TECHNICAL *Summary*

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Dynamic Cone Penetration Test (DCPT) for Subgrade Assessment

Introduction

In-situ penetration tests have been widely used in geotechnical and foundation engineering for site investigation in support of analysis and design. The standard penetration test (SPT) and the cone penetration test (CPT) are two typical in-situ penetration tests. The dynamic cone penetration test shows features of both the CPT and the SPT. The DCPT is similar to the SPT in test. It is performed by dropping a hammer from a certain fall height and measuring a penetration depth per blow for each tested depth. The shape of the dynamic cone is similar to that of the penetrometer used in the CPT.

In road construction, there is a need to assess the adequacy of the subgrade to behave satisfactorily beneath a pavement. A recently completed Joint Transportation Research Program project showed that the DCPT can be used to evaluate the mechanical properties of compacted subgrade soils. In the present implementation project, the application of the DCPT is further investigated.

Present practice in determining the adequacy of a compacted subgrade is to determine the dry density and water content by the sand-cone method or with a nuclear gauge. However, the use of the resilient modulus (Mr) has become mandatory for pavement design. To find the Mr, a time-consuming testing procedure is required which demands significant effort. Therefore a faster and easier alternative for compaction control in road construction practice is desired. To this end, the present project aimed to take a first step in the generation of data to create appropriate correlations among subgrade parameters and DCPT results.

The present research project consists of field testing, laboratory testing, and analysis of the results. The field testing includes the DCPT and nuclear gauge tests. In the planning stage, several road construction sites were selected for the field testing. For the selected road construction sites, both the DCPT and nuclear tests were performed at the same location, allowing a comparison between DCPT and nuclear test results. Soil samples for the selected project sites were also obtained for the laboratory testing program.

Findings

For clayey sand classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), the equation for the dry density in terms of PI can be used for predicting γd using field DCP tests.

Since such predictions using the DCPT are subject to considerable uncertainty, DCPT should be performed for compaction control in combination with a few conventional test methods, such as the nuclear gage. These can be used to anchor or calibrate the DCPT correlation for specific sites, reducing the uncertainty in the predictions. Site-specific correlations do appear to be of better quality.

The DCPT should not be used in soil with gravel. Unrealistic PI values could be obtained and the penetrometer shaft could be bent.

Implementation

Results from the field testing, laboratory testing and analysis lead to the following conclusions and recommendations:

1) Field DCP Tests were performed at seven sites. Four sites contained clayey sands, one contained a well graded sand with clay and two

contained a poorly graded sand. For each test location, in-situ soil density and moisture contents were measured using a nuclear gauge at three different depths. The relationship between the soil properties and the penetration index were examined. Though the data shows considerable scatter, a trend appears to exist, particularly if each site is considered separately, the penetration index decreases as the dry density increases and slightly increases as moisture content increases. It may be possible to improve the correlation by normalizing the quantities in a different way and by obtaining more data.

2) For clayey sand classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), the equation for the dry density was derived in terms of the PI as follows:

$$\gamma_d = \left(10^{1.5} \times PI^{-0.14} \times \sqrt{\frac{\sigma'_V}{p_A}}\right)^{0.5} \times \gamma_W$$

where PI = penetration index in mm/blow; and pA = reference stress (100kPa).

This equation can be used to predict γd from the measured PI value. The actual γd will be in a range defined by the calculated γd ± 1.63 kN/m3.

3) To investigate the relationship between the shear strength of poorly graded sand and the penetration index, direct shear tests were performed on samples obtained from the field. The results of the direct shear tests also show considerable scatter.

4) For clayey sands and well-graded sands with clay classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), unconfined compression tests were conducted. The test results show some correlation with the penetration index (PI). It was observed that PI decreases as unconfined compressive strength increases. Additionally, the resilient modulus was calculated from su at 1.0% strain using the Lee (1997) equation. The following correlation was developed between Mr and PI:

Mr=-3279PI + 114100

where Mr=resilient modulus in kPa; and PI=penetration index in mm/blow

This relationship should be used with caution since it is derived from a very weak correlation based on highly scattered data for different sites. There is a need for further study to gather sufficient data to refine this relationship into a reliable equation.

5) For clayey sand classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), the equation for the dry density in terms of PI can be used for predicting γd using field DCP tests.

6) Since such predictions using the DCPT are subject to considerable uncertainty, DCPT should be performed for compaction control in combination with a few conventional test methods, such as the nuclear gage. These can be used to anchor or calibrate the DCPT correlation for specific sites, reducing the uncertainty in the predictions. Site-specific correlations do appear to be of better quality.

7) The DCPT should not be used in soil with gravel. Unrealistic PI value could be obtained and the penetrometer shaft could be bent.

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16. Abstract

In-situ penetration tests have been widely used in geotechnical and foundation engineering for site investigation in support of analysis and design. The standard penetration test (SPT) and the cone penetration test (CPT) are two typical in-situ penetration tests. The dynamic cone penetration test shows features of both the CPT and the SPT. The DCPT is performed by dropping a hammer from a certain fall height and measuring penetration depth per blow for each tested depth. The DCPT is a quick test to set up, run, and evaluate on site. Due to its economy and simplicity, better understanding of DCPT results can reduce efforts and cost for evaluation of pavement and subgrade soils.

Present practice in determining the adequacy of a compacted subgrade is to determine the dry density and water content by either the sand-cone method or the nuclear gauge. The use of the resilient modulus (M_r) has recently become mandatory for pavement design. To find the M_r, a time-consuming test is required which demands significant effort. Therefore, a faster and easier alternative for compaction control in road construction practice is desired**.** To this end, the present project is a step towards the generation of sufficient data to create appropriate correlations between subgrade parameters and DCPT results.

The present research considers several subgrade soils at different road construction sites. Each soil is tested in the field and in the laboratory. The field testing includes the DCPT and nuclear density gauge tests. Based on analysis of this testing, the relationships between the DCPT results and the subgrade parameters such as unconfined compression strength and resilient modulus are obtained.

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Dynamic Cone Penetration Test (DCPT) for Subgrade Assessment

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TABLE OF CONTENTS

CHAPTER 1. INTRODUCTION 1

1.1 Introduction 1

1.2 Problem Statement 2

1.3 Research Objective 3

1.4 Project Outline 3

CHAPTER 2. DYNAMIC CONE PENETRATION TEST AND ITS APPLICATION 4

2.1 Description of Dynamic Cone Penetration Test (DCPT) 4

2.2 Relationship between Penetration Index (PI) and CBR Values 9

2.3 Relationship between PI and Compaction Properties 10

2.4 PI – Shear Strength Relationship 14

CHAPTER 3. DYNAMIC CONE PENETRATION TESTS ON SUBGRADE SOILS 17

3.1 Introduction 17

3.2 Reconstruction Site of I-65 in Hobart, IN 19

3.3 Reconstruction Site of US49 in Valparaiso, IN 27

3.4 Reconstruction Site of I-80/I-94 in Gary, IN 35

3.5 Road Widening Construction Site of US35 in Knox, IN 44

3.6 Reconstruction Site of Lindberg Road at West Lafayette, IN 53

3.7 Reconstruction Site of I-65/County Road 100E in Lebanon, IN 63

3.8 Reconstruction Site of US36 in Bainbridge, IN 71

3.9 Analysis of the Results from Field DCP and Laboratory Tests 80

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS 86

4.1 Conclusions 86

4.2 Recommendations 88

LIST OF REFERENCE 89

LIST OF TABLES

Table 2.1 Correlations between CBR and PI (after Harison 1987 and Gabr et al. 2000) 12
Table 2.2 Basic properties of test materials (after Ayers et al. 1989) 15
Table 2.3 Relationship between PI and shear strength (after Ayers et al. 1989) 16
Table 3.1 Test sites for DCPT 18
Table 3.2 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-65 in Hobart, IN 21
Table 3.3 Result of Unconfined Compressive Test and corresponding Penetration Index from field DCPT for the site of I-65 in Hobart, IN 22
Table 3.4 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US49 in Valpariso, IN 29
Table 3.5 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of US49 in Valpariso, IN 30
Table 3.6 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-80/I-94 in Gary, IN 37
Table 3.7 Result of Direct Shear Test with different normal stress for the site of I-80/I94 in Gary, IN 38
Table 3.8 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US35 in Knox, IN 46
Table 3.9 Result of Direct Shear Test with different normal stress for the site of US35 in Knox, IN 47
Table 3.10 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of Lindberg Road in West Lafayette, IN 55
Table 3.11 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of Lindberg Road in West Lafayette, IN 56
Table 3.12 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I65/County Road100E in Lebanon, IN 65
Table 3.13 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of I65/County Road100E in Lebanon, IN 66
Table 3.14 Total and Dry Soil Densities and Moisture Contents measured from

nuclear gauge for the site of US36 at Bainbridge, IN 73
Table 3.15 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of US36 at Bainbridge, IN 74

LIST OF FIGURES

Figure 2.1 Structure of Dynamic Cone Penetrometer 6
Figure 2.2 Dynamic Cone Penetration Test 7
Figure 2.3 Typical DCPT results 8
Figure 2.4 PI versus compaction parameters from laboratory results (after Harison 1987) 13
Figure 3.1 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-65 in Hobart, IN 22
Figure 3.2 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 1) 23
Figure 3.3 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 2) 23
Figure 3.4 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 3) 24
Figure 3.5 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 4) 24
Figure 3.6 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 5) 25
Figure 3.7 Particle size distribution for the site of I-65 in Hobart, IN 25
Figure 3.8 Relationship between Dry Density and Penetration Index from field DCPT for the site of I-65 in Hobart, IN 26
Figure 3.9 The Relationship between Moisture Content and Penetration Index from field DCPT for the site of I-65 in Hobart, IN 26
Figure 3.10 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US49 in Valpariso, IN 30
Figure 3.11 Log of DCPT for the site of US49 in Valpariso, IN (Station: 18+850, Test No. 1) 31
Figure 3.12 Log of DCPT for the site of US49 in Valpariso, IN (Station: 18+840, Test No. 2) 31
Figure 3.13 Log of DCPT for the site of US49 in Valpariso, IN (Station: 18+846, Test No. 3) 32
Figure 3.14 Log of DCPT for the site of US49 in Valpariso, IN (Station: 18+828, Test

No. 4) 32
Figure 3.15 Particle size distribution for the site of US49 in Valpariso, IN 33
Figure 3.16 Relationship between Dry Density and Penetration Index from field DCPT for the site of US49 in Valpariso, IN 33
Figure 3.17 Relationship between Moisture Content and Penetration Index from field DCPT for the site of US49 in Valpariso, IN 34
Figure 3.18 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-80/I-94 in Gary, IN 38
Figure 3.19 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 1) 39
Figure 3.20 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 2) 39
Figure 3.21 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 3) 40
Figure 3.22 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 4) 40
Figure 3.23 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 5) 41
Figure 3.24 Particle size distribution for the site of I-80/I-94 in Gary, IN 41
Figure 3.25 Relationship between Dry Density and Penetration Index from field DCPT for the site of I-80/I-94 in Gary, IN 42
Figure 3.26 Relationship between Moisture Content and Penetration Index from field DCPT for the site of I-80/I-94 in Gary, IN 42
Figure 3.27 Result of Direct Shear Test with different normal stress for the site of I-80/I-94 in Gary, IN 43
Figure 3.28 Relationship between PI and Shear Strength with different normal stress for the site of I-80/I-94 in Gary, IN 43
Figure 3.29 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US35 in Knox, IN 47
Figure 3.30 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 1) 48
Figure 3.31 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 2) 48

Figure 3.32 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 3) 49
Figure 3.33 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 4) 49
Figure 3.34 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 5) 50
Figure 3.35 Particle size distribution for the site of US35 in Knox, IN 50
Figure 3.36 Relationship between Dry Density and Penetration Index from field DCPT for the site of US35 in Knox, IN.......... 51
Figure 3.37 Relationship between Moisture Content and Penetration Index from field DCPT for the site of US35 in Knox, IN.......... 51
Figure 3.38 Result of Direct Shear Test with different normal stress for the site of US35 in Knox, IN.......... 52
Figure 3.39 Relationship between PI and Shear Strength with different normal stress for the site of US35 in Knox, IN 52
Figure 3.40 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of Lindberg Road in West Lafayette, IN.......... 57
Figure 3.41 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+189, Test No. 1) 57
Figure 3.42 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+200, Test No. 2) 58
Figure 3.43 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+211, Test No. 3) 58
Figure 3.44 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+222, Test No. 4) 59
Figure 3.45 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+233, Test No. 5) 59
Figure 3.46 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+245, Test No. 6) 60
Figure 3.47 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+256, Test No. 7) 60
Figure 3.48 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+269, Test No. 8) 61

Figure 3.49 Particle size distribution for the site of Lindberg Road in West Lafayette, IN ... 61
Figure 3.50 Relationship between Dry Density and Penetration Index from field DCPT for the site of Lindberg Road in West Lafayette, IN ... 62
Figure 3.51 Relationship between Moisture Content and Penetration Index from field DCPT for the site of Lindberg Road in West Lafayette, IN ... 62
Figure 3.52 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I65/County Road100E in Lebanon, IN ... 66
Figure 3.53 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 1) ... 67
Figure 3.54 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 2) ... 67
Figure 3.55 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 3) ... 68
Figure 3.56 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 4) ... 68
Figure 3.57 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 5) ... 69
Figure 3.58 Particle size distribution for the site of I65/County Road100E in Lebanon, IN ... 69
Figure 3.59 Relationship between Dry Density and Penetration Index from field DCPT for the site of I65/County Road100E in Lebanon, IN ... 70
Figure 3.60 Relationship between Moisture Content and Penetration Index from field DCPT for the site of I65/County Road100E in Lebanon, IN ... 70
Figure 3.61 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US36 at Bainbridge, IN ... 74
Figure 3.62 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+505, Test No. 1) ... 75
Figure 3.63 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+506, Test No. 2) ... 75
Figure 3.64 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+722, Test No. 3) ... 76
Figure 3.65 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+724,

Test No. 4).. 76
Figure 3.66 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+574, Test No. 5).. 77
Figure 3.67 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+577, Test No. 6).. 77
Figure 3.68 Particle size distribution for the site of US36 at Bainbridge, IN 78
Figure 3.69 Relationship between Dry Density and Penetration Index from field DCPT for the site of US36 at Bainbridge, IN... 78
Figure 3.70 Relationship between Moisture Content and Penetration Index from field DCPT for the site of US36 at Bainbridge, IN... 79
Figure 3.71 Relationship between Moisture Content and Dry Density............................... 82
Figure 3.72 Relationship between Dry Density and Penetration Index............................... 82
Figure 3.73 Relationship between Moisture Content and Penetration Index 83
Figure 3.74 Relationship between Unconfined Compressive Strength and Penetration Index 83
Figure 3.75 Relationship between s_u at 1.0% strain and Penetration Index 84
Figure 3.76 Relationship between Resilient Modulus and Penetration Index..................... 84
Figure 3.77 Relationship between normalized Dry density and Penetration Index 85

IMPLEMENTATION REPORT

In geotechnical and foundation engineering in-situ penetration tests have been widely used for site investigation in support of analysis and design. The standard penetration test (SPT) and the cone penetration test (CPT) are the two in-situ penetration tests often used in practice. The SPT is performed by driving a sampler into the ground by hammer blows uses a dynamic penetration mechanism, while in the CPT a cone penetrometer is pushed quasi-statically into the ground. In the DCPT, a cone penetrometer is driven into the ground, so that the DCPT shows some features of both the CPT and SPT.

Quality road construction requires an assessment of the adequacy of a subgrade to behave satisfactorily beneath a pavement. Present practice in determining the adequacy of a compacted subgrade is to determine the dry density and water content by the sand-cone method or with a nuclear gauge. The use of the resilient modulus (M_r) has recently become mandatory for pavement design. To find the M_r, a time-consuming test is required, which demands significant effort.

The DCP is operated by two persons, and is a quick test to set up, run, and evaluate on site. Due to its economy and simplicity, better understanding of the DCPT results can reduce significantly the efforts and cost for evaluation of pavement and subgrade soils. The intention of this project is to generate sufficient data to create appropriate correlations among subgrade parameters and DCPT results.

The present research project consists of field testing, laboratory testing, and analysis of the results. The field testing includes the DCPT and nuclear tests. In the planning stage, several road construction sites were selected for the field testing. For the selected road construction sites, both the DCPT and nuclear tests were performed at the same location allowing comparison between DCPT and nuclear test results. Soil samples for the selected project sites were also obtained for the laboratory testing program.

Results from the field testing, laboratory testing and analysis lead to the following conclusions and recommendations:

Conclusions

(1) Field DCP Tests were performed at seven sites. Four sites contained clayey sands, one contained a well graded sand with clay and two contained a poorly graded sand. For each test location, in-situ soil density and moisture contents were measured using a nuclear gauge at three different depths. The relationship between the soil properties and the penetration index were examined. Though the data shows considerable scatter, a trend appears to exist, particularly if each site is considered separately, the penetration index decreases as the dry density increases and slightly increases as moisture content increases. It may be possible to improve the correlation by normalizing the quantities in a different way and by obtaining more data.

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where PI = penetration index in mm/blow; and p_A = reference stress (100kPa).

This equation can be used to predict γ_d from the measured PI value. The actual γ_d will be in a range defined by the calculated $\gamma_d \pm 1.63\,kN/m^3$.

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(4) For clayey sands and well-graded sands with clay classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), unconfined compression tests were conducted. The test results show some correlation with the penetration index (PI). It was observed that PI decreases as unconfined compressive strength increases. Additionally, the resilient modulus was calculated from s_u at 1.0% strain using the Lee (1997) equation. The following correlation was developed between M_r and PI:

$$M_r = -3279PI + 114100$$

where M_r=resilient modulus in kPa; and PI=penetration index in mm/blow

This relationship should be used with caution since it is derived from a very weak correlation based on highly scattered data for different sites. There is a need for further study to gather sufficient data to refine this relationship into a reliable equation.

Recommendations

(1) For clayey sand classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), the equation for the dry density in terms of PI can be used for predicting γ_d using field DCP tests.

(2) Since such predictions using the DCPT are subject to considerable uncertainty, DCPT should be performed for compaction control in combination with a few conventional test methods, such as the nuclear gage. These can be used to anchor or calibrate the DCPT correlation for specific sites, reducing the uncertainty in the predictions. Site-specific correlations do appear to be of better quality.

(3) The DCPT should not be used in soil with gravel. Unrealistic PI values could be obtained and the penetrometer shaft could be bent.

CHAPTER 1. INTRODUCTION

1.1 Introduction

In geotechnical and foundation engineering, in-situ penetration tests have been widely used for site investigation in support of analysis and design. The standard penetration test (SPT) and the cone penetration test (CPT) are two typical in-situ penetration tests. While the SPT is performed by driving a sampler into the soil with hammer blow, the CPT is a quasi-static procedure.

The dynamic cone penetration test (DCPT) was developed in Australia by Scala (1956). The current model was developed by the Transvaal Roads Department in South Africa (Luo, 1998). The mechanics of the DCPT shows features of both the CPT and SPT. The DCPT is performed by dropping a hammer from a certain fall height measuring penetration depth per blow for a certain depth. Therefore it is quite similar to the procedure of obtaining the blow count N using the soil sampler in the SPT. In the DCPT, however, a cone is used to obtain the penetration depth instead of using the split spoon soil sampler. In this respect, there is some resemblance with the CPT in the fact that both tests create a cavity during penetration and generate a cavity expansion resistance.

In road construction, there is a need to assess the adequacy of a subgrade to behave satisfactorily beneath a pavement. Proper pavement performance requires a satisfactorily performing subgrade. A recent Joint Transportation Research Program project by Luo (1998) was completed showing that the DCPT can be used to evaluate the mechanical properties of compacted subgrade soils. In the present implementation project, the application of the DCPT is further investigated.

1.2 Problem Statement

Present practice in determining the adequacy of a compacted subgrade is to determine the dry density and water content by the sand-cone method or with a nuclear density gauge. This testing is done with the expectation that successful performance in-service will occur if the compaction specifications are found to be fulfilled. In addition, the use of the resilient modulus (M_r) has also become mandatory for pavement design. To find the M_r, another time consuming test is required which demands significant effort.

There is much interest in finding a quick positive way to assure the presence of desired behavior parameters in a subgrade. The quality of a subgrade is generally assessed based on the dry density and water content of soils compared with the laboratory soil compaction test results. This connection is based on the observation that the strength of soils and compressibility of soils is well-reflected by dry density. While the sand cone method was a common approach to evaluate a subgrade in practice in the past, use of the nuclear gauge is currently very popular. The nuclear gauge is quick and very convenient to obtain the in-situ soil density and water content. However it uses nuclear power and requires a special operator who has finished a special training program and has a registered operating license. Therefore, a safer and easier alternative for the compaction control of road construction practice is desired.

1.3 Research Objective

The goal of this project is to generate sufficient data to create appropriate correlations among subgrade parameters and DCPT results. Successful completion will allow road construction engineers to assess subgrade adequacy with a relatively quick, easy-to-perform test procedure avoiding time-consuming testing. It is expected to cover the range of fine-textured soils encountered in practice. Detailed objectives are:

(1) Generation of sufficient data to allow development of initial correlations.

(2) Investigation of the relationship between DCPT results and subgrade parameters such as soil density, water content, and resilient modulus.

1.4 Project Outline

The present research project consists of field testing, laboratory testing, and analysis of the results. The field testing includes the DCPT and nuclear tests. In the planning stage, several road construction sites were selected for the field testing. For the selected road construction sites, both the DCPT and nuclear tests were performed at the same location allowing a comparison between DCPT and nuclear test results. Soil samples for the selected project sites were also obtained for the laboratory testing program.

Based on the field and laboratory test results, the relationship between the DCPT results and subgrade parameters such as unconfined compression strength and resilient modulus will be investigated.

CHAPTER 2. DYNAMIC CONE PENETRATION TEST AND ITS APPLICATION

2.1 Description of Dynamic Cone Penetration Test (DCPT)

The dynamic cone penetration test (DCPT) was originally developed as an alternative for evaluating the properties of flexible pavement or subgrade soils. The conventional approach to evaluate strength and stiffness properties of asphalt and subgrade soils involves a core sampling procedure and a complicated laboratory testing program such as resilient modulus, Marshall tests and others (Livneh et al. 1994). Due to its economy and simplicity, better understanding of the DCPT results can reduce significantly the effort and cost involved in the evaluation of pavement and subgrade soils.

Figure 2.1 shows a typical configuration of the dynamic cone penetrometer (DCP). As shown in the figure, the DCP consists of upper and lower shafts. The upper shaft has an 8 kg (17.6 lb) drop hammer with a 575 mm (22.6 in) drop height and is attached to the lower shaft through the anvil. The lower shaft contains an anvil and a cone attached at the end of the shaft. The cone is replaceable and has a 60 degree cone angle. As a reading device, an additional rod is used as an attachment to the lower shaft with marks at every 5.1 mm (0.2 in).

In order to run the DCPT, two operators are required. One person drops the hammer and the other records measurements. The first step of the test is to put the cone tip on the testing surface. The lower shaft containing the cone moves independently from the reading rod sitting on the testing surface throughout the test. The initial reading is not usually equal to 0 due to the disturbed loose state of the ground surface and the self-weight of the testing equipment. The value of the initial reading is counted as initial penetration corresponding to blow 0. Figure 2.2 shows the penetration result from the first drop of the hammer. Hammer blows are repeated and the penetration depth is measured for each hammer drop. This process is continued until a desired penetration depth is reached.

As shown in Figure 2.3, DCPT results consist of number of blow counts versus penetration depth. Since the recorded blow counts are cumulative values, results of DCPT in general are given as incremental values defined as follows,

$$PI = \frac{\Delta D_p}{\Delta BC} \qquad (2.1)$$

where PI = DCP penetration index in units of length divided by blow count; ΔD_p = penetration depth; BC = blow counts corresponding to penetration depth ΔD_p. As a result, values of the penetration index (PI) represent DCPT characteristics at certain depths.

