FINAL REPORT

Performance-Based Quality Assurance/Quality Control (QA/QC) Acceptance Procedures for In-Place Soil Testing Phase 3

FDOT Contract No. BDV31-977-25

UF Contract Nos. 00102168 & 00102731

Submitted By:

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Developed for the

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DISCLAIMER

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

The Briaud compaction device (BCD) is manufactured by Roctest Ltd., Saint-Lambert, Quebec.

The Clegg impact soil tester (CIST) is marketed in the United States by Lafayette Instrument, Lafayette, IN.

The GeoGauge or soil stiffness gauge (SSG) is manufactured by Humboldt Mfg. Co., Elgin, IL.

The geophone-type lightweight deflectometer (LWD) is manufactured by Dynatest International, Glostrup, Denmark.

The accelerometer-type LWD is manufactured by Zorn Instruments, Stendal, Germany.

The 'dirt' seismic pavement (or property) analyzer (DSPA) is manufactured by Geomedia Research and Development, El Paso, TX.

METRIC CONVERSION FACTORS (from FHWA)

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
		FORCE and PRESSURE or STRESS		
lbf	pound force	4.45	newtons	N
lbf/in^2	pound force per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS TO ENGLISH UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
TEMPERATURE (exact degrees)				
^oC	Celsius	1.8C+32	Fahrenheit	^oF
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
FORCE and PRESSURE or STRESS				
N	newtons	0.225	pound force	lbf
kPa	kilopascals	0.145	pound force per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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16. Abstract

One of the objectives of this study was to evaluate soil testing equipment based on its capability of measuring in-place stiffness or modulus values. As design criteria transition from empirical to mechanistic-empirical, soil test methods and equipment that measure properties such as stiffness and modulus and how they relate to Florida materials are needed. Requirements for the selected equipment are that they be portable, cost effective, reliable, accurate, and repeatable. A second objective is that the selected equipment measures soil properties without the use of nuclear materials. The current device used to measure soil compaction is the nuclear density gauge (NDG). Equipment evaluated in this research included lightweight deflectometers (LWD) from different manufacturers, a dynamic cone penetrometer (DCP), a GeoGauge, a Clegg impact soil tester (CIST), a Briaud compaction device (BCD), and a seismic pavement analyzer (SPA). Evaluations were conducted over ranges of measured densities and moistures. Testing (Phases I and II) was conducted in a test box and test pits. Phase III testing was conducted on materials found on five construction projects located in the Jacksonville, Florida, area.

Phase I analyses determined that the GeoGauge had the lowest overall coefficient of variance (COV). In ascending order of COV were the accelerometer-type LWD, the geophone-type LWD, the DCP, the BCD, and the SPA which had the highest overall COV. As a result, the BCD and the SPA were excluded from Phase II testing.

In Phase II, measurements obtained from the selected equipment were compared to the modulus values obtained by the static plate load test (PLT), the resilient modulus (M_R) from laboratory testing, and the NDG measurements. To minimize soil and moisture content variability, the single spot testing sequence was developed. At each location, test results obtained from the portable equipment under evaluation were compared to the values from adjacent NDG, PLT, and laboratory M_R measurements. Correlations were developed through statistical analysis. Target values were developed for various soils for verification on similar soils that were field tested in Phase III.

The single spot testing sequence also was employed in Phase III, field testing performed on A-3 and A-2-4 embankments, limerock-stabilized subgrade, limerock base, and graded aggregate base found on Florida Department of Transportation construction projects.

The Phase II and Phase III results provided potential trend information for future research—specifically, data collection for in-depth statistical analysis for correlations with the laboratory M_R for specific soil types under specific moisture conditions. With the collection of enough data, stronger relationships could be expected between measurements from the portable equipment and the M_R values.

Based on the statistical analyses and the experience gained from extensive use of the equipment, the combination of the DCP and the LWD was selected for in-place soil testing for compaction control acceptance. Test methods and developmental specifications were written for the DCP and the LWD. The developmental specifications include target values for the compaction control of embankment, subgrade, and base materials.

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EXECUTIVE SUMMARY

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compaction control acceptance. Test methods and developmental specifications were written for the DCP and the LWD. The developmental specifications include target values for the compaction control of embankment, subgrade, and base materials.

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Chapter 1: Introduction

1.1 Background

Earthwork construction acceptance by the Florida Department of Transportation (FDOT) requires in-place (field) testing conducted with a nuclear density gauge (NDG) to determine dry density, which is the current measure for acceptance. The in-place dry density is compared to the laboratory dry density. Specifications require the in-place density to be at least a certain percentage of the laboratory density.

As design criteria transition from empirical to mechanistic-empirical (M-E), test methods that measure properties such as stiffness and modulus and how they relate to Florida conditions should be investigated.

1.1.1 Problem Statement Number 1

Current Quality Assurance/Quality Control (QA/QC) procedures do not measure mechanistic or performance-based properties as defined by the *Mechanistic-Empirical Pavement Design Guide* (MEPDG).

M-E design criteria are based on the use of resilient modulus (M_R) as the primary input parameter when characterizing soil and base material stiffness. M_R is determined by laboratory testing. Field testing is measured using the plate load test (PLT). Our research focused on the study of the various methods for determining the modulus of the in-place soil and base materials.

1.1.2 Problem Statement Number 2

Alternatives to the current nuclear density test method are being evaluated due to the cost of the associated radiation safety program needed for the operation of nuclear density testing equipment.

1.2 Statement of Hypothesis

In-place soil stiffness/modulus measurements can be substituted for density specifications for compaction control and verification of M-E pavement design criteria. To prove or disprove the hypothesis, the purpose of this project was to determine whether the selected equipment and test procedures provide equal or better precision when compared to the existing soil density QA/QC acceptance program for soil compaction.

1.3 Research Objectives

On this project we identified testing equipment and test methods to produce a stiffness/modulus-based equivalent (or correlation) measurement and conducted the following protocol:

1. Evaluate in-place soil testing equipment. The selected equipment shall have the capability of measuring soil stiffness/modulus values. The selected equipment shall be portable, cost effective, reliable, accurate, and repeatable. This equipment will be used to evaluate in-place soil properties over ranges of measured density and moisture. Although the dynamic cone penetrometer (DCP) does not measure stiffness/modulus directly, it is included and will be used in conjunction with equipment that measures stiffness/modulus such as the lightweight deflectometer (LWD).
2. Compare the stiffness/modulus values obtained with the selected portable testing equipment to the modulus values obtained by static PLTs, the resilient modulus values (M_R) obtained in the soils laboratory, and the densities obtained with the NDG.

