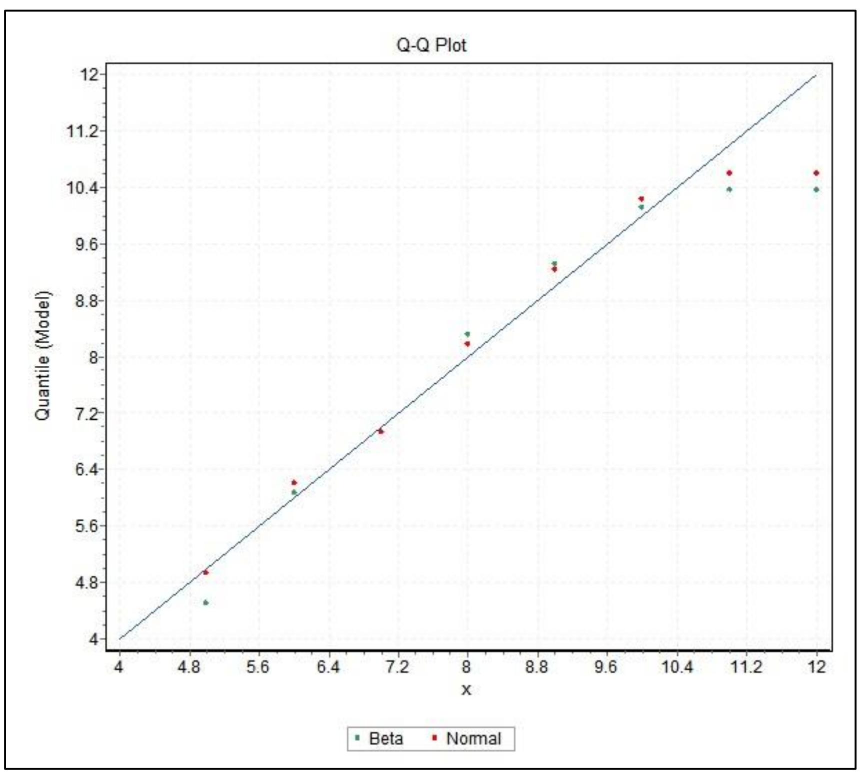
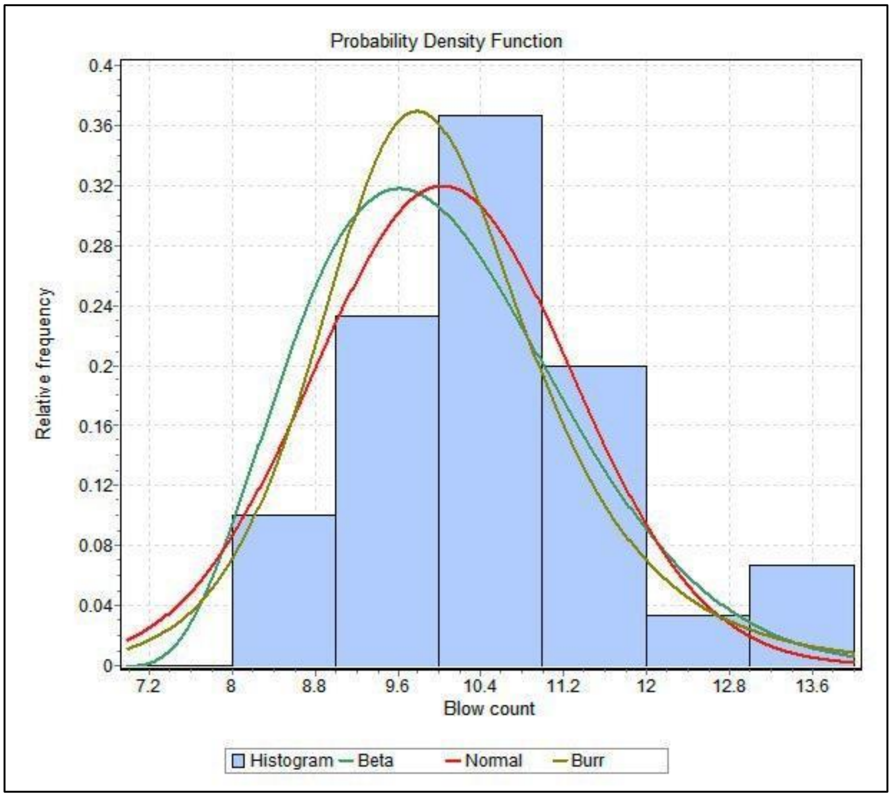


Figure 6.12 Group A: DCP-C-A3-1-KO-(1, 2, 6)



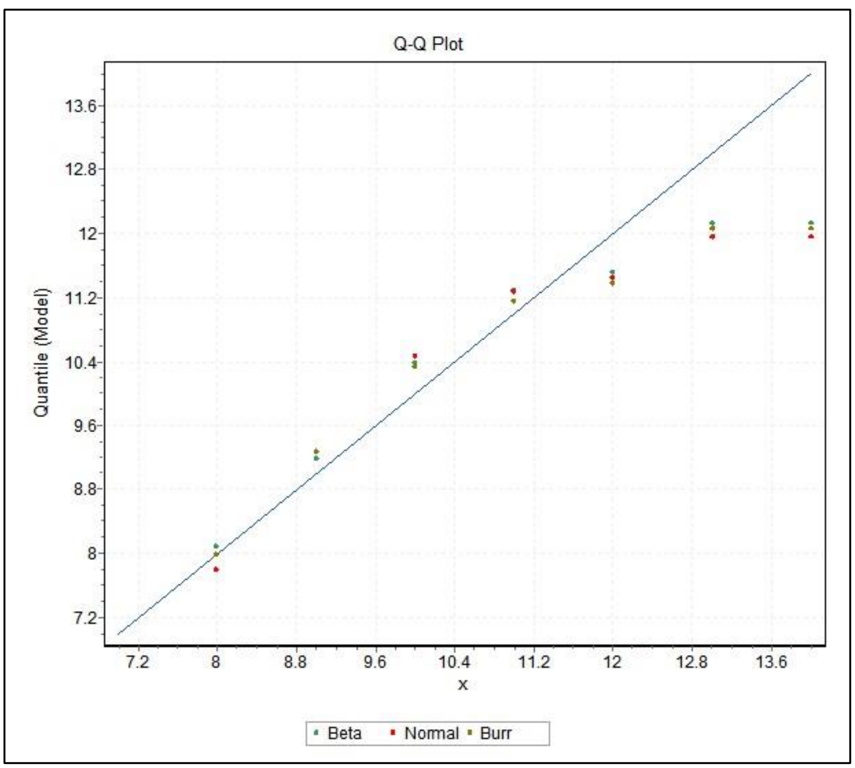
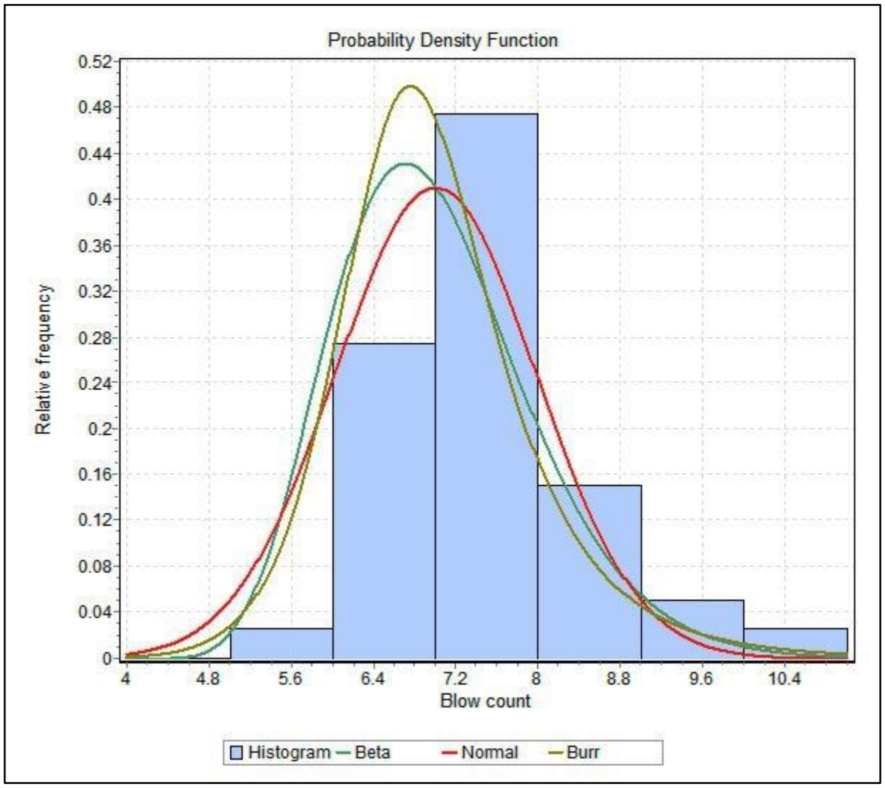


Figure 6.13 Group B: DCP-C-A3-1-KO-(4, 5, 7)



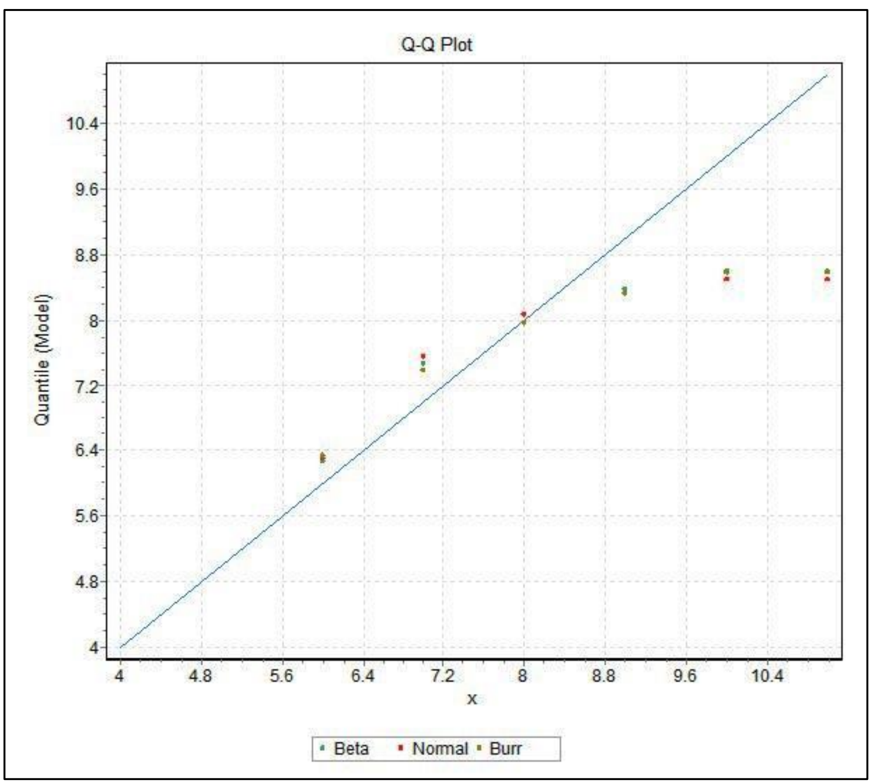
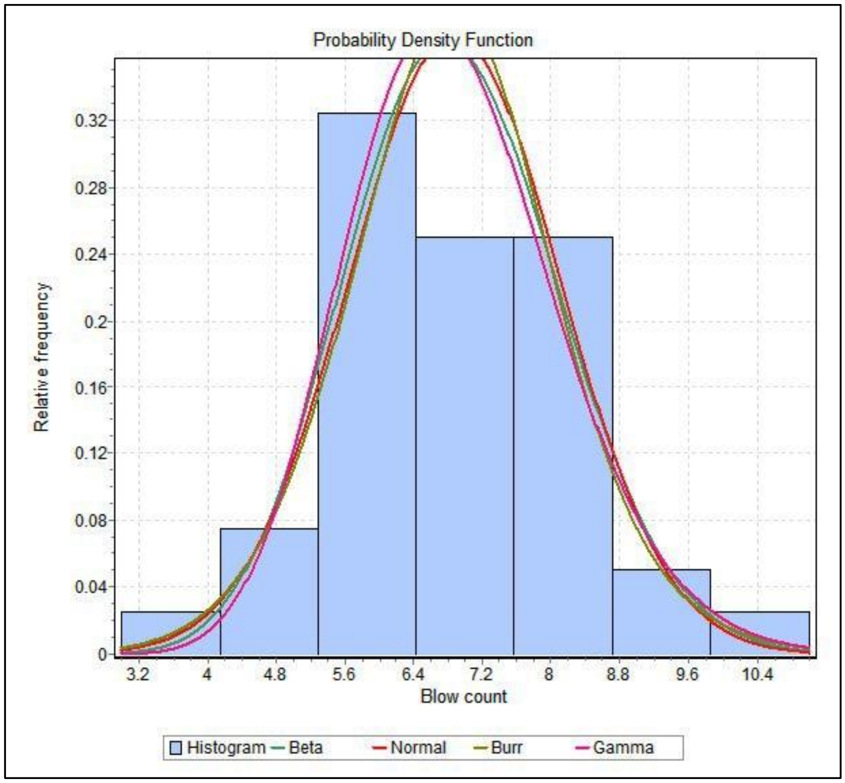


Figure 6.14 Group C: DCP-F-A6-3-BL-(1, 2, 3, 4)



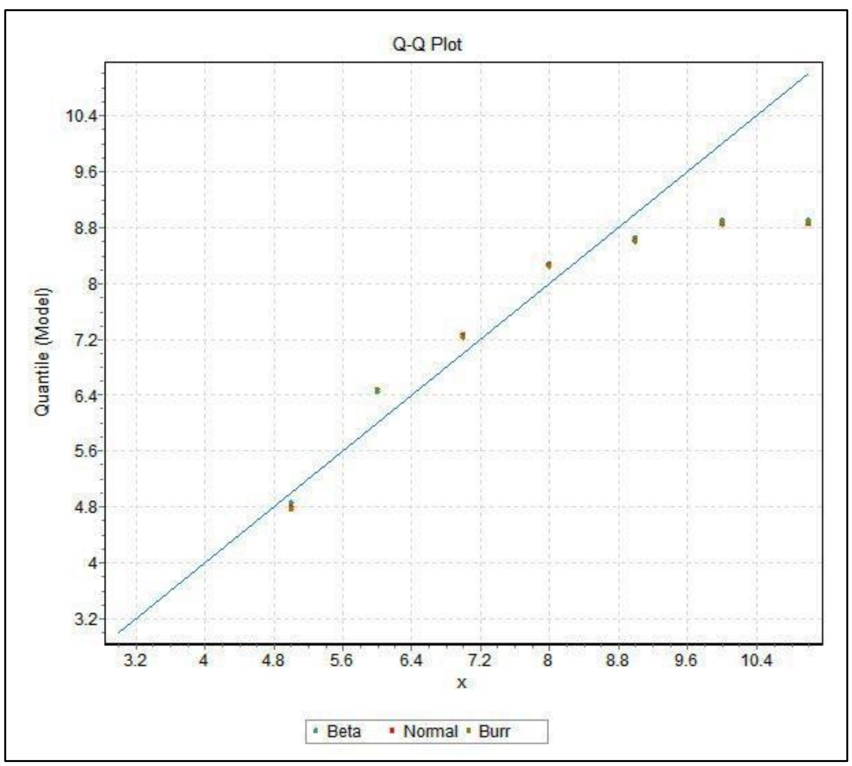


Figure 6.15 Group D: DCP-F-A6-2-NV-(1, 2, 3, 4)

Table 6.5 Comparison of parameters of Beta and Normal distributions

Test ID	**Beta Distribution**					**Normal Distribution**		
	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 statistics
a, b: shape parameters of Beta distribution
L, U: Lower and upper bound of Beta distribution
σ: standard deviation
μ: mean

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 objective behind grid testing was to: i) limit the area in which the DCP tests were performed and ii) 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 tests were performed at a center-to center-spacing of at least 1 foot (≃30 cm or approximately 15 DCP cone tip diameters). This distance was chosen so as to reduce the probability of adjacent tests affecting each other. Figure 7.1 shows the schematic and the accompanying picture from the field of the grid testing.

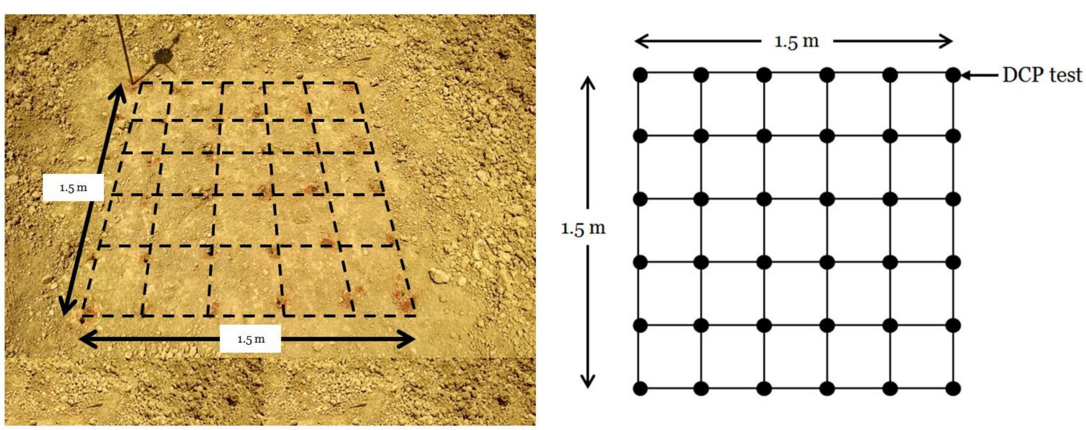


Figure 7.1 Grid testing in field and schematic

To assess the *in situ* density of the compacted soil, 4 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 7.2; 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 7.3 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 suggest a good fit for both of the distributions. Table 7.2 shows the distribution fitting parameters for the DCP grid testing.

Table 7.2 Distribution fitting parameters for the field grid testing

Test ID	**Beta Distribution**					**Normal Distribution**		
	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

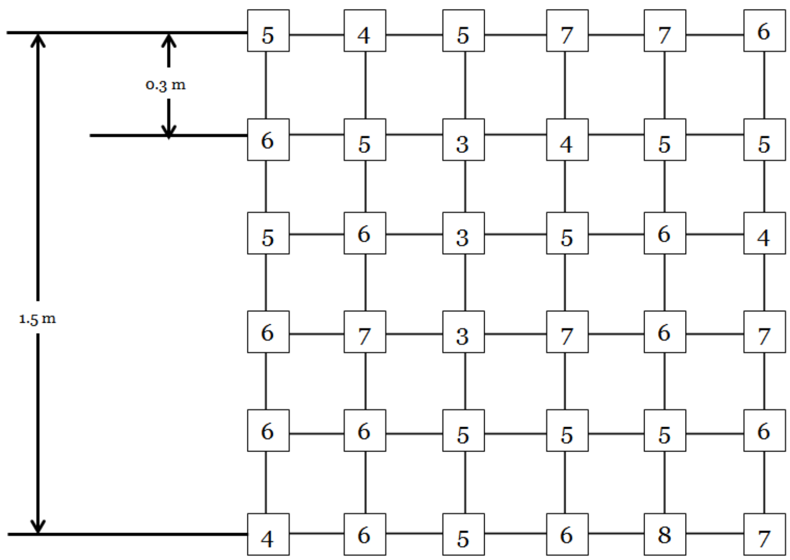
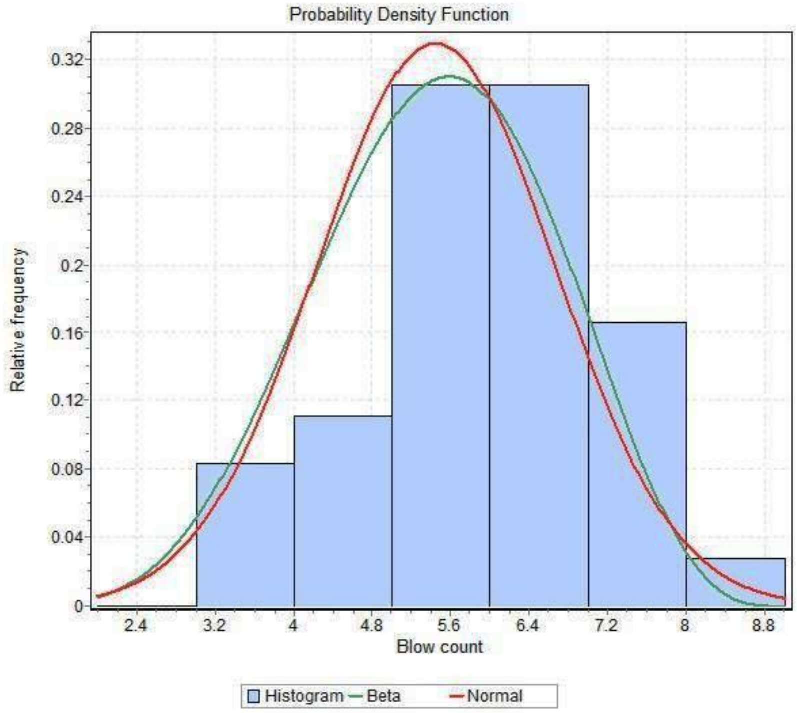


Figure 7.2 DCP blow count for 6 inch penetration at specific locations on the grid



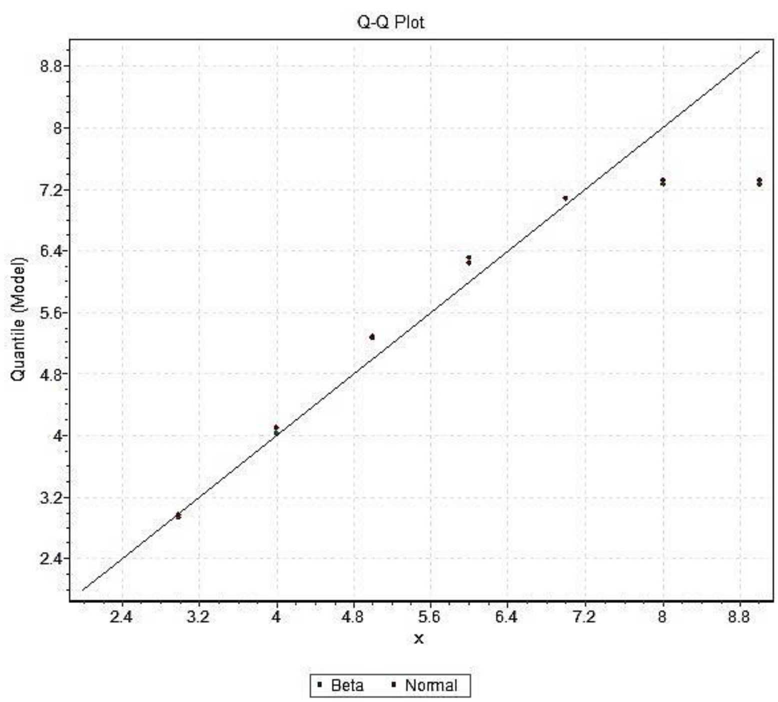


Figure 7.3 Probability density function and QQ plot for grid test results

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, we can carry out computer simulations.

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 this simulation, 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 7.3 gives the results of the simulations performed. It presents various percentiles and the corresponding values of standard deviation and variance obtained from the simulations. From this table, it can be seen that to be reasonably conservative, we can take the standard deviation corresponding to the 90 percentile for our calculations.

Table 7.3 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

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 approach a solution to 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 of sets of n 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 2011). Therefore, knowing that: (i) the population is normally distributed, (ii) the standard deviation of the population, and (iii) 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, such that we can be reasonably assured 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 confidence interval (CI) is given by:

$$CI\ length = 2 \times z_{\alpha/2} \times \frac{\sigma}{\sqrt{n}} \tag{6.17}$$

where $z_{\alpha/2}$ represents the confidence level, 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 (6.17), 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 $\pm$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, the sample size is equal to 7. Therefore, the mean of 7 DCP tests performed within a reasonably small area (area of 1.5 m by 1.5 m) should give us a good estimate of the mean of the population.

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 values of the properties of compacted soil in the field can be characterized using random field theory. 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 such that the mean of the DCP blow count varies in a certain way across space, while the covariance (ratio of standard deviation to the mean) remains constant.

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 7.4, 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 he RC 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 2008). Therefore, conservatively the separation length between test sets should not be more than 10 m. Based on this, the following procedure could be followed to do quality control of compacted soil:

1. Perform one test set, comprised of 7 DCP tests, on one corner of the compacted subgrade;
2. Perform a second test set (comprising of 7 tests) at a distance of 10 meters (distance A) from the first test set along the length of the compacted structure;

3. Perform the next test set at distance of 20 meters (distance B) 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 A).
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 to 5 meters and 10 meters, respectively, due to the higher variability associated with compaction of manufactured sands.

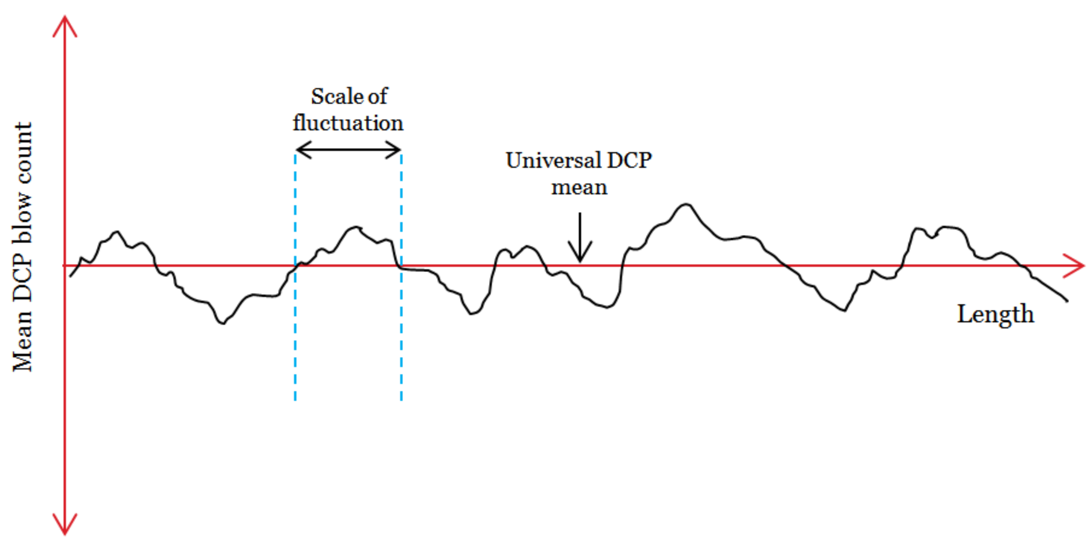


Figure 7.4 Scale of fluctuation

CHAPTER 8. SUMMARY AND CONCLUSIONS

The DCP is an ideal tool for assessing the quality of the compaction at a jobsite. Not only it is light and easy to transport and perform, it is 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. Extensive research in the sphere of the development of various correlations for the DCP has been carried out over the past few decades. Correlations have been developed between the DCP blow count and quality control parameters, such as CBR, resilient modulus and relative compaction. As a result, the DCP has become an accepted QA/QC testing device across the road construction industry.