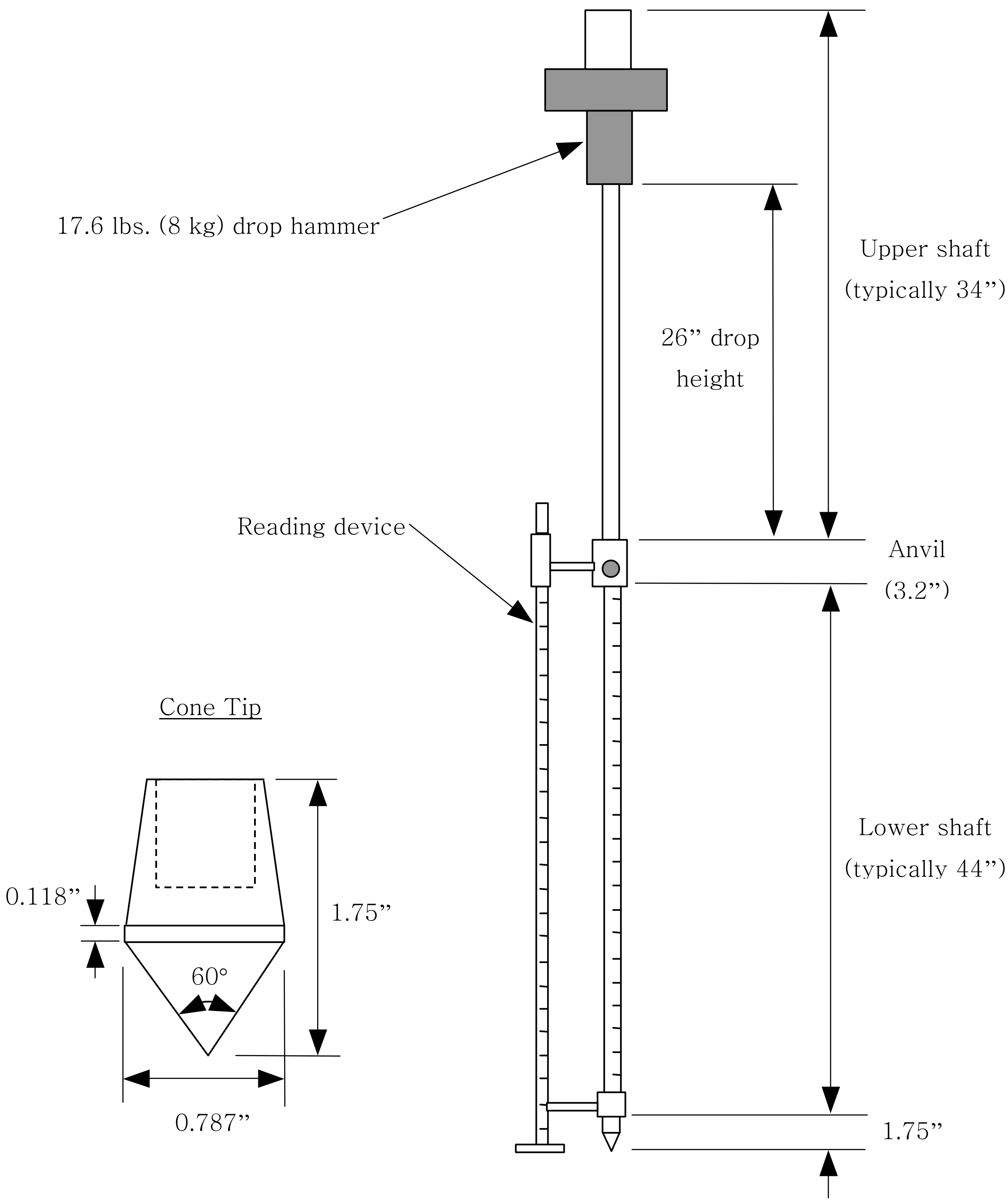
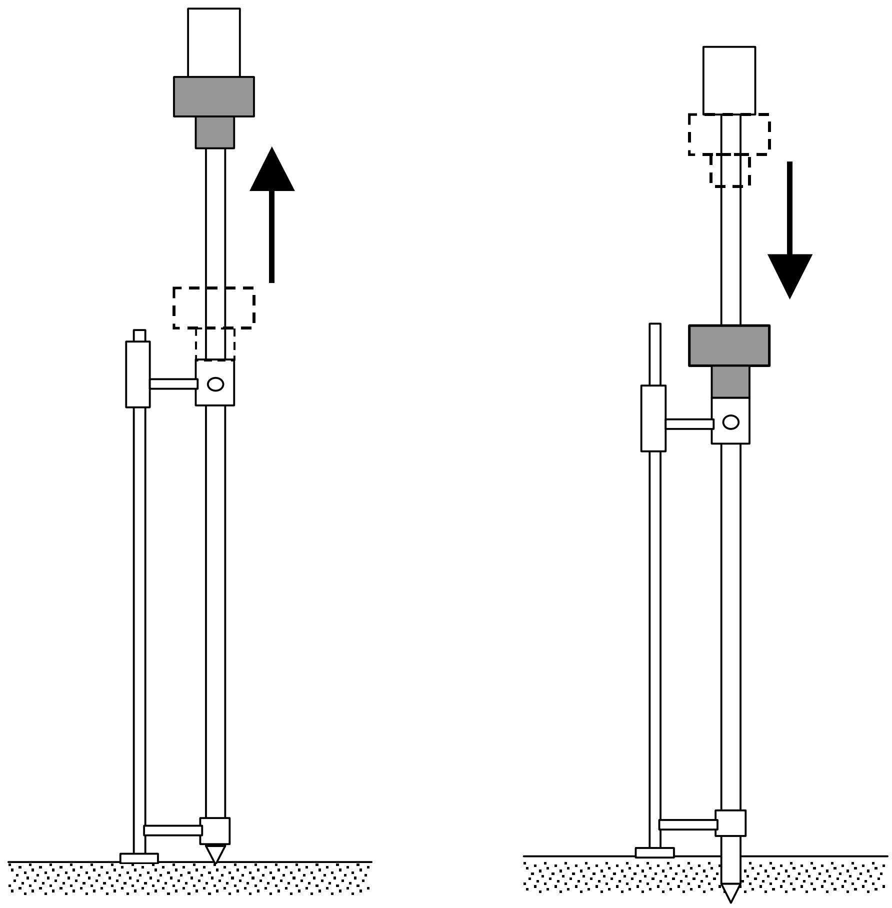
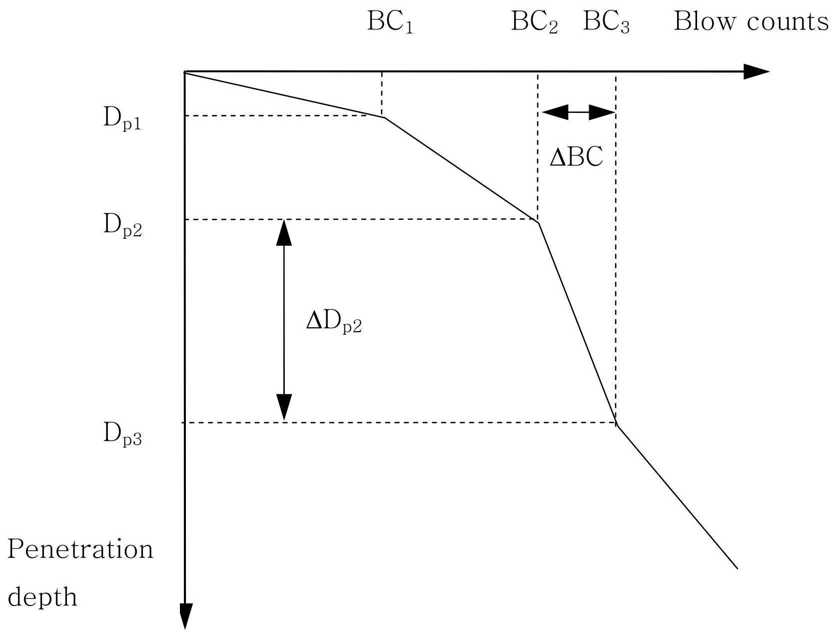


Figure 2.1 Structure of Dynamic Cone Penetrometer

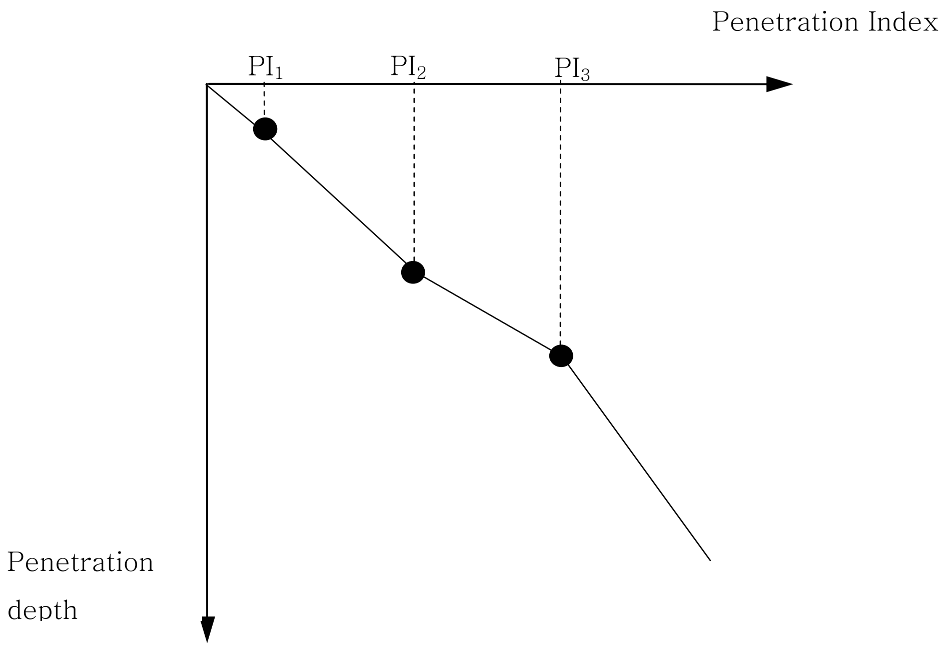


(a) Before hammer dropping (b) After hammer dropping

Figure 2.2 Dynamic Cone Penetration Test



(a)



(b)

Figure 2.3 Typical DCPT results

2.2 Relationship between Penetration Index (PI) and CBR Values

Several authors have investigated relationships between the DCP penetration index PI and California Bearing Ratio (CBR). CBR values are often used in road and pavement design. Two types of equations have been considered for the correlation between the PI and CBR. Those are the log-log and inverse equations. The log-log and inverse equations for the relationship can be expressed as the following general forms:

log-log equation: $\log CBR = A - B \cdot (\log PI)^{C}$ (2.2)

inverse equation: CBR = D(PI)E + F (2.3)

where CBR = California Bearing Ratio; PI = penetration index obtained from DCPT in units of mm/blow or in/blow; A ,B, C, D, E, and F = regression constants for the relationships. Based on statistical analysis of results from the log-log and inverse equations, Harison (1987) concluded that the log-log equation produces more reliable results while the inverse equation contains more errors and is not suitable to use. Considering the log-log equations, many authors have proposed different values of A, B, and C for use in (2.2). For example, Livneh (1987) and Livneh, M. (1989) proposed the following relationships based on field and laboratory tests:

$$\log CBR = 2.20 - 0.71 \cdot (\log PI)^{1.5} \quad (2.4)$$

$$\log CBR = 2.14 - 0.69 \cdot (\log PI)^{1.5} \quad (2.5)$$

where CBR = California Bearing Ratio; PI = DCP Penetration Index. Although (2.5) was suggested based on (2.4), differences in results from (2.4) and (2.5) are small. After further examination of results by other authors, Livneh et al. (1994) proposed the following equation as the best correlation:

$$\log CBR = 2.46 - 1.12 \cdot (\log PI) \qquad (2.6)$$

Table 2.1 summarizes typical log-log equations suggested by different authors for the CBR-PI correlation.

2.3 Relationship between PI and Compaction Properties

The CBR and DCPT have similar testing mechanisms. Thus, results from the tests may reflect similar mechanical characteristics. Compared to work done for PI-CBR relationships described in the previous section, investigations of the PI - compaction properties relationships were insufficiently performed. This condition may be because the compaction properties, including dry unit weight and moisture content, are affected by a number of different factors. The compacted unit weight itself also depends on the moisture content.

Although limited information concerning these relationships appears in the literature, a typical relationship can be found in Harison (1987) and Ayers et al. (1989). Harison (1987) performed a number of laboratory tests including CBR, compaction, and DCP tests for different types of soils. According to Harison (1987), values of PI are a function of both moisture content and dry unit weight. Although generalized equations for the relationships were not proposed, certain correlations between the parameters were observed. Figure 2.4 shows the typical trend of PI with respect to values of dry unit weight and moisture content. In the figure, values of PI increase as the dry unit weight increases. This result appears to be reasonable since denser soils would result in higher penetration resistance.

Figure 2.4 (c) shows a trend of PI values with moisture contents corresponding to the compaction curve. As shown in the figure, the PI value decreases with increasing moisture contents up to the optimum moisture content (OMC) for a given compaction energy. This point corresponds to the maximum dry unit weight for a given compaction energy. After the OMC, PI values increase again with increasing moisture content. It should be noted that the values of PI in Figure 2.4 (c) were obtained for the soil states following the compaction curve. Also, although the same dry unit weight was considered, the PI value tends to be higher for higher moisture contents.

Table 2.1 Correlations between CBR and PI (after Harison 1987 and Gabr et al. 2000)

Author	Correlation	Field or laboratory based study	Material tested
Kleyn (1975)	log (CBR) = 2.62 - 1.27· log(PI)	Laboratory	Unknown
Harison (1987)	log (CBR) = 2.56 - 1.16· log(PI)	Laboratory	Cohesive
Harison (1987)	log (CBR) = 3.03 - 1.51· log(PI)	Laboratory	Granular
Livneh et al. (1994)	log (CBR) = 2.46 - 1.12· log(PI)	Field and laboratory	Granular and cohesive
Ese et al. (1994)	log (CBR) = 2.44 - 1.07· log(PI)	Field and laboratory	ABC*
NCDOT (1998)	log (CBR) = 2.60 - 1.07· log(PI)	Field and laboratory	ABC* and cohesive
Coonse (1999)	log (CBR) = 2.53 - 1.14· log(PI)	Laboratory	Piedomont residual soil
Gabr (2000)	log (CBR) = 1.40 – 0.55· log(PI)	Field and laboratory	ABC*

*Aggregate base course

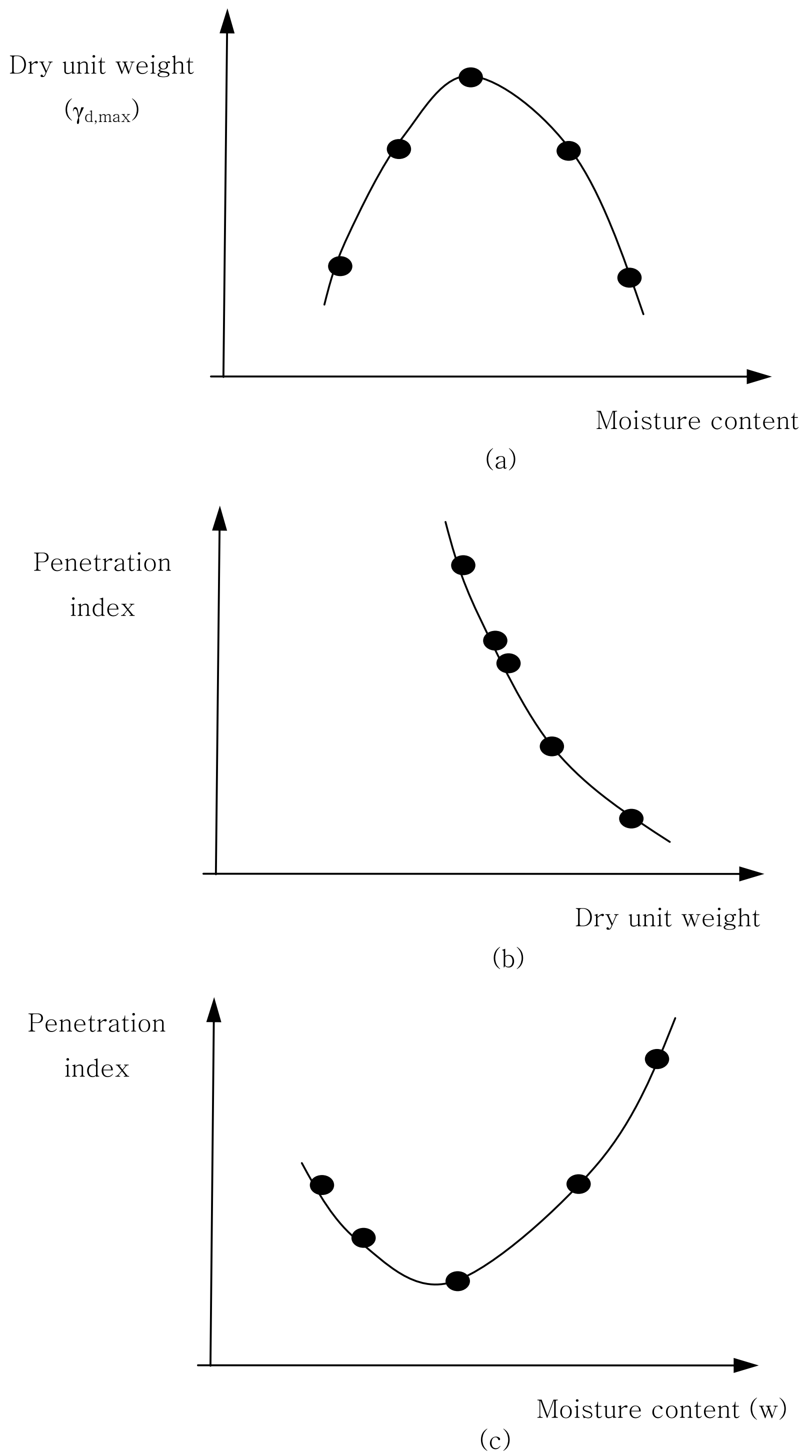


Figure 2.4 PI versus compaction parameters from laboratory results (after Harison 1987)

2.4 PI – Shear Strength Relationship

Ayers et al. (1989) proposed a correlation between values of PI and the shear strength of granular soils. The goal of the study was to evaluate the efficiency of the DCPT for estimating shear strength of granular material as a quick and economical in-situ testing approach. The work was done for soil samples obtained from a typical track section. Laboratory DCP and triaxial tests were performed to obtain PI and shear strength values, respectively. The test samples included sand, dense-graded sandy gravel, crushed dolomitic ballast, and ballast with varying amounts of non-plastic crushed dolomitic fines. Table 2.2 shows the basic properties of the tested materials.

Similarly to results by Harison (1987), it was observed that the values of PI decrease as the unit weight of soils increases. Based on a series of laboratory test results, Ayers (1989) developed correlations between the value of PI and the shear strength of soils**.** Table 2.3 shows the correlations between the PI and shear strength for the different materials and confining stress levels. It was also found that, for a given unit weight or relative density, the values of PI decrease as the confining stress increases. This indicates that the effect of confining stress on the penetration index of DCPT exists, which is consistent to findings by Livneh et al. (1994).

Table 2.2 Basic properties of test materials (after Ayers et al. 1989)

Material	G_S	C_u[1]	C_c[2]	Max. grain size (mm)	D_{10} (mm)	D_{30} (mm)	D_{60} (mm)
Sand	2.65	5.1	0.87	4.83	0.229	0.483	1.168
Sandy gravel	2.55	80.0	1.01	25.4	0.102	0.914	8.128
Crushed dolomitic ballast	2.63	1.7	0.99	38.1	18.03	23.11	29.97
Ballast with 7.5% NF[3]	2.63	3.0	1.67	38.1	9.906	22.09	29.46
Ballast with 15% NF[3]	2.63	9.2	5.22	38.1	3.048	21.08	27.94
Ballast with 22.5% NF[3]	2.62	15.1	8.41	38.1	1.778	20.07	26.92

[1]Cu: Coefficient of uniformity

[2]Cc: Coefficient of curvature

[3]NF: Non-plastic fines

Table 2.3 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(PI)
	206.9	DS* = 192.1 – 95.8(PI)

CHAPTER 3. DYNAMIC CONE PENETRATION TESTS ON SUBGRADE SOILS

3.1 Introduction

Field dynamic cone penetration tests (DCPT) were performed on subgrade soils at seven road construction sites. For each test site, the tests were conducted at several different locations. In order to measure in-situ soil densities and water contents, the nuclear gauge was used for each test location where the DCP tests were conducted. For a laboratory testing program, soil samples were obtained from the testing sites. A list of the laboratory tests performed in this study is as follows:

(1) grain size distribution tests;

(2) atterberg limit tests for cohesive soils;

(3) specific gravity tests;

(4) minimum and maximum density tests for granular soils;

(5) direct shear tests;

(6) unconfined compression tests for cohesive soils.

The laboratory testing program conducted in this study aims at characterizing the subgrade soils of the test sites as well as relating the measurement from the DCPT to various soil parameters. Table 3.1 shows a description of the test sites in which DCPTs were performed.

Table 3.1 Test sites for DCPT

Number	Location	Road	Station No.	Soil type
1	Hobart, IN	I-65	59+395	Clayey sand
2	Valpariso, IN	US 49	18+840, 18+846, 18+828 and 18+850	Well graded sand with clay
3	Gary, IN	I-80/I-94	342+000	Poorly graded sand
4	Knox, IN	US 35	2+150	Poorly graded sand
5	W. Lafayette, IN	Lindberg Road	1+189, 1+200, 1+211, 1+222, 1+233, 1+245, 1+256 and 1+269	Clayey sand
6	Lebanon, IN	I-65/County Road 100E	72+137	Clayey sand
7	Bainbridge	US36	10+505, 10+506, 10+722, 10+724 and 10+577	Clayey sand

3.2 Reconstruction Site of I-65 in Hobart, IN

Field DCP tests were performed on subgrade soils at the I-65 road construction site in Hobart, Indiana. Construction at the site was to rebuild the existing road and replace old pavement. Since the project did not include replacement of the subgrade soils, the tests were done on the existing subgrade soils exposed after removing the old pavement. Five DCP tests were conducted at several different locations around station 59+395. For each testing location, in-situ soil densities and moisture contents were also measured using the nuclear gauge at depths of 5.1 cm (2 in), 15.2 cm (6 in), and 30.5 cm (12 in) from the soil surface.

Table 3.2 and Figure 3.1 show in-situ total and dry soil densities and moisture contents measured from the nuclear gauge. DCPT logs are shown in Figure 3.2 through Figure 3.6.

The laboratory tests were performed to characterize the soils of test site. A sieve analysis and Atterberg limit test were conducted. The soil's specific gravity (G_S) was determined to be 2.71. Figure 3.7 shows the particle size distribution from the result of sieve analysis. The liquid limit (LL) and plastic limit (PL) are 23.3 and 17.2 respectively. The plastic index (IP) is 6.1. The soil is a clayey sand (SC).

The relationships of dry density, moisture content and the penetration index (PI) are shown in Figure 3.8 and Figure 3.9 respectively.

Unconfined compression tests were conducted in the laboratory on a sample with similar dry density and moisture content to those tested to those tested in the field. A PI value for a corresponding dry unit weight can be obtained from the results of the field

DCPT. According to the results of Lee (1997), the relationship between resilient modulus (M_r) and stress in psi at 1% axial strain in an unconfined compressive test is as follows,

$$M_r = 695.4\ (s_u)_{1.0\%} - 5.93\ [(s_u)_{1.0\%}]^2$$

The M_r can be estimated from $(s_u)_{1.0\%}$ using this equation. Table 3.3 shows the results of the unconfined compression test and the corresponding penetration index for a given moisture content and dry density.

Table 3.2 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-65 in Hobart, IN

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
1	5.1	12.2	22.1	19.7
	15.2	14.6	21.2	18.5
	30.5	13.6	21.9	19.3
	Average	13.5	21.7	19.1
2	5.1	9.5	22.8	20.8
	15.2	9.8	22.6	20.6
	30.5	9.3	22.6	20.7
	Average	9.5	22.7	20.7
3	5.1	12.4	21.7	19.3
	15.2	11.7	21.4	19.2
	30.5	11.3	21.9	19.7
	Average	11.8	21.7	19.4
4	5.1	10.5	22.3	20.2
	15.2	10.2	22.4	20.3
	30.5	9.8	22.5	20.5
	Average	10.2	22.4	20.3
5	5.1	10.6	22.3	19.8
	15.2	10.5	21.9	19.8
	30.5	10.1	21.8	20.2
	Average	10.4	22.0	19.9

Table 3.3 Result of Unconfined Compressive Test and corresponding Penetration Index from field DCPT for the site of I-65 in Hobart, IN

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)
18.4	205.6	55.89	36180.0	10.2
19.0	598.3	274.7	126139.8	10.2
22.0	332.8	269.8	125027.1	5.1

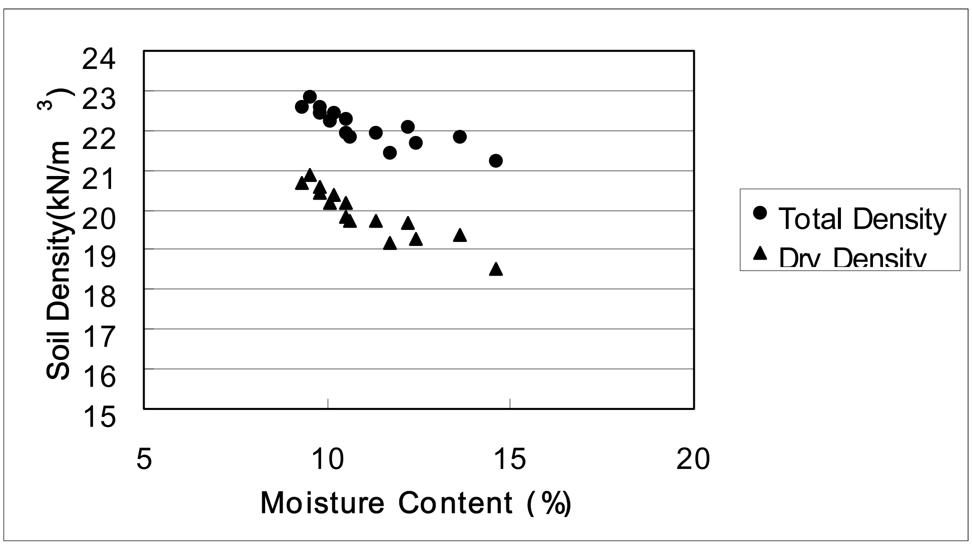


Figure 3.1 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-65 in Hobart, IN

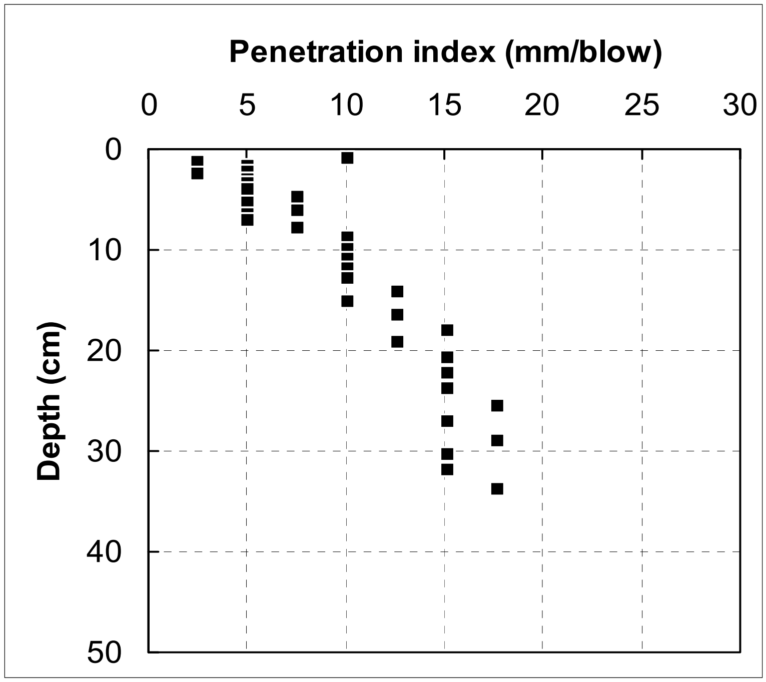


Figure 3.2 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 1)

