ASSESSMENT OF DENSITY AND SHEAR STRENGTH OF EASTERN SAUDI SANDS USING DYNAMIC CONE PENETRATION TESTING (DCPT)

BY

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بسم الله الرحمن الرحيم

IN THE NAME OF ALLAH, MOST GRACIOUS, MOST MERCIFUL

This Humble Work Is

Dedicated to My Beloved

Mother and Wife

and to All My Family Members

in Admiration and Affection

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All thanks and merits are to ALLAH (subhana wa taala) for his guidance and continuous support to start and complete this work. I really would like to first thank my mother who always respects my ideas and allows me to find my way. I would also like to thank my uncle, Abdullah A. Ba-Ghomyan, who was the reason I am a civil engineer.

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LIST OF ABBREVIATIONS

CBR	:	California Bearing Ratio
C_c	:	Coefficient of Curvature
CC	:	Cement Content
CPT	:	Cone Penetration Test
C_u	:	Coefficient of Uniformity
DCPI	:	Dynamic Cone Penetration Index
DCPT	:	Dynamic Cone Penetration Test
D_r	:	Relative Density
E_s	:	Modulus of Subgrade Reaction
FWD	:	Falling Weight Deflectometer
G_s	:	Specific Gravity
GRP	:	Glass Reinforced Plastic
H	:	Height
LC	:	Lime Content
M_R	:	Resilient Modulus
PI	:	Plasticity Index
PLT	:	Plate Load Test
R^2	:	Coefficient of Determination
SPT	:	Standard Penetration Test
SSG	:	Soil Stiffness Gauge
t	:	Curing Time
w	:	Water Content
ϕ	:	Angle of Internal Friction
γ_d	:	Dry Density

ABSTRACT

Full Name : ABDULRAHMAN MOHAMMED ABDULRAHMAN HAMID

Thesis Title : ASSESSMENT OF DENSITY AND SHEAR STRENGTH OF EASTERN SAUDI SANDS USING DYNAMIC CONE PENETRATION TESTING (DCPT)

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Rapid and accurate in-situ measurement of soil properties is still a challenge facing the construction industry and there is a need for new and advanced devices and methods. Dynamic cone penetration test (DCPT) is an effective device used for field quality assessment of soils. DCPT was used to predict the engineering properties of sand in which it is difficult to obtain undisturbed sand samples. Furthermore, it is very difficult to perform the conventional density tests, such as the sand replacement (sand cone) method, especially when loose or submerged sandy soil is encountered.

This study was aimed to evaluate the effect of dry density, silt content and water level on the shear strength and penetration resistance using DCPT for sands. The soil sample was brought from a site near King Fahd University of Petroleum & Minerals and classified as poorly graded sand (SP) according to USCS. DCPTs were performed on sand samples with different silt content (1%, 4% and 8%) and different relative densities (40%, 60% and 90%) where it was compacted in a large scale mold (1600 mm in diameter and 1500 mm height). Test results indicated that the increase in the dry density and silt content increased the penetration resistance which means a decrease in the dynamic cone penetration index (DCPI). Further, the results showed that variations in water table level had a significant effect on DCPT results. In addition, the effective stress of sand was changed due to pore water pressure in which the penetration resistance revealed out a significant influence on the stiffness of sand. Regressions were developed to correlate the various parameters.

In addition to the laboratory testing, a comprehensive field testing program conducted in Al-Jubail and Rass Al-Khair was utilized to validate laboratory testing and evaluate the potential use of the DCPT to assess the compaction during the construction of backfill. DCPT has proven to be an effective tool in the assessment of compaction and density of sand backfill, in the field.

MASTER OF SCIENCE DEGREE

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DHAHRAN, SAUDI ARABIA

ملخص الرسالة

الاسم الكامل: عبدالرحمن محمد عبدالرحمن حميد

عنوان الرسالة: تقييم كثافة وقوة رمال شرق المملكة باستخدام اختبار الاختراق المخروطي الديناميكي

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يمثل قياس خواص التربة في الموقع بشكل سريع ودقيق تحديا امام الصناعة الانشائية مما زاد الاحتياج لإستخدام اجهزة متطورة واساليب جديدة ومبتكرة. ويعتبر اختبار الاختراق المخروطي الديناميكي (DCPT) فعال لاستكشاف وتقييم التربة في العديد من المواقع. ان جهاز الاختراق المخروطي الديناميكي يستخدم بشكل واسع في تقييم خواص التربة الهندسية دون التسبب باي اخلال بخواصها الهندسية، حيث لوحظ انه من الصعب اجراء اختبارات الكثافة التقليدية على الرمل متدني الكثافة او المغمور بالمياه.

تهدف هذه الدراسة الى تقييم تاثير الكثافة الجافة للرمل ومحتوى الطمي ومستوى ارتفاع المياة الجوفية على قياس القوة ومقاومة الاختراق للرمل وذلك باستخدام اختبارالاختراق المخروطي الديناميكي، وقد تم احضار عينة الرمل من موقع قريب من جامعة الملك فهد للبترول والمعادن بالظهران وتم اجراء الاختبارات الاولية للتربة حيث تم تصنيفها بانها ذات تدرج حبيبي منتظم، واجريت اختبارات الاختراق المخروطي الديناميكي على عينات الرمل التي تحتوي على نسب مختلفة (1%، 4%، 8%) من الطمي (silt) وكثافة نسبية مختلفة (40% ، 60%، 90%)، حيث تم تجهيز العينات بعناية فائقة في قالب كبير بقطر 1600 مم وارتفاع 1500 مم. وقد دلت نتائج الاختبارات على ان الزيادة في الكثافة الجافة ومحتوى الطمي يسببان زيادة في مقاومة الاختراق (مما يعني انخفاضا في مؤشر الاختراق المخروطي الديناميكي). كما اظهرت النتائج ان التغيرات في مستوى المياة الجوفية لها تاثير كبير وفعال على مؤشر الاختراق المخروطي الديناميكي وذلك لحدوث تغير في ضغط المياة في مسام الرمل مما ادى الى تاثير كبير على صلابة التربة والذي تبين من المقاومة للاختراق.

إضافة الى ذلك فقد تم الاستفادة من اختبارات ميدانية في موقعيين احدهما في الجبيل والاخر في راس الخير وذلك لتأكيد دقة النتائج المعملية و لتقييم إمكانية استخدام اختبار الاختراق المخروطي الديناميكي لتقييم دمك التربة اثناء اعمال الردم، وقد اثبتت النتائج ان اختبار الاختراق المخروطي الديناميكي هو اداة فعالة لتقييم الدمك والكثافة في الموقع.

درجة الماجستير

قسم الهندسة المدنية والبيئية

جامعة الملك فهد للبترول والمعادن

الظهران، المملكة العربية السعودية

CHAPTER 1

INTRODUCTION

1.1 General

Density is referred to as the ratio of the mass to the volume. There are various methods to determine in-situ density of soil such as the sand replacement method (sand cone), balloon method, nuclear gauge method, and electrical resistivity method. The soil density is one of the essential engineering properties of the soil that helps in the engineering design such as the design of foundations, dams, retaining walls, embankment, etc. The nuclear gauge is nowadays the most common device used to measure in-situ density of soil because it provides rapid and accurate results with the minimal effect of human error. Unfortunately, there are many restricted regulations for using such a device with a nuclear source due to its health negative effects (Adams et al., 2007). Furthermore, the nuclear gauge has a limited reach of about 300 mm only.

Because of the lack of cohesion, it is essentially impossible to obtain undisturbed samples of sand. The dynamic cone penetration test (DCPT) has been used widely for field exploration and quality assessment of subsoil layers. DCP testing can be used in the characterization of subgrade and base material properties in many ways. Perhaps the most important advantage of the DCP device related to its ability to provide a continuous record of relative soil strength with depth (Burnham et al., 1993). Dynamic cone penetrometer device is distinguished by its economy and simplicity to operate and its superiority to provide repeatable results and rapid property assessment. DCPT has the main features of both the CPT and the SPT (Salgado and Yoon, 2003).

The main objective of this research was to develop proper correlations for density and shear strength of eastern Saudi sand with DCPT results. Further, research was aimed to investigate the effect of silt content and water table levels on dynamic cone penetration test results. In addition, this study was aimed to evaluate the potential use of DCPT to assess compaction during construction.

1.2 Significance of Investigation

Saudi Arabia is witnessing unparalleled development of all types of construction, particularly in the various urban and industrial areas, especially in the Eastern Province of Saudi Arabia. In recent years, there is a tendency to attract investments from international organizations in major projects, especially in the Industrial City of Jubail. As a result of this massive development, special problems in the soil have emerged. For example, some projects facing the nightmare of poor soils to withstand the unprecedented loads which force the practicing engineers to develop radical solutions to eradicate this problem by stabilizing the soil or replace the existing soil by another one that has more strength and stiffness. As a result, it is difficult to check the physical properties, such as density of the entire area, by conventional methods, and how to assess their properties in-situ instead of getting the soils to the laboratory. Therefore, it is important to find out a way through which we can assess the density and strength using a simple, effective and inexpensive device.

1.3 Research Objectives

The specific objectives of this investigation were:

1. Developing sufficient data to generate proper correlations for density and shear strength of eastern Saudi sand with dynamic cone penetration test (DCPT) results taking into account the following parameters:
   - Different densities of the sands;
   - Different silt content; and
   - Different water table levels.
2. Evaluating the potential use of the dynamic cone penetration test (DCPT) to assess the compaction during construction.

1.4 Research Methodology

This research included several tasks aimed at achieving the above objectives. First, a comprehensive literature review was conducted to have a good knowledge about dynamic cone penetration test applications and calibration chamber test that was used to develop the correlations for density and shear strength with dynamic cone penetration test results. Based on this literature review, which is presented in Chapter 2, a large scale chamber was manufactured at the Main Workshop at KFUPM.

Details of the laboratory experimental program were presented in Chapter 3. The laboratory research focused on correlation for density and shear strength of sand with dynamic cone penetration test results under controlled laboratory conditions. In addition, the effect of water table and silt content on dynamic cone penetration test operations was investigated. In Chapter 4, field data at two sites was studied to evaluate the potential use of the dynamic cone penetration test (DCPT) to assess the compaction strength during the construction of backfill. Based on laboratory and field tests result, data analysis was discussed in Chapter 5. Figure 1.1 summarizes the research methodology adopted in this investigation.

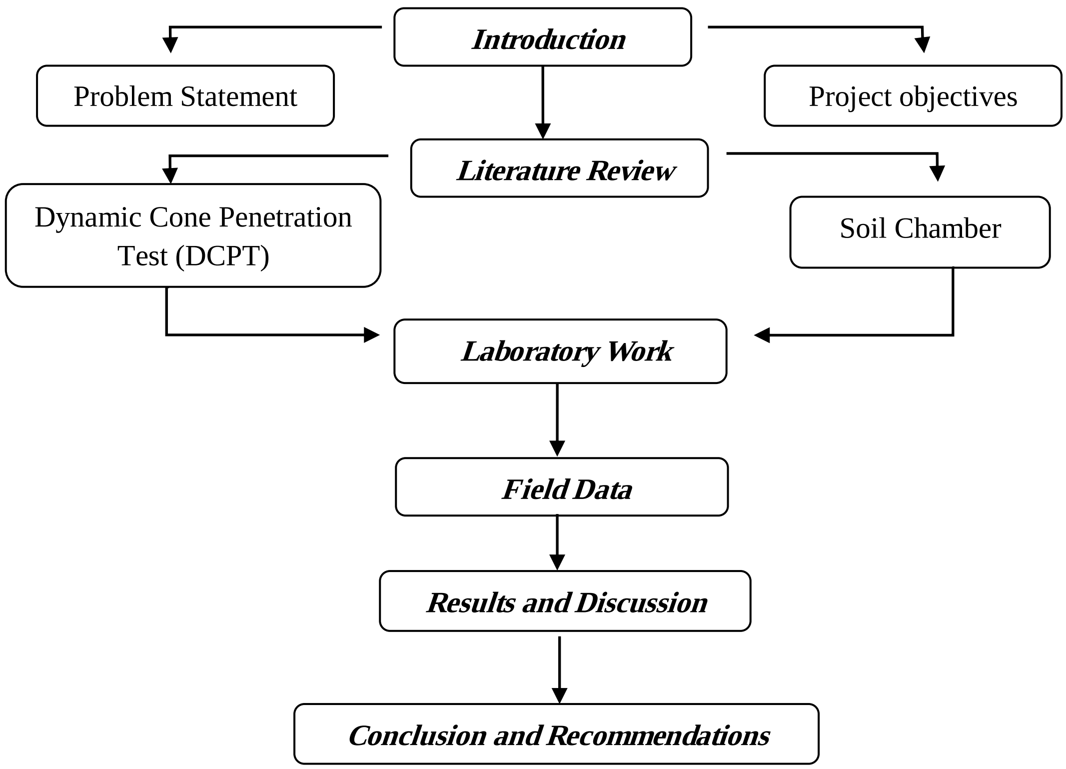


Figure 1-1: Research methodology

CHAPTER 2

LITERATURE REVIEW

2.1 Dynamic Cone Penetration (DCP)

The Dynamic Cone Penetrometer (DCP) is an instrument that can be used in the characterization of subgrade and base material properties in many ways. It can also be used for the assessment of compaction quality for sand backfilling. Perhaps the greatest advantage of the DCP device lies in its ability to provide a continuous record of relative soil strength with depth (Burnham et al., 1993). It is a hand held instrument planned to penetrate soils to depths of 1 m with a 20 mm diameter cone, a 60-degree cone, and a hammer of 8 kg weight, as shown in Figure 2.1 and Figure 2.2. The test has been standardized in ASTM D 6951. Two people are commonly required to force the DCP setup into the soil. However, the manpower can be reduced to one person by using electronic device to record the DCPT data.

DCPT users can observe soil layers by plotting a graph of depth versus penetration index (DCPI). There are many applications of the DCPT including correlations to soil properties such as angle of internal friction, relative density, moisture content, dry density and void ratio.

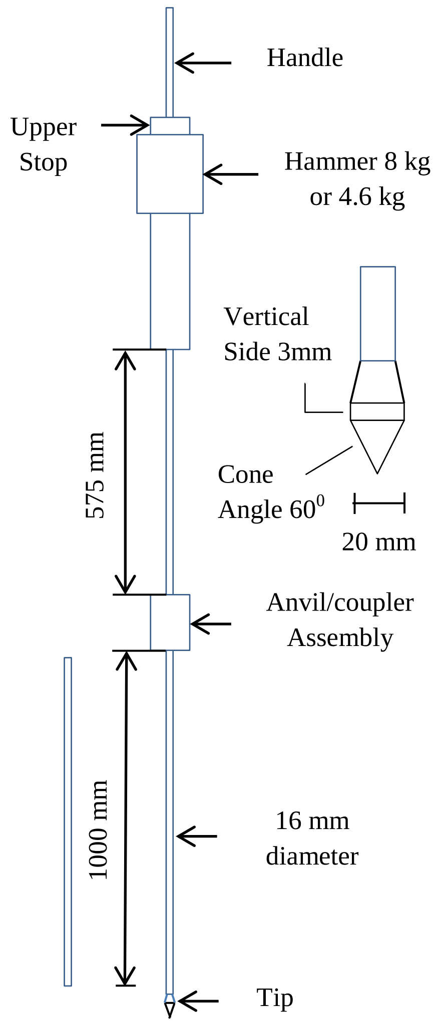
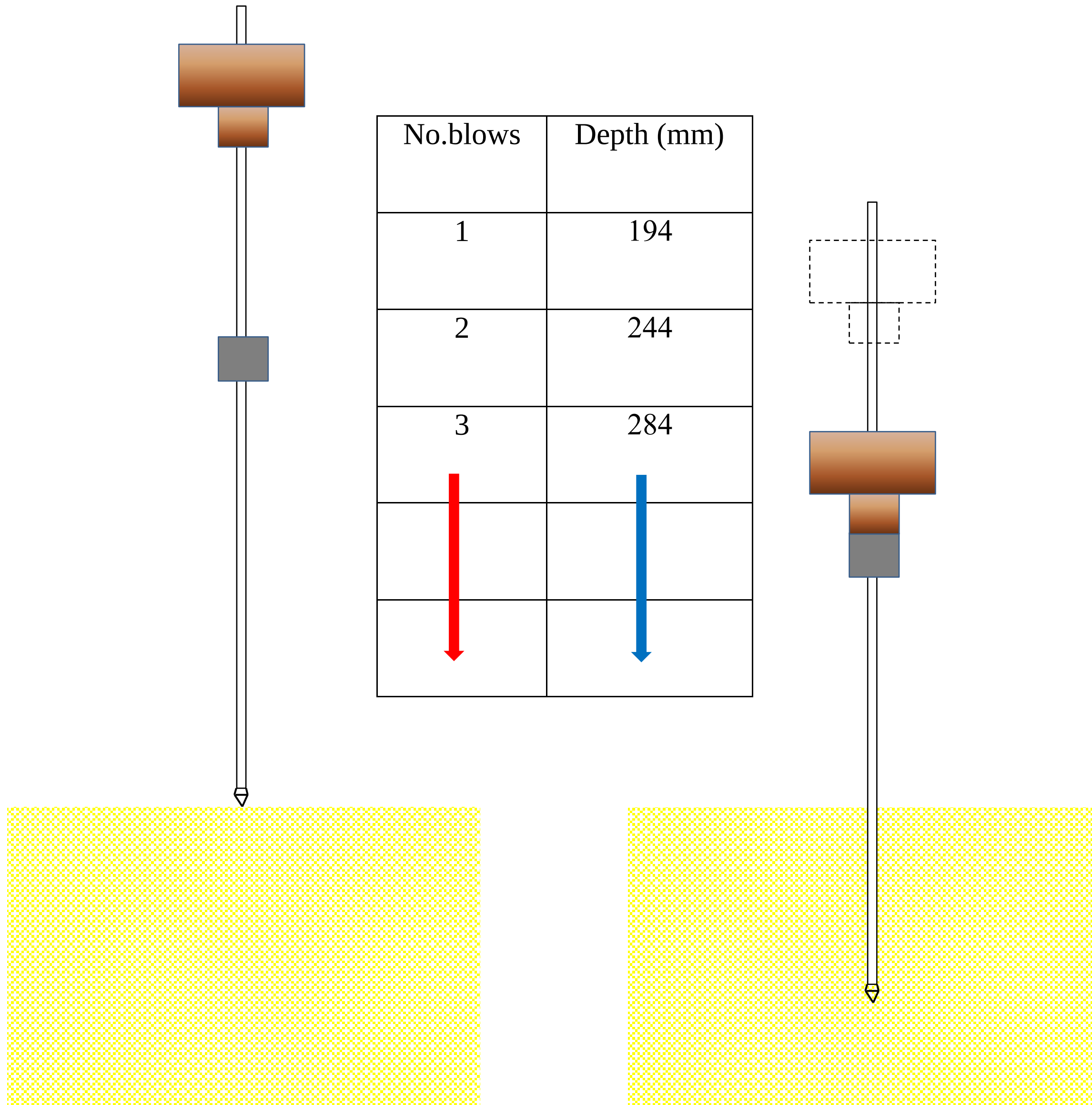


Figure 2-1: Schematic of standard DCP



(a) Before hammer dropping

(b) After hammer dropping

Figure 2-2: Dynamic cone penetration test (ASTM D 6951).

2.1.1 DCP Historical Developments and Applications

Although initial DCP had a 30-degree cone, 60-degree cone become more popular in latest years due to its durability in high-strength materials. The cone angle in the current ASTM D 6951 method is 60 degrees. Some applications of the DCP include correlations to resilient modulus, CBR, unconfined compressive strength, and shear strengths, as well as its use in quality control of compaction of fill and performance evaluation of pavement layers (Amini, 2003).

Scala (1956) presented the dynamic cone penetrometer based on the previous designs in Switzerland. The drop height of hammer was 508 mm and the hammer weight was 9 kg. The cone angle was 30 degrees. Scala penetrometer was used with an extension to a depth of 1.8 m. He introduced the theoretical relationship between the applied energy; soil resistance and penetration rate, and developed the DCP-CBR correlation and used DCP for pavement design.

Gawith and Perrin (1962) reported the use of the same DCP in Australia and using a DCP-CBR correlation curve. In South Africa, Van Vuuren (1969) developed the modern DCP by adjusting the penetrometer, which was in use in Australia. It was made of a 10 kg hammer sliding on a 16 mm rod dropping from 460 mm height. The cone was 20 mm in diameter.

Sowers and Hedges (1966) introduced a DCP device with 6.8 kg hammer, falling 508 mm on the driving rod. The cone point was enlarged to reduce the circumferential resistance. It was used for field exploration and substantiation of soil conditions at individual footings.

Since 1973, the DCP has been used in South Africa (Kleyn, 1975). The type used in South Africa involved an 8 kg hammer dropping from 575 mm height with a 30-degree cone having a diameter (cone) of 20 mm. Kleyn was one of the developers who discovered the linear relationship between DCPI and CBR on a log-log scale.

Kleyn et al. (1982) recorded several applications of the DCP in pavement design, road construction, and pavement evaluation and monitoring. They stated that the DCP measures in-situ CBR rather than laboratory soaked CBR, and that the DCP correlates better with pavement's field performance than the laboratory soaked CBR. In a particular test, it was established that DCP can discover the deterioration of pavement materials very well. However, Kleyn and Savage (1982) excluded the cemented materials since they carry loads and are subject to fatigue damage. The DCP did not evaluate these materials in a way that relate to their behavior in the field. They presented a design and evaluation method for thin surface of unbound gravel pavements using DCP.

Smith and Pratt (1983) developed a correlation between DCPI (30-degree cone, hammer weighted 9.08 kg, and dropping 508 mm) and in-situ CBR tests in clayey materials, as summarized in Table 2.4. They reported that the DCP results were as acceptable as the in-situ CBR while the coefficient of variation (C_v) of DCP tests was smaller than that of the in-situ CBR tests at the same place. They compared the CBR values for materials molded at field moisture content and density and in-situ CBR and suggested in-situ CBR and DCP relations.

Harison (1986, 1987) developed theoretical explanation for the linear log-log relation of DCP and CBR. He conducted 72 pairs of DCP and CBR tests on clay-like, well-graded sand, and well-graded gravel samples prepared in standard CBR molds and presented correlation equations, as shown in Table 2.4. The regression analysis showed that the log-log model relates DCP and CBR better than the inverse model. It was determined that moisture content and dry density had significant effects on CBR and DCP. It was also concluded that the soaking process did not have a significant effect on the calibration.

Livneh and Ishai (1987) used a dynamic cone penetrometer with a 30-degree cone for pavement evaluation. They developed a correlation between DCPI and CBR, based on laboratory and field tests on a wide range of natural and compacted soils, as summarized in Table 2.4. However, they did not provide the soil classification and other soil parameters. Based on the CBR-DCPI correlation, they developed methods for evaluation of airport and highway pavement in addition to evaluation of the dynamic stiffness modulus and load classification number. Livneh (1987) reported that the coefficient of variation of the CBR results for any particular material was significantly higher than that of the DCP.

Chua (1988) presented a one dimensional model for DCP penetration to estimate the elastic modulus of the soil. He reported the results as series of graphs for different soils that correlated DCPI to elastic modulus.

Chua and Lytton (1988) used a DCP with an accelerometer fixed on top of the handle to analyze the dynamics of the system. They developed a simple model of springs and dashpots representing hammer-rod-soil interactions. In addition, they verified the ability of determination of the damping ratio of the soil.

Harison (1989) developed a new correlation between DCPI and CBR, which is corrected to account for the confinement effects of laboratory CBR tests, as summarized in Table 2.5. He also shows that the DCP test results in lower coefficient of variation than the CBR, and therefore, it is more repeatable than CBR test.

Ayers et al. (1989) studied DCPI-shear strength correlations for a range of granular materials. The equations correlate the DCPI to deviator stress under different confining pressures. They used sand, sandy gravel, and crushed dolomitic ballast with different percentage of fines. They emphasized the role of the confining pressure under field loading conditions, as summarized in Table 2.6.

Livneh et al. (1992) explained a pneumatic automated DCP, which needs a compressor for operation. That system was able to run 24 blows and more per minute. They compared the data from the automated and manual DCP. While the regression analysis showed that the manual DCP resulted in higher values than the automated DCP, the statistical analysis showed that they are indistinguishable. However, CBRs from the automated system were on average 14% smaller than the ones obtained from the manual DCP. They examined the effect of blow rate in sandy clay, but it was found not significant.

Weintraub (1993) developed an automated DCP to measure bearing strength of unsurfaced airfield. The mechanical design procedure and appropriate details were described in his work. He also reported that the results of the manual DCP and the automated DCP are not similar and they have developed the following correlations:

- For all five sites combined

$$DCP = 2.27\ ADCP - 0.12 \qquad (R^2 = 0.85) \qquad (2.1)$$

- For the three sand sites

$$DCP = 2.3\ ADCP - 0.04 \qquad (R^2 = 0.94) \qquad (2.2)$$

Where:

DCP: Dynamic cone penetration index (mm/blow)

ADCP: Automated dynamic cone penetration index (mm/blow)

Weintraub (1993) summarized the following advantages of the automated DCP:

- Ease of operation over large number of tests
- Field testing can be performed by one operator
- Operator error reduced with electronic blow counter
- Higher blow rate decreases testing time
- Data acquisition easier to document with separate measuring rod

Burnham and Johnson (1993) studied the use of the DCP for in-situ characterization of soil profiles. They explained examples of its application in structural evaluation of existing pavements, embankment and back-fill construction control, preliminary soil surveys, and supplementing foundation testing for design purposes. They reported that the DCPT can efficiently and effectively provide a view of strength characteristics throughout a soil or roadbed structure.

Webster et al. (1994) observed the minimum penetration depth required in DCP to measure the strength of surface layers. They reported that the required depth is 2.5 to 28 cm for materials ranging from highly plastic clay to poorly graded sand. It was also shown that the thickness and location of a weak soil layer in a pavement can be determined using DCPT.

Ese et al. (1994) indicated that a DCPI of less than 2.6 mm/blow in the well-graded gravel base layer was critical to have a good serviceability in a highway. It was determined that the DCP tests during snow melting give the best correlation to the serviceability of a highway in Norway. They reported variation of DCP values due to variations of the moisture content. In DCP-CBR correlation, they showed that this correlation is independent of moisture content and dry density.

Bratt et al. (1995) developed a DCPI to dry density correlation. They showed that DCP could substitute moisture-density tests for compaction construction control of embankment and subgrade examination.

Truebe et al. (1995) evaluated the strength of a low volume road of Forest Service in United State of America by using DCPT. They presented a DCP to in-situ CBR correlation for the aggregate surface and subgrade. In addition, they reported that the DCP is a useful tool for rapidly evaluating material strength properties.

Livneh et al. (1995) verified the vertical confinement effect of granular layers, cohesive layers, and rigid structural layers on clayey materials. They studied the effect of upper asphalt layers on the DCPI of granular materials. In addition, their findings have indicated that no vertical confinement effect exists by the upper granular layer on the DCP values of the cohesive subgrade beneath them. However, DCP measurement in granular soil depends on the vertical confinement. They mentioned that for pavement evaluation purposes, any DCP measurement should be conducted through a narrow boring in asphalt layer and not after removal of a wide strip of asphalt.

Hassan (1996) studied existing correlations between DCPI and resilient modulus for sand and fine-grained soils. The specimens were Oklahoma soils molded and compacted in small mold with 6 inch diameter and 12 inch height. The experimental results showed that in fine-grained soils, the increase in moisture content above the optimum values significantly increase DCPI, while an increase in soil dry density decreases DCPI. However, and an increase in confining pressure does not significantly affect DCPI. In granular soils, it was indicated that the confining pressure is an important factor affecting DCPI. Nevertheless, this effect is less for materials with higher coefficient of uniformity. He also developed a correlation between DCPI and resilient modulus in fine-grained soils at optimum moisture content.

Al-Refeai and Al-Suhaibani (1997), at king Saud University in Saudi Arabia, have developed from laboratory study unique models between dynamic cone penetration index and CBR for a number of different soil types ranging from clay to gravely sand and they have indicated that dynamic cone penetration test can be used to predict CBR values with relatively high accuracy.

Burnham (1997) studied application of DCP as a quality control device in granular base layer compaction and the backfill compaction of pavement edge drain trenches in Minnesota Department of Transportation (MnDOT). A limiting DCPI value for each particular subgrade soil and base type was proposed as incorporated in MnDOT specifications. He presented a correlation between DCPI and the necessary remedial thickness of granular backfill/lime modification.

Chai and Roslie (1998) correlated the number of blows required by DCP to penetrate 30 cm to subgrade modulus back-calculated from Falling Weight Deflectometer (FWD). Parker et al. (1998) developed and automated a DCP type where the instrument was fixed on a trailer. The system was planned to lift the hammer, record data, and remove the rod after penetration.

Luo et al. (1998) developed field and laboratory relationship between penetration index, dry density, water content and resilient modulus. They have shown that, in the future, it is possible to give enough data and obtain correlations between penetration index, dry density, water content and resilient modulus.

Coonse (1999) has implemented DCP and CBR tests on remolded residual clayey soils in laboratory. He verified that the CBR and DCP show the same strength response to change in moisture content while the compaction effort is constant and to change in compaction effort while the moisture is constant around optimum. By comparing the results of CBR and DCP tests in soaked and unsoaked samples, he indicated that soaked specimen loose strength and both tests identified such behavior. It was also indicated that the change of moisture content can significantly change the strength of the cohesive soil. He also confirmed the influence of the mold size on DCP and CBR test results. He also derived a correlation between DCP and CBR for CH and CL materials, as summarized in Table 2.5.

Chen et al. (1999) established a strong correlation between a 30-degree DCP cone results and the elastic modulus from FWD (Falling Weight Deflectometer) in commonly encountered clayey and silty soils in Kansas. They showed a correlation equation along with 95 percent confidence lines.

Siekmeier et al. (2000) applied the DCP test, falling weight deflectometer (FWD), and soil stiffens gauge (SSG) on subgrade and granular base for several projects and they linked the modulus results from these devices. They showed that there is a weaker suitable correlation between the strength that was measured with the DCP and the elastic modulus from the FWD and SSG.

Gabr et al. (2000) investigated the use of the DCP device for evaluation of the pavement distress state. As a result, a model for evaluating the pavement stress level based on the DCP result was developed. In addition, they have created a correlation between DCP data and CBR based on laboratory and field study for aggregate base course material, as summarized in Table 2.5.

Livneh et al. (2000) stated that if the extension DCP rod is used, there is a significant change in CBR value when a standard equation of CBR-DCP is applied. Based on in situ testing for clay and granular subsoil, they evaluated that the deduction of CBR value that was calculated from in situ DCP index have to be 20% for the depth of DCP rod exceeding 1 m and 10% for 1.5 m. In this study, they suggested an equation to estimate the developing skin friction along rod of DCP that affects the DCP results when the DCP rod is not in the vertical position.

George and Uddin (2000) used manual and trailer-mounted automated DCPs in their examination to determine the subgrade resilient modulus of subgrade soils in the state of Mississippi. They reported that there was no difference between the manual and automated DCP measurements. They determined the subgrade moduli by using laboratory triaxial tests and analyzing the deflection profiles obtained from the FWD.

Chen et al. (2001) conducted more than 60 DCP tests on two test pavements. They conducted DCPT in 3 different behaviors through asphalt concrete, a narrow borehole in asphalt concrete and directly on the base of highway. Average DCPI was used for correlations to CBR and then to elastic modulus using the correlation proposed by Powell et al. (1984). The elastic modulus obtained from DCP was then compared with those obtained by FWD-MDD (Falling Weight Deflectometer-Multi Depth Deflectometer) tests and by resilient modulus laboratory tests. They reported that the elastic modulus of the base and subgrade layers determined by DCP and FWD-MDD tests are very close and laboratory determined subgrade modulus were slightly higher than those.

Konrad and Lachance (2001) studied the effect of grain size on penetration resistance and they used a 51-mm diameter cone in dynamic penetration test in base and subbase materials. They also correlated the penetration index to the elastic modulus from plate load test in unbound base and subbase materials.

Rahim and George (2002) conducted DCP and automated DCP (which is mounted on a trailer) tests on a top of subgrade through drilled holes at 12 sites in Mississippi. They obtained Shelby tube samples and tested them to calculate Resilient Modulus. They also developed a correlation between DCPI and other soil properties to resilient modulus by two different equations for coarse-grained and fine-grained soils.

Herrick and Jones (2002) used DCPT with a 2 kg hammer for measuring soil compaction in agricultural and rangelands. They used an adjustable hammer drop height to have the flexibility, which allowed them to use a single instrument on a broad range of soils without any loss in sensitivity.

Amini (2003) studied the application of DCP in pavement design and construction. He warned the use of DCP for materials with an aggregate size larger than 50 mm.

Salgado and Yoon (2003) have investigated the relationships between subgrade parameters and DCP and they have created correlations such as a correlation between dry density and DCP results for clayey sand in terms of plasticity index (PI) as follows:

$$\gamma_d = \left\{ 10^{1.5} * \mathrm{PI}^{-0.14} \, \mathrm{X} \sqrt{\frac{\sigma_v}{P_A}} \right\}^{0.5} * \gamma_w \tag{2.3}$$

In this study, they investigated several subgrade soils at different road construction sites. Each soil was tested in the field and in the laboratory; the DCPT and nuclear density gauge tests were used on the field testing, as shown in Figure 2.3 and Figure 2.4. They determined the relationships between the DCPT results and the subgrade parameters such as resilient modulus and unconfined compressive strength, as summarized in Table 2.1 and Table 2.2.

Zhang et al. (2004) indicated good correlations among the test data from the Dynamic Cone Penetration test, Falling Weight Deflectometer, and Plate Load Test, which can be used in the future for the quality control of backfills. In this research, laboratory and field tests were conducted on backfill materials and subgrade soils. They developed the following correlations:

DCP versus PLT

$$E_{PLT} = -0.34 * (N_{DCP})^2 + 13.97 * N_{DCP} - 13.67 \quad (2 < N_{DCP} < 15) \qquad (2.4)$$

Where:

E_{PLT}: Elastic Modulus in MPa

N_{DCP}: No. of blows per 10 cm

DCP versus FWD

$$M_{FWD} = -0.14\, X(N_{DCP})^2 + 10.61 * N_{DCP} - 17.11 \quad (2 < N_{DCP} < 15) \qquad (2.5)$$

Where:

M_{FWD}: Resilient Modulus in MPa

N_{DCP}: No. of blows per 10 cm

Rahim et al. (2004) introduced a model based on the pore collapse theory and cylindrical cavity expansion to predict DCP penetration resistance based on cohesion, angle of internal friction and initial porosity. They showed that for small initial porosity, the penetration resistance was strongly dependent on internal angle of friction but it was not as sensitive to cohesion. In general, the described angles of internal frictions versus DCPIs were scattered.

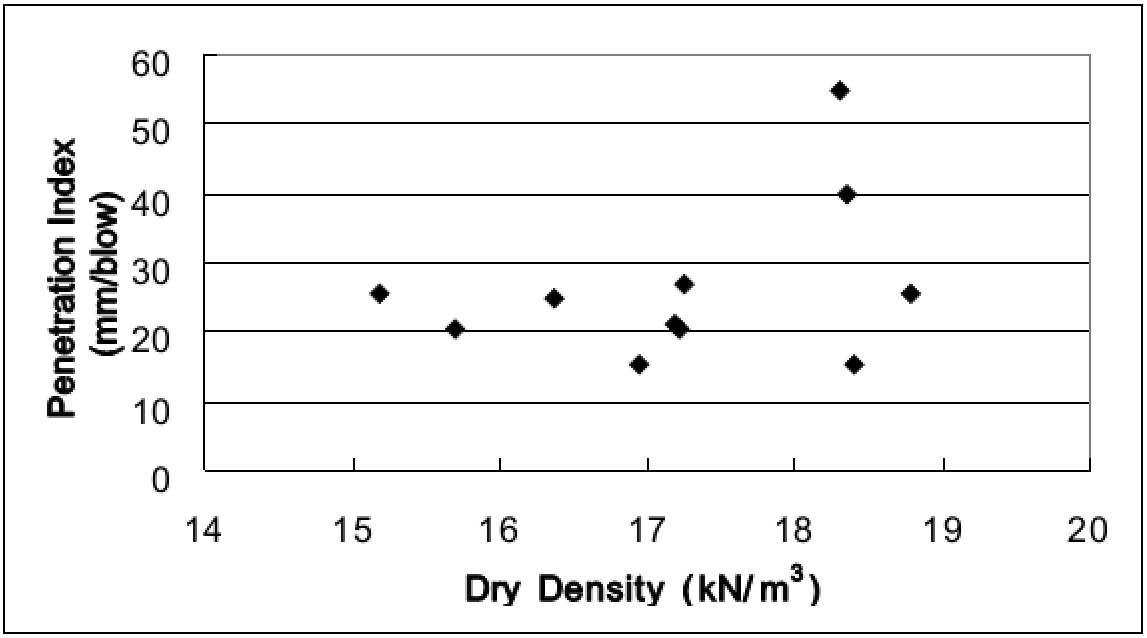


Figure 2-3: Relationship between dry density and penetration index from field DCPT

(after Salgado and Yoon, 2003) .

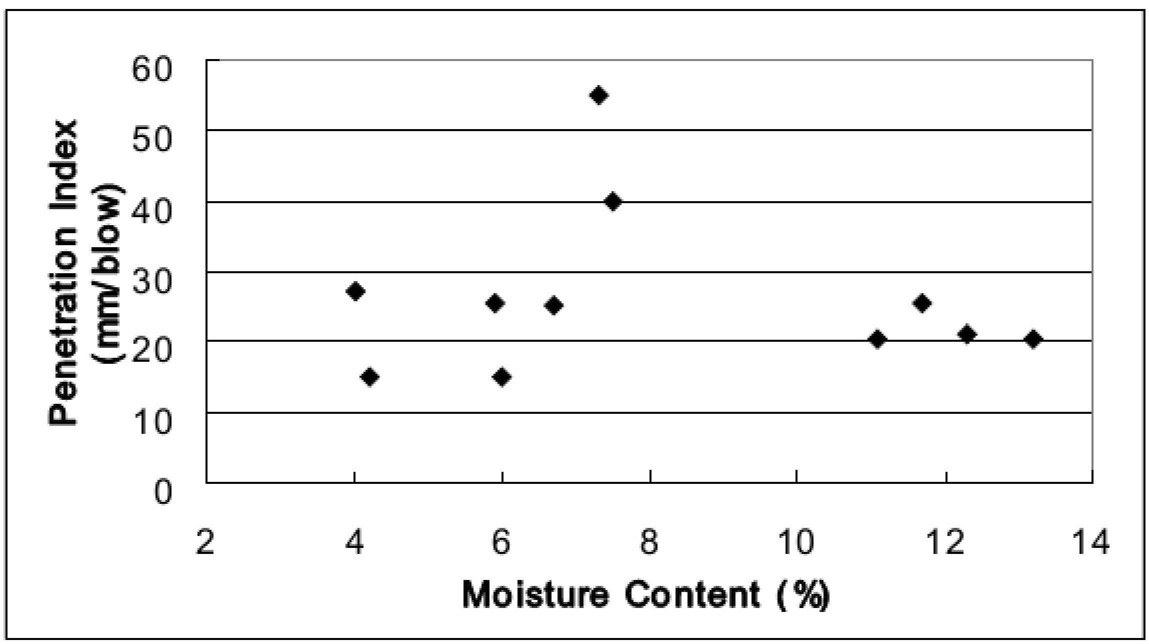


Figure 2-4: Relationship between moisture content and penetration index from field DCPT(after Salgado and Yoon, 2003).

Table 2-1: Result of unconfined compressive test and corresponding penetration index from field DCPT for the site of Lindberg Road in West Lafayette, IN (after Salgado and Yoon, 2003).

Dry Density (kN/m^3)	Unconfined Compressive Strength (kN/m^2)	S_u at 1% strain (kN/m^2)	Resilient Modulus (kN/m^2)	Penetration Index (mm/blow)
19.1	278.1	168.5	92749.7	21.9
19.4	419.3	210.3	108206.8	17.8
19.2	305.3	152.0	85830.5	15.2

Table 2-2: Result of unconfined compression test and corresponding penetration index from field DCPT for the site of I65/County Road100E in Lebanon, IN (after Salgado and Yoon, 2003).

Dry Density (kN/m^3)	Unconfined Compressive Strength (kN/m^2)	S_u at 1% strain (kN/m^2)	Resilient Modulus (kN/m^2)	Penetration Index (mm/blow)
18.6	117.3	18.0	12205.4	17.8
19.0	283.8	94.0	57743.3	13.5
20.3	549.2	175.8	95688.9	29.3

Chen et al. (2005) found a correlation between DCPI and elastic modulus from FWD tests. They compared their relationship with the elastic modulus obtained using the DCPI-CBR correlation of Webster et al. (1992), as shown in Table 2.5, and CBR to elastic modulus correlation suggested by Powell et al. (1984). The DCPI was corrected to take into account the effect of overburden pressure in case of conducting the test through a drilled hole in the asphalt layer.

Edil and benson (2005) conducted several tests on the exposed subgrade and subbase to the maximum depth of 38 cm across the State of Wisconsin. They obtained the DCPI from the weighted average of penetration rates without excluding any data points for further analysis and correlations. They observed a linear relationship among SSG (Soil Stiffness Gauge) stiffness in regular scale and DCPI averaged from depth 0 to 152 mm in logarithmic scale. They also showed that in plots of DCPI versus unit weight or water content, a general pattern can be observed but data points were so dispersed that a unique correlation could not be developed.

Herath et al. (2005) developed two models to predict the resilient modulus from dynamic cone penetration test results based on laboratory and field studies on four cohesive soils. One model predicted the resilient modulus of cohesive soils from the dynamic cone penetration index. The other model was for the prediction of the resilient modulus of cohesive soils from the dynamic cone penetration index, moisture content, dry unit weight, and plasticity index of cohesive soils. The following two models were proposed to predict the M_r of cohesive soils:

Table 2-3: Summary of full-scale trench test information (after Zhang et al. 2004)

Trench Number	Material	Section Number	Compaction Effort	Standard Proctor γ_d (kN/m^3)	Field Moisture Content (%)	Field Dry Density γ_d (kN/m^3)	Average N_{DCP} (Blows per 10 cm)	Modulus E_{PLT} (MPa)	Modulus M_{FWD} (MPa)
1	Sand	1	Light*	16.8	3.7	16.1	1.5	15.38	27
		2	Medium**			17.1	3.6	40.03	77
		3	Heavy***			17.2	5.3	-	-
2	RAP	1	Light	18.8	8.4	15.8	3.3	18	44
		2	Medium			16.9	6.2	32	78
		3	Heavy			18	14	105.2	139
3	Crushed Limestone	1	Light	21.4	5.1	18.9	2.8	29.95	40
		2	Medium			19.1	4.7	35.17	70
		3	Heavy			21.1	17.5	96.5	92

*: One pass of vibratory plate compactor

**: Four passes of vibratory plate compactor

***: Four passes of vibratory plate compactor + four passes of Wacker Packer compaction.

$$M_r = 16.28 + \frac{928.24}{(DCPI)} \quad (2.6)$$

$$M_r = 520.62 \left\{\frac{1}{(DCPI)^{0.7362}}\right\} + 0.40 \left\{\frac{\gamma_d}{w}\right\} + 0.44\ PI \quad (2.7)$$

Where:

M_r: Resilient Modulus in MPa

DCPI: Dynamic Cone Penetration Index (mm/blow)

γ_d: Dry Density in (kN/m^3)

w: Water Content (%)

PI: Plasticity Index (%)

Abu-Farsakh et al. (2005) showed that, based on laboratory and field study for DCP, PLT, FWD, and CBR, dynamic cone penetration test can be used to evaluate subgrade and pavement layers. They also developed empirical correlations from DCP results with PLT elastic modulus, FWD resilient modulus, and CBR. In addition, they indicated that the DCP test was efficient tool for compaction control.