1.3.1 Significance, Use, and Implementation

Accurate and reliable soil compaction measurements are important for assessing the operational performance and service life of pavement foundation systems. Performance-related compaction control testing, which has the ability to support traffic loads without undue deflection or creating stresses that damage a pavement structure, is expected to increase compaction uniformity as well as inspector safety and productivity (Kim et al., 2010).

1.3.2 QA/QC Acceptance

Although considerable research has been conducted in an effort to compare measured soil property values to those obtained by the equipment selected for this research, the literature review did not reveal precise correlations necessary to employ these methods in the QA/QC acceptance process. One of the main objectives of this research is to evaluate alternative methods of soil compaction control under controlled conditions.

Numerous studies related to compaction control have been conducted by state DOTs, universities, and national research organizations. The results of these previous studies have been determined to be acceptable in this research and were used to reduce duplication in the overall testing effort, allowing focus on the most viable test methods for specific materials and conditions found in Florida.

1.4 Laboratory Testing

Laboratory and field testing designed into this research project builds on the findings from the literature review. Phase I testing was conducted in a soil test box and soil test pits. Phase II testing continued the testing started in Phase I. Phase III field testing utilized the devices that exhibited the best performance characteristics in the previous two phases.

1.4.1 Phase I Test Box and Test Pits

Phase I testing was conducted in order to evaluate in-place soil testing devices and their potential for determining in-place soil moduli. An aluminum test box and soil test pits located at the FDOT's State Materials Office (SMO) were utilized as part of this testing effort. Several soil types—A-3; low fines A-2-4 (12% passing the 200 mesh sieve); high fines A-2-4 (24% passing the 200 mesh sieve); stabilized subgrade (A-3 soil blended with limerock); and limerock base (calcium carbonate from the Ocala formation)—were examined.

The LWD, GeoGauge (also known as soil stiffness gauge [SSG]), Briaud compaction device (BCD), "dirt" seismic pavement analyzer (DSPA), Clegg impact soil tester (CIST), and DCP were evaluated.

1.4.2 Phase II Test Pits

Phase II also was conducted at the SMO test pit facility. This phase focused on accuracy defined as the difference between true and measured values. The single spot testing sequence (SSTS) approach was used in an attempt to minimize the variability expected from separate testing locations in the test pit. Density, moisture, and particle size are the important variables that this method would help control. The data from each device were compared to those obtained from an adjacent 12-in. static PLT modulus measurement, an NDG test measurement, and a laboratory resilient modulus (M_R) test run in the FDOT State Materials Laboratory from a material sample obtained as close as possible to the equipment evaluation test location.

1.5 Phase III Field Testing

Phase III consisted of the use of test sections on FDOT roadway construction projects. Test sections were selected based on their capability of providing a wide range of materials and conditions comparable to those obtained in the pit testing results from Phase II.

1.6 Data Collection and Analysis

Phase I pertained to the precision or repeatability of an individual piece of equipment under controlled conditions. Statistical analyses, such as the coefficient of variation (COV), were used to identify the equipment exhibiting the highest level of precision. Phase II pertained to accuracy when the measurements obtained from the selected equipment were compared to soil test values such as the in-place dry density determined by the NDG, the modulus determined by the static PLT, and the M_R test run in the laboratory. The measured soil values were used as the baselines for statistical comparisons. Phase III consisted of equipment testing using LWDs, a DCP, a GeoGauge, and a CIST. Comparisons of the measurements obtained by the equipment being evaluated to the measurements determined by the static PLT, the M_R test run in the laboratory, and the in-place dry density determined by the NDG were conducted.

Specifically, the COV was calculated in Phase I to assess relative variation for test results obtained from the different pieces of equipment. Correlation analysis was conducted in Phase II to determine the strength of the linear relationship and to test the significance of such a relationship between laboratory-tested M_R and results from the different pieces of equipment used for the in-place testing. Regression analysis was conducted further to quantify the linear relationship between the laboratory-tested M_R and equipment measurements for different soil types under different moisture conditions. The prediction expressions were generated from the regression models and validated using in-place test results in Phase III.

Chapter 2: Literature Review

The method for determining the in-place (field) measurements required for acceptance testing typically is accomplished through the use of the NDG. For decades, the NDG has been used for earthwork construction acceptance by both the contractor and the FDOT. A sample of each material is laboratory tested to determine its dry density. From this, an established means exists to compare field density results to the laboratory density value. Laboratory procedures include the Standard Proctor Test, Standard Method of Test for Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in. Drop (AASHTO, 2010) or the Modified Proctor Test, Standard Method of Test for Moisture-Density Relations of Soils Using a 10-lb Rammer and an 18-in. Drop (AASHTO, 2010). The assumption is that if the material is well compacted and has a high in-place density, it also will meet the minimum design requirements.

Concerns, however, exist:

1. Density is not a function of M_R. Therefore, field measurements are related indirectly to design criteria.
2. NDG owners and operators are subject to numerous licensing and training requirements per their radiation safety programs.
3. New and emerging technologies are available to measure in-place modulus/stiffness.

2.1 Review of Previous Research

Historically, pavement design relied on empirical procedures. Layer types and their dimensions were based on the American Association of State and Highway Transportation Officials (AASHTO) road tests performed during the 1950s. Subgrade, base, and surface layers were selected based on their strength and the strength of the underlying materials.

The *AASHTO Guide for Design of Pavement Structures* (AASHTO, 1993) incorporated the M_R of component materials into the design process. In 2002, the AASHTO MEPDG included the M_R of each supporting layer (embankment, subgrade, and base) in the design process. In 2008, AASHTO published the *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* and in 2011 released the first version of the accompanying software. The *Design Guide* and software are based on M-E principles and, as such, are a significant departure from the previous empirically based pavement design procedures. Moving from empirical to M-E design procedures provides a number of advantages, including the evaluation of a broader range of vehicle loadings, material properties, and climatic effects; improved characterization of the existing pavement layers; and improved reliability of pavement performance predictions (Pierce and McGovern, 2014).

2.1.1 Terminology

A goal of this research was to determine if a measurement of modulus or stiffness can be used to ensure uniform compaction from the top of the layer(s) to the bottom of the layer(s). By doing so, there will be assurance that each soil component has adequate load and deformation characteristics that will allow the highway system to perform as needed.