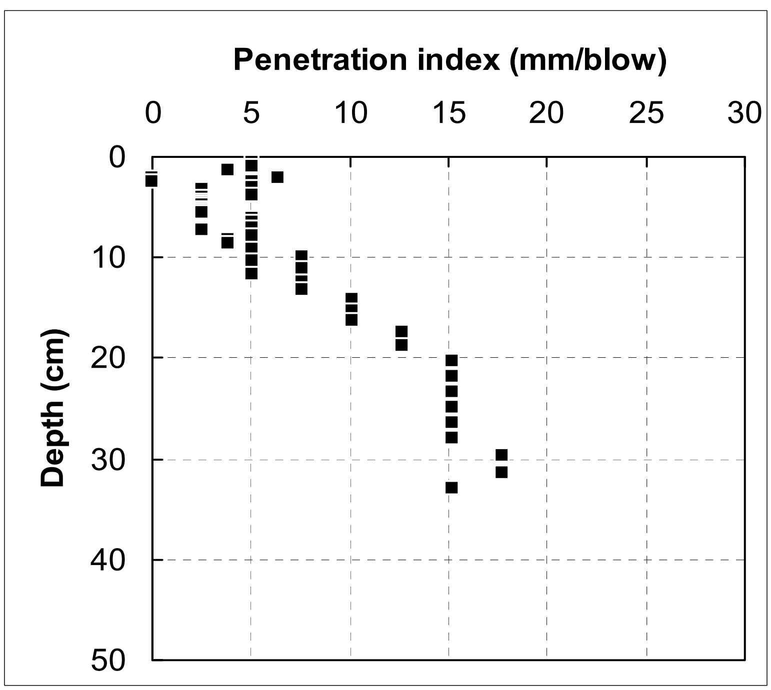


Figure 3.3 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 2)

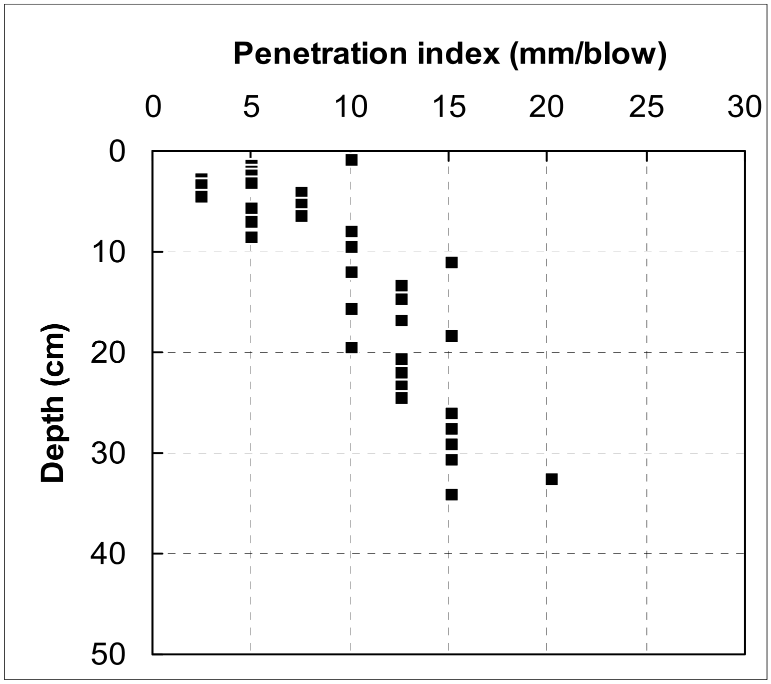


Figure 3.4 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 3)

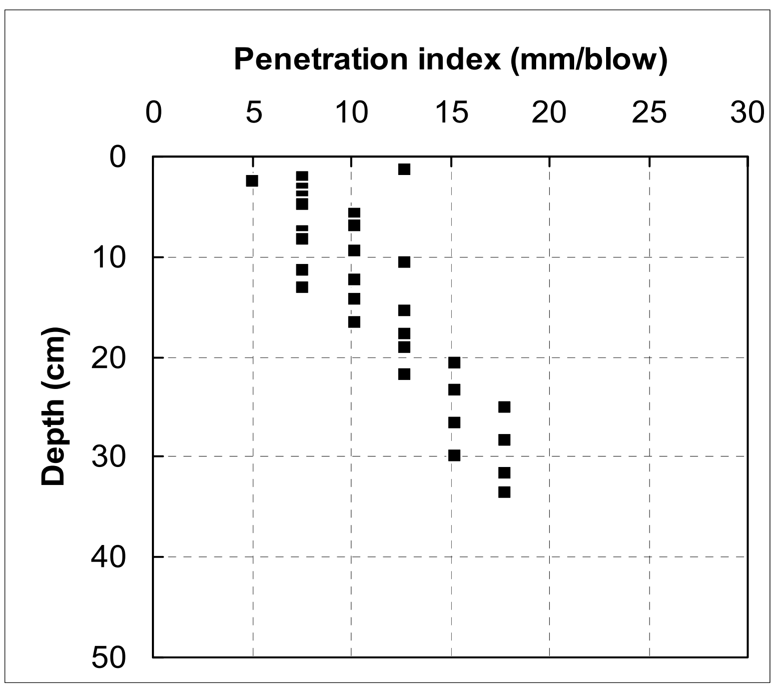


Figure 3.5 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 4)

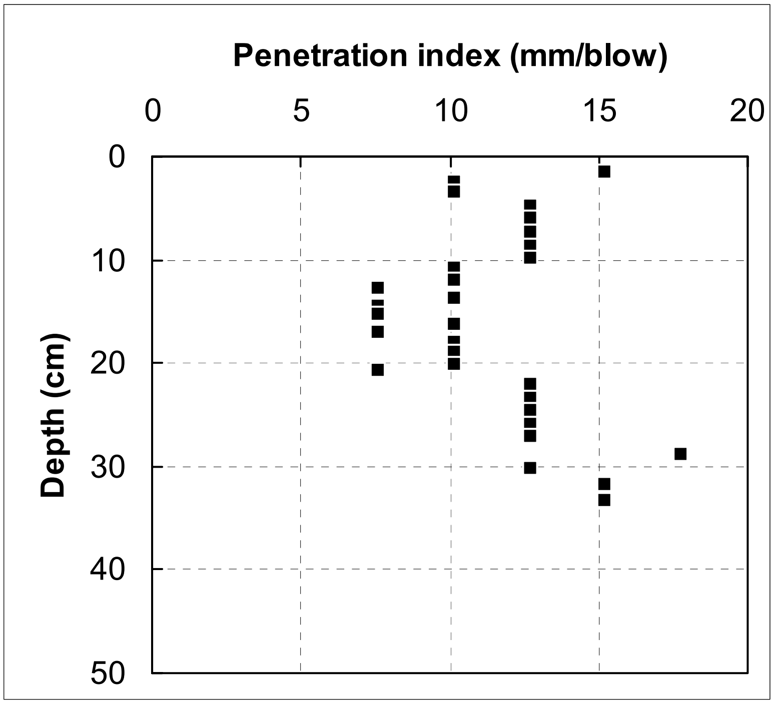


Figure 3.6 Log of DCPT for the site of I-65 in Hobart, IN (Station: 59+395, Test No. 5)

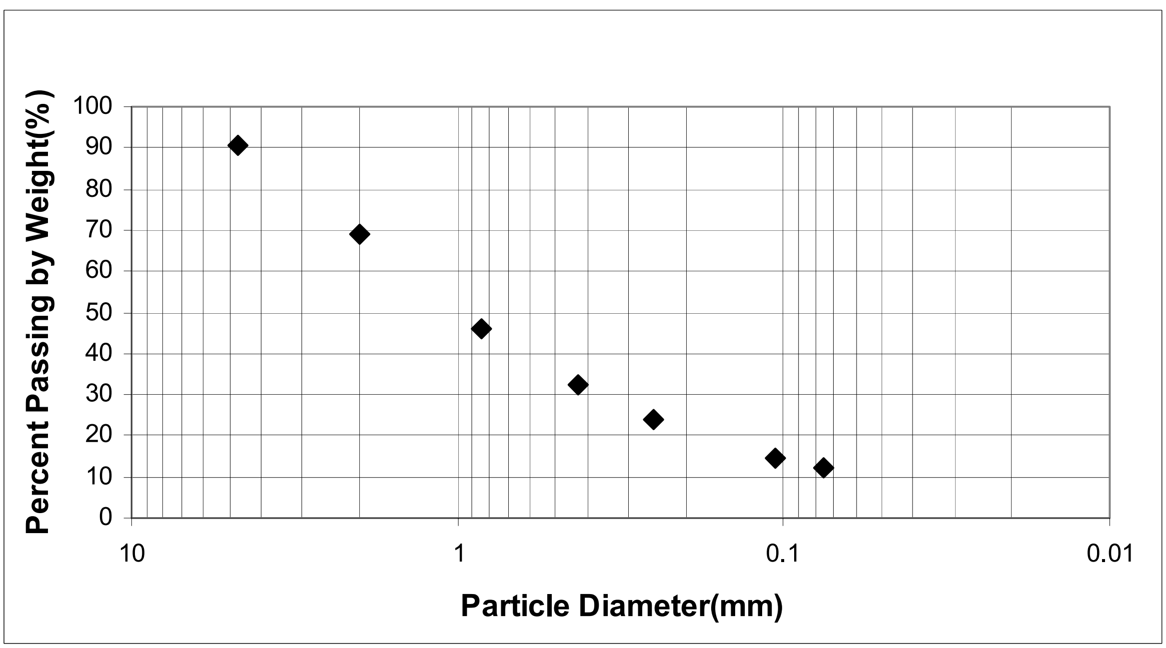


Figure 3.7 Particle size distribution for the site of I-65 in Hobart, IN

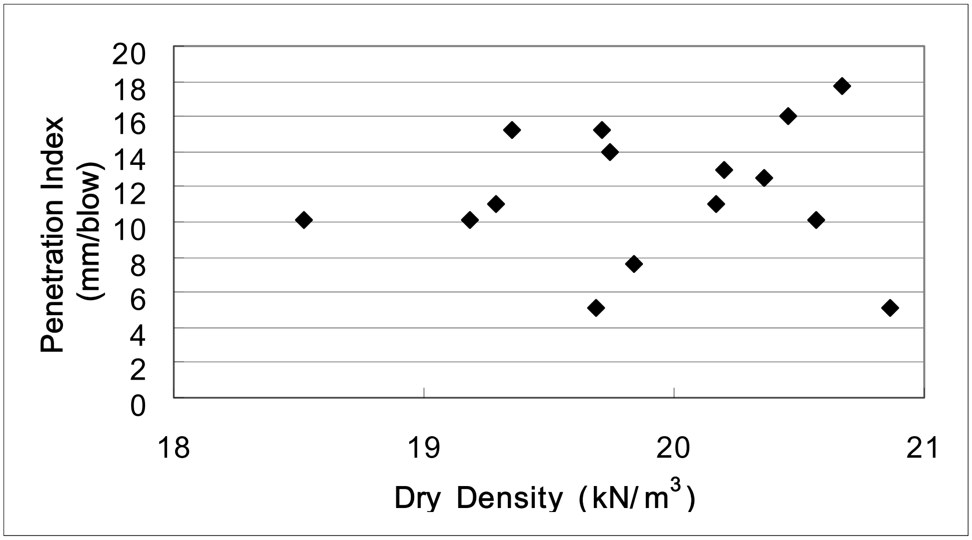


Figure 3.8 Relationship between Dry Density and Penetration Index from field DCPT for the site of I-65 in Hobart, IN

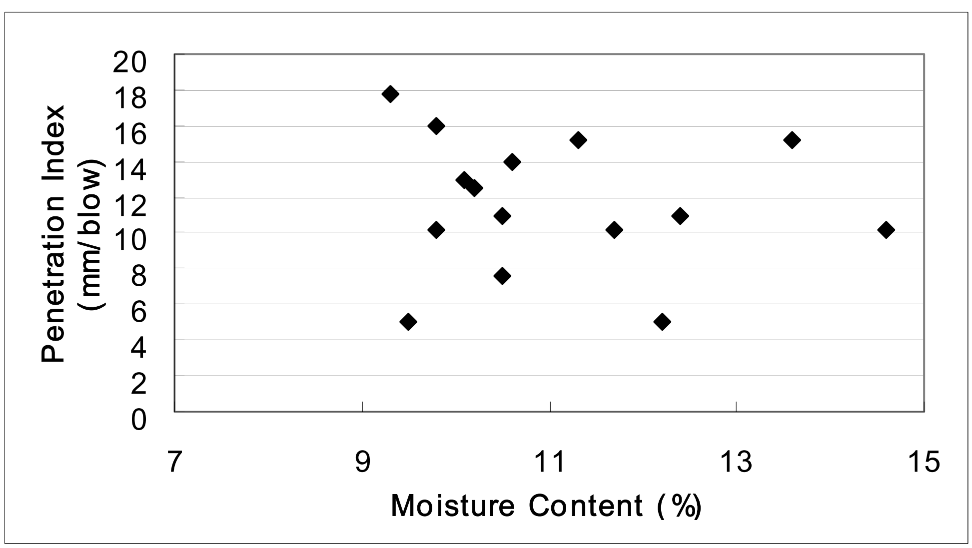


Figure 3.9 The Relationship between Moisture Content and Penetration Index from field DCPT for the site of I-65 in Hobart, IN

3.3 Reconstruction Site of US49 in Valpariso, IN

Field DCP Tests were performed on subgrade soils at a US49 road construction site in Valpariso, Indiana. Construction at the site was to rebuild the existing road and replace old pavement. The subgrade soil was compacted, since it was covered by the old US49 road. The tests were conducted on the existing subgrade soil exposed after removing the old pavement. Four DCP tests were performed at different locations (Station 18+850, 18+840, 18+846 and 18+828). For each testing location, in-situ soil densities and moisture contents were measured with a nuclear gauge at the same location as the DCPT. The values were evaluated at the depths of 5.1 cm (2 in), 15.2 cm (6 in), and 30.5 cm (12 in) from the soil surface. Table 3.4 and Figure 3.10 show in-situ total and dry soil densities and moisture contents measured from the nuclear gauge. The DCPT logs are shown in Figure 3.11 through Figure 3.14.

To characterize the soils of the test site, the laboratory tests were conducted. A sieve analysis and Atterberg limit test were performed. The liquid limit (LL) and plastic limit (PL) are 24.1 and 16.4 respectively. The plastic index (IP) is 7.7. The particle size distribution from the result of the sieve analysis is shown in Figure 3.15. The coefficient of curvature (Cc) and uniformity (Cu) are 1.28 and 11.0 respectively. The specific gravity is 2.65. The soil is a well graded sand with clay (SW-SC).

The relationships between dry density, moisture content and the penetration index (PI) are shown in Figure 3.16 and Figure 3.17 respectively.

To correlate the penetration index and soil strength, unconfined compression tests were conducted in the laboratory. The samples were prepared with similar dry density and moisture content to those measured in the field. The measured value of unconfined compressive strength, s_u at 1% strain and resilient modulus calculated using Lee's equation (1997) were obtained. From the result of field DCPT, the corresponding PI values with similar dry unit weight were obtained. The results of unconfined compression tests are shown in Table 3.5.

Table 3.4 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US49 in Valparaiso, IN

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
1	5.1	11.8	20.1	18.0
	15.2	11.4	20.8	18.7
	30.5	10.7	21.2	19.2
	Average	11.3	20.7	18.6
2	5.1	10.8	20.5	18.5
	15.2	10.6	21.1	19.1
	30.5	10.2	21.6	19.5
	Average	10.5	21.1	19.0
3	5.1	12.1	21.1	18.8
	15.2	12.6	21.3	18.9
	30.5	12.3	21.5	19.2
	Average	12.3	21.3	18.9
4	5.1	9.3	16.6	15.2
	15.2	8.5	18.6	17.2
	30.5	7.5	19.6	18.2
	Average	8.4	18.3	16.9

Table 3.5 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of US49 in Valpariso, IN

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	261.0	75.5	47624.0	20.3
19.0	487.7	198.4	104103.8	10.2
17.1	206.2	113.7	67936.1	15.0

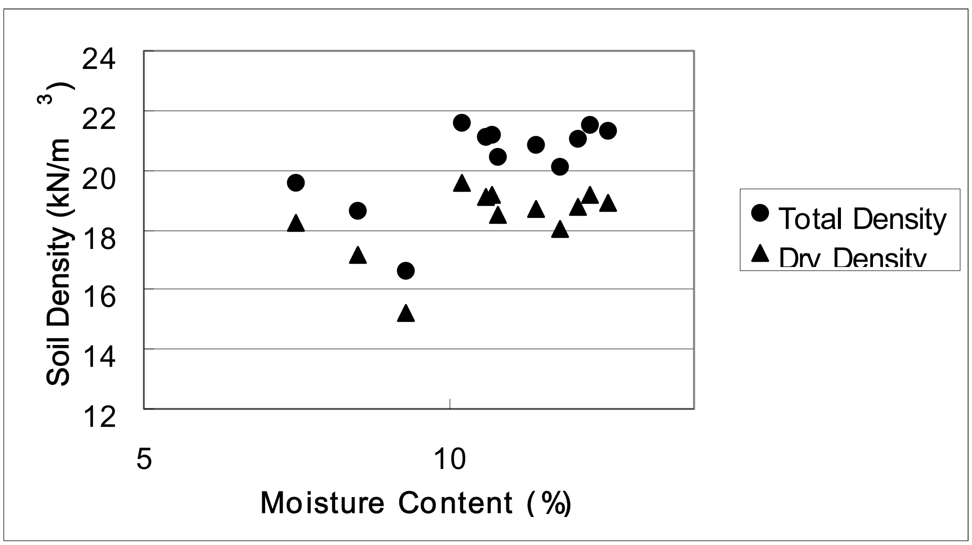


Figure 3.10 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US49 in Valpariso, IN

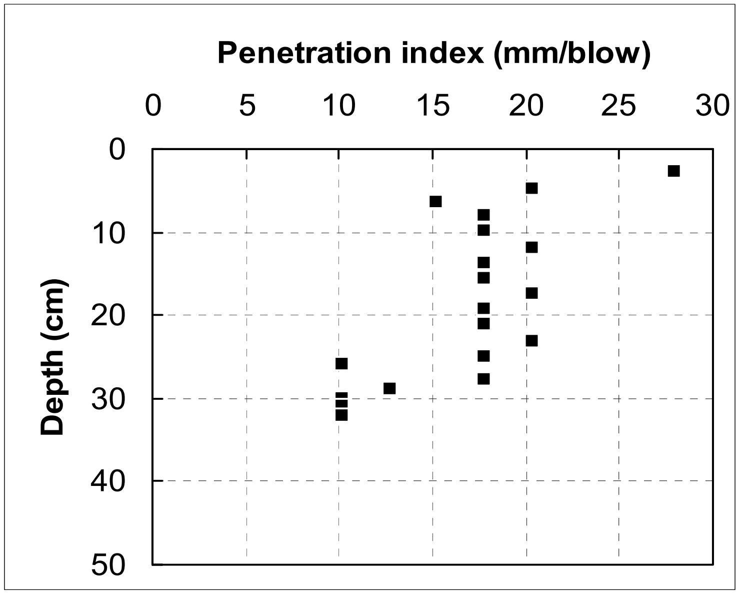


Figure 3.11 Log of DCPT for the site of US49 in Valpariso, IN (Station: 18+850, Test No. 1)

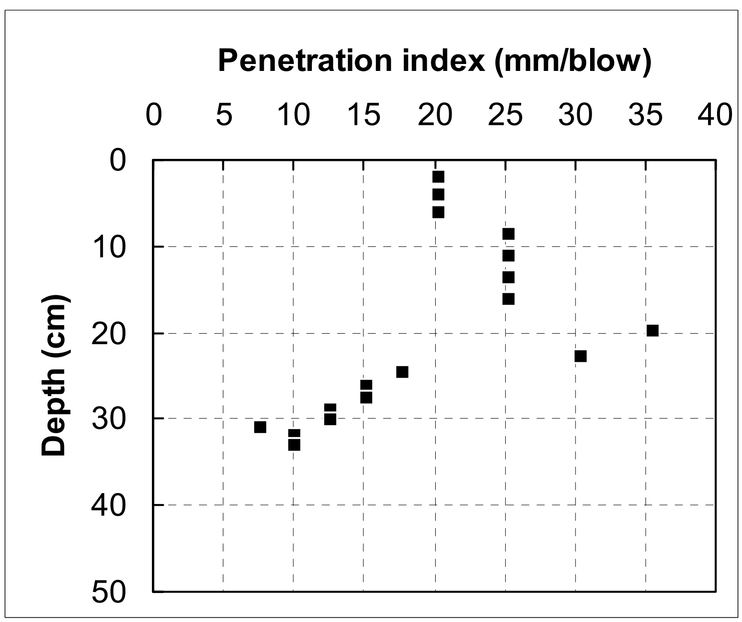


Figure 3.12 Log of DCPT for the site of US49 in Valpariso, IN (Station: 18+840, Test No. 2)

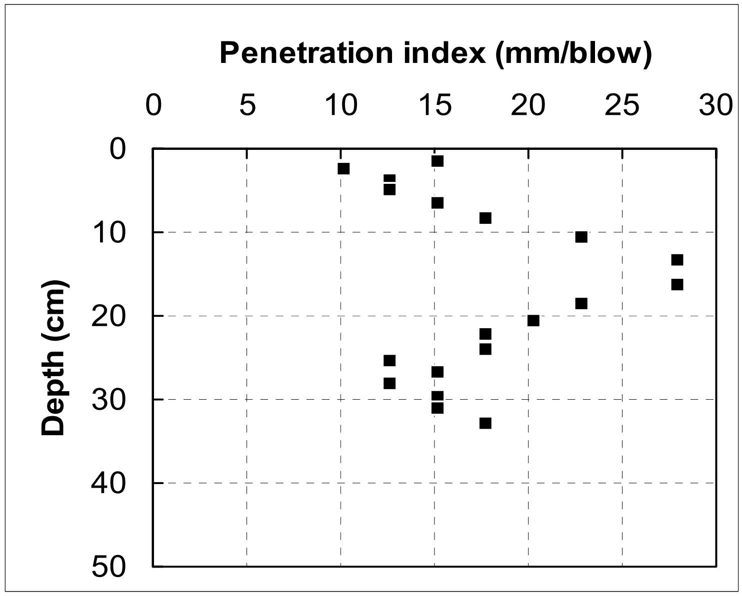


Figure 3.13 Log of DCPT for the site of US49 in Valparaiso, IN (Station: 18+846, Test No. 3)

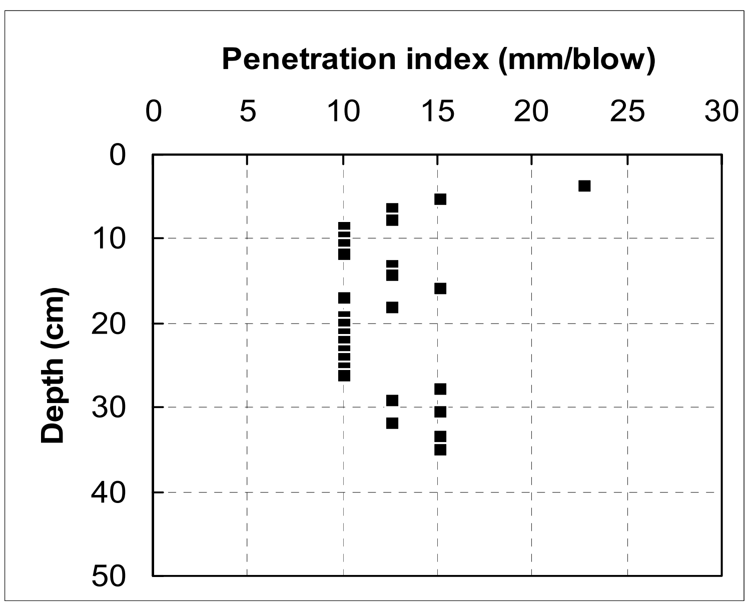


Figure 3.14 Log of DCPT for the site of US49 in Valparaiso, IN (Station: 18+828, Test No. 4)