Enayatpour et al. (2006) developed correlations between unconfined strength, curing time and dynamic cone penetration index based on laboratory study of treated expansive soils that were stabilized by cement and lime. They also explained the effects of stabilizer, dosages and curing periods on dynamic cone penetration measurements. They developed the following correlations:

$$q_c = 470 + 104.3 * CC + 201 * t + 4052.7\ DCPI \quad (2.8)$$

$$q_c = 341.2 - 26.2 \ * \ LC + 21.6 \ * \ t + 335.7 \ DCPI \tag{2.9}$$

Where:

q_c: Unconfined Compressive Strength (UCS) of the soil in kPa.

CC: Cement Content

LC: Lime Content

t: Curing time in days

Dai and Kremer (2006) summarized specifications and implementation of the DCP testing in Minnesota and other states. They conducted tests with DCP (equipped with DCP-DAS) and other tests on several construction projects in the State of Minnesota. They suggested a modified DCP specification for road construction projects and testing procedure.

Ampadu and Arthur (2006) developed, based on tests on compacted gravel in a road construction site in Ghana, a correlation between DCPI and the level of compaction. They decided that this correlation depends on the material and the water content, and the proposed equation is not unique.

Wu and Sargand (2007) showed that DCP is a practical device for evaluation of base and subgrade during construction. They showed that DCP can greatly develop the quality monitoring of pavement unbound materials. They used an automated DCP in their research, and they stated that it reduced the required time to run one test to one-fifth. However, very small penetration rates were observed in some of the tests, which they related to non-homogenous nature of subgrade soil and presence of small rocks. They suggested accepting DCP into pavement design methods, since they proved the validity of DCP to measure the soil strength.

Mohammadi et al. (2007) indicated that the dynamic cone penetration test can reliably predict the moduli obtained from PLT and CBR values, and DCPT can be used to evaluate the in-situ stiffness characteristics of compacted soils, subgrade, base layers, and embankments. In this study, series of laboratory tests on poorly graded sands and clayey silt were performed by varying the moisture content and the dry density. They developed the following correlations:

$$\text{Log CBR} = 2.256 - 0.954\ \text{Log PR} \qquad (R^2 = 0.65) \tag{2.10}$$

$$E_{PLT(i)} = 6925/(6.1 + PR^{1.4}) \qquad (R^2 = 0.65) \tag{2.11}$$

$$E_{PLT(R2)} = 6925\ \text{X}\ PR^{-1.29} \qquad (R^2 = 0.79) \tag{2.12}$$

Where:

CBR: California Bearing Ratio

$E_{PLT(i)}$: Initial Elastic Modulus

$E_{PLT(R2)}$: Reloading Elastic Modulus

PR: Penetration Rate (mm/blow)

Booth et al. (2008) raised some concerns about validity of DCP to CBR correlations after comparing laboratory CBR values with those obtained from correlation equations from tests in sandy slightly gravelly silt and silty very gravelly sand. Puppala (2008) reviewed

DCPI to resilient modulus correlations. He stated that the DCP was used by different transportation agencies for years to estimate the moduli of compacted subgrades and granular soils. However, he warned that the majority of the correlations were site specific and empirical in nature and their use for other soils required careful examination and engineering decision.

Rao et al. (2008) focused on exploring correlations between falling weight deflectometer (FWD) results and the results obtained from DCP test and CBR test. Regression models were established to qualify the prediction of CBR values based on the observed values of FWD modulus and DCP index.

Mohammadi et al. (2008) developed the relationships between Dynamic Penetration Index, relative density, modulus of elasticity, shear modulus, modulus of subgrade reaction, and the friction angle of the soil with a high coefficient of determination (more than 90%). They used a mold with 700 mm diameter and 700 mm height and conducted DCPTs and PLTs. They developed the following correlations:

DCPI versus D_r (%)

$$D_r = 189.93/(DCPI)^{0.53} \qquad (R^2 = 0.98) \tag{2.13}$$

DCPI versus modulus of elasticity

$$E_{PLT(i)}(MPa) = 55.033/(DCPI)^{0.54} \quad (R^2 = 0.83) \tag{2.14}$$

$$E_{PLT(R^2)}(MPa) = 53.73/(DCPI)^{0.74} \quad (R^2 = 0.94) \tag{2.15}$$

DCPI versus shear modulus

$$G_{PLT}(\text{MPa}) = 75.74/(\text{DCPI})^{0.99} \qquad (R^2 = 0.93) \tag{2.16}$$

DCPI versus modulus of subgrade reaction

$$E_s(\text{MN/m}^3) = 898.36/(\text{DCPI})^{0.9} \qquad (R^2 = 0.95) \tag{2.17}$$

DCPI versus shear strength

$$\phi'(\text{deg}) = 52.16/(\text{DCPI})^{0.13} \qquad (R^2 = 0.9) \tag{2.18}$$

$$\phi'(\text{deg}) = 26.31 + 0.21(D_r) \qquad (R^2 = 0.9) \tag{2.19}$$

George et al. (2009) have indicated that an increase in the fines content from 10 to 92%, for blended laterite soils, caused a decrease in the maximum dry density, and an increase in the optimum moisture content and the dynamic cone penetration index. Further, they have developed the correlations between various parameters and DCP results.

Kim et al. (2010) have evaluated the use of the dynamic cone penetration test and the Clegg hammer test results to develop criteria for soil compaction quality control. In this research, minimum required dynamic cone penetration blow accounts have proposed for various type of soil based on field and laboratory experimental program.

2.1.2 Existing DCP Correlations

The DCP-CBR Correlations

According to many researches (Al-Refeai and Al-Suhaibani, 1997; Coonse, 1999; Ese et al., 1994; Gabr et al., 2000; Harison, 1989, 1986; Livneh and Ishai, 1987; Livneh, 1987; Smith and Pratt, 1983a; Truebe et al., 1995; and Webster et al., 1994, 1992), several

DCP-CBR correlations have been developed for different material in laboratory and field. These correlations are listed in the following Tables 2.4 and 2.5 and Figures 2.5 and 2.6:

The DCP- Shear Strength Correlations

Ayers et al. (1989) developed a correlation between dynamic cone penetration index and the shear strength of granular soils. The objective of that study was to assess the shear strength using DCPT index for granular material as a rapid and inexpensive in-situ testing method. Laboratory DCP and triaxial tests were performed to get penetration index and shear strength values. The test samples included sand, dense-graded sandy gravel, crushed dolomitic ballast, and ballast with varying amounts of non-plastic crushed dolomitic fines.

Ayers (1989) developed correlations between the dynamic cone penetration index (DCPI) and the shear strength of soils and provided equations, as summarized in Table 2.6, for various confining stress (34.5, 103.4 and 206.9 kPa) in the following form:

$$DS = A - B\ (DCPI) \qquad (2.20)$$

Where:

DS: Shear Strength,

A and B: Regression Coefficients.

Table 2.6 shows the correlations between the penetration indexes and shear strength for the different materials and confining stress levels. It was also discovered that for a given unit weight or relative density, the values of penetration index (DCPI) decrease as the confining stress increases.

Table 2-4: DCP-CBR correlations (1983-1987)

Author	Correlation	Limitation	Location
(Smith and Pratt, 1983)	log CBR = 2.555 - 1.145*(log DCPI)	Clayed materials	Field
Sampson (1984)	ln CBR = 5.80 - 0.95*(ln DCPI)	All tests	Laboratory
	ln CBR = 5.93 - 1.1*(ln DCPI)	Plastic materials only	
	ln CBR = 6.15 - 1.248*(ln DCPI)	Materials with PI > 6	
	ln CBR = 5.70 - 0.82*(ln DCPI)	Materials with PI < 6	
	ln CBR = 5.86 - 0.69*(ln DCPI)	Materials with PI = 0	
Harrison (1986)	log CBR = 2.81 - 1.32*(log DCPI)	For clay, SW, and GW is limited to CBR=2-17, CBR=17-45, and CBR=55-100 respectively	Laboratory
	log CBR = 2.70 - 1.12*(log DCPI)	Granular materials with DCPI< 10 mm/blow	
	log CBR = 2.56 - 1.16*(log DCPI)	Cohesive soils (MH) with DCPI= 10-70 mm/blow	
	log CBR = 3.03 - 1.51*(log DCPI)	Sand (SW) with DCPI=5-15 mm/blow	
	log CBR = 2.55 - 0.96*(log DCPI)	Gravel (GW) with DCPI=4-10 mm/blow	
	log CBR = 2.76 - 1.28*(log DCPI)	Soaked CBR (all materials)	
	log CBR = 2.83 - 1.33*(log DCPI)	Unsoaked CBR (all materials)	
Livneh (1987) and Livneh and Ishai (1987)	log CBR = 2.20 - 0.71*$(\log DCPI)^{1.5}$	Granular soils	Field and laboratory

Table 2-5: DCP-CBR correlations (1989-2005)

Author	Correlation	Limitation	Field or laboratory based study
Harrison (1989)	log CBR = 2.55 - 1.14*(log DCPI)	For all types of materials	Laboratory
Webster et al. (1992)	$CBR = 292/DCPI^{1.12}$	For SW, SC, SM-SC, SP-SM, CL, CH, and GC materials	Field
Webster et al. (1994)	$CBR = 1/(0.017019*DCPI)^{2}$	For CL materials only when DCPI < 18 mm/blow	Field
Webster et al. (1994)	CBR = 1 / 0.002871*DCPI	For CH materials only. Based on data DCPI > 20 mm/blow	Field
Ese et al., (1994)	log CBR = 2.669 - 1.065*(log DCPI)	Materials consisted of well-graded gravel with 9 to 19% fines	Laboratory
Truebe et al. (1995)	$CBR = 320/DCPI^{0.943}$	The materials tested include aggregate surface (20 < CBR < 86) and MH or ML subgrade (6 < CBR > 22). The equation is valid for 4 ≤ DCPI ≤40.	Field
Al-Refeai and Al-Suhaibani (1997)	log CBR = 3.24 - 1.50*(log DCPI)	For poorly graded sand, 10 ≤ DCPI ≤50 mm/blow	Laboratory
	log CBR = 2.80 - 1.46*(log DCPI)	For silty sand, 4 ≤ DCPI ≤35 mm/blow	
	log CBR = 2.54 - 1.23*(log DCPI)	For CL or ML, 4 ≤ DCPI ≤35 mm/blow	
	log CBR = 2.50 - 1.07*(log DCPI)	For all materials, 4 ≤ DCPI ≤50 mm/blow	
Coonse (1999)	log CBR = 2.53 - 1.14*(log DCPI)	For clayey soils and for 25 ≤ DCPI ≤80	Laboratory
Gabr et al. (2000)	log CBR = 2.53 - 1.14*(log DCPI)	Piedmont subgrade soil and aggregate base coarse material	Laboratory and Field
	log CBR = 2.40 - 0.55*(log DCPI)	For aggregate base coarse material	
Karunaprema & Edirisinghe (2002)	log CBR = 2.182 - 0.872*(log DCPI)	For residual clayey and silty sand	Laboratory and Field
	log CBR = 1.966 - 0.667*(log DCPI)	For very clayey or silty gravel	
Abu-Farsakh et al. (2005)	$CBR = 1161.1/DCPI^{1.52}$	Materials tested include CL, CL-ML, SP, GP, GW-GC, and cement treated soil. The equation is valid for DCPI between 7.5 to 70 mm/blow	laboratory

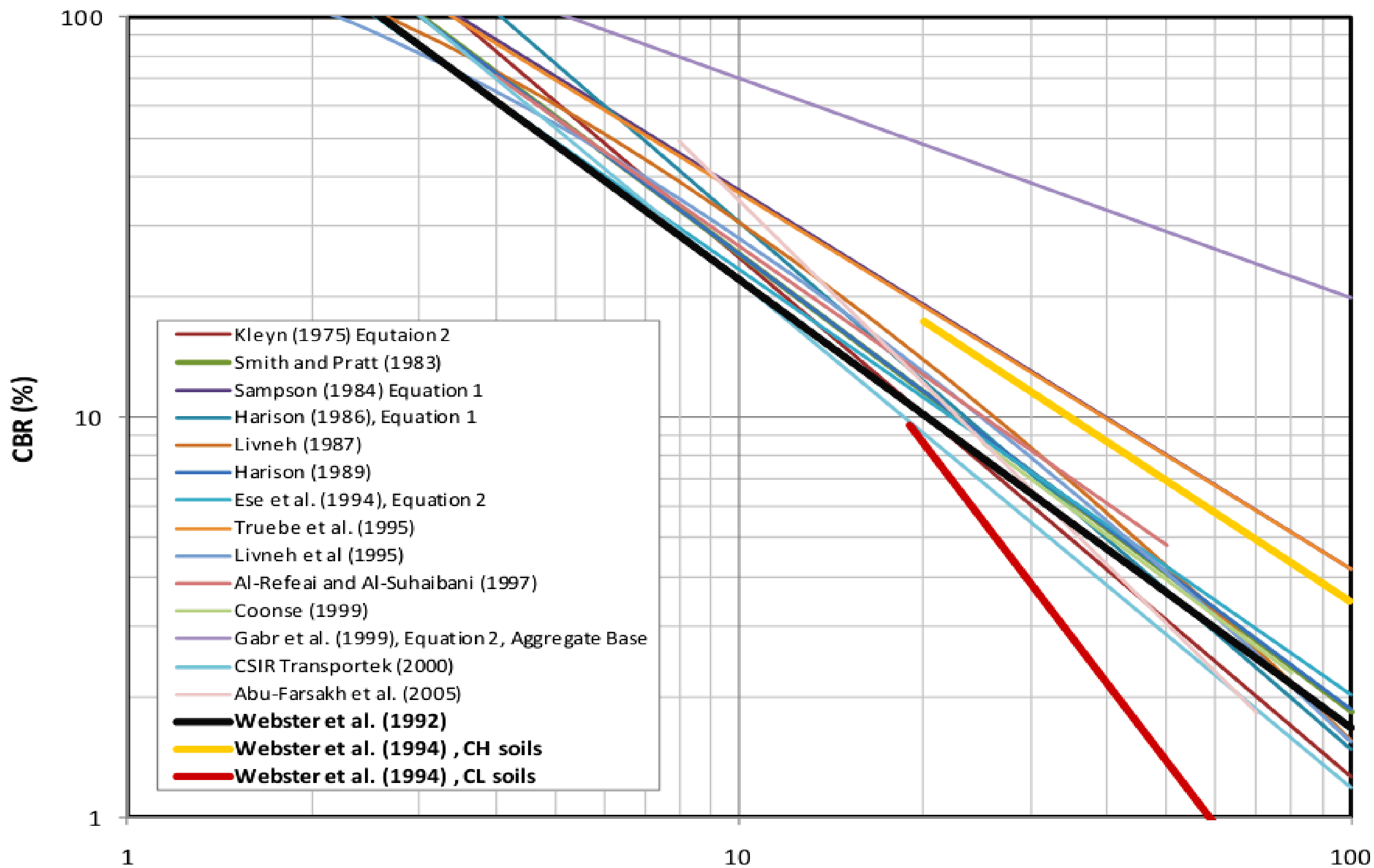


Figure 2-5: Schematic representation of DCPI-CBR correlations (Kianirad, 2011).

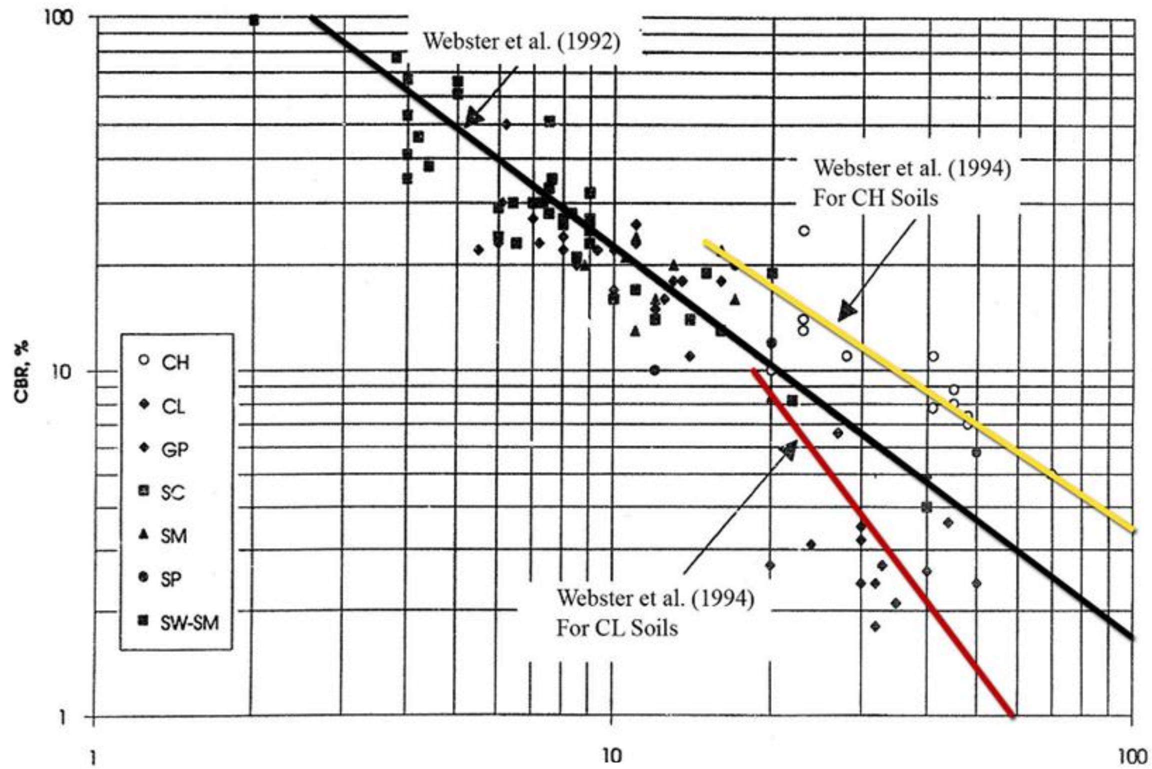


Figure 2-6: DCPI and CBR test data versus correlation equations
(after Webster et al., 1994)

Table 2-6: Relationship between PI and shear strength (after Ayers et al. 1989)

Material	Confining stress (kPa)	Correlation
Sand	34.5	DS = 41.3 - 12.8(PI)
	103.4	DS = 100.4 - 23.4(PI)
	206.9	DS = 149.6 - 12.7(PI)
Sandy gravel	34.5	DS = 51.3 - 13.6(PI)
	103.4	DS = 62.9 - 3.6(PI)
	206.9	DS = 90.7 - 5.8(PI)
Crushed dolomitic ballast	34.5	DS = 64.1 - 13.3(PI)
	103.4	DS = 139.0 - 40.6(PI)
	206.9	DS = 166.3 - 16.2(PI)
Ballast with 7.5% NF	34.5	DS = 87.2 - 78.7(PI)
	103.4	DS = 216.1 - 213.9(PI)
	206.9	DS = 282.1 - 233.2(PI)
Ballast with 15% NF	34.5	DS = 47.5 - 0.45(PI)
	103.4	DS = 184.2 - 215.5(PI)
	206.9	DS = 206.4 - 135.7(PI)
Ballast with 22.5% NF	34.5	DS = 49.7 - 23.1(PI)
	103.4	DS = 133.1 - 68.6(DCPI)
	206.9	DS = 192.1 - 95.8(DCPI)

DS = Shear strength

PI = penetration index

NF = Non plastic fines

2.1.3 DCP Features

The advantages of DCP test are summarized by Livneh (1987), Ayers et al. (1989), Burnham et al. (1993), Webster et al. (1994), MnDOT (1996), Karunaprema and Edirisinghe (2002), Salgado and Yoon (2003), and Wu and Sargand (2007) and listed as follows.

- It characterizes the in-situ strength of soil with depth,
- It could be used to determine the thickness and depth of underlying soil layers,
- It is widely used for field exploration and quality assessment of subsoil layers,
- It is repeatable, relatively inexpensive and reliable,
- It can be used in soils with a wide range of particle sizes and strengths,
- It is man-portable and the maintenance is simple and inexpensive,
- It is simple enough to be used by an inexperienced person,
- It could be used to verify whether a stabilized soil has achieved its potential stiffness,
- It requires less penetration depth than the CPT to measure the surface layer strength,
- It is relatively fast and usually does not take more than 10 min (time varies depending on the strength of the material and maximum depth of penetration)

2.1.4 DCP Problems

The first problem with DCP is the removal of the instrument after deep tests in some cases (Weintraub, 1993, Wu and Sargand, 2007). Using disposable cone tips, as suggested by Webster et al. (1992), may be one solution. However, ASTM D 6951 suggests using an extraction jack if disposable cone tips are not used.

Ayers et al, (1989) has shown that the maximum aggregate size has an important influence on the test results. They indicated that the maximum aggregate size of around 38 mm to be where the DCP is no longer a viable test. However, Webster et al. (1992) reported that DCP is not suitable for soils having significant amount of aggregates greater than 50 mm.

The third problem is that the physical raise and drop of the hammer could be a source of error in a DCPT. Webster et al. (1992) presented that the user has to ensure that the hammer is touching the bottom of the handle but not lifting the cone before it is allowed to drop. They also stated that the worker should be careful not to exert any downward or upward force on the handle and not to influence the free fall of the hammer by hand movement.

Application of skin friction can cause incorrect results in cohesive materials. Buncher and Christiansen (1992) stated that, after comparing the Electric Cone Penetrometer results with DCP and in-situ CBR, the DCP is very susceptible to skin friction in cohesive soils. For the same reason, Webster et al. (1992) suggested to limit the depth of penetration to 30 cm in highly plastic clays and clean and lubricate the rod after each test to reduce sticking of clay to the rod. However, Webster et al. (1994) stated that oiling the rod does not improve test results in CH soils significantly.

The fifth problem is the tests in loose soils because the 8 kg hammer causes excess penetration (Webster et al. 1992). In this case, no DCP measurement is possible close to the surface in dry sand or gravel (Webster et al., 1992) due to the lack of confining pressure in the procedure. Using a different hammer mass is suggested by ASTM D 6951 after Webster et al. (1992). However, there is no experimental data enough for using a 4.6 kg hammer.

The sixth problem is that the manual reading and recording the number of blows and depth of the DCP could also cause some mistakes (Webster et al., 1992). Since the DCPT requires one operator to lift and drop the hammer while keeping the device vertical, another operator should keep track of the penetration after each blow. To solve this problem, some innovation tools are added to DCP. For example, Kessler Inc. (Kessler Soils Engineering Products, Inc., 2007) presented tools to write the number of blows for each set of blows on a removable tape along the ruler, or use a magnetic ruler data collection device. Applied Research Associates Inc. developed a DCP Data Acquisition System (DCP-DAS), which uses a string potentiometer to automatically measure the depth of penetration and number of drops (Vertek, 2010). Otherwise, data collection and analysis are time consuming if no automated measurement system and software are used.

2.2 Soil Chamber

Soil chamber is a well-established technique for standardization of in situ testing devices in sandy soils, where the soil sample is prepared in the chamber and tested under controlled boundary conditions (Schnaid and Houlsby, 1991). With respect to the scale effects in the chamber, there are two factors that were considered. First, the ratio of the diameter of the DCP to that of the sand particles sizes. The particles of soil will have an effect on the test results if the size of the soil particles becomes too large compared with the DCP diameter. Related to this factor, Bolton et al. (1999) have shown that there are important effects due to soil particle on cone penetration and they have concluded that the cone diameter (B) should be at least twenty times greater than the mean particle diameter D_{50}.

Second, the ratio of the diameter of chamber to the diameter of the DCP is an additional important factor. Parkin et al. (1980) have recommended that the ratio of the diameter of chamber to the diameter of the DCP for dense sand is desirable at minimum of 50 and for loose sand is desirable at minimum of 20. Been et al. (1986) have suggested that the ratio of the diameter of chamber to the diameter of the DCP for dense sand must be larger than 50 to reduce the chamber size effect on the test results.

Schnaid and Houlsby (1991) have recommended that the ratio of the diameter of chamber to the diameter of the DCP should be at least 50 in dense sand to reduce the chamber size effects. Salgado et al. (1998) have proposed that the ratio of the diameter of chamber to the diameter of the DCP should be larger than 150 to minimize the effect of cone resistance.

Mehdiahmadi (2000) indicated that, based on experimental observation and numerical simulation for loose and dense sand under different boundary conditions and different sizes of chamber, calibration chamber size effect is not important for loose sand, but it can be significant for dense sand.

Bałachowski (2006) showed that, based on laboratory study for five boundary conditions, the size and the boundary conditions effect would be less significant in more compressible materials like silty or carbonated sands.

Mohammadi and Robertson (2008) proposed that, based on numerical results in very dense and dilatant sand, for practical purpose, the ratio of the diameter of chamber to cone diameter is desirable at 80 or more for dense sand. Also, the results indicated that the zone influence around a penetrating cone is large for dense sand compared with loose sand.

The calibration chamber has been used effectively as a research tool to simulate in situ soil in the laboratory and it has been used in creating interpretation procedures for cone penetration test in sand (Hsu and Huang, 1999). There are many applications of calibration chamber test for other types of in situ tests. Related to these applications, Been and Kosar (1991) have described the application of calibration chamber testing to study the hydraulic fracture phenomena on oil sand while Huang et al. (1991) have presented a model by conducting pressuremeter inside calibration chamber.

CHAPTER 3

LABORATORY EXPERIMENTAL PROGRAM

In order to fulfill the objectives of this proposed investigation, an experimental program was conducted to evaluate the effects of dry density and silt content on the dynamic cone penetration test (DCPT) results and to develop correlations between density and dynamic cone penetration index (DCPI) of sands. In addition, the experimental program was aimed to investigate the effect of variations in water table level on DCPI. The first phase of the program constituted the initial characterization of the sands. Relevant ASTM standard testing procedures were adopted for the characterization of the sands. The initial characterization included grain size distribution, specific gravity and relative density (maximum and minimum dry density). The second phase involved the construction of the large chamber and preparation of samples for testing. The third phase consisted of performing DCPTs on sands with different silt content and different relative density, to achieve the objectives of this research. The flowchart of the experimental program is shown in Figure 3.1.

Three different densities of the sand (loose, medium and dense) were used with different proportions of silt content and water table level. These sand samples were prepared in a large chamber (1600 mm diameter and 1500 mm height) and subjected to dynamic cone penetration test, as shown in Figure 3.2.

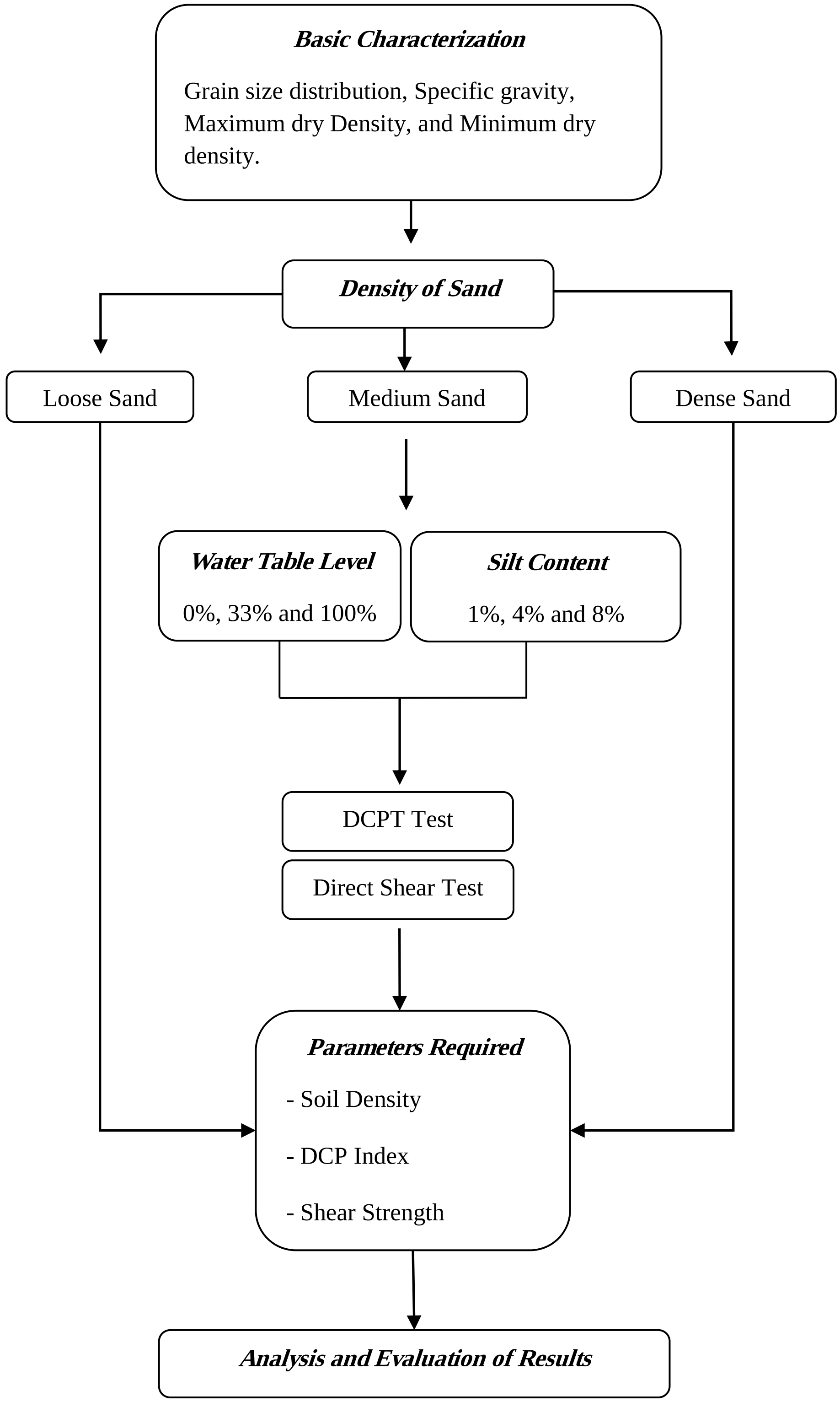


Figure 3-1: Flowchart of the experimental program.

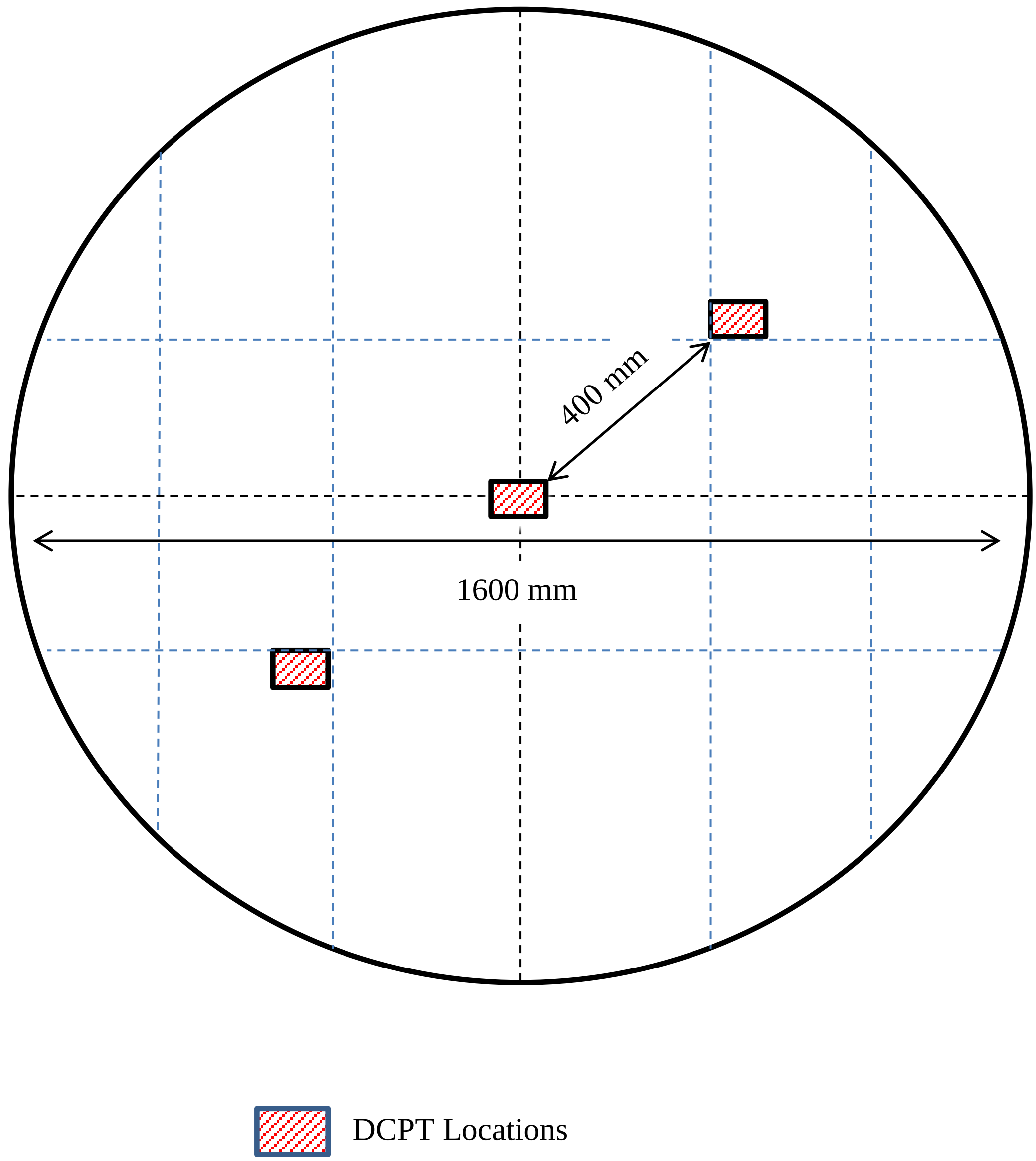


Figure 3-2: Layout of testing location for dry sand

Collection of Sand and Stone Filler

3.1.1 Collection of Sand

The sand used in this investigation was brought from the dunes in Dhahran near King Fahd University of Petroleum and Mineral (KFUPM). The soil was a dune sand, light yellowish in color and had a uniform gradation.

3.1.2 Collection of Stone Filler

The stone filler used in the testing program was brought from stone crusher that is located approximately 200 km faraway from Dammam towards Riyadh. The filler was non-plastic and light whitish in color which was sieved through ASTM D 422 and collected in containers. The filler material was mixed with sand at different percentage for 10 minutes using a mixer, as shown in Figure 3.3.

3.2 Test Chamber and Density Procedure

3.2.1 Test chamber

Soil chamber is a well-established technique for standardization of in situ testing devices in sandy soils, where the soil sample is prepared in the chamber and tested under controlled boundary conditions (Schnaid and Houlsby, 1991). The soil chamber used in this study was a cylindrical glass fiber reinforced plastic pipe (GRP pipe) that was designed and manufactured at Future Pipe Industries, in the Dammam Second Industrial Area. The chamber is 1600 mm in diameter and 1500 mm in height with a wall thickness of 25 mm. GRP pipe was fixed using adhesive material (epoxy) on stiff square steel plate

Figure 3-3: Collection and preparation of sand-silt mixture.

(1700 x 1700 mm and 300 mm high) that was manufactured at KFUPM maintenance shop. In order to facilitate the removal of the water from the chamber after the tests, two drain hose openings were made at the bottom of the chamber. The drain holes are 25 mm in diameter on two opposite sides and they are connected by two valves to allow water to inter into and out of the chamber. In order to control the water level in the chamber during the test, the valves were connected to transparent pipes to monitor the water level. Filter fabric sheets were attached to the chamber from the inside face at the openings of the valves to prevent soil migration through the openings, as shown in Figures 3.4 and 3.5.

3.2.2 Density Control Procedure

The achievement of the required densities was a challenge in this study. To meet this challenge, several preliminary experimental works were conducted to determine the most appropriate and accurate way to obtain the required density. The different sand densities were calibrated in small mold (750 mm diameter and 450 mm height) by performing more than one trial. Sand pluviation was adopted for sample preparation where different nozzle openings and funnel height were adopted to produce different densities. Once the required density was achieved, the specimen was then prepared in the large scale mold (1600 mm diameter and 1500 mm height).

In this study, two techniques were conducted to get the required density, pluviation and vibration techniques. The large funnel was used to prepare loose sand samples in the chamber with the desired density. The relative density of the loose sand deposited in the chamber was controlled by two factors: (1) the opening size of nozzle at the bottom of the funnel (25 mm maximum diameter), and (2) the sand drop height (1000 mm), as shown in Figure 3.6. However, the medium and dense sand samples were prepared with pluviation and vibration technique; whereby the sample in the testing mold was compacted in five 300 mm thick lifts. Compaction effort was applied using a 300 mm long vibrating rod and circular plate with 300 mm diameter and 17 kg weight whereby the plate contains 50 mm opening in the middle to allow the rod to inter in and out the sand sample. The relative density of medium and dense sand deposited in the chamber is controlled by vibration time; medium sand samples were vibrated once for a second while dense sand samples were vibrated once for ten seconds, by inserting the rod in the sand sample, as depicted in Figure 3.7.

3.3 Preliminary Characterization of Sands and Silt

The experimental program was intended to develop correlations for density and shear strength with the dynamic cone penetration test results of sands at different densities and with different percentages of silt and to study the effect of water table levels on DCPT results. The collected material (sand and sand-silt mixture) was subjected to preliminary tests including grain size analysis, specific gravity and relative density tests. These tests were performed on the material to assess the basic engineering properties of the natural sand and sand with different silt content. The following paragraphs describe the details of these tests.

3.3.1 Grain Size Distribution

The grain size analysis was conducted for natural sand and sand with different silt content using dry sieving technique (ASTM D 422). The air dried sands were sieved through a set

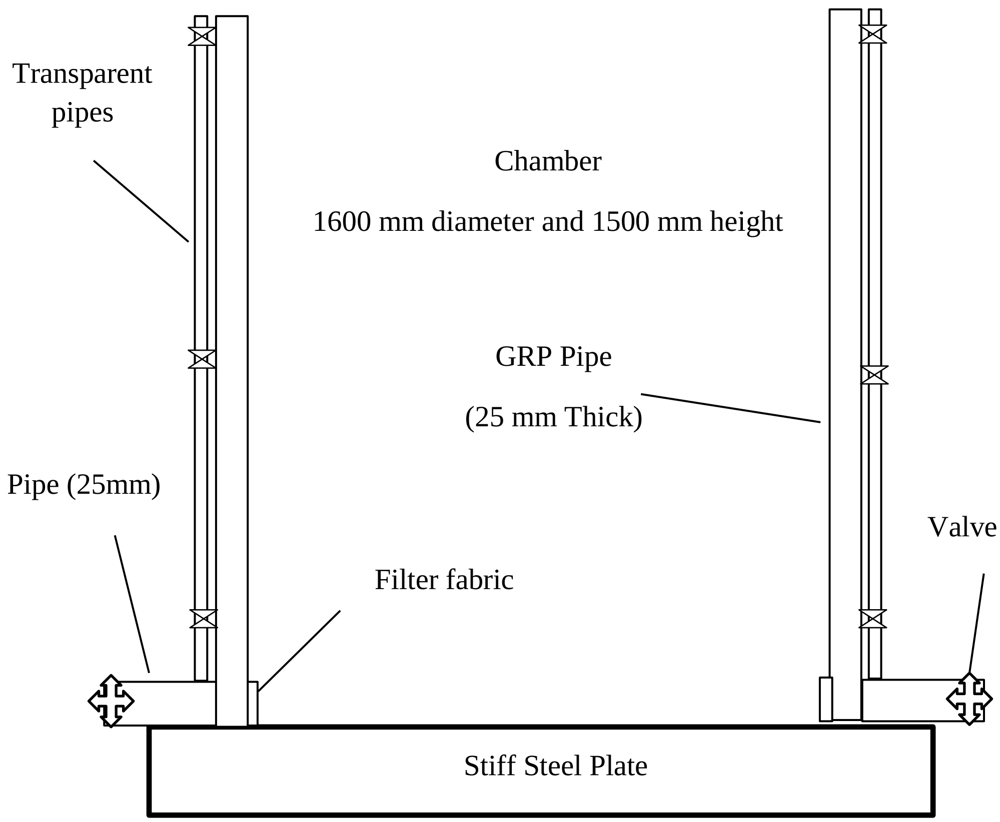


Figure 3-4: Schematic diagram of the soil chamber.

Figure 3-5: Test chamber in laboratory with scaffolding for sample preparation.

Figure 3-6: Preparation of loose sand.

Figure 3-7: Preparation of medium and dense sand.

of sieves with opening sizes of 0.85 mm, 0.425 mm, 0.25 mm, 0.15 mm, 0.106 mm, and 0.075 mm (ASTM sieve numbers: 10, 20, 40, 60, 100, 140 and 200), with a pan at the bottom. The amount of soil retained on each sieve was collected and their masses were recorded.

3.3.2 Specific Gravity Test

The specific gravity of soil is used as a parameter in the determination of some important properties of the soil such as unit weight, void ratio and particle analysis. For the natural sand and sand with different content of fine percentage, the specific gravity test was conducted according to ASTM D 854 procedure. The average of three tests was taken to represent the specific gravity of the soil.

3.3.3 Relative Density Determination

The relative density tests (ASTM D 4253 and D 4254) were performed to determine the maximum and minimum dry densities. For the minimum dry density determination, oven-dry sand was poured in the standard mold maintaining a free fall of sand of about 0.5 inch by using a funnel and moving in a spiral path to minimized segregation. By knowing the weight of sand in the mold and the volume of the mold, the minimum density of sand was determined, as reported in ASTM D 4253.

For the determination of maximum dry density, the oven-dry sand was poured into the mold using the pluviation method to minimize segregation. For densification of the sand, the side of the mold was struck few times using a rubber hammer. Then, a surcharge weight was placed on the top of the sand. The mold was placed and fixed on the vibrating table. Vibration was carried out for 8 minutes at 60 Hz. By knowing volume of the mold and the weight of sand in the mold, the maximum dry density could be determined, as specified in ASTM D 4254.

3.3.4 Direct Shear Test

In order to develop the correlation between shear strength and dynamic cone penetration index (DCP), the shear strength parameters of sand were determined. Direct shear tests were performed on sand in dry condition at different relative densities (40%, 60% and 90%) and different silt content (1%, 4% and 8%). These tests were performed according to ASTM D 5321.

CHAPTER 4

FIELD DATA

In order to validate the laboratory results, it was decided to review some of the well-controlled filed DCP testing with similar testing conditions. Field data have been obtained from two projects in eastern Saudi Arabia. the two projects were executed by Aiban (2012) where dynamic cone penetration tests (DCPTs) were performed on sand at two different sites; one in Al-Jubal, and the other in Ras Al-Khair, at Kingdom of Saudi Arabia. For each test site, the dynamic cone penetration test and nuclear gauge test were conducted at several different locations. The DCP tests were conducted in accordance with ASTM D 6951, using 8 kg hammer. In order to measure accurately the in situ soil densities and water contents, the nuclear gauge was conducted for different test locations where the DCP tests were conducted.