Its ease of use has made it a standard test in many DOTs across the USA. 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, there is an aspect of statistical variability that is associated with the test results.

Such variability needs to be accounted for in the process of development of QA/QC correlations. The statistical procedure outlined in chapter 3 addresses specifically this matter.

In this research study, in order to develop suitable QA/QC correlations for the DCP, soils were classified into 2 categories (coarse-grained or fine-grained soils). The effect of fabric on the resulting mechanical behavior of soils was considered; it was found that for clay contents above about 20%, the behavior of the soil changes from sand like to clay like. To account for all the dominant factors that control the mechanical behavior of soil and its response to DCP loading, the decision as to which category a soil belongs – sand-like soil or clay-like soil – was based on the compaction characteristics, plasticity index and fines content of the soils.

It was observed that the DCP blow counts for coarse-grained soils had a good correlation with the optimum water content obtained from the standard Proctor compaction test performed in the field or in the laboratory. The blow counts for penetration of 0-to-12 inches decreased when the OMC of soils compacted to at least 95% increased. The demarcation between manufactured and natural coarse-grained soils was also clear. Correlations were developed for manufactured coarse-grained soils with respect to the OMC and C_u. Figure 8.1, Figure 8.2 and Figure 8.3 show the correlations developed for coarse-grained soils.

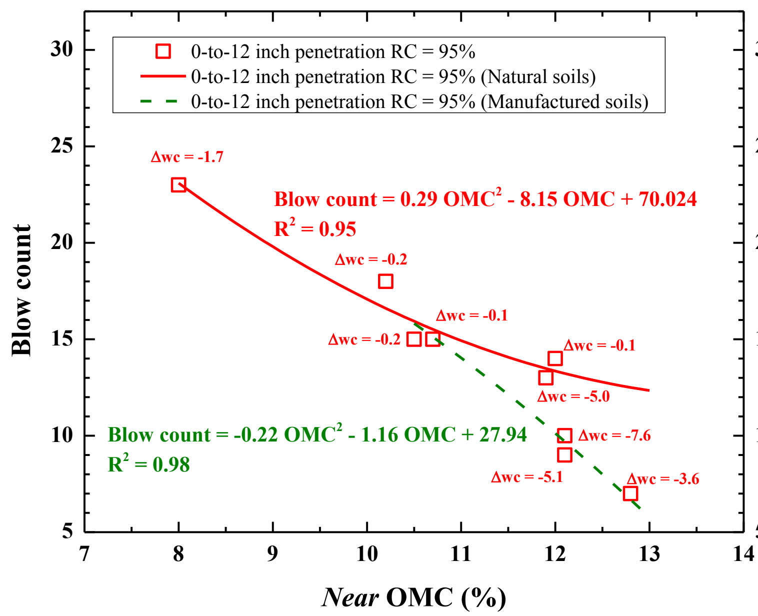


Figure 8.1 DCP blow count for 0-12 inch penetration for coarse-grained soils

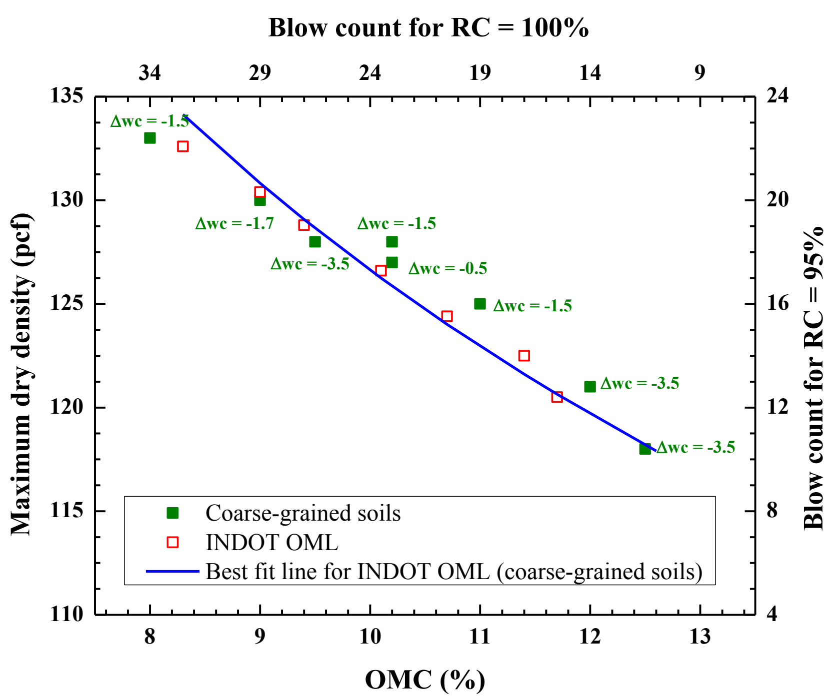


Figure 8.2 DCP blow count for 0-12 inch penetration for coarse-grained soils for 95 and 100% RC

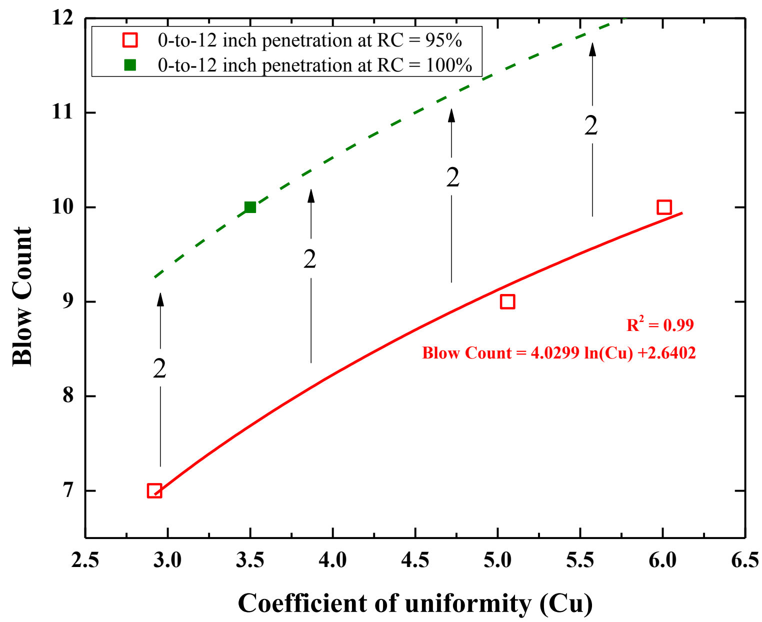


Figure 8.3 DCP blow count for 0-12 inch penetration for manufactured sands for 95 and 100% RC

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. Figure 8.4 shows the correlations developed for the fine-grained soils.

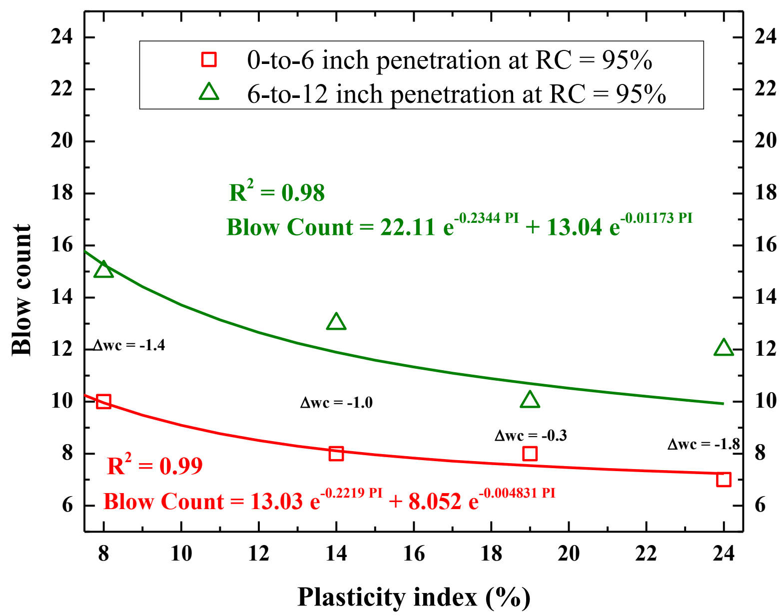


Figure 8.4 Blow count correlations for fine-grained soils

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.

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 examined, and a 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 the compacted soil to further refine the large-scale testing procedure proposed.

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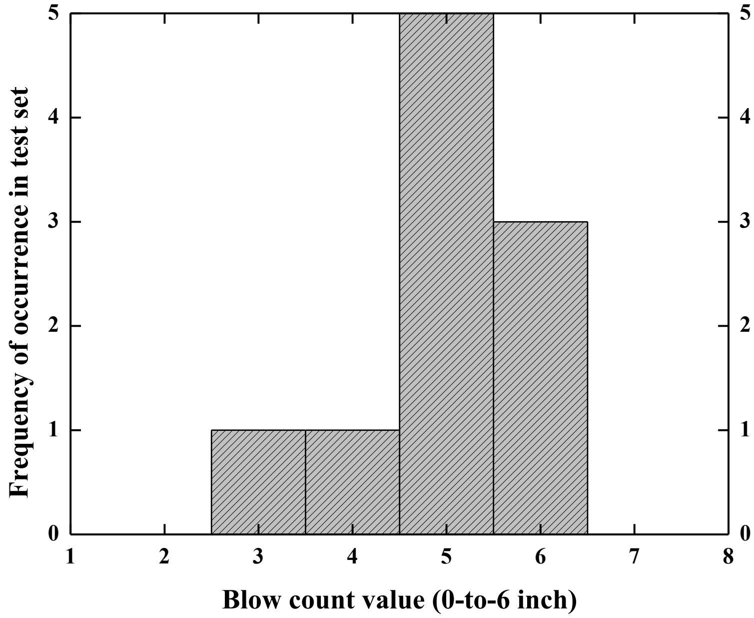
APPENDIX

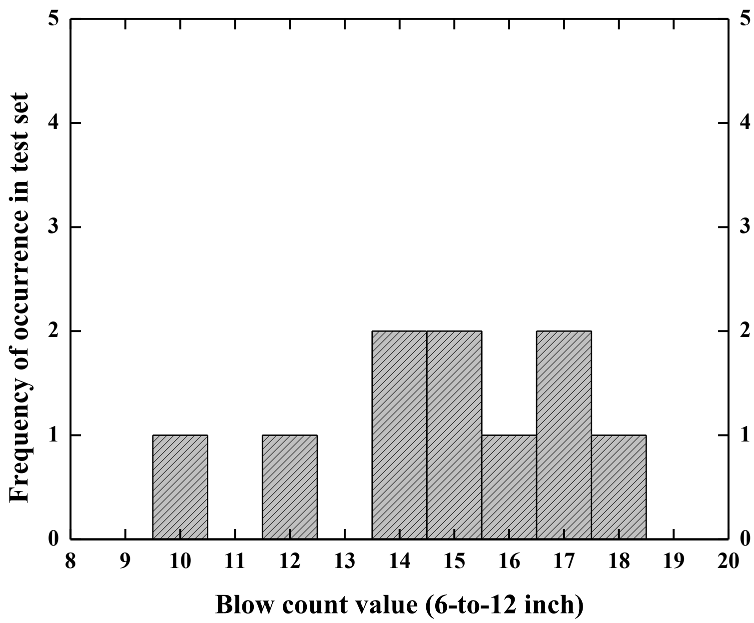
APPENDIX

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.

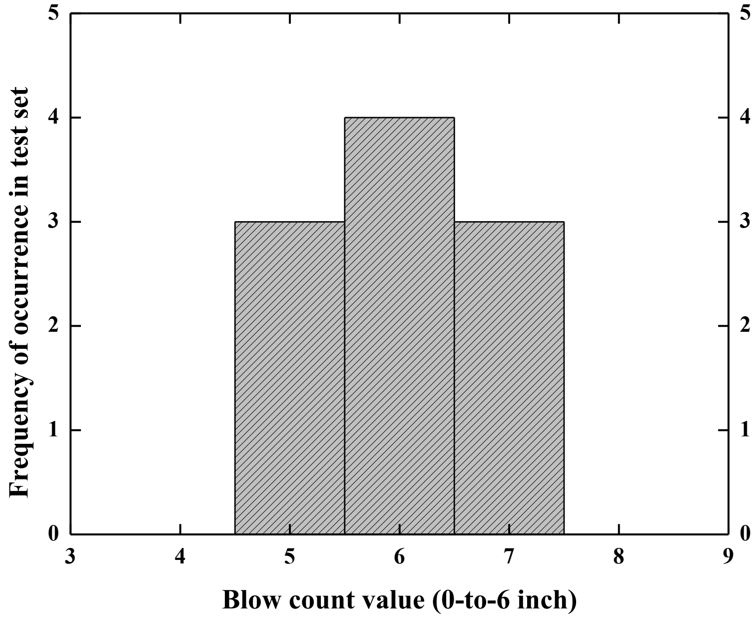


(a)

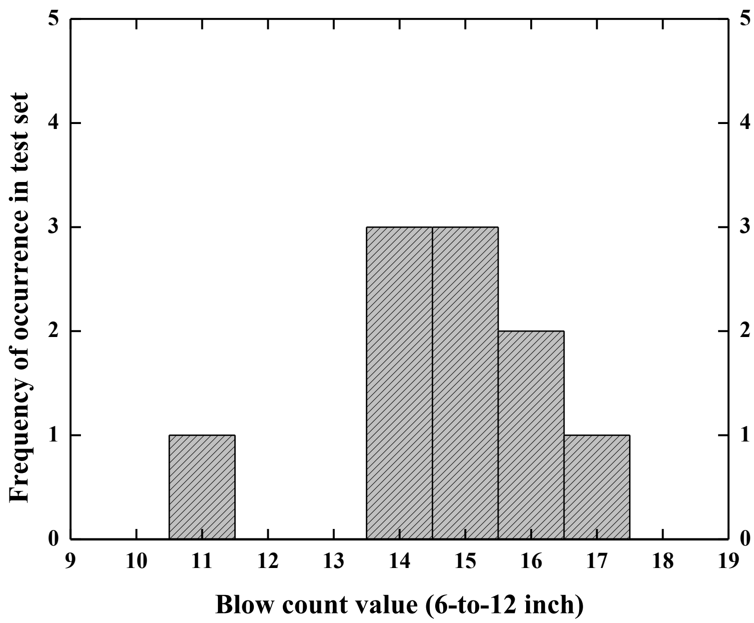


(b)

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

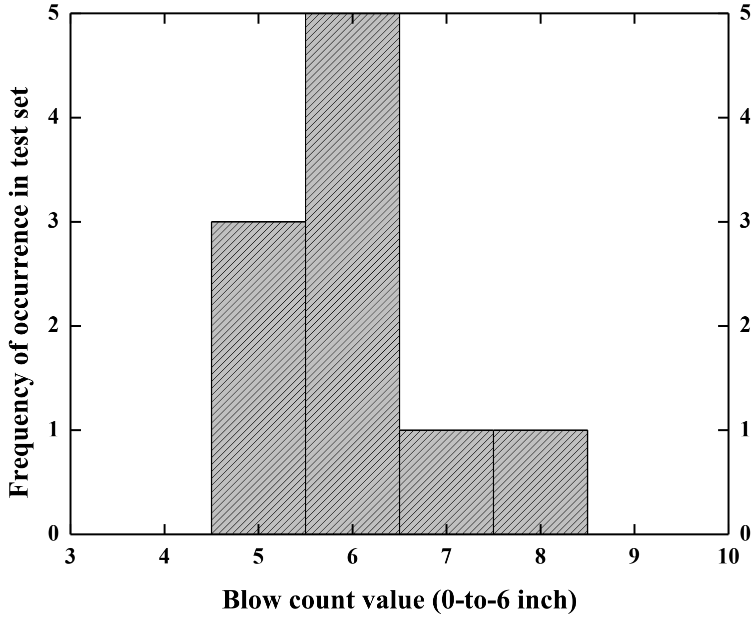


(a)

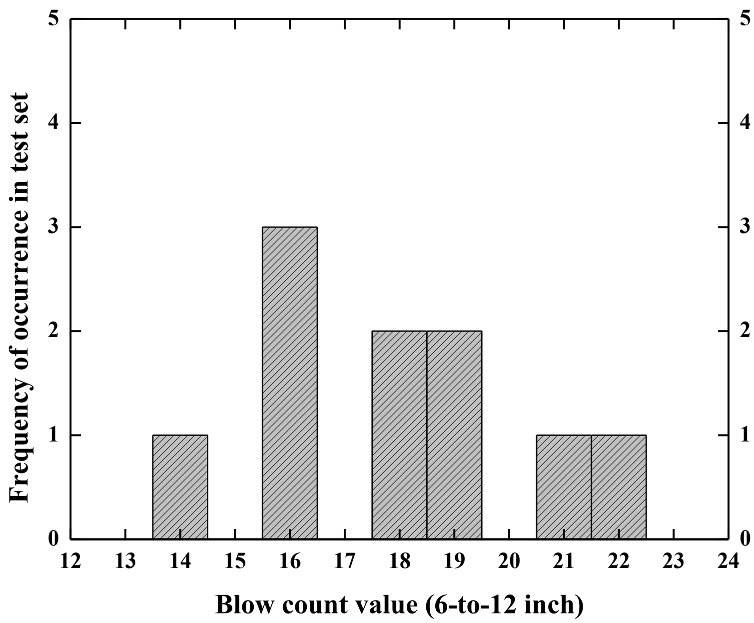


(b)

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

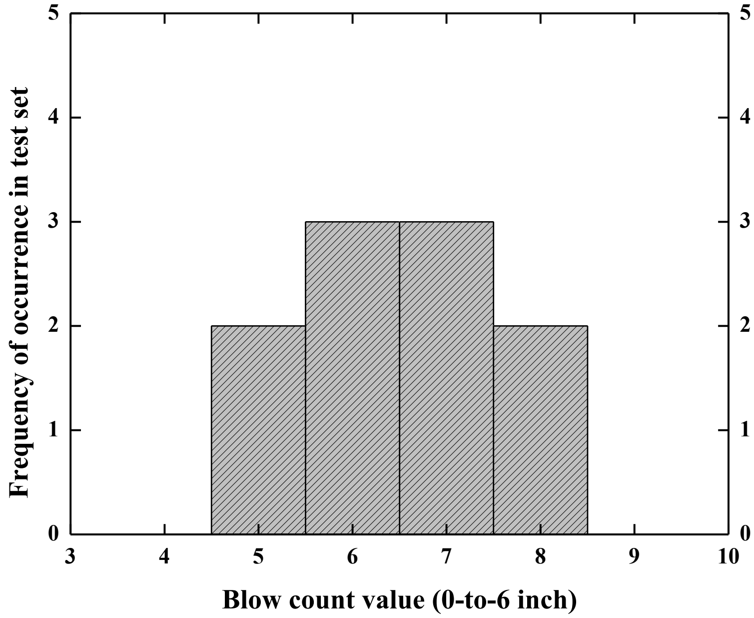


(a)

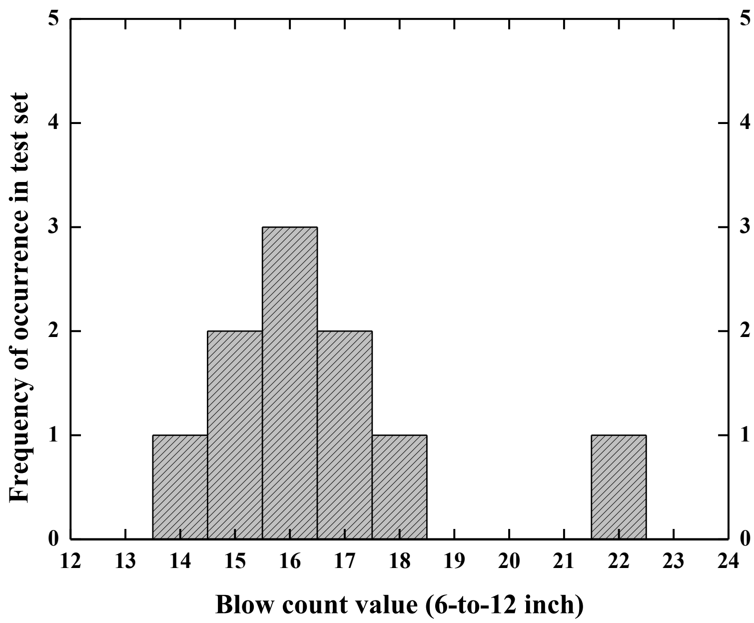


(b)

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

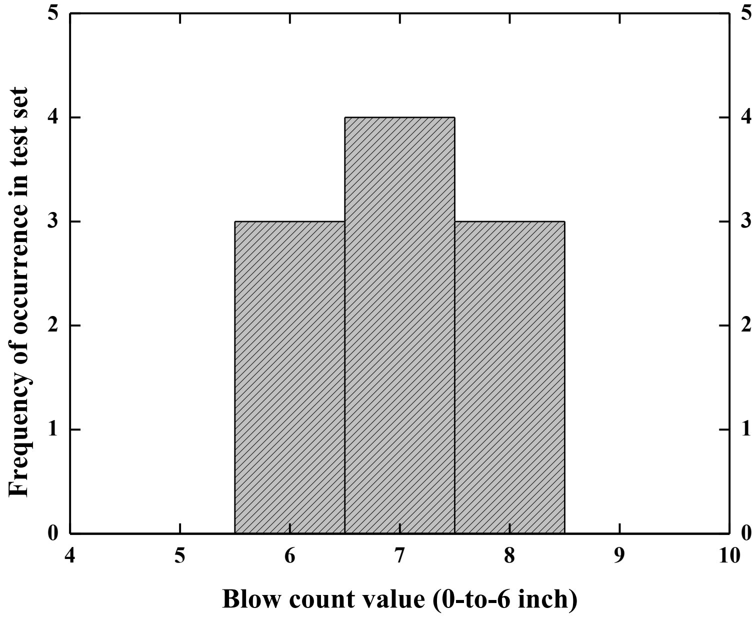


(a)

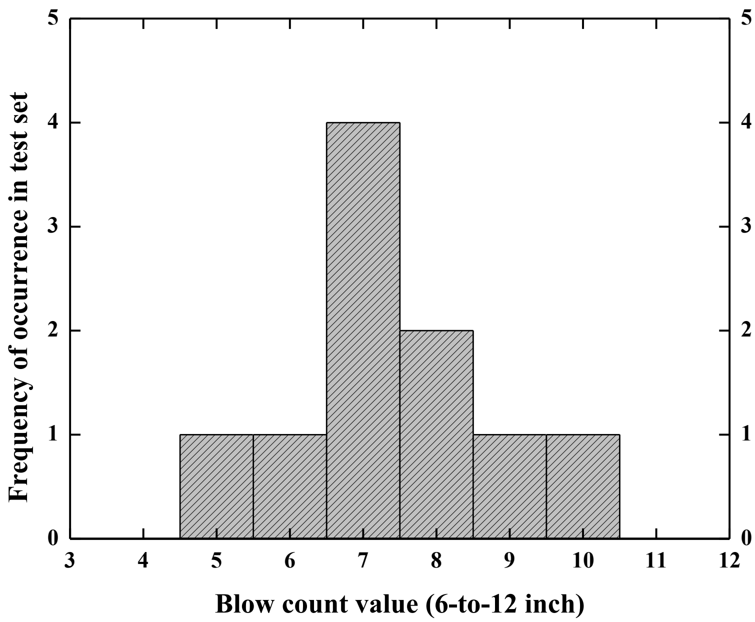


(b)

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

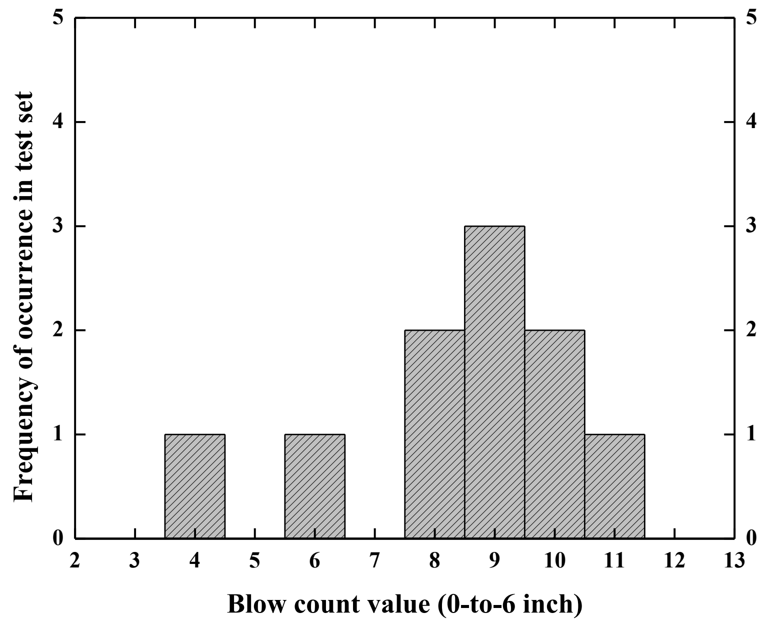


(a)

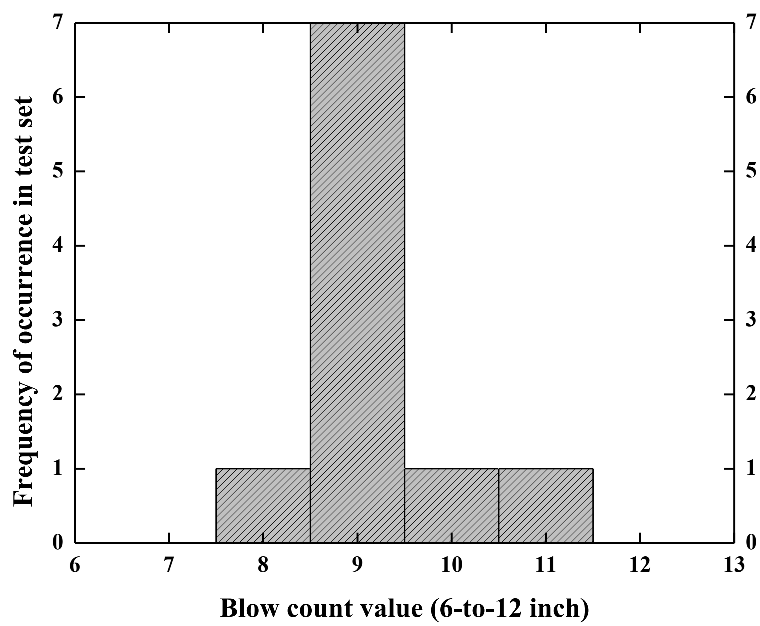


(b)