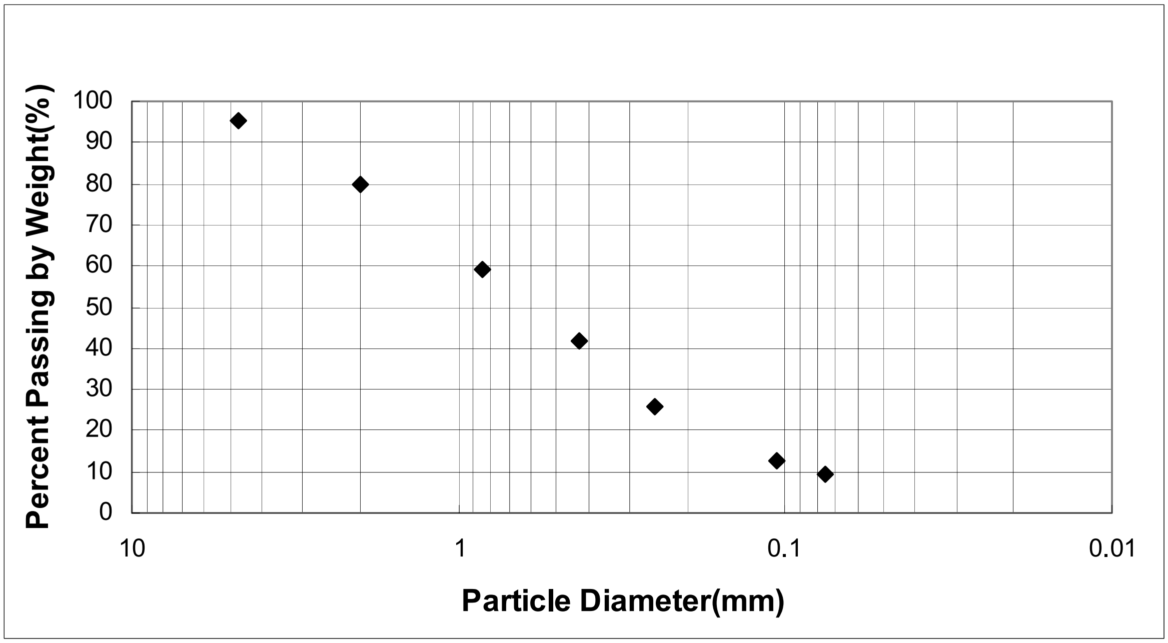


Figure 3.15 Particle size distribution for the site of US49 in Valpariso, IN

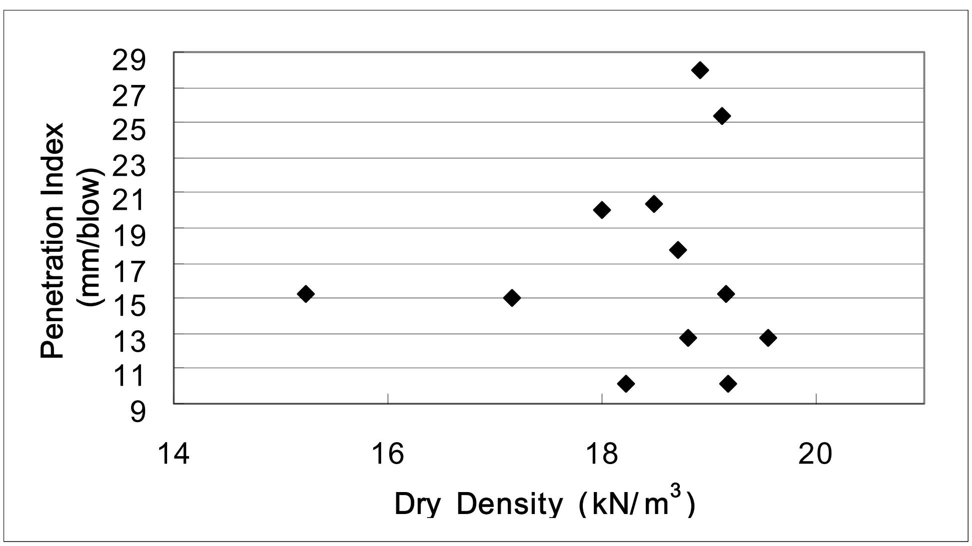


Figure 3.16 Relationship between Dry Density and Penetration Index from field DCPT for the site of US49 in Valpariso, IN

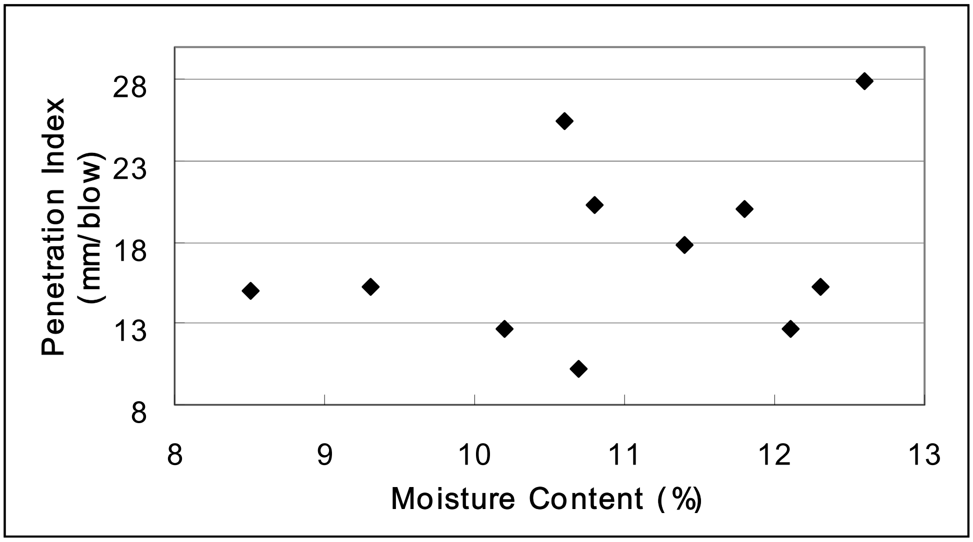


Figure 3.17 Relationship between Moisture Content and Penetration Index from field DCPT for the site of US49 in Valpariso, IN

3.4 Reconstruction Site of I-80/I-94 in Gary, IN

Field DCP Tests were performed on subgrade soils at an I-80/I94 road construction site in Gary, Indiana. Construction at the site was to rebuild the existing road and replace old pavement. Therefore, the subgrade soils were compacted. Five DCP tests were performed at different locations around station 342+000. In-situ soil densities and moisture contents were measured with a nuclear gauge at the same location as the DCPT. The values were evaluated at the depths of 5.1 cm (2 in), 15.2 cm (6 in), and 30.5 cm (12 in) from the soil surface.

Table 3.6 and Figure 3.18 show in-situ total and dry soil densities and moisture contents measured by the nuclear gauge. The DCPT logs are shown in the Figure 3.19 through Figure 3.23.

To characterize the tested soil, a sieve analysis, specific gravity and minimum and maximum density tests were conducted in laboratory. The result of the sieve analysis is shown in Figure 3.24. The coefficient of curvature (Cc) and uniformity (Cu) are 1.5 and 1.67 respectively. The soil is classified as a poorly graded sand (SP). The specific gravity is 2.65. The relative density (Dr) is commonly used to indicate the in- situ denseness or looseness of granular soil. From the laboratory tests, the minimum dry density, with an e_{max} of 0.88, is 13.8 kN/m^3 and the maximum dry density, with an e_{min} of 0.56, is 16.7 kN/m^3. The tube method was used for the minimum dry density test. The average dry density of the site is 16.6 kN/m^3. From these results, the Dr value is 98%. The soils of the site were well compacted.

Figure 3.25 and Figure 3.26 show the relationship between dry density, moisture content and the penetration index (PI) respectively.

Direct shear tests were performed in the laboratory corresponding to the field DCP tests No. 3,4, and 5. The samples were prepared with the same average moisture content and dry unit weight for each test location. The results of direct shear tests are shown in Table 3.7 and Figure 3.27. The contours of the relationship between PI and shear strength with different normal stress is shown in Figure 3.28.

Table 3.6 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-80/I-94 in Gary, IN

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
1	5.1	15.0	17.6	15.4
	15.2	13.6	18.6	16.4
	30.5	11.7	18.9	17.0
	Average	13.4	18.4	16.2
2	5.1	15.2	18.1	15.8
	15.2	14.6	19.6	17.1
	30.5	13.2	19.4	17.2
	Average	14.3	19.0	16.7
3	5.1	15.6	17.9	15.5
	15.2	15.4	18.6	16.1
	30.5	15.8	19.2	16.6
	Average	15.3	18.5	16.1
4	5.1	14.8	19.0	16.6
	15.2	13.3	19.4	17.1
	30.5	14.1	19.0	16.6
	Average	14.0	19.1	16.8
5	5.1	7.1	18.0	16.8
	15.2	7.1	18.6	17.3
	30.5	6.5	18.6	17.5
	Average	6.9	18.4	17.2

Table 3.7 Result of Direct Shear Test with different normal stress for the site of I-80/I94 in Gary, IN

Dry unit weight (kN/m^3)	Moisture Content (%)	Friction Angle (Φ°)	Corresponding Penetration Index (mm/blow)	Shear Strength (kN/m^2)		
				Normal stress (32.4 kN/m^2)	Normal stress (95.2 kN/m^2)	Normal stress (189.0 kN/m^2)
16.8	14.1	37.7	11.66	29.6	85.3	151.3
17.2	6.9	36.2	20.8	28.2	75.7	144.5
16.1	15.6	36.6	15.1	25.7	71.5	140.3

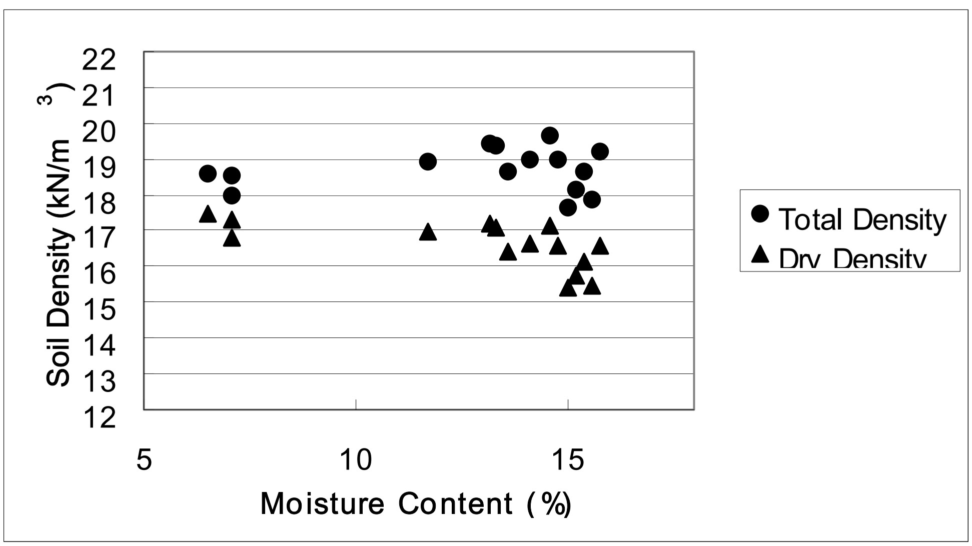


Figure 3.18 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I-80/I-94 in Gary, IN

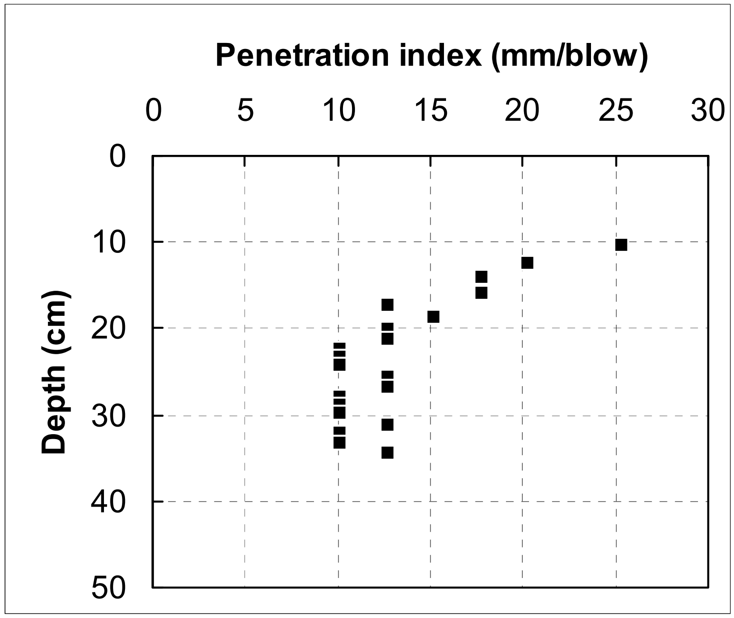


Figure 3.19 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 1)

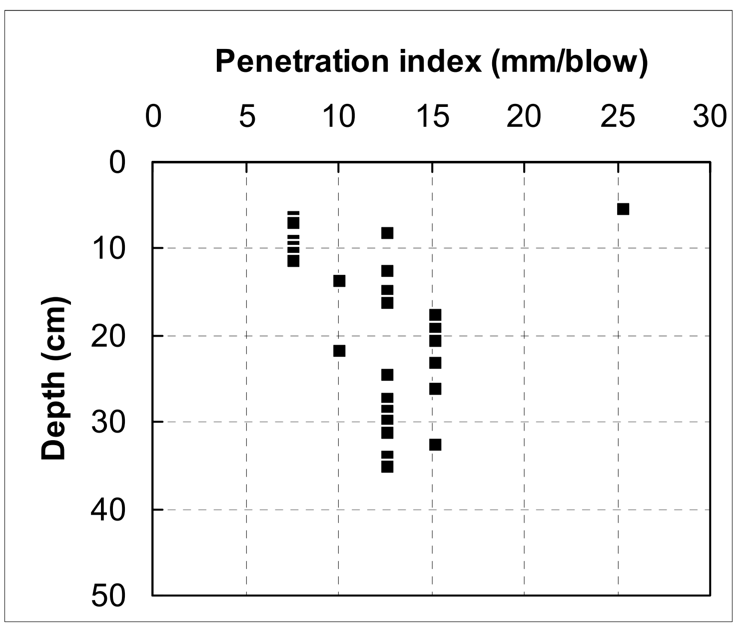


Figure 3.20 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 2)

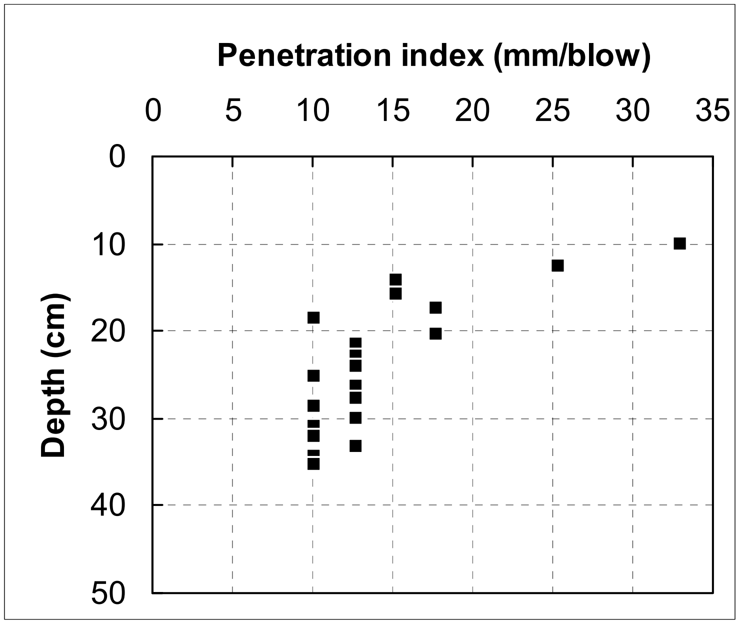


Figure 3.21 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 3)

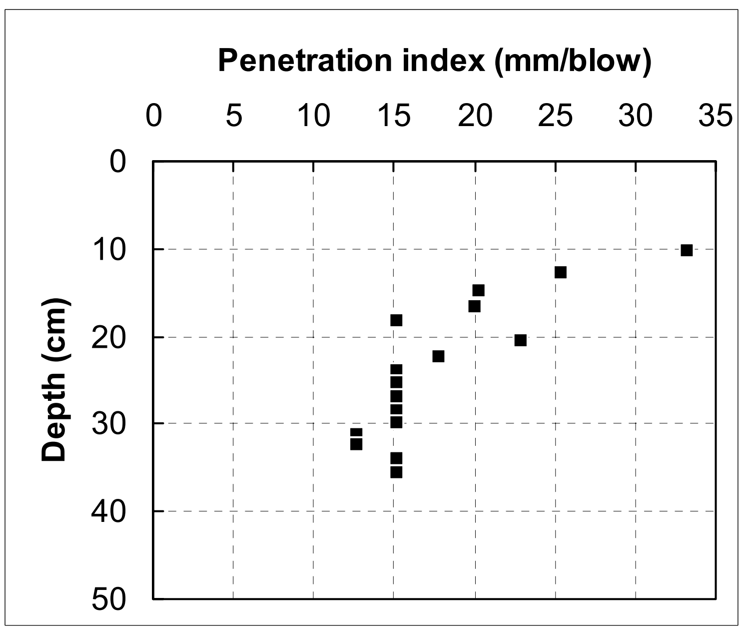


Figure 3.22 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 4)

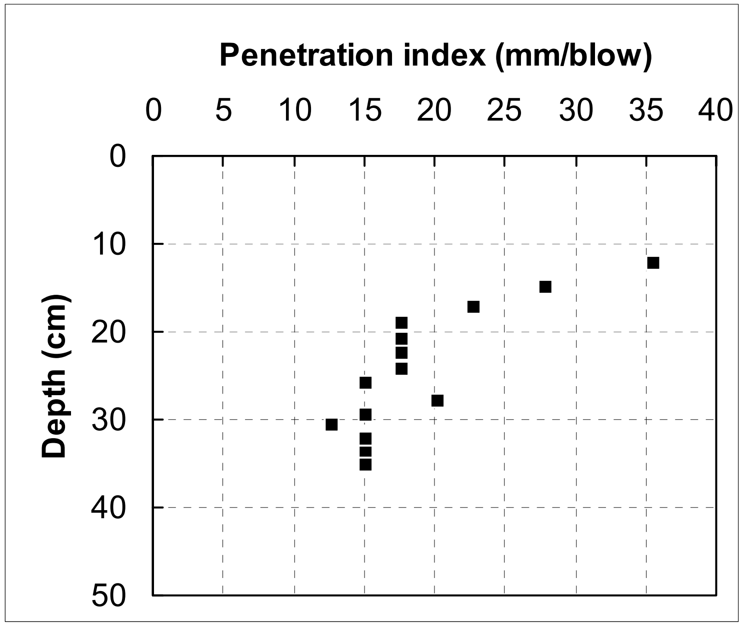


Figure 3.23 Log of DCPT for the site of I-80/I-94 in Gary, IN (Station: 342+000, Test No. 5)

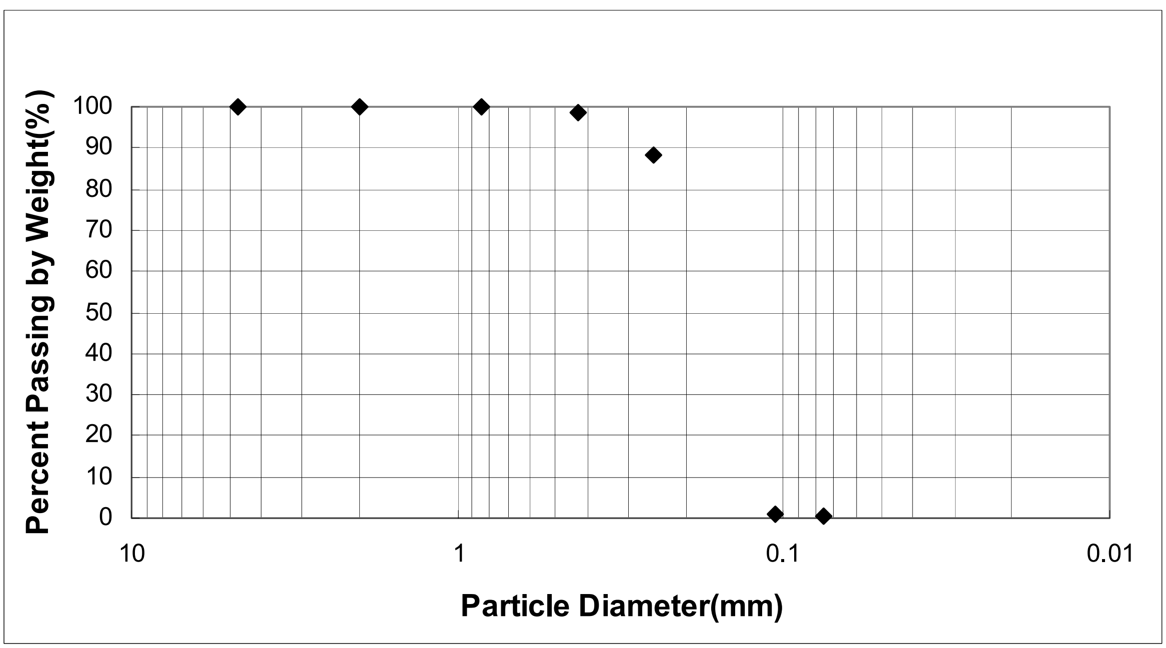


Figure 3.24 Particle size distribution for the site of I-80/I-94 in Gary, IN

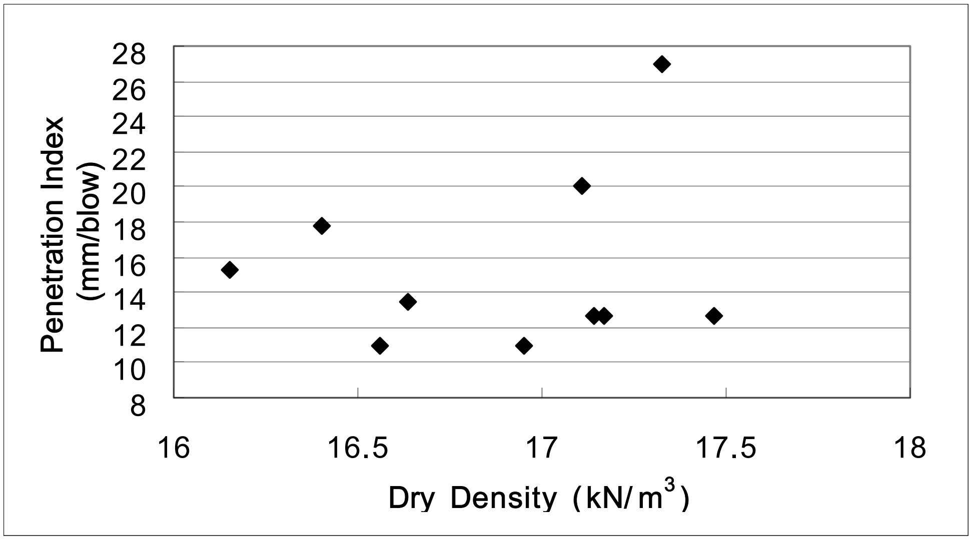


Figure 3.25 Relationship between Dry Density and Penetration Index from field DCPT for the site of I-80/I-94 in Gary, IN

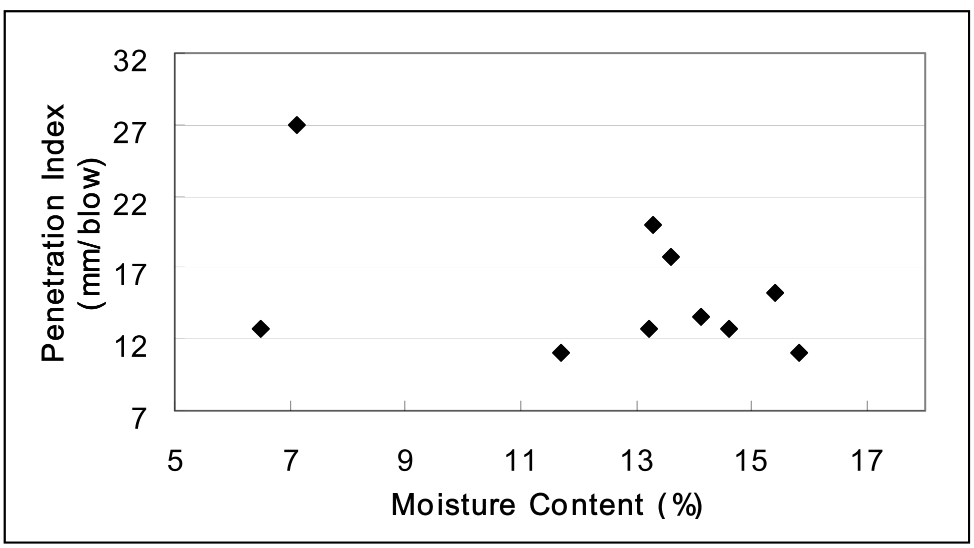


Figure 3.26 Relationship between Moisture Content and Penetration Index from field DCPT for the site of I-80/I-94 in Gary, IN

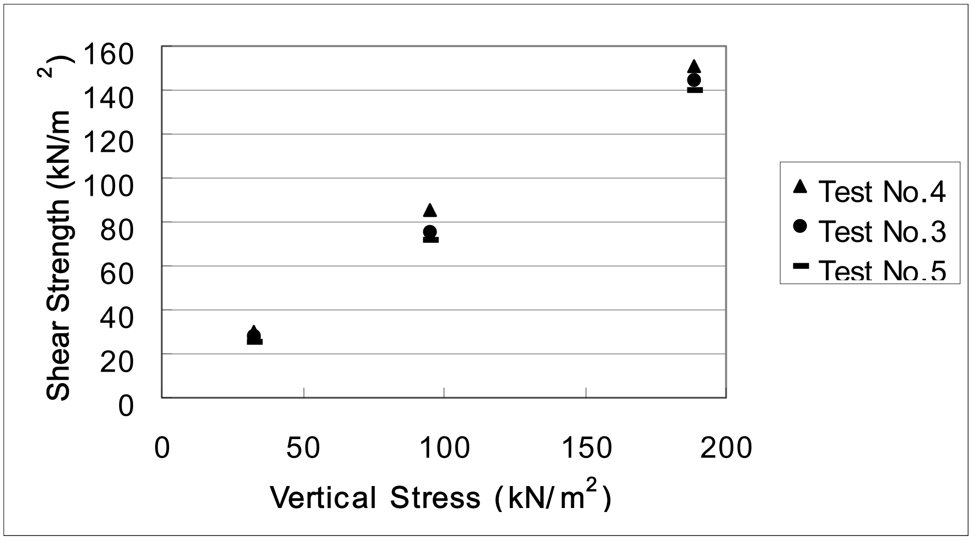


Figure 3.27 Result of Direct Shear Test with different normal stress for the site of I-80/I-94 in Gary, IN

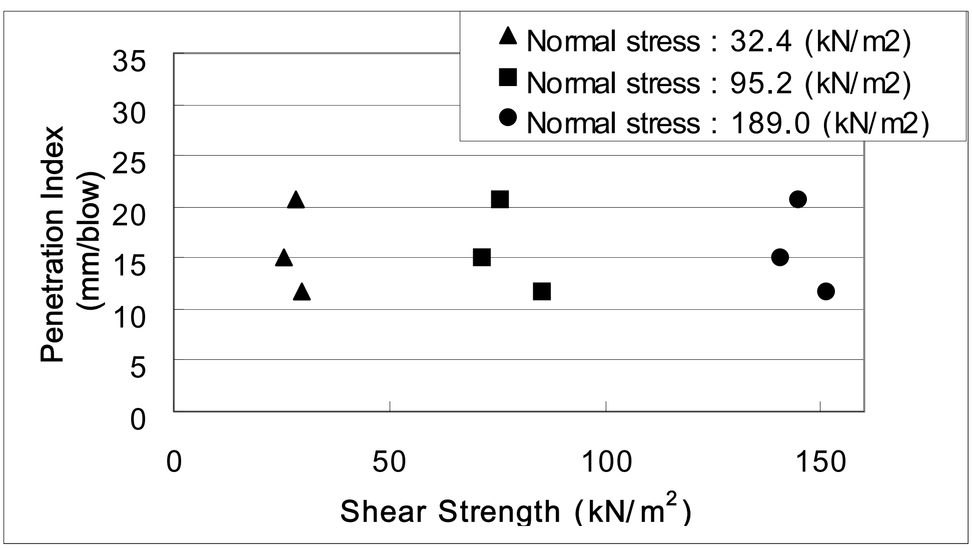


Figure 3.28 Relationship between PI and Shear Strength with different normal stress for the site of I-80/I-94 in Gary, IN

3.5 Road Widening Construction Site of US35 in Knox, IN

Field DCP Tests were performed on subgrade soils at a US35 road widening construction site in Knox, Indiana. Construction at the site was to rebuild the existing road and replace old pavement. The tests were conducted on the existing subgrade soils exposed after removing the old pavement. The subgrade soils were compacted. Five DCP tests were performed at several different locations around station 2+150. Also in-situ soil densities and moisture contents were measured using a nuclear gauge at depths of 5.1 cm (2 in), 15.2 cm (6 in), and 30.5 cm (12 in) from the soil surface. In-situ total and dry soil densities and moisture contents measured from the nuclear gauge are shown in Table 3.8 and Figure 3.29. The DCPT logs are shown in Figure 3.30 through Figure 3.34.