4.1 Case Study 1: Al-Jubail, Kingdom of Saudi Arabia

A field study was performed to assess the density using dynamic cone penetration test and nuclear gauge test. The study area was allocated in Al-Jubail in Kingdom of Saudi Arabia. The dynamic cone penetrometer (DCP) was used to assess the variations of the density for a depth of the backfill, which ranged from 1.2 m to 2.4 m. The nuclear gauge dry density data for the top 1.2 to 1.6 m of soil backfill within the different phases of compacted areas was used for the correlation with the DCP data. The nuclear gauge data along with the DCP data within the same location were used as a reference for the correlation. This implies that the soils tested at the different locations possessed the same/similar characteristics. In this study, forty five (45) DCP tests were performed to accomplish such assessment and nuclear testing was performed at three random locations. These reference locations were referred to as the control locations and the readings were the control readings. The plot plan showing the location of the different phase was provided on Figure 4.2.

Control/ Reference Testing

A filed testing program was designed and carried out to evaluate the DCP-density correlation. The location of testing program is shown in Figure 4.2. Three control pits were dug out to explore the layering of the soil and to measure the density variation with depth using nuclear gauge, for the top 1200 to 1600 mm. Figure 4.1 and Figure 4.3 show the DCP and nuclear gauge setups during the data collection. The backfill material consisted mainly of sand.

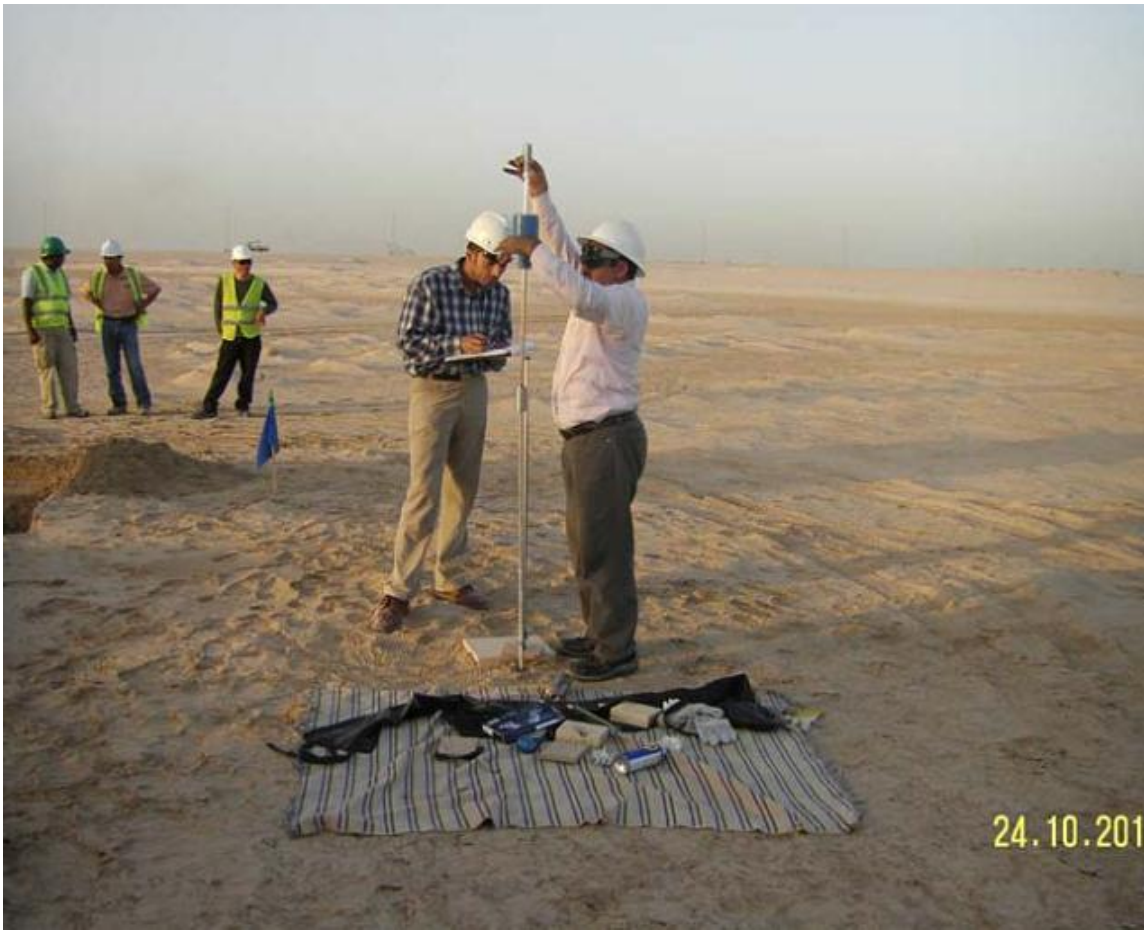


Figure 4-1: Typical photographs showing the DCP in operations (Aiban, 2012).

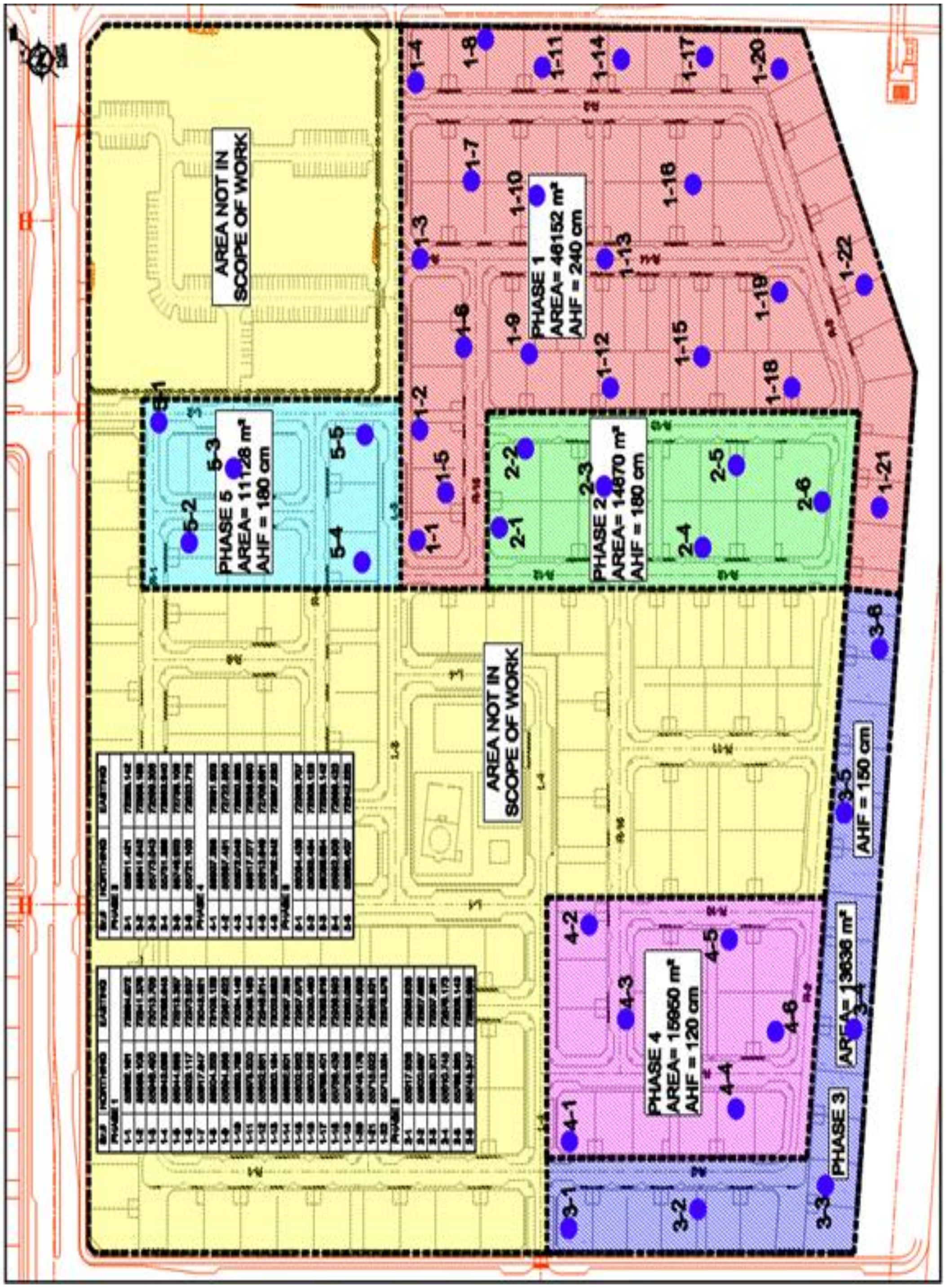


Figure 4-2: Plot plan showing the different phases and testing locations, Al- Jubail site (550 x 500 m) (Aiban, 2012).

Figure 4-3: Typical photographs showing the measurements of the dry density variations with depth using the nuclear gauge (Aiban, 2012).

4.2 Case Study 2: Ras Al-Khair, Kingdom of Saudi Arabia

A field study, using the DCP, was performed to examine the compactness of the top 5 m of soil within the area allocated in Ras Al-Khair, Kingdom of Saudi Arabia. The nuclear gauge dry density data was used for the correlation with the DCP data for the top 1.8 m of soil at two different locations. The nuclear data along with the DCP data within the same locations were used as a reference for the correlation. This implies that the soils tested at the different locations possess the same/similar characteristics. A total of 29 DCP tests were performed at randomly selected locations, as shown in the plot plan in Figure 4.4. The DCP tests were conducted in accordance with ASTM D 6951, using 8 kg hammer. These DCP testing was carried out to the depth of 5 m or refusal (difficult penetration). Due to the high density of the material and existence of little cementation, refusal was encountered within the top one meter (1 m). Several attempts were made to penetrate at slow penetration rate (high resistance and thus high blow counts) to the targeted 5 m depth; however, such attempts were not all successful. Three DCP rods were lost upon retrieval in such dense layers. Only 7 DCP tests penetrated more than 3 m.

Control/ Reference Testing

A filed testing program was designed and carried out to comply with the intended objectives. The location of control testing is shown in Figure 4.4. Two control pits were dug out to explore the layering of the soil and to measure the density variation with depth, for the top 1800 mm. Figure 4.5 and Figure 4.6 show the DCP and nuclear gauge setups during the data collection, respectively. The material consisted mainly of sand. In some places, there seemed to be little cementation at the top layers.

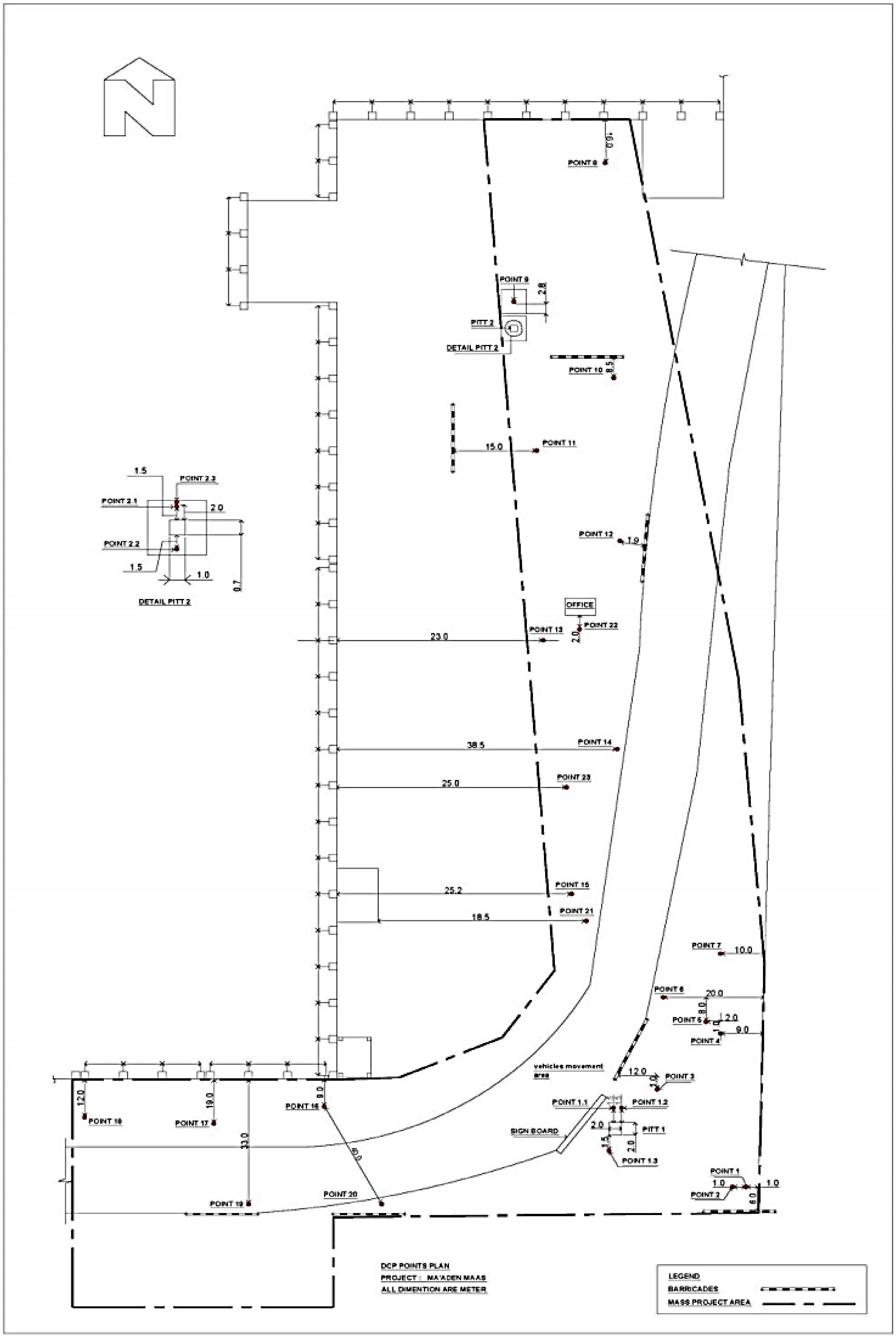


Figure 4-4: Plot plan showing the different phases and testing locations (Aiban, 2012).

Figure 4-5: Typical photographs showing the DCP in operations at Ras Al-Khair (Aiban, 2012).

Figure 4-6: Typical photographs showing the measurements of the dry density variations with depth using the nuclear gauge at Ras Al-Khair (Aiban, 2012).

CHAPTER 5

RESULTS AND DISCUSSIONS

The methodology of the laboratory tests performed in this research and field DCP data analysis were presented in Chapter 3 and Chapter 4, respectively. In this chapter, the characterization test results of sand with different silt content were discussed. In addition, dynamic cone penetration and nuclear gauge test results were presented and discussed in details.

5.1 Laboratory Testing Results

5.1.1 Properties of Sand

The sand used in this investigation was light yellowish dune sand from the Dhahran area with uniform gradation. The grain-size distribution curve is shown in Figure 5.1. The curve shows that 87.8% of the soil passed ASTM sieve No. 40 and the particles lie in the range of 0.1 to 0.9 mm and the effective size (D_{10}) was 0.146 mm, (D_{30}) was 0.2 mm, and (D_{60}) was 0.29 mm. The coefficient of curvature (C_c) was 0.94 and coefficient of uniformity (C_u) was 1.98. Therefore, the soil can be classified as poorly graded sand (SP) according to the Unified Soil Classification System and A-3 according to the AASHTO classification system. The maximum dry density (ASTM D 4253) was 1.84 gram/cm^3 and the minimum dry density (ASTM D 4254) was 1.63 gram/cm^3. The properties of natural dune sand and dune sand with different silt contents are summarized in Tables 5.1, 5.2, 5.3 and 5.4, respectively. The effects of silt content on the compaction and density characteristics were presented in Table 5.3. It is noticed that as the silt content increased, for up to 8%, the maximum dry density increased. This is mainly attributed to the fact that silt is filling the voids of the poorly graded sand (Mitchell and Soga, 2005).

5.1.2 Direct Shear Test Results

In order to determine the peak friction angles of sand, direct shear tests were conducted on sand samples at different relative densities and with different silt content. Figures 5.2 to 5.10 presented the relationship between shear stress and horizontal displacement for different relative densities and silt content. For each parameter, three different normal stresses (27.8, 55.5 and 83.3 kN/m^2) were applied. Table 5.4 shows the summary of direct shear test results for different silt content. It was observed that an increase in the percentage of silt content from 1 to 8% for different relative densities has resulted in an increase in the peak friction angles. This is attributed to the dense packing when adding silt which fills some of the voids of the poorly graded sand (Mitchell and Soga, 2005).

5.1.3 Effect of Sand Density on DCPI

In this investigation, the effect of sand density on the dynamic cone penetration index (DCPI) was studied for different silt content. It should be mentioned that to insure the repeatability and accuracy of results, three DCP tests were performed for the same set of parameters, density and silt content. The results are very consistent as shown in Figures 5.11 through 5.22. The dynamic cone penetration test was conducted up to a depth of 1500 mm. The DCP index value (DCPI) was calculated by dividing the total penetration

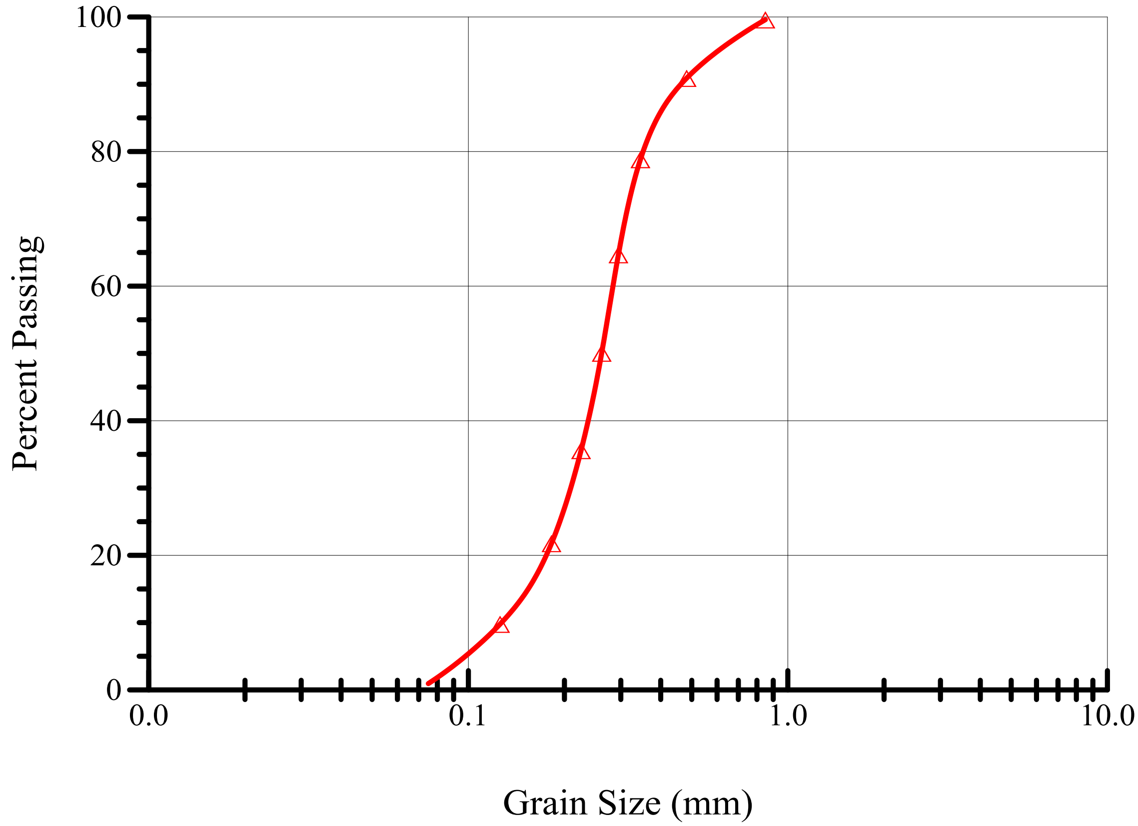


Figure 5-1: Grain-size distribution curve for the natural sand used in this investigation.

Table 5-1: Properties of natural dune sand

Property	Unit	Value
Particle size range	mm	0.1- 0.9
Specific gravity		2.68
D_{10}	mm	0.146
D_{30}	mm	0.2
D_{60}	mm	0.29
C_u		1.98
C_c		0.94
AASHTO Classification		A-3
USCS Classification		SP
Maximum dry density (ASTM D 4253)	gram/cm^3	1.84
Minimum dry density (ASTM D 4254)	gram/cm^3	1.63

Table 5-2: Sand characteristics with different silt content

Silt Content	Loose Sand		Medium Sand		Dense Sand	
	Relative Density (%)	Dry Density (ton/m^3)	Relative Density (%)	Dry Density (ton/m^3)	Relative Density (%)	Dry Density (ton/m^3)
1%	40	1.7	60	1.75	90	1.82
4%	40	1.71	60	1.75	90	1.85
8%	40	1.74	60	1.80	90	1.88

1 ton = 1000 kg

Table 5-3: Variations in sand density and relative density with different silt content

Silt Content	Max. Density (gram/cm³)	Min. Density (gram/cm³)	Specific Gravity	Max. Void Ratio	Min. Void Ratio
1% (natural sand)	1.84	1.63	2.68	0.68	0.48
4%	1.88	1.64	2.65	0.66	0.41
8%	1.92	1.65	2.62	0.63	0.33

Table 5-4: Summary of direct shear test results of sand in a dry state

Silt Content	Loose Sand	Medium Sand	Dense Sand
	ϕ_{peak}	ϕ_{peak}	ϕ_{peak}
1%	45°	46°	48°
4%	45.5°	46.5°	49°
8%	46°	47.5°	50°

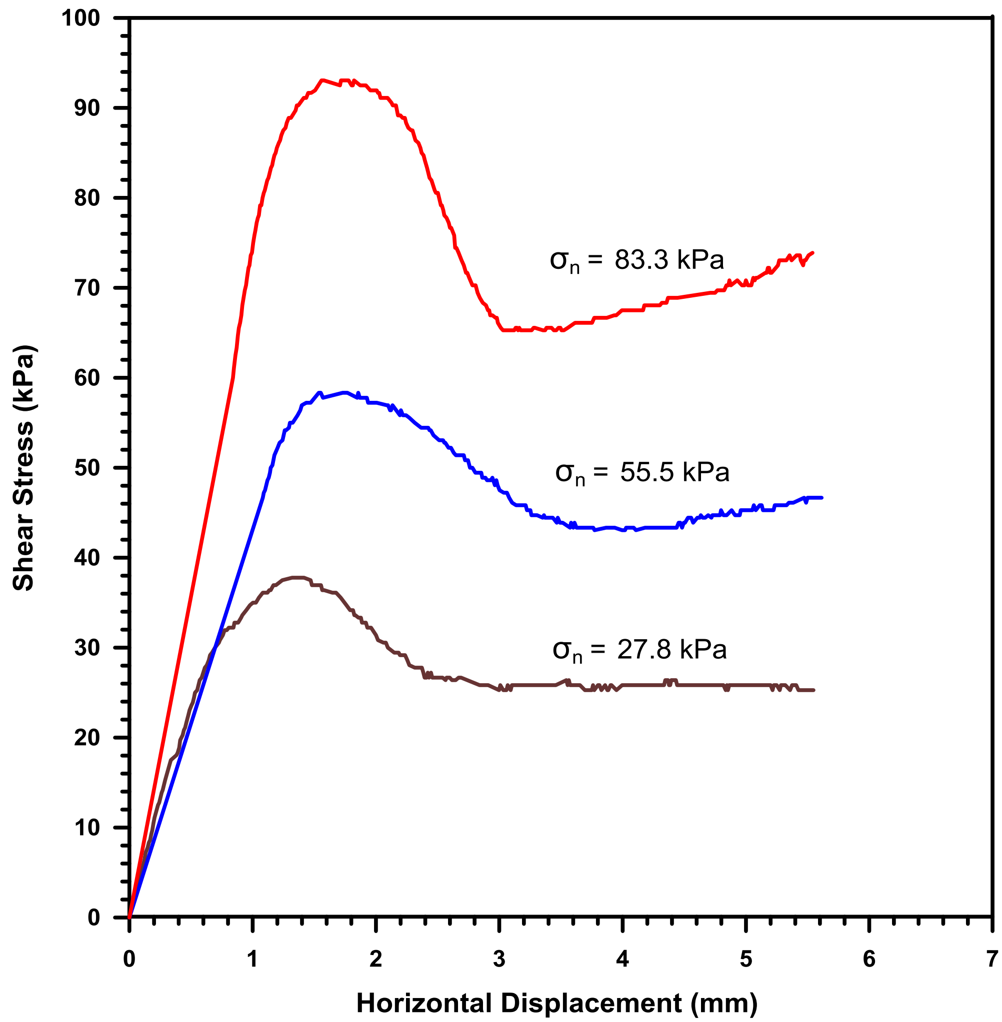


Figure 5-2: Result of direct shear test for loose dry natural sand (1% silt content).

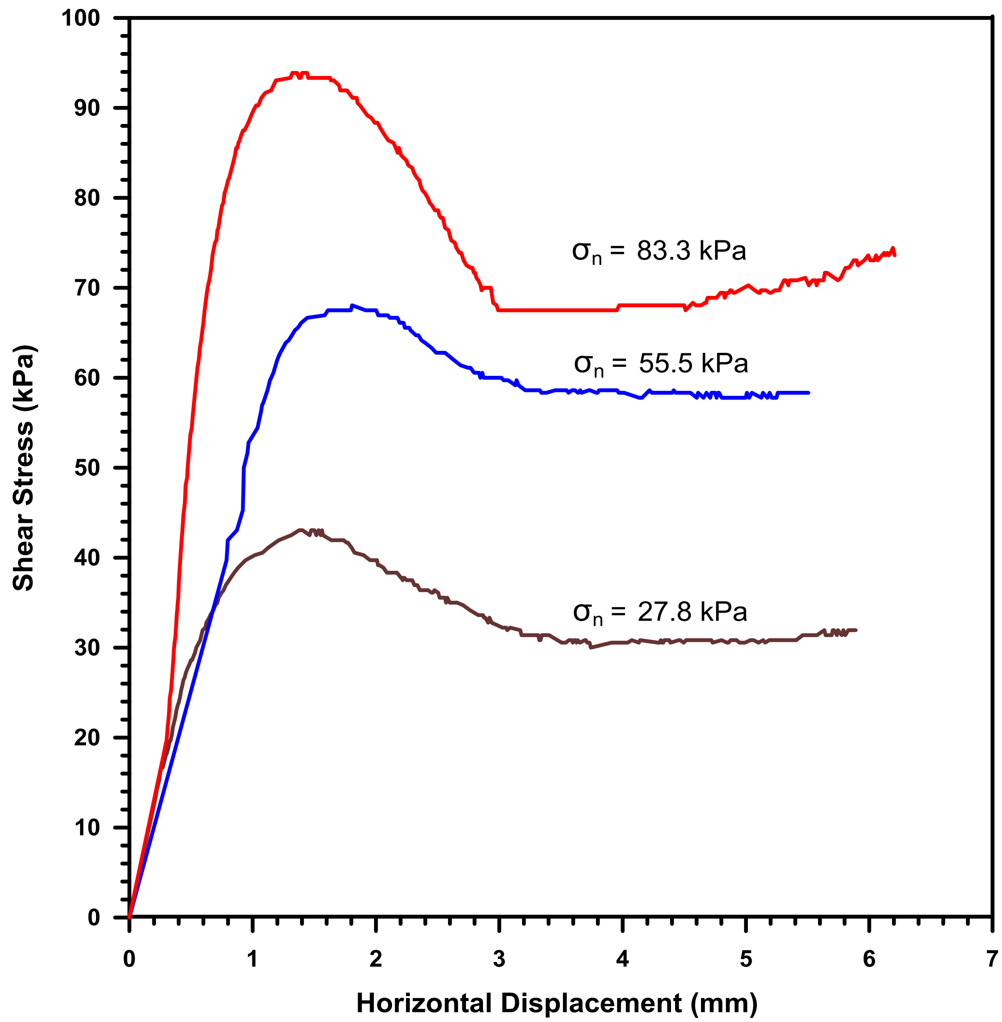


Figure 5-3: Result of direct shear test for medium dense dry natural sand (1% silt content).

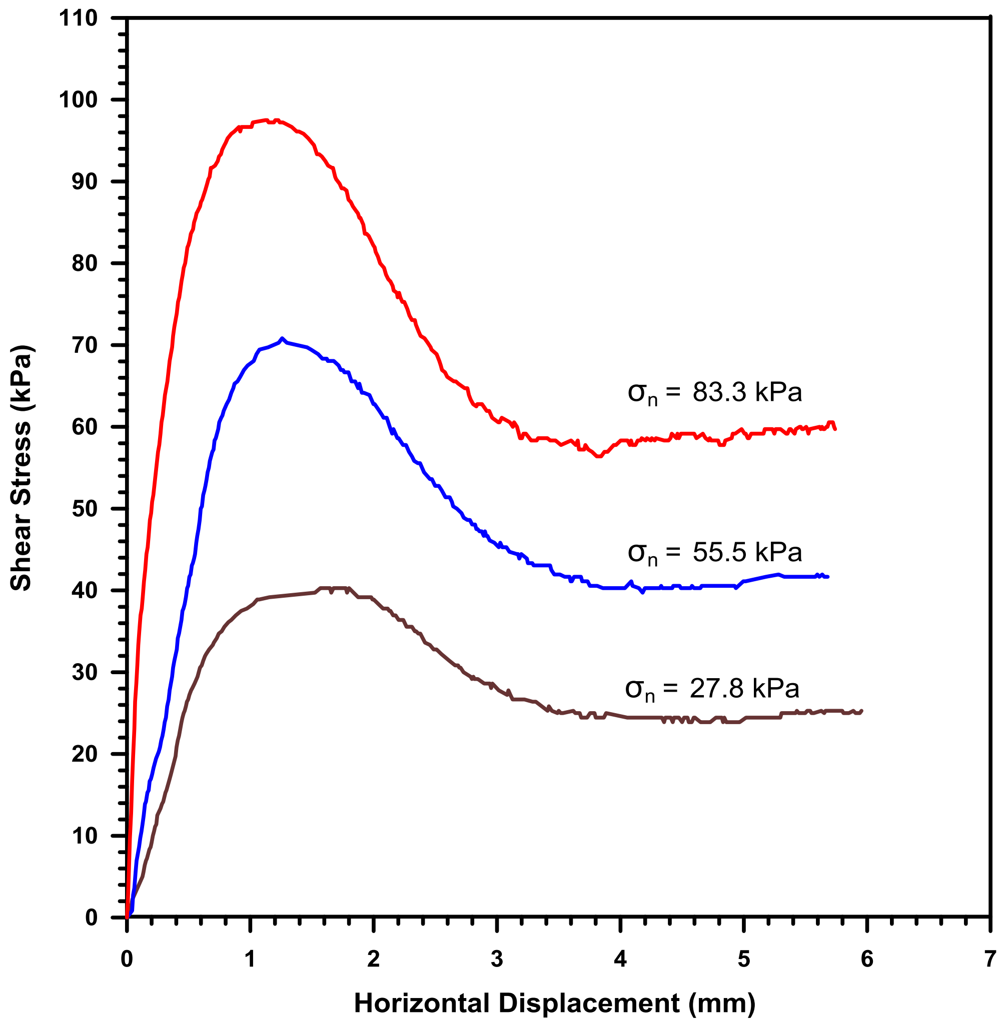


Figure 5-4: Result of direct shear test for dense dry natural sand (1% silt content).

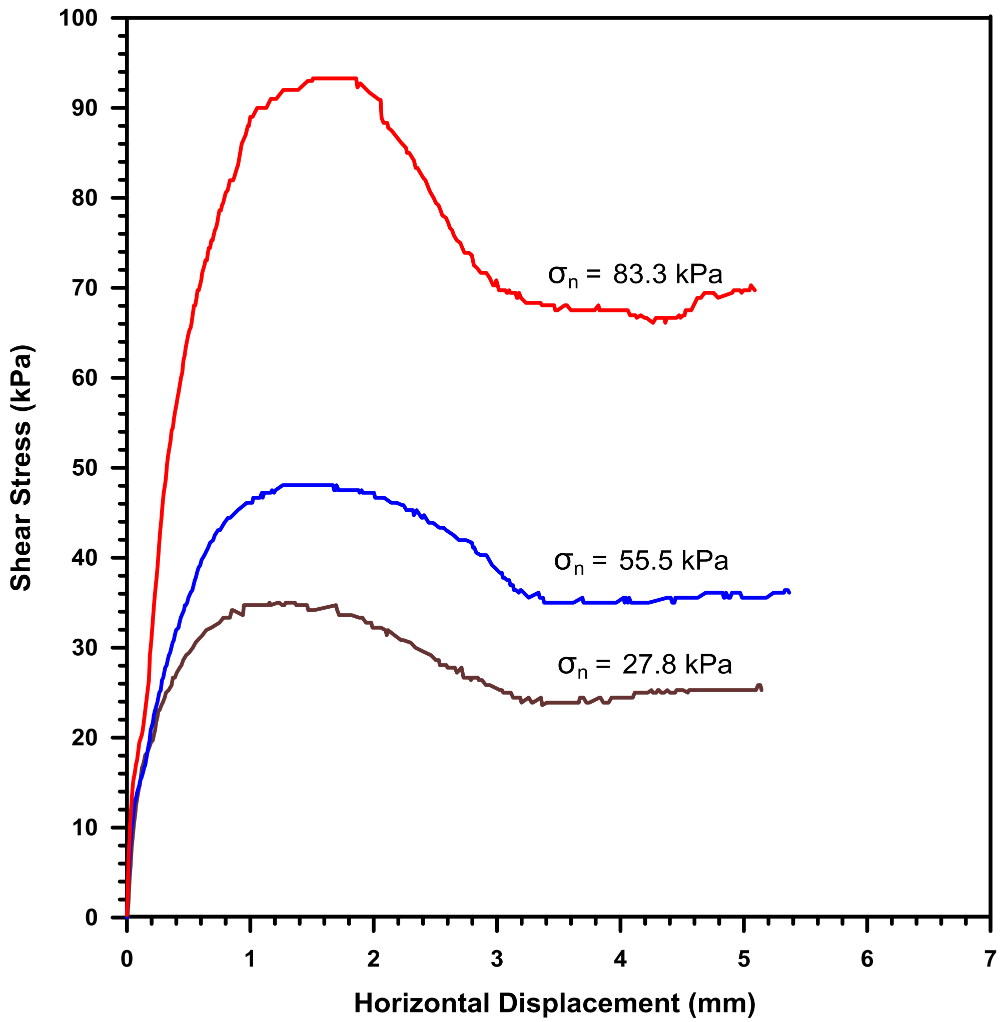


Figure 5-5: Result of direct shear test for loose dry natural sand (4% silt content).

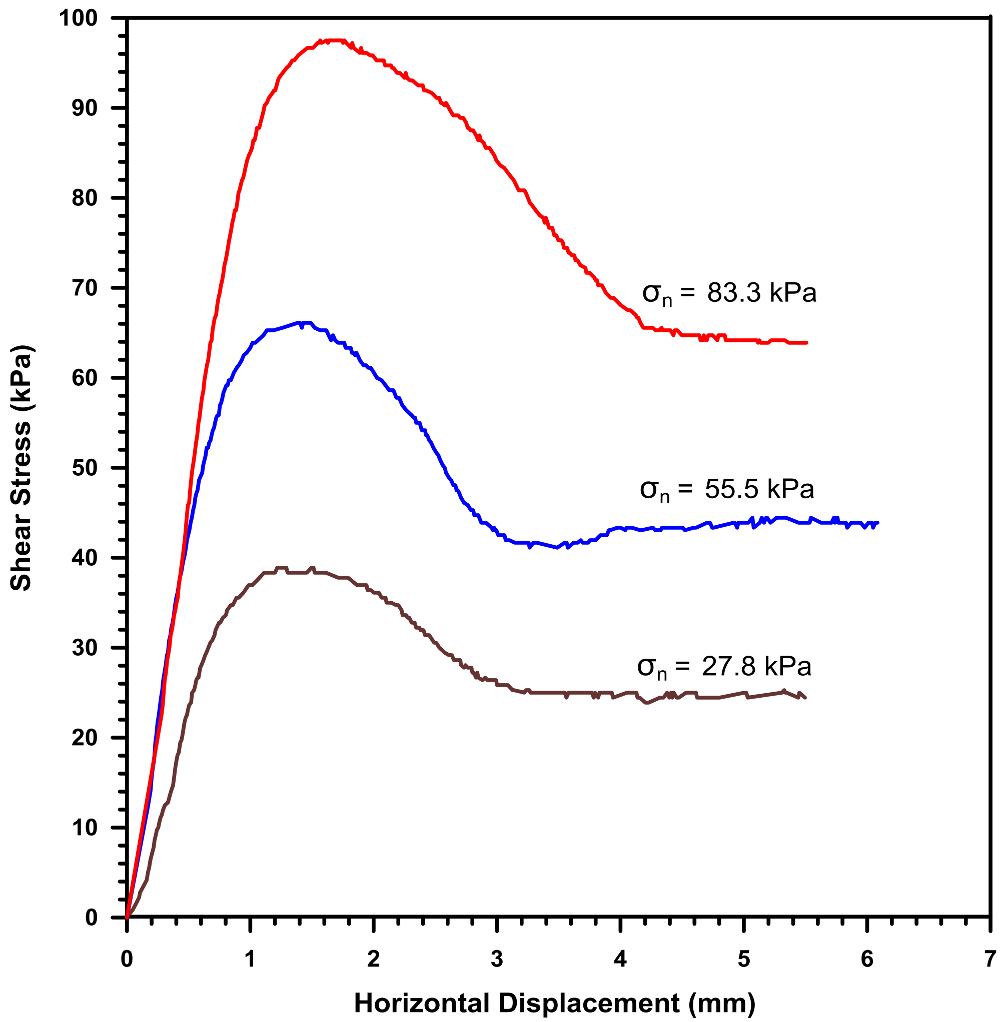


Figure 5-6: Result of direct shear test for medium dense dry natural sand (4% silt content).

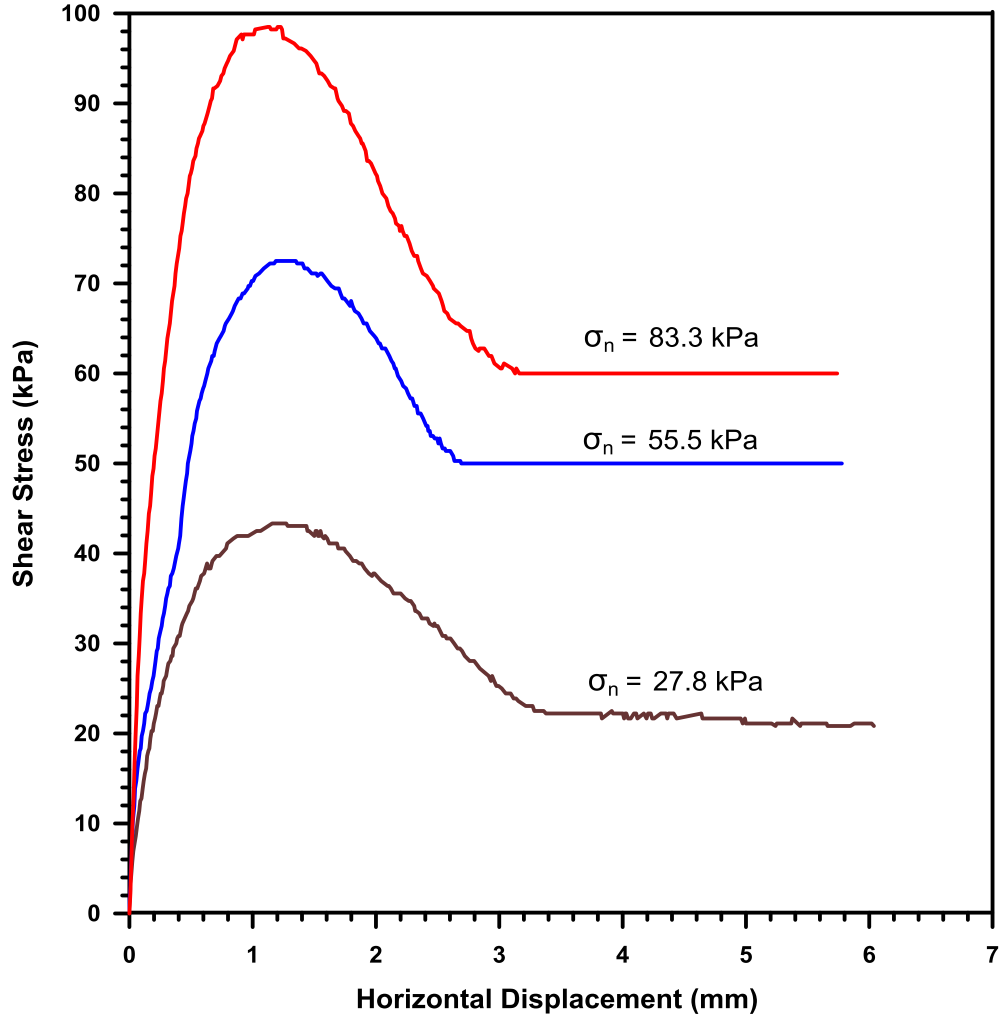


Figure 5-7: Result of direct shear test for dense dry natural sand (4% silt content).

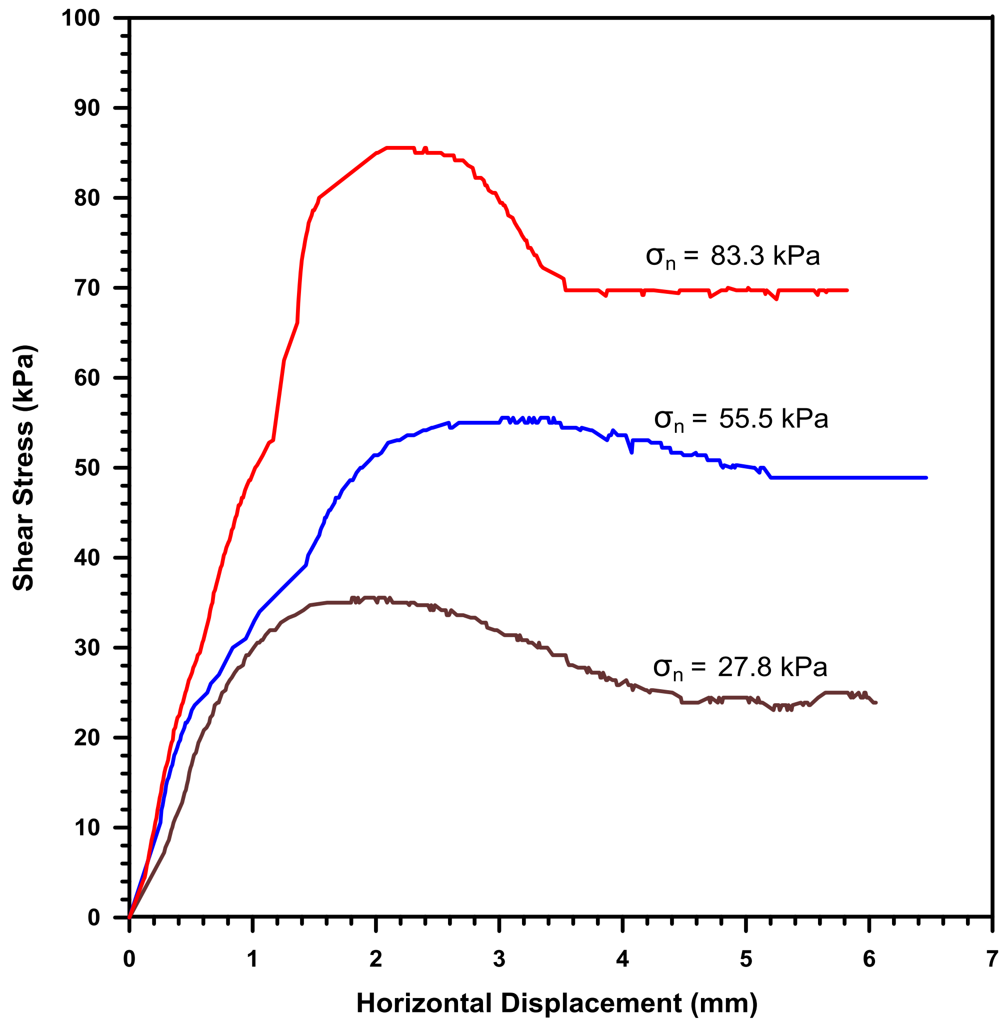


Figure 5-8: Result of direct shear test for loose dry natural sand (8% silt content).