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

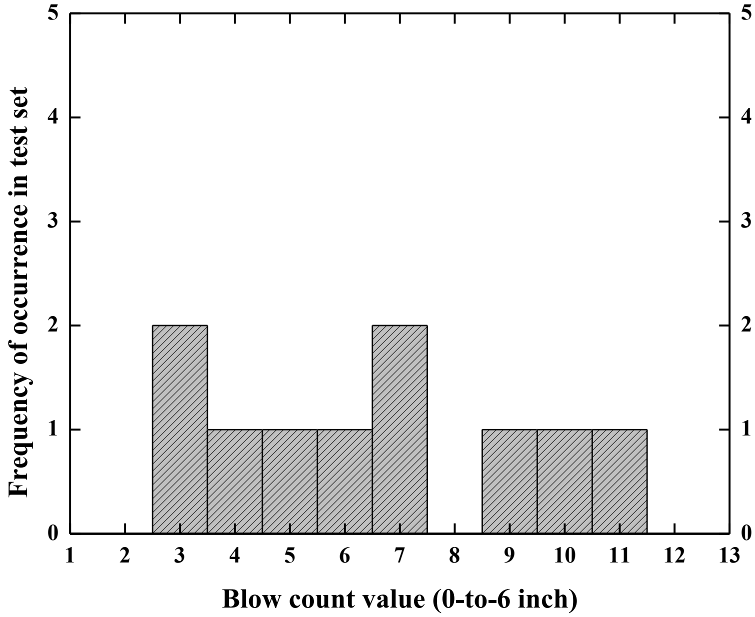


(a)

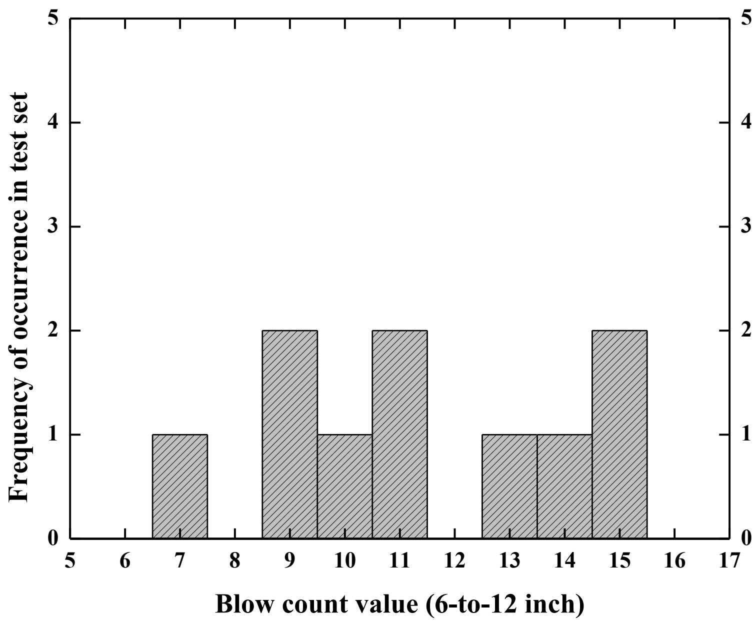


(b)

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

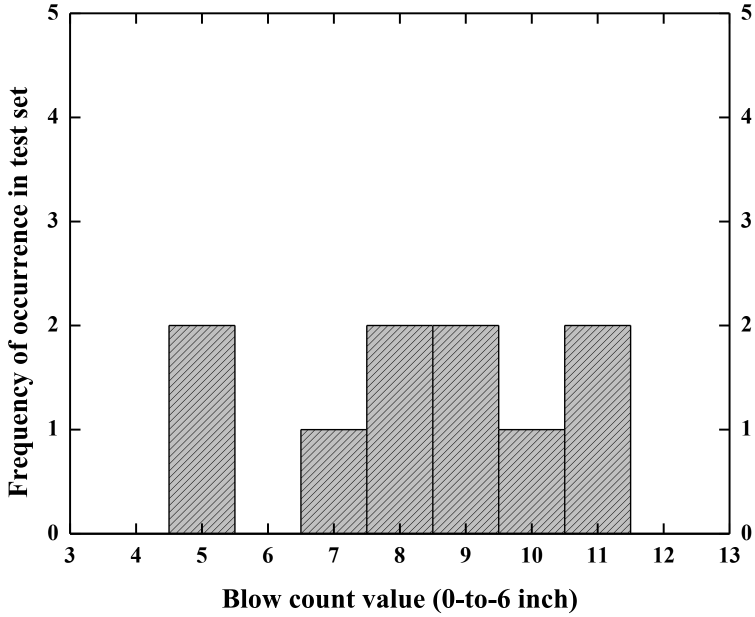


(a)

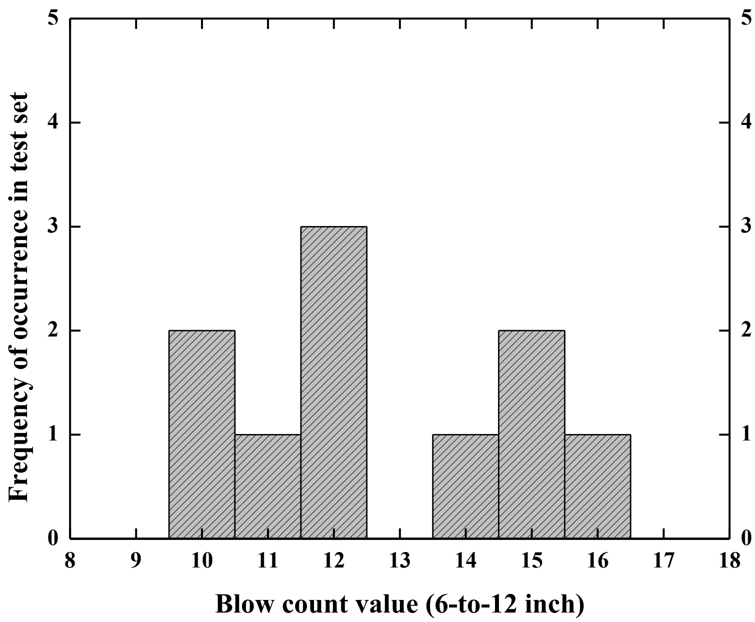


(b)

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

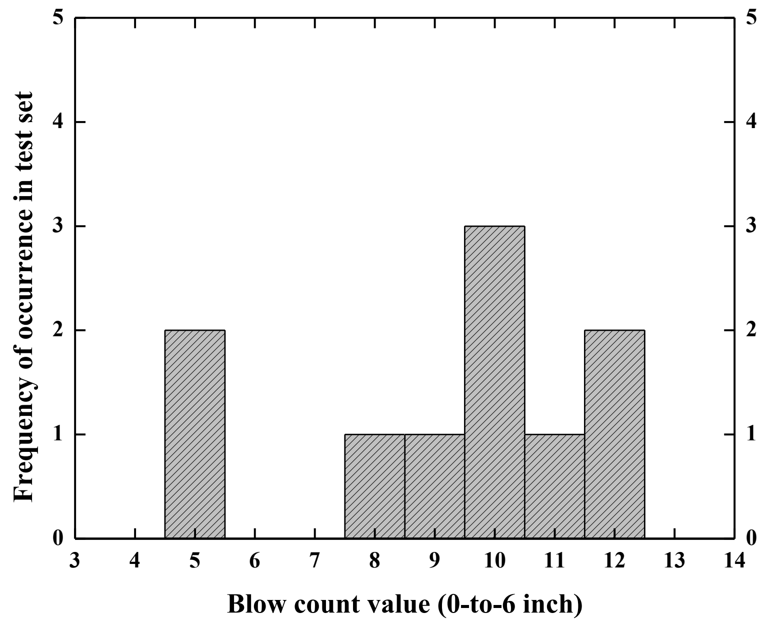


(a)

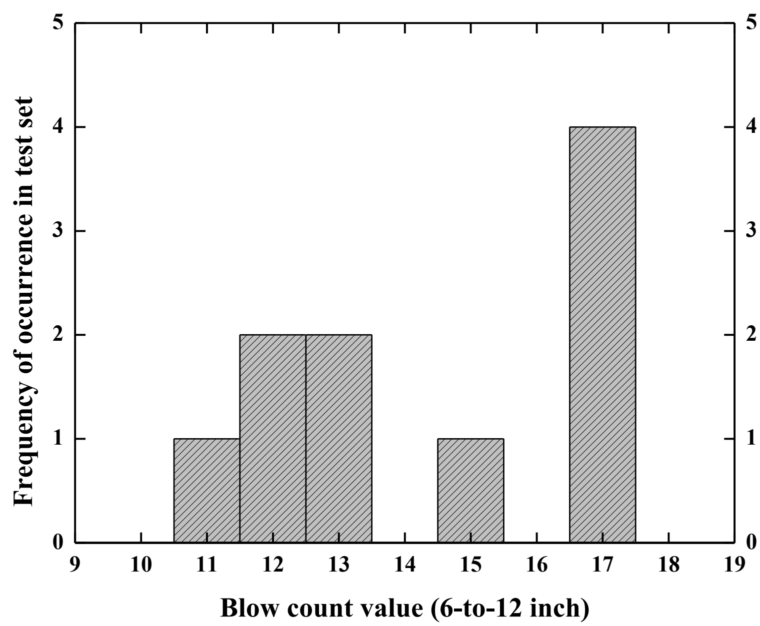


(b)

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

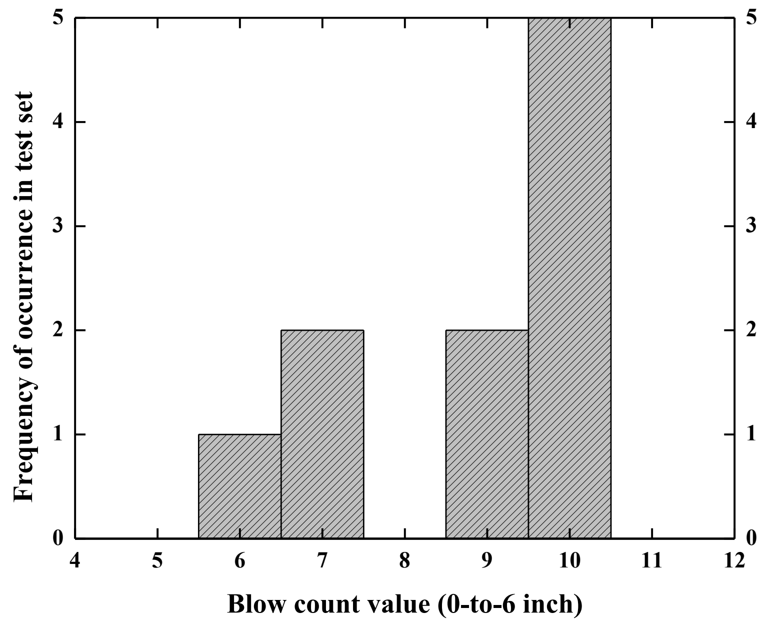


(a)

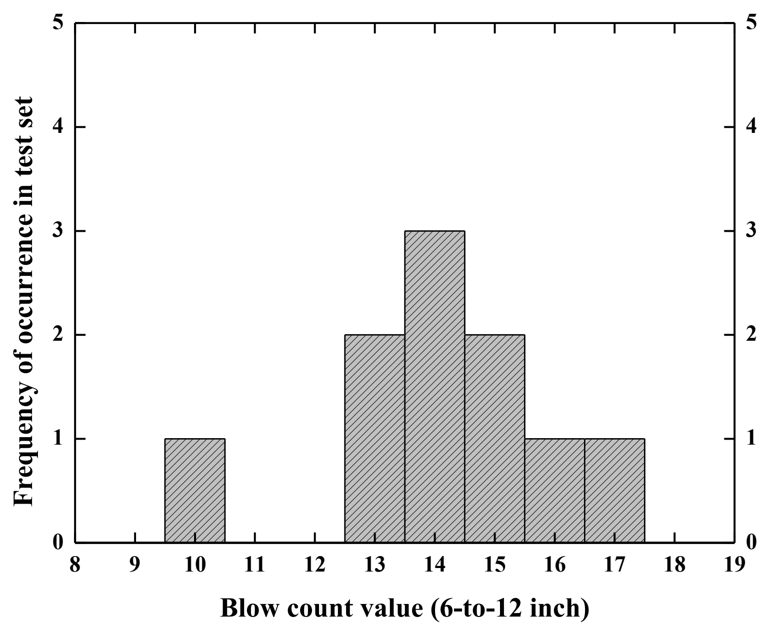


(b)

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

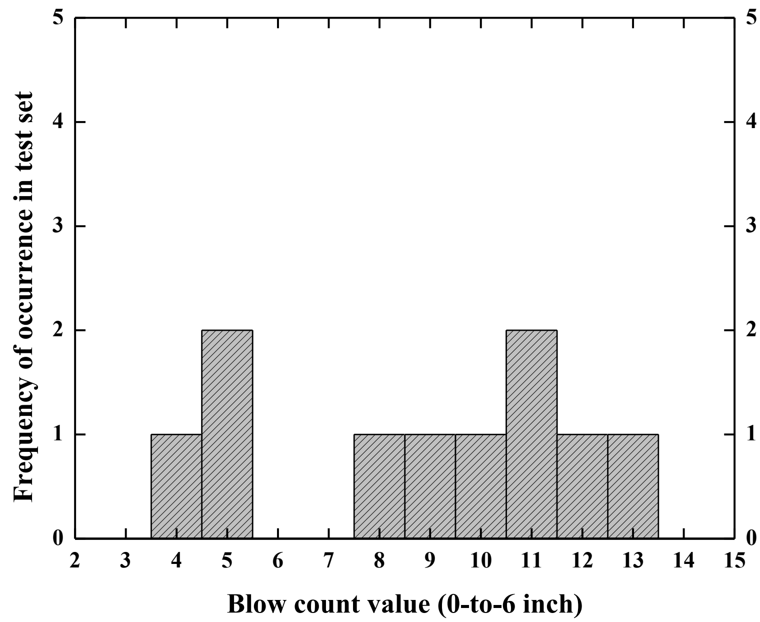


(a)

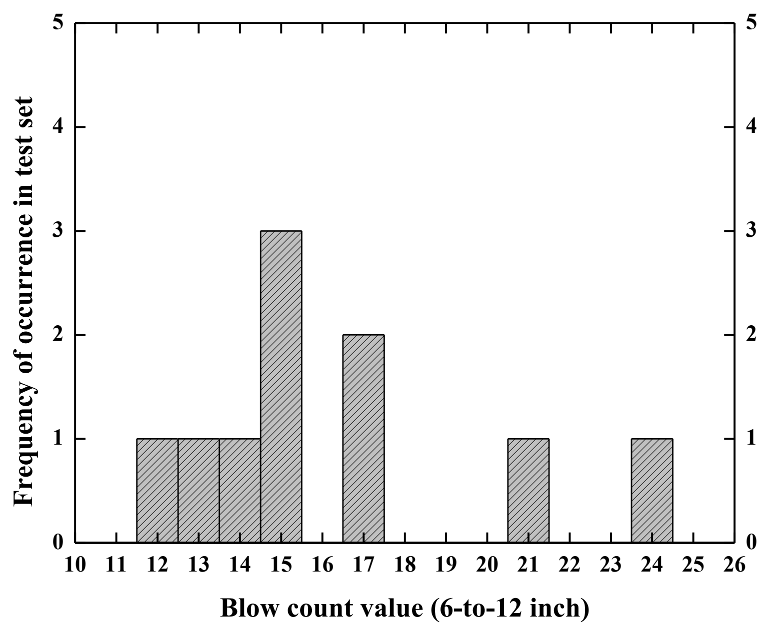


(b)

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

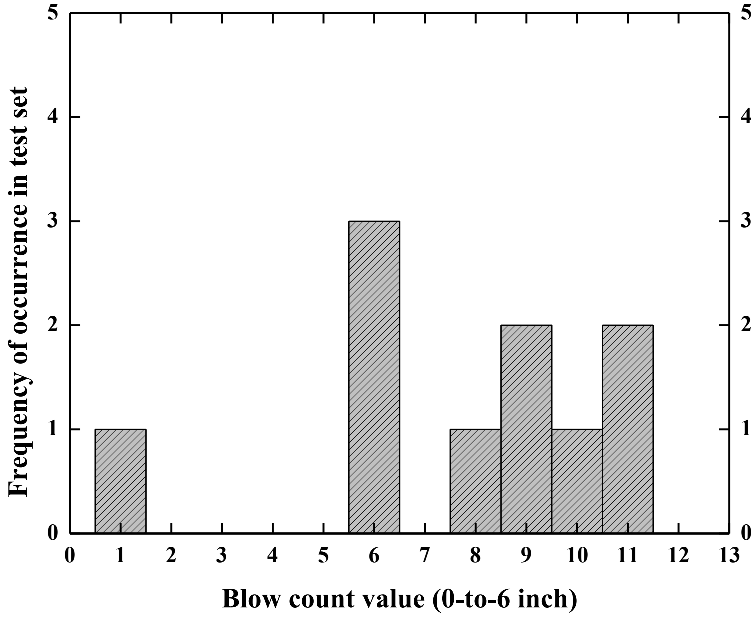


(a)

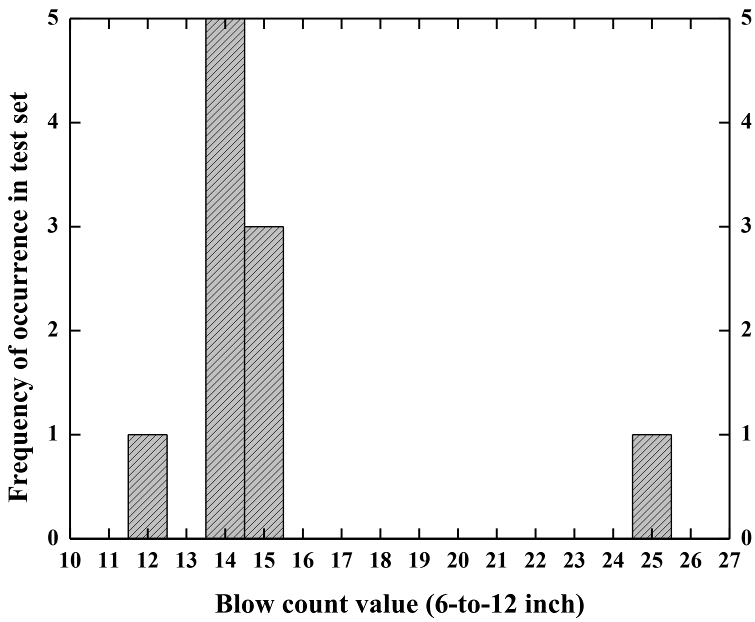


(b)

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

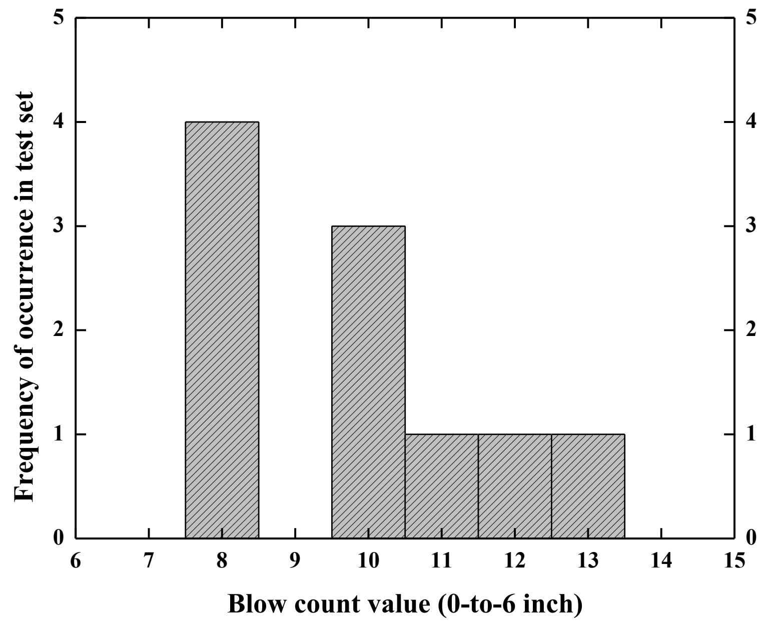


(a)



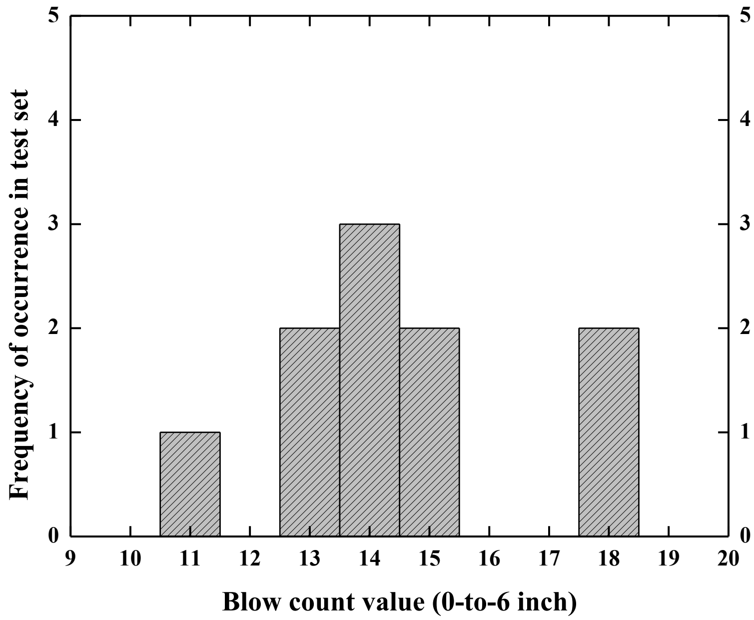
(b)

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



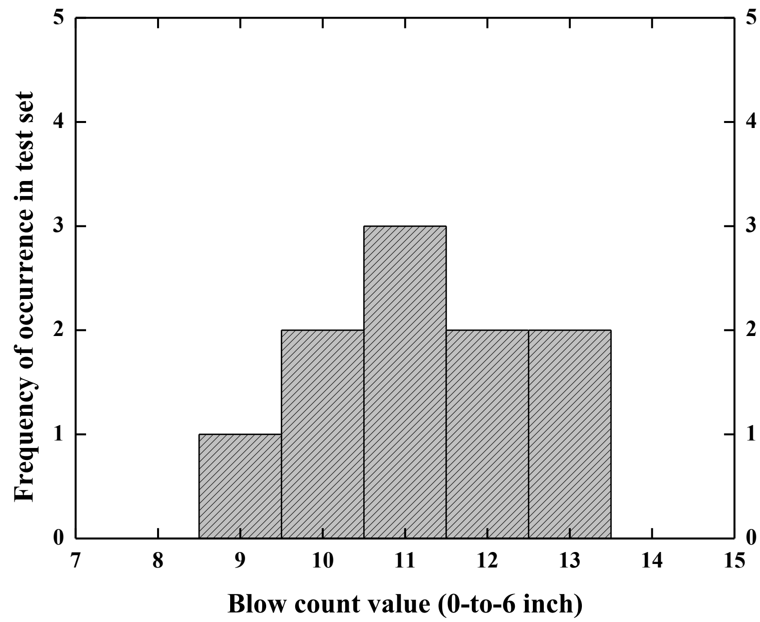
(a)

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



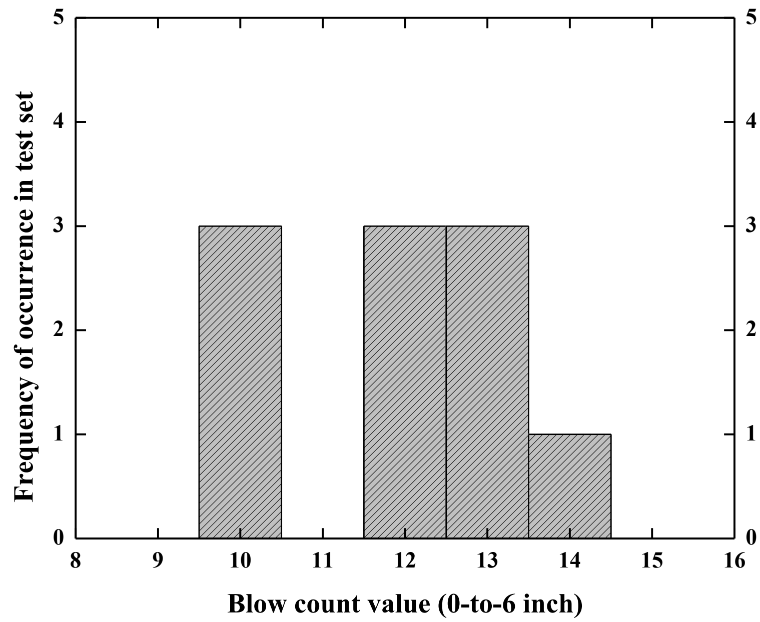
(a)

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



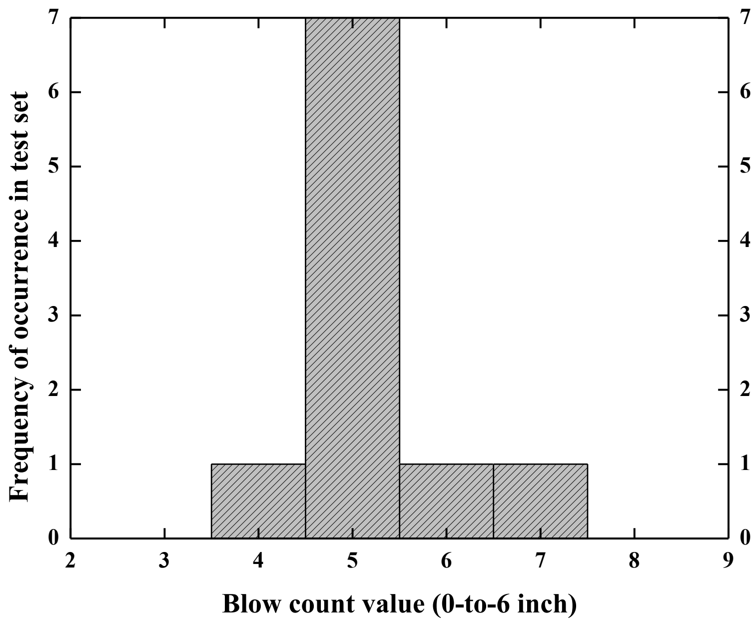
(a)