Sieve analysis, specific gravity and minimum and maximum density tests were performed to characterize the tested soil. Figure 3.35 shows the result of the sieve analysis. The coefficient of curvature (Cc) and uniformity (Cu) are 1.26 and 2.67 respectively. The soil is a poorly graded sand (SP). The specific gravity is 2.64. The minimum dry density is 13.9 kN/m^3 with an e_{max} of 0.86 and the maximum dry density is 17.3.7 kN/m3 with an e_{min} of 0.50. The tube method was used for the minimum dry density test. The average dry density of the site is 17.18 kN/m^3. From these results the relative density (Dr) is 98%. The soils of the site were well compacted.

The relationship between the dry density, moisture contents and the penetration index (PI) are shown in Figure 3.36 and Figure 3.37, respectively.

Direct shear tests were performed in the laboratory corresponding to the field DCP tests Nos. 2,3 and 5. The samples were prepared with the same average moisture content and average dry unit weight for each test location. Table 3.9 and Figure 3.38 show the result of direct shear tests. The relationship between PI and shear strength with different normal stresses is shown in Figure 3.39.

Table 3.8 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US35 in Knox, IN

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
1	5.1	4.7	18.0	17.2
	15.2	4.2	17.7	16.9
	30.5	4.0	17.9	17.2
	Average	4.3	17.9	17.1
2	5.1	6.7	17.5	16.4
	15.2	6.0	19.5	18.4
	30.5	5.9	19.9	18.8
	Average	6.2	19.0	17.8
3	5.1	8.5	18.9	17.4
	15.2	7.3	19.7	18.3
	30.5	7.5	19.7	18.3
	Average	7.7	19.4	18.0
4	5.1	13.2	19.2	17.0
	15.2	13.2	19.5	17.2
	30.5	12.3	19.3	17.2
	Average	12.9	19.3	17.1
5	5.1	10.8	18.1	16.3
	15.2	11.1	17.4	15.7
	30.5	11.7	17.0	15.2
	Average	11.2	17.5	15.7

Table 3.9 Result of Direct Shear Test with different normal stress for the site of US35 in Knox, IN

Dry unit weight (kN/m^3)	Moisture Content (%)	Friction Angle (Φ°)	Corresponding Penetration Index (mm/blow)	Shear Strength (kN/m^2)		
				Normal stress (32.4 kN/m^2)	Normal stress (95.2 kN/m^2)	Normal stress (189.0 kN/m^2)
17.9	6.2	34.2	18.2	28.1	70.1	134.5
18.0	7.8	37.8	50.3	28.8	73.8	149.8
15.7	11.2	33.5	25.1	21.9	68.3	126.2

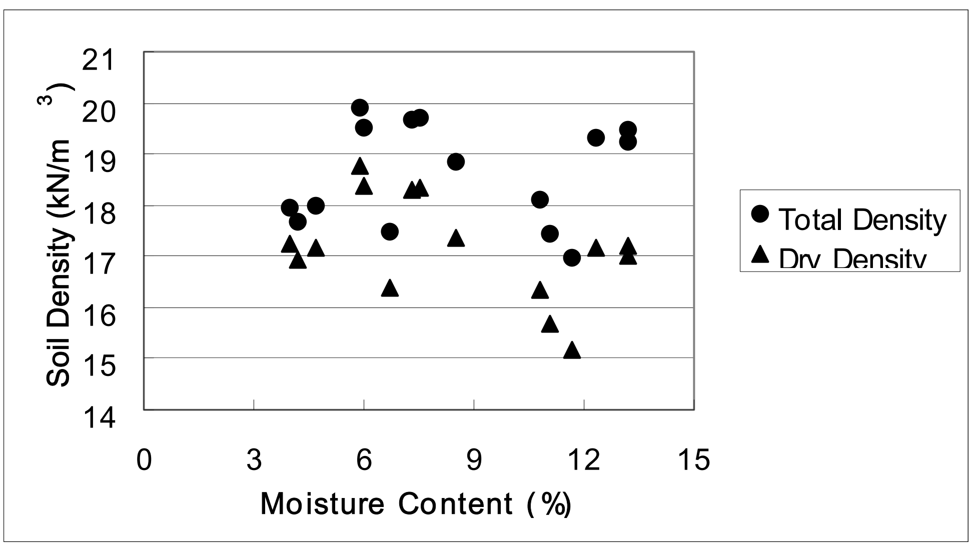


Figure 3.29 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US35 in Knox, IN

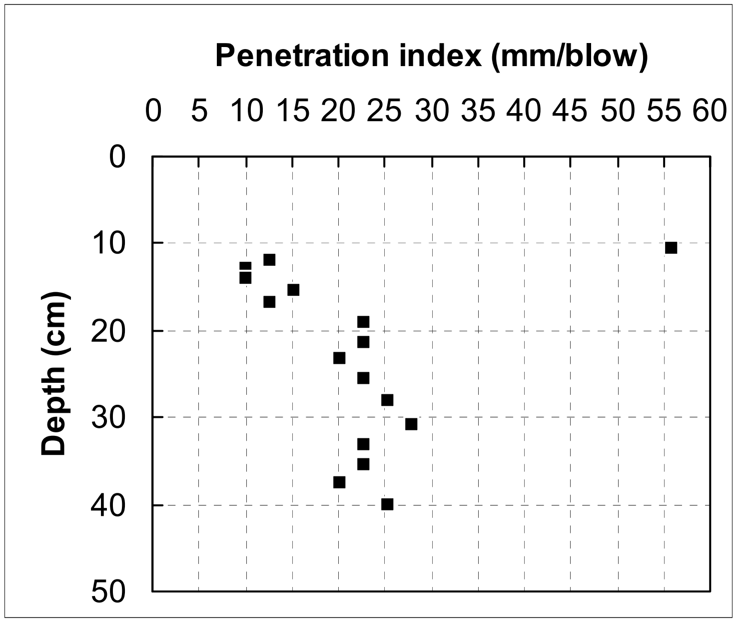


Figure 3.30 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 1)

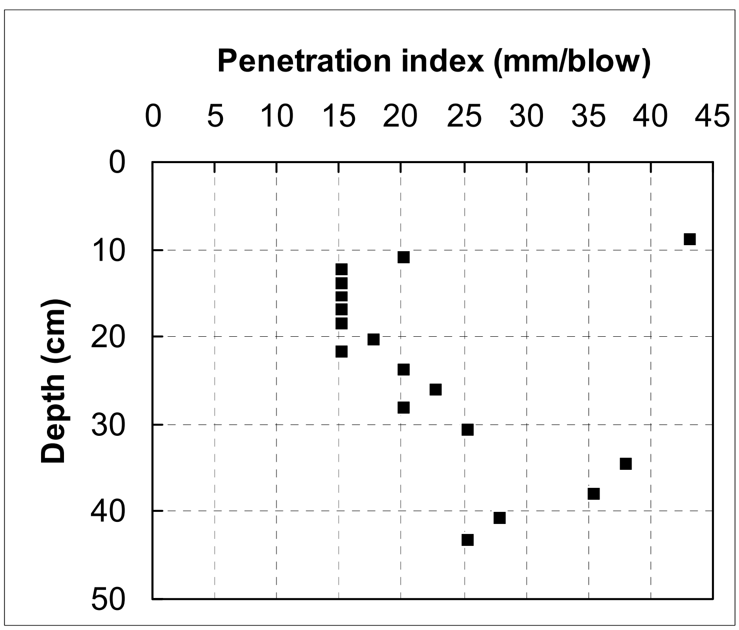


Figure 3.31 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 2)

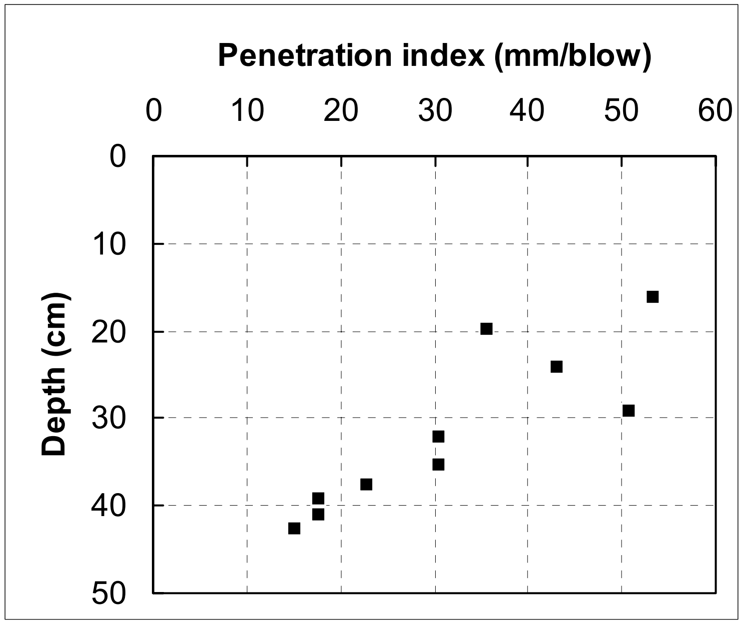


Figure 3.32 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 3)

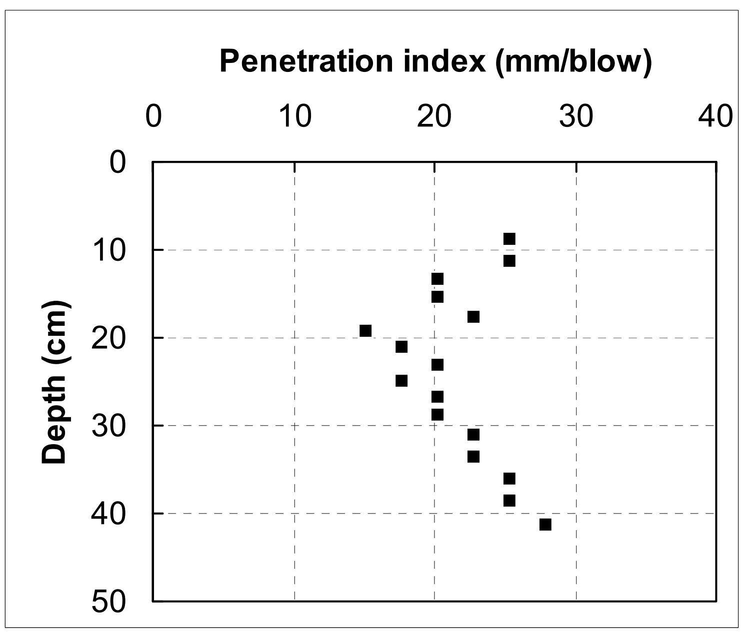


Figure 3.33 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 4)

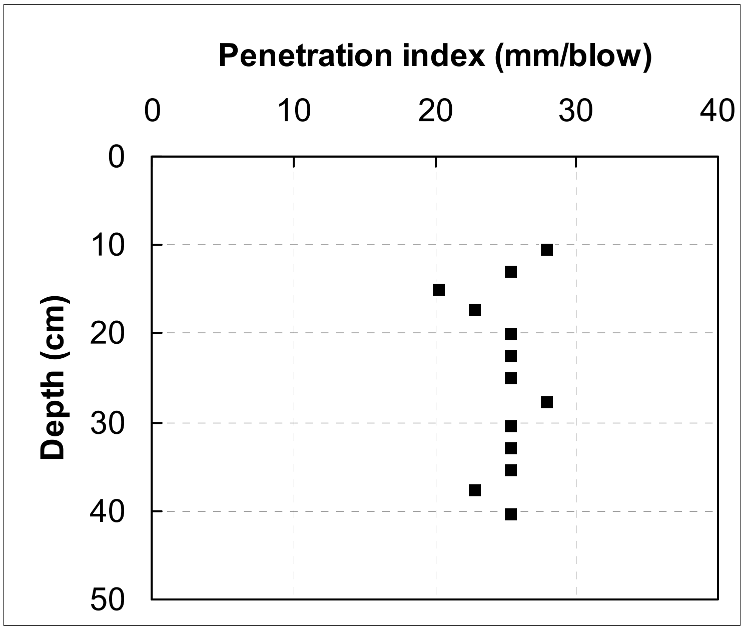


Figure 3.34 Log of DCPT for the site of US35 in Knox, IN (Station: 2+150, Test No. 5)

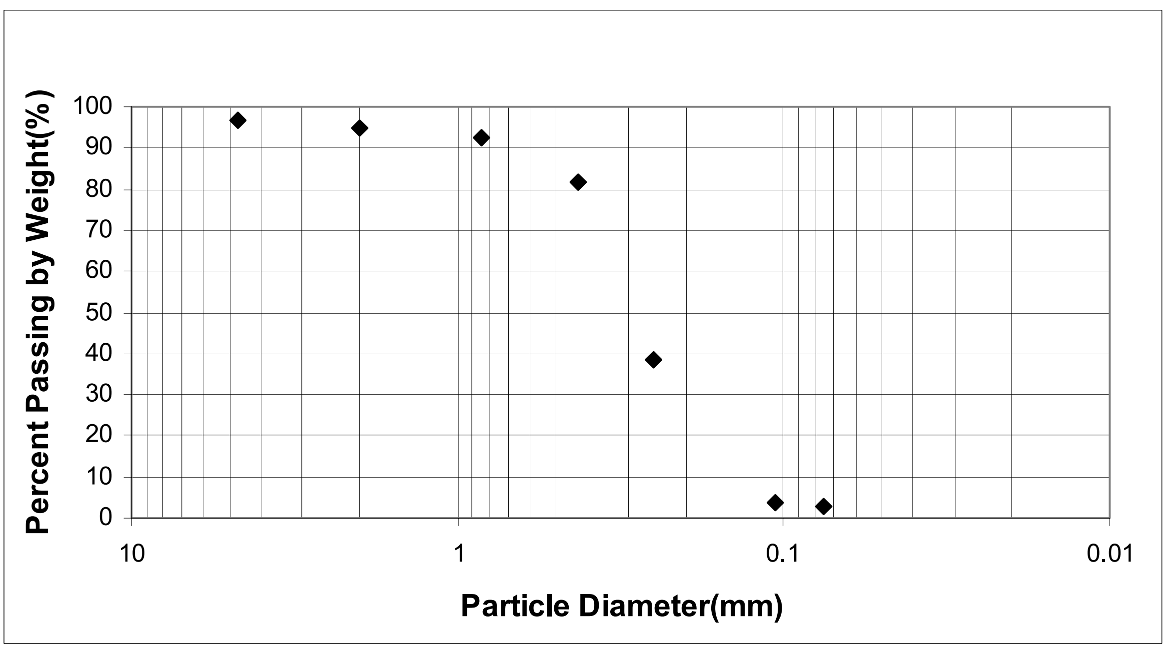


Figure 3.35 Particle size distribution for the site of US35 in Knox, IN

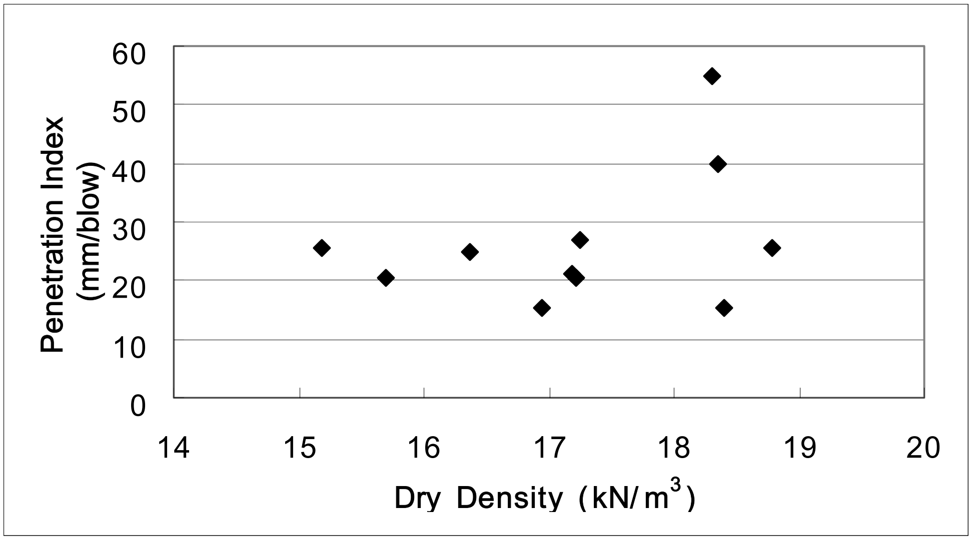


Figure 3.36 Relationship between Dry Density and Penetration Index from field DCPT for the site of US35 in Knox, IN

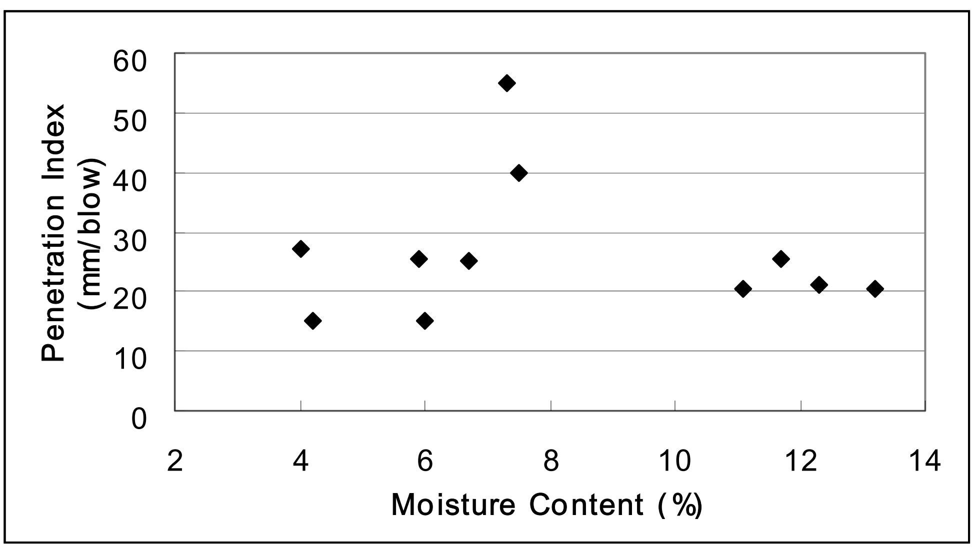


Figure 3.37 Relationship between Moisture Content and Penetration Index from field DCPT for the site of US35 in Knox, IN

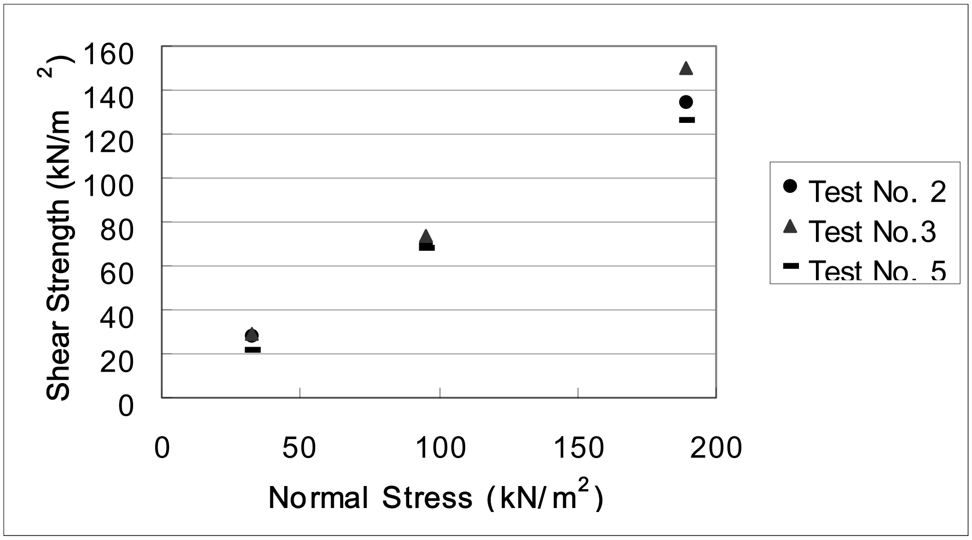


Figure 3.38 Result of Direct Shear Test with different normal stress for the site of US35 in Knox, IN

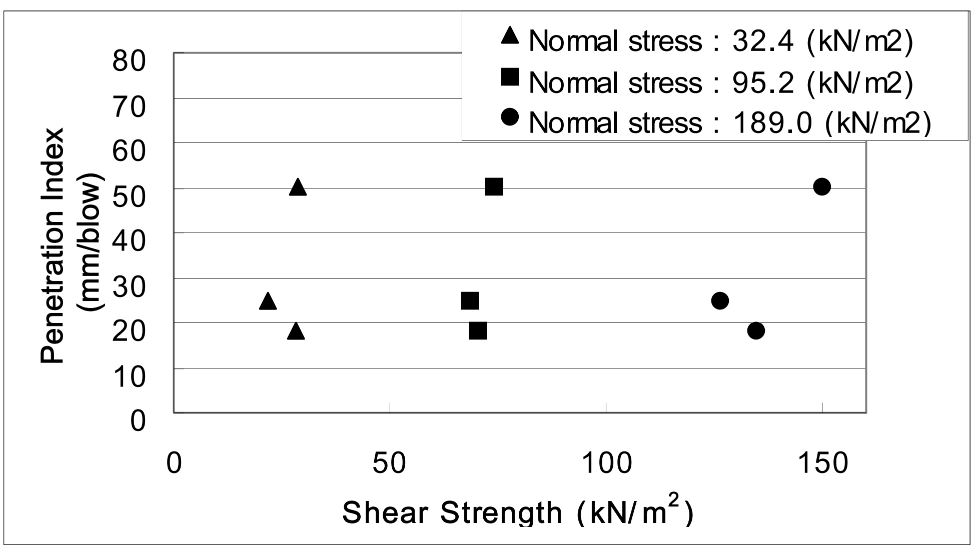


Figure 3.39 Relationship between PI and Shear Strength with different normal stress for the site of US35 in Knox, IN

3.6 Reconstruction Site of Lindberg Road at West Lafayette, IN

Field DCP Tests were performed on subgrade soils at a reconstruction site on Lindberg Road in West Lafayette, Indiana. Construction at the site was to rebuild the existing road and replace old pavement. A clayey sand subgrade embankment was built on the existing road. Eight DCP tests were conducted at several different locations (Station 1+189, 1+200, 1+211, 1+222, 1+233, 1+245, 1+256 and 1+269). Also, in-situ soil densities and moisture contents were measured using the nuclear gauge for each testing location at depths of 5.1 cm (2 in), 15.2 cm (6 in), and 30.5 cm (12 in) from the soil surface. Table 3.10 and Figure 3.40 show in-situ total and dry soil densities and moisture contents measured with the nuclear gauge. The DCPT logs are shown in Figure 3.41 through Figure 3.48.

To characterize the soils of the test site, laboratory tests were performed. A specific gravity test, sieve analysis and Atterberg limit test were conducted. The soil's specific gravity (G_S) is 2.71. From the results of the sieve analysis, the particle size distribution is shown in Figure 3.49. The liquid limit (LL) and plastic limit (PL) are 22.5 and 14.0 respectively from the Atterberg limits tests. The plastic index (IP) is 8.49. The soil is a clayey sand (SC).

The relationships between dry density, moisture content and the penetration index (PI) are shown in Figure 3.50 and Figure 3.51respectively.

The unconfined compression tests were conducted in the laboratory on samples prepared with similar dry densities and moisture contents to the soil in the field. A corresponding PI value with similar dry unit weight can be obtained from the result of the field DCPT. Resilient modulus was calculated using Lee's (1997) equation. Table 3.11

shows the unconfined compressive strength, s_u at 1% strain, resilient modulus and the penetration index from the field DCPT for different dry density.