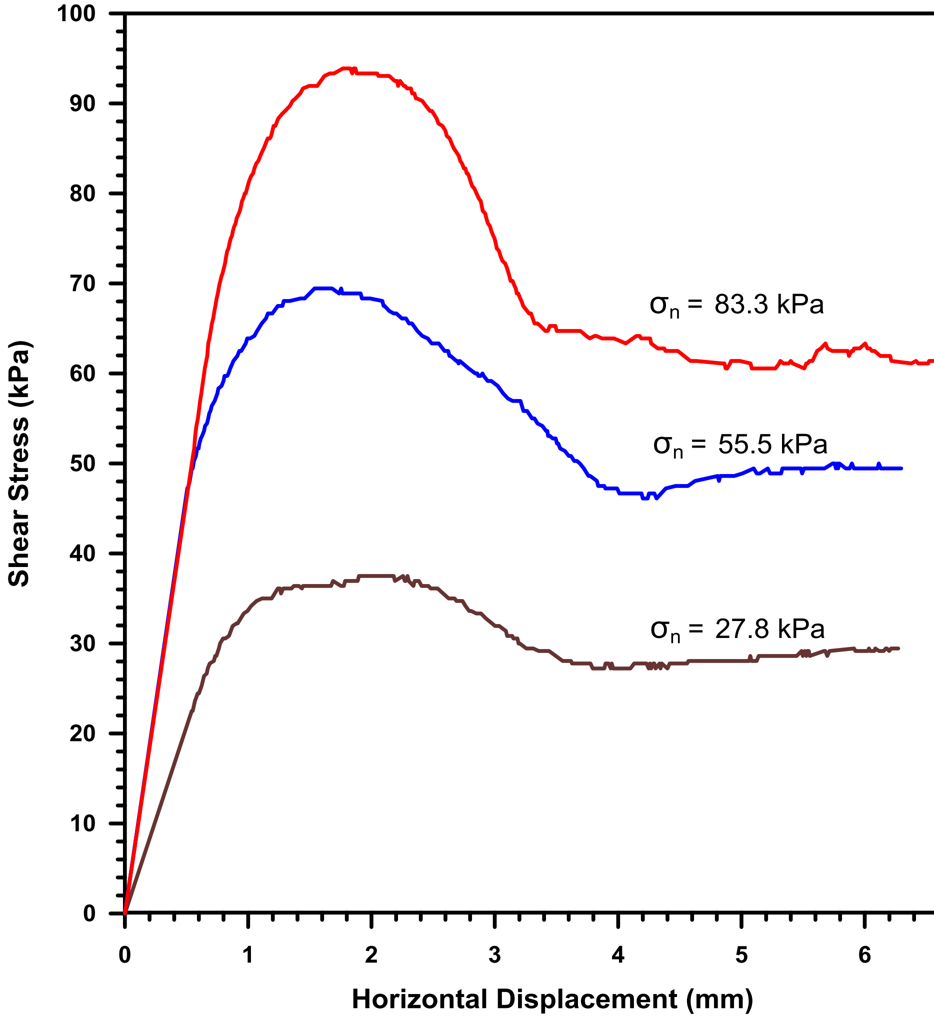


Figure 5-9: Result of direct shear test for medium dense dry natural sand (8% silt content).

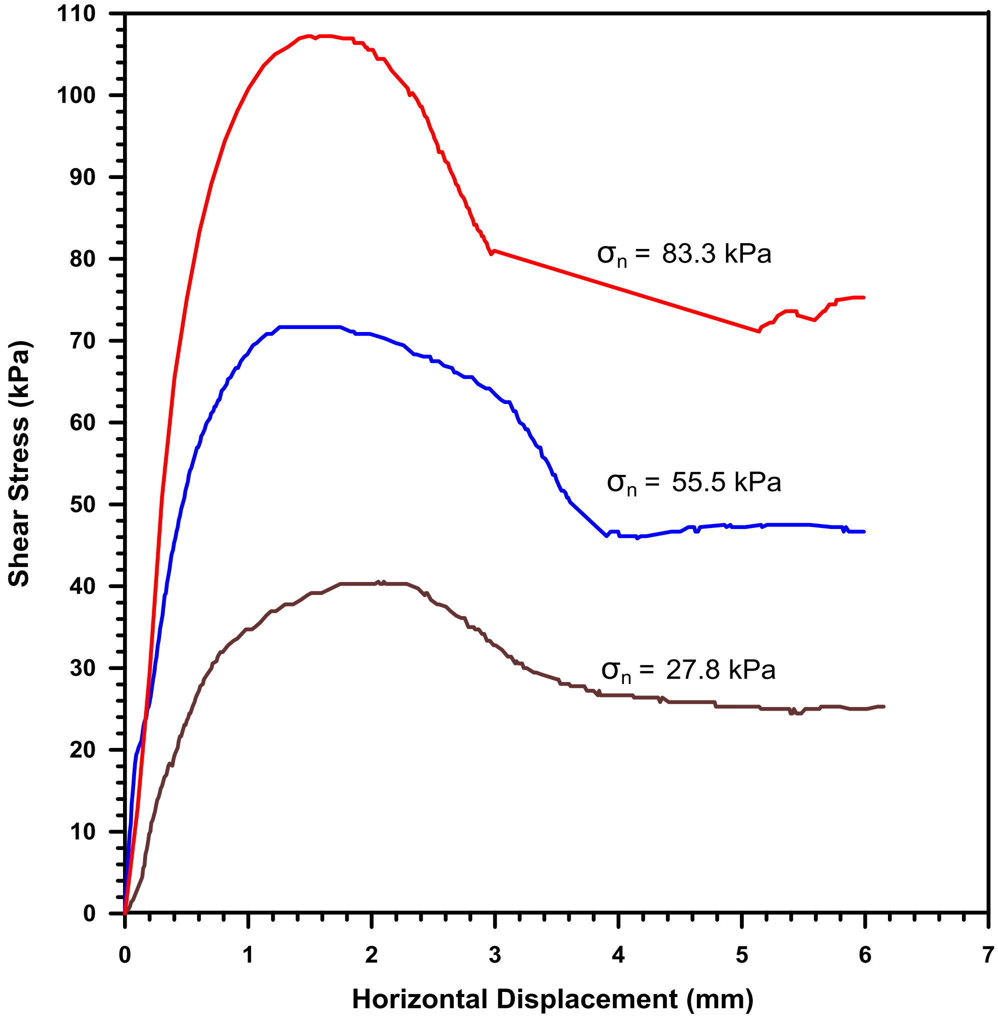


Figure 5-10: Result of direct shear test for dense dry natural sand (8% silt content).

depth in mm by the number of blows (mm/blow) for the DCPTs and reported for each sample.

DCP Test Results on Natural Sand (with 1% Silt Content)

Typical results obtained from dynamic cone penetration tests were presented in Figures 5.11 through 5.14, to illustrate the effect of variations of densities of natural sand (1% silt content) on DCPT results.

Figure 5.11 presents the variation of penetration resistance with depth (up to 1500 mm) for three dynamic cone penetration tests on loose sand to check the repeatability of the results. It should be noted that there is no significant penetration resistance in loose sand layer; which is mainly due to the low density and low confining pressure effects on the dynamic cone penetration test results. Loose soils tend to compress as the cone penetrates and thus offering low resistance.

Figure 5.12 presents the variation of penetration resistance with depth for dynamic cone penetration tests on dry medium dense sand. The dynamic cone penetration test was conducted on three different locations. It should be noted that there was a significant increase in the penetration resistance for the medium dense sand layer, compared to the loose sand, due to the increase in density and the vertical confining pressure effects.

Figure 5.13 presents also the variation of penetration resistance with depth for dynamic cone penetration tests, but on very dense sand. Three dynamic cone penetration tests were conducted in this case. It should be noted that there is a high penetration resistance for dense sand. It has been observed as a result of this resistance a noticeable change in the

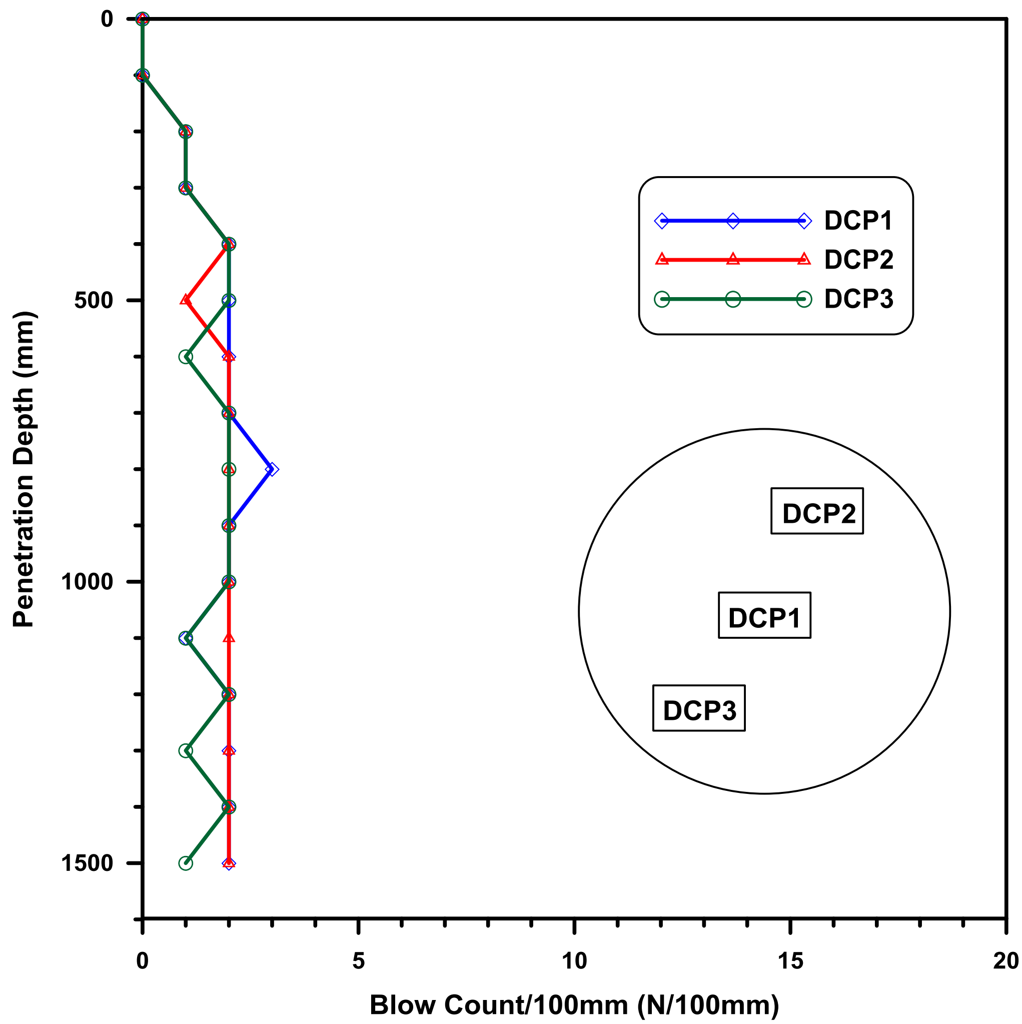


Figure 5-11: DCP resistance for natural loose dry sand (1% silt content).

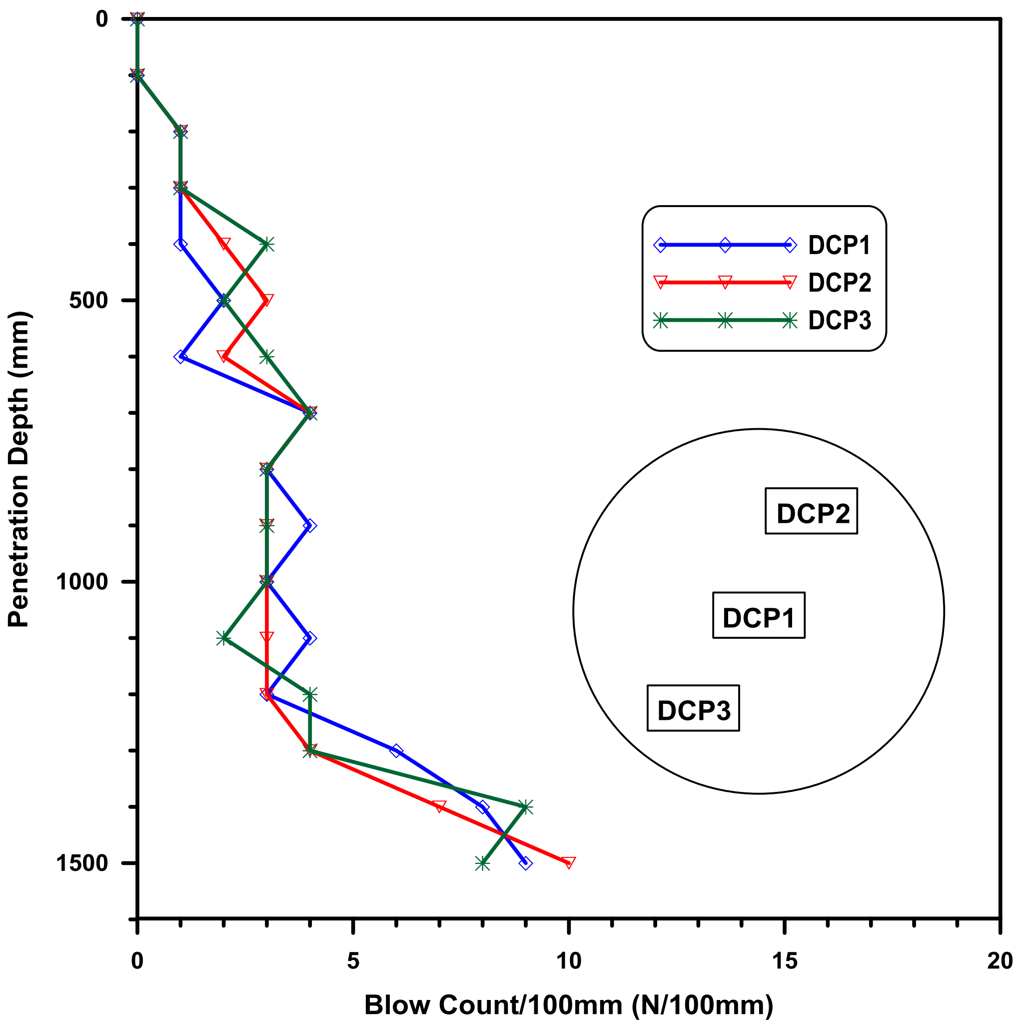


Figure 5-12: DCP resistance for natural medium dense dry sand (1% silt content).

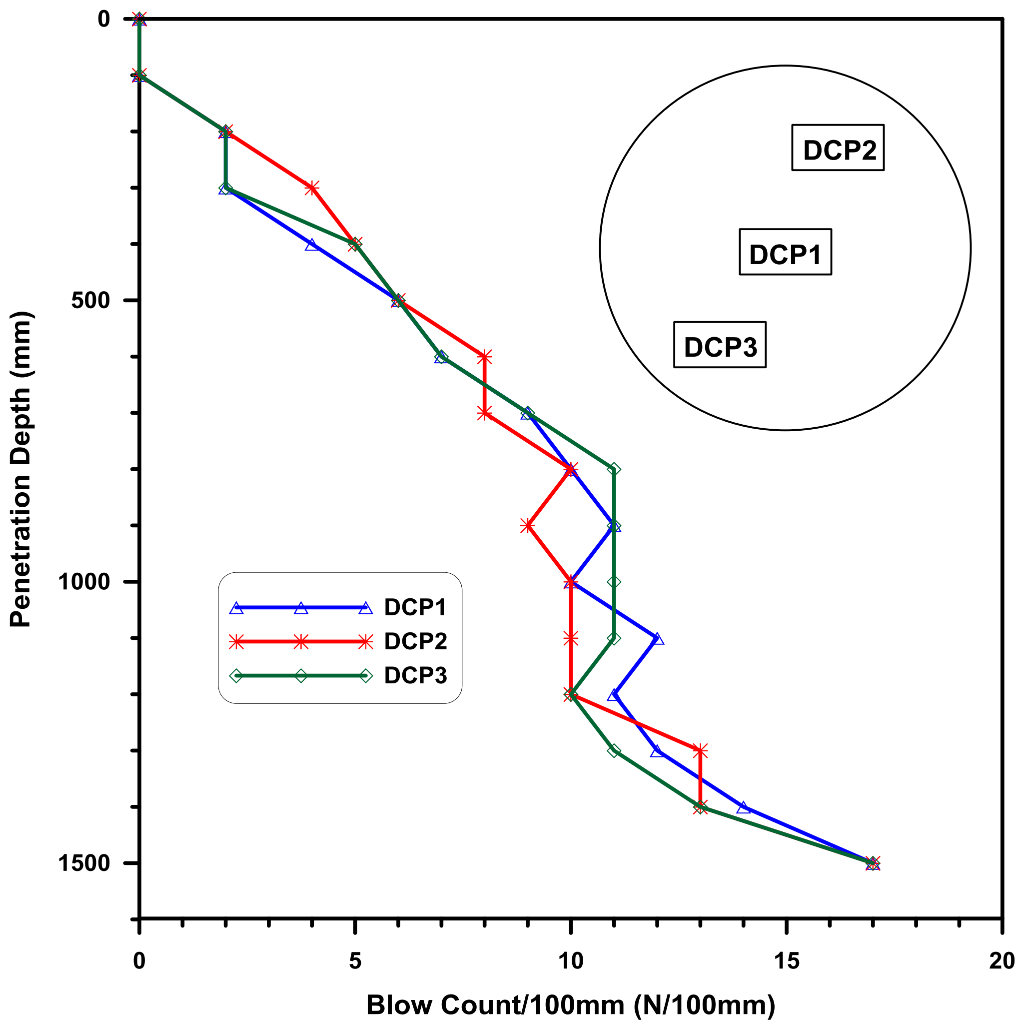


Figure 5-13: DCP resistance for natural dense dry sand (1% silt content).

results of dynamic cone penetration index (DCPI) compared to the situation of loose and medium sands.

The repeatability of the DCP test results is an important consideration that was evaluated for all test sets. To evaluate the repeatability, several tests (usually three) were carried out for each density set. Figure 5.14 shows the results of the nine series tests undertaken for three different relative densities (D_r) for natural sand (1% silt content). It was difficult to assess such density variations. On the contrary, for the loose samples where no densification was done, the DCP resistance did not change with depth. The high resistance was also attributed to the tendency of dense sands to dilate upon penetration of the cone and thus the resistance increased.

DCP Test Results on Sand with 4% Silt Content

Dynamic cone penetration tests were performed on dune sand with 4% silt content at different densities. Figure 5.15 presents the variation of DCP resistance with depth (up to 1500 mm) for three tests on loose sand with 4% silt content. It was observed that the penetration resistance was found to be similar to that observed in the loose sand layer with 1% silt content.

Figure 5.16 depicts the variation of DCP resistance with depth for on medium sand with 4% silt content. Three dynamic cone penetration tests were conducted on different locations. The results show relatively higher DCP resistance in the medium sand compared to the situation of loose sand due to the density effect.

In the case of dense sand with 4% silt content, as depicted in Figure 5.17, the soil seems very densely compacted for depths below 600 mm; a situation showing the effect of density, thereby producing high penetration resistance (more than 10 Blows/ 100 mm) in most tested locations. At a depth of 600 to 1500 mm from the surface, the penetration resistance was marginally reduced but still indicating a very dense material.

Figure 5.18 shows the results of the nine (three series) tests undertaken for three different relative densities (D_r).

DCP Test Results on Sand with 8% Silt Content

Nine DCP tests were performed on sand with 8% silt content at three different relative densities. Figure 5.19 presents the variation of penetration resistance with depth for three dynamic cone penetration tests on loose sand with 8% silt content. It was observed that the penetration resistance was similar to that observed in the loose sand with 1% and 4% silt contents for the top portion (between depth of 300 and 1000 mm). In addition, the bottom portion (below 1000 mm) shows relatively high penetration resistance. This may indicate a higher density for this portion.

Figure 5.20 presents the DCP resistance for medium dense sand with 8% silt content. It could be noted that there is an increase in the DCP penetration resistance in the medium sand compared to the situation of loose sand due to density effects.

Figure 5.21 presents the case of dense sand with 8% silt content. The penetration resistance was found to be much more than that observed in the dense sand with 4% silt content particularly at top layer (from surface to a depth of 700 mm). However, the penetration resistance of the layer between 700 to 1500 mm seems denser than other densities and the average DCP was in excess of 13 Blows/100mm, for most of the tested

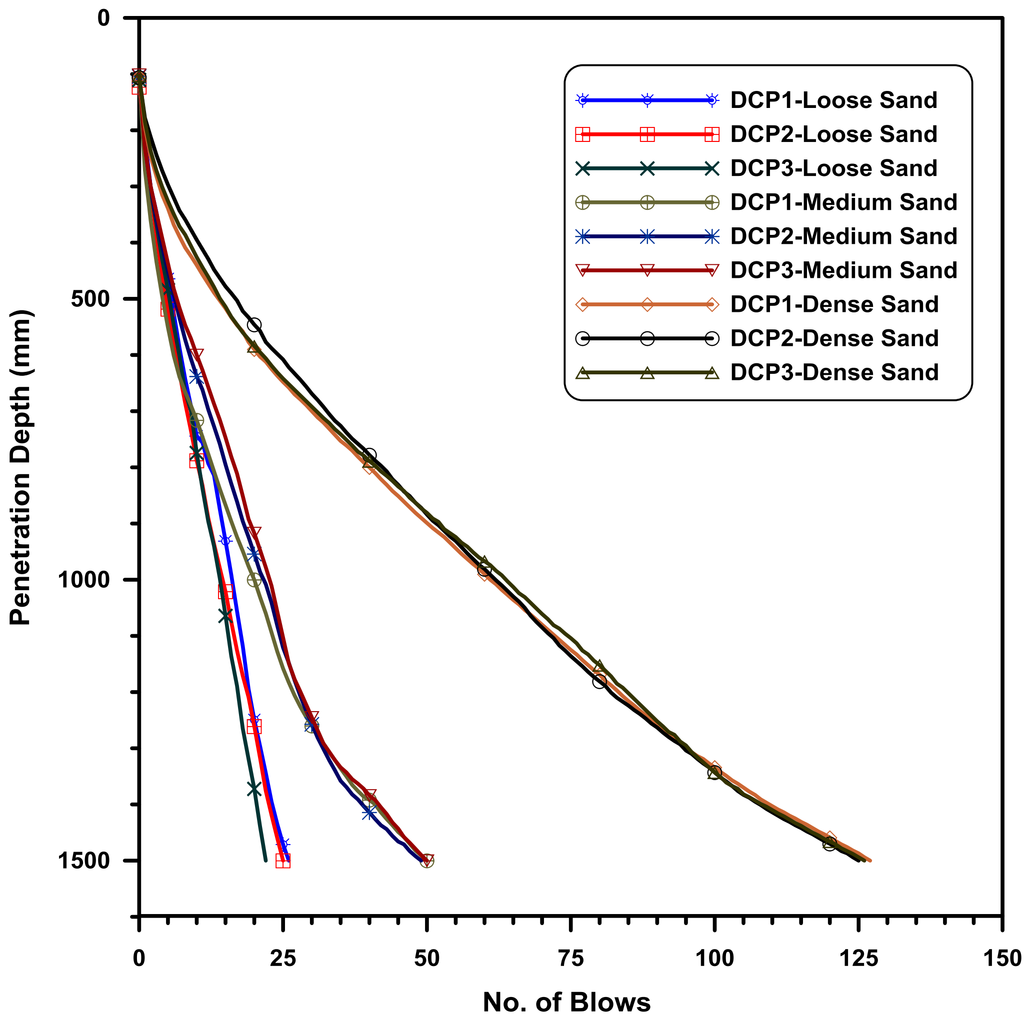


Figure 5-14: Variation of No. of blows with depth for natural dry sand (1% silt content) at different densities.

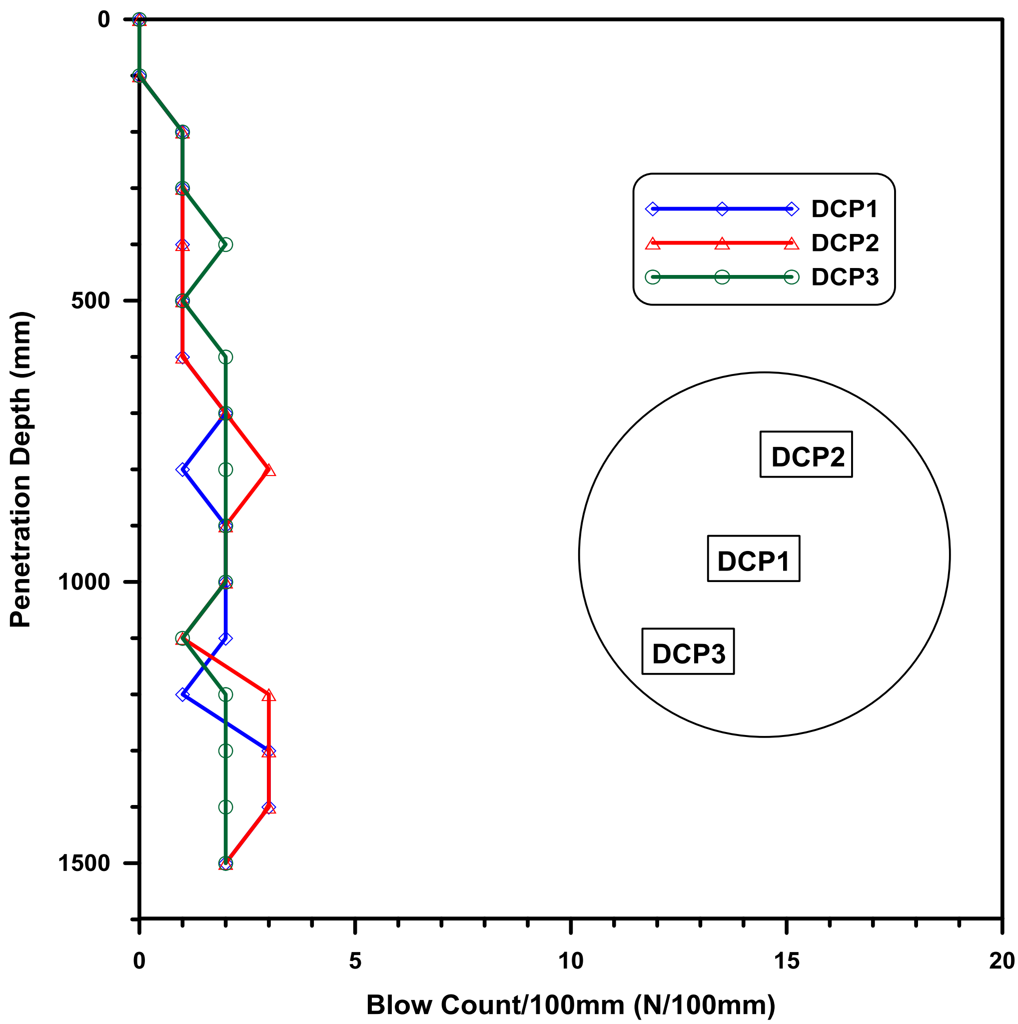


Figure 5-15: DCP resistance for loose dry sand with 4% silt content.

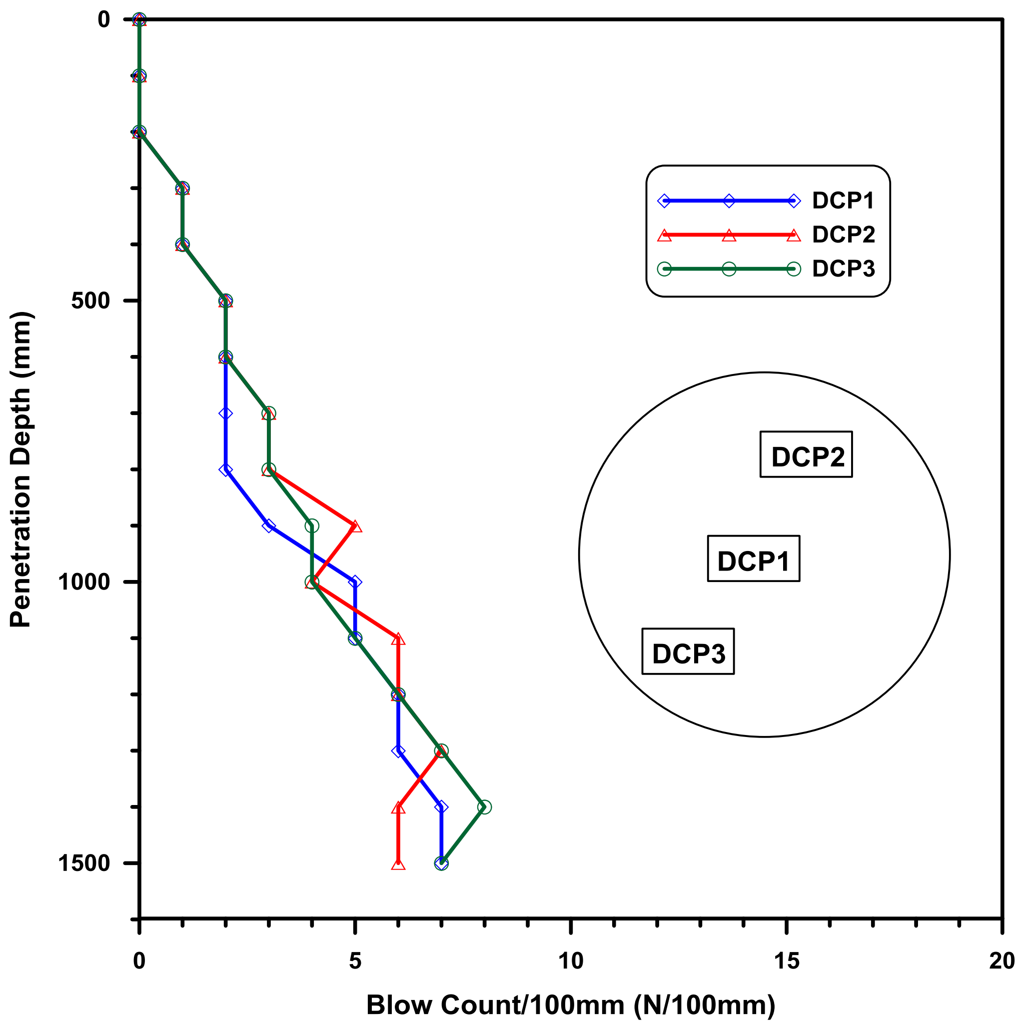


Figure 5-16: DCP resistance for medium dense dry sand with 4% silt content.

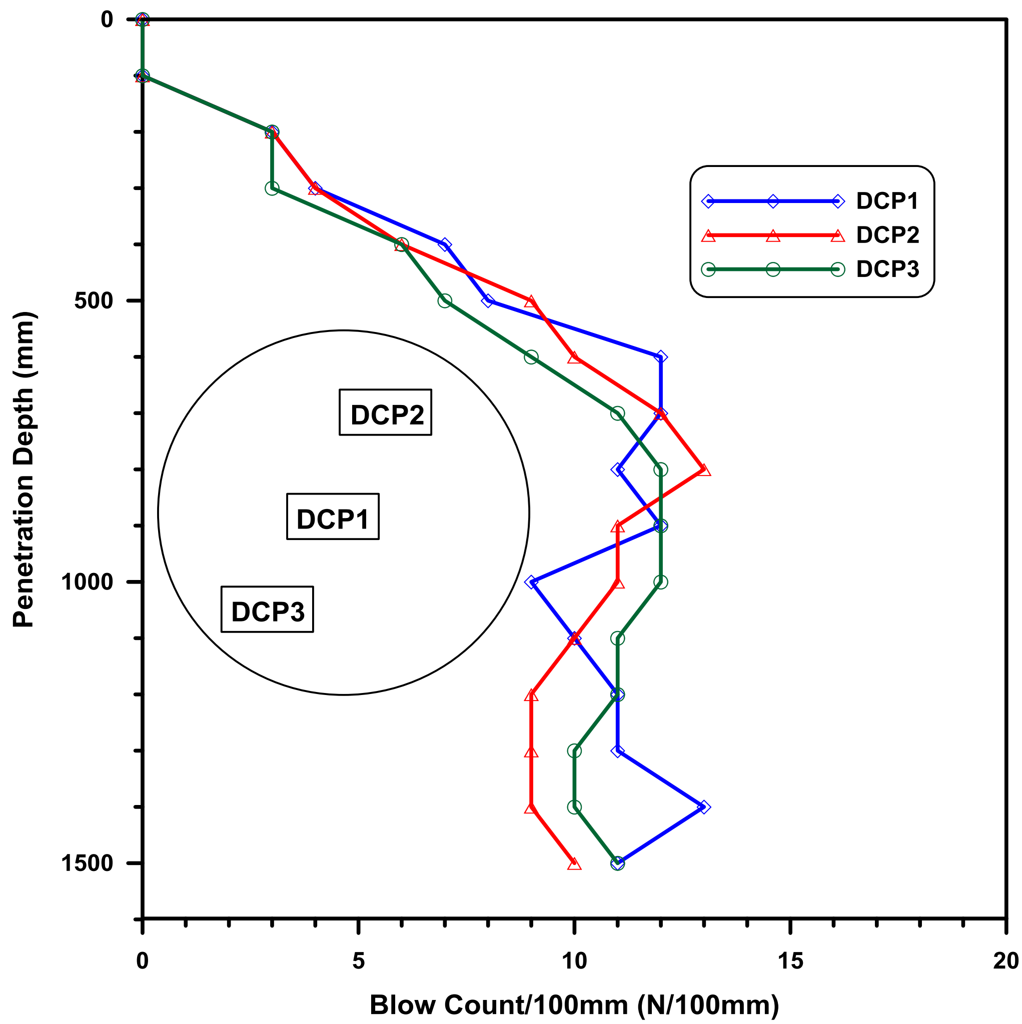


Figure 5-17: DCP resistance for dense dry sand with 4% silt content.

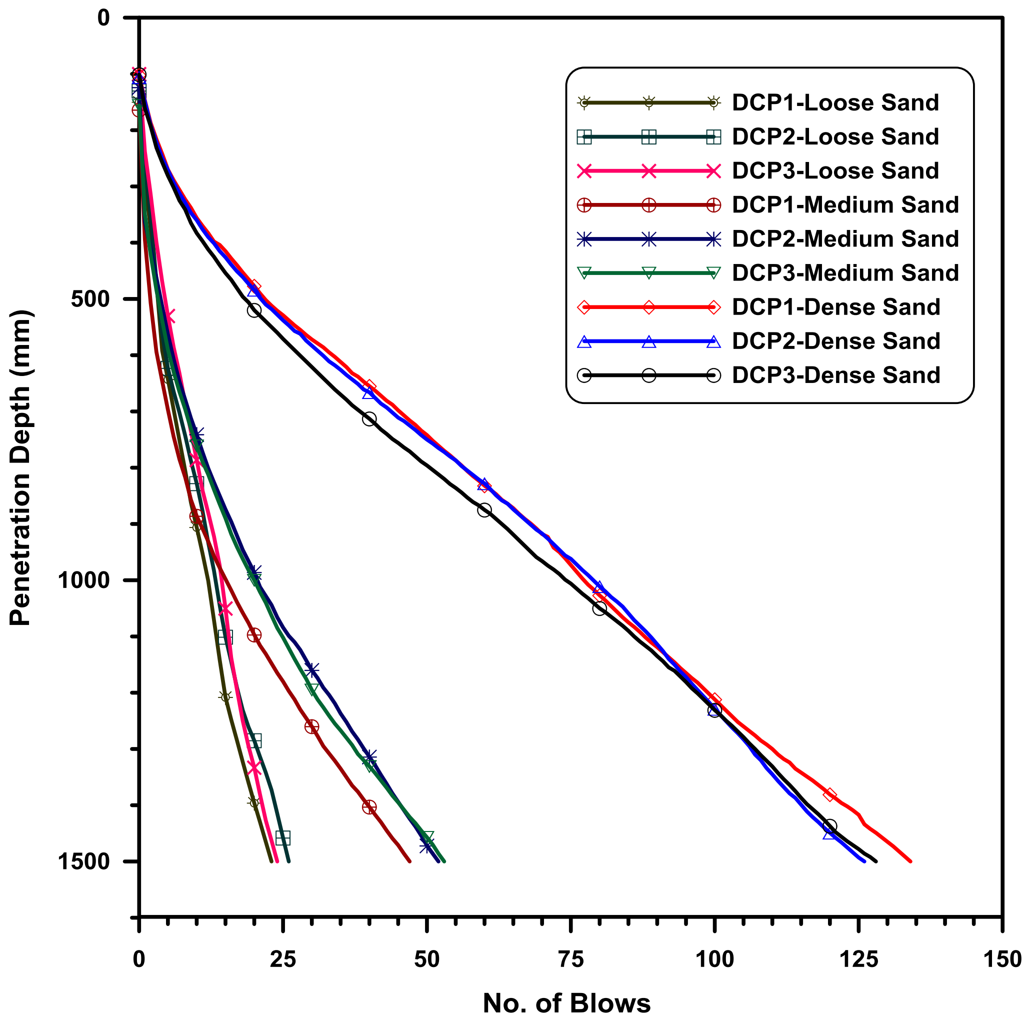


Figure 5-18: Variation of No. of blows with depth for dry sand with 4% silt content at different densities.

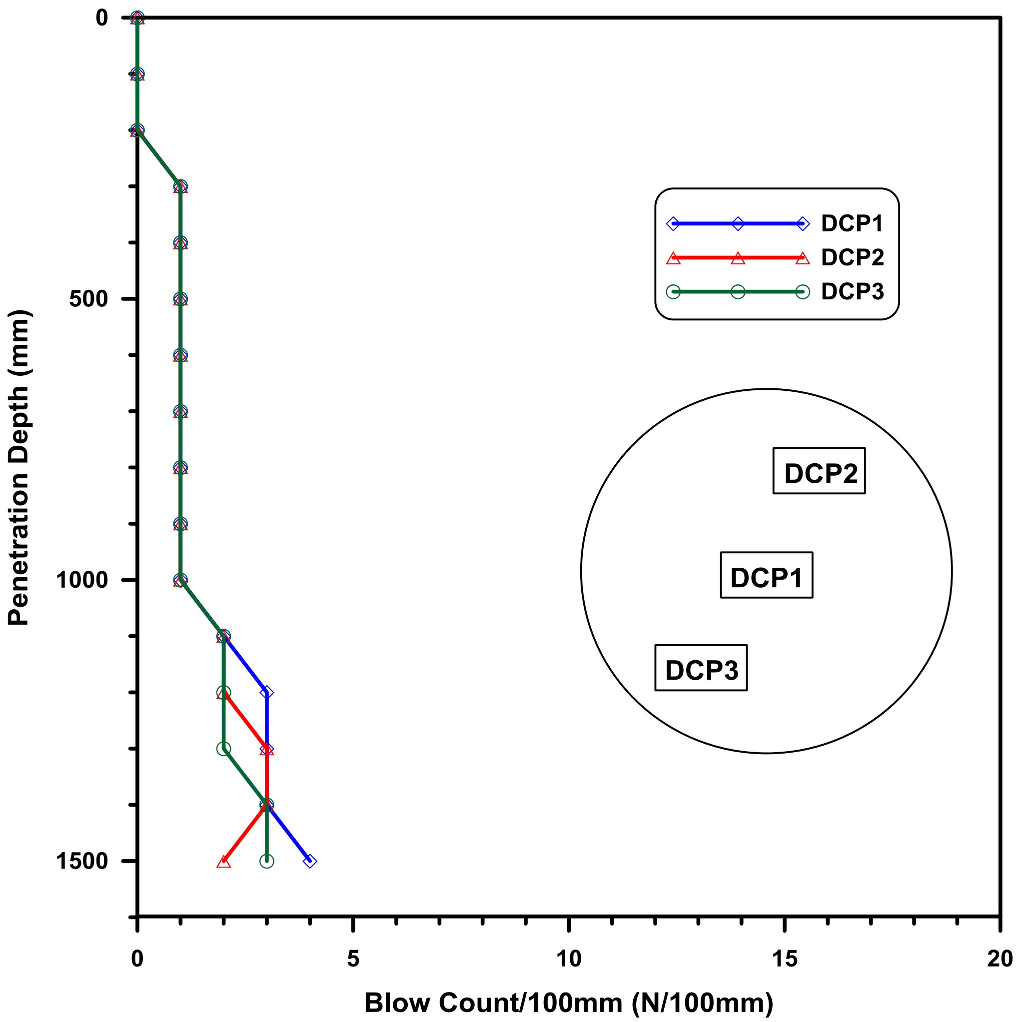


Figure 5-19: DCP resistance for loose dry sand with 8% silt content.