Stiffness and modulus (Young's modulus, *E*, or resilient modulus, M_R) are two different mechanistic properties. Density, the current acceptance criteria, is not a measure of either E or M_R.

2.1.2 Theory

Elastic modulus often is referred to as Young's modulus after Thomas Young, who published the concept in 1807. *E* can be determined for any solid material and represents a constant ratio of stress and strain (stiffness).

$$E = \frac{stress}{strain} \qquad \text{(Eq. 2.1)}$$

A material is elastic if it is able to return to its original shape or size immediately after being stretched or squeezed. Almost all materials are elastic to some degree as long as the applied load does not cause it to deform permanently. Thus, the "flexibility" of any object or structure depends on its elastic modulus and geometric shape.

The elastic modulus is basically the slope of its stress-strain plot within the elastic range. The initial straight-line portion of the curve is the elastic range (Pavement Interactive 2007).

Resilient modulus (M_R) is a measure of stiffness. M_R is a design parameter for pavement systems. Knowledge of the M_R of each pavement layer will determine how the pavement responds to loading. M_R is defined as the ratio of the applied deviator stress to the recoverable or "resilient" strain. When a load is applied to a material, stress occurs. This stress is equal to the load divided by the contact area. While the stress may remain constant, the magnitude of deformation due to the load will vary. The properties of the material—in our case, soil—influence the deformation. A portion of the deformation may be recoverable or "resilient" while the remainder is unrecoverable or "plastic."

Strain is a critical design consideration. It is the ratio of the object's deformation to its original dimension in a given direction. It can be measured vertically, horizontally, or longitudinally. Loading a soil will cause it to compress. Unloading it will allow it to rebound. By measuring these distances, recoverable resilient strain and unrecoverable resilient strain values can be obtained (Buchanan, 2007; AASHTO, 2006).

M_R generally is obtained through laboratory testing. A commonly used procedure is AASHTO T 307, Standard Method for Determining the Resilient Modulus of Soil and Aggregate Materials (AASHTO, 1999).

2.2 State Departments of Transportation

Since the 1990s, researchers and inventors have attempted to discover equipment and methodology to replace the NDG. Although several devices are available commercially, many of which exhibited great initial promise, subsequent trials often disputed earlier claims of accuracy and repeatability. This lack of confidence has led to limited use by both agencies and contractors. Numerous studies have been conducted and/or funded by federal and state agencies, universities, trade groups, and manufacturers. Although significant amounts of data have been generated, there is no consensus on the equipment and test procedure needed for QA/QC in-place soil compaction acceptance criteria other than the NDG and the sand-cone method.

The development of procedures for comparing the properties of the in-place earthwork construction to the design criteria is ongoing. The study of M-E design (performance-based specifications) continues not only by the FDOT but by DOTs of other states, such as Minnesota, Indiana, Missouri, and Nebraska. A previous survey indicated that all state DOTs with the exception of Minnesota are using the NDG to determine acceptable soil compaction or density. (Mooney et al., 2008). The need for a proven method of determining in-place modulus is recognized on both state and national levels.

2.2.1 Minnesota Department of Transportation

The Minnesota Department of Transportation (Mn/DOT) supports ongoing research for development of alternative methods to determine in-place soil properties. Research pertaining to the DCP dates back to the early 1990s. From those efforts, Mn/DOT has adopted DCP specifications that require a maximum allowable penetration index based on the type of soil tested. In the 1990s Mn/DOT also participated in the initial evaluations of the GeoGauge/soil stiffness gauge (SSG) (Siekmeier et al., 2000; Burnham and Johnson, 1993; Burnham, 1997; Siekmeier et al., 1999; Davich et al., 2006; Swenson et al., 2006).

Mn/DOT research has included significant work with LWDs leading to the adoption of an LWD pilot specification. This specification requires either the construction of a control strip to find an LWD target value or comparison testing whereby LWD results are compared to the results from other test methods such as the DCP or sand cone. Target values for both the DCP and LWD were compiled from extensive field testing (Siekmeier et al., 2009). Similar tables from the Siekmeier project are included in NCHRP Synthesis 456 (Nazzal, 2014).

Modified Dynamic Cone Penetrometer Method
http://www.dot.state.mn.us/materials/gradingandbasedocs/DCP/TestProceduremod.doc

Lightweight Deflectometer Method
http://www.dot.state.mn.us/materials/gradingandbasedocs/LWD/2105_2106_2211_2215_LWD01_032812.pdf

Text versions of Mn/DOT test procedures and specifications can be found in Appendix H of this report.

2.2.2 Indiana Department of Transportation

In an attempt to replace the NDG, the Indiana Department of Transportation (INDOT) has conducted both laboratory and field research projects. Based on the findings from their research, as of March 2014, INDOT has disposed of 92 of their 106 NDGs. The NDGs were replaced with 60 Zorn LWDs and 117 Kessler DCPs. These LWDs and DCPs are used in the field for soil compaction acceptance (Siddiki, 2014). Test methods are listed as follows.

ITM-508: Field Determination of Deflection Using Lightweight Deflectometer
http://www.in.gov/indot/div/mt/itm/pubs/508_testing.pdf

ITM-509: Field Determination of Strength Using Dynamic Cone Penetrometer
http://www.in.gov/indot/div/mt/itm/pubs/509_testing.pdf

Links to the field testing methods for determination of moisture and maximum dry density for use with the LWD and DCP test methods are provided as follows.

ITM-506: Field Moisture Determination
http://www.in.gov/indot/div/mt/itm/pubs/506_testing.pdf

ITM-512: Field Determination of Maximum Dry Density and Optimum Moisture Content of Soil
http://www.in.gov/indot/div/mt/itm/pubs/512_testing.pdf

Text versions of ITM-508 and ITM-509 can be found in Appendix H of this report.

2.2.3 Missouri Department of Transportation

Missouri Department of Transportation (MoDOT) Specification Section 304 Aggregate Base Course includes the use of the DCP for testing Type 7 aggregate base used in both roadway and shoulder construction. (www.**modot**.org/...**specs**/Sec0304) The text version of this specification can be found in Appendix H of this report.

2.2.4 Nebraska Department of Transportation

A 2011 study funded by the Nebraska Department of Roads (NDOR) investigated new QC and QA technologies for both hot mix asphalt (HMA) and soils. For soils, non-nuclear technologies—the electrical density gauge (EDG), the moisture density indicator (MDI), and the LWD—were investigated. Data analyses showed that the accuracies of the non-nuclear soil gauges were somewhat lower than that of the nuclear gauge. However, the researchers concluded that with an improved methodology to create soil models for the EDG and standardized ways to develop LWD target values, the EDG and LWD could have a similar or better accuracy than the NDG (Cho et al., 2011).

2.2.5 Florida Department of Transportation

The FDOT has an extensive research history pertaining to alternative soil compaction control devices (Bloomquist et al., 2003; Bloomquist and Ellis, 2008). The BCD, CIST, DSPA, and DCP have been used by FDOT personnel in either the laboratory or the field (Horhota, 1996) studied the spectral analysis of surface waves (SASW) test method for FDOT applications. The FDOT was one of the first DOTs to use the falling weight deflectometer (FWD) in the early 1980s.

Additional summaries of research done by others can be found in Appendix A of this report.

2.3 Equipment and Procedures

Although many of the devices used to measure compaction control of soils have been available for several years, their use in state DOT QA/QC acceptance decisions is limited. Information for these devices follows as well as a description of the test method and applicable American Society for Testing and Materials (ASTM) procedure, if available. Similar methodologies have been explored by the hot mix asphalt industry (Von Quintus et al., 2009).

2.3.1 GeoGauge/Soil Stiffness Gauge (SSG)

- Description: The GeoGauge or SSG induces small soil displacements using a harmonic oscillator. Sensors are used to measure associated force and displacement. These data are used to compute soil stiffness. Low-strain cyclic loading is applied by the apparatus about a static load. This method applies to silty and clayey materials containing greater than 20% fines. In such cases, the relationship between stiffness and dry density or dry unit weight is sensitive to the water content. The stiffness and modulus of silty and clayey materials also change with moisture content. In addition, for silty and clayey materials with high concentration of fines, higher stiffness does not necessarily ensure adequate compaction. (Humboldt, 2007).
- Our research using the GeoGauge has indicated that it is simple to use and provides stiffness and modulus values. Proper seating of the device's foot on the surface is critical for accurate measurements.
- ASTM D6758-08 Standard Test Method for Measuring Stiffness and Apparent Modulus of Soil and Soil-Aggregate In-Place by Electro-Mechanical Method. The apparatus and procedure described by this standard provide a means for measurement of the stiffness of

a layer of soil or soil-aggregate mixture from which a Young's modulus may be determined for an assumed Poisson's ratio. The stiffness, in force per unit displacement, is determined by imparting a small, measured force to the surface of the ground, measuring the resulting surface velocity and calculating the stiffness. This is done over a frequency range, and the results are averaged (ASTM, 2008).

- When referring to the GeoGauge, manufactured by Humboldt Scientific, Inc., earlier literature tends to use the term *Soil Stiffness Gauge* or *SSG*. Later literature typically refers to the device by its trade name, the GeoGauge. The terms *Soil Stiffness Gauge*, *SSG*, and *GeoGauge* are used interchangeably throughout this document.

2.3.2 Dynamic Cone Penetrometer (DCP)

- <u>Description:</u> During a DCP test, a slender shaft is driven into compacted subgrades and bases using a sliding hammer weight. The rate of penetration is measured. Data from tests are analyzed to produce a penetration index. In brief, the DCP is a miniature version of the Standard Penetration Test (SPT) method (Kessler, 2010).
- Our research using the DCP has provided an indication that it is inexpensive due to its simple construction and does not require software for operation. The DCP used in this study, however, was equipped with a magnetic ruler allowing for data to be collected electronically. The DCP was ineffective on stabilized subgrade and base materials due to difficulty in penetrating those layers to the required depth, a minimum of 6 in. Per ASTM D6951, the standard hammer is 8 kg [17.6 lb]. There is an optional hammer weighing 4.6 kg [10.1 lb]. Only the 8-kg [17.6-lb] hammer DCP was used in this research project.
- <u>ASTM D6951-09 Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications.</u> This test method is used to assess in situ strength of undisturbed soil and/or compacted materials. The penetration rate of the 8-kg DCP can be used to estimate in situ California Bearing Ratio (CBR) as well as to identify strata thickness, shear strength of strata, and other material characteristics. The DCP typically is used to assess material properties down to a depth of 39 in. The DCP can be used to estimate the strength characteristics of fine- and coarse-grained soils, granular construction materials, and weak stabilized or modified materials. The DCP cannot be used in highly stabilized or cemented materials or for granular materials containing a large percentage of aggregates greater than 2 in. The DCP may be used to assess the density of a fairly uniform material by relating density to penetration rate on the same material. In this way, undercompacted or "soft spots" can be identified, even though the DCP does not measure density directly. A field DCP measurement results in a field or in situ CBR and will not normally correlate with the laboratory or soaked CBR of the same material. The test is thus intended to evaluate the in situ strength of a material under existing field conditions (ASTM, 2009).

2.3.3 Lightweight Deflectometer (LWD)

- Description: Similar to the full-scale falling weight deflectometer, the LWD utilizes both dynamic force and velocity measurements to measure soil stiffness. Transducer and accelerometer measurements are converted to elastic stiffness (Young's Modulus) of the base or subgrade system using equations that assume underlying layers as homogeneous elastic half-space. The portable LWD typically consists of a mass (often 10 kg), an accelerometer or geophone, and a data collection unit. Some uses of data include QA/QC of compacted layers, structural evaluation of load carrying capacity, and determination of thickness requirements for highway and airfield pavements. LWDs are designed to be small enough to be moved and operated by one person (Zorn, undated; Dynatest, 2006).
- Our research using LWDs provided an indication that they are more difficult to use and less reliable relative to other devices.
- ASTM E2583-07 (2011) Standard Test Method for Measuring Deflections with a Lightweight Deflectometer (LWD). This test method covers the determination of surface deflections as a result of the application of an impulse load. The resulting deflections are measured at the center of the applied load and also may be measured at various distances away from the load. Deflections may be either correlated directly to pavement performance or used to determine in situ material characteristics of the pavement layers (ASTM, 2011b).
- ASTM E2835-11 Standard Test Method for Measuring Deflections Using a Portable Impulse Plate Load Test Device. This test method uses plate deflection resulting from the application of an impulse load. The deflection is measured at the center of the top of the load plate. If the load plate is in "perfectly uniform" contact with the unbound material under the plate, then deflection of the load plate should be equal to the deflection of the surface of the unbound material under test. However, with typical unbound materials, a 100% uniform contact seldom can be achieved. Accordingly, the test surface shall be as clean and smooth as possible with loose granules and protruding material removed. For gravel surfaces, it is recommended that a thin layer of fine sand be placed over the test point so that an approximately uniform contact between the load plate and the surface is established (ASTM, 2011a).

2.3.4 Clegg Impact Soil Tester (CIST)

- Description: The CIST, also known as the Clegg hammer, was developed by Dr. Baden Clegg in the Department of Civil Engineering at the University of Western Australia in the 1970s. The device consists of two basic components—a flat-ended cylindrical mass (i.e., hammer) and a guide tube. Four hammer masses are available—4.5 kg (standard), 2.25 kg (medium), 0.5 kg (light), and 20 kg (heavy). The 4.5-kg mass is the "general purpose" hammer for roadworks, earthworks, airstrips, etc. The two lighter hammers are used primarily for turf or sand testing. The heavy hammer is for testing through a larger zone or on the top course of flexible pavements. The mass is dropped manually from a predetermined height, and its deceleration is recorded as it makes impact with the ground surface. Deceleration rates are correlated to stiffness, and results—or an impact value (IV)—are displayed on a digital readout (Clegg, 2005).

- Our research using the CIST has provided an indication that it is simple to use and relatively inexpensive. The 4.5-kg standard weight device used in this research was ineffective on loosely compacted soils.

2.3.5 Seismic Pavement Analyzer (SPA)

- Description: The seismic pavement analyzer (SPA) is designed to determine the Young's modulus of elasticity and shear modulus of pavement layers. The portable SPA (PSPA) typically is used for pavement material properties, and the "dirt" SPA (DSPA) is used on constructed subgrades and bases to determine the layer properties. This device relies on the surface wave method, which was developed in the early 1980s (Dynatest, 2006).
- Our research using the DSPA provided an indication that it is more expensive than the other devices. It requires a field laptop computer for data collection, making it more difficult to use. Variability in the results, which often require interpretation, is an additional issue that requires a highly skilled operator.
- ASTM Standards: No ASTM standard is associated with this device.

2.3.6 Briaud Compaction Device (BCD)

- Description: The BCD uses strain gauges to measure the bending that occurs on a plate resting on the ground surface when a known mass is dropped on it. Results are correlated to soil modulus. Testing assumes that soil stiffness is proportional to plate bending. (Roctest, 2011).
- Our research using the BCD provided an indication that it is somewhat difficult to use due to the application of downward force by the operator needed to conduct the test. This difficulty in usage contributed to variability in results, particularly between operators.
- ASTM Standards: No ASTM standard is associated with this device.

Chapter 3: Phase I Test Box and Test Pit

3.1 Phase I Testing Plan

In Phase I existing technologies for determining in-place soil properties were investigated. The objective of this phase was to identify the devices that produce results equally or more precise than currently acceptable methods for verifying soil compaction density (e.g., the nuclear density gauge).

3.1.1 Equipment Procurement

A Zorn Model 3000 LWD and a Kessler Model K100 DCP were purchased by the FDOT prior to the commencement of equipment evaluation testing. The existing CIST units were found to be inoperative and unrepeatable. A new CIST was purchased during the second quarter of this project. The existing Carl Bro/Keros Prima 100 LWD also was found to be unreliable and unrepeatable. A Dynatest 3031 LWD was purchased during the third quarter of this project. This purchase followed thorough use of a similar LWD loaned to the FDOT by the supplier, Dynatest Equipment, Inc. Photographs of the Zorn LWD and Kessler DCP are presented in Figure 3.1.

For the purpose of this report, the LWD manufactured by Zorn Instruments (accelerometer-type) will be designated as LWD-1. The LWD manufactured by Dynatest Equipment, Inc. (geophone-type) will be designated as LWD-2. The LWD manufactured by Carl Bro/Keros will be designated as LWD-P.

Figure 3.1: Photographs of testing equipment purchased
(LWD-1, left; DCP, right)

The following devices were tested during Phase I of this study:

1. LWD-1
2. LWD-2
3. LWD-P

4. DCP
5. DSPA
6. BCD
7. CIST
8. GeoGauge
9. Mini-PLT
10. PLT

3.1.2 Material

To minimize variability, initial test box testing was limited to soil classified as A-2-4 according to AASHTO M 145 Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes. A native Florida soil obtained from the Orange Heights area was used. This soil is representative of a typical soil that meets the requirements for compacted embankment as specified by FDOT's Standard Specification for Road and Bridge Construction Section 120 Excavation and Embankment. Laboratory results indicate material previously stockpiled at the FDOT's SMO was suitable (see Figure 3.2).

Figure 3.2: Photograph of A-2-4 soil stockpiled at the FDOT's State Materials Office

3.1.3 Moisture

Optimum moisture and maximum dry density are determined for each soil as specified in AASHTO T 99. For testing, the following moisture contents will be used:

- Optimum
- ±2% above optimum
- ±2% below optimum

Target moistures are obtained by adding water to a mixer (Figure 3.3) filled with the selected soil. "Speedy" moisture tests (FM 5-507) and microwave oven heating (ASTM D4643) are used to verify moisture contents.

Figure 3.3: Photograph of mixer used to achieve target moisture contents

3.2 Test Box

Phase I equipment evaluations were conducted in a 3 ft. by 3 ft. by 2 ft. open-top aluminum container that was reinforced with aluminum channels to minimize flexure (see Figure 3.4).

Figure 3.4: Photograph of aluminum soil test box

3.2.1 Compaction

Before testing, a uniform compactive effort is applied to the material in the test box with a manual rammer. The compactive effort and moisture determination are detailed in Section 3.4.

3.2.2 Test Box Procedure

The following procedure was used for each material to be tested in the test box.

1. A sample of the material is obtained, and its moisture content is determined by oven-drying the specimen to a constant mass to accurately determine moisture content.
2. Sufficient material is obtained to fill the test box. Based upon results from (1), water is added to the soil, if needed, to achieve the correct water content.
3. Soil is placed in the test box in two 8-in. lifts. Each lift is compacted per the procedure outlined previously.

A moisture test also is conducted using the speedy moisture tester for informational purposes.

3.2.3 Test Box Testing Sequence

The testing sequence is critical. Some tests cause large soil disturbances while others are non-destructive. Therefore, the following sequence is used:

1. GeoGauge
2. BCD
3. DSPA
4. LWDs
5. DCP
6. CIST
7. NDG
8. Mini-PLT

The mini-PLT is conducted last. It is the most labor intensive and requires the most time to be completed. Because of this, it may be possible for the soil to dry, which would affect all tests after the PLT. Therefore, all other tests are completed prior to the mini-PLT to minimize the influence of moisture variation. When the test box is not in use (between tests), it is covered in Visqueen to minimize moisture loss.

Photographs of several devices are presented in Figures 3.5 through 3.9.

Figure 3.5: Photograph of LWD devices in east test pit
(From left to right: LWD-P, LWD-2, and LWD-1)

Figure 3.6: Photograph of DSPA in test box upper lift

Figure 3.7: Photograph of BCD in test box

Figure 3.8: Photograph of mini-PLT in test box with modified clamping devices

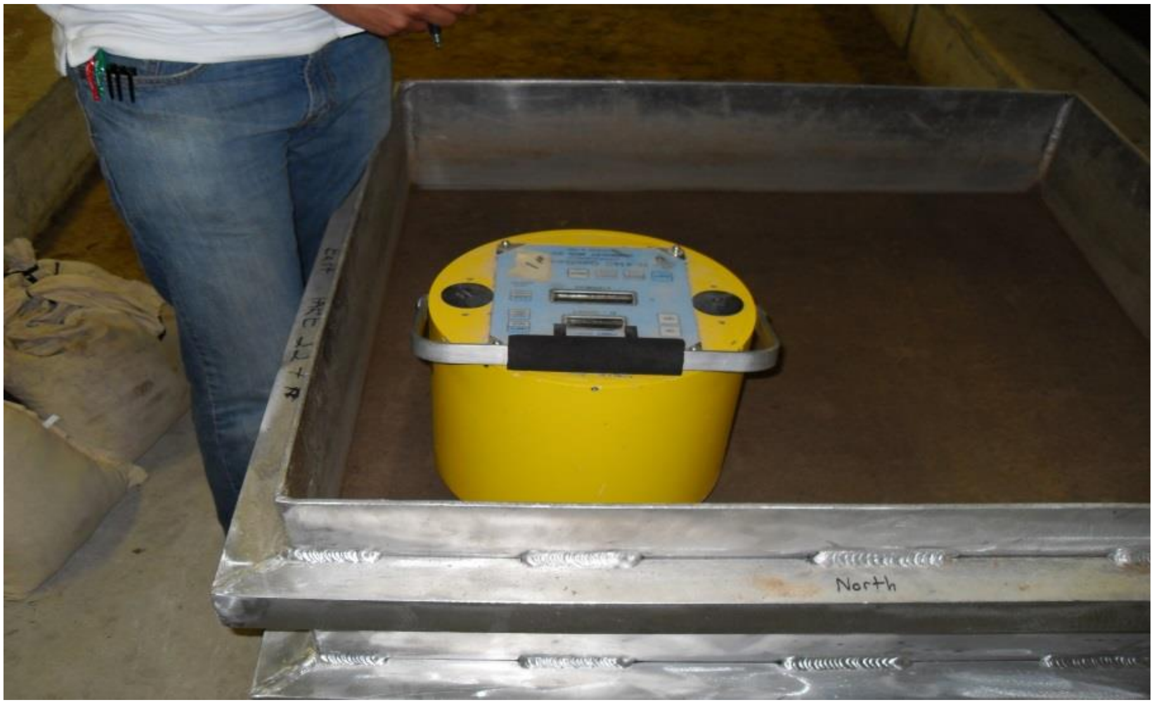

Figure 3.9: Photograph of GeoGauge in test box

3.3 Phase I Testing

3.3.1 Test Series 1 and 2

One of the goals of the first round of tests is to determine the depth of influence for each apparatus. To prepare the test box for these initial tests series, soil is mixed with water, brought to its optimum moisture content, added to the test box, and compacted to a lift thickness of 8 in. A test series is conducted where several pieces of equipment are tested. Next, another 8-in. compacted lift is added to the test box. A second test series is conducted. Moisture contents are verified with a microwave oven. Mass differences between moist and dry specimens are calculated (as specified in ASTM D4643). Moisture data also are collected using the NDG and speedy moisture tester (ASTM D4944); however, these data are not used for moisture adjustment.

Collecting data from the initial 8-in. soil layer helped investigators familiarize themselves with the various equipment used during this study. These data, however, were not used in the final statistical analysis. A summary of data collected during the first two test series is presented in Table 3.1.

Table 3.1: Data generated during first two test series - Phase I

Test Date	Test Types	Additional Comments
01-25-2013	NDG, wet density, percent moisture, dry density (typical), GeoGauge, DSPA, LWD-P, LWD-1, BCD, speedy moisture tester	Tests only on lower 8-in. compacted lift
01-28-2013	NDG	Tests on upper 8-in. compacted lift

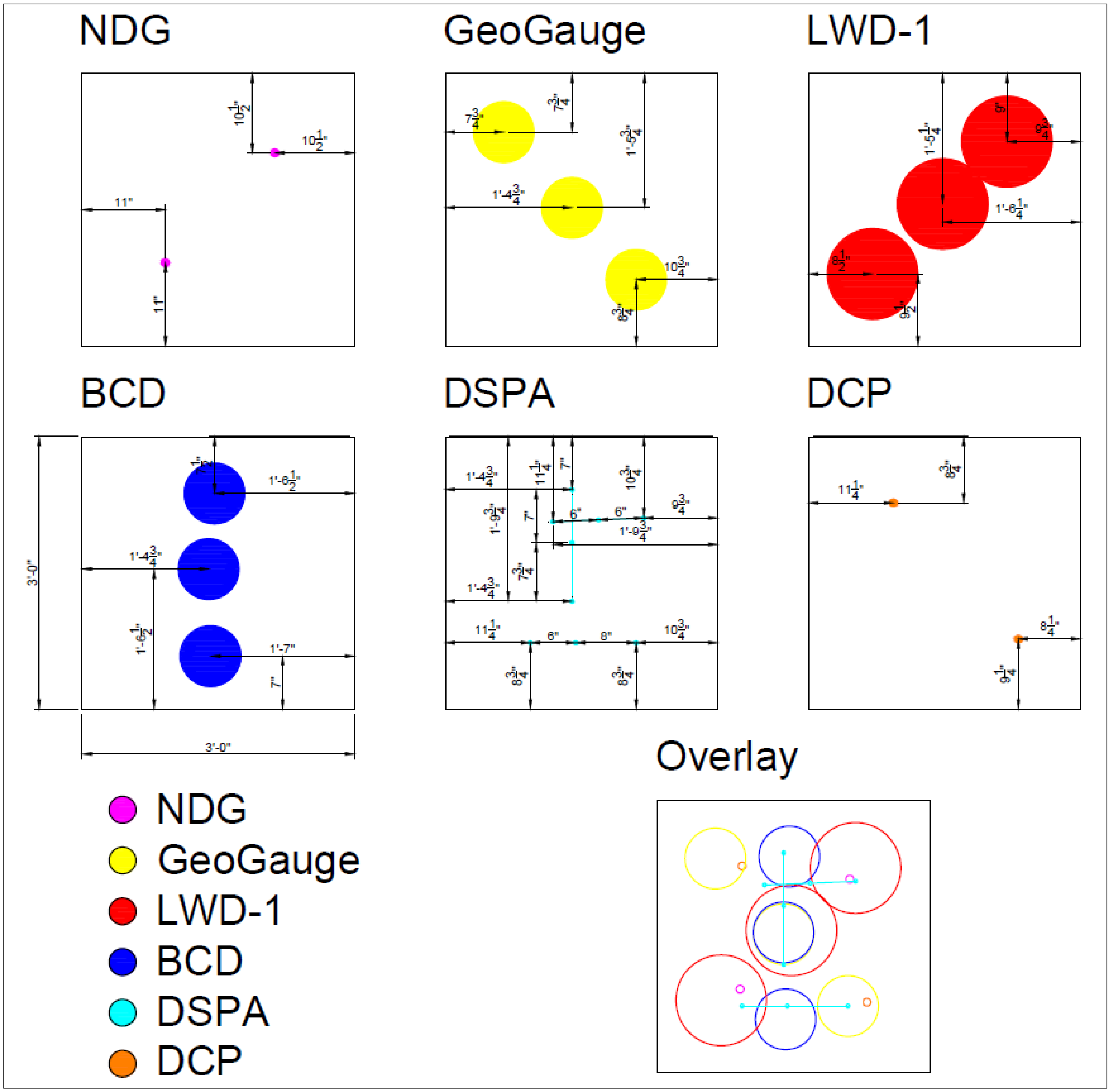

Figure 3.10: Testing device orientation in the test box

3.3.2 Test Box Preparation for Third Test Series

Test box preparation for the third test series is similar to procedures outlined previously. Soil is mixed, brought to optimum water content, and added to the test box in 8-in. (compacted) increments. However, tests are conducted on the entire 16-in. depth (as opposed to one test series per lift). Between tests, the test box is covered with 6-mil (0.006-in.) Visqueen to minimize moisture loss due to evaporation. A summary of information associated with this test series is presented in Table 3.2.

Table 3.2: Data generated during third test series - Phase I

Test Date	Test Types	Additional Comments
01-30-2013	NDG	Tests indicated that moisture had dropped below optimum (indicating draining)
01-31-2013	GeoGauge, DSPA, LWD-1, LWD-P, BCD, DCP	LWD-P malfunctioned
02-01-2013	LWD-P, mini-PLT, speedy moisture tester	LWD-P would not communicate with PDA; mini-PLT malfunctioned
02-04-2013	Percent moisture at upper lift, 8 in. below surface, bottom 8 in.	Data showed significantly higher water contents in the bottom of the test box (indicating draining)
02-06-2013	LWD-1, LWD-2, LWD-P	LWD-P continued to malfunction

These data appeared to indicate that water was draining from the top of the test box over time. As a result, a modified procedure was needed to ensure consistent moisture content during a test series.

3.3.3 Test Box Preparation for Subsequent Test Series

Because of the variability in moisture content, a new preparation procedure was developed. One-half of the test box soil (as determined by its mass) is removed from the box. This soil is transferred to the mixer, mixed for ten minutes, and brought to optimum moisture content. Meanwhile, the remaining soil is moved from the test box to a temporary storage unit. Once soil in the mixer is at optimum moisture, it is transferred from the mixer to the test box. This soil is compacted to an 8-in. lift and planed. The soil in the temporary storage unit is added to the mixer, mixed with water, and brought to its optimum moisture content. Once at optimum moisture, it is added to the test box, compacted to another 8-in. lift, and leveled. Test series are conducted on the full 16-in. soil depth and compared with NDG tests. A summary is presented in Table 3.3.

Table 3.3: Data generated during subsequent test series - Phase I

Test Date	Test Types	Additional Comments
02-08-2013	NDG, GeoGauge, BCD, DSPA, LWD-1, LWD-2, DCP, mini-PLT, speedy moisture tester	
02-12-2013	Speedy moisture tester	Box was allowed to sit idle for four days; moisture tested at various depths; results appeared to indicate some draining had occurred
02-15-2013	NDG, GeoGauge, BCD, DSPA, LWD-1, DCP, mini-PLT, speedy moisture tester	Box was prepared on 02-14-2013 and covered with Visqueen
02-26-2013	NDG, GeoGauge, LWD-1, BCD, DSPA, DCP, NDG, mini-PLT, speedy moisture tester, wet density, dry density	Box was prepared on 02-25-2013 and covered with Visqueen
03-01-2013	NDG, GeoGauge, BCD, DSPA, DCP, mini-PLT, speedy moisture tester	Box was prepared on 02-28-2013 and covered with Visqueen
03-07-2013	NDG, GeoGauge, BCD, DSPA, LWD-1, DCP, Mini-PLT	
03-08-2013	NDG, percent moisture, speedy moisture tester	Tests were conducted using box prepared from previous day

3.4 Compactive Effort and Moisture Determination

Required soil volume per lift is computed per Eq. 3.1:

$$V_{box} * \rho_{soil} = W_{reqd} \qquad \text{(Eq. 3.1)}$$

where V_{box} is the volume per lift; ρ_{soil} is the density of the soil at optimum moisture; and W_{reqd} is the weight of soil required. Based on the box surface area (35.4 in. by 35.5 in. = 1256.7 in.^{2}), an 8-in. lift, and an optimum density of 112 lbs. per cubic ft., approximately 5.818 cubic ft. of soil is required per lift.

Throughout mixing, moisture is determined by microwave oven. The amount of water needed for the optimum moisture content is then determined and adjusted. Research pertaining to moisture probes has been limited to a review of the literature including manufacturers' data. An easy-to-use, cost-effective moisture probe producing results to the desired accuracy was not identified.

3.5 Phase I Test Pit Equipment Evaluations

3.5.1 Test Pit Description

Phase I equipment evaluations were conducted in FDOT's SMO east test pit. This 24 ft. by 9 ft. test pit is underlain by approximately 24 in. of sand/river rock. The test pit is equipped with a system to control its water table (Figures 3.11 and 3.12). However, investigators' previous experience in the test pit has shown that this water-level adjustment mechanism is approximate. Therefore, moisture contents are verified throughout Phase I testing.

Figure 3.11: Photograph of east test pit sump pump
(Used to control water level)

Figure 3.12: Photograph of east test pit Magnetrol water level control system

3.5.2 Test Pit Testing Plan

During Phase I testing, three levels of moisture content per soil type—approximately at optimum moisture, slightly above optimum moisture, and slightly below optimum moisture—are used. Each piece of equipment was tested at 24 locations at each of the moisture percentages. Careful measurement ensured proper spacing from the pit walls and from previous tests. See Figure 3.13.

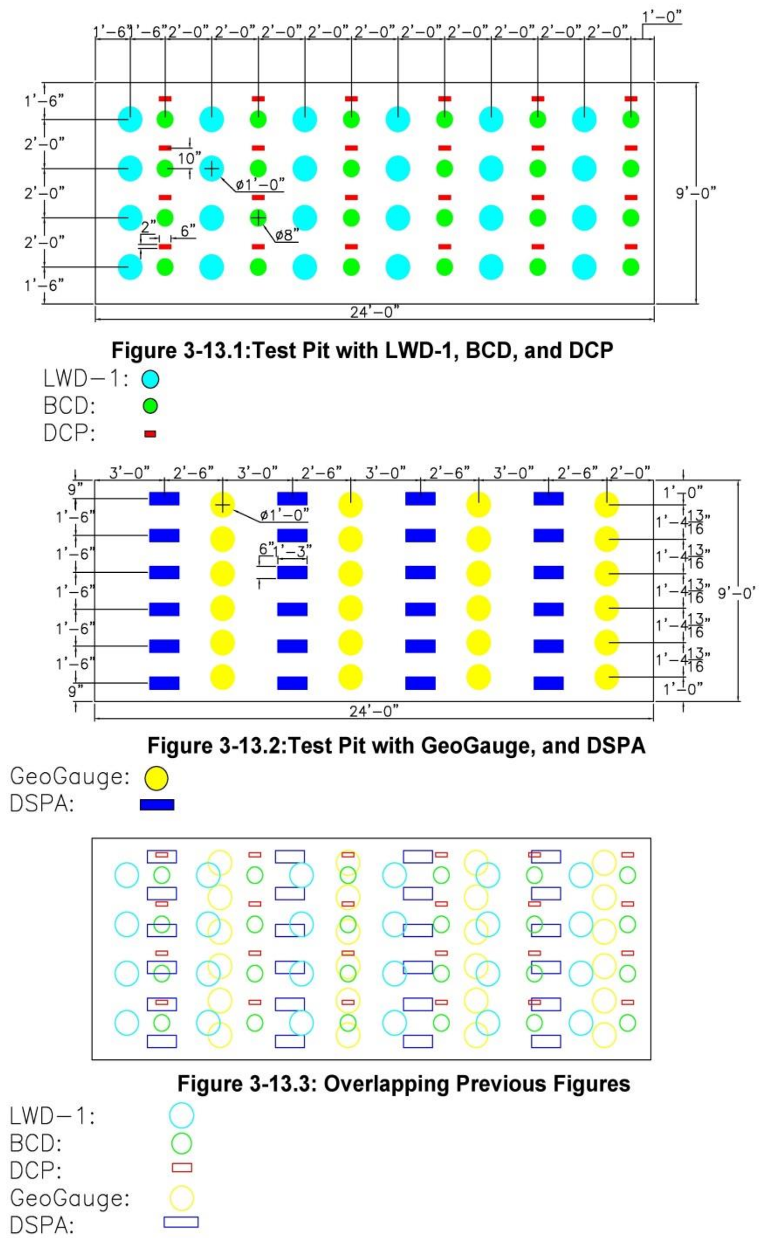

Figure 3.13: Plan of Phase I testing locations in the test pit

3.6 Phase I Test Pit Testing

Table 3.4 indicates test type and test dates for the A-3 soil in the east test pit. Photographs of testing are presented in Figures 3.14 through 3.16.

Table 3.4: Phase I tests: east test pit A-3 soil

Test Date	Test Type
03-13-2013	LWD-1, BCD
03-14-2013	DCP
03-15-2013	DSPA, GeoGauge
03-18-2013	12-in. static PLT

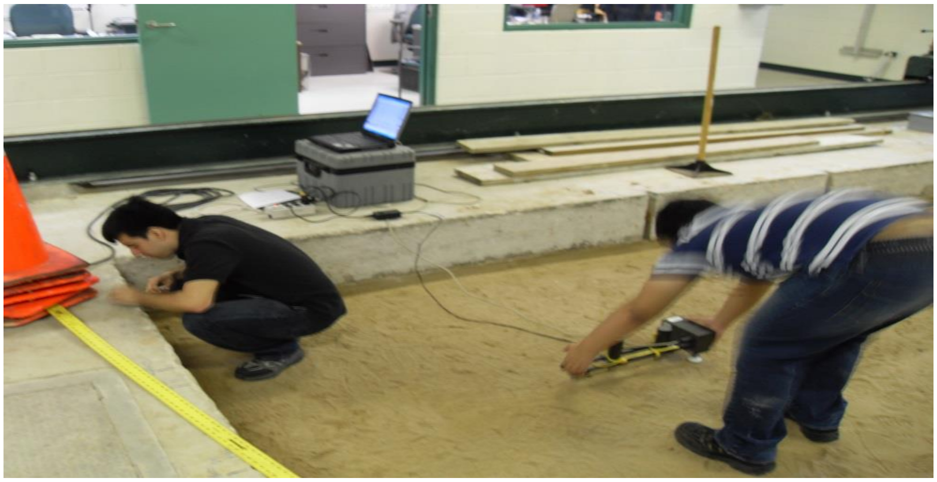

Figure 3.14: Photograph of east test pit layout and DSPA

Figure 3.15: Photograph of east test pit and DCP