Table 3.10 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of Lindberg Road in West Lafayette, IN

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
1	5.1	11.7	18.2	16.3
	15.2	10.1	21.6	19.6
	30.5	9.1	24.7	22.6
	Average	10.3	21.5	19.5
2	5.1	11.8	17.8	15.7
	15.2	10.2	21.3	19.3
	30.5	9.2	24.3	22.3
	Average	10.4	21.1	19.1
3	5.1	10.8	18.4	16.6
	15.2	10.0	21.1	19.2
	30.5	8.2	24.1	22.2
	Average	9.7	21.2	19.3
4	5.1	10.4	19.3	17.5
	15.2	9.3	22.2	20.3
	30.5	8.5	25.2	23.2
	Average	9.4	22.2	20.3
5	5.1	12.2	19.1	17.0
	15.2	10.6	21.6	19.5
	30.5	9.1	24.8	22.8
	Average	10.6	21.8	19.8
6	5.1	11.3	19.0	17.1
	15.2	9.9	21.3	19.3
	30.5	8.4	24.5	22.6
	Average	9.9	21.6	19.7

continued

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
7	5.1	11.2	18.9	17.0
	15.2	10.0	21.6	19.6
	30.5	8.8	24.8	22.8
	Average	10.0	21.7	19.8
8	5.1	11.6	18.5	16.6
	15.2	10.2	21.3	19.3
	30.5	8.5	24.4	22.5
	Average	10.1	21.4	19.5

Table 3.11 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of Lindberg Road in West Lafayette, IN

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

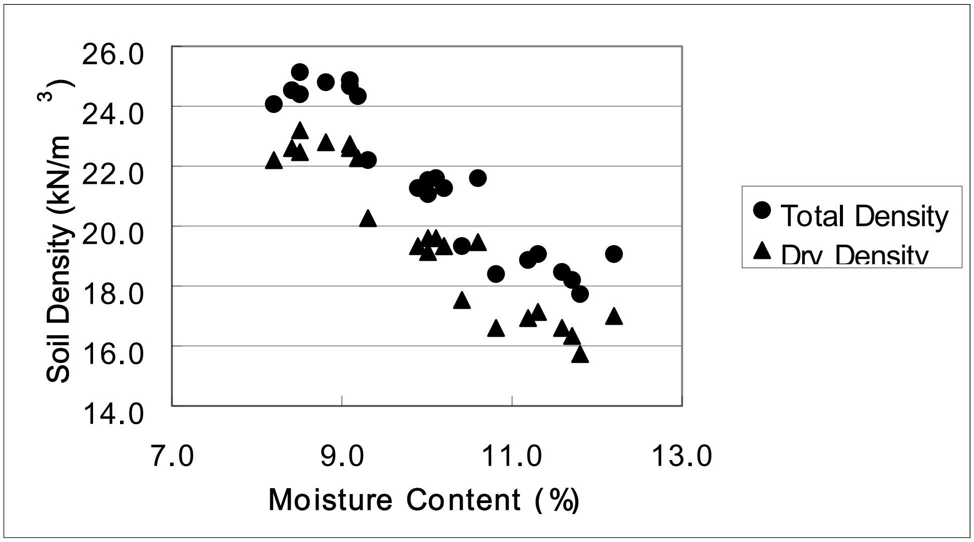


Figure 3.40 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of Lindberg Road in West Lafayette, IN

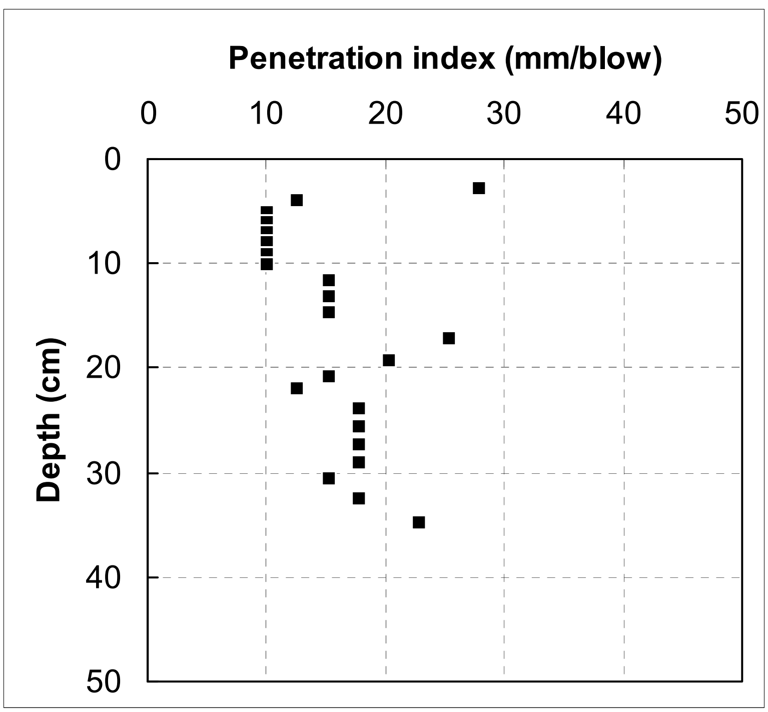


Figure 3.41 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+189, Test No. 1)

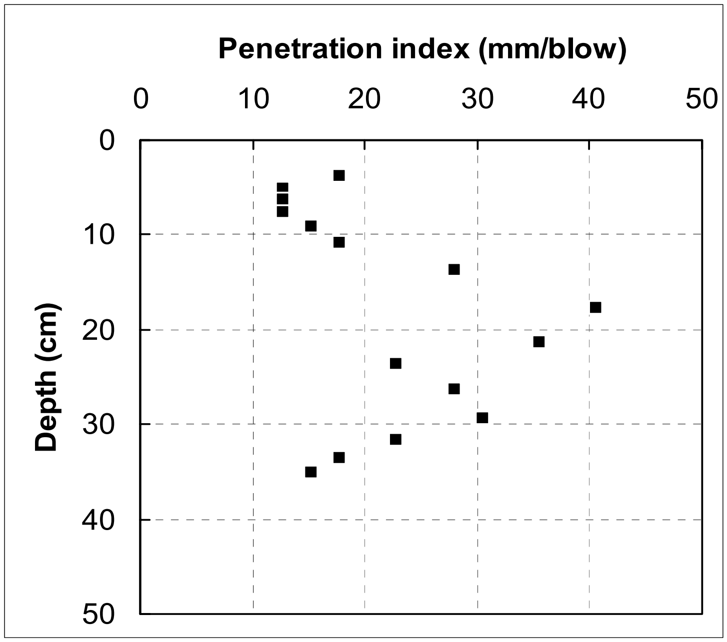


Figure 3.42 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+200, Test No. 2)

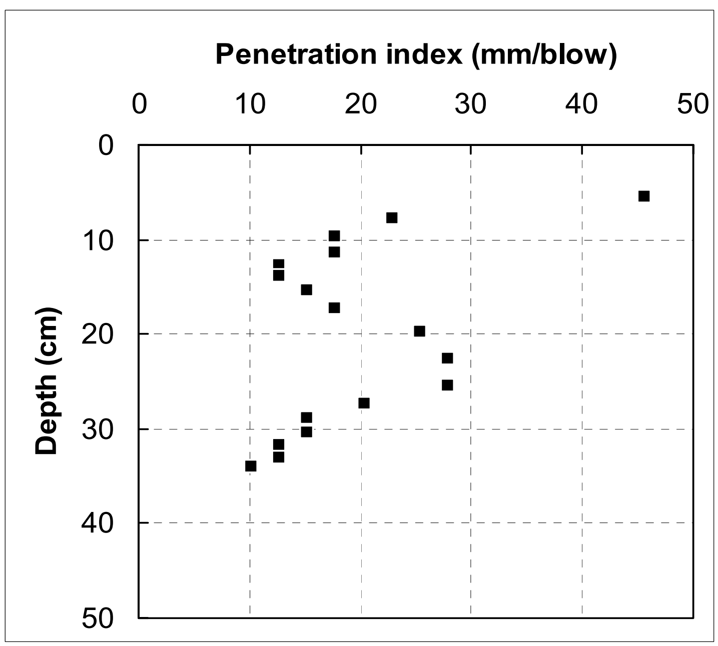


Figure 3.43 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+211, Test No. 3)

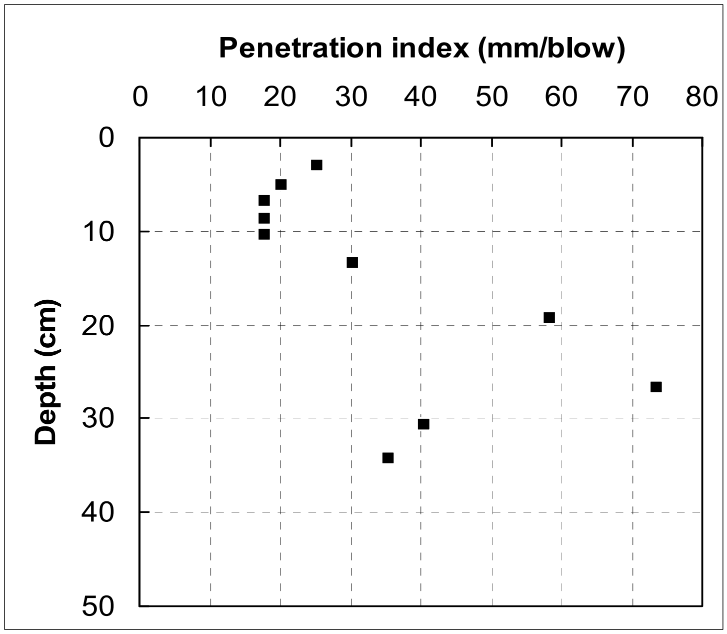


Figure 3.44 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+222, Test No. 4)

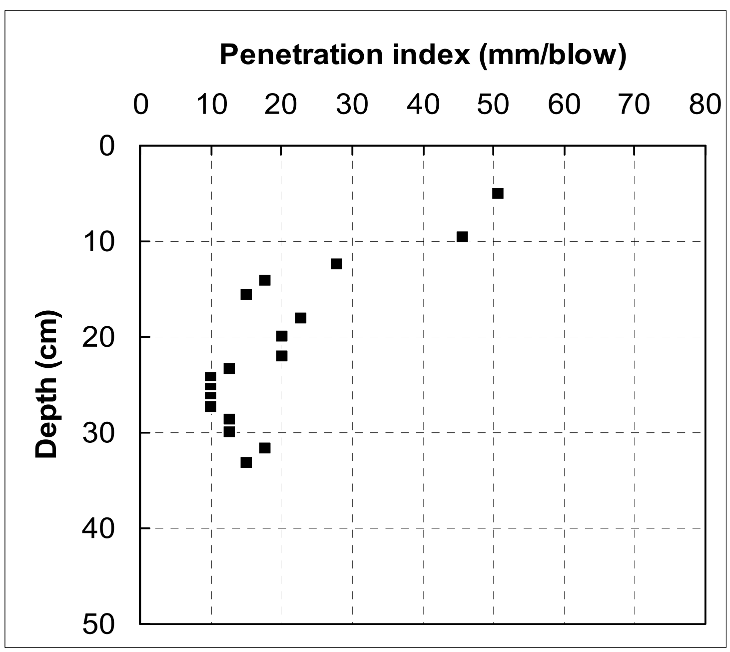


Figure 3.45 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+233, Test No. 5)

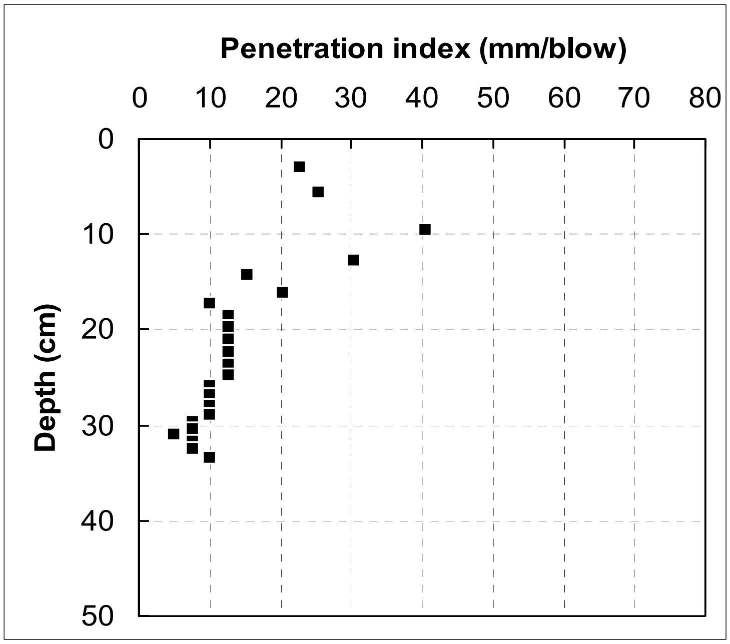


Figure 3.46 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+245, Test No. 6)

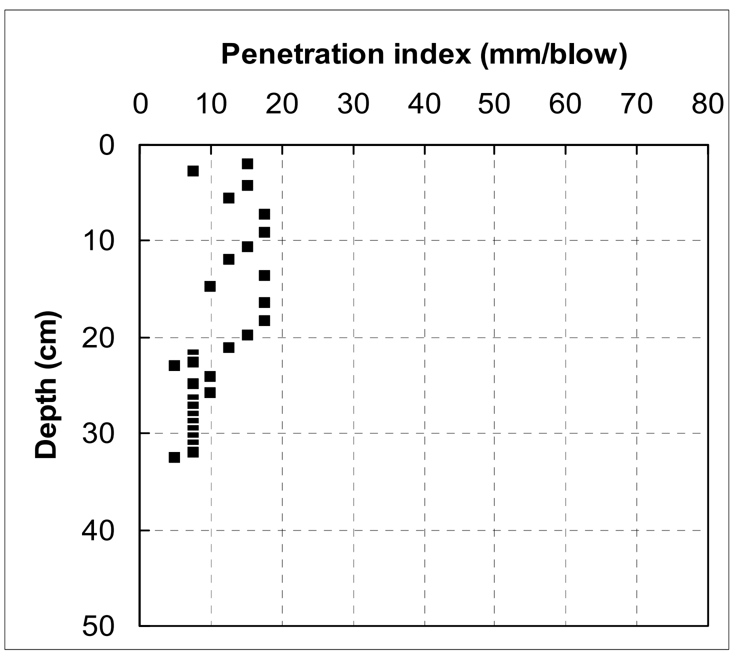


Figure 3.47 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+256, Test No. 7)

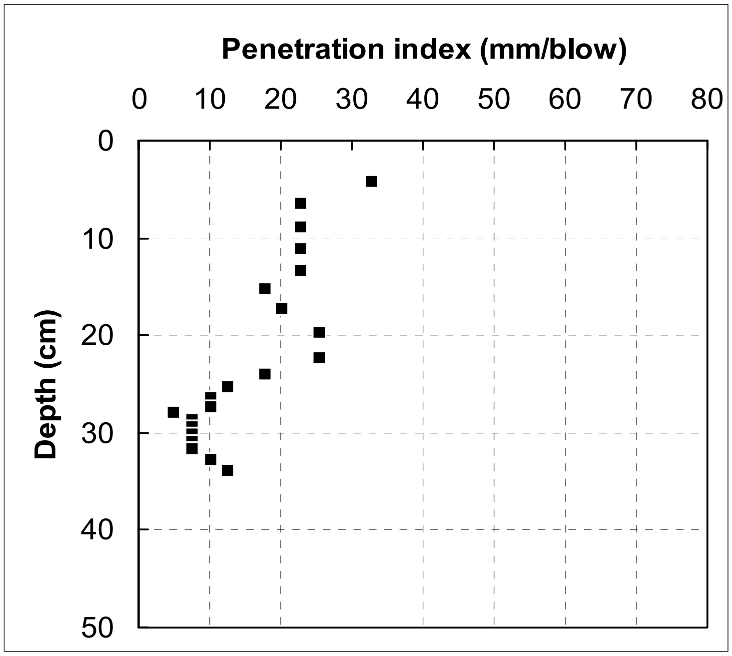


Figure 3.48 Log of DCPT for the site of Lindberg Road in West Lafayette, IN (Station: 1+269, Test No. 8)

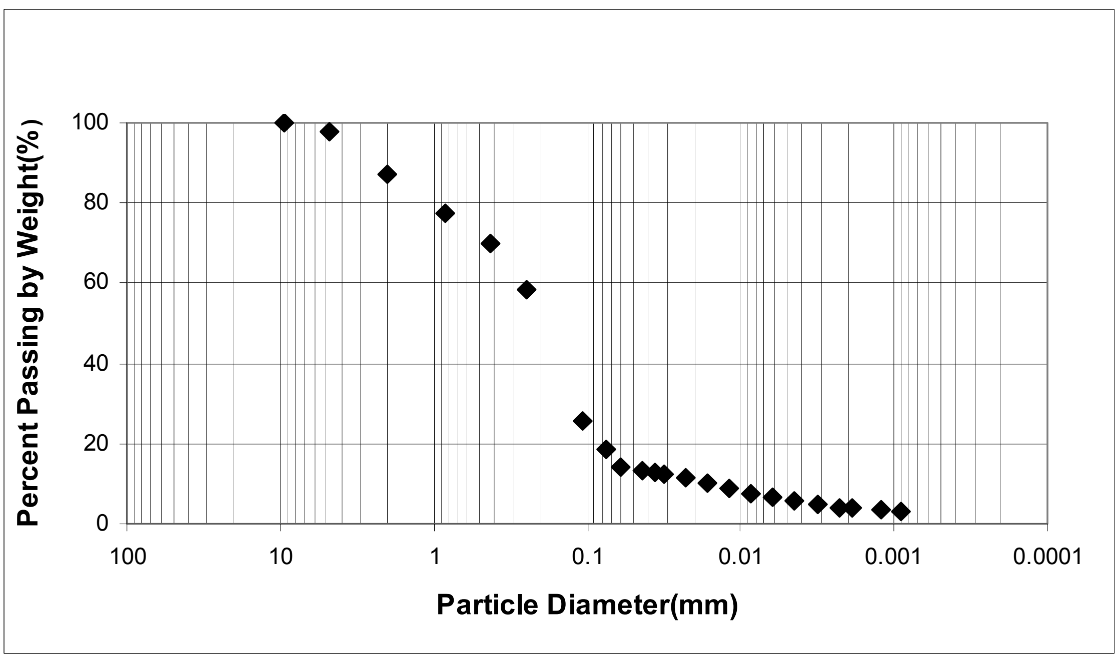


Figure 3.49 Particle size distribution for the site of Lindberg Road in West Lafayette, IN

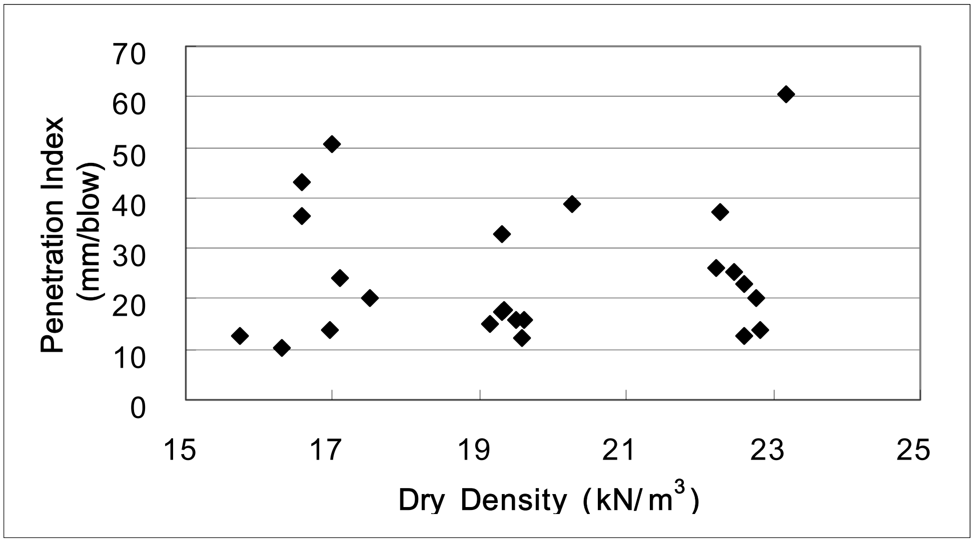


Figure 3.50 Relationship between Dry Density and Penetration Index from field DCPT for the site of Lindberg Road in West Lafayette, IN

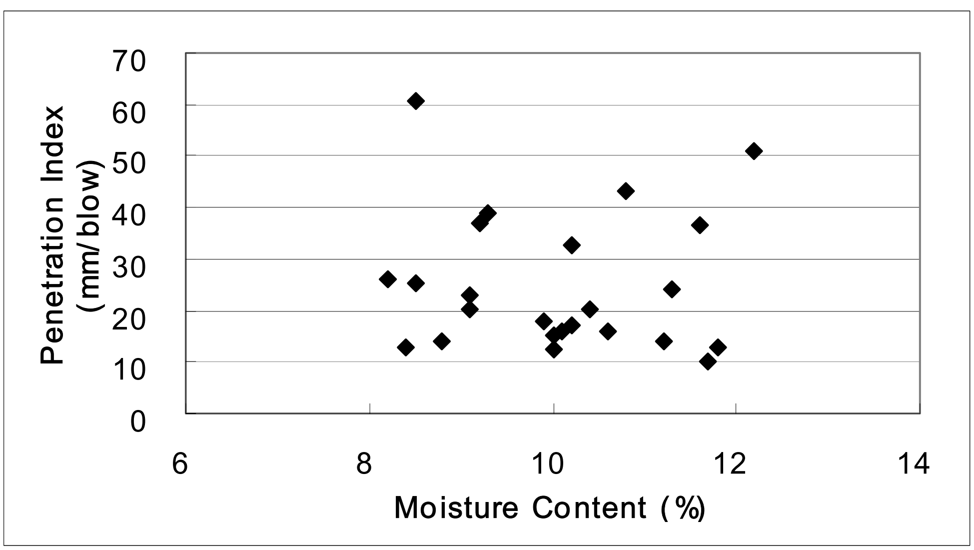


Figure 3.51 Relationship between Moisture Content and Penetration Index from field DCPT for the site of Lindberg Road in West Lafayette, IN

3.7 Reconstruction Site of I-65/County Road 100E in Lebanon, IN

Field DCP Tests were performed on subgrade soils at the I-65/County road 100E construction site in Lebanon, Indiana. The project was for deck reconstruction and lane widening of the county road 100E overpass. The tests were performed on the existing soil after removing the old pavement. Five DCP tests were conducted at several different locations around station 72+137. In-situ soil densities and moisture contents were measured with a nuclear gauge for each testing location at depths of 5.1 cm (2 in), 15.2 cm (6 in), and 30.5 cm (12 in) from the soil surface. Table 3.11 and Figure 3.52 show in-situ total and dry soil densities and moisture contents measured with the nuclear gauge. The DCPT logs are shown in Figure 3.53 through Figure 3.57.

To characterize the tested soils, laboratory tests, such as a specific gravity, sieve analysis and Atterberg limit test were conducted. The soil's specific gravity (GS) is 2.69. The result of the sieve analysis is shown in Figure 3.58 to evaluate a particle size distribution. From the Atterberg limit test the liquid limit (LL) and plastic limit (PL) are 20.9 and 15.3, respectively, and the plastic index (IP) is 5.6. The soil is a clayey sand (SC).

The relationships between dry density, moisture content and the penetration index (PI) are shown in Figure 3.59 and Figure 3.60, respectively.

The unconfined compression tests were conducted in the laboratory on samples, which were prepared with similar dry densities and moisture contents to the soil in the field. These densities and moisture contents were chosen to correspond to those tested with the DCP. From Lee's (1997) equation, a resilient modulus was calculated. Table 3.13 shows the unconfined compressive strength, s_u at 1% strain, resilient modulus and the penetration index from the field DCPT for different dry densities.