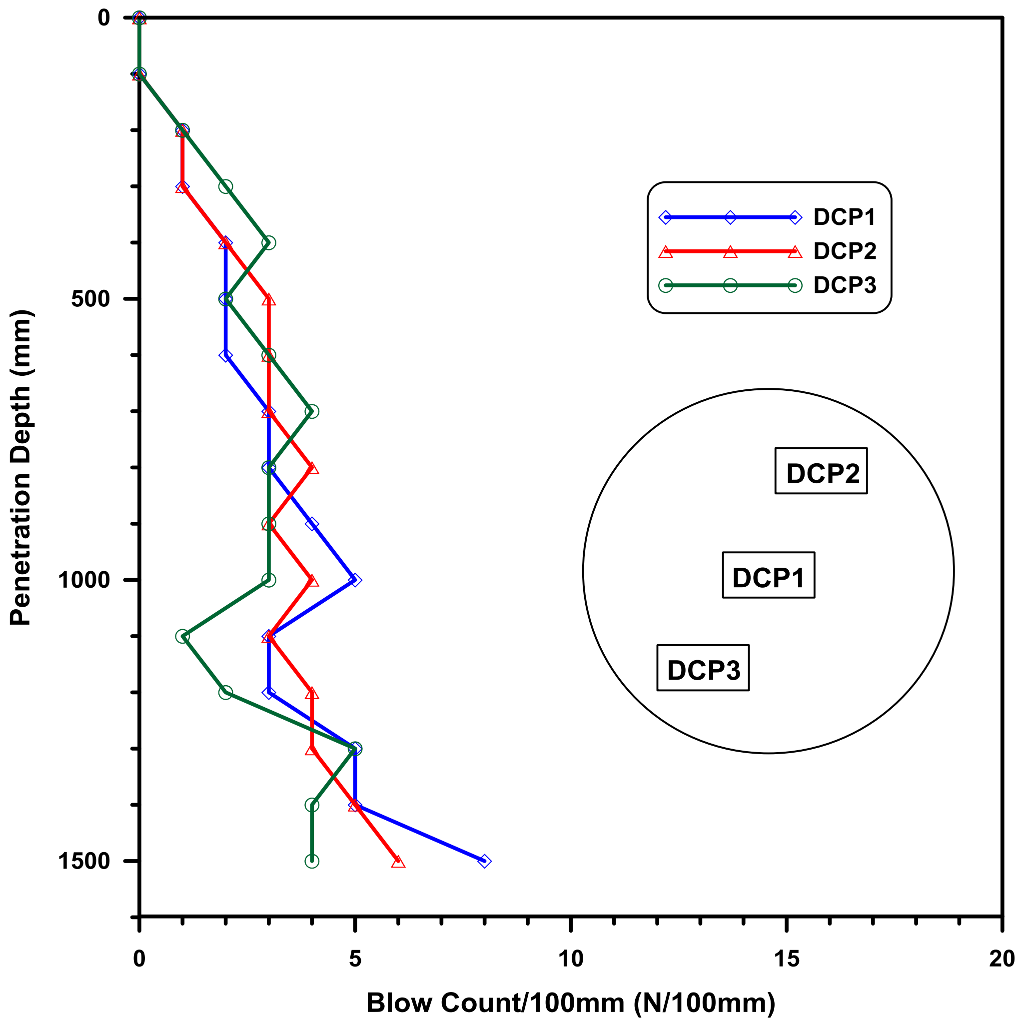


Figure 5-20: DCP resistance for medium dense dry sand with 8% silt content.

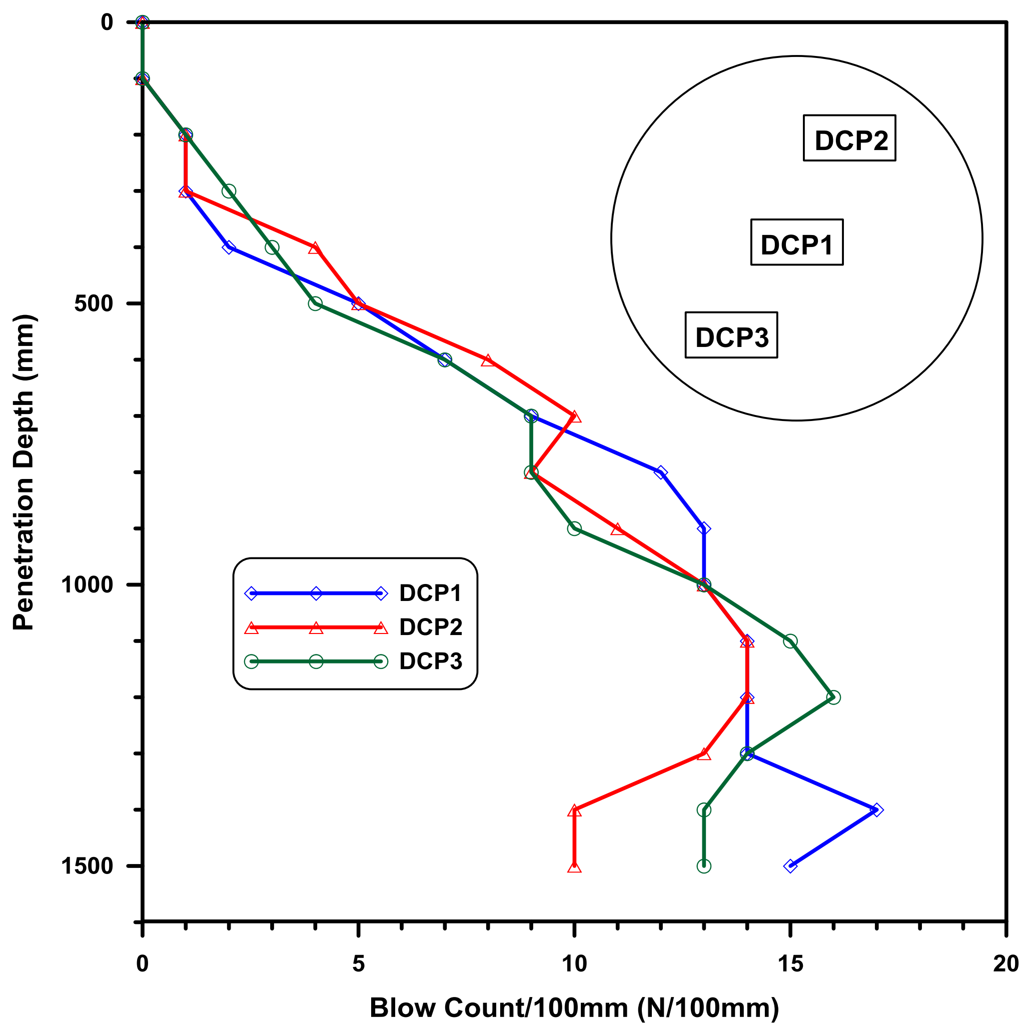


Figure 5-21: DCP resistance for dense dry sand with 8% silt content.

locations. Figure 5.22 summarizes the variation of number of blows/100 for the three relative densities.

It was observed that the blow counts per unit depth increased significantly as the density of sand increased for different silt content (1%, 4% and 8%). This is ascribed to the fact that compacted dry soils have higher stiffness and offer greater resistance to penetration in the case of sandy and silty soils (Kim and Kim, 2006) and the vertical confining pressure has increased with depth.

5.1.4 Effect of Silt Content on DCPT Result

Based on DCP investigations performed on sands (loose, medium, and dense), the effect of variations in the percentage of silt content on the penetration resistance was also analyzed. The variation of the dynamic cone penetration index (DCPI) measured in mm/blow is presented in Figure 5.23. It could be observed that an increase in the percentage of relative density from 40% to 90% has resulted in a corresponding decrease in the dynamic cone penetration index (DCPI) from 63 to 8 mm/blow. This is attributed to the interlocking as the material gets denser. Silt particles fit into the voids between larger sand particles and, therefore, the void ratio of sand-silt mixtures decreased with the increase in silt content that indicated a significant positive influence on the sand density (Mitchell and Soga, 2005) despite that fact that it is not affecting the DCPI significantly.

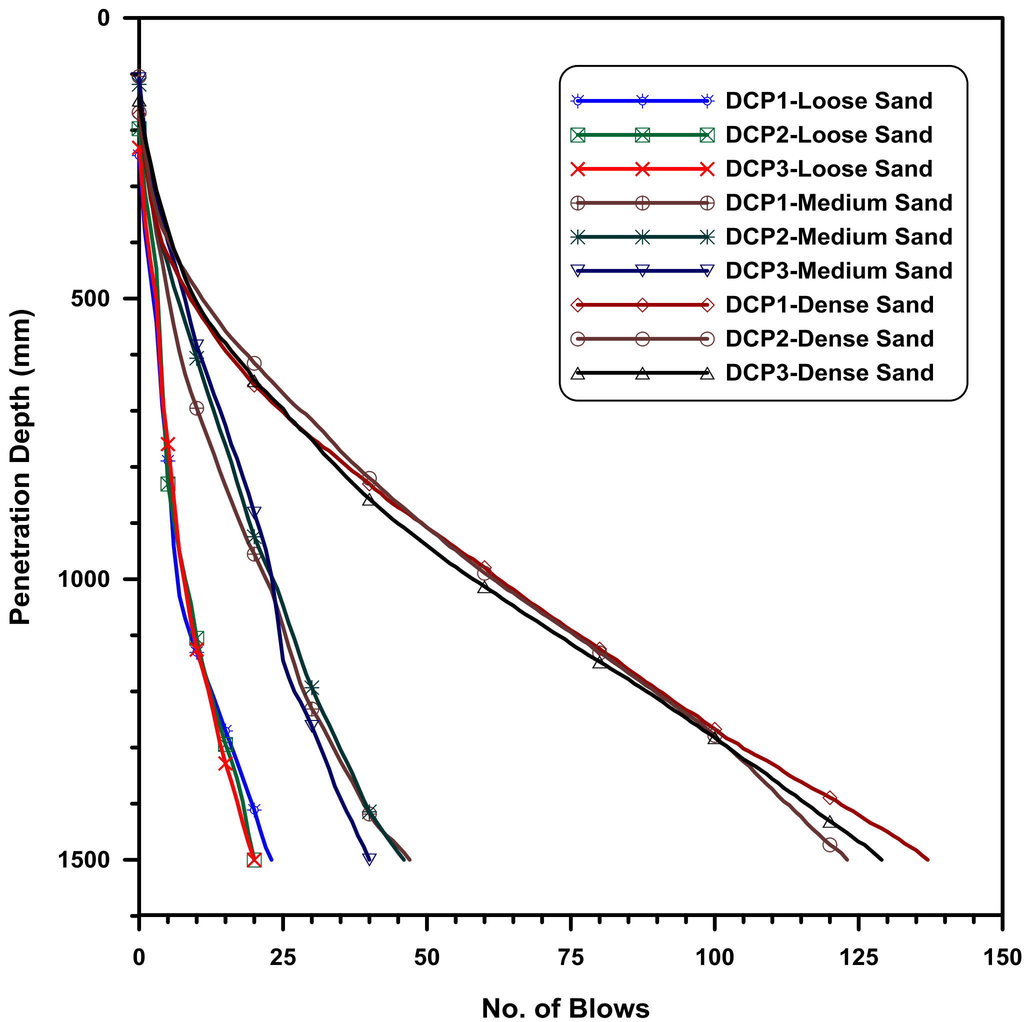


Figure 5-22: Variation of No. of blows with depth for dry sand with 8% silt content at different densities.

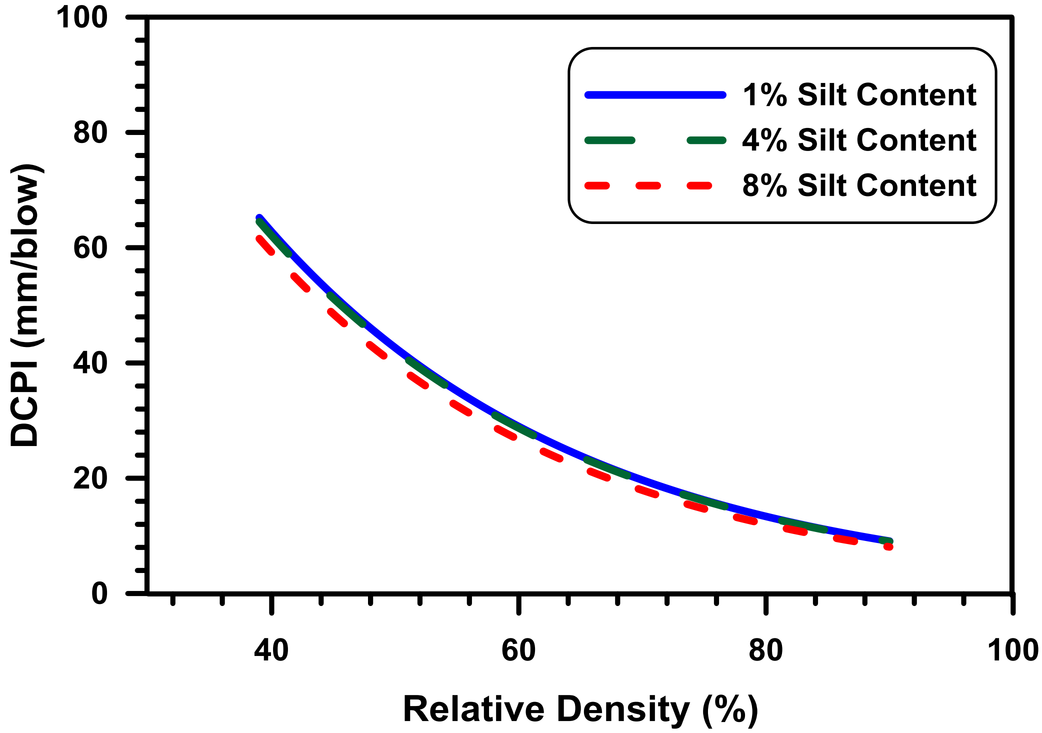


Figure 5-23: Variations of DCPI with relative densities for dry soil having different silt contents.

5.1.5 Effect of Water Level on DCPT Results

In this investigation, the effects of water table level on the DCP resistance were studied. Three levels of water table were investigated; at the base of the chamber (zero level), at 466 mm (i.e. one third of the layer thickness of the soil) above the base and fully submerged sand. Furthermore, the effect of negative pore water pressure on the DCPT results was also studied. Two DCPTs were conducted on the medium dense sand for each case, as shown in Figures 5.24 to 5.27.

Figure 5.25 shows the increase in the DCP resistance with a sudden change at the water level. The highest DCP resistance occurred at a depth of 634 from the surface of the sand sample while the water level was at a level of 466 mm from the bottom of the chamber. This high value (5 Blows/ 100 mm) occurred at the level having suction (capillary zone). It should be noticed that the DCP resistance decreased below the suction zone since the pore pressure was tending to be positive as the point gets lower towards the base of chamber. The capillary zone was estimated to be around 300 mm above the water table level. On the other hand, Figure 5.24 depicted the results of DCPT on the dry case which show the smallest DCP resistance (3 Blows/ 100 mm).

In the case of fully submerged sand, as shown in Figure 5.26, the DCP resistance value between 2 and 3 Blows/ 100 mm. The sand pores were filled with water and the soil is assumed to be saturated below the water level. However, hydrostatic pressure results from the water weight, and the uplift force due to buoyancy that reduced the effective weight of water-filled sand, which leads to smaller skeletal forces for submerged sand compared to dry sand (Mitchell and Soga, 2005). Therefore, the effective stress of sand

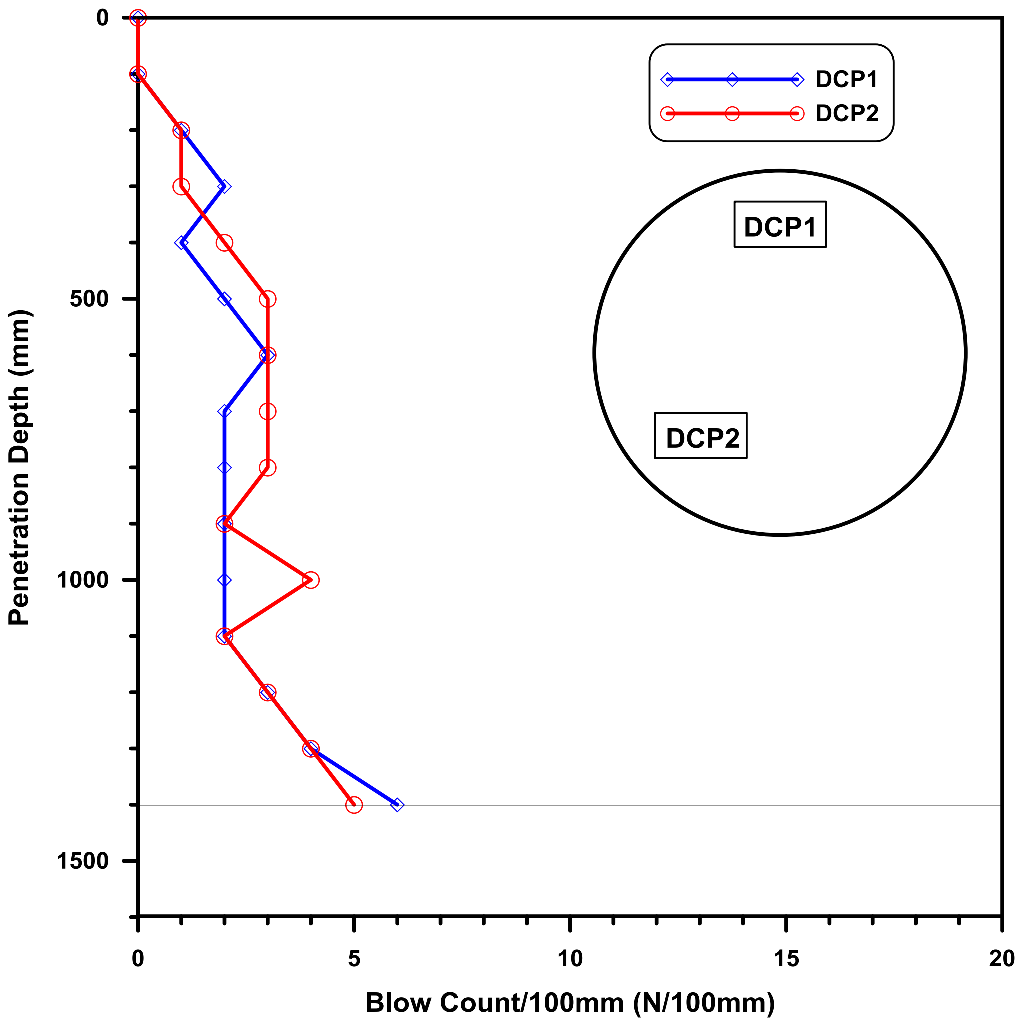


Figure 5-24: DCP resistance for medium dense dry natural sand.

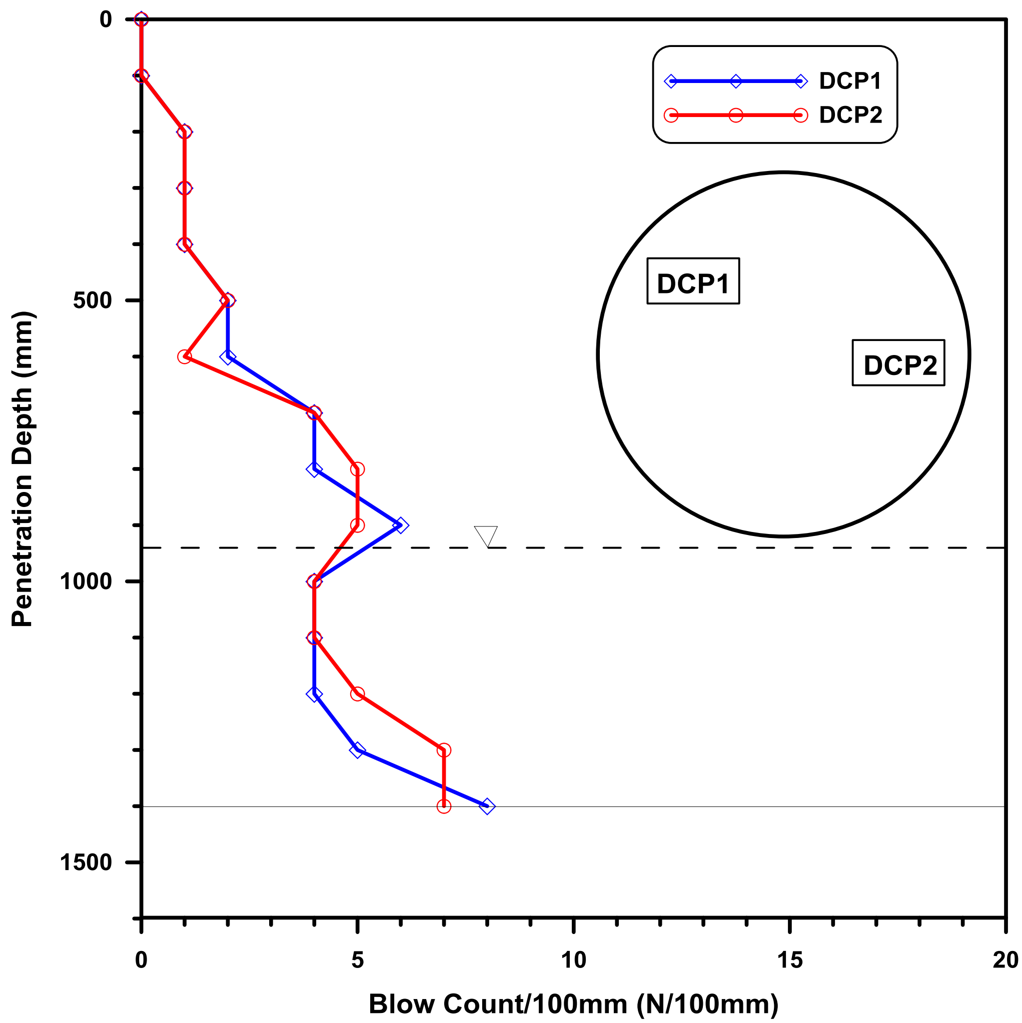


Figure 5-25: Effect of water table level (WT level = 466 mm) on DCP resistance of medium dense natural sand.

has decreased due to positive pore water pressure. These observations for submerged sand confirmed that the penetration resistance decreased when the pore pressure was positive and thus indicating a significant negative influence on the stiffness of sand.

In the case of drainage where the water was removed from the sample and testing was done right after drainage, as depicted in Figure 5.27, the DCP resistance value was 8 Blows/ 100 mm at a depth 700 mm. The high resistance was attributed to the presence of negative pore water pressure when compared to those for submerged or dry sand. Increases in DCP resistance of as much as 166% for a relative density of 60% were recorded, as shown in Figure 5.28. Suction, in particular, increased the shear strength and may add to the stiffening of the soil deformation response (Russell and Khalili, 2006; Masin and Khalili, 2008). Figure 5.28 summarized the four cases for the effect of water level on the DCPT results.

5.1.6 Dynamic Cone Penetration Test Correlations

Based on the laboratory results of the present research, correlations were established between DCPI with density, relative density, void ratio and angle of internal friction of the sand with different silt content. Statistical approach has been applied to find the best correlations of the results with a high coefficient of determination.

DCPI vs Density

Based on the data developed in table 5.3 and Figures 5.14, 5.18 and 5.22, the relations of dynamic cone penetration index (DCPI) and dry density of sand with different silt content (1%, 4% and 8%) were depicted in Figures 5.29, 5.30 and 5.31, respectively, and the best correlations could be described as follows:

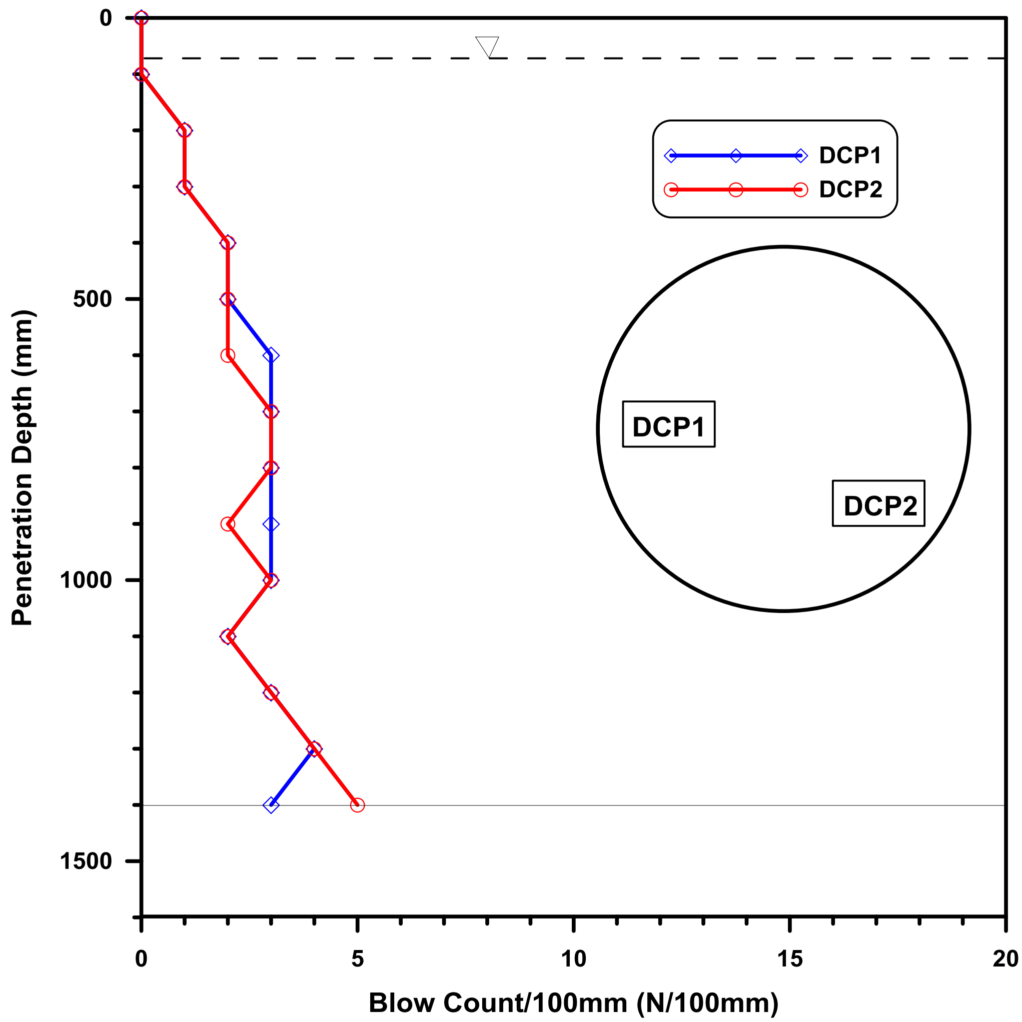


Figure 5-26 Effect of full submergence (WT level = 1400 mm) on DCP resistance of medium dense natural sand.

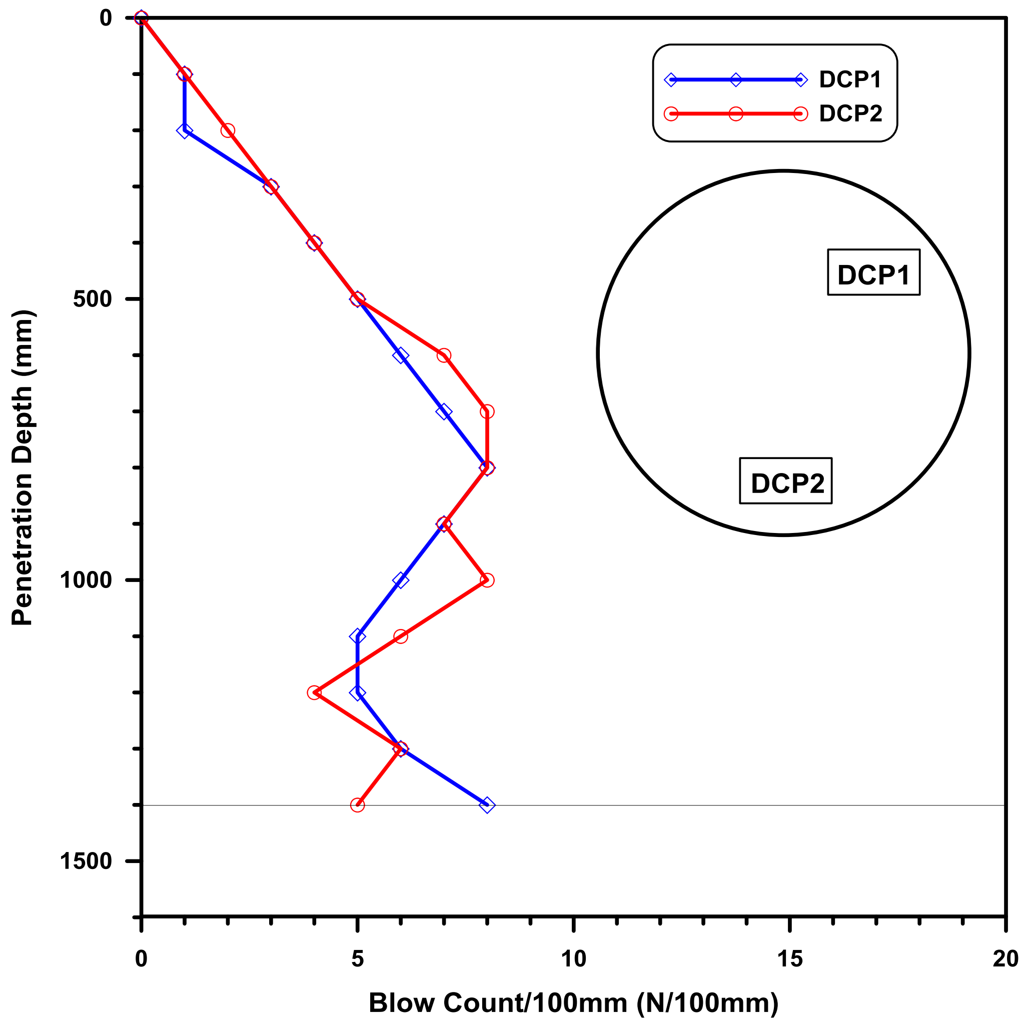


Figure 5-27: Effect of negative pore pressure on DCP resistance of medium dense natural sand.

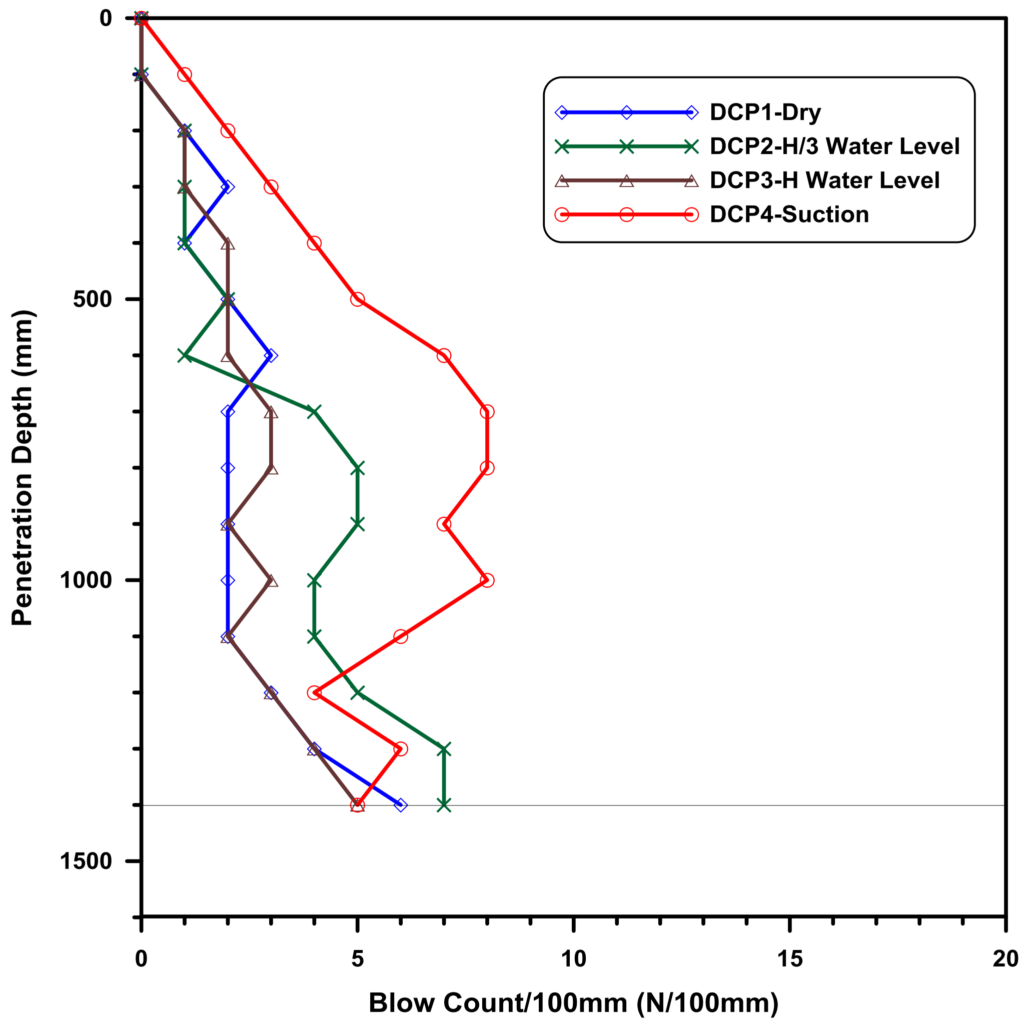


Figure 5-28: Effect of water level for four cases of medium dense natural sand.

For 1% silt content: $\gamma_d = 1.96/(DCPI)^{0.03}$ $(R^2 = 0.99)$ (5.1)

For 4% silt content: $\gamma_d = 2.03/(DCPI)^{0.04}$ $(R^2 = 0.99)$ (5.2)

For 8% silt content: $\gamma_d = 2.04/(DCPI)^{0.04}$ $(R^2 = 0.99)$ (5.3)

Where:

γ_d: Dry density (ton/m^3)

DCPI: Dynamic cone penetration index

R^2: Coefficient of determination

It was observed that an increase in the sand density resulted in a corresponding decrease in the DCPI for different silt content.

DCPI vs Relative Density

The relative density is an appropriate parameter to describe the consistency of sands (Coduto, 2001). Based on the data developed in table 5.3 and Figures 5.14, 5.18 and 5.22,the correlations of dynamic cone penetration index (DCPI) and relative density (D_r) of sand with different silt content (1%, 4%, and 8%) were illustrated by Figures 5.32, 5.33, and 5.34, respectively. The data in these figures indicate that an increase in the relative density resulted in a decrease in the DCPI for different silt content. From the data in these figures, the best correlations between the dynamic cone penetration index (DCPI) and the relative density (D_r) are as follows:

For 1% silt content: $D_r = 230.55/(DCPI)^{0.42}$ $(R^2 = 0.98)$ (5.4)

For 4% silt content: $D_r = 231/(DCPI)^{0.42}$ $(R^2 = 0.98)$ (5.5)

For 8% silt content: $D_r = 214/(DCPI)^{0.40}$ $(R^2 = 0.98)$ (5.6)

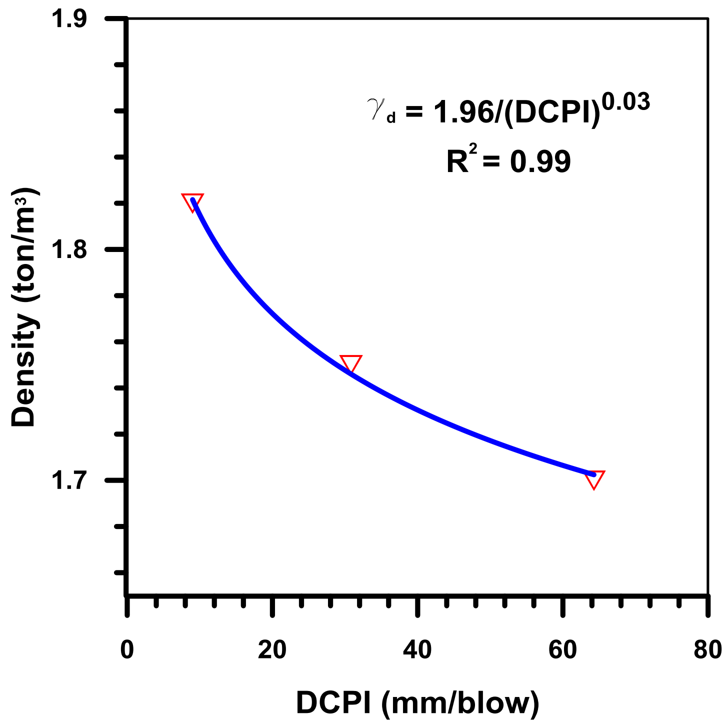


Figure 5-29: Correlation between DCPI and dry density of sand with 1% silt content.

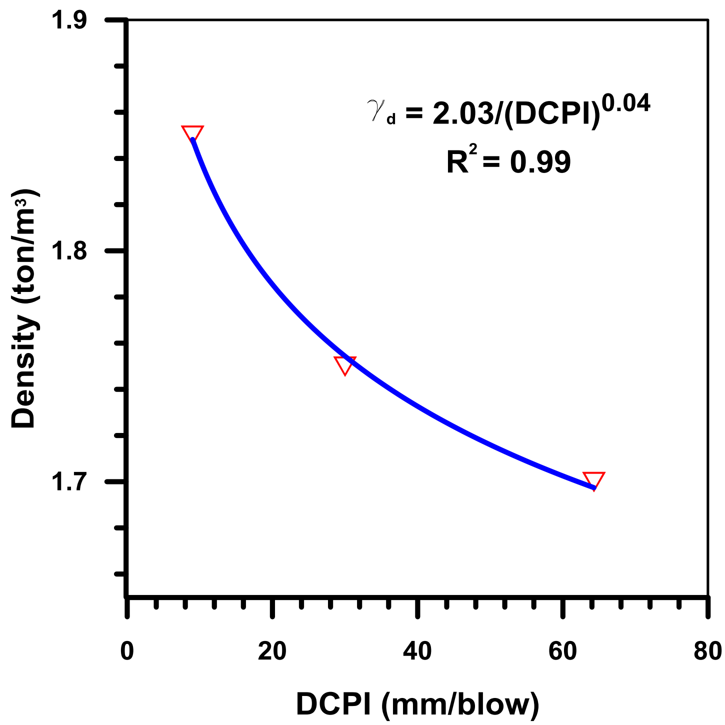


Figure 5-30: Correlation between DCPI and dry density of sand with 4% silt content.

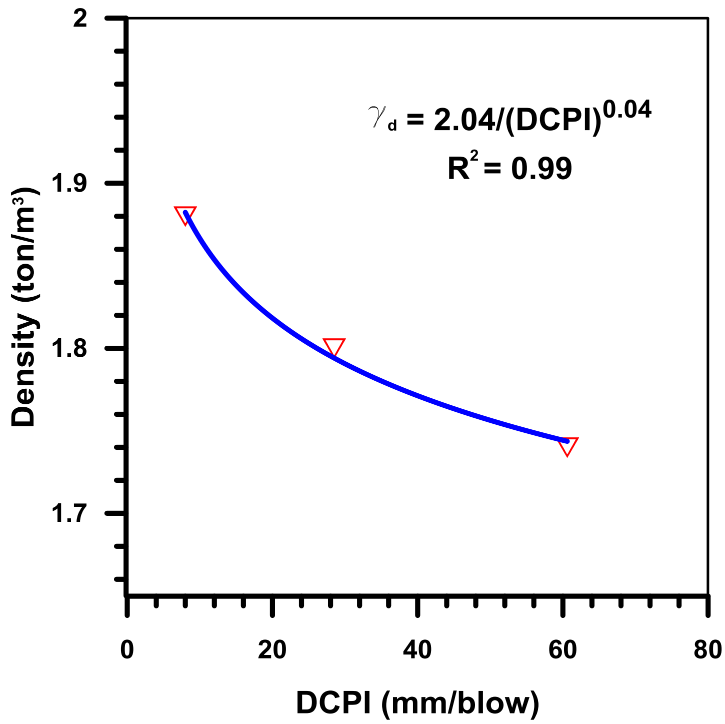


Figure 5-31: Correlation between DCPI and dry density of sand with 8% silt content.

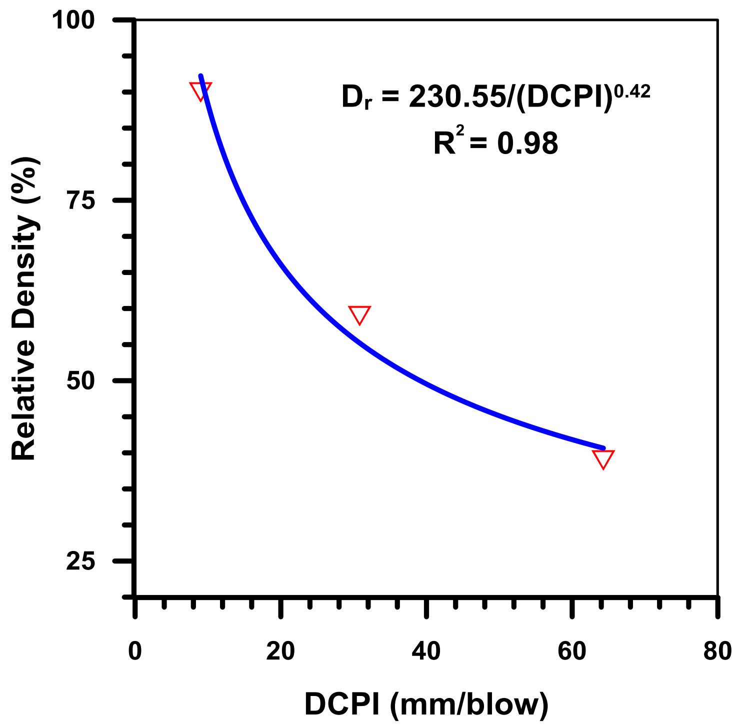


Figure 5-32: Correlation between DCPI and D_r of sand with 1% silt content.

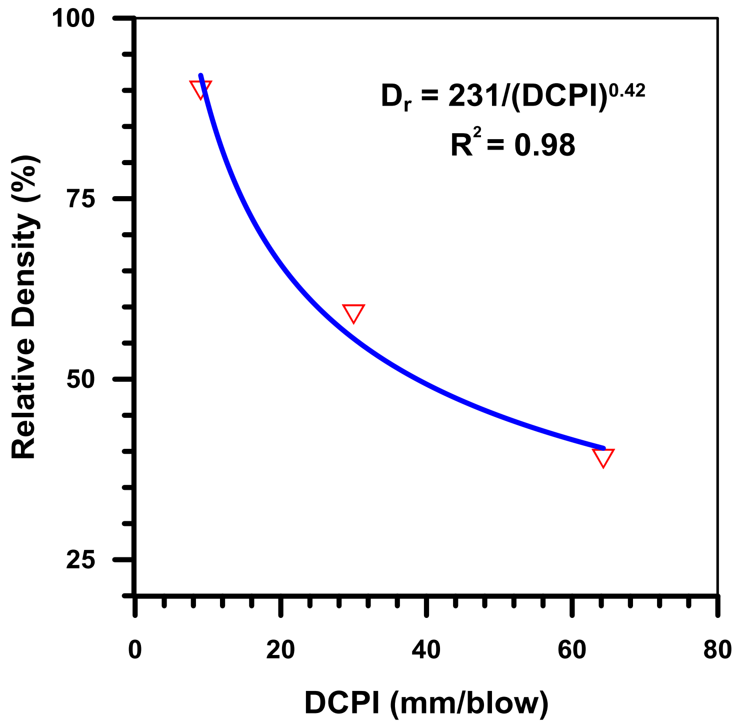


Figure 5-33: Correlation between DCPI and D_r of sand with 4% silt content.