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

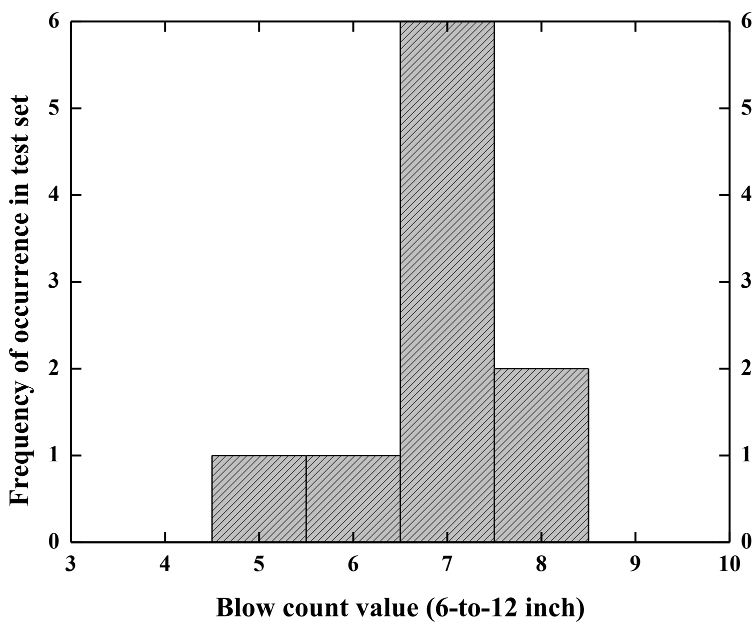


(a)

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

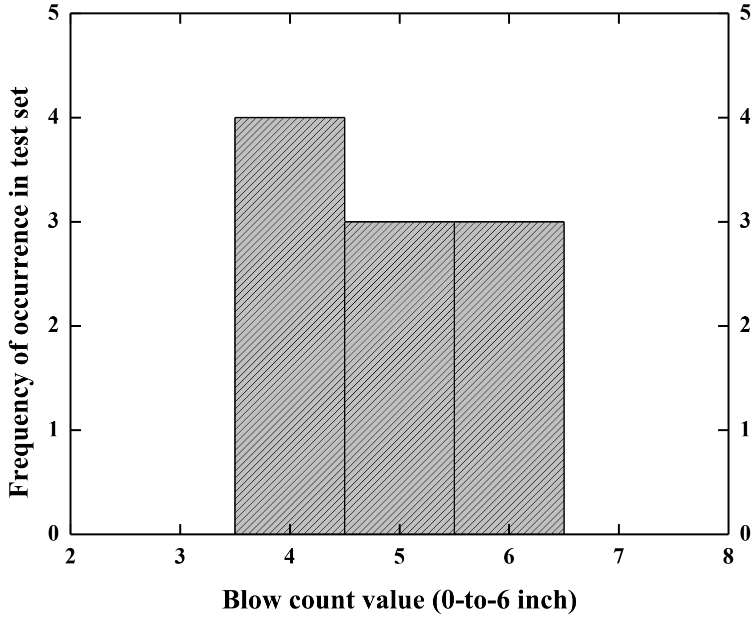


(a)

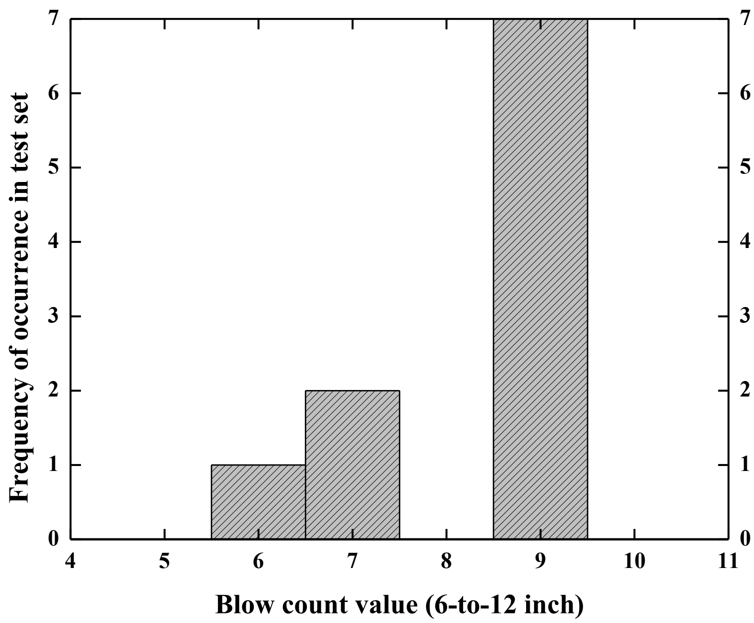


(b)

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

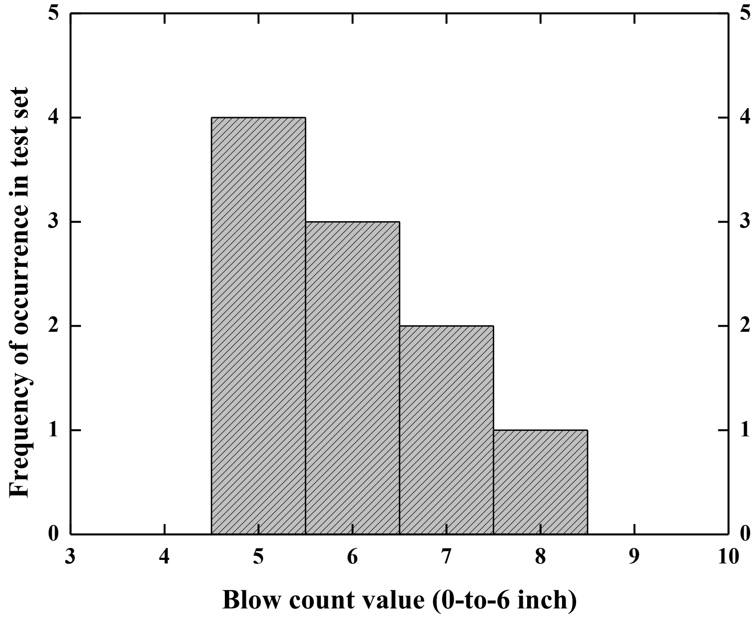


(a)

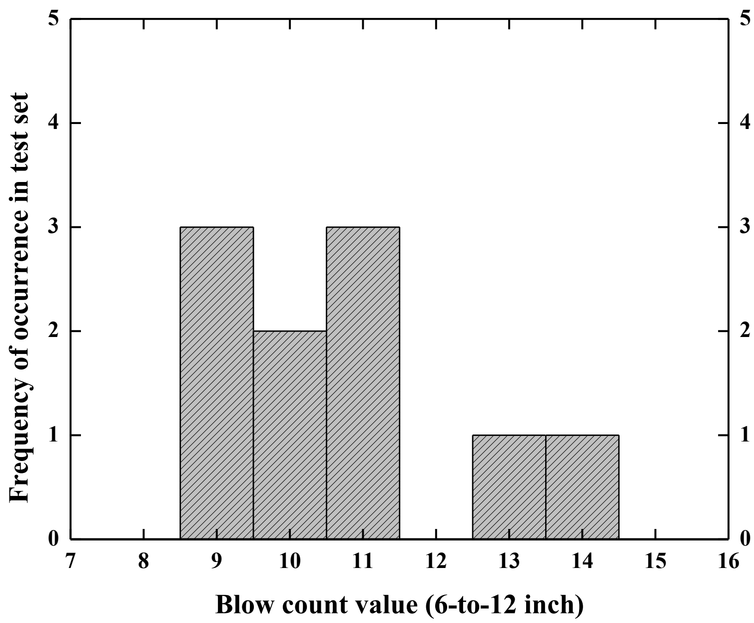


(b)

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

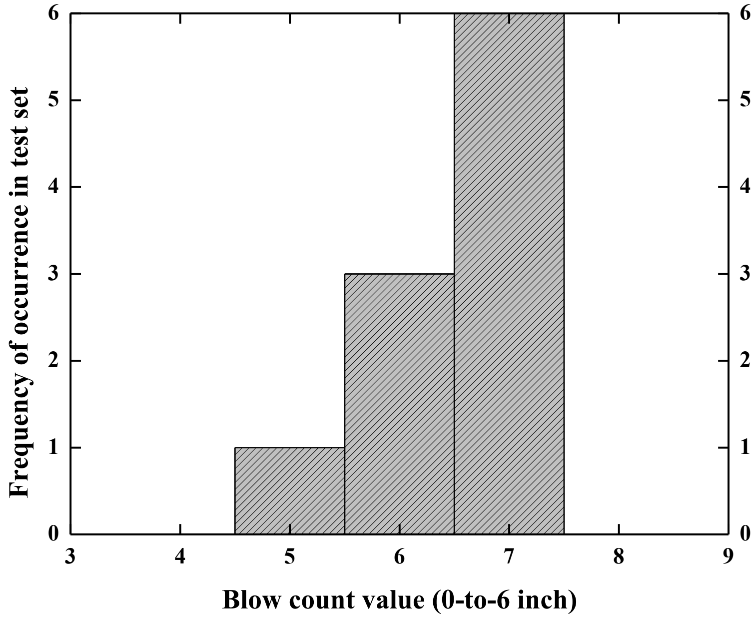


(a)

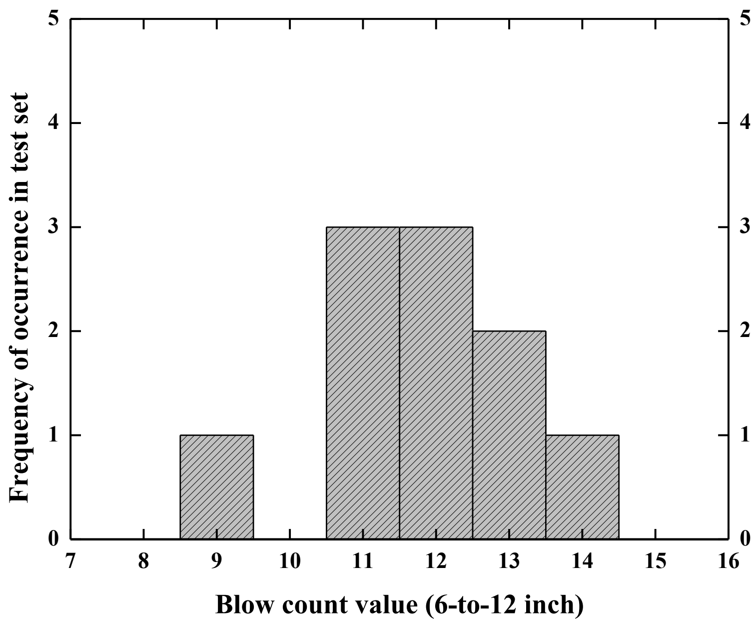


(b)

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

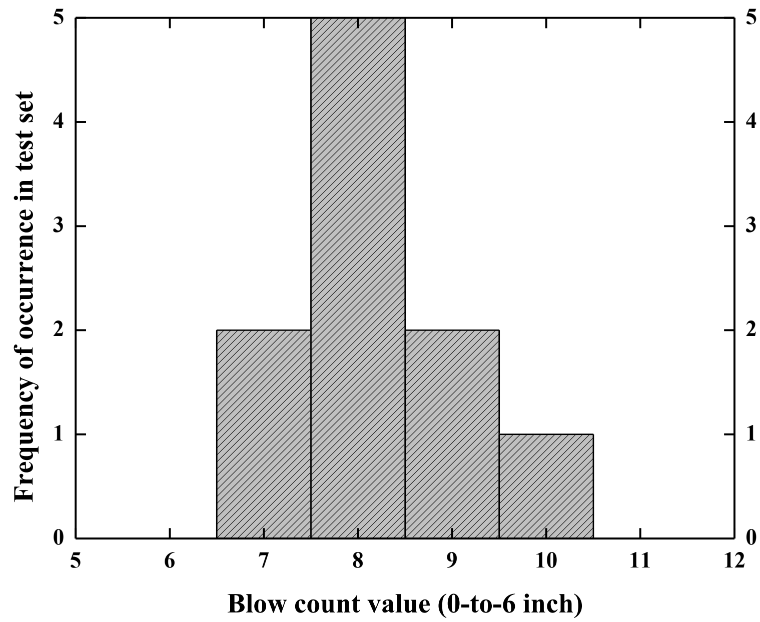


(a)

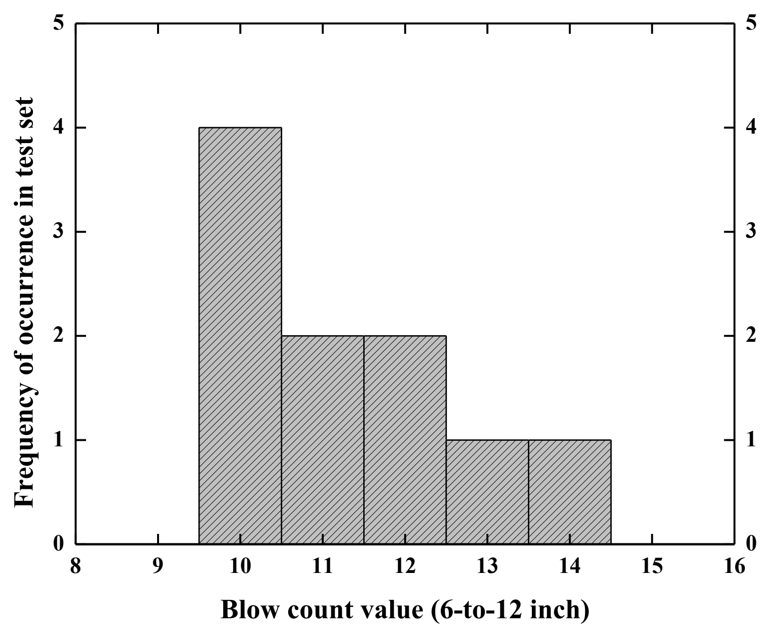


(b)

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

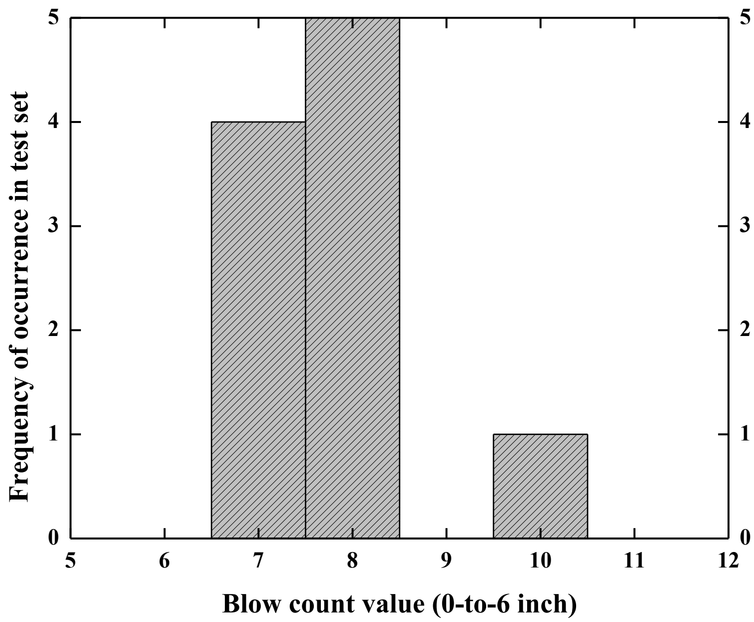


(a)

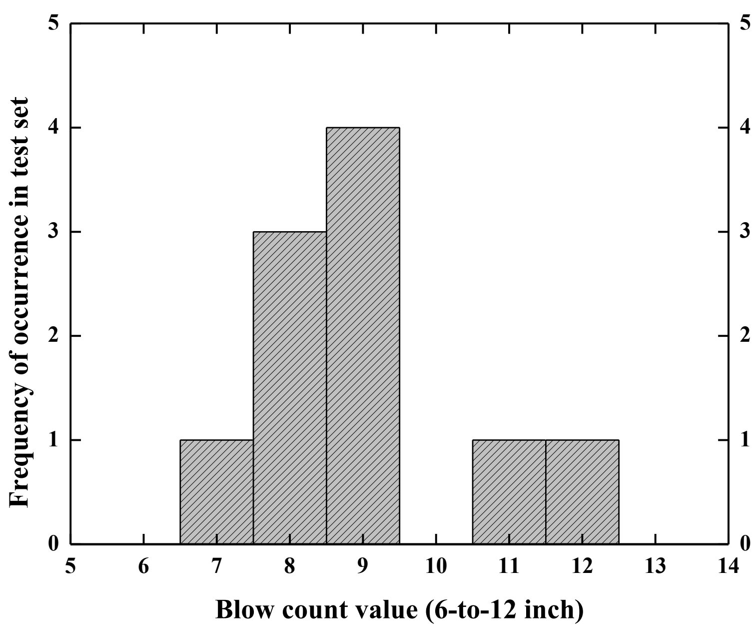


(b)

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

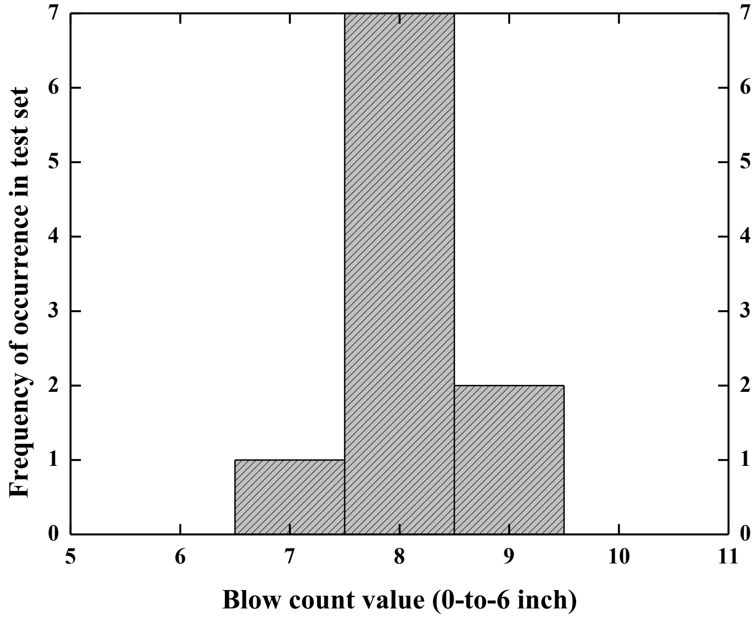


(a)

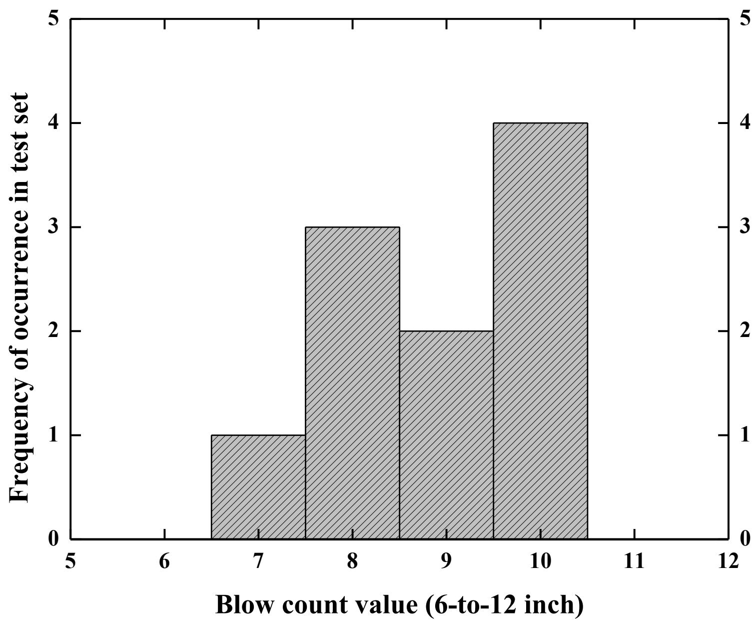


(b)

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

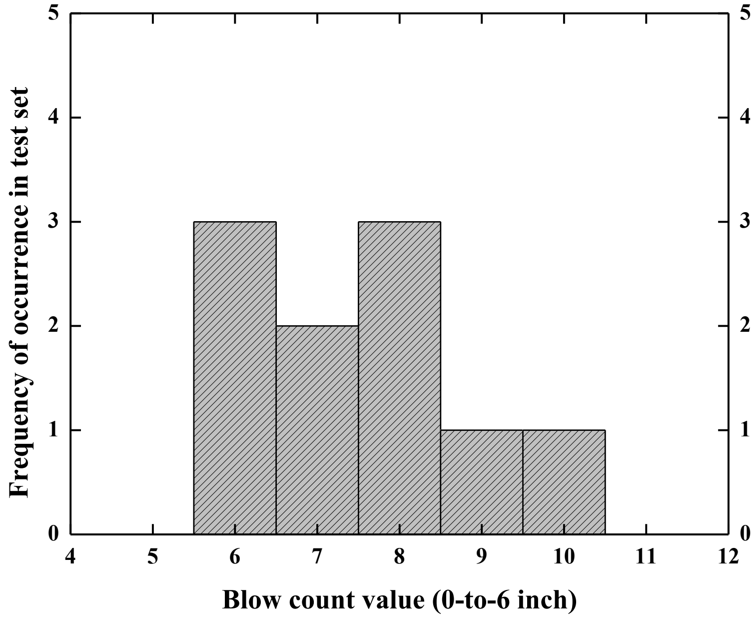


(a)

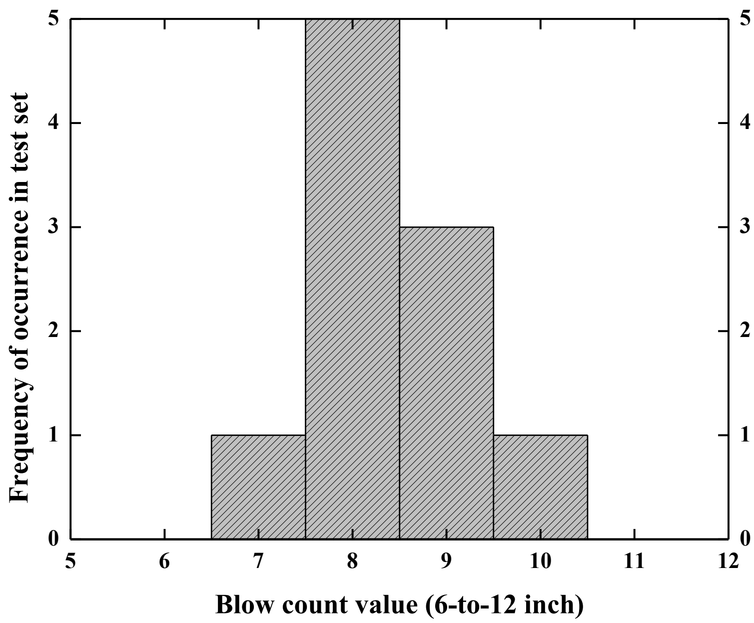


(b)

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

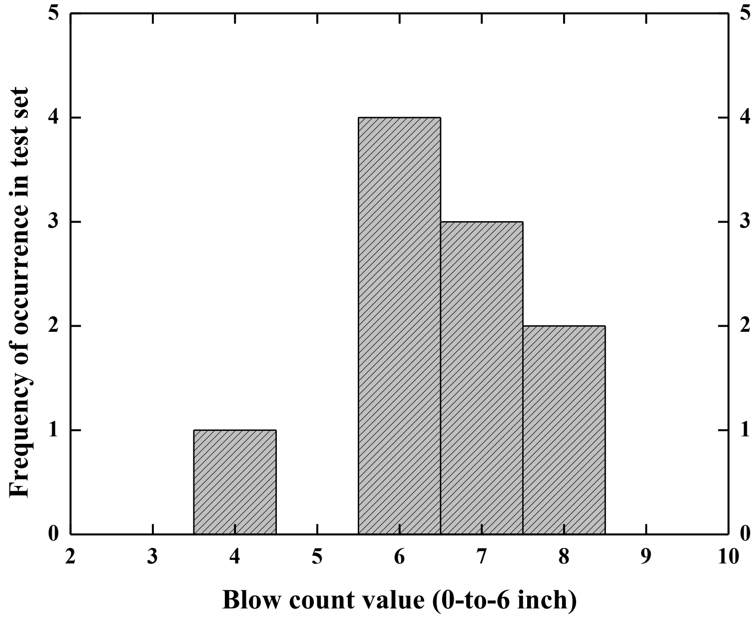


(a)

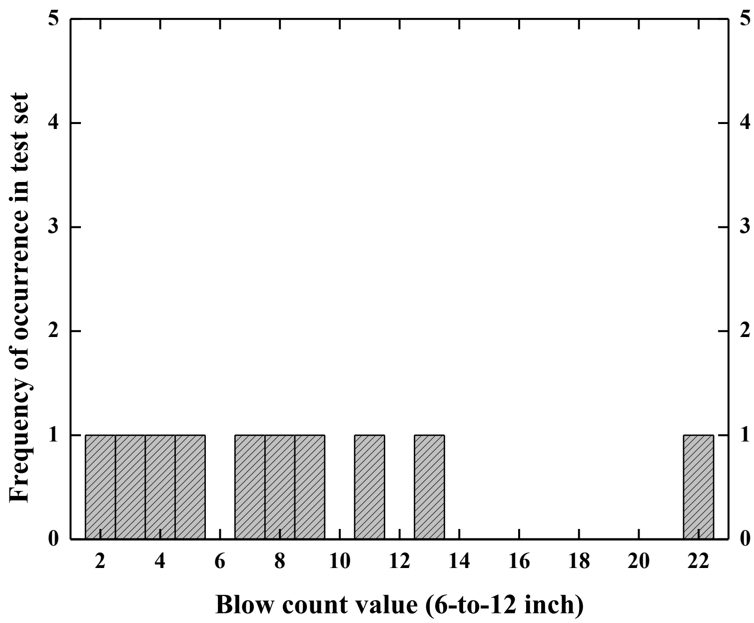


(b)

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

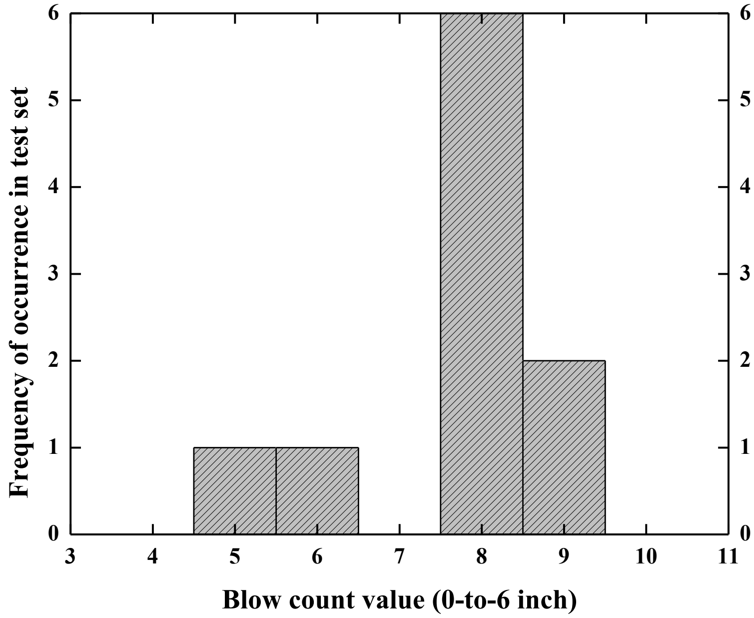


(a)