Table 3.12 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I65/County Road100E in Lebanon, IN

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
1	5.1	14.6	19.6	17.1
	15.2	12.8	21.0	18.6
	30.5	13.0	21.4	19.0
	Average	13.5	20.7	18.2
2	5.1	16.2	19.9	17.1
	15.2	16.0	20.5	17.7
	30.5	15.7	20.9	18.1
	Average	16.0	20.4	17.6
3	10.2	13.7	20.7	18.2
	15.2	12.5	21.6	19.1
	30.5	12.5	22.2	19.7
	Average	12.9	21.5	19.0
4	10.2	11.4	20.1	18.1
	15.2	10.7	21.9	19.8
	30.5	9.7	22.4	20.4
	Average	10.6	21.5	19.4
5	10.2	11.5	21.2	19.0
	15.2	11.3	21.5	19.4
	30.5	11.2	22.2	20.0
	Average	11.3	21.7	19.5

Table 3.13 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of I65/County Road100E in Lebanon, IN

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

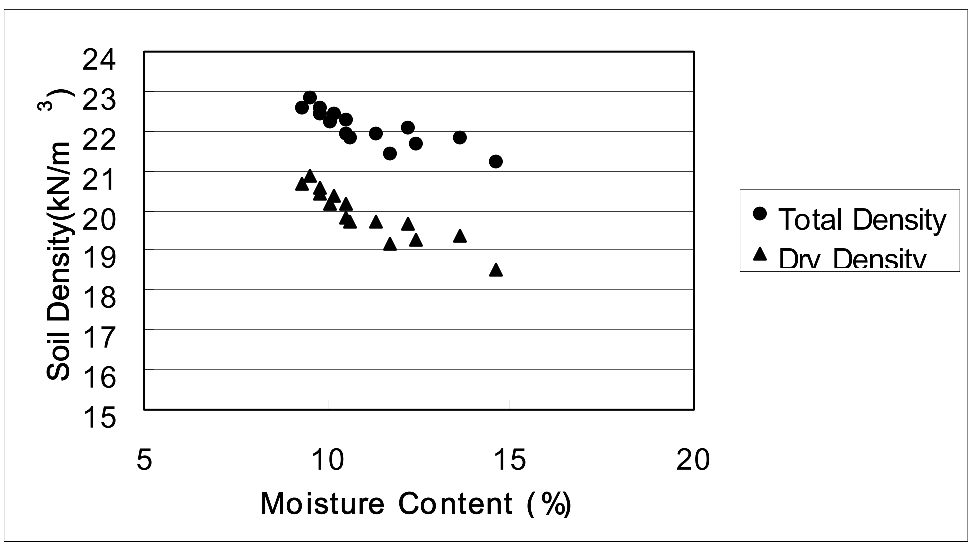


Figure 3.52 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of I65/County Road100E in Lebanon, IN

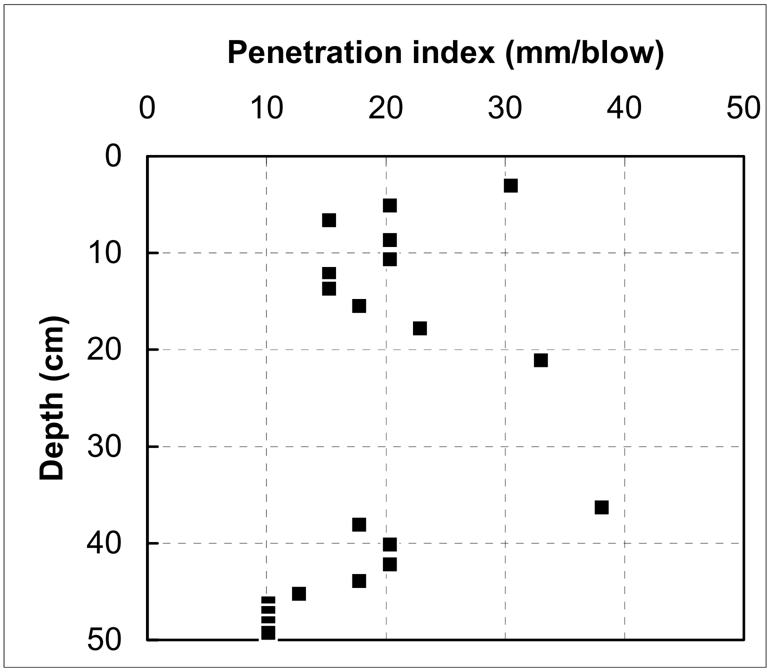


Figure 3.53 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 1)

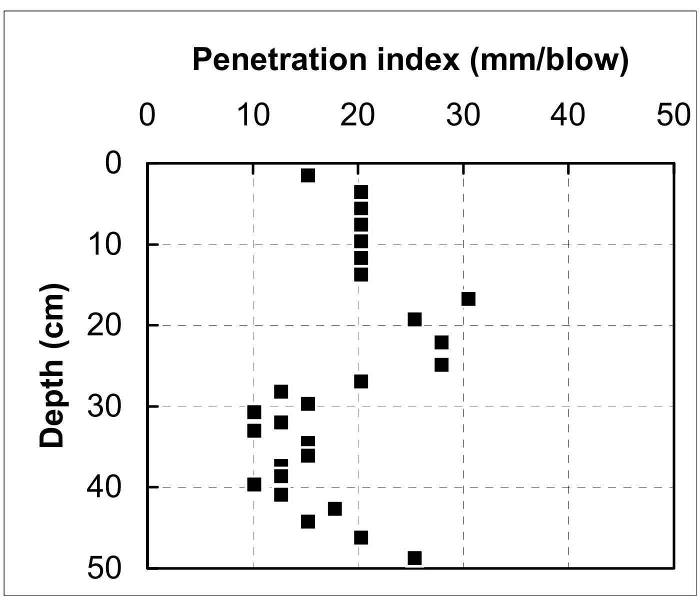


Figure 3.54 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 2)

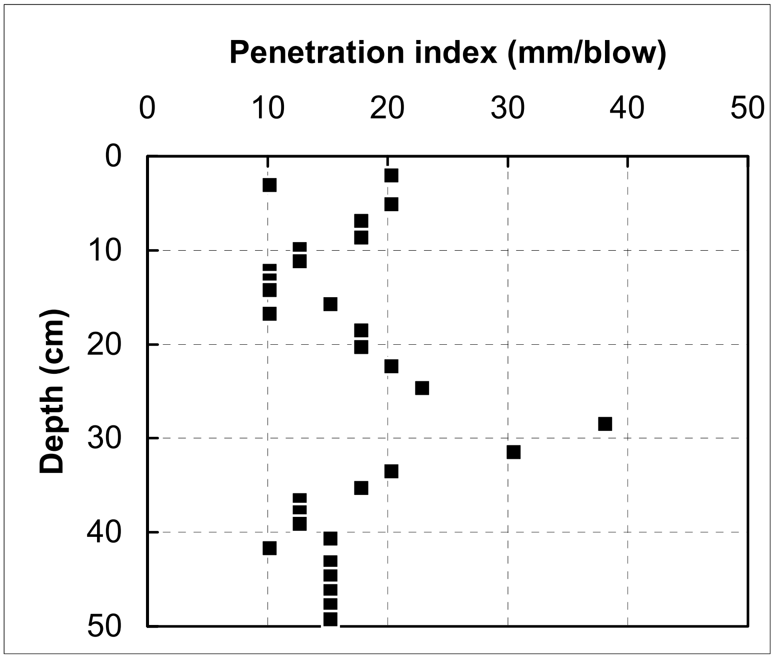


Figure 3.55 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 3)

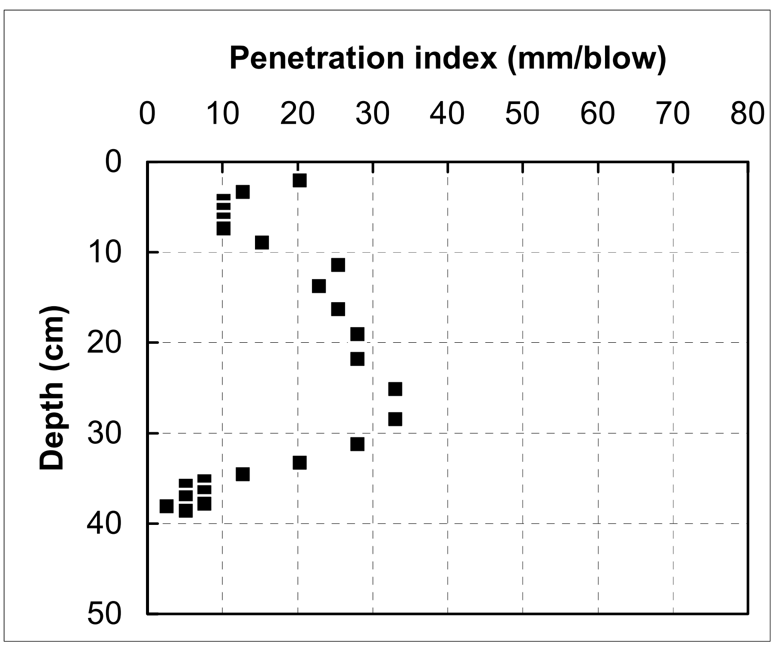


Figure 3.56 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 4)

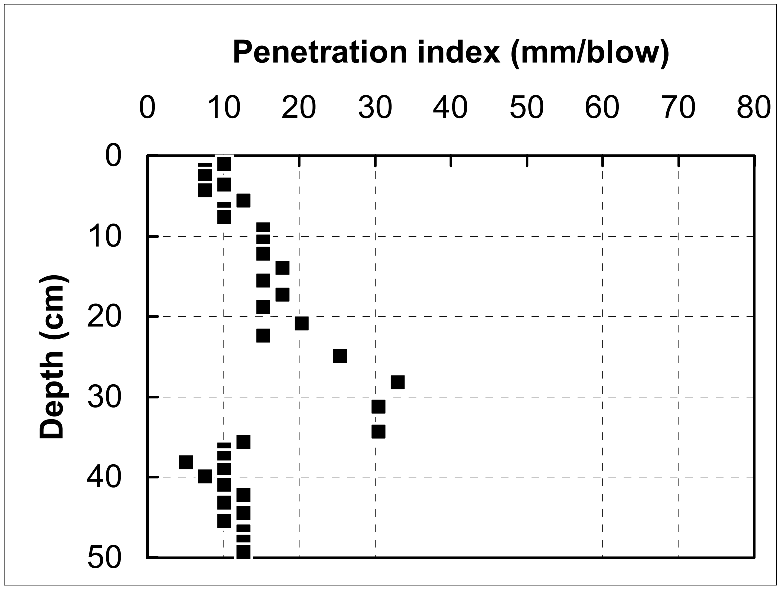


Figure 3.57 Log of DCPT for the site of I65/County Road100E in Lebanon, IN (Station: 72+137, Test No. 5)

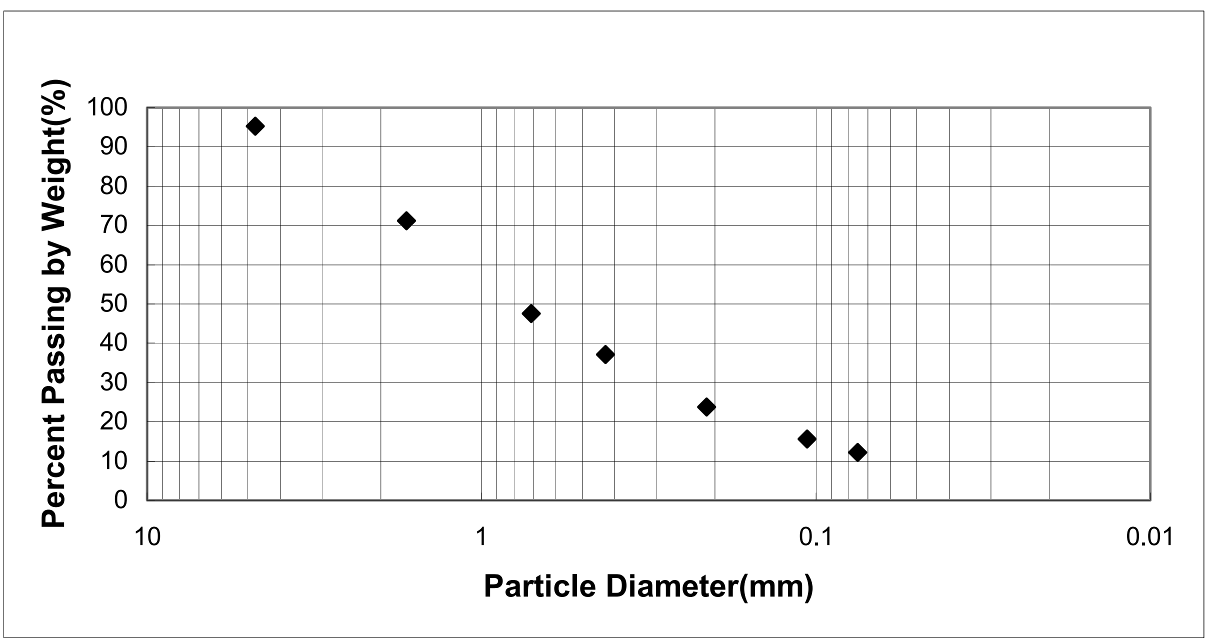


Figure 3.58 Particle size distribution for the site of I65/County Road100E in Lebanon, IN

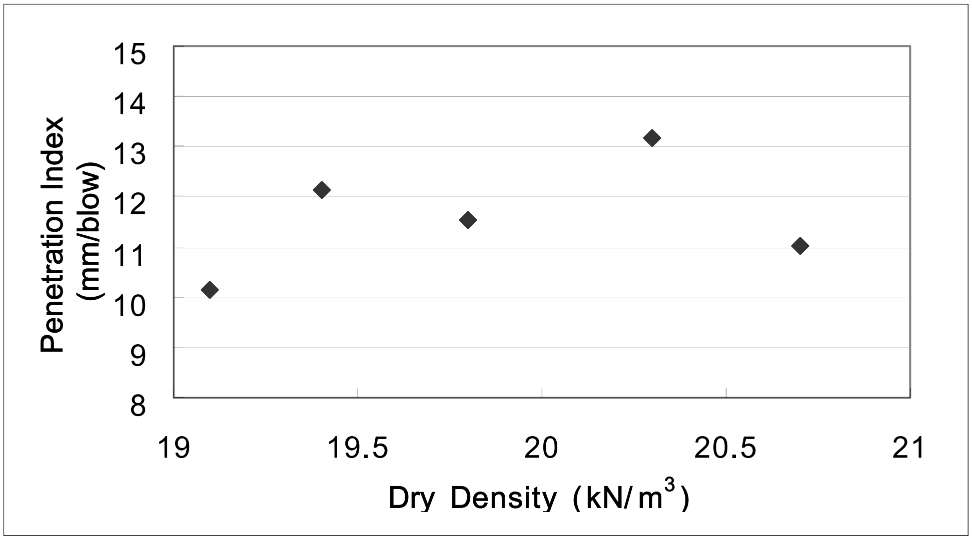


Figure 3.59 Relationship between Dry Density and Penetration Index from field DCPT for the site of I65/County Road100E in Lebanon, IN

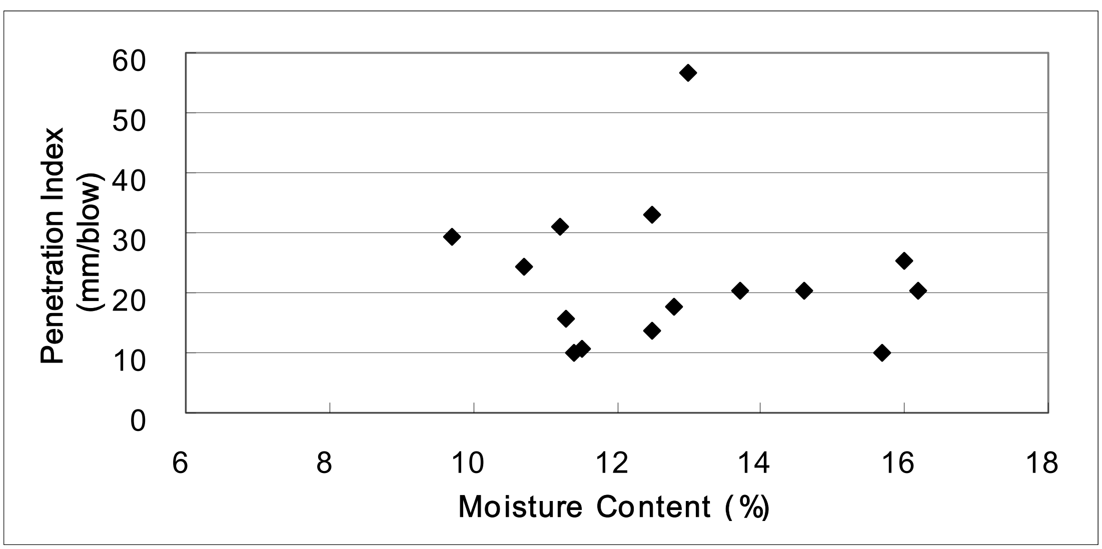


Figure 3.60 Relationship between Moisture Content and Penetration Index from field DCPT for the site of I65/County Road100E in Lebanon, IN

3.8 Reconstruction Site of US36 in Bainbridge, IN

Six field DCP Tests were conducted on subgrade soils at a reconstruction site of US36 in Bainbridge, Indiana. Construction at the site was to rebuild the existing road and replace old pavement. The clayey sand subgrade was exposed after removing the old pavement. The top 2in of subgrade soil was cut down. The DCP tests were conducted at several different locations (Stations No. 10+505, 10+506, 10+722, 10+724, 10+574 and 10+577). Also in-situ soil densities and moisture contents were measured using the nuclear gauge for each testing location at depths of 5.1 cm (2 in), 15.2 cm (6 in), and 30.5 cm (12 in) from the soil surface. In-situ total and dry soil densities and moisture contents measured from the nuclear gauge are shown in Table 3.14 and Figure 3.61. The DCPT logs are shown in Figure 3.62 through Figure 3.67.

In the laboratory, a specific gravity test, sieve analysis and Atterberg limit test were conducted. The soil's specific gravity (GS) is 2.70. From the result of the sieve analysis, the particle size distribution is shown in Figure 3.68. The liquid limit (LL) and plastic limit (PL) are 34.8 and 15.6, respectively, from the Atterberg limit test. The plastic index (IP) is 19.2. The soil is a clayey sand (SC).

Figure 3.69 and Figure 3.70 show the relationships between dry density, moisture content and the penetration index (PI), respectively.

The unconfined compression tests were conducted in the laboratory on samples prepared with similar dry densities and moisture contents to those tested with the DCP in the field. Resilient modulus was calculated using Lee's (1997) equation. Table 3.15 shows the unconfined compressive strength, s_u at 1% strain, resilient modulus and the penetration index from the field DCPT for different dry densities

Table 3.14 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US36 at Bainbridge, IN

Test No.	Depth (cm)	Moisture content (%)	Total unit weight (kN/m^3)	Dry unit weight (kN/m^3)
1	5.1	18.2	19.4	16.4
	15.2	17.6	20.5	17.5
	30.5	17.6	20.6	17.6
	Average	17.8	20.2	17.2
2	5.1	12.9	19.8	17.5
	15.2	12.1	20.2	18.0
	30.5	12.4	20.5	18.2
	Average	12.5	20.1	17.9
3	5.1	19.2	19.7	16.5
	15.2	18.2	20.3	17.1
	30.5	17.8	20.1	17.0
	Average	18.4	20.0	16.9
4	5.1	18.2	20.3	17.2
	15.2	17.4	20.5	17.5
	30.5	18.6	20.2	17.0
	Average	18.1	20.3	17.2
5	5.1	23.3	17.2	14.0
	15.2	19.6	19.2	16.0
	30.5	17.9	17.2	20.3
	Average	20.3	17.9	16.8
6	5.1	16.5	20.0	17.2
	15.2	16.9	20.2	17.3
	30.5	16.5	20.3	17.4
	Average	16.6	20.2	17.3

Table 3.15 Result of Unconfined Compression Test and corresponding Penetration Index from field DCPT for the site of US36 at Bainbridge, IN

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)
17.6	151.5	30.1	20152.5	23.9
18.2	87.2	8.1	5583.1	17.78
19.6	168.4	33.0	21992.7	10.3

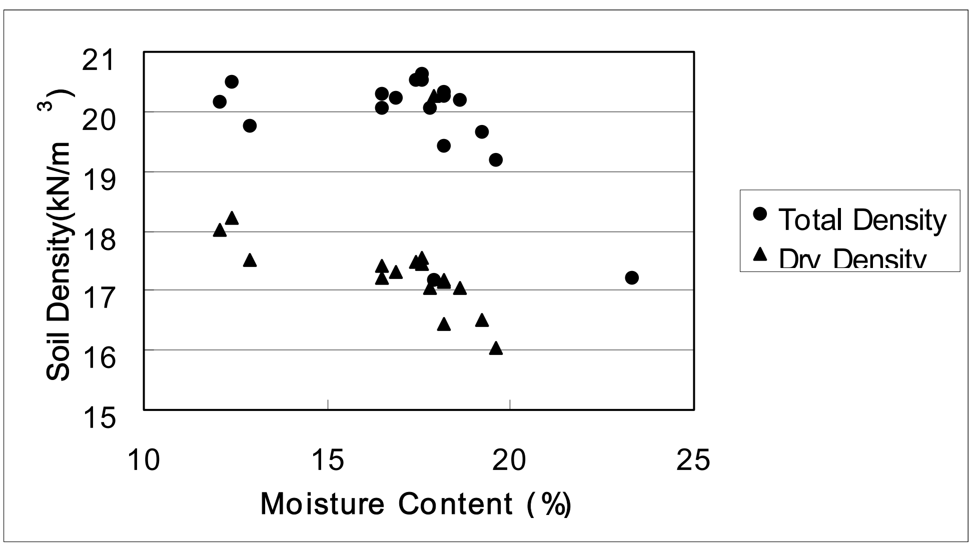


Figure 3.61 Total and Dry Soil Densities and Moisture Contents measured from nuclear gauge for the site of US36 at Bainbridge, IN

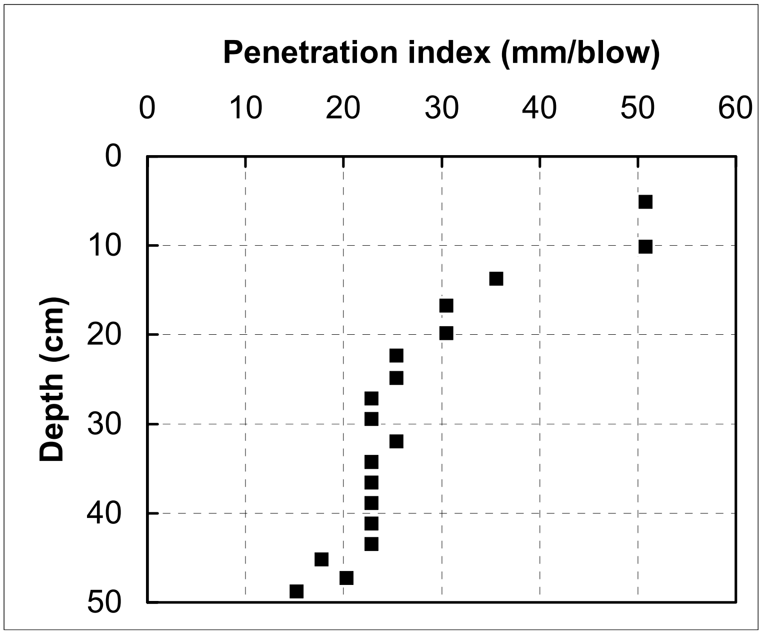


Figure 3.62 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+505, Test No. 1)

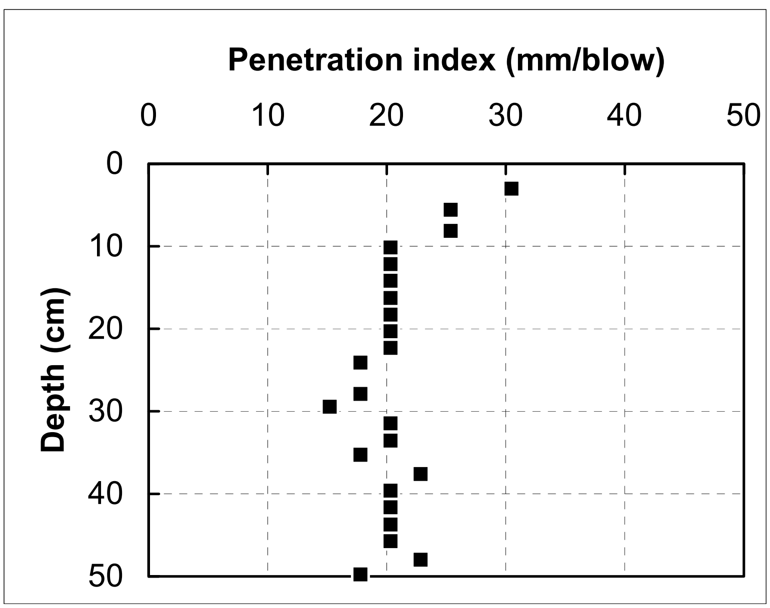


Figure 3.63 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+506, Test No. 2)

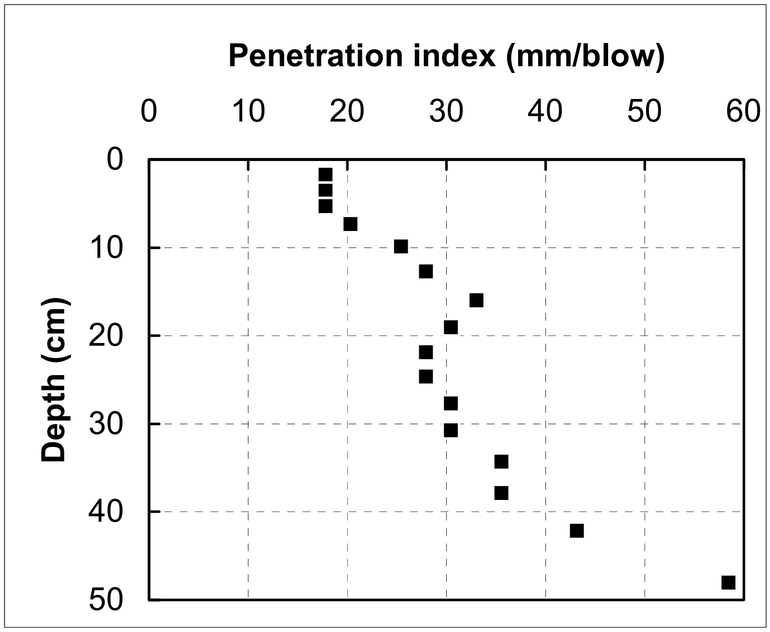


Figure 3.64 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+722, Test No. 3)

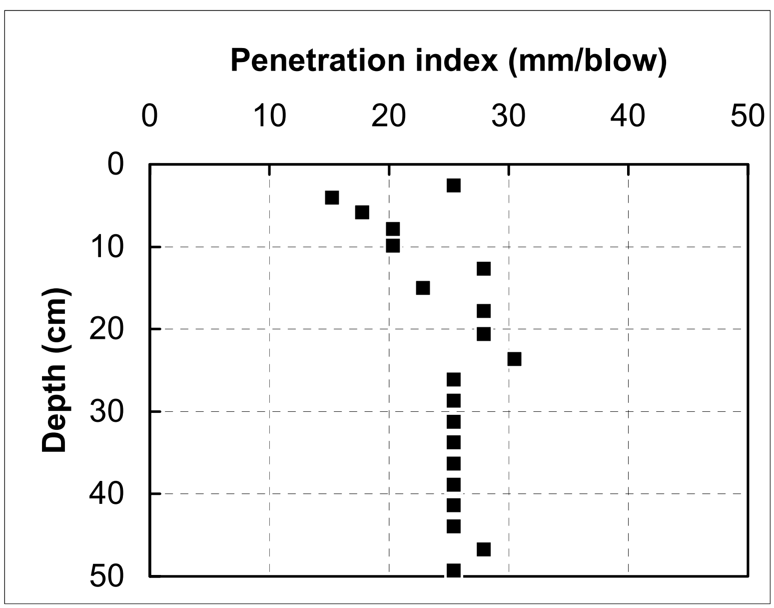


Figure 3.65 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+724, Test No. 4)

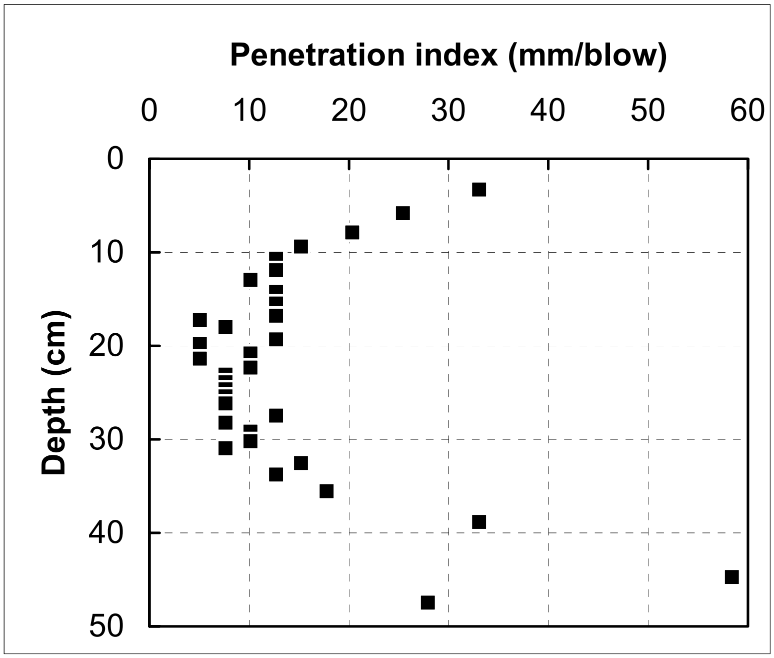


Figure 3.66 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+574, Test No. 5)