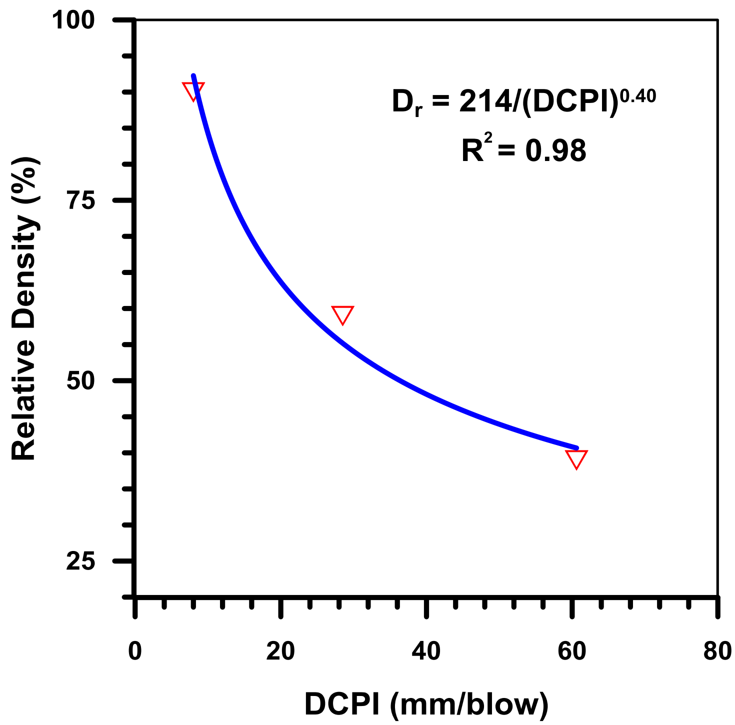


Figure 5-34: Correlation between DCPI and D_r of sand with 8% silt content.

DCPI vs Void Ratio

Based on the data developed in table 5.3 and Figures 5.14, 5.18 and 5.22, the correlations of dynamic cone penetration index (DCPI) and void ratio of sand with different silt content (1%, 4%, and 8% ± 0.2) were illustrated in Figures 5.35, 5.36, and 5.37, respectively. Figure 5.38 summarizes the relationships between the void ratio and DCPI for different silt content. Here, it was observed that an increase in the percentage of silt content from 1 to 8% resulted in a decrease in the void ratio which leads to decrease the DCPI. This is attributed to the fact that silt particles fit into the voids between larger sand particles, so the void ratio of sand-silt mixtures decreased with increased in silt content. These figures suggested good correlations between the dynamic cone penetration index (DCPI) and the void ratio (e) that presented on the following equations:

For 1% silt content: $e = 0.463 + 0.002*(DCPI)$ $(R^2 = 0.96)$ (5.7)

For 4% silt content: $e = 0.418 + 0.002*(DCPI)$ (R2 = 0.91) (5.8)

For 8% silt content: $e = 0.384 + 0.002*(DCPI)$ $(R^2 = 0.97)$ (5.9)

DCPI vs Friction Angle

Based on the data developed in table 5.4 and Figures 5.14, 5.18 and 5.22, the correlations of DCPI and friction angle of sand with different silt content (1%, 4% and 8%) were illustrated by Figures 5.39, 5.40 and 5.41. It was also observed that an increase in the friction angle resulted in a decrease in the DCPI. Similar observations were made by Mohammadi et al. (2008). The suggested correlations between the DCPI and the peak friction angle (ϕ_{peak}) could be presented by the following best fitting equations:

For 1% silt content: $\phi_{peak} = 51.58/(DCPI)^{0.032}$ $(R^2 = 0.99)$ (5.10)

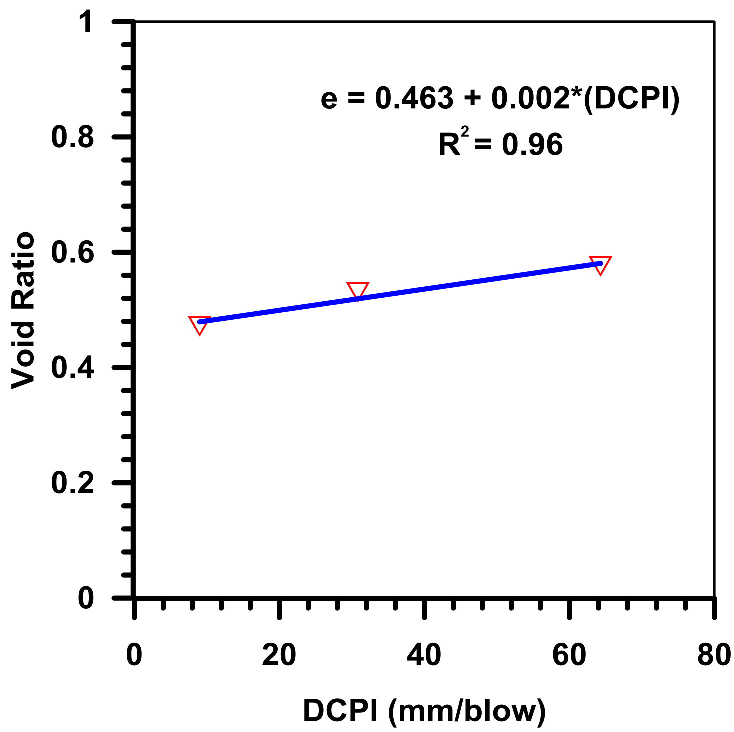


Figure 5-35: Correlation between DCPI and void ratio of sand with 1% silt content.

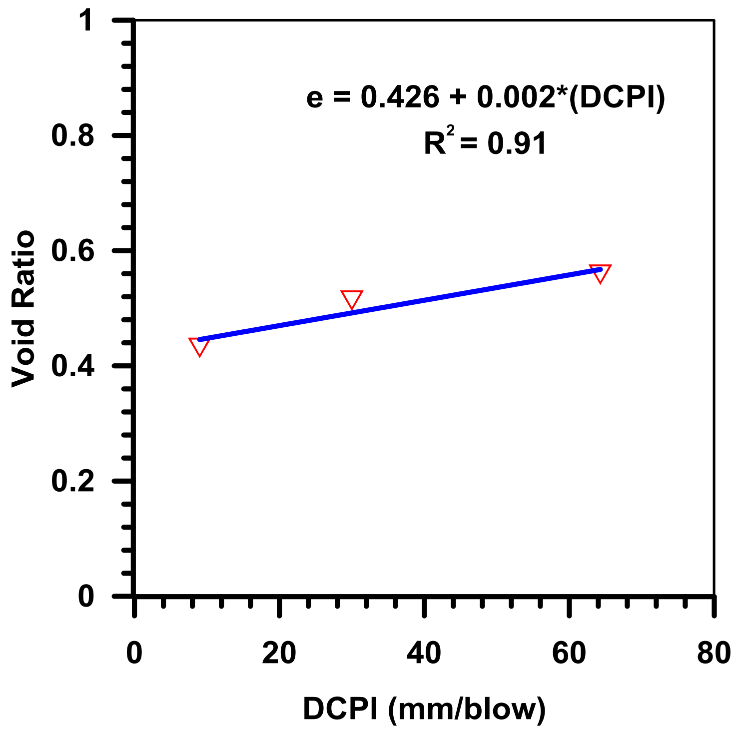


Figure 5-36: Correlation between DCPI and void ratio of sand with 4% silt content.

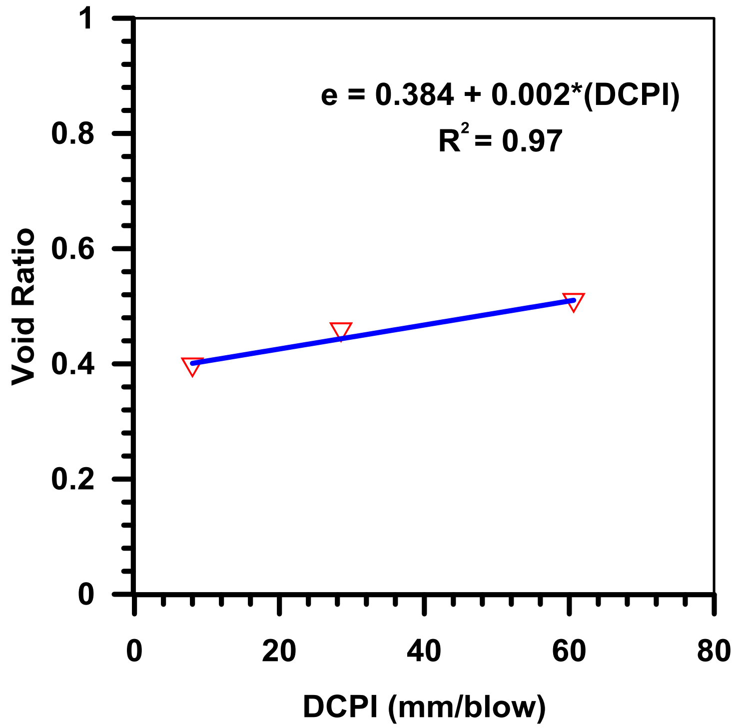


Figure 5-37: Correlation between DCPI and void ratio of sand with 8% silt content.

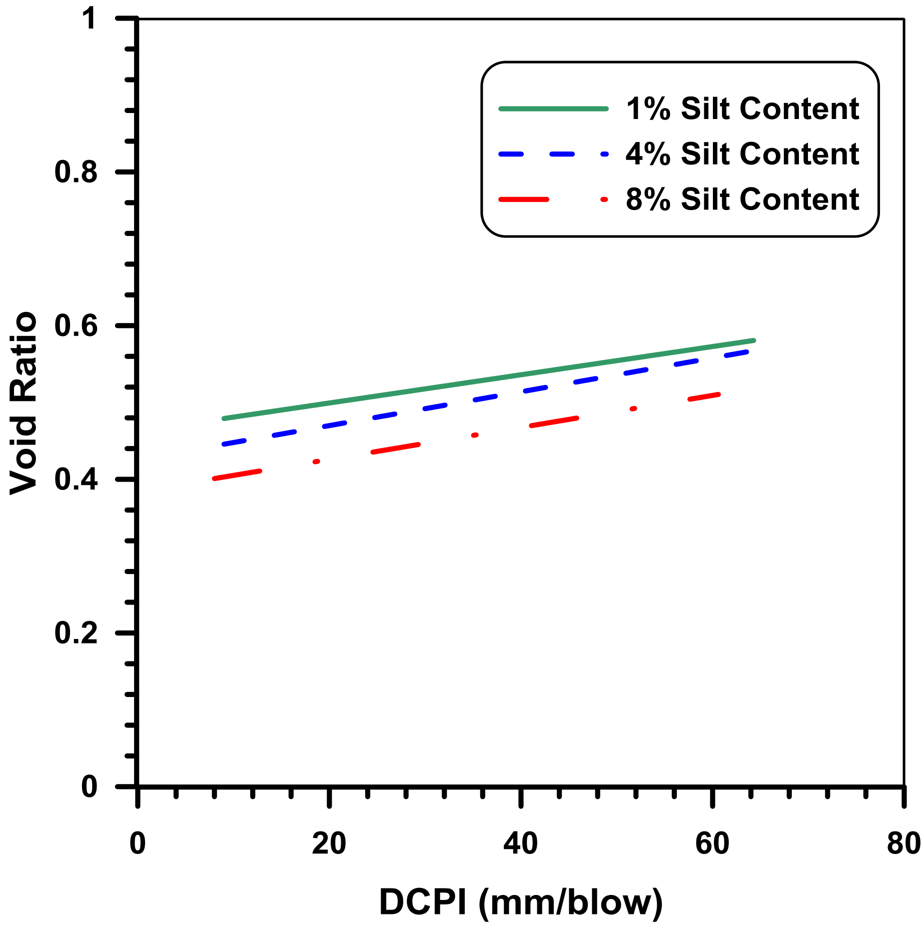


Figure 5-38: Correlation between DCPI and void ratio of sand with different silt content.

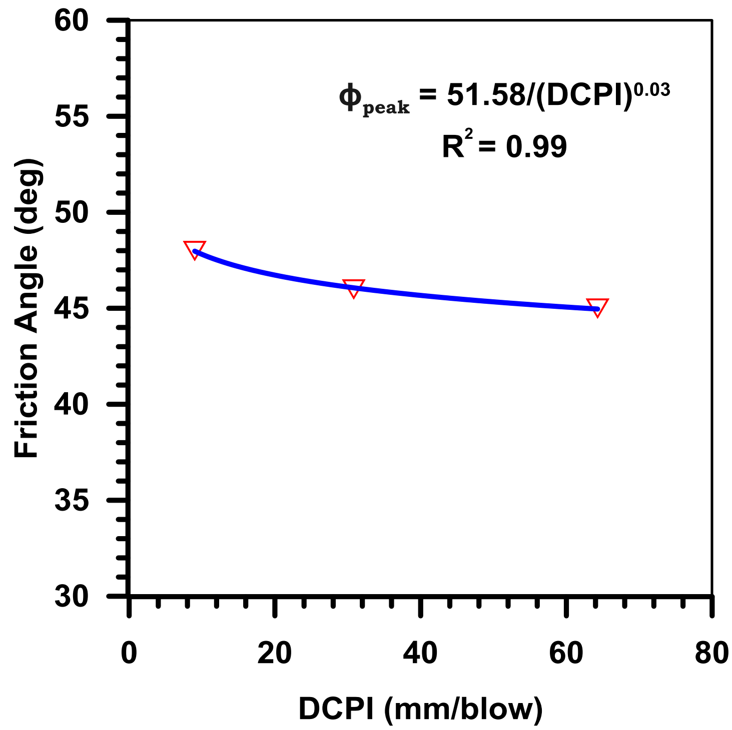


Figure 5-39: Correlation between DCPI and peak friction angle of sand with 1% silt content.

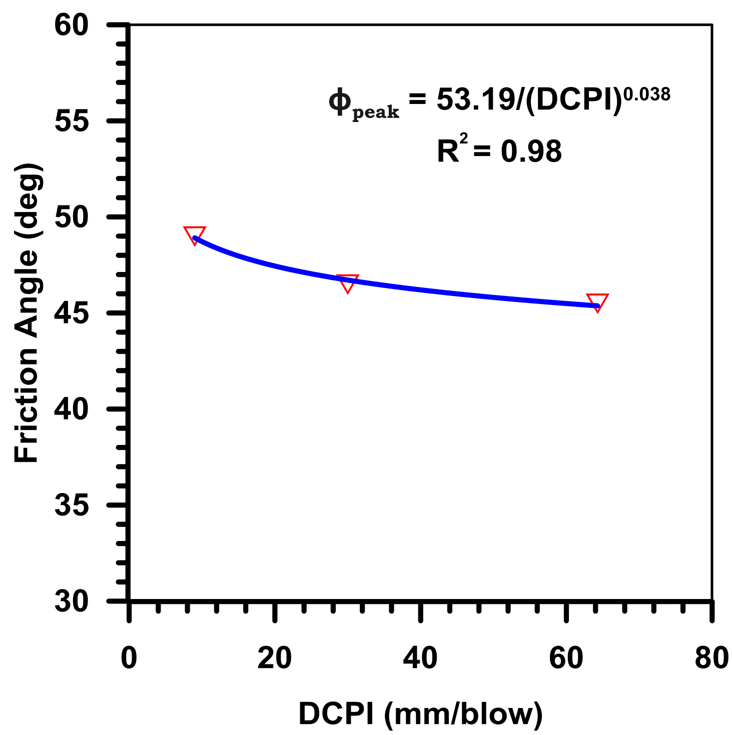


Figure 5-40: Correlation between DCPI and peak friction angle of sand with 4% silt content.

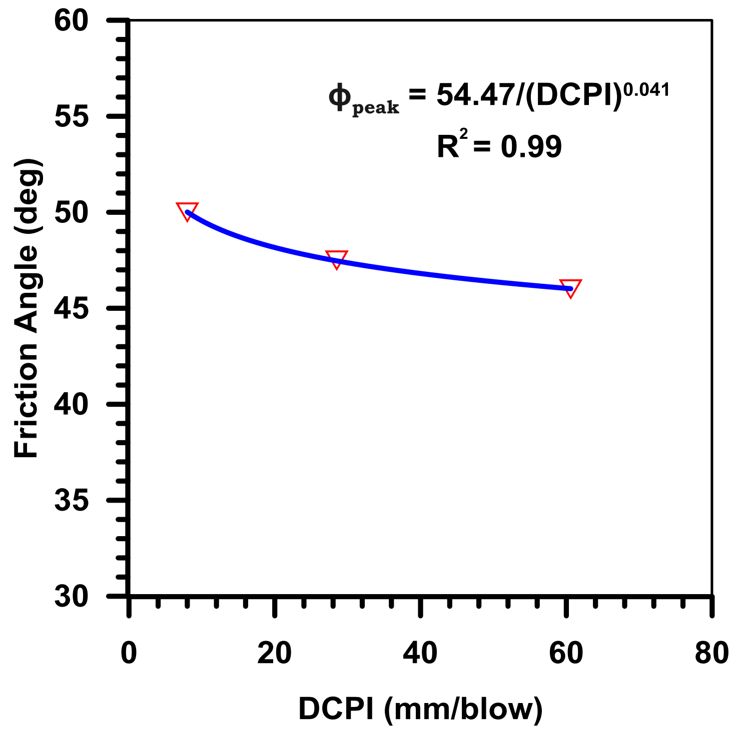


Figure 5-41: Correlation between DCPI and peak friction angle of sand with 8% silt content.

For 4% silt content: $\phi_{peak} = 53.19/(DCPI)^{0.038}$ ($R^2 = 0.98$) (5.11)

For 8% silt content: $\phi_{peak} = 54.47/(DCPI)^{0.041}$ ($R^2 = 0.99$) (5.12)

Relative Density vs Peak Friction Angle

Several correlations between relative density (Dr) and friction angle (ϕ) have been proposed by different researchers including Meyerhof (1959) and Mohammadi et al (2008). These researchers indicated that an increase in the relative density resulted in an increase in the friction angle of sand. In this investigation, the correlations of relative density and peak friction angle of sand with different silt content (1%, 4% and 8%) were illustrated by Figures 5.42, 5.43, and 5.44, respectively. Figure 5.45 summarizes these data. It could be observed that an increase in the relative density from 40 to 90% resulted in an increase in the peak friction angle. These figures indicate that the best correlations between the relative density (D_r) and the peak friction angle (ϕ_{peak}) could be presented in the following equations:

For 1% silt content: $\phi_{peak} = 42.5 + 0.06*(D_r)$ ($R^2= 0.99$) (5.13)

For 4% silt content: $\phi_{peak} = 42.5 + 0.07*(D_r)$ ($R^2= 0.98$) (5.14)

For 8% silt content: $\phi_{peak} = 42.75 + 0.08*(D_r)$ ($R^2= 0.99$) (5.15)

Tables 5.5, 5.6 and 5.7 summarized the developed equations for different silt content (1%, 4% and 8%) between DCPI and different parameters (density, relative density, void ratio and peak friction angle). The determination coefficient between DCPI and other parameters were mostly greater than 0.90.

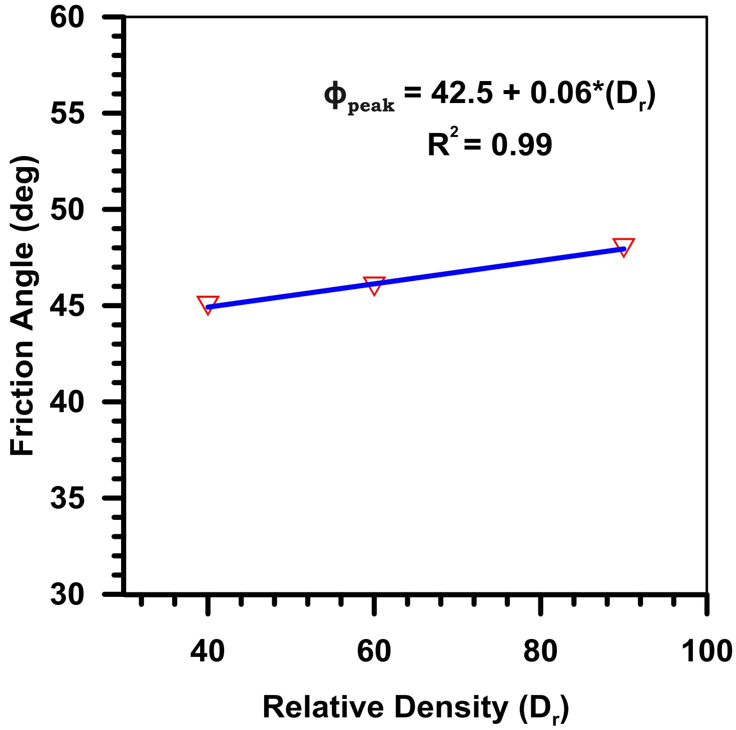


Figure 5-42: Correlation between relative density and peak friction angle of sand with 1% silt content.

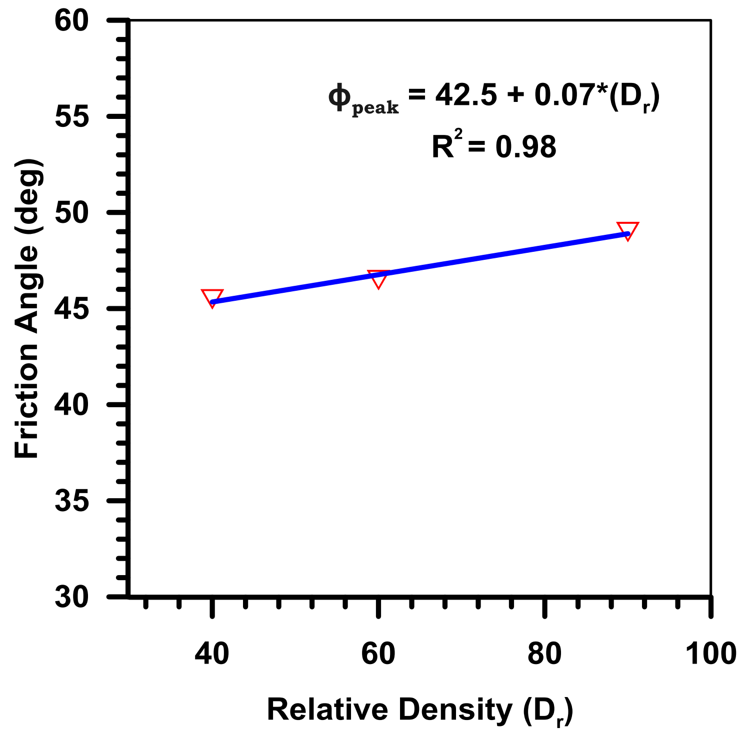


Figure 5-43: Correlation between relative density and peak friction angle of sand with 4% silt content.

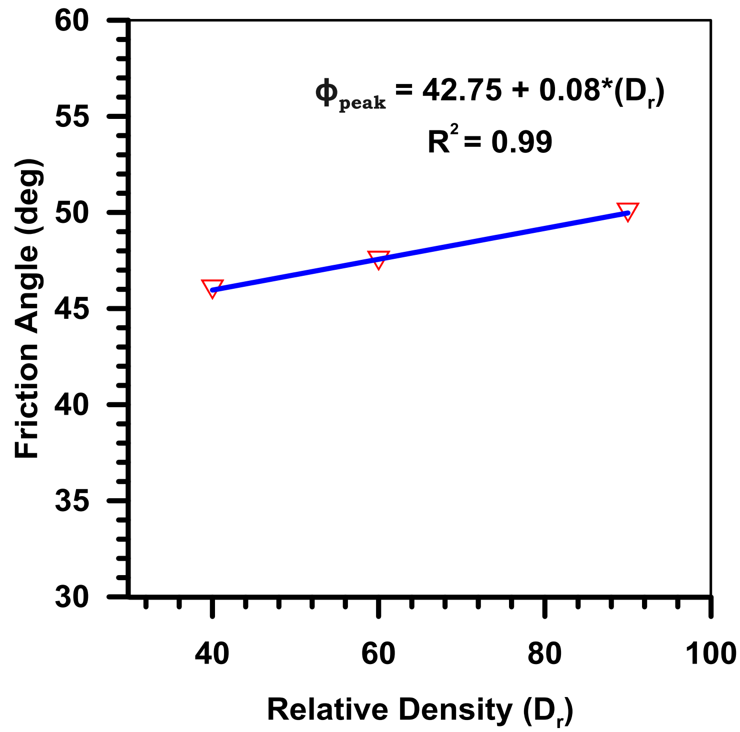


Figure 5-44: Correlation between relative density and peak friction angle of sand with 8% silt content.

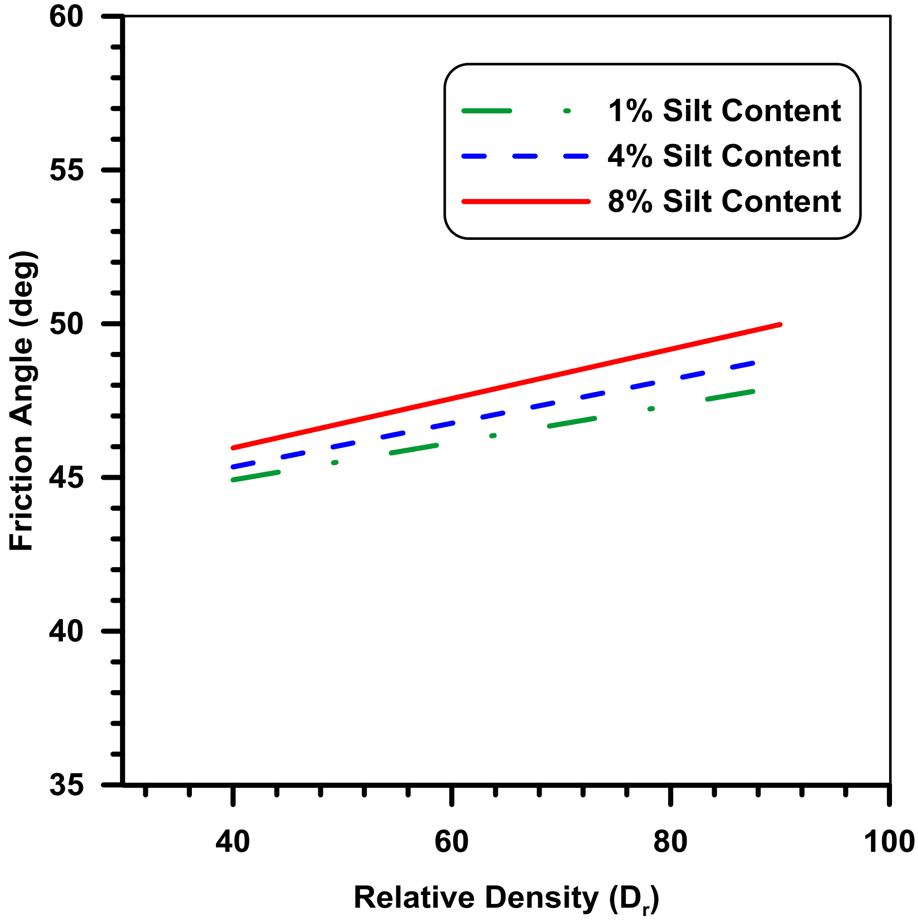


Figure 5-45: Correlation between relative density and peak friction angle of sand with different silt content.

Table 5-5: Summary of developed equations for 1% silt content

Parameter	Equations	Determination coefficient (R^2)
γ_d - DCPI	$\gamma_d = 1.96/(DCPI)^{0.03}$	0.99
D_r - DCPI	$D_r = 230.55/(DCPI)^{0.42}$	0.98
e - DCPI	e = 0.463 + 0.002*(DCPI)	0.96
ϕ_{peak} - DCPI	$\phi_{peak} = 51.58/(DCPI)^{0.031}$	0.99
ϕ_{peak} - D_r	$\phi_{peak} = 42.5 + 0.06*(D_r)$	0.99

Table 5-6: Summary of developed equations for 4% silt content

Parameter	Equations	Determination coefficient (R^2)
γ_d - DCPI	$\gamma_d = 2.03/(DCPI)^{0.04}$	0.99
D_r - DCPI	$D_r = 231/(DCPI)^{0.42}$	0.98
e - DCPI	e = 0.426 + 0.002*(DCPI)	0.91
ϕ_{peak} - DCPI	$\phi_{peak} = 53.19/(DCPI)^{0.038}$	0.98
ϕ_{peak} - D_r	$\phi_{peak} = 42.5 + 0.07*(D_r)$	0.98

Table 5-7: Summary of developed equations for 8% silt content

Parameter	Equations	Determination coefficient (R^2)
γ_d - DCPI	$\gamma_d = 2.04/(DCPI)^{0.04}$	0.96
D_r - DCPI	$D_r = 214/(DCPI)^{0.40}$	0.98
e - DCPI	$e = 0.384 + 0.002*(DCPI)$	0.97
ϕ_{peak} - DCPI	$\phi_{peak} = 54.47/(DCPI)^{0.041}$	0.99
ϕ_{peak} - D_r	$\phi_{peak} = 42.75 + 0.08*(D_r)$	0.99

5.2 Field Data

During the past two decades, there has been a dramatic construction revolution in the cities of the Arabian Gulf with the advent of oil in the Gulf region, which led to the proliferation of buildings and infrastructure over vast areas, that has forced engineers to search for ways and devices to explore and evaluate the properties of the soil and natural factors affecting them. In this section, the potential use of dynamic cone penetration test was studied in two projects.

5.2.1 Al-Jubail Site Results

The data provided by Aiban (2012) includes total of 45 DCP tests performed at randomly selected locations, distributed over the five phases of a large housing project area in Jubail. These DCP testings were carried out to the depth of the backfill for each phase. The backfill consisted of poorly graded dune sand and the backfilling thickness was variable depending on the original ground topology. The DCP tests were conducted in accordance with ASTM D 6951, using 8 kg hammer. The variation of dry soil density with depth was determined using a nuclear gauge and the results were provided in Tables 5.8 to 5.11.

Materials Properties

The backfill material consisted mainly of poorly graded dune sand. In some places, especially on access roads, the top 200 to 300 mm consisted of graded base material. Some oversize gravel was encountered in the access roadways.

Table 5-8: Total and dry soil densities and moisture contents measured using nuclear gauge for Al-Jubail Project in a point near the site office (Aiban, 2012)

Depth (mm)	Moisture content (%)	Total unit weight(gram/cm²)	Dry unit weight(gram/cm²)
101.6	3.60	1.77	1.70
203.2	3.50	1.87	1.81
304.8	3.10	1.96	1.90
406.4	2.40	1.76	1.72
508	2.60	1.80	1.76
609.6	2.90	1.85	1.80
711.2	3.60	1.59	1.53
812.8	3.30	1.67	1.62
914.4	3.10	1.72	1.67
1016	4.20	1.54	1.47
1117.6	4.90	1.63	1.55
1219.2	4.40	1.73	1.65
1320.8	6.00	1.56	1.47
1422.4	5.50	1.64	1.56
1524	6.30	1.72	1.61
1625.6	8.40	1.23	1.28
1727.2	7.80	1.63	1.43
1828.8	8.00	1.70	1.57
Average	4.64	1.69	1.62

Table 5-9: Total and dry soil densities and moisture contents measured using nuclear gauge for Al-Jubail Project in a point in Phase (1) (Aiban, 2012)

Depth (mm)	Moisture content (%)	Total unit weight(gram/cm²)	Dry unit weight (gram/cm²)
101.6	1.7	1.60	1.57
203.2	1.7	1.62	1.67
304.8	1.4	1.75	1.72
406.4	3	1.78	1.72
508	2.7	1.78	1.73
609.6	2.6	1.84	1.79
711.2	4.1	1.64	1.58
812.8	4.1	1.63	1.64
914.4	3.4	1.78	1.72
1016	5.1	1.71	1.63
1117.6	4.6	1.80	1.72
1219.2	4.7	1.85	1.79
1320.8	7.3	1.68	1.65
1422.4	6.1	1.80	1.70
1524	6.1	1.84	1.73
Average	3.91	1.74	1.69

Table 5-10: Total and dry soil densities and moisture contents measured using nuclear gauge for Al-Jubail Project in a point in Phase (2) (Aiban, 2012)

Depth (mm)	Moisture content (%)	Total unit weight(gram/cm²)	Dry unit weight(gram/cm²)
101.6	2.2	1.72	1.65
203.2	2.4	1.79	1.75
304.8	2	1.89	1.85
406.4	4.1	1.72	1.65
508	3.9	1.78	1.71
609.6	4.4	1.82	1.75
711.2	2.7	1.61	1.57
812.8	3.2	1.66	1.61
914.4	2.6	1.71	1.66
1016	4.9	1.51	1.44
1117.6	4.6	1.62	1.55
1219.2	4.5	1.71	1.63
1320.8	5.7	1.54	1.46
1422.4	5.4	1.62	1.54
1524	5.2	1.57	1.65
Average	3.9	1.69	1.63

Table 5-11: Total and dry soil densities and moisture contents measured using nuclear gauge for Al-Jubail Project in a point in Phase (4) (Aiban, 2012)

Depth (mm)	Moisture content (%)	Total unit weight (gram/cm²)	Dry unit weight (gram/cm²)
101.6	4.3	1.76	1.68
203.2	3.7	1.80	1.74
304.8	3.7	1.80	1.74
406.4	6.7	1.63	1.52
508	6	1.67	1.57
609.6	5.9	1.71	1.62
711.2	5.6	1.53	1.45
812.8	5.9	1.64	1.54
914.4	5.7	1.73	1.64
1016	10.5	1.46	1.32
1117.6	10.2	1.59	1.44
1219.2	10.5	1.71	1.55
Average	6.6	1.67	1.57

DCP Testing for Different Phases

The field data curves were shown in Figures 5.46 to 5.59. In general, the following observations could be made:

a) Phase 1: Twenty two (22) DCP tests were performed. The DCP readings were relatively low for most of the locations. It to be noted that the readings for the top 300 mm are always neglected due to the disturbance. For many locations, the DCP counts were reasonable for the layer between 0.4 and 0.7 m. Below 1.5 m, some locations showed good DCP data (more than 15 Blows/100mm), and thus the density values were expected to be acceptable, as shown in Figure 5.46. On the other hand, some DCP tests had values of 5 Blows/100mm, throughout the backfill thickness; which indicates poor compaction. This is the case for 68% of the tested locations within phase 1, as shown in Figures 5.49 to 5.53.

b) Phase 2: Six (6) DCP tests were performed. The DCP data were relatively low for the top 1.5 m, for most of the tested locations. Below 1.5 m, some locations exhibited better DCP data, and thus the density values were expected to be acceptable, as shown in Figure 5.47. On the other hand, some DCP tests displayed values less than 6 to 7 Blows/100 mm throughout the backfill thickness; thereby indicating poor compaction. This is the case for 67% of the tested locations within Phase 2, as shown in Figures 5.54 and 5.55.

c) Phase 3: Six (6) DCP tests were performed. The DCP data were relatively good for most of the tested locations, in general. However, the top portion (between depths of 0.3 to 0.7 m) seems to be better than other phases and the DCP was in excess of 7 Blows/100mm, for most of the tested locations. Below 1.0 m, most of the tested locations exhibited a reduction in the DCP readings, and thus indicating low density values, as shown in Figures 5.56 to 5.57. This is, again, the case for 50% of the tested locations within Phase 3.

d) Phase 4: Six (6) DCP tests were performed. The DCP data were relatively low in some of the tested locations in general. Two DCP tests exhibited low values (5 Blows/100mm). In addition, the bottom portion (below 1.0 m) showed lower blow counts compared to the corresponding portions above, as shown in Figures 5.48 and 5.58.

e) Phase 5: Five (5) DCP tests were performed. The DCP data were relatively good between depths of 0.4 and 0.7 m. However below 0.7 m, the DCP blow count decreased and most of the tested locations had a blow count less than 5 Blow/100mm, as shown in Figure 5.59.

It was clear from the field DCP data, in general, that the compaction was not uniform both in the vertical direction and lateral. This is a clear indication of the absence of a good quality control and good backfilling procedure. Furthermore, the density values seem to be on the low side for most of the tested field locations. This shows clearly how effective the DCP testing in quantifying the field compaction for sandy soils up to depths of 5 m. It is providing a unique tool for quality control of large areas specially when using ground improvement techniques such as dynamic compaction.

5.2.2 Ras Al-Khair Site Results

A total of 29 DCP tests were performed by Aiban (2012) at randomly selected locations, distributed over the Ras Al-khair project area. These DCP testing's were carried out to the

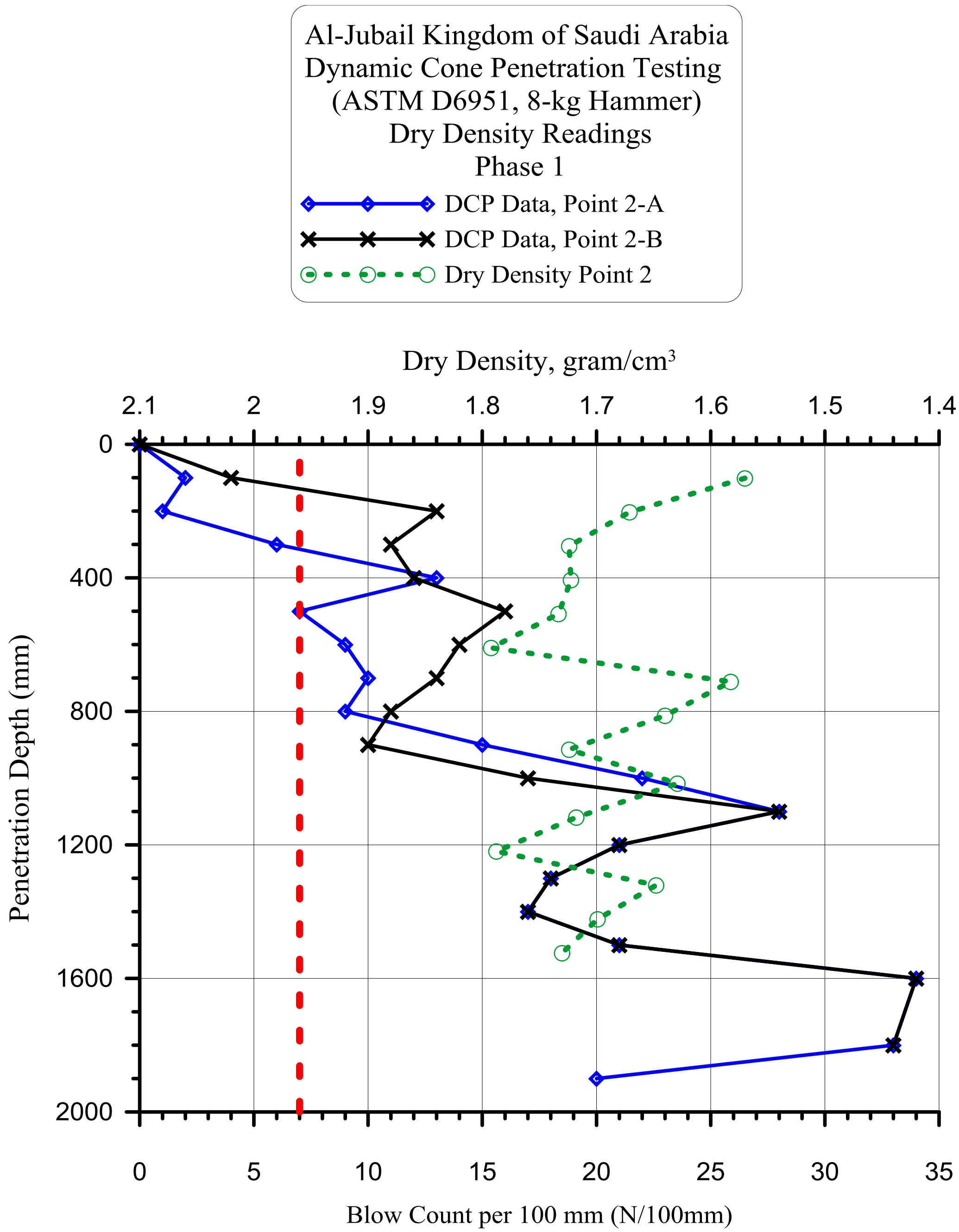


Figure 5-46: Variations of the DCP data and dry density data with depth for a selected location between Phase 1 area and the temporary office building area (Aiban, 2012).

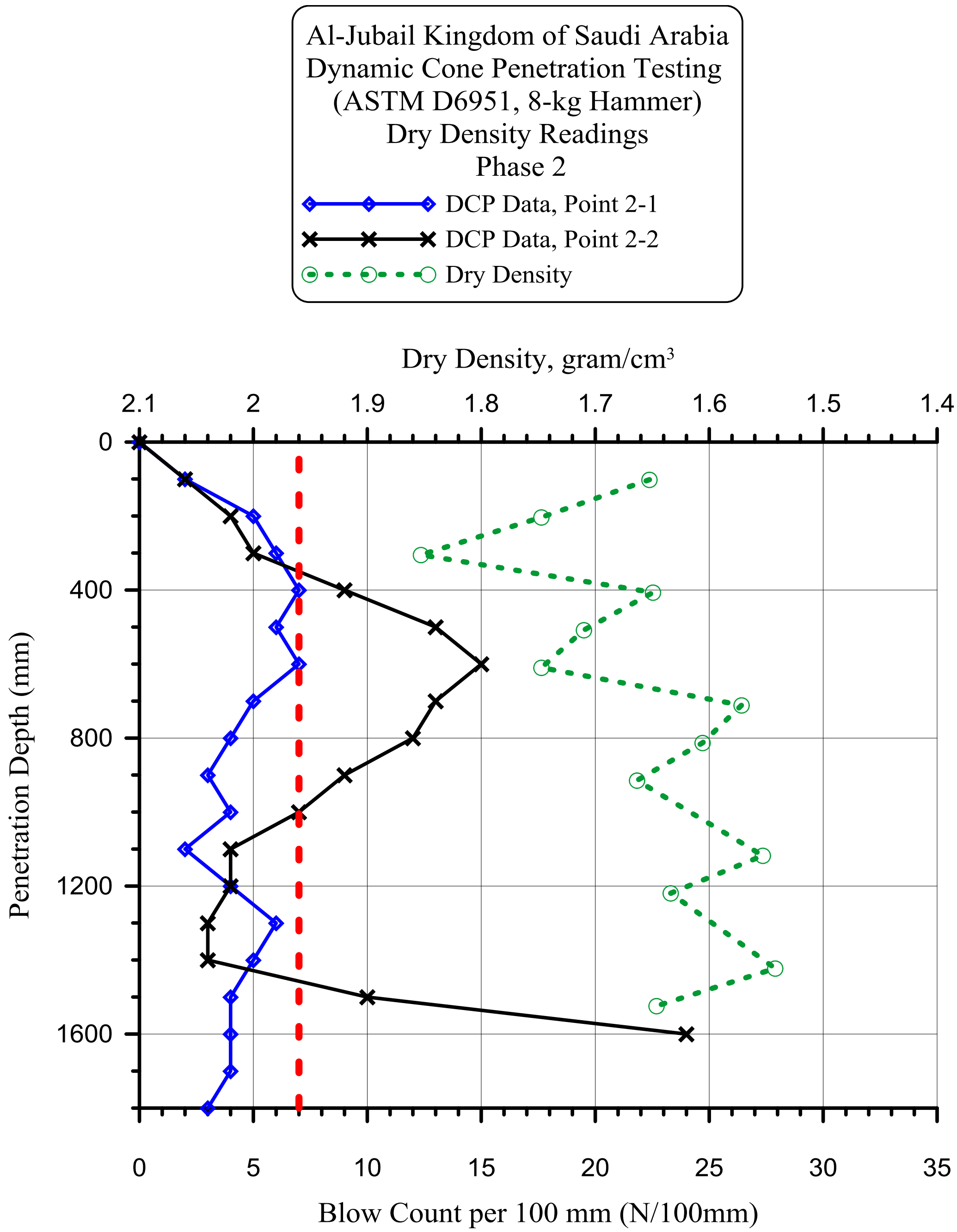


Figure 5-47: Variations of the DCP data and dry density data with depth for a selected location within Phase (2) (Aiban, 2012).