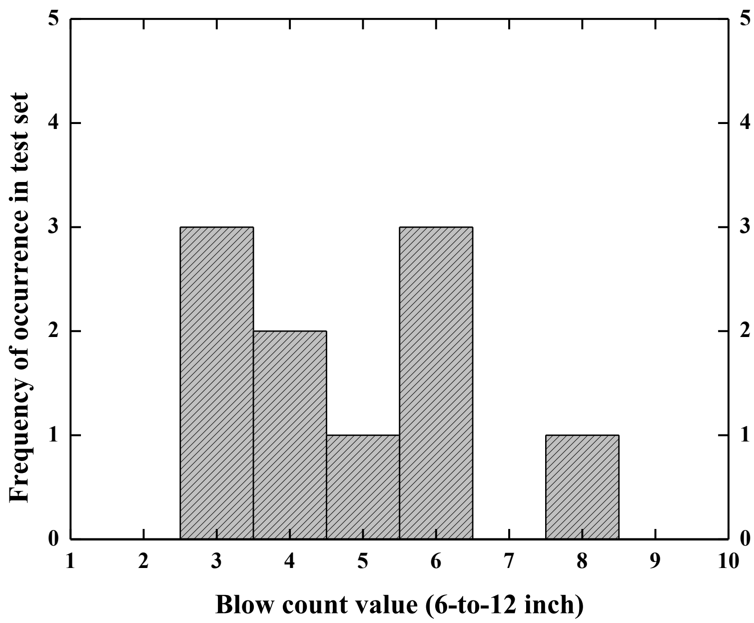


(b)

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

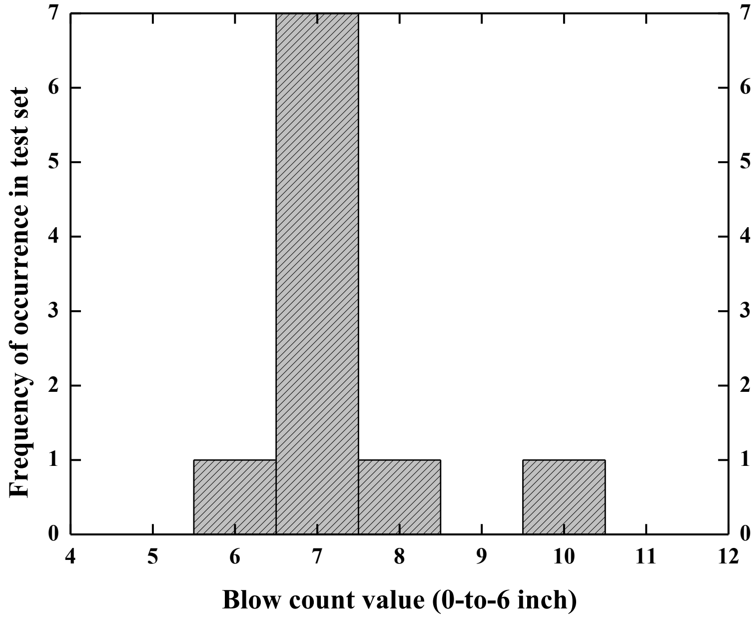


(a)

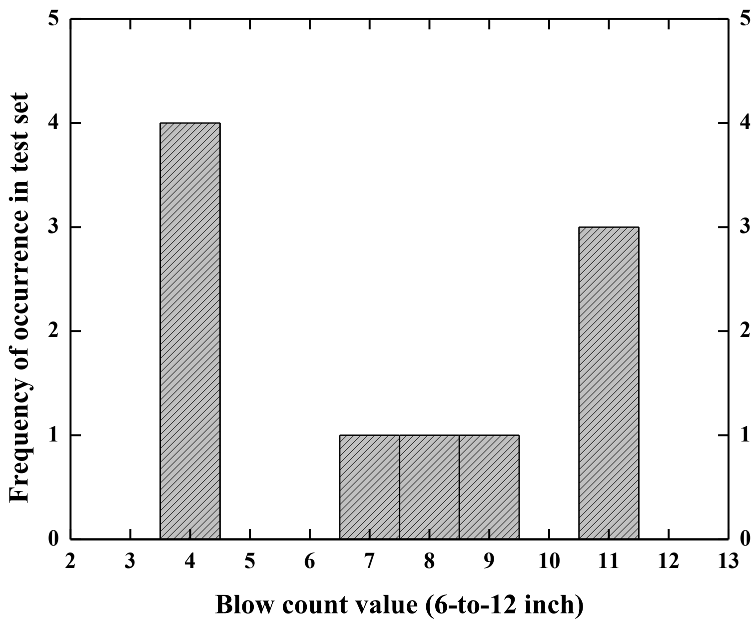


(b)

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

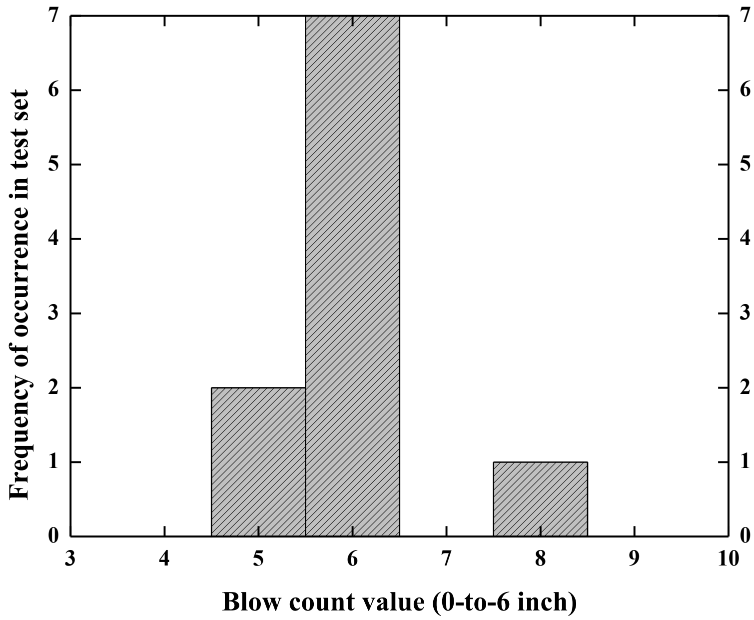


(a)

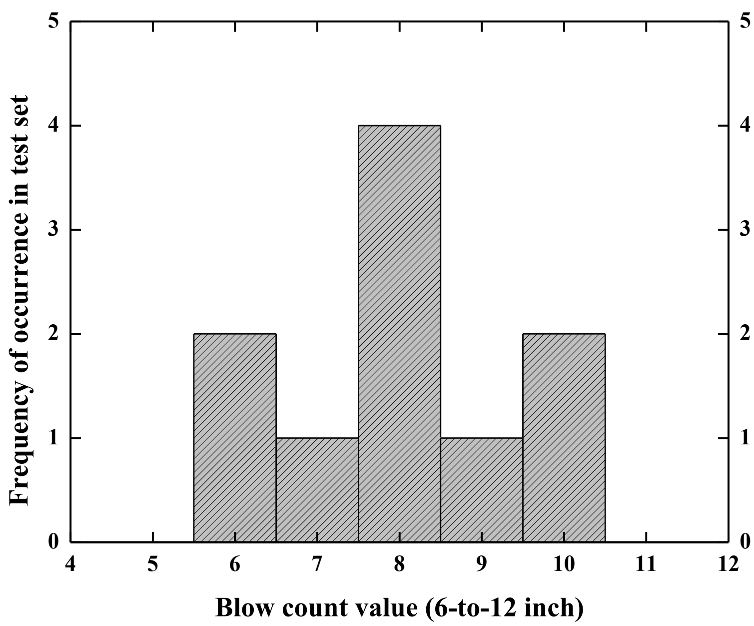


(b)

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

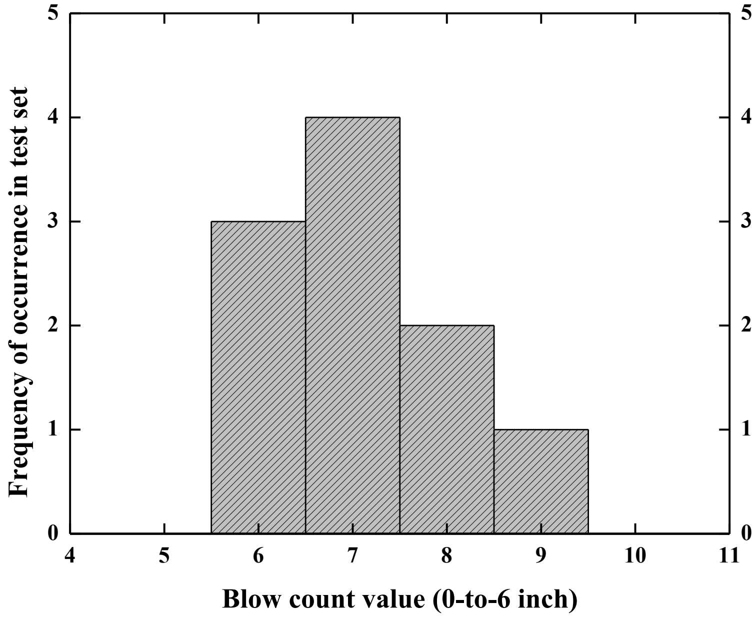


(a)

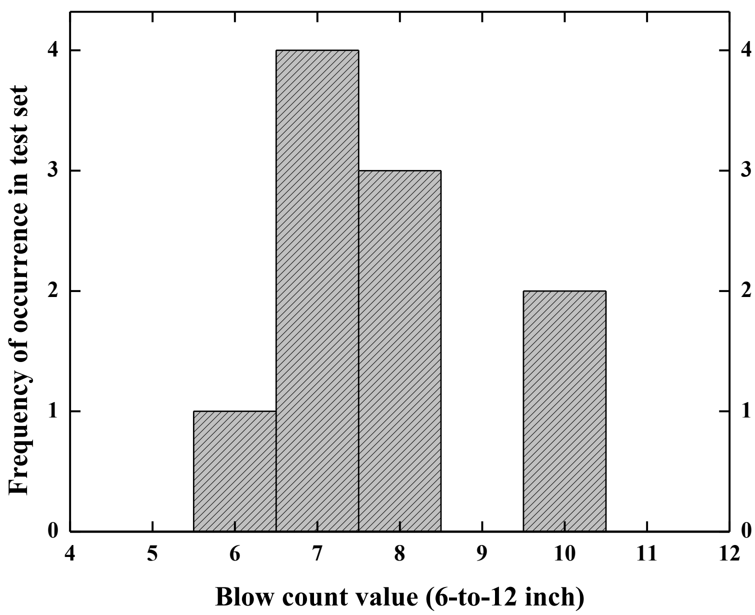


(b)

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

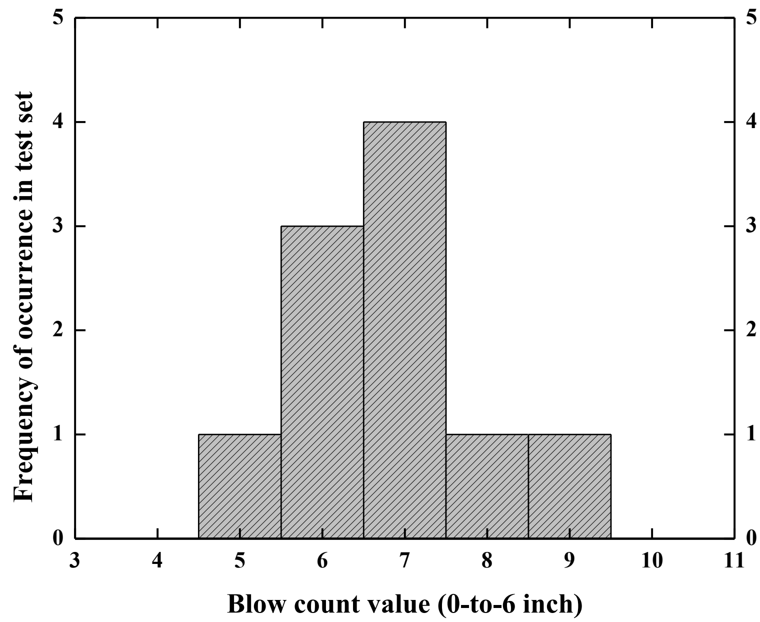


(a)

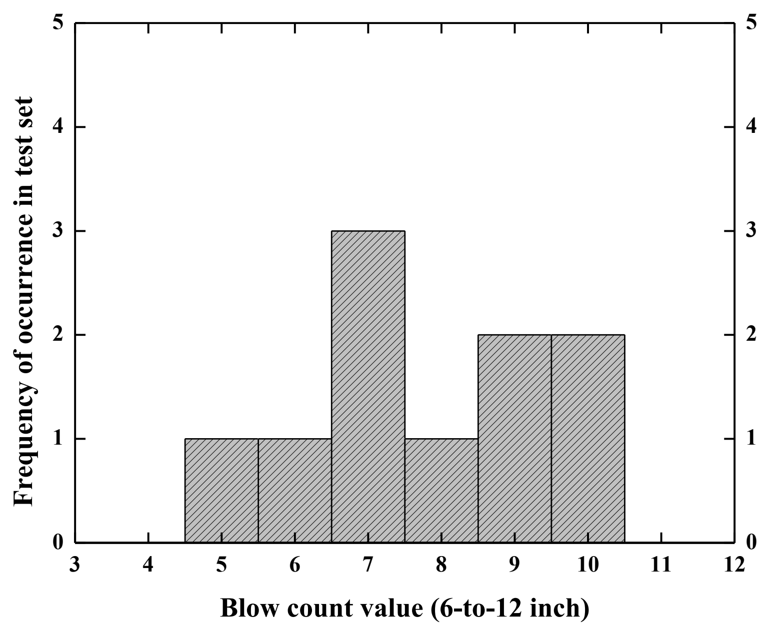


(b)

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

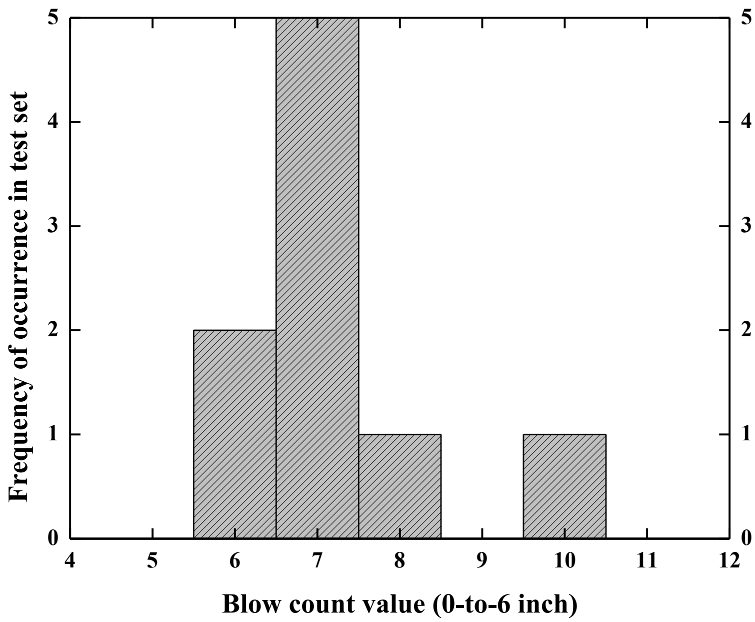


(a)

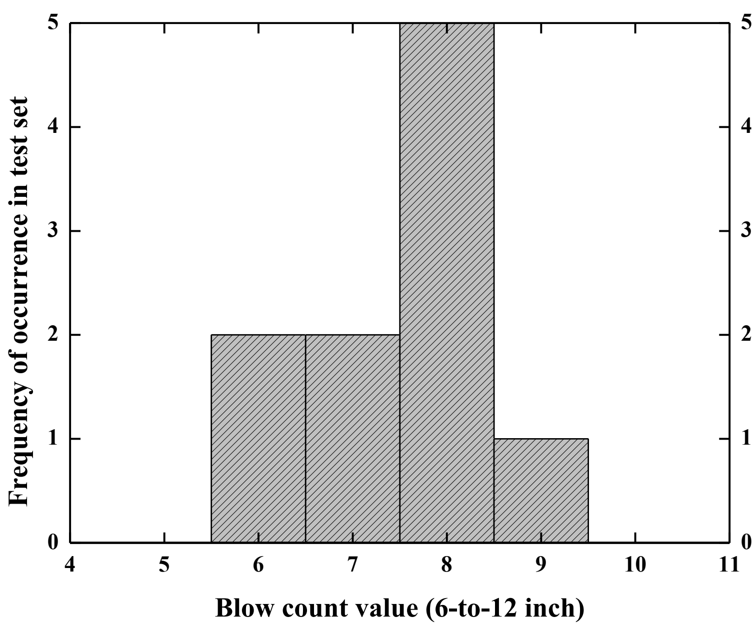


(b)

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

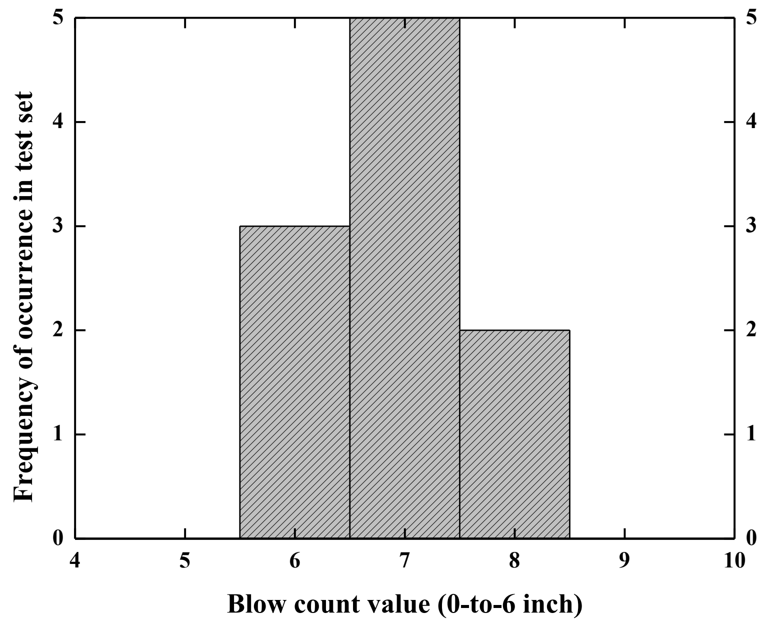


(a)

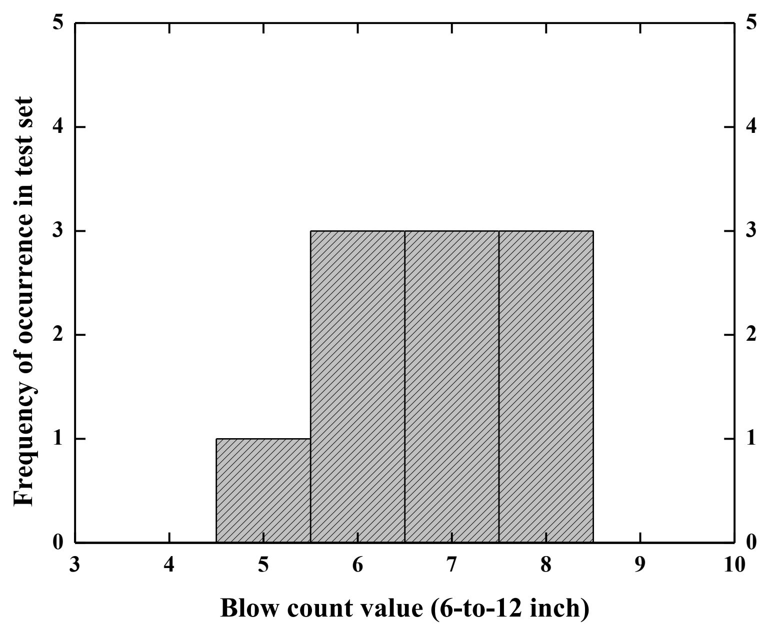


(b)

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



(a)



(b)

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

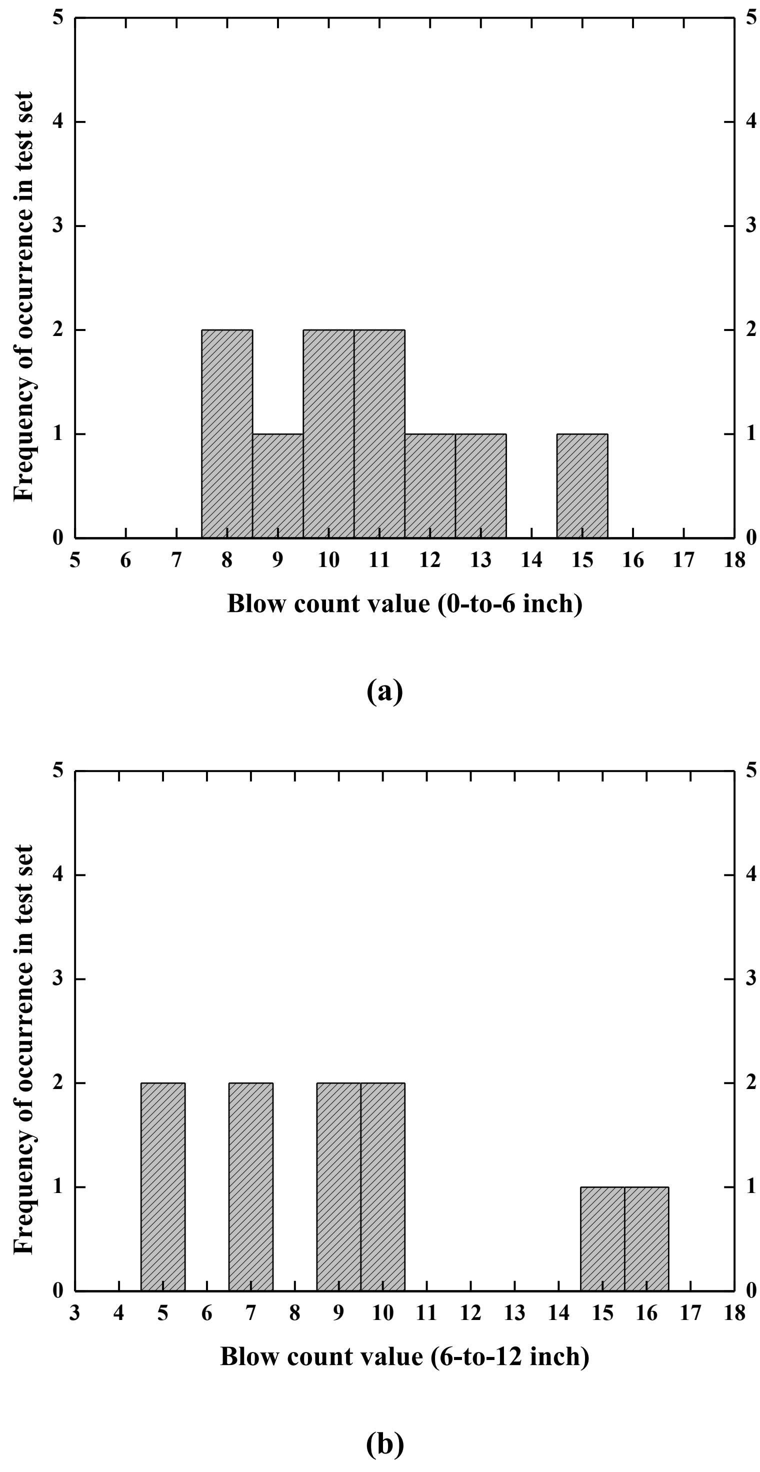
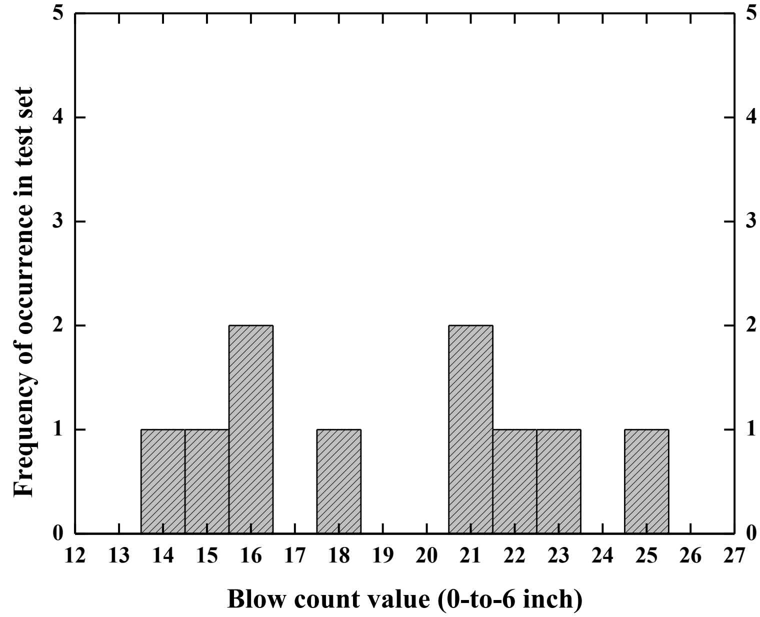
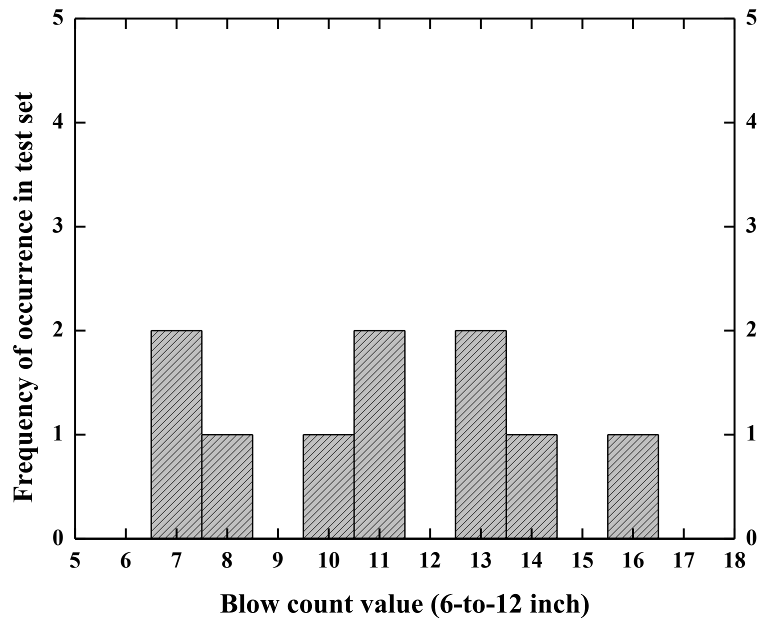