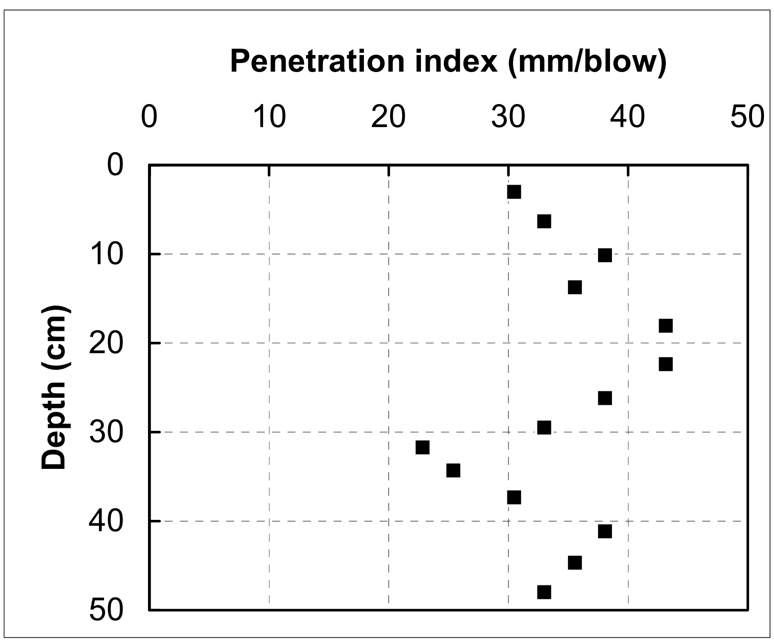


Figure 3.67 Log of DCPT for the site of US36 at Bainbridge, IN (Station: 10+577, Test No. 6)

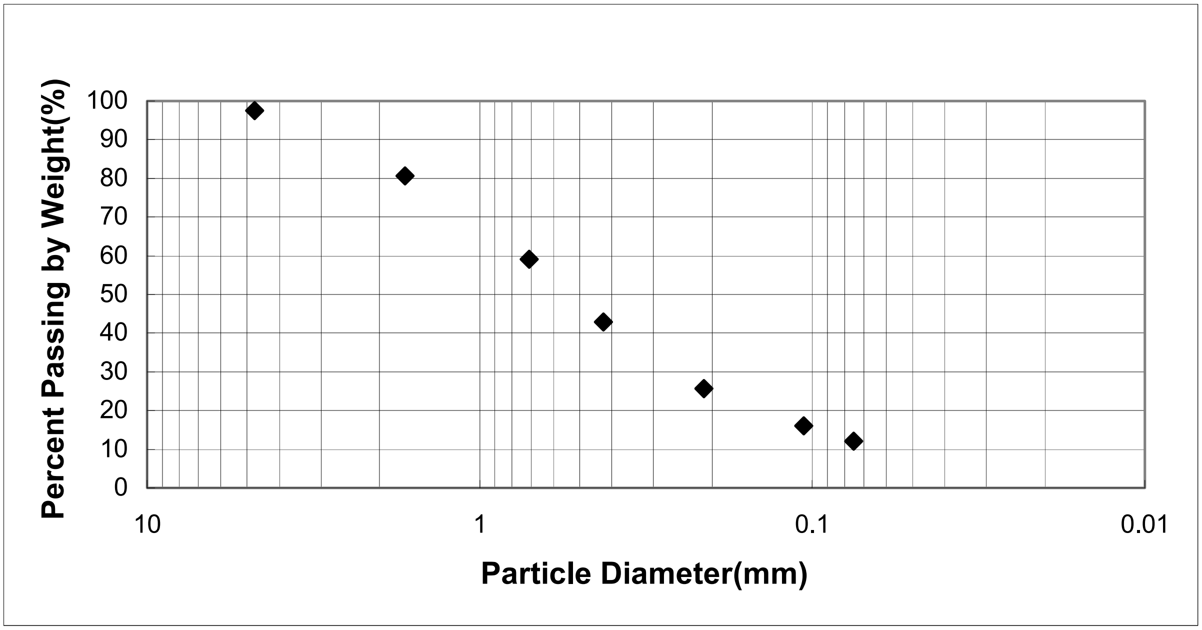


Figure 3.68 Particle size distribution for the site of US36 at Bainbridge, IN

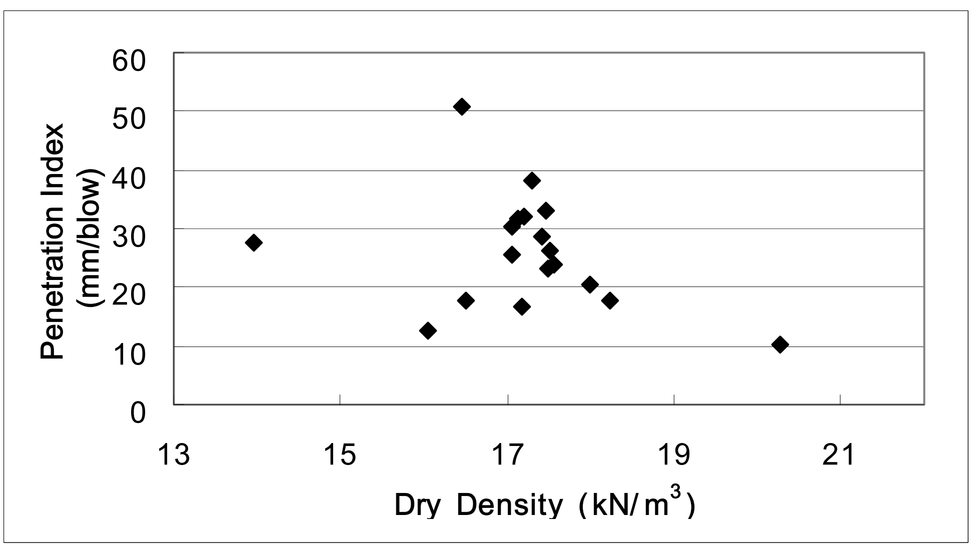


Figure 3.69 Relationship between Dry Density and Penetration Index from field DCPT for the site of US36 at Bainbridge, IN

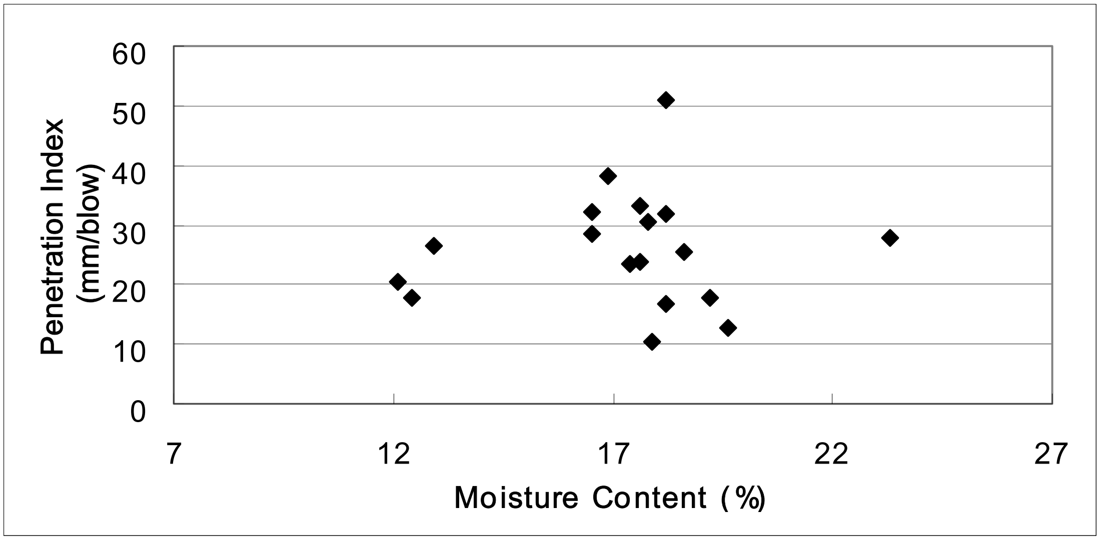


Figure 3.70 Relationship between Moisture Content and Penetration Index from field DCPT for the site of US36 at Bainbridge, IN

3.9 Analysis of the Results from Field DCP and Laboratory Tests

The field DCP tests and laboratory tests done in this project were presented in Sections 3.2 through 3.8. A relationship between dry density and moisture content based on the data for the seven different sites is shown in Figure 3.71. The relationships between penetration index, dry density and moisture content are shown in Figure 3.72 and Figure 3.73. To get a better correlation between penetration index and dry density, the dry density of the clayey sand is normalized using γ_w and the vertical effective stress. Figure 3.77 shows the relationship between dry density of clayey sand and penetration index where

$$R = \frac{\left(\gamma_d / \gamma_w\right)^2}{\left(\sigma'_V / p_A\right)}$$

The equation for the dry density was derived in terms of the PI as follows,

$$\gamma_d = \left(10^{1.5} \times PI^{-0.14} \times \sqrt{\frac{\sigma'_V}{p_A}}\right)^{0.5} \times \gamma_W$$

This equation can be used to predict γ_d using PI value. The γ_d value calculated from this equation has an error range of $\pm 1.63\,\text{kN/m}^3$. Note that, had we considered site-specific correlations, the resulting correlations would be better, as suggested by the different symbols for each site appearing in Figure 3.71. There is no clear relationship between γ_d and PI for well-graded or poorly-graded sand.

The unconfined compression tests that were conducted for clayey sand (I-65 site in Hobart, Lindberg Road site in West Lafayette, I-65/County Road 100E site in Lebanon and US36 site in Bainbridge, IN) and well graded sand with clay (US49 site in Valpariso, IN) are shown in Figure 3.74. Figure 3.75 and Figure 3.76 show that the penetration index decreases as either the unconfined compressive strength or $(s_u)_{1.0\%}$ decrease. The resilient modulus for soils from different sites was obtained using the Lee (1997) equation. Figure 3.76 shows the relationship between the resilient modulus and the penetration index. The equation for the resilient modulus in terms of the PI was developed as follows,

$$M_r = -3279PI + 114100$$

where Mr=resilient modulus in kPa; and PI=penetration index in mm/blow.

This equation should be used carefully, since it is derived from scattered and limited data. More data are needed to develop a complete database.

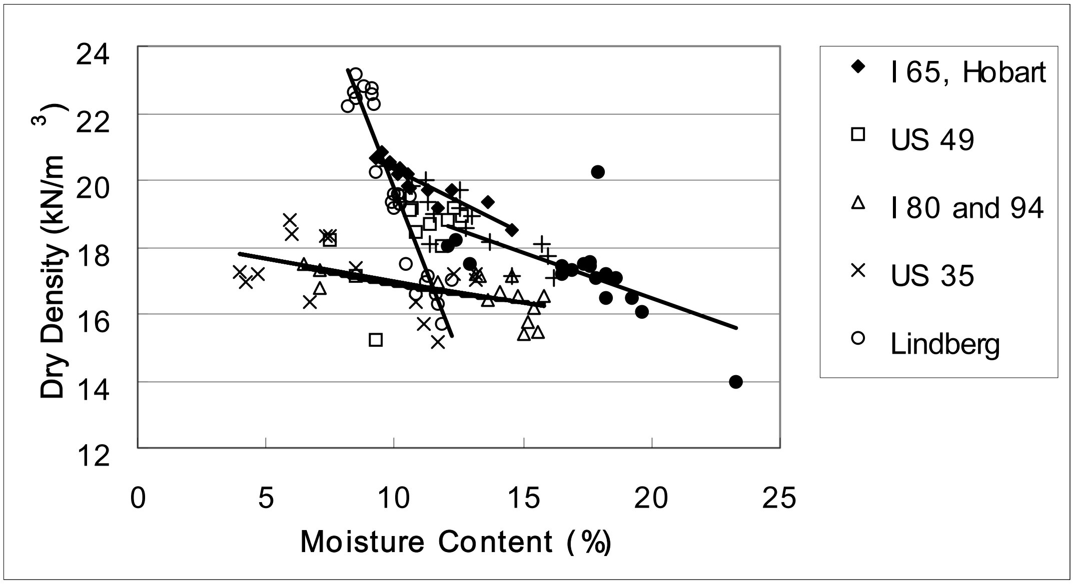


Figure 3.71 Relationship between Moisture Content and Dry Density

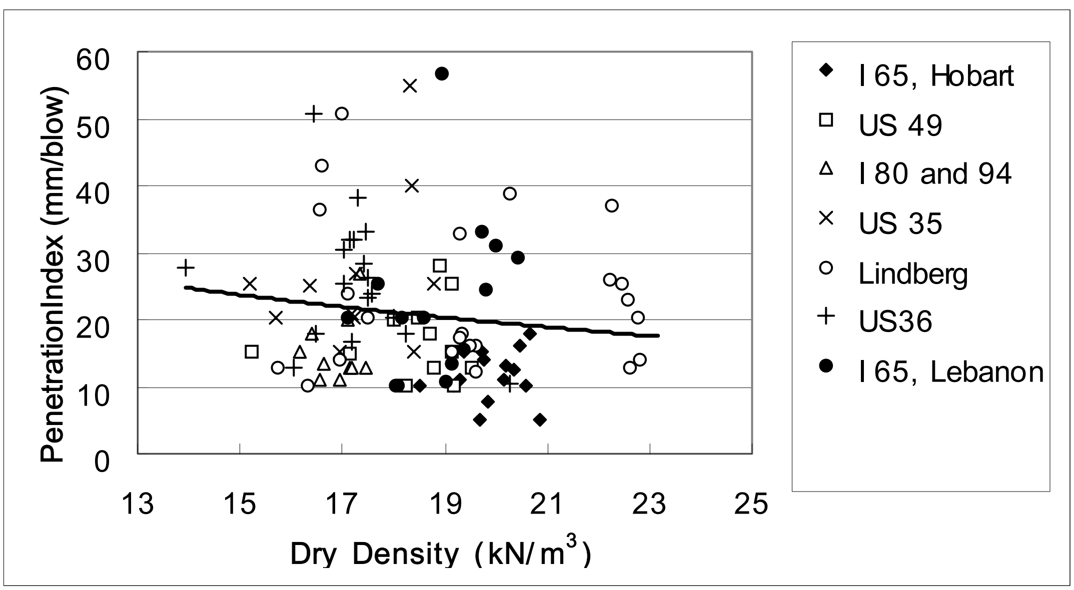


Figure 3.72 Relationship between Dry Density and Penetration Index

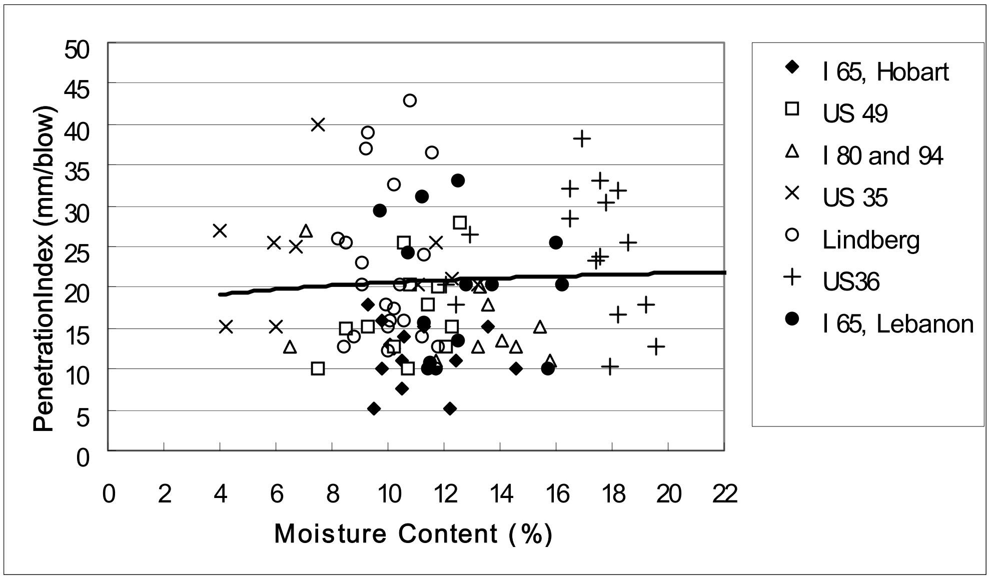


Figure 3.73 Relationship between Moisture Content and Penetration Index

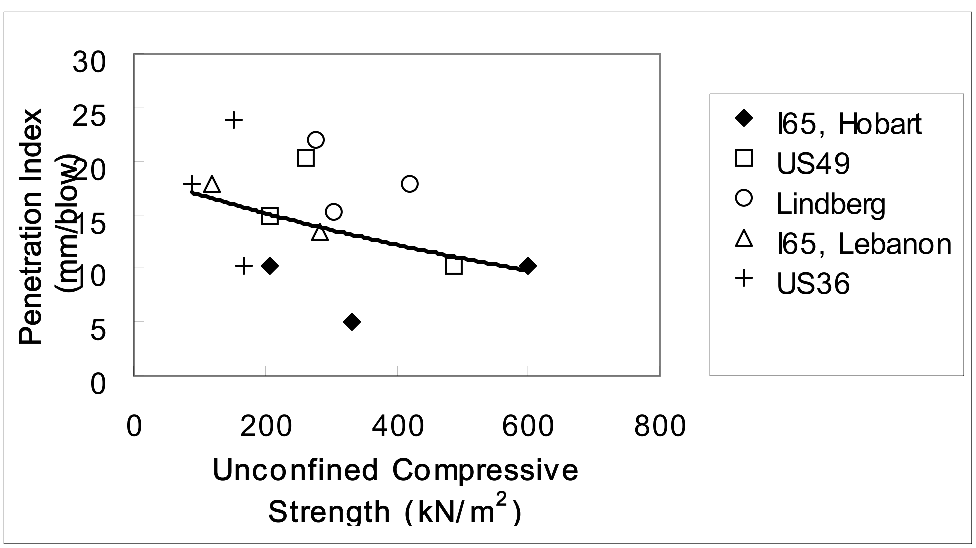


Figure 3.74 Relationship between Unconfined Compressive Strength and Penetration Index

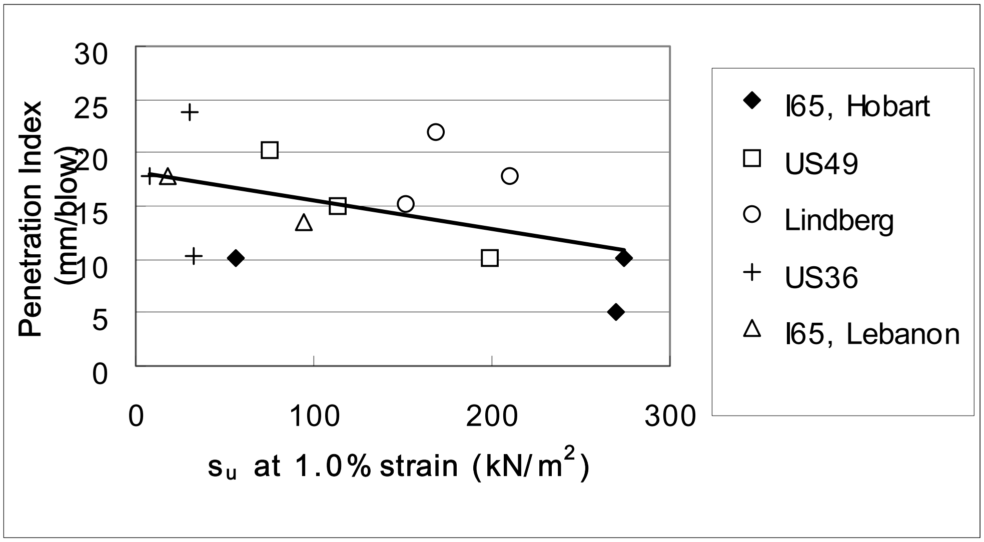


Figure 3.75 Relationship between s_u at 1.0% strain and Penetration Index

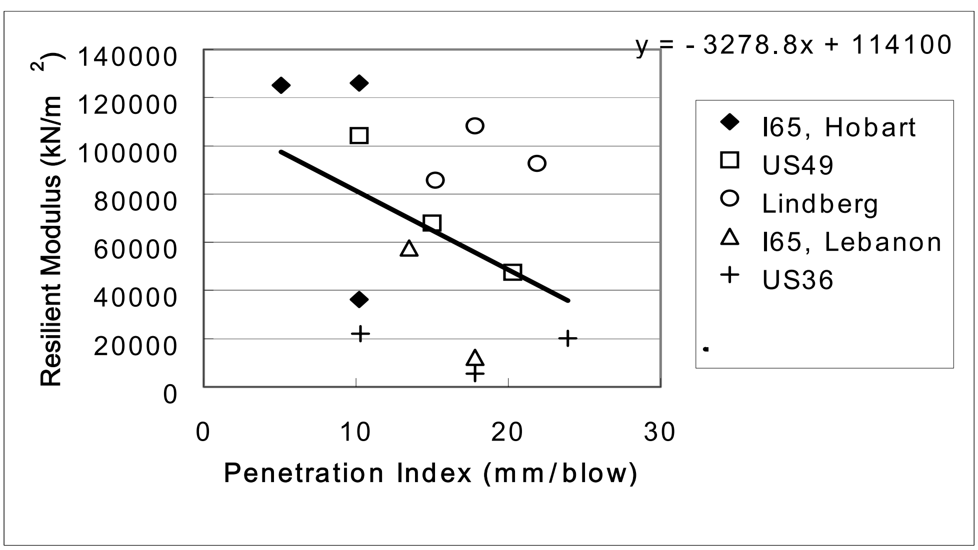


Figure 3.76 Relationship between Resilient Modulus and Penetration Index

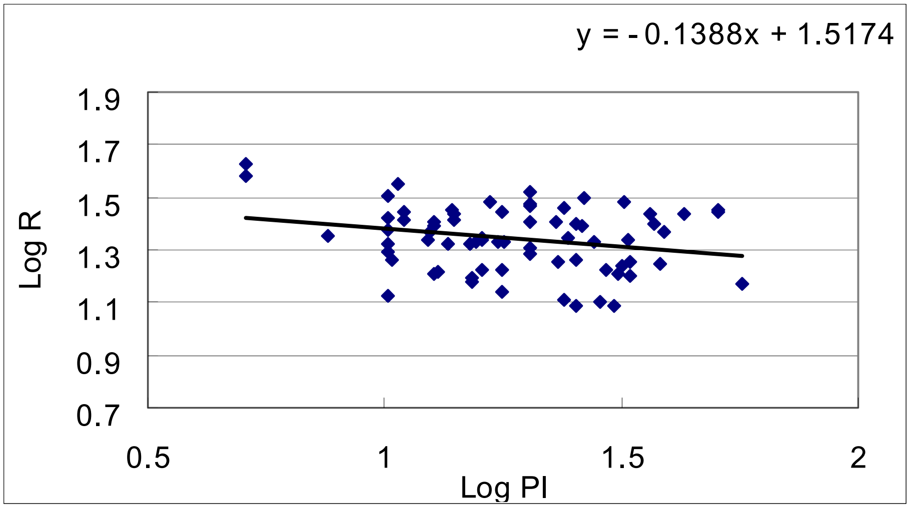


Figure 3.77 Relationship between normalized Dry density and Penetration Index

CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

(1) Field DCP Tests were performed at seven sites. Four sites contained clayey sands, one contained a well graded sand with clay and two contained a poorly graded sand. For each test location, in-situ soil density and moisture contents were measured using a nuclear gauge at three different depths. The relationship between the soil properties and the penetration index were examined. Though the data shows considerable scatter, a trend appears to exist, particularly if each site is considered separately, the penetration index decreases as the dry density increases and slightly increases as moisture content increases. It may be possible to improve the correlation by normalizing the quantities in a different way and by obtaining more data.

(2) For clayey sand classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), the equation for the dry density was derived in terms of the PI as follows:

$$\gamma_d = \left(10^{1.5} \times PI^{-0.14} \times \sqrt{\frac{\sigma'_V}{p_A}} \right)^{0.5} \times \gamma_W$$

where PI = penetration index in mm/blow; and p_A = reference stress (100kPa).

This equation can be used to predict γ_d from the measured PI value. The actual γ_d will be in a range defined by the calculated γ_d $\pm 1.63\,\text{kN/m}^3$.

(3) To investigate the relationship between the shear strength of poorly graded sand and the penetration index, direct shear tests were performed on samples obtained from the field. The results of the direct shear tests also show considerable scatter.

(4) For clayey sands and well-graded sands with clay classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), unconfined compression tests were conducted. The test results show some correlation with the penetration index (PI). It was observed that PI decreases as unconfined compressive strength increases. Additionally, the resilient modulus was calculated from s_u at 1.0% strain using the Lee (1997) equation. The following correlation was developed between M_r and PI:

$$M_r=-3279PI + 114100$$

where M_r=resilient modulus in kPa; and PI=penetration index in mm/blow

This relationship should be used with caution since it is derived from a very weak correlation based on highly scattered data for different sites. There is a need for further study to gather sufficient data to refine this relationship into a reliable equation.

4.2 Recommendations

(1) For clayey sand classified in accordance with the United Classification System (sandy loam classified in accordance with INDOT standard specifications Sec. 903), the equation for the dry density in terms of PI can be used for predicting γ_d using field DCP tests.

(2) Since such predictions using the DCPT are subject to considerable uncertainty, DCPT should be performed for compaction control in combination with a few conventional test methods, such as the nuclear gage. These can be used to anchor or calibrate the DCPT correlation for specific sites, reducing the uncertainty in the predictions. Site-specific correlations do appear to be of better quality.

(3) The DCPT should not be used in soil with gravel. Unrealistic PI values could be obtained and the penetrometer shaft could be bent.

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