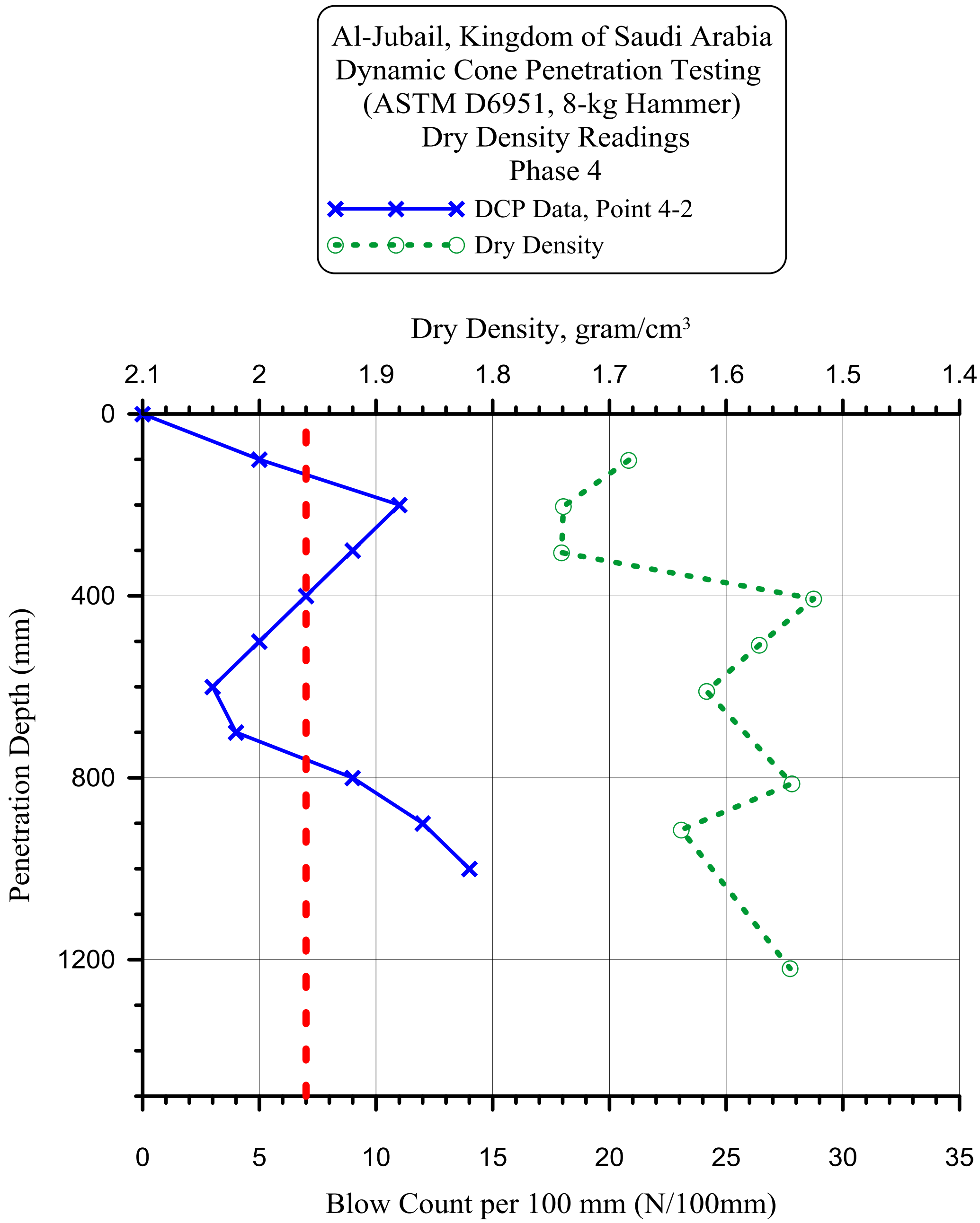


Figure 5-48: Variations of the DCP data and dry density data with depth for a selected location within Phase (4) (Aiban, 2012).

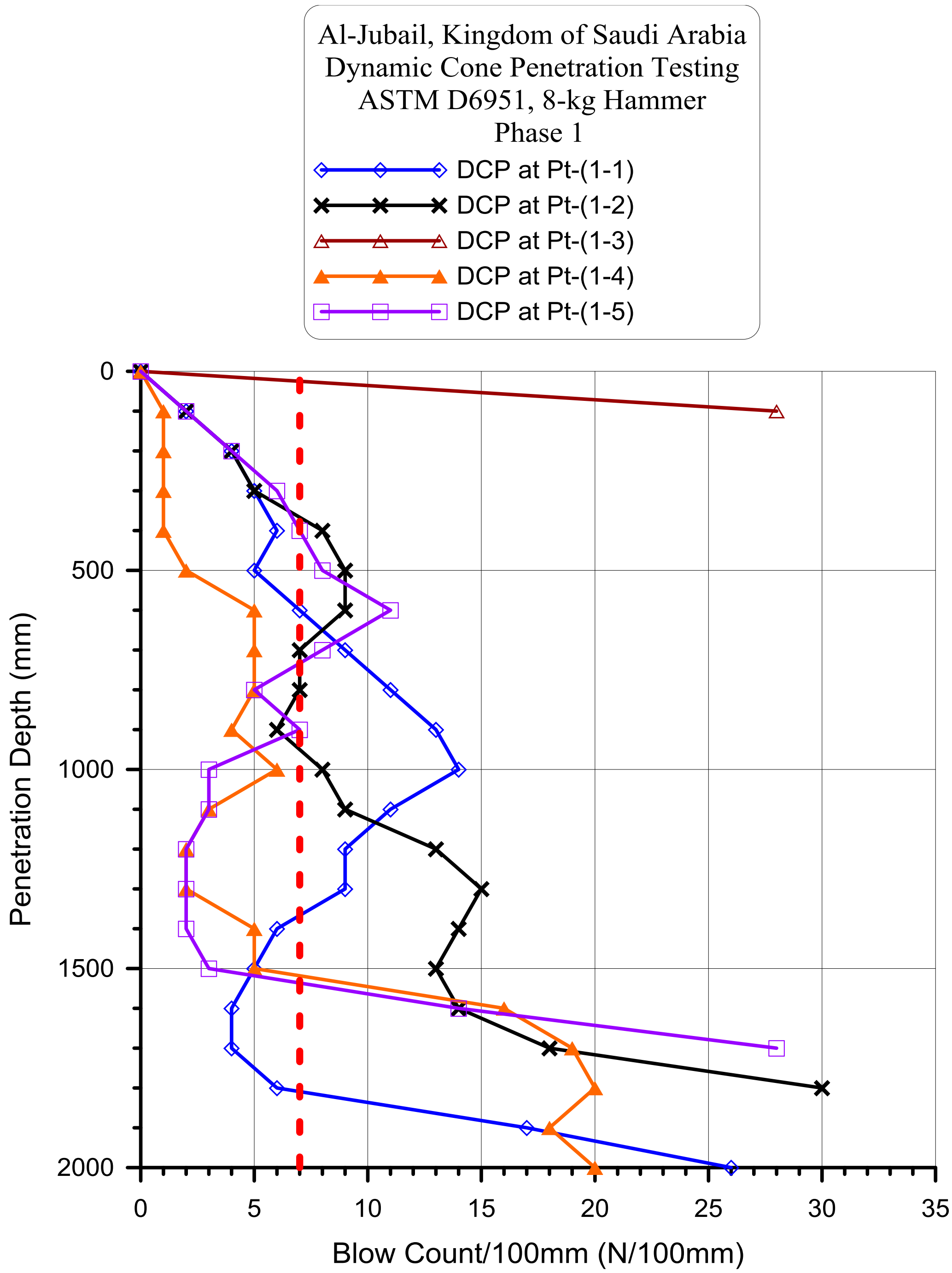


Figure 5-49: Variations of the DCP data with depth for selected locations within Phase (1), Part (a) (Aiban, 2012).

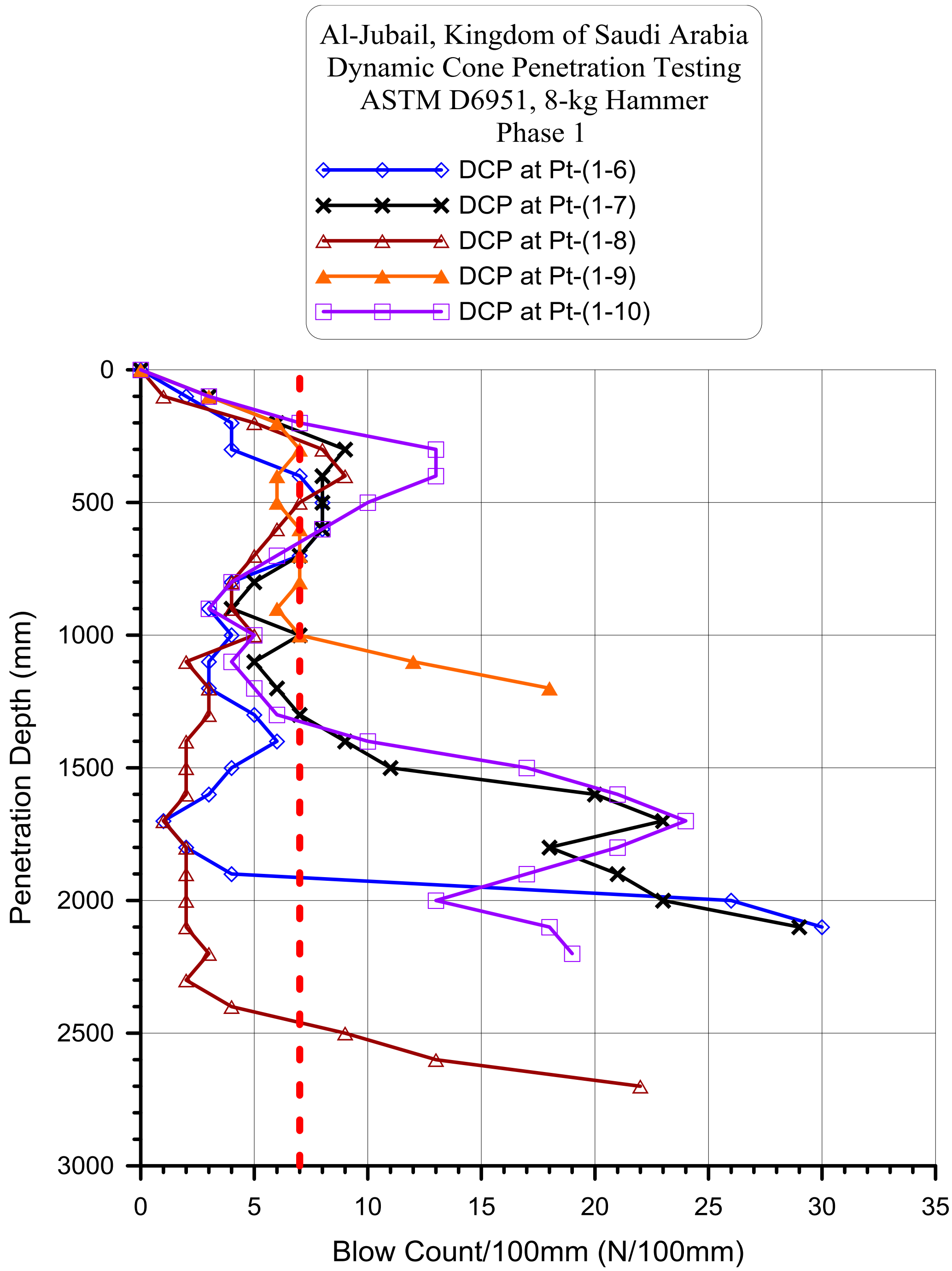


Figure 5-50: Variations of the DCP data with depth for selected locations within Phase (1), Part (b) (Aiban, 2012).

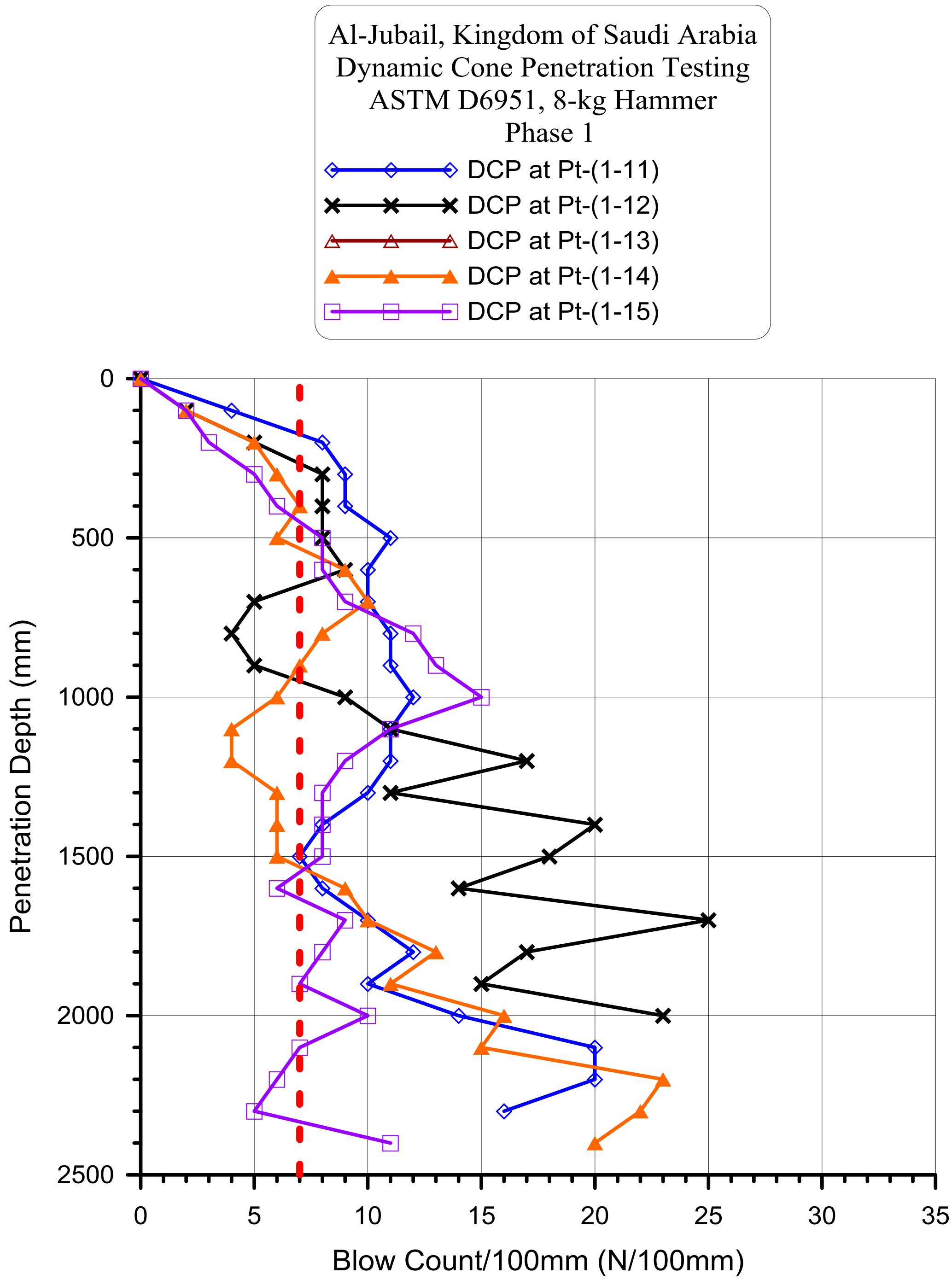


Figure 5-51: Variations of the DCP data with depth for selected locations within Phase (1), Part (c) (Aiban, 2012).

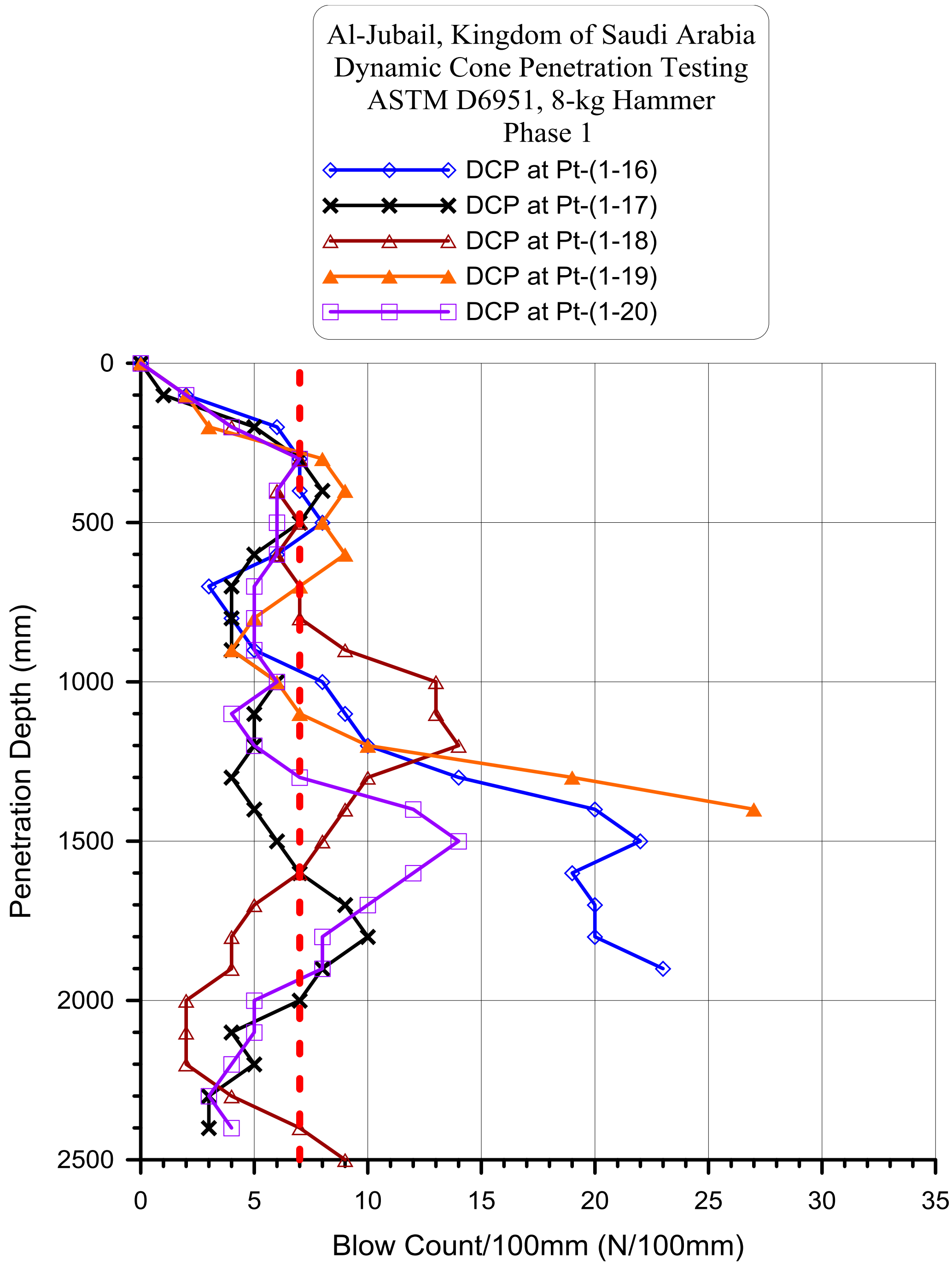


Figure 5-52: Variations of the DCP data with depth for selected locations within Phase (1), Part (d) (Aiban, 2012).

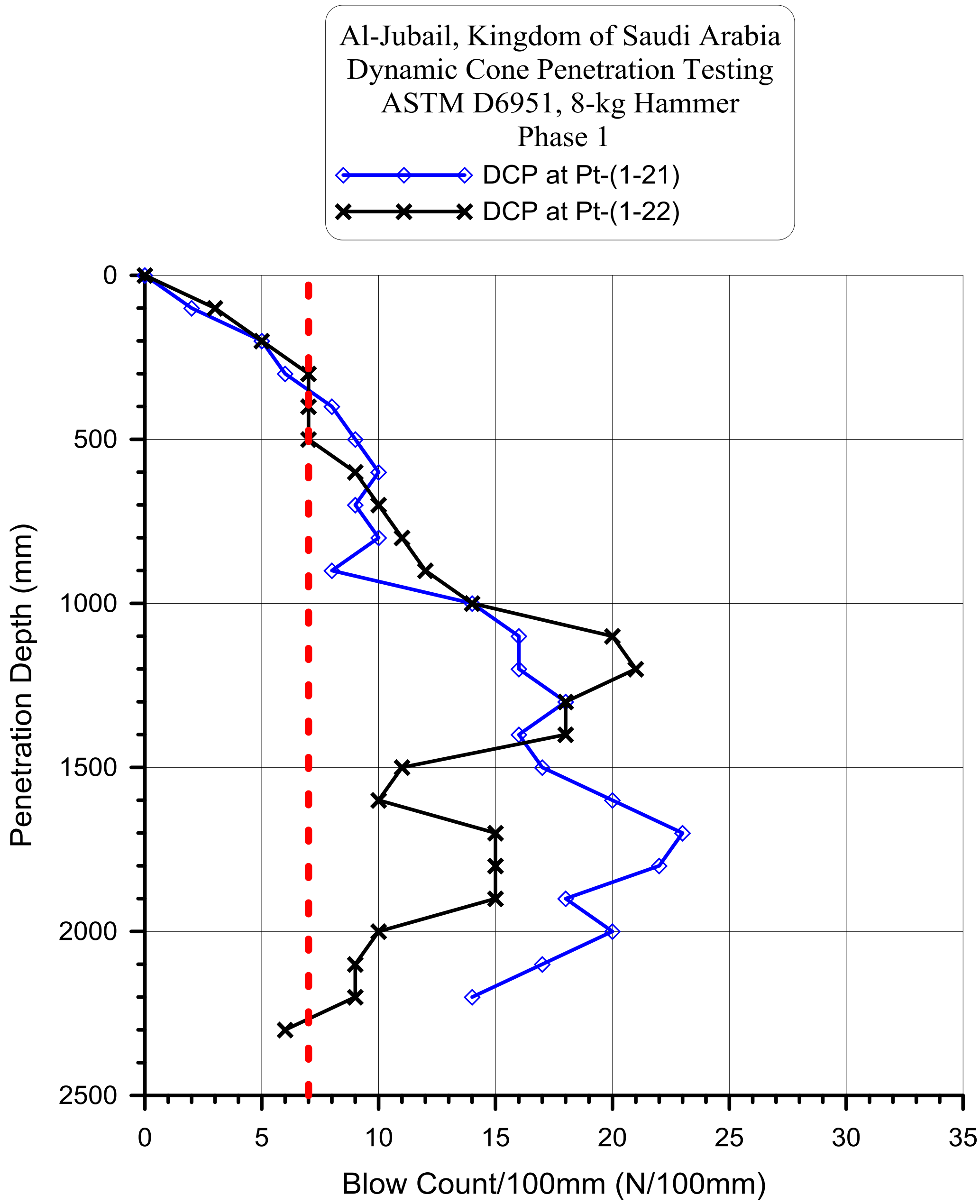


Figure 5-53: Variations of the DCP data with depth for selected locations within Phase (1), Part (e) (Aiban, 2012).

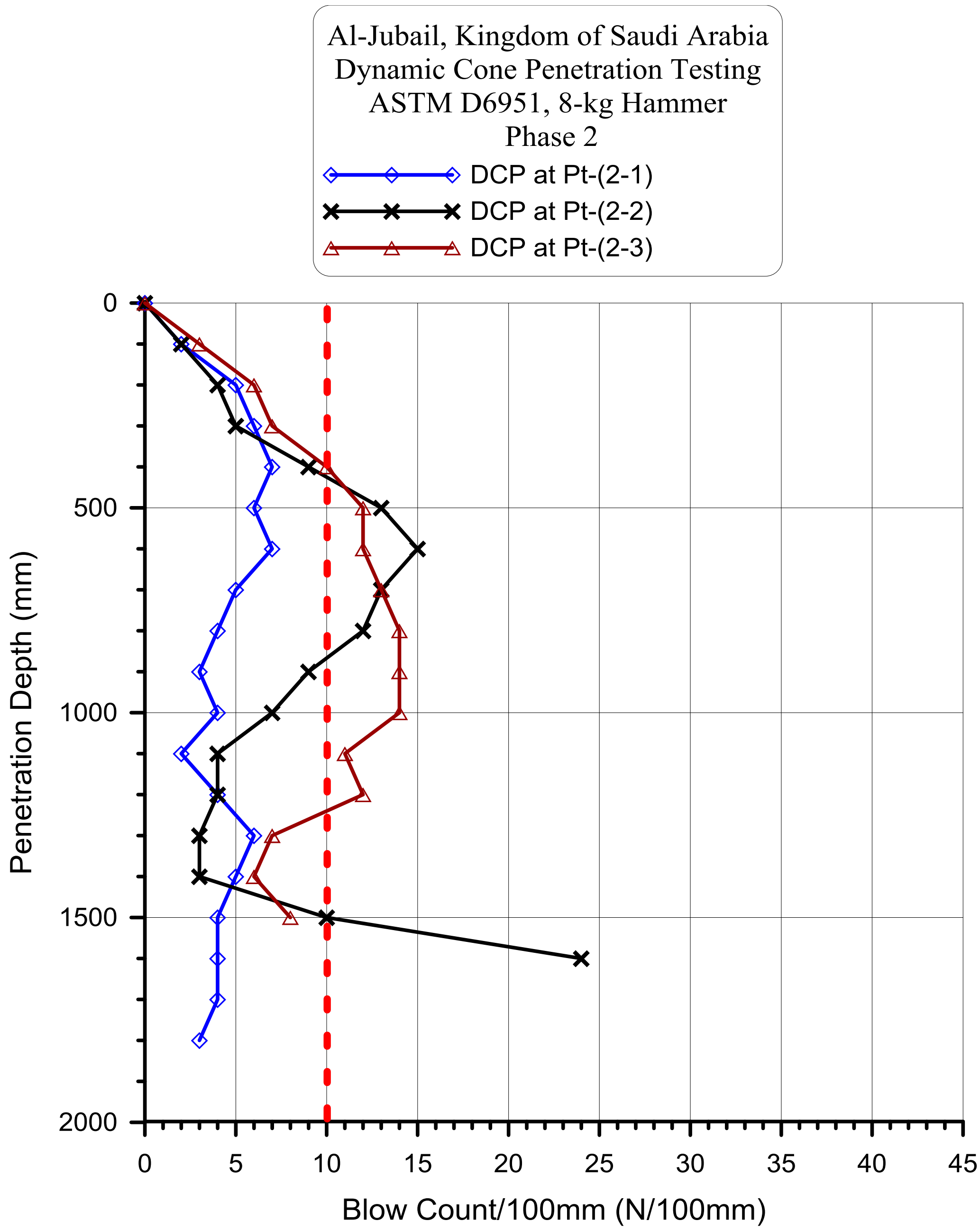


Figure 5-54: Variations of the DCP data with depth for selected locations within Phase (2), Part (a) (Aiban, 2012).

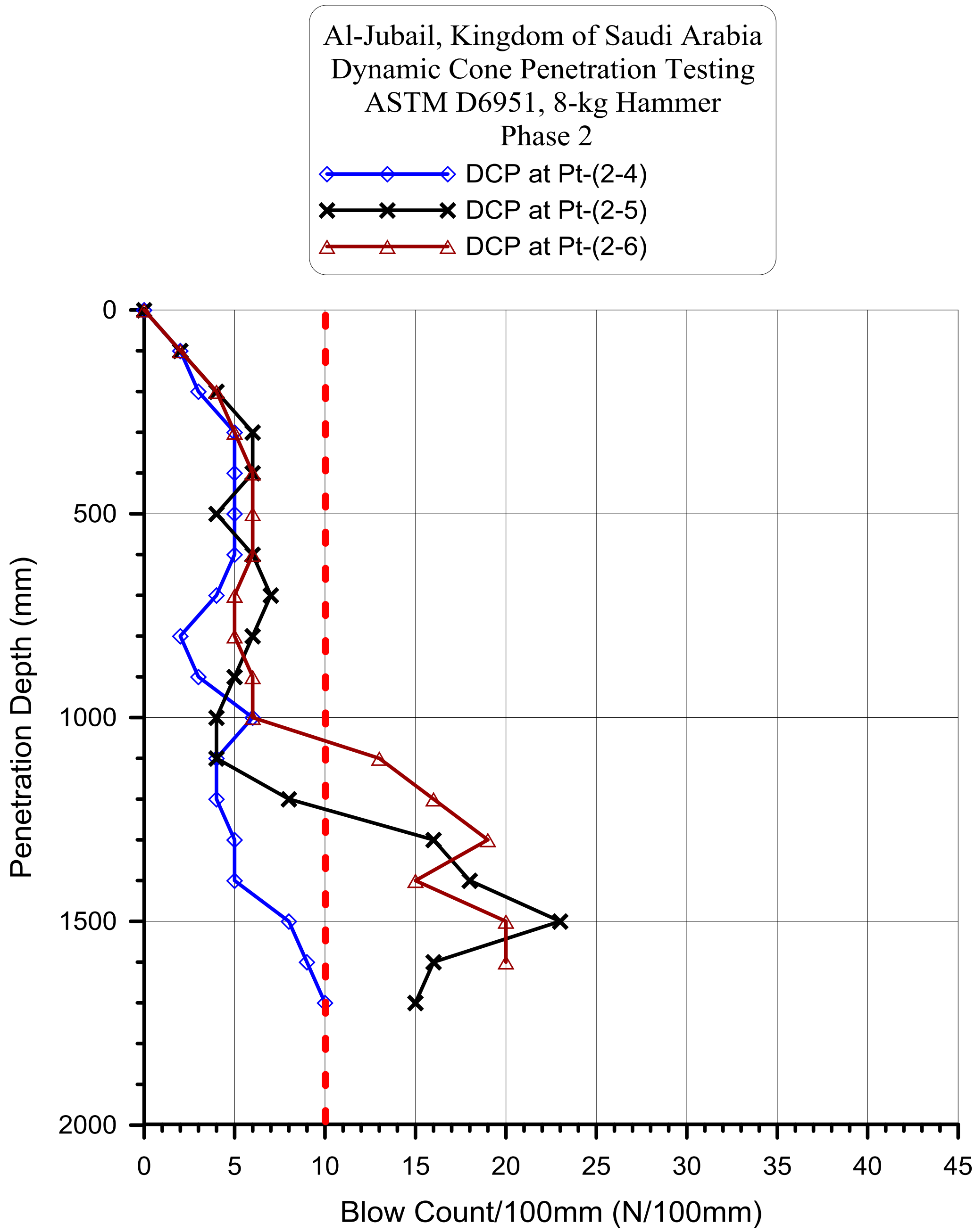


Figure 5-55: Variations of the DCP data with depth for selected locations within Phase (2), Part (b) (Aiban, 2012).

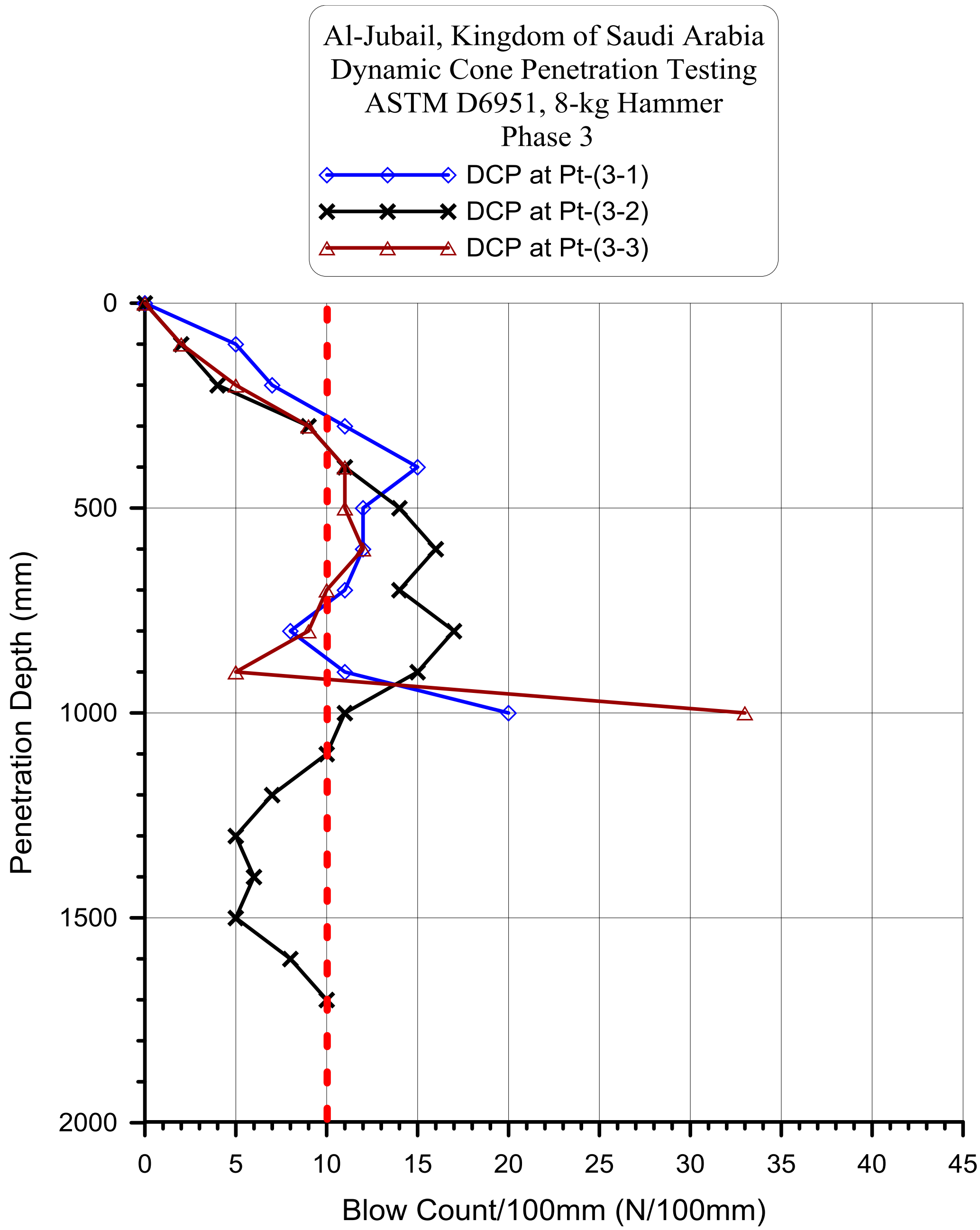


Figure 5-56: Variations of the DCP data with depth for selected locations within Phase (3), Part (a) (Aiban, 2012).

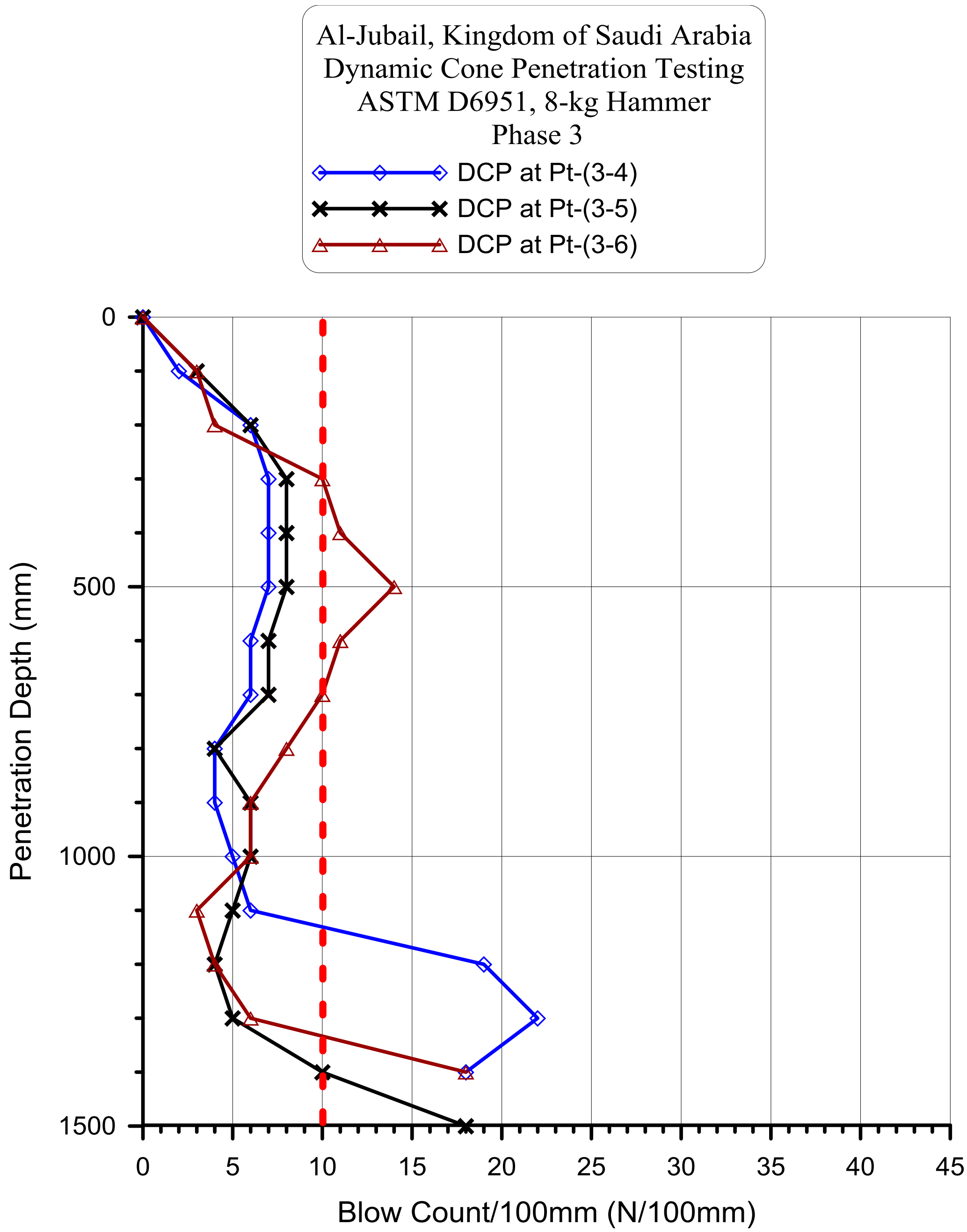


Figure 5-57: Variations of the DCP data with depth for selected locations within Phase (3), Part (b) (Aiban, 2012).

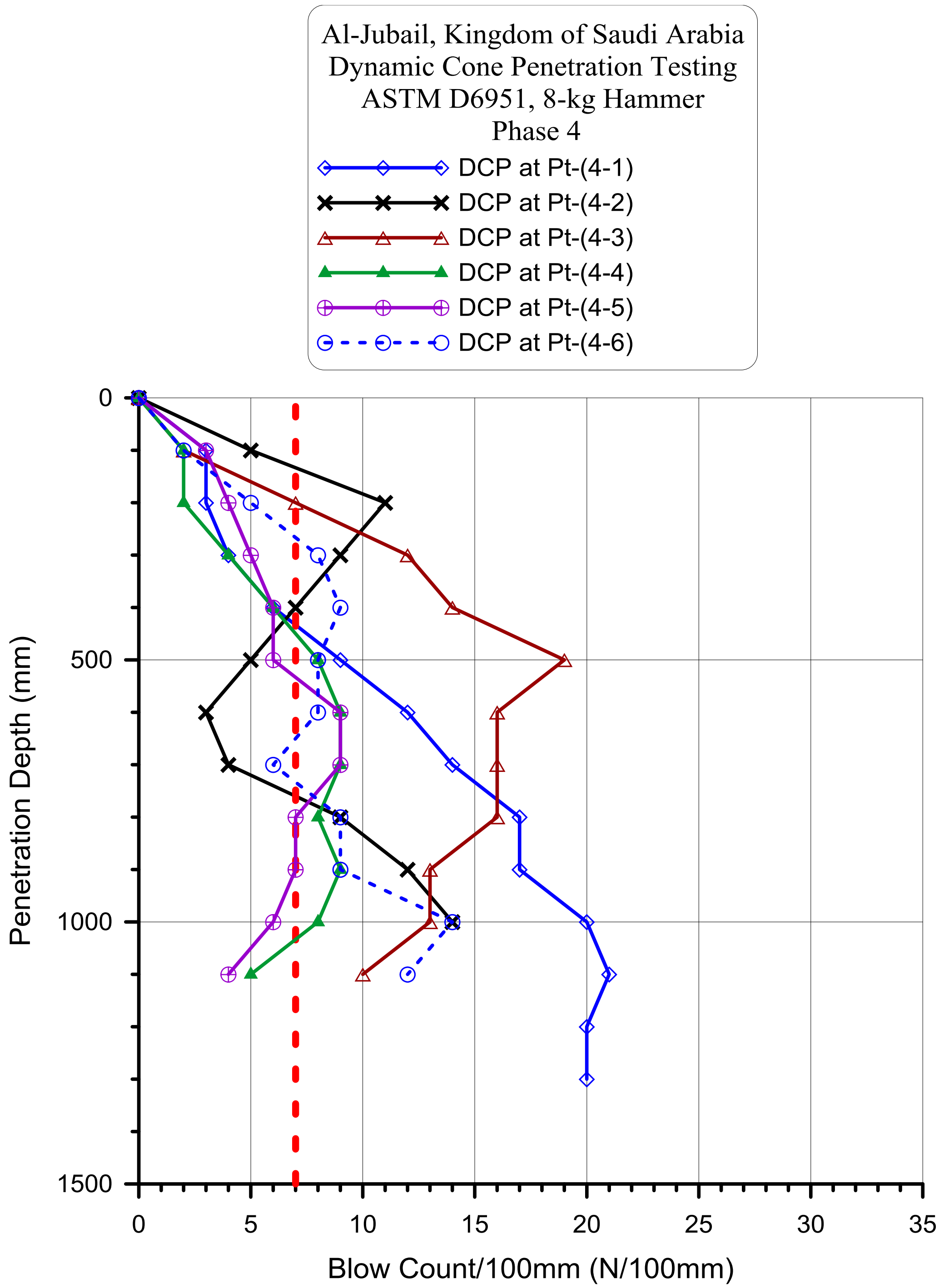


Figure 5-58: Variations of the DCP data with depth for selected locations within Phase (4) (Aiban, 2012).