Figure A.33 Frequency vs DCP blow count for DCP-F-A6-3-NV-1 (a) 0-to-6 inches and (b) 6-to-12 inches

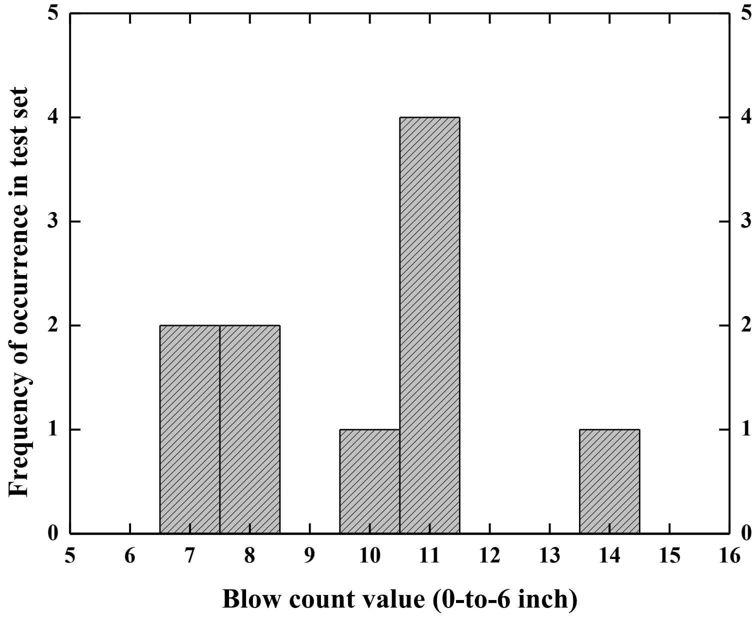


(a)

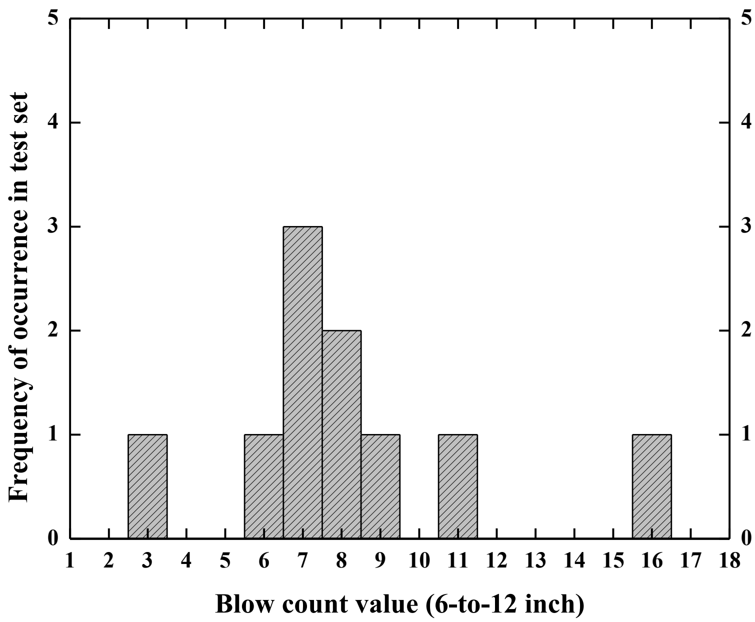


(b)

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

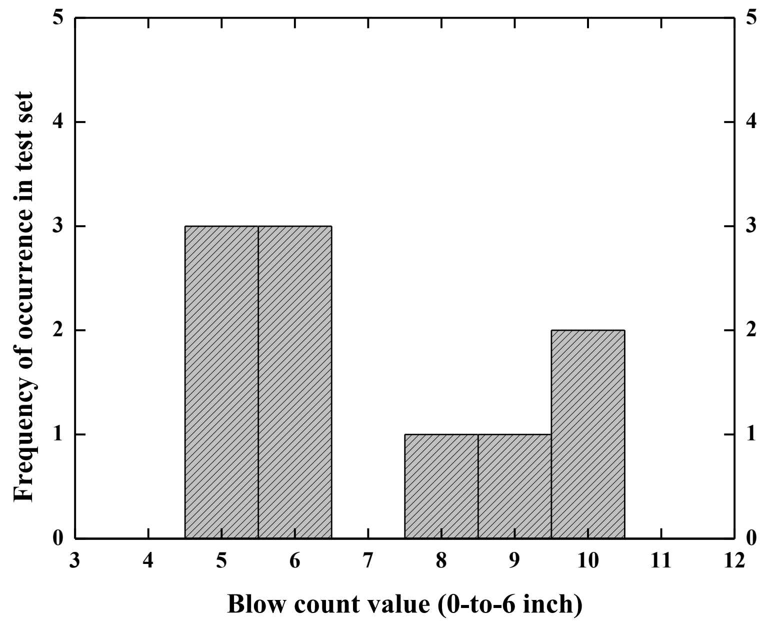


(a)

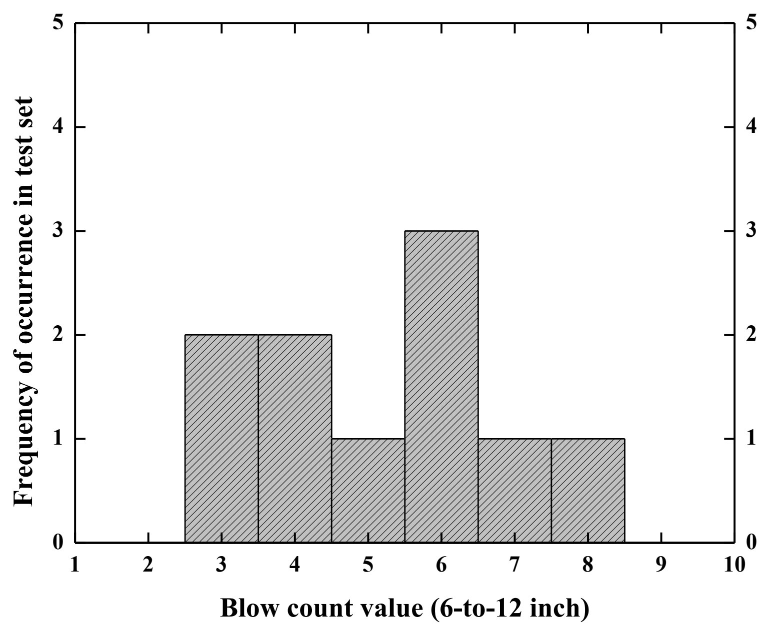


(b)

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

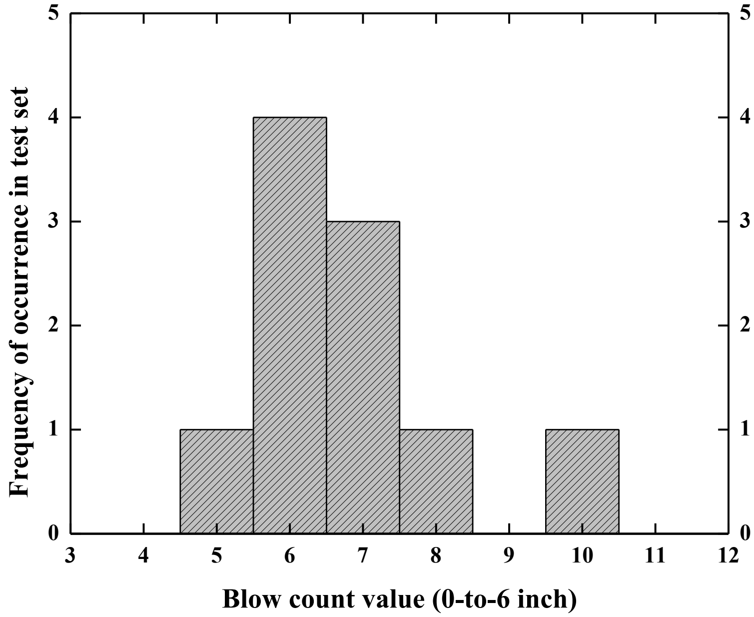


(a)

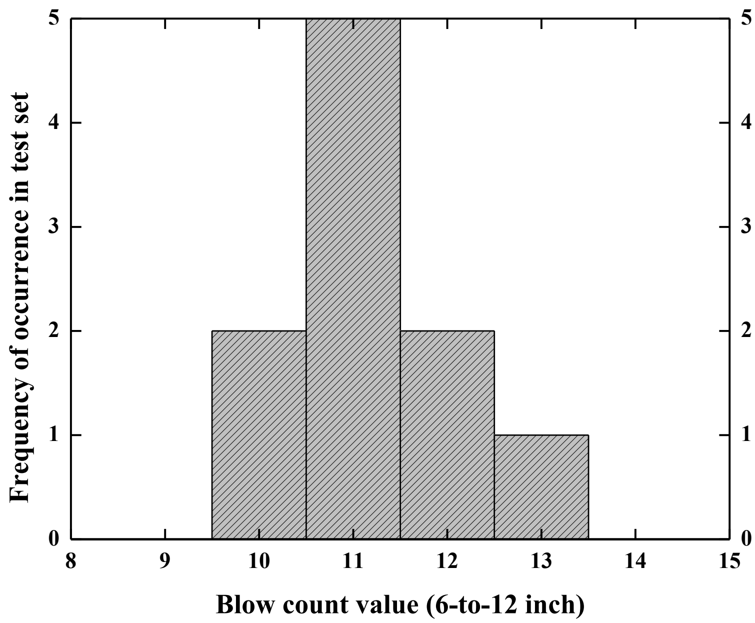


(b)

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

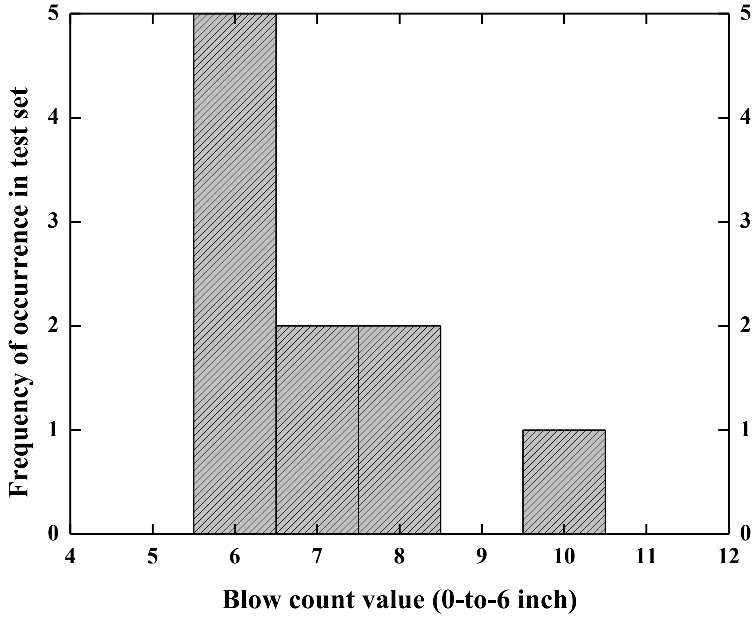


(a)

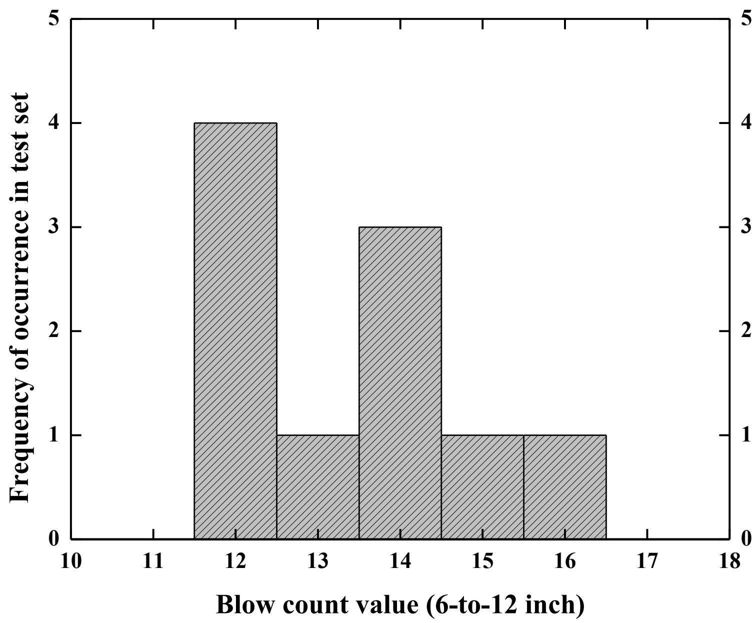


(b)

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

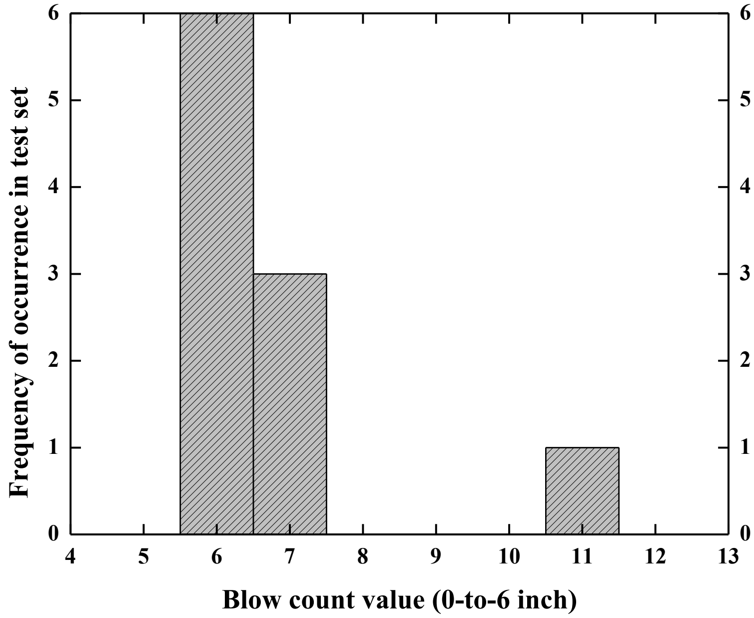


(a)

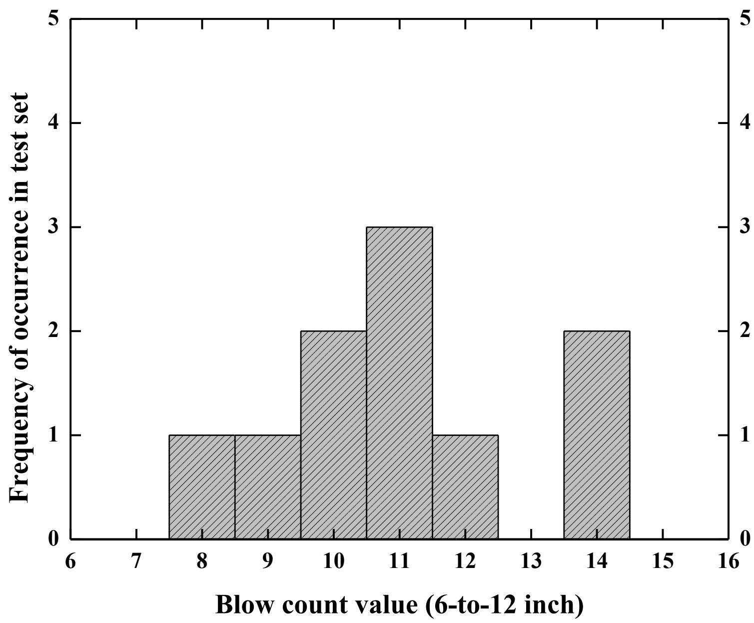


(b)

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

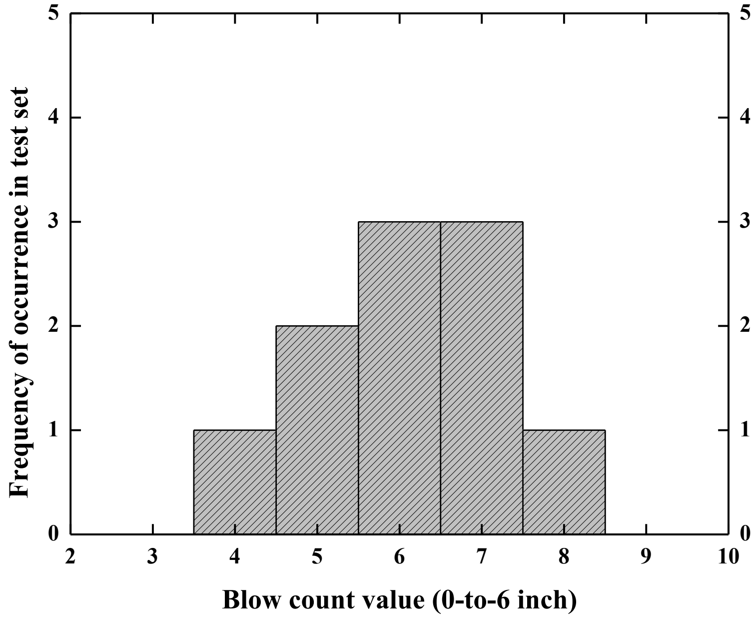


(a)

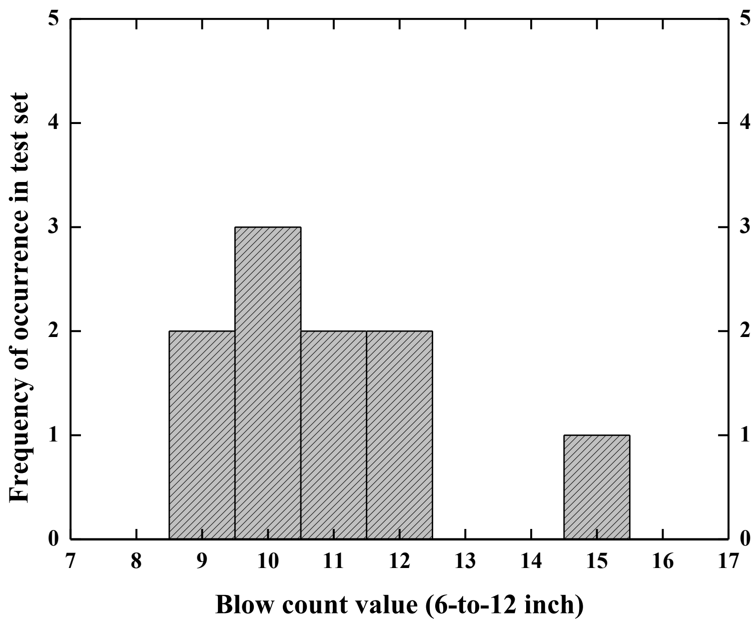


(b)

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

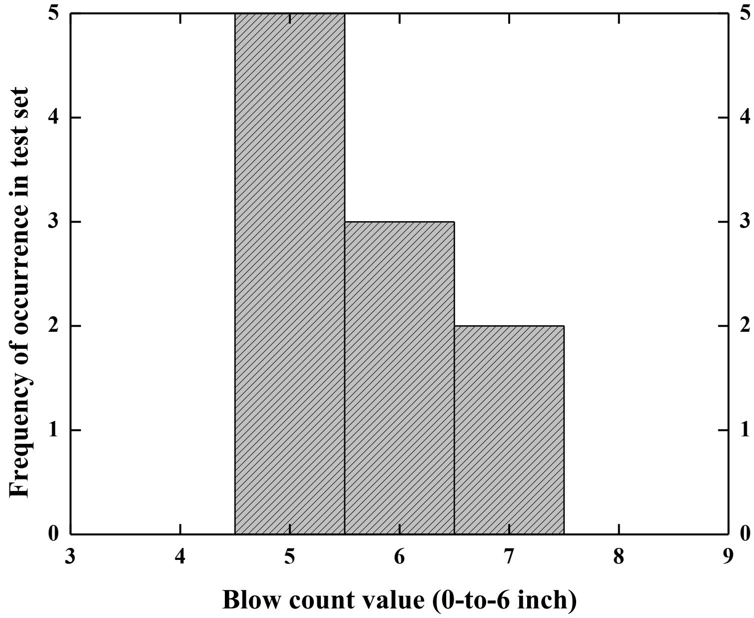


(a)

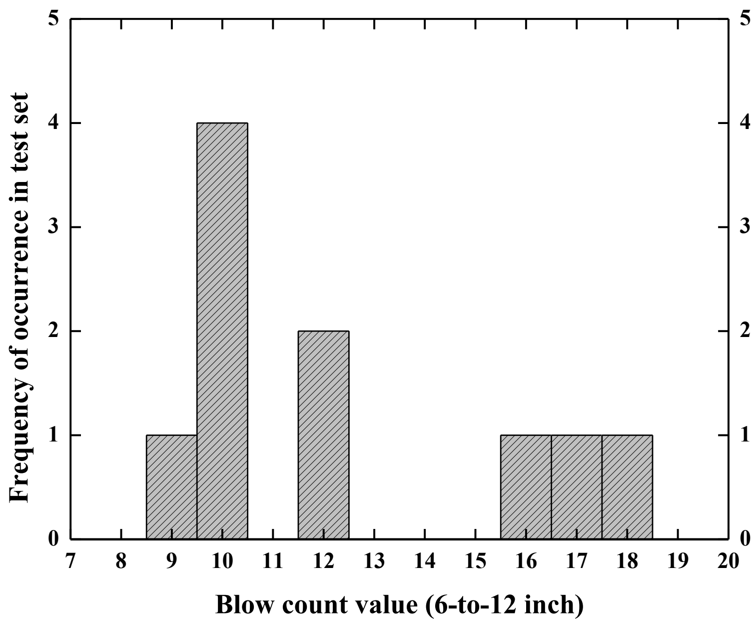


(b)

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

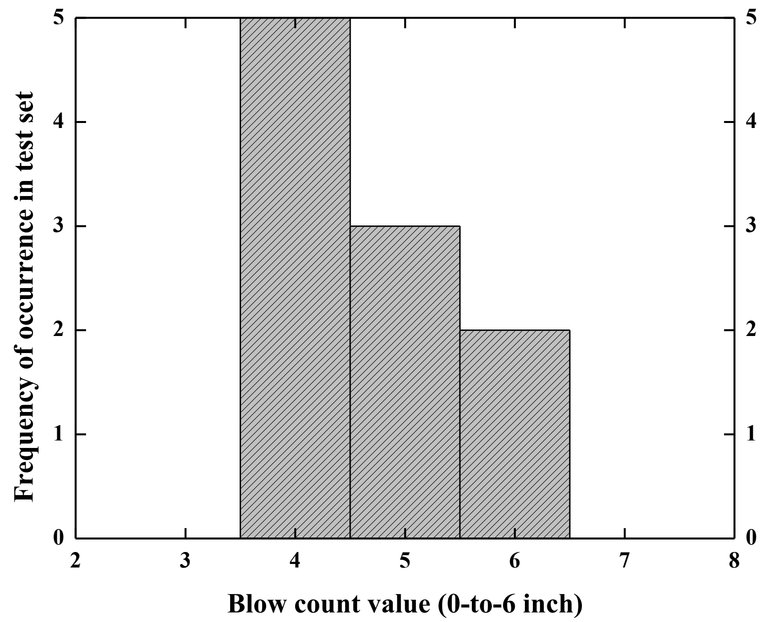


(a)

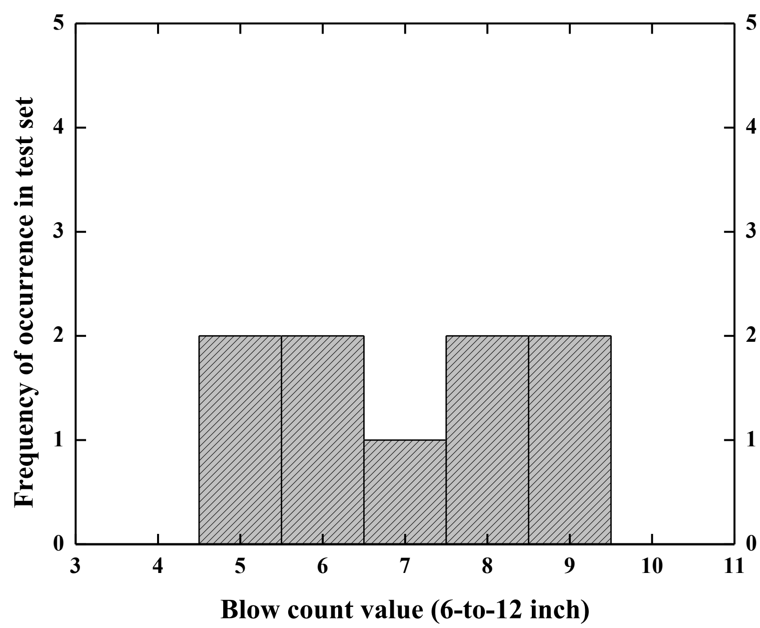


(b)

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

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.

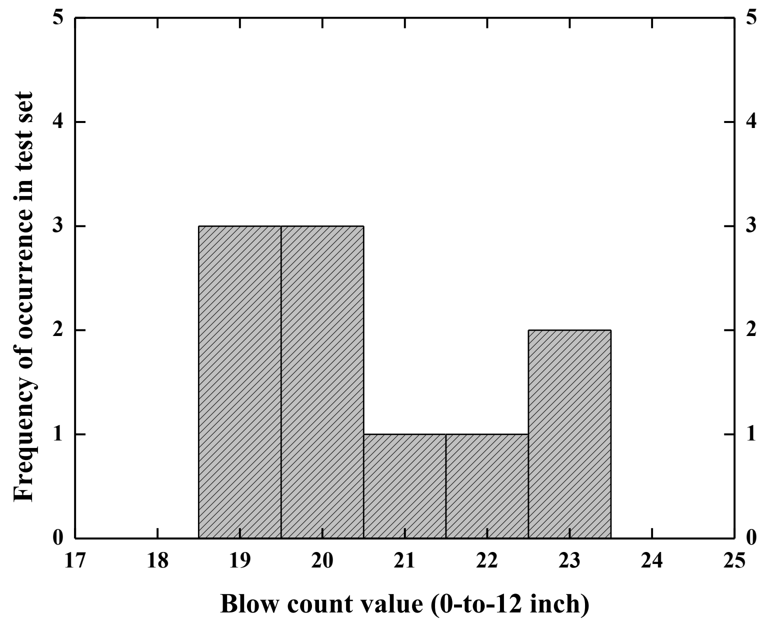


Figure B.1 Frequency vs DCP blow count for DCP-C-A1-1-DE-1 (0-to-12 inch penetration)

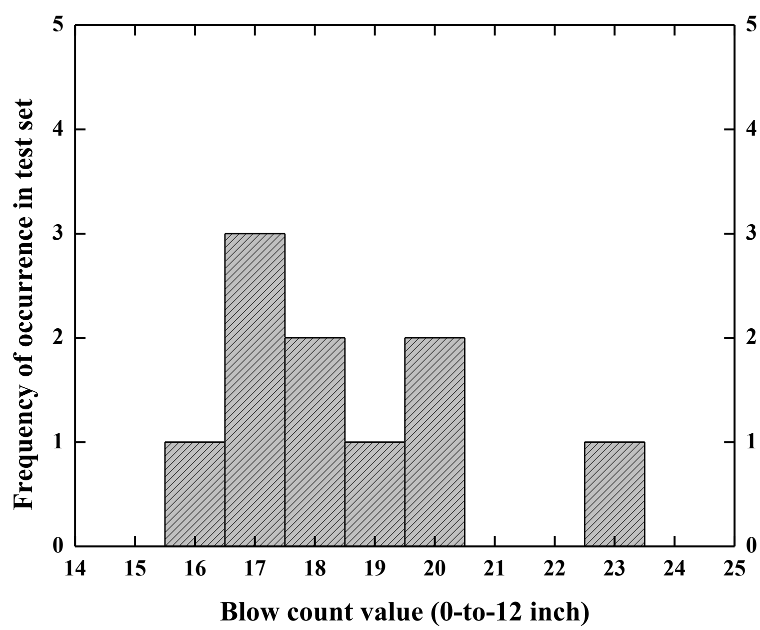


Figure B.2 Frequency vs DCP blow count for DCP-C-A1-1-DE-2 (0-to-12 inch penetration)

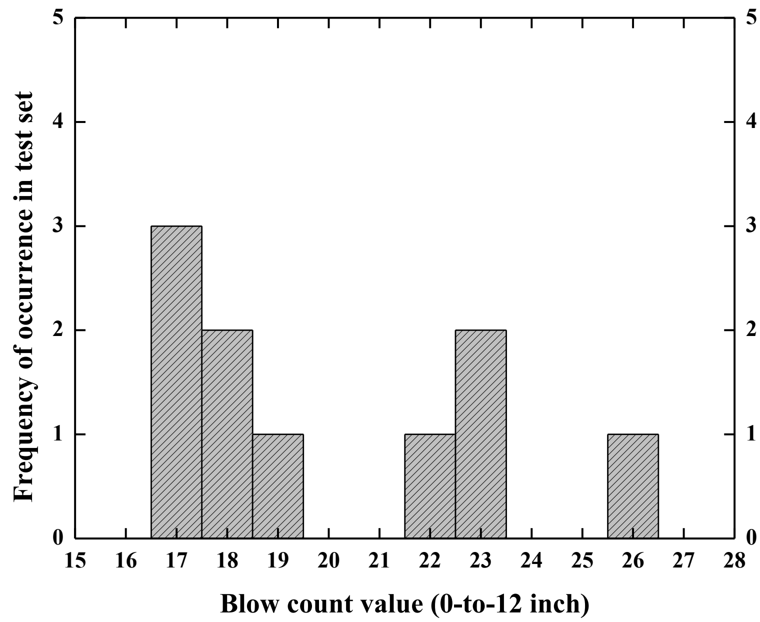


Figure B.3 Frequency vs DCP blow count for DCP-C-A1-1-DE-3 (0-to-12 inch penetration)

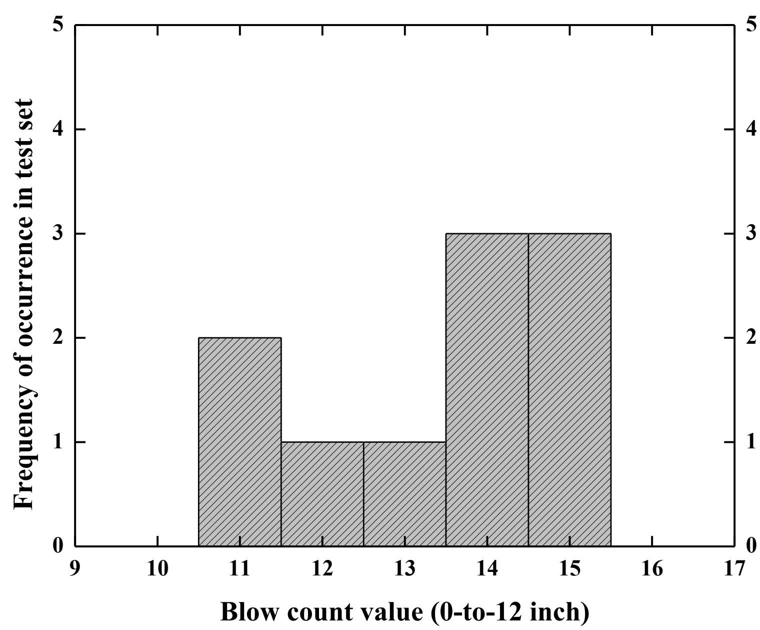


Figure B.4 Frequency vs DCP blow count for DCP-C-A1-1-DE-4 (0-to-12 inch penetration)

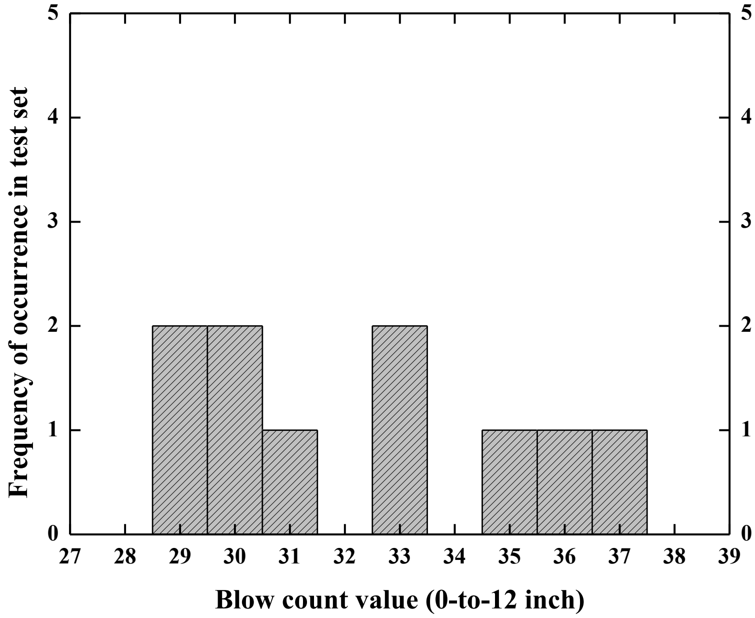


Figure B.5 Frequency vs DCP blow count for DCP-C-A1-1-DE-5 (0-to-12 inch penetration)

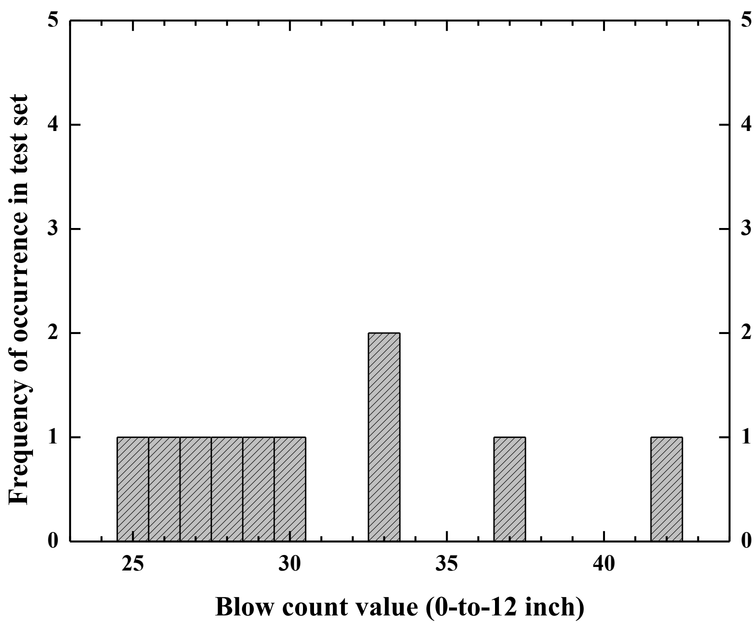


Figure B.6 Frequency vs DCP blow count for DCP-C-A1-1-DE-6 (0-to-12 inch penetration)

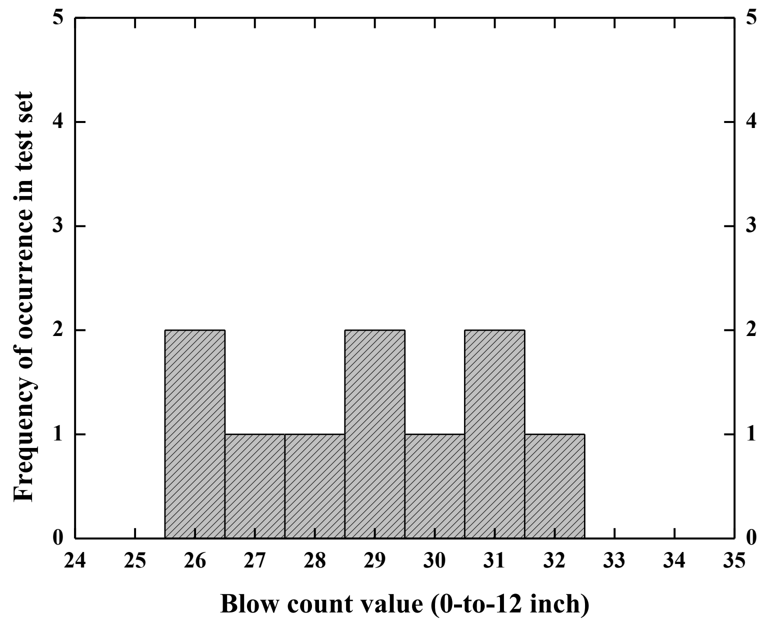


Figure B.7 Frequency vs DCP blow count for DCP-C-A1-1-DE-7 (0-to-12 inch penetration)

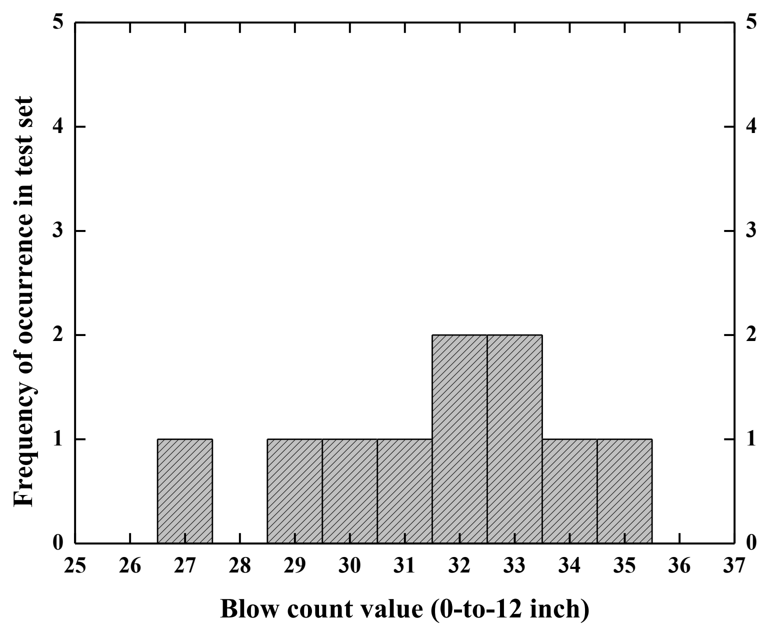


Figure B.8 Frequency vs DCP blow count for DCP-C-A1-1-DE-8 (0-to-12 inch penetration)

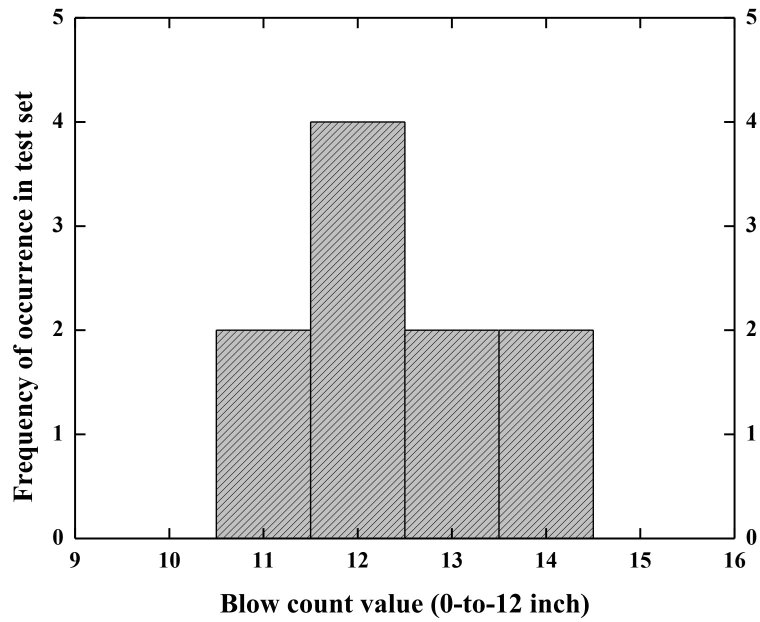


Figure B.9 Frequency vs DCP blow count for DCP-C-A1-1-DE-9 (0-to-12 inch penetration)

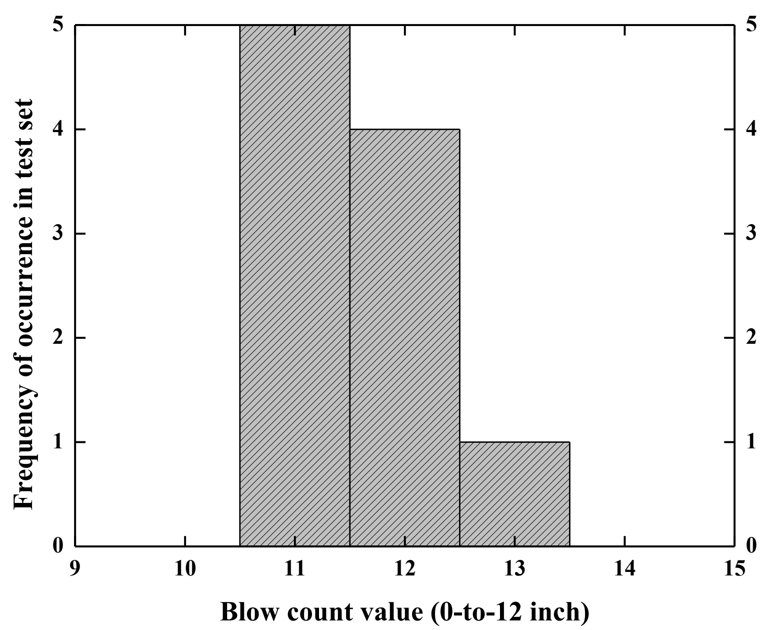


Figure B.10 Frequency vs DCP blow count for DCP-C-A1-1-DE-10 (0-to-12 inch penetration)

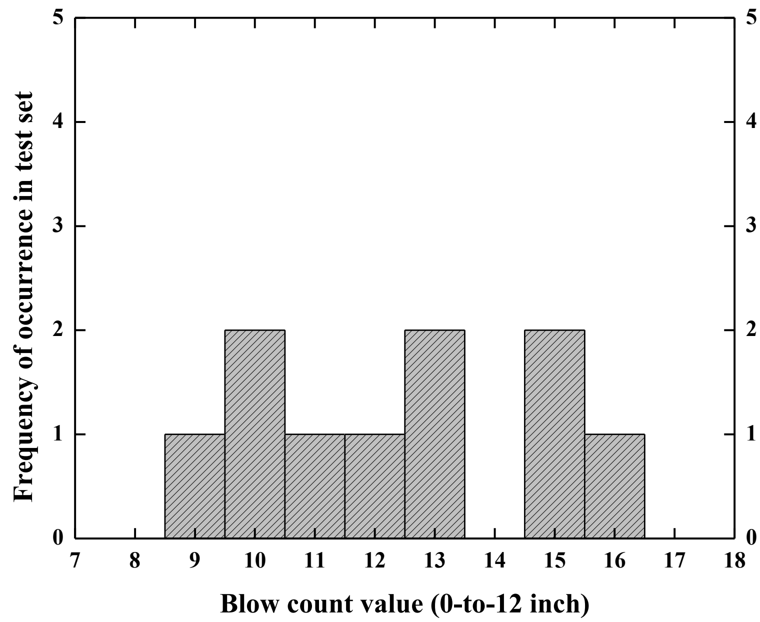


Figure B.11 Frequency vs DCP blow count for DCP-C-A1-1-DE-11 (0-to-12 inch penetration)

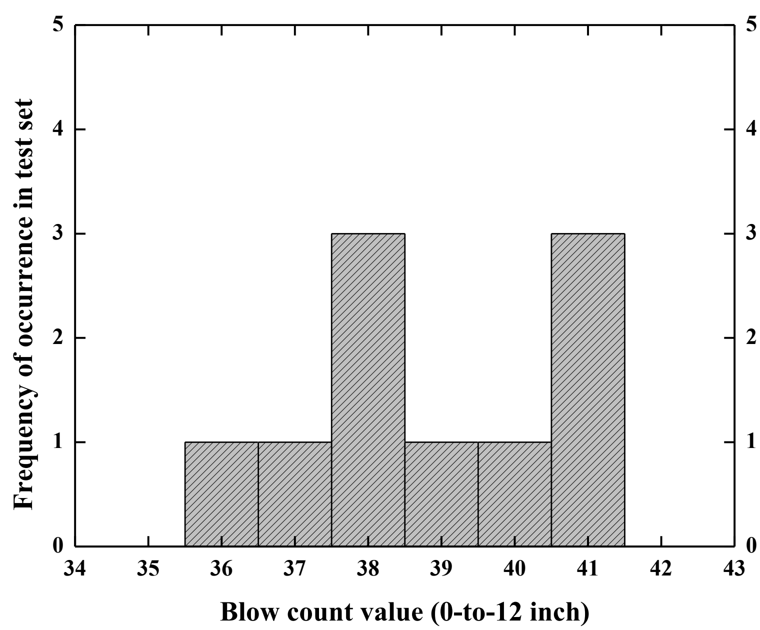


Figure B.12 Frequency vs DCP blow count for DCP-C-A1-2-DE-1 (0-to-12 inch penetration)

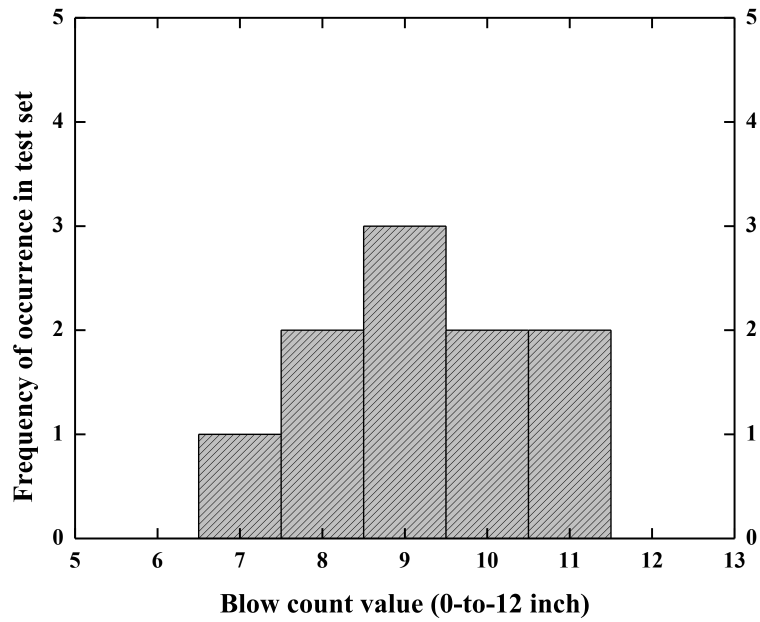


Figure B.13 Frequency vs DCP blow count for DCP-C-A1-2-DE-2 (0-to-12 inch penetration)

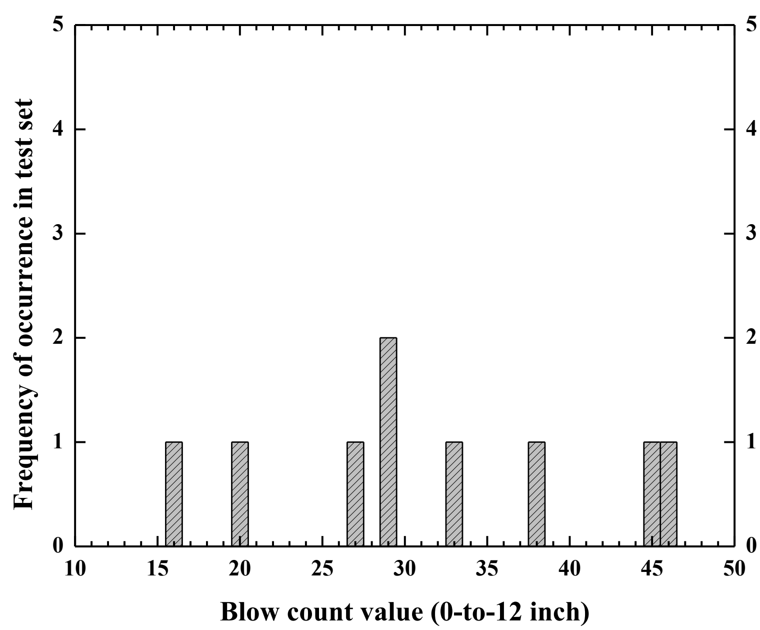


Figure B.14 Frequency vs DCP blow count for DCP-C-A1-2-DE-3 (0-to-12 inch penetration)

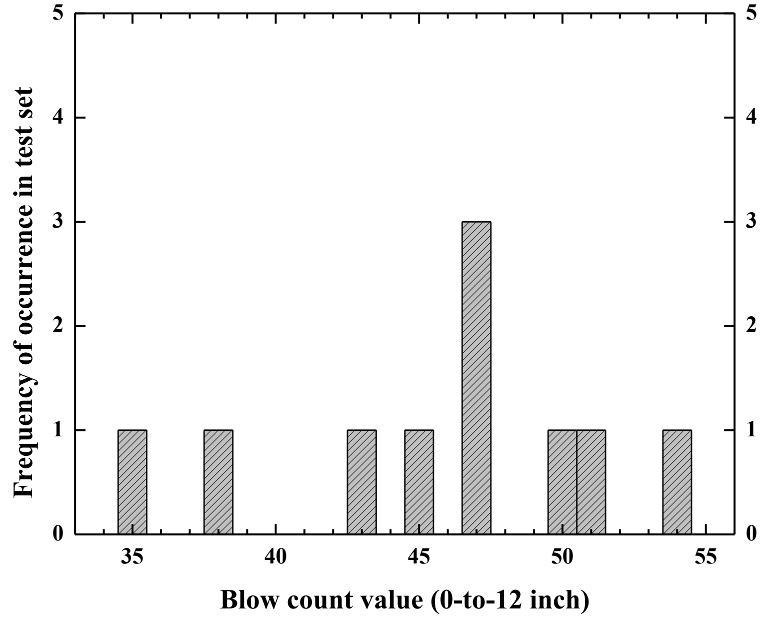


Figure B.15 Frequency vs DCP blow count for DCP-C-A1-2-DE-4 (0-to-12 inch penetration)

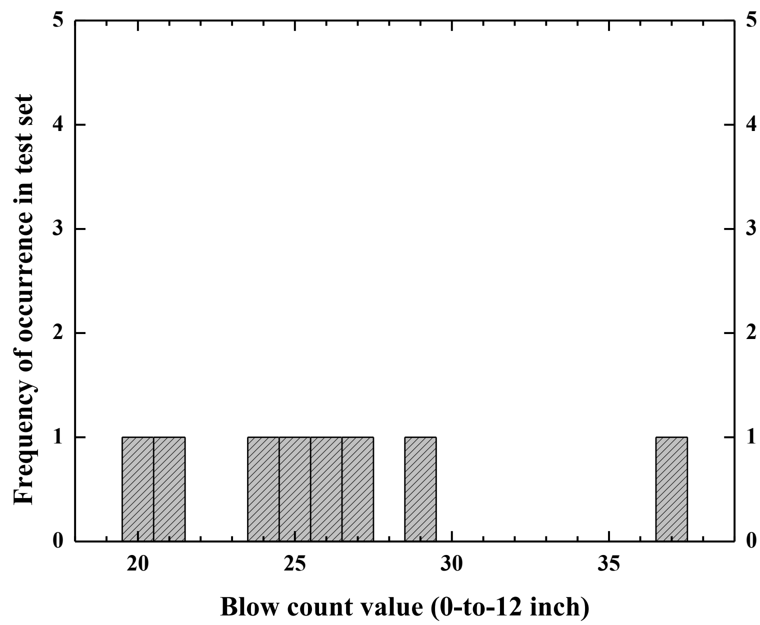


Figure B.16 Frequency vs DCP blow count for DCP-C-A1-2-DE-5 (0-to-12 inch penetration)

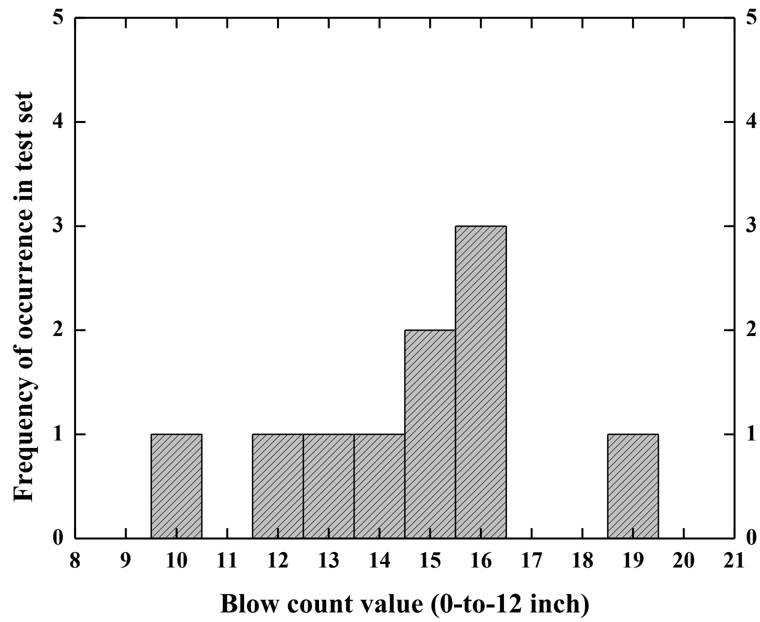


Figure B.17 Frequency vs DCP blow count for DCP-C-A1-2-DE-6 (0-to-12 inch penetration)

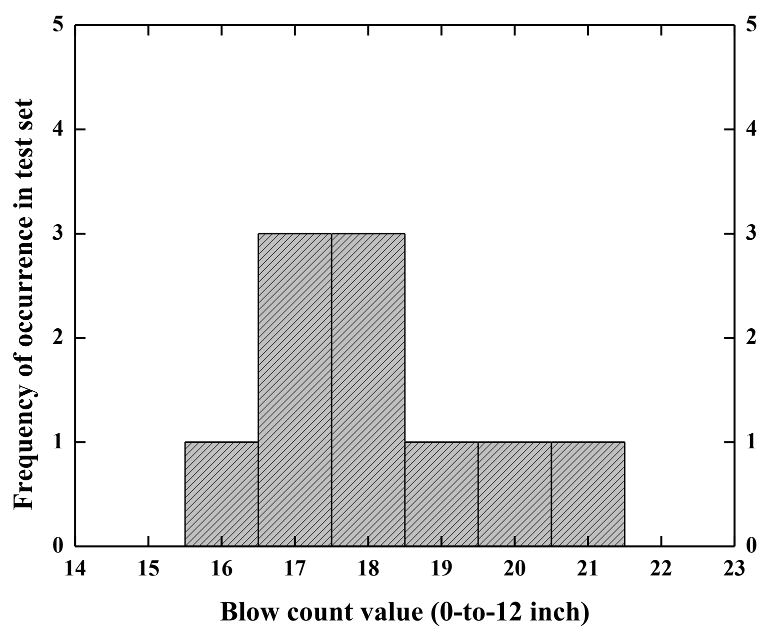


Figure B.18 Frequency vs DCP blow count for DCP-C-A1-2-DE-7 (0-to-12 inch penetration)

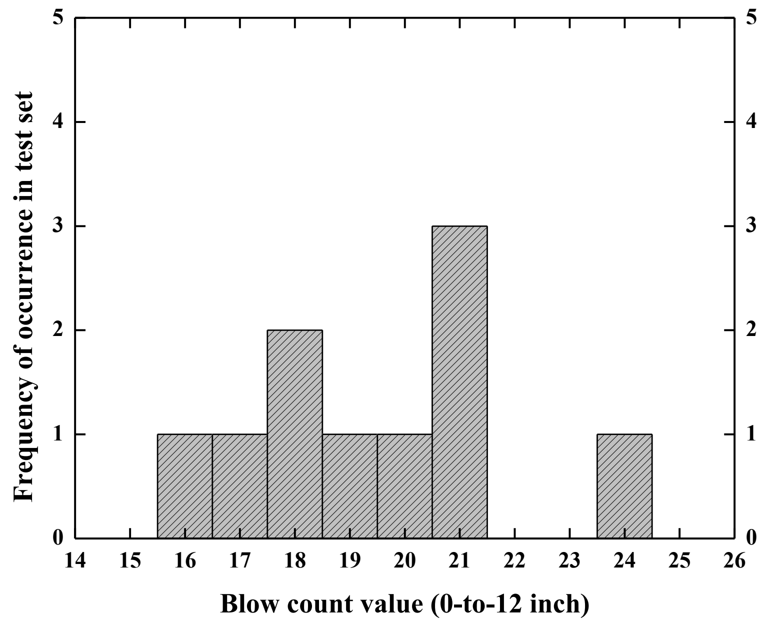


Figure B.19 Frequency vs DCP blow count for DCP-C-A1-2-DE-8 (0-to-12 inch penetration)

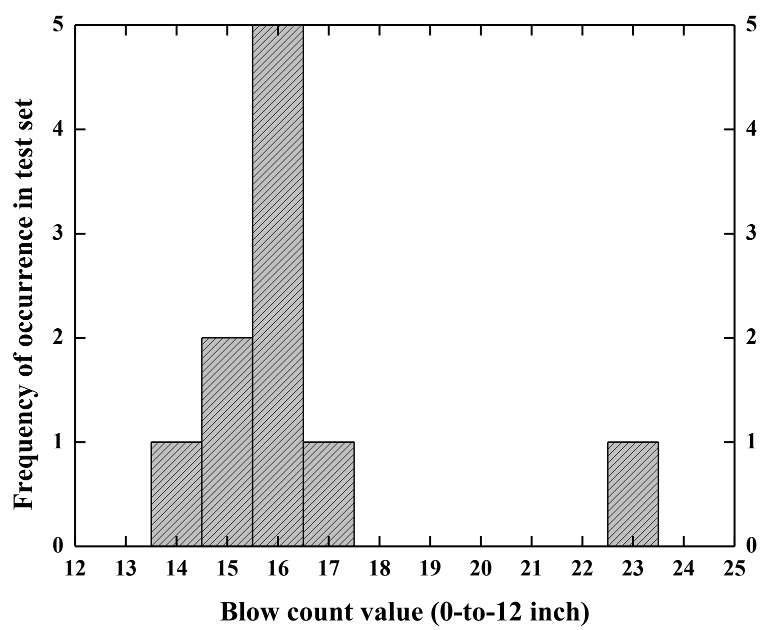


Figure B.20 Frequency vs DCP blow count for DCP-C-A1-2-DE-9 (0-to-12 inch penetration)

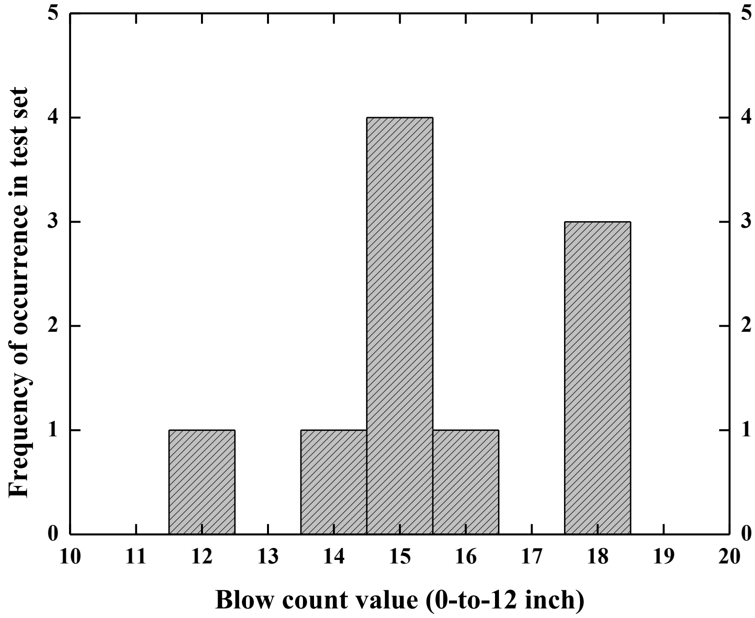


Figure B.21 Frequency vs DCP blow count for DCP-C-A1-2-DE-10 (0-to-12 inch penetration)

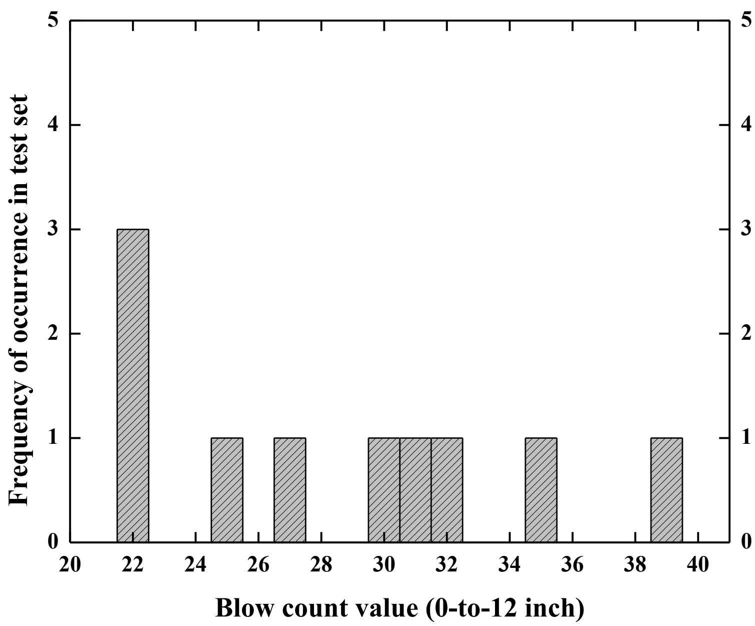


Figure B.22 Frequency vs DCP blow count for DCP-C-A1-3-DE-1 (0-to-12 inch penetration)

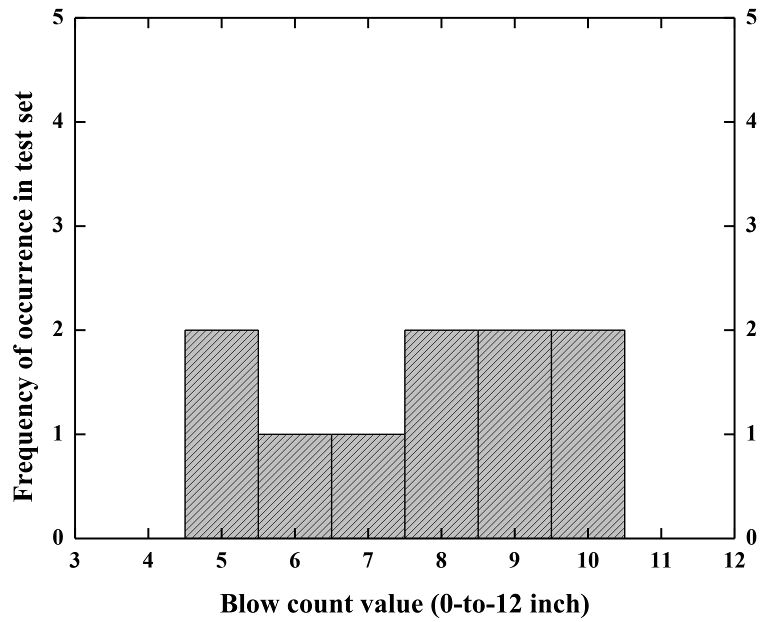


Figure B.23 Frequency vs DCP blow count for DCP-C-A1-3-KO-1 (0-to-12 inch penetration)

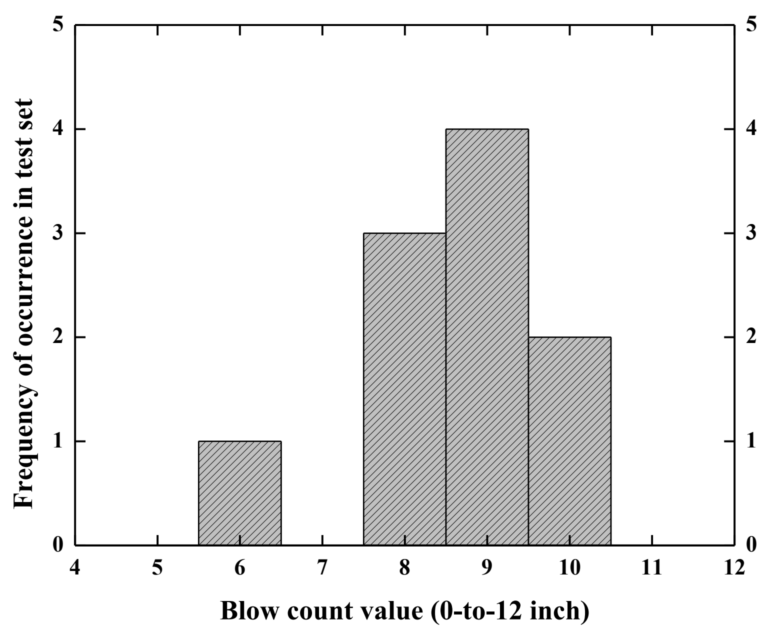


Figure B.24 Frequency vs DCP blow count for DCP-C-A1-3-KO-2 (0-to-12 inch penetration)

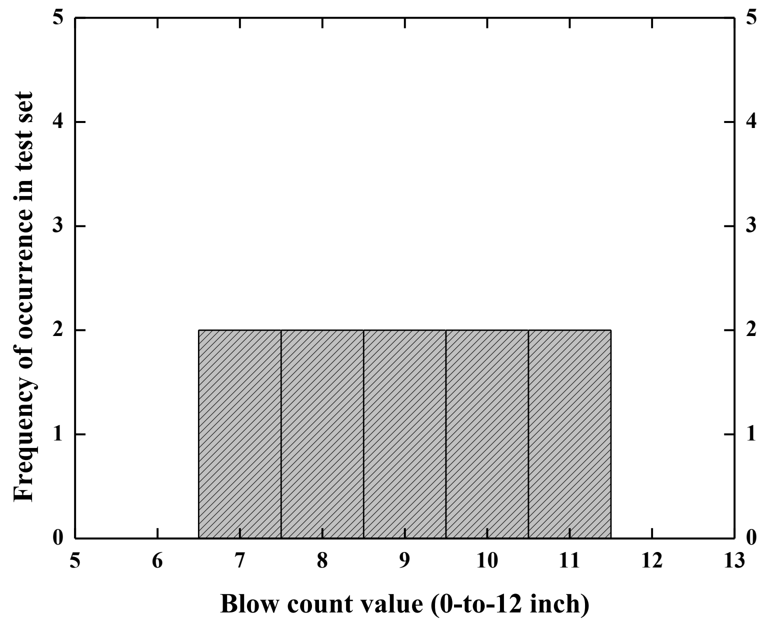


Figure B.25 Frequency vs DCP blow count for DCP-C-A1-3-KO-3 (0-to-12 inch penetration)

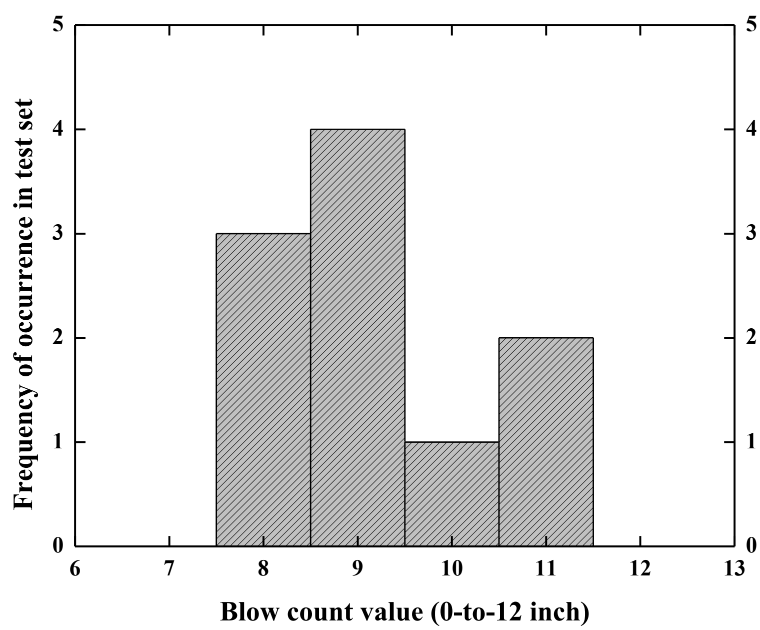


Figure B.26 Frequency vs DCP blow count for DCP-C-A1-3-KO-4 (0-to-12 inch penetration)

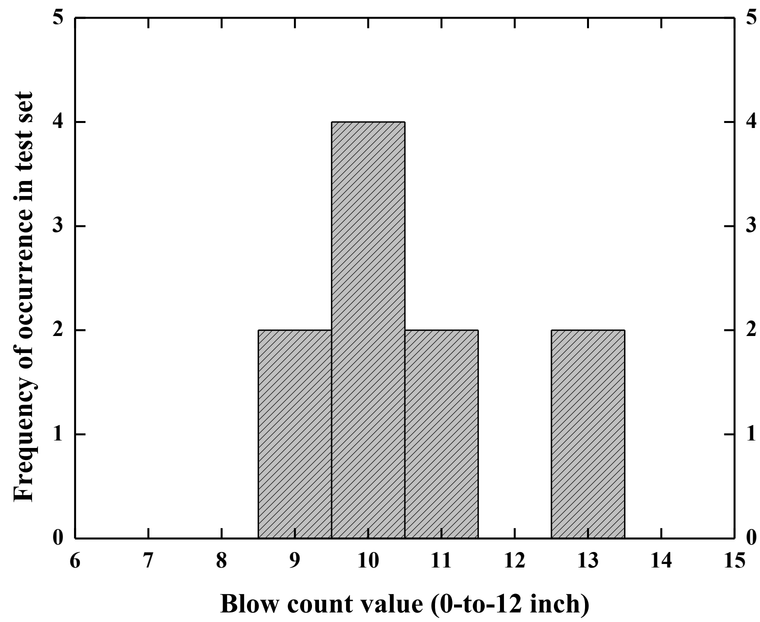


Figure B.27 Frequency vs DCP blow count for DCP-C-A1-3-KO-5 (0-to-12 inch penetration)

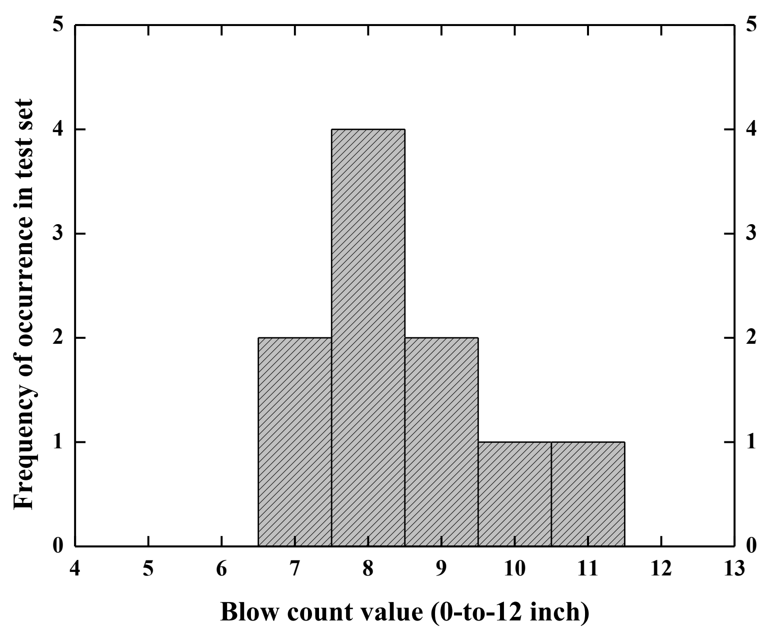


Figure B.28 Frequency vs DCP blow count for DCP-C-A1-3-KO-6 (0-to-12 inch penetration)

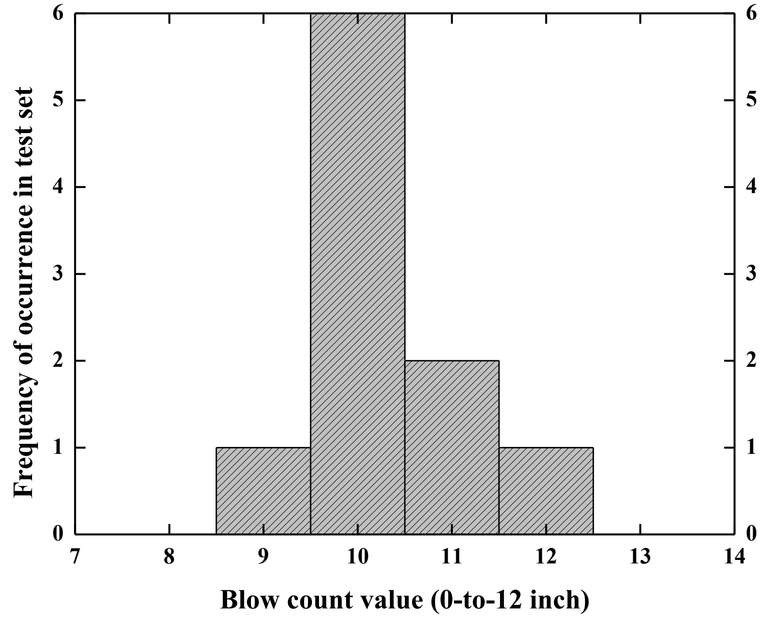


Figure B.29 Frequency vs DCP blow count for DCP-C-A1-3-KO-7 (0-to-12 inch penetration)

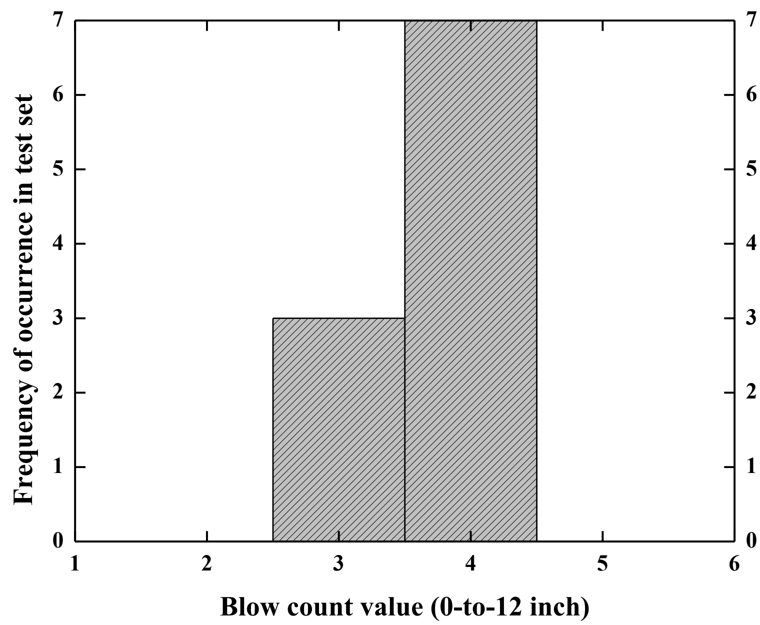


Figure B.30 Frequency vs DCP blow count for DCP-C-A1-3-KO-8 (0-to-12 inch penetration)

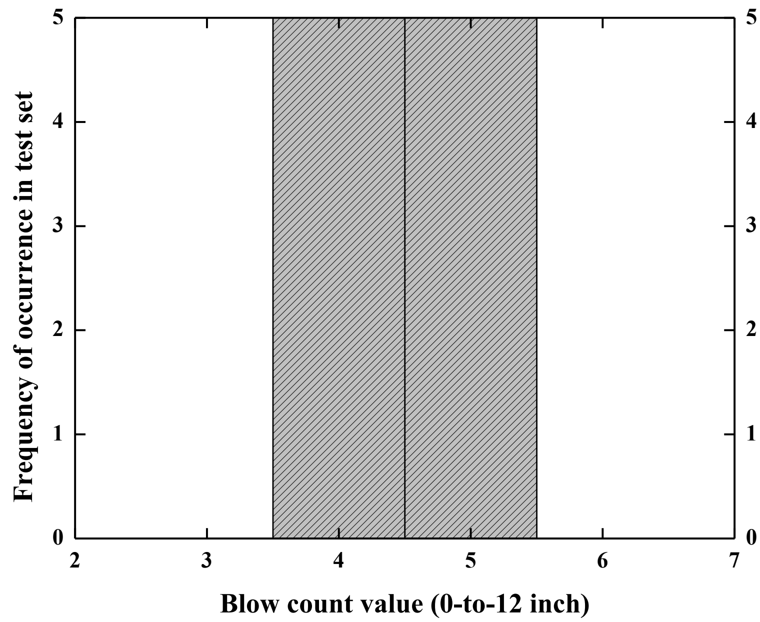


Figure B.31 Frequency vs DCP blow count for DCP-C-A1-3-KO-9 (0-to-12 inch penetration)

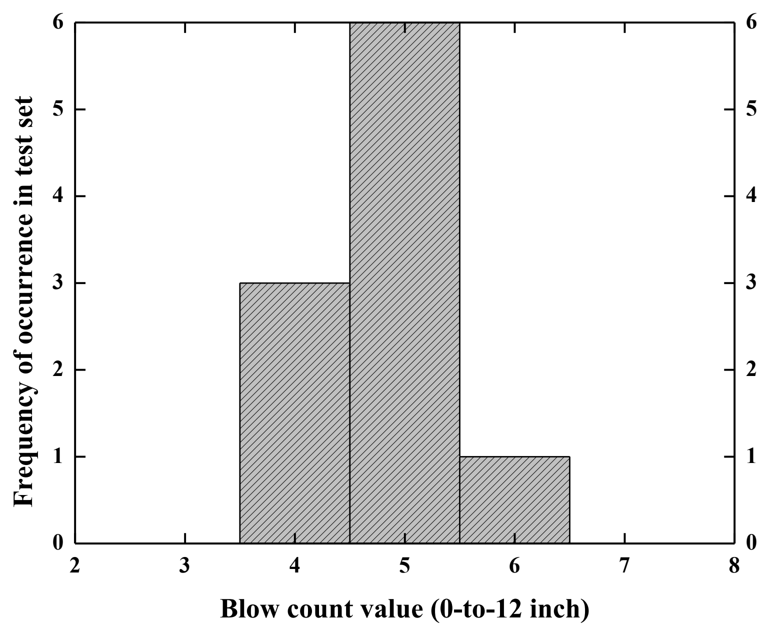


Figure B.32 Frequency vs DCP blow count for DCP-C-A1-3-KO-10 (0-to-12 inch penetration)

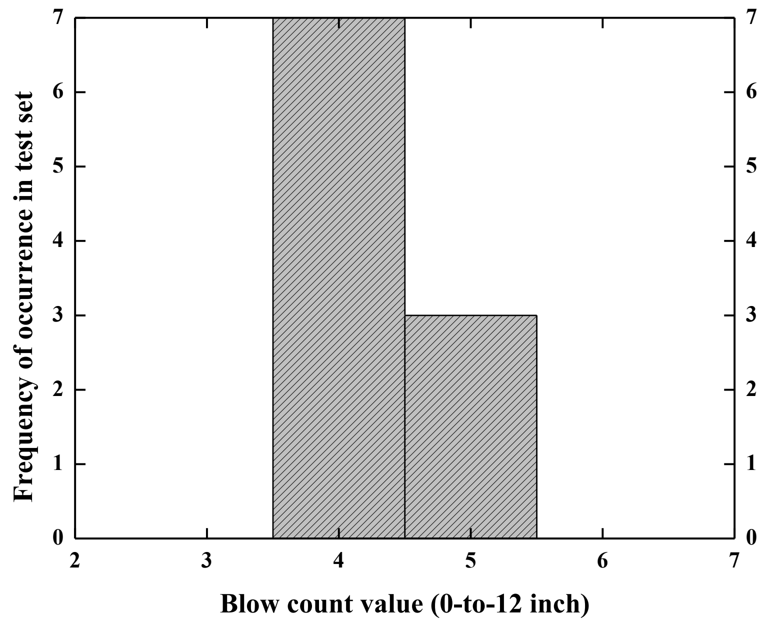


Figure B.33 Frequency vs DCP blow count for DCP-C-A1-3-KO-11 (0-to-12 inch penetration)