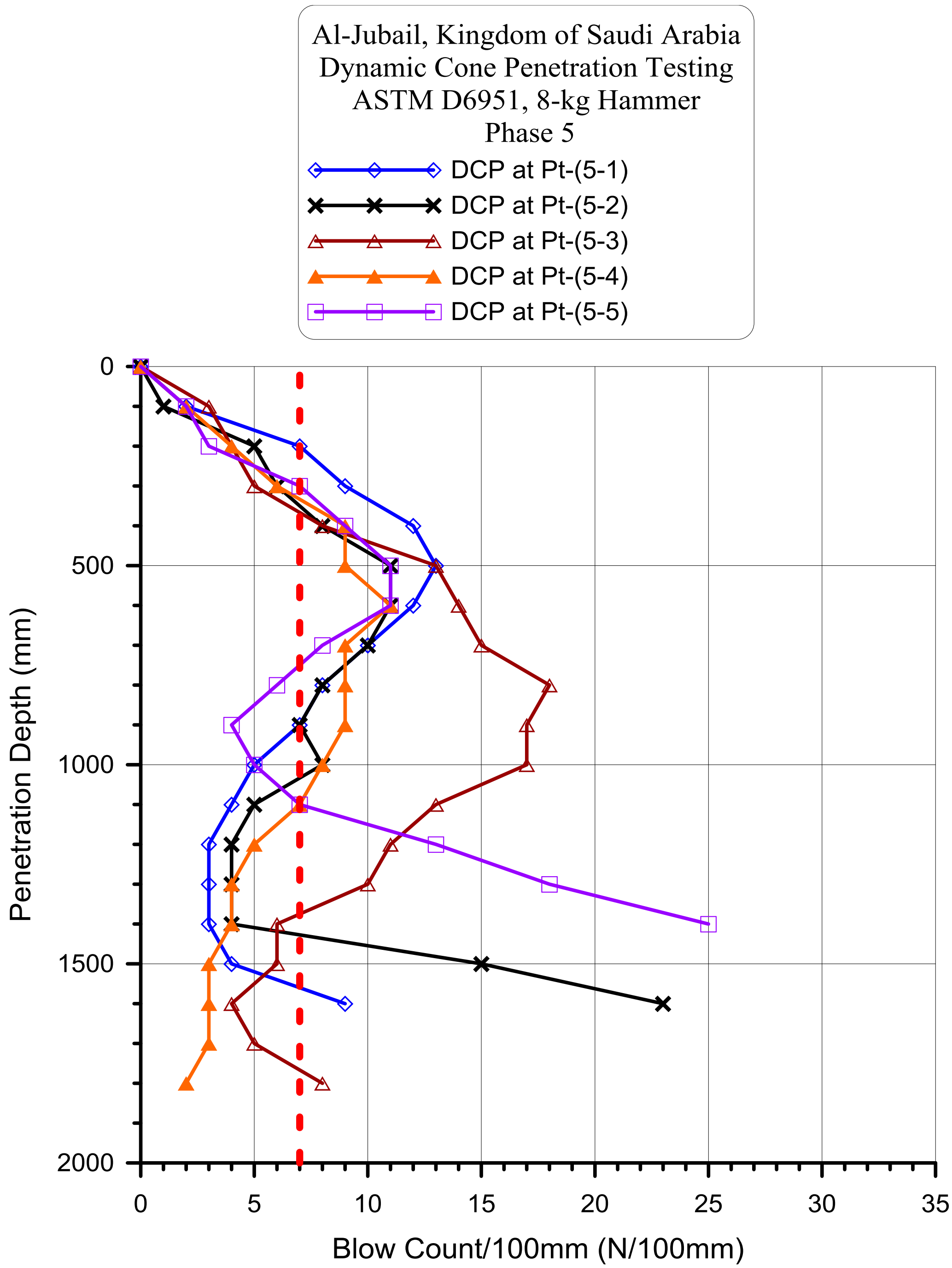


Figure 5-59: Variations of the DCP data with depth for selected locations within Phase (5) (Aiban, 2012).

depth of 5 m or refusal (difficult penetration). Due to the high density of the material and existence of little cementation, refusal was encountered within the top one meter (1 m). Several attempts were made to penetrate at slow penetration rate (high resistance and thus high blow counts) to the targeted 5 m depth; however, such attempts were not all successful. The three DCP rods were lost upon retrieval in such dense layers, as shown in Figures 5.62, 5.64 and 5.65. Only 7 DCP tests penetrated more than 3 m. The DCP tests were conducted in accordance with ASTM D 6951, using 8 kg hammer. The variation of dry soil density with depth was determined using a nuclear gauge and the results were tabulated in Tables 5.13 and 5.14.

Materials Properties

During the excavation of the two pits for the nuclear density testing, soil samples from the different levels that might be showing different layers, were retrieved and tested for grain size distribution. Four samples were tested in accordance with the relevant ASTM D 422 standard. The material was sand and the fines content was less than 12% and thus the soils could be classified as sands with silty fines according to the Unified Soil Classification System. The fines were non-plastic, however, while sampling, it was observed that the silt was providing some light cementation at the top layers. This is a reason for the high penetration resistance.

DCP Testing Different Locations

The collected field data revealed that the soil compactness was very good for all the tested locations for the top 5 m, as shown in Figures 5.60 to 5.67. The density of the sand backfill was reflected by the DCP results that indicates good correlation between the two techniques, dynamic cone penetration test and nuclear gauge. In general, there is consistency in the degree of compaction within the tested locations. The soil seems very densely compacted at the top layer (around 500 mm); a situation resulting from traffic loading during the construction activities. Furthermore, these top layers (within 1000 mm) show little cementation between the sand particles as reflected by producing high penetration resistance in most of the tested locations. Below about 500 to 1000 mm of the thick top layer, the penetration resistance was reduced but still indicating a very dense material.

A DCP blow count larger than 5 for sands was indicating a very dense material. This is confirmed by the nuclear gauge density reading at the two control pits, as shown in Figures 5.60 and 5.61. It was observed that an increase in the dry density resulted in an increase in the number of blows. Such consistency was clear both laterally and with depth. Based on the field investigation it appears that the soil compaction was consistent and shows a sandy material at a very dense state. The top 500 mm was always an exception; it could be loose due to disturbance or very dense due to traffic loading and little cementation from dust.

The field data curves are presented in Figures 5.60 to 5.67. In general, the following observations could be made:

a) The top 500 to1000 mm (excluding the top 200 mm or so) show very high blow count indicting dense materials with little cementation. The readings for the top 200 to 300 mm were always neglected due to the disturbance and/or cementation.
b) Below the 500 to 1000 mm from the surface, the DCP data were relatively high (i.e., in excess of 8 to 10) for most of the locations.

c) It was clear from the DCP data, in general, that the material was dense as a natural material without any compaction other than the traffic and compaction from the construction activities of the different projects. This is a clear indication of the absence of loose layers within the top 5 m of the material.

Table 5-12: Total and dry soil densities and moisture contents measured using nuclear gauge for Ras Al-Khair Project of location 1 (near DCP 1-1, 1-2, 1-3) (Aiban, 2012)

Depth (mm)	Moisture content (%)	Total unit weight (gram/cm^2)	Dry unit weight (gram/cm^2)
101.6	2.1	1.94	1.90
203.2	2.1	1.97	1.93
304.8	2.2	2.05	2.01
406.4	5.1	1.73	1.65
508	4.2	1.82	1.75
609.6	3.5	1.96	1.90
711.2	6.9	1.62	1.52
812.8	5.5	1.78	1.69
914.4	6.1	1.81	1.71
1016	8.5	1.60	1.48
1117.6	7.3	1.75	1.63
1219.2	6.9	1.85	1.73
1320.8	7.5	1.68	1.55
1422.4	7.2	1.76	1.64
1524	7.3	1.70	1.59
1625.6	6.9	1.72	1.61
1727.2	8	1.78	1.65
1828.8	6.3	1.87	1.76
Average	5.8	1.80	1.70

Table 5-13: Total and dry soil densities and moisture contents measured using nuclear gauge for Ras Al-khair Project of location 2 (near DCP 2-1 and 2-2) (Aiban, 2012)

Depth (mm)	Moisture content (%)	Total unit weight (gram/cm²)	Dry unit weight (gram/cm²)
101.6	1.6	1.83	1.80
152.4	1.8	1.88	1.84
203.2	2	1.89	1.85
254	1.7	1.89	1.86
304.8	1.7	1.96	1.93
406.4	4	1.90	1.82
457.2	4.1	1.94	1.87
508	3.6	1.98	1.91
558.8	3.3	1.96	1.90
609.6	3.1	2.02	1.96
711.2	3.7	1.67	1.62
762	3.8	1.77	1.71
812.8	4.1	1.81	1.74
863.6	3.7	1.88	1.81
914.4	4.3	1.98	1.90
1016	6.3	1.72	1.62
1066.8	6.2	1.81	1.70
1117.6	5.4	1.85	1.76
1168.4	5.4	1.90	1.80
1219.2	5.3	1.99	1.89
1320.8	8	1.71	1.58
1371.6	6.4	1.94	1.82
1422.4	6.7	1.98	1.86
1473.2	6.3	2.01	1.89
1524	6.3	2.06	1.94
1625.6	11	1.80	1.63
1676.4	9.7	1.87	1.70
1727.2	9.6	1.91	1.75
1778	9.7	1.92	1.75
1828.8	9.6	1.99	1.81
Average	5.28	1.89	1.80

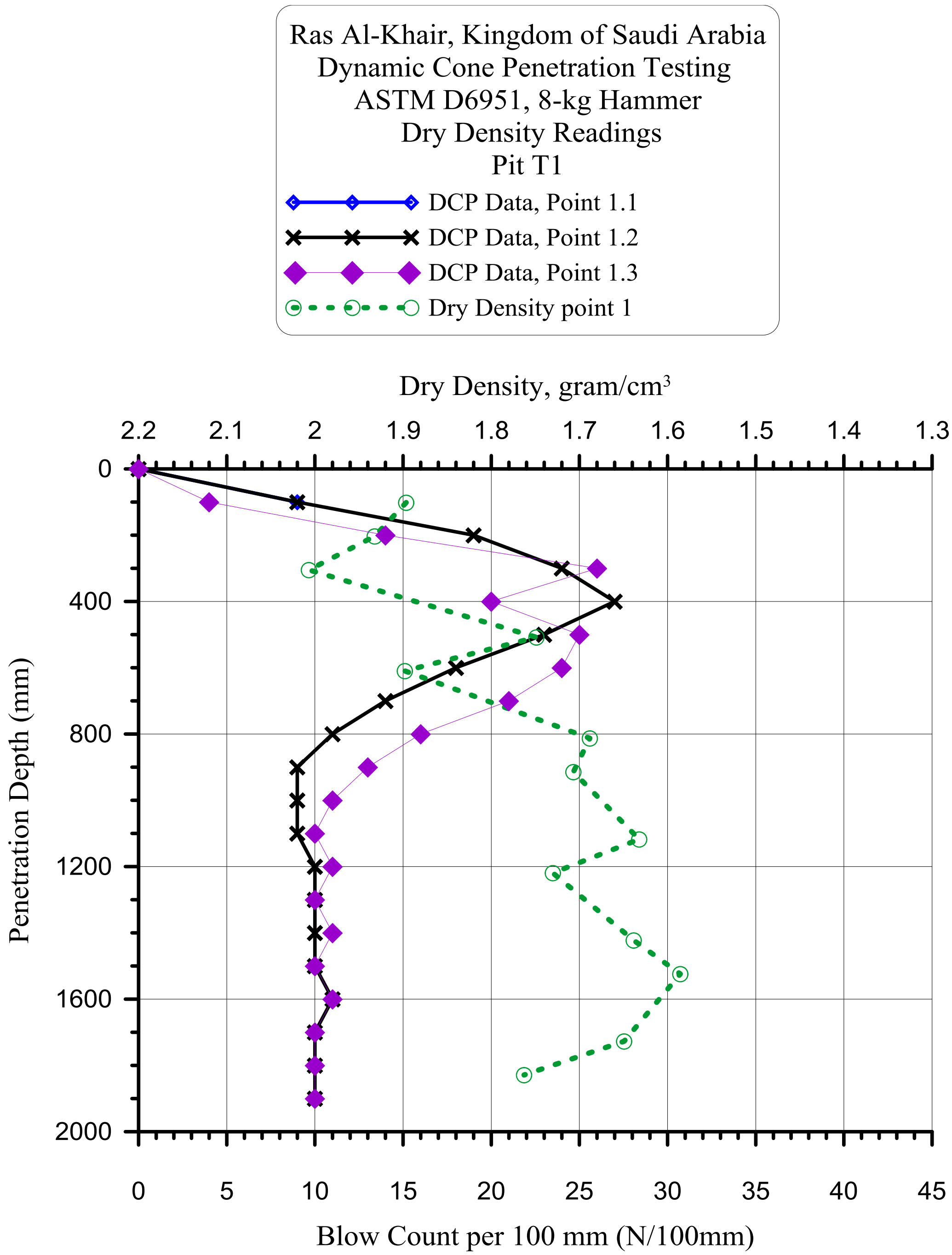


Figure 5-60: Variations of the DCP data and dry density data with depth for Pit 1 (location 1) (Aiban, 2012).

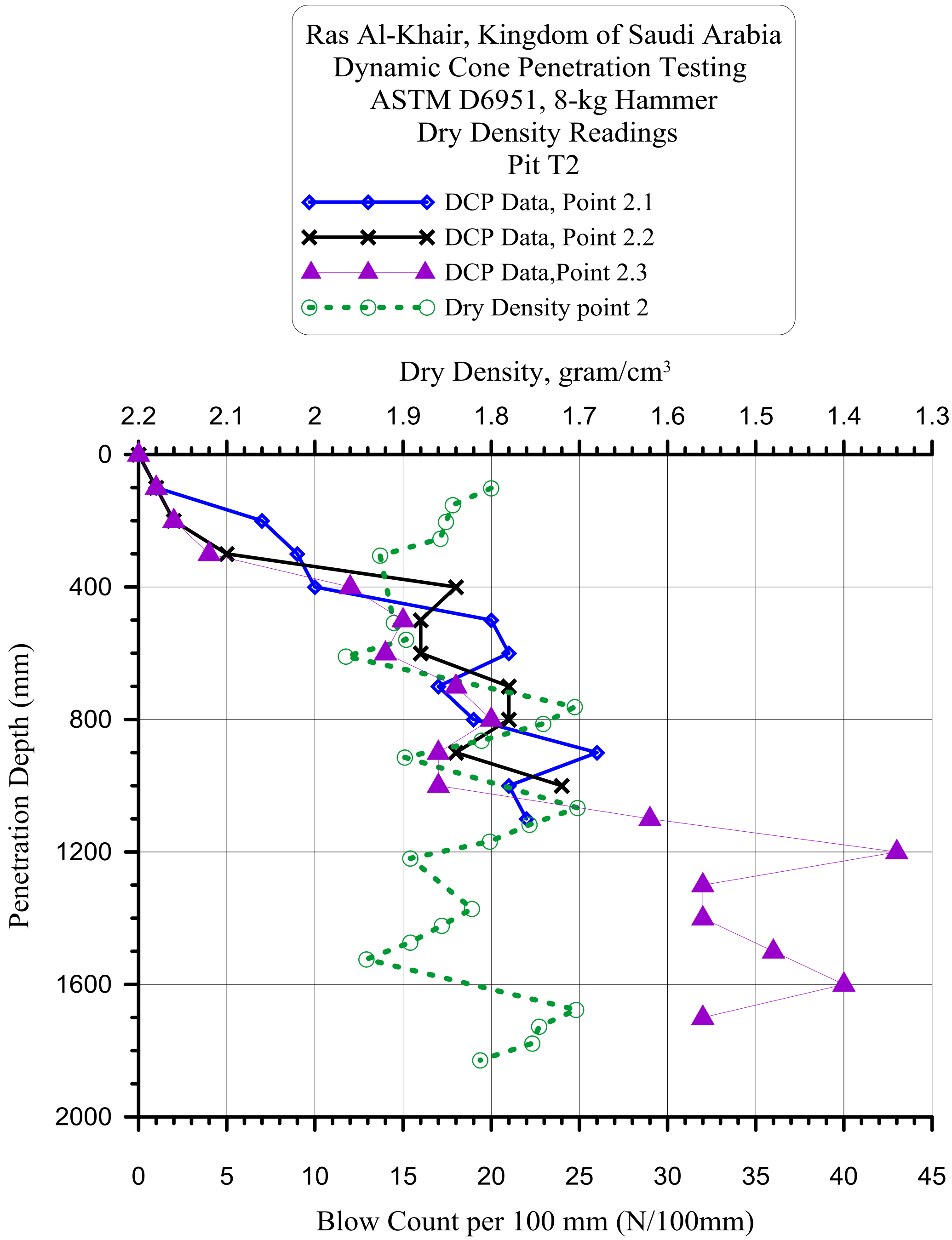


Figure 5-61: Variations of the DCP data and dry density data with depth for Pit 2 (location 2) (Aiban, 2012).

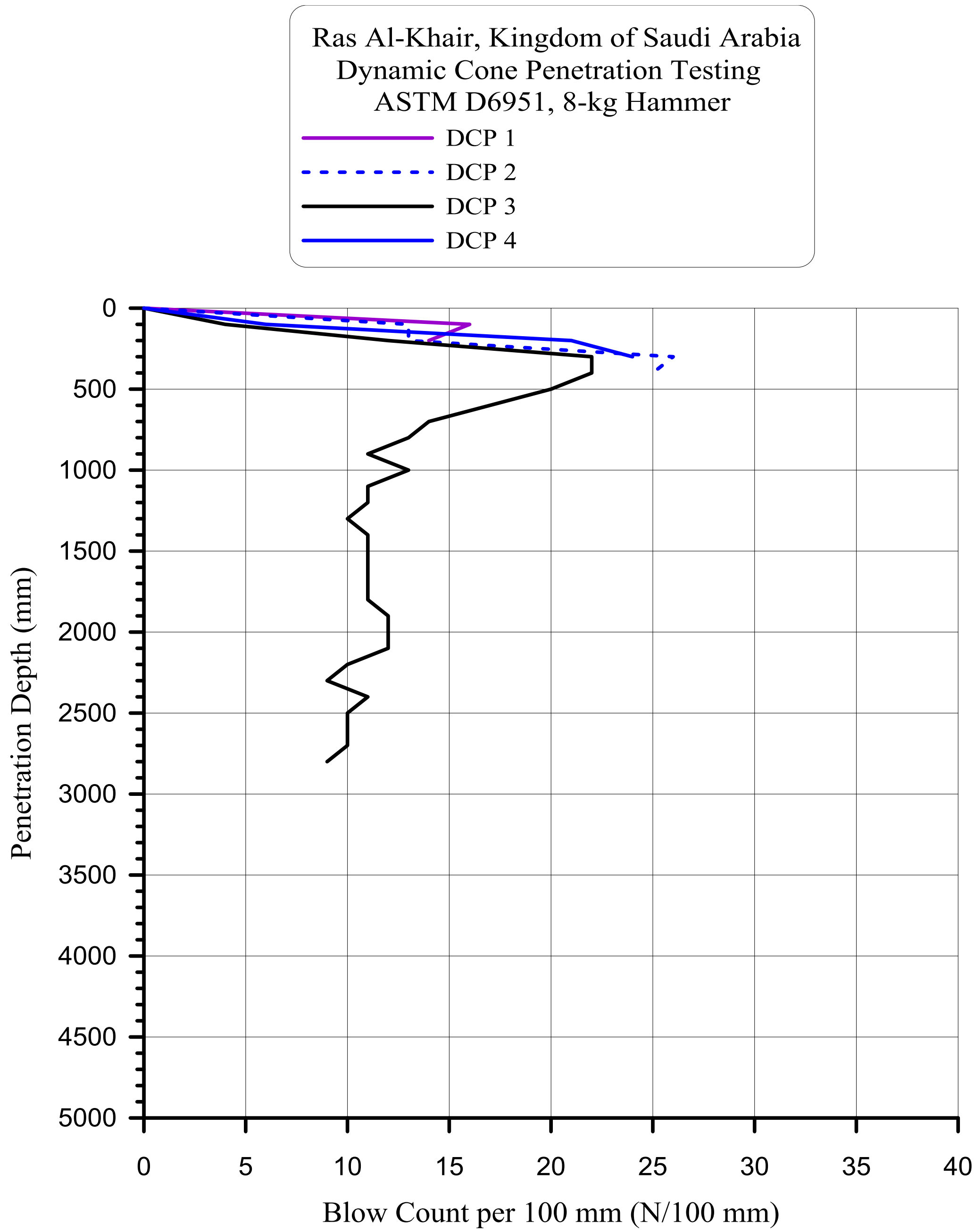


Figure 5-62: Variations of the DCP data with depth for selected locations, Part (a) (Aiban, 2012).

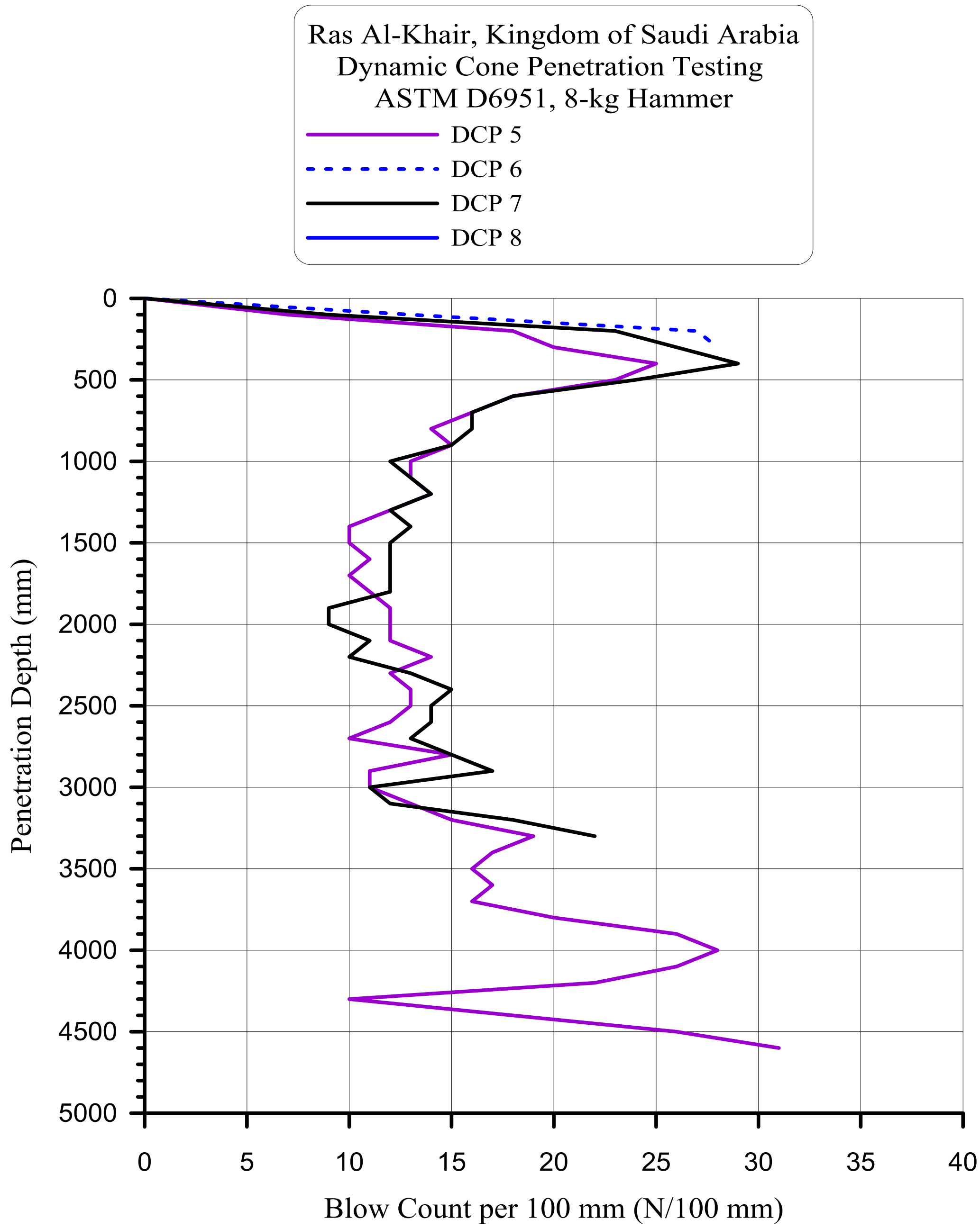


Figure 5-63: Variations of the DCP data with depth for selected locations, Part (b) (Aiban, 2012).

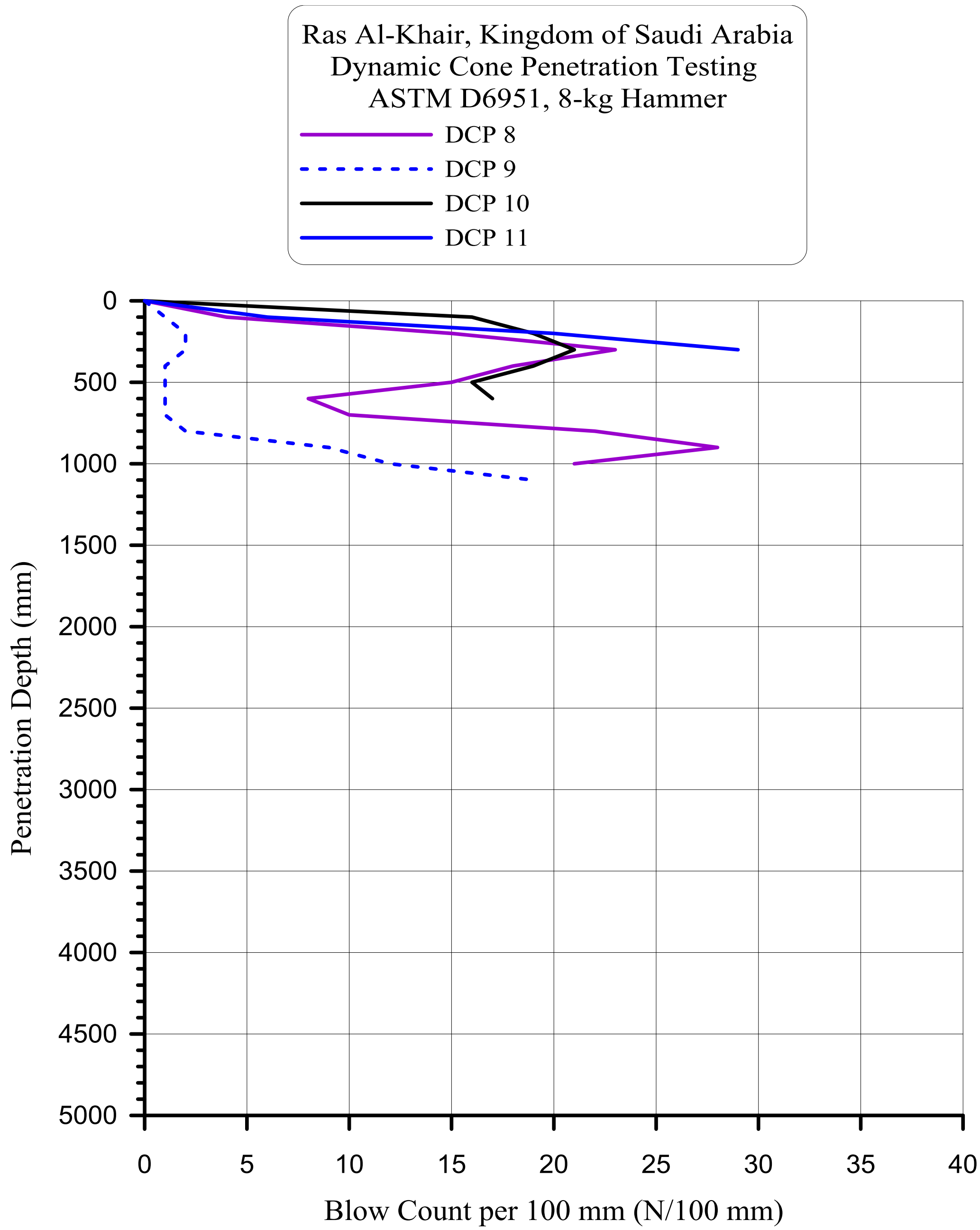


Figure 5-64: Variations of the DCP data with depth for selected locations, Part (c) (Aiban, 2012).

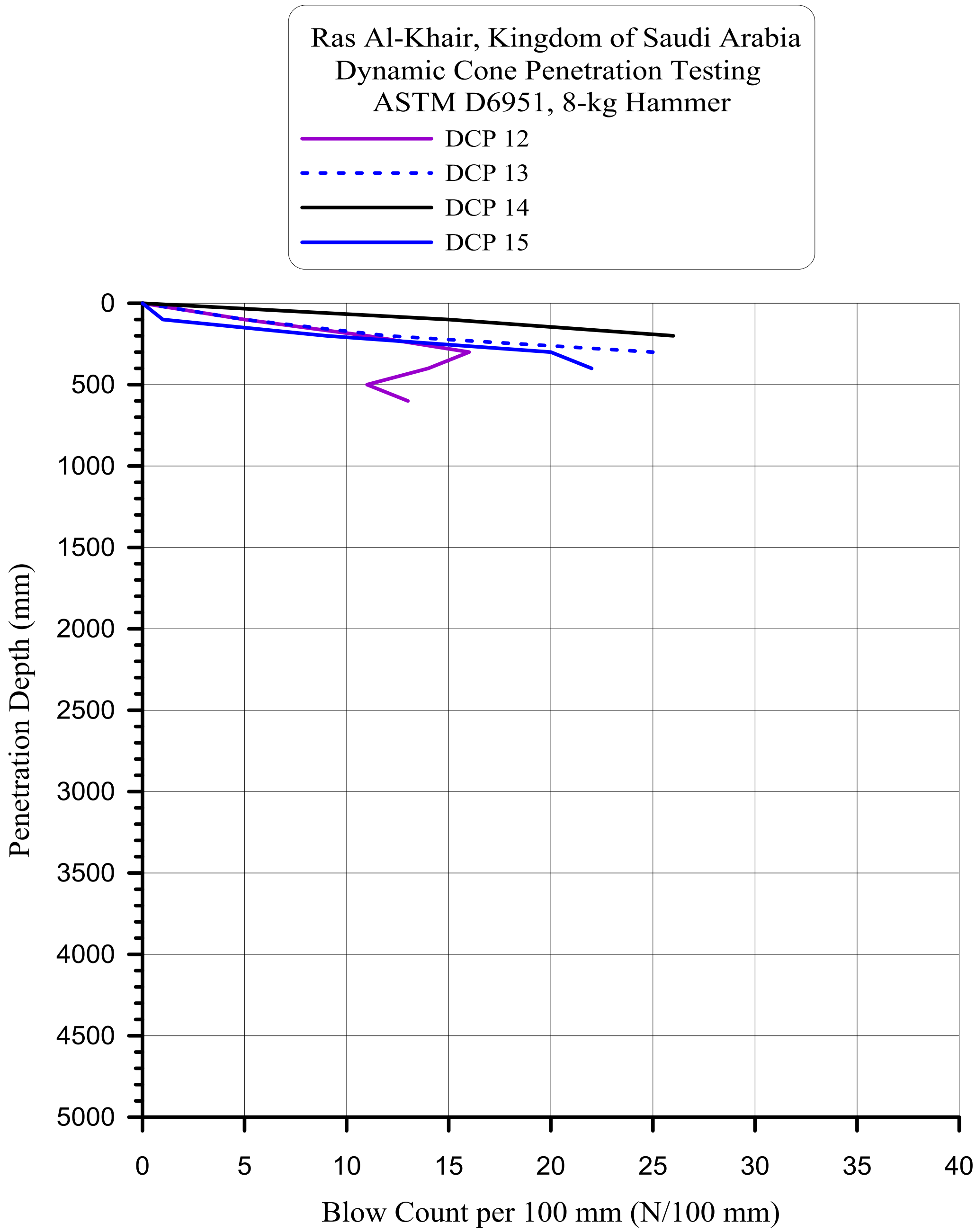


Figure 5-65: Variations of the DCP data with depth for selected locations, Part (d) (Aiban, 2012).

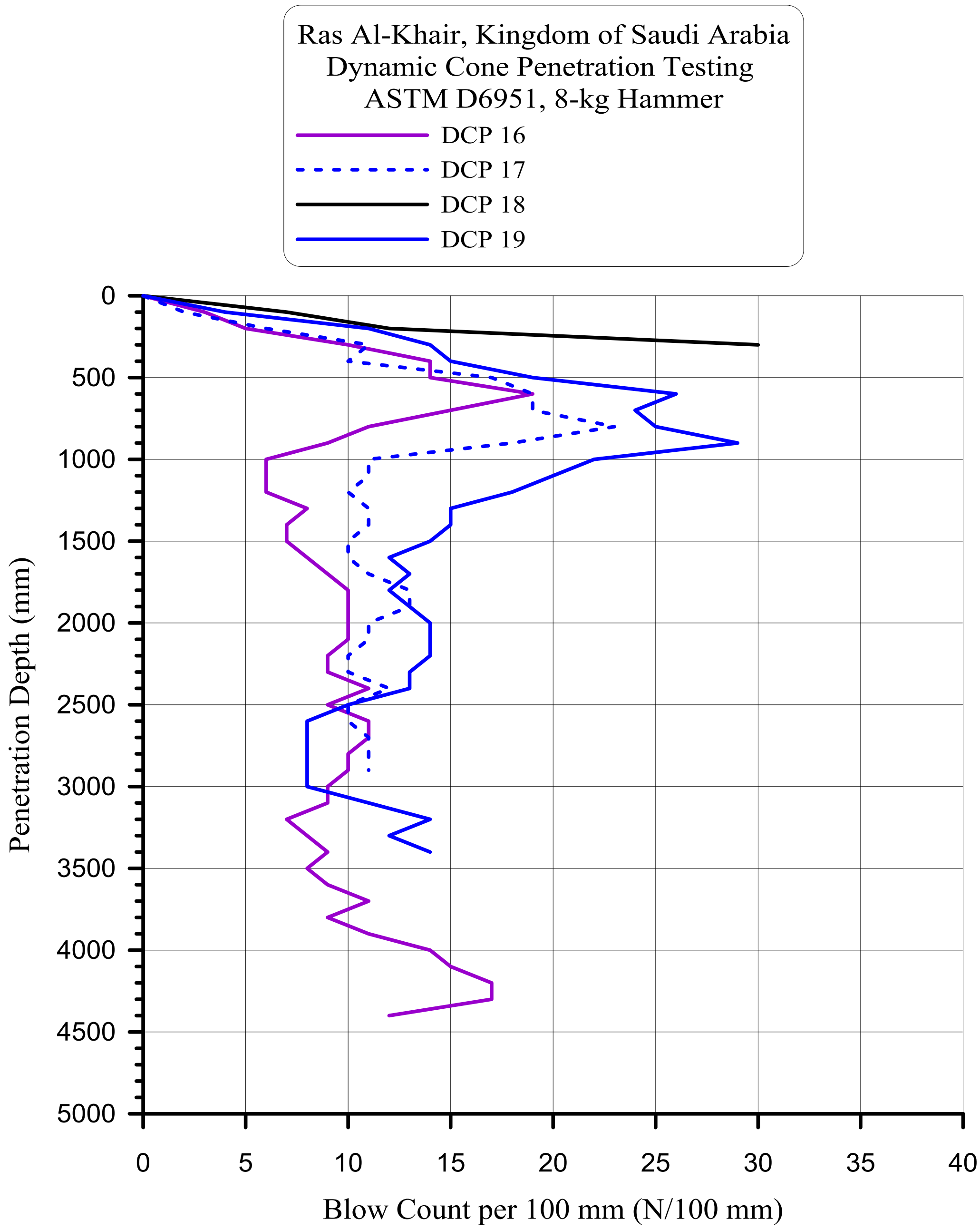


Figure 5-66: Variations of the DCP data with depth for selected locations, Part (e) (Aiban, 2012).

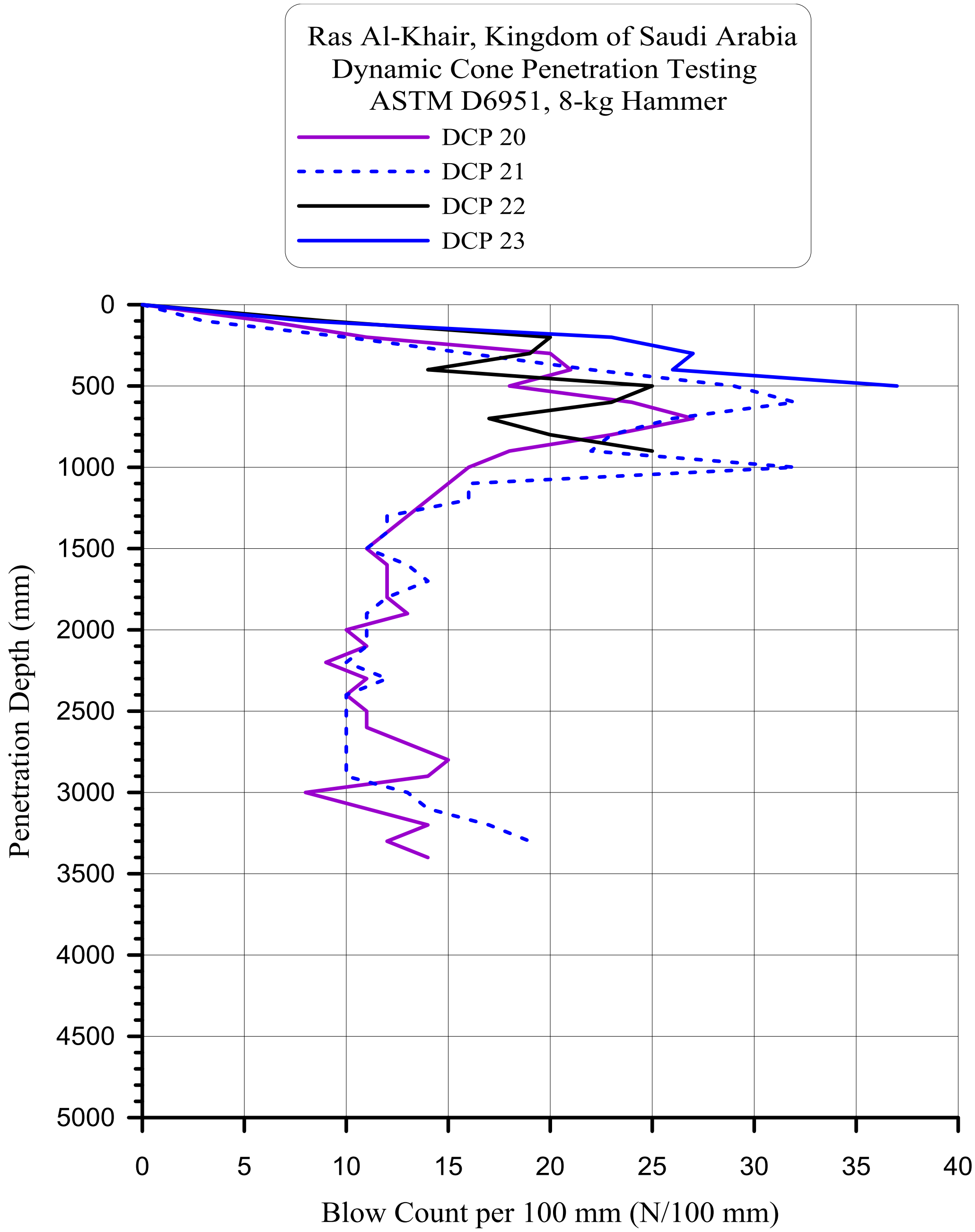


Figure 5-67: Variations of the DCP data with depth for selected locations, Part (f) (Aiban, 2012).

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6. 1 Summary

The Dynamic Cone Penetrometer (DCP) is an instrument that can be used to evaluate the compaction of sand backfills and base material. Perhaps the greatest advantage of the DCP device lies in its ability to provide a continuous record of relative soil stiffness with depth and its simple use and operation. Another advantage is that it can be conducted in very confined spaces without the need for trucks or heavy machines that may not have enough access to the site or may damage the existing installations. Good correlations were obtained between DCP data and material properties such as density and angle of internal friction.

6. 2 Conclusions

Based on laboratory study, the following conclusions could be drawn:

1. The blow counts per unit depth increased significantly as the density of sand increased for different silt content (1%, 4% and 8% ± 0.2). This is mainly due to the reason that compacted dry soils have close packing of particles and thus higher penetration resistance and higher stiffness. In addition, the vertical confining pressure tends to increase with depth thereby increasing blow count.

2. An increase in the relative density from 40% to 90% has resulted in a corresponding decrease in the dynamic cone penetration index.

3. The shear strength has a significant effect on dynamic cone penetration test results. It was observed that an increase in the friction angle resulted in a decrease in the DCPI.

4. Variations in water table level have a significant effect on DCPT result. At the height of 466 mm of water level (i.e. one third of the layer thickness of the soil) above the base was observed change in the resistance of the sand, which led to a distinct change in the test results compared to the situation of the dry sand which showed a higher resistance as a reaction to the capillary pressure; as a result of this resistance a noticeable change in the results of dynamic cone penetration test are recorded. In the case of submerged sand, the effective stress of sand has been decreased due to positive pore water pressure that confirms that the penetration resistance indicating a significant negative influence on the stiffness of sand. In the case of negative pore pressure, the DCP resistance increases significantly as much as 166% when compared to those for submerged or dry sand, due to the presence of negative pore pressure that increased the shear strength and stiffness of sand.

5. Statistical analyses were used to determine the best correlations of the results. All the developed correlation between dynamic cone penetration index (DCPI) and other parameters were reliable with a determination coefficient mostly greater than 0.90.

Based on field study, the following conclusions are drawn:

1 Based on the field investigation on Al-Jubail, it appeared that the soil compaction was not consistent and lower than the project requirements/specifications. All field data clearly indicate that the sand backfill was loose and susceptible to compression upon wetting, vibrations or change in loading. The DCP-nuclear gauge data clearly indicated the following:

   a. There is a good correlation between the dry density obtained from the nuclear gauge and the dynamic cone penetration (DCP) readings. A density of sand of 1.73 gram/cm^3 (95% relative compaction) or more corresponds to DPC blow count values of 7 Blows/100 mm (or 70 Blows/ meter) or more. Lower DCP blow counts resulted in lower density values.
   b. In such evaluation processes, it is always practical to ignore the field data/readings for the top 300 mm layer due to the disturbance over time which will usually result in density values that would be much less than the normal undisturbed value for sands.
   c. The field data have clearly indicated the non-homogeneity of the sand compaction for all phases. The variations were clear for the same location (variation with depth) and between different locations. These variation clearly indicate the superiority of the DCP testing and its potential use for quality control of mass deep backfilling (within 3 to 5 meters).

2 Based on the field investigation on Rass-Alkhair, it appears that the soil compaction was consistent and displayed a sandy material at a very dense state.

The top 500 mm was always exception; it could be loose due to disturbance or very dense due to traffic loading and little cementation from dust. All field data clearly indicated that the sandy material at the top 5 m of the Ras Al-khair site was similar in density. The DCP-nuclear gauge data clearly indicated the following:

a. There is a good correlation between the dry density obtained from the nuclear gauge and the dynamic cone penetration (DCP) readings. A density of sand of 1.73 gram/cm^3 (95% relative compaction) or more corresponds to DPC blow count value of 7 Blows/100 mm or more. Lower DCP blow counts resulted in lower density values. The data obtained by the DCP was rather qualitative and not quantitative and, at this stage, should be used for comparison purposes only.
b. In such evaluation process, it is always practical to ignore the field data/readings for the top 200~300 mm due to the disturbance over time which will usually result in density values that would be much less or much more (for cemented material) than the normal undisturbed value for sands.
c. The field data have clearly indicated the homogeneity of the sand compaction below one meter of the site soil. The variations were minimal.

3 The dynamic cone penetration test has proven to be an effective tool in the assessment of in situ strength and stiffness of sand backfill.

6. 3 Recommendations for Future Study

Following are the recommendations for future research based on the experimental findings of this research program:

1. Dynamic cone penetration test should be performed on sand with percentages of silt content more than 8%.
2. In order to study the effect of vertical confining stress on sand layer, a comprehensive testing program should be designed to conduct different confining stresses on sand and performing dynamic cone penetration test.
3. Dynamic cone penetration test should be studied for other types of soil like sabkha, clay and marl.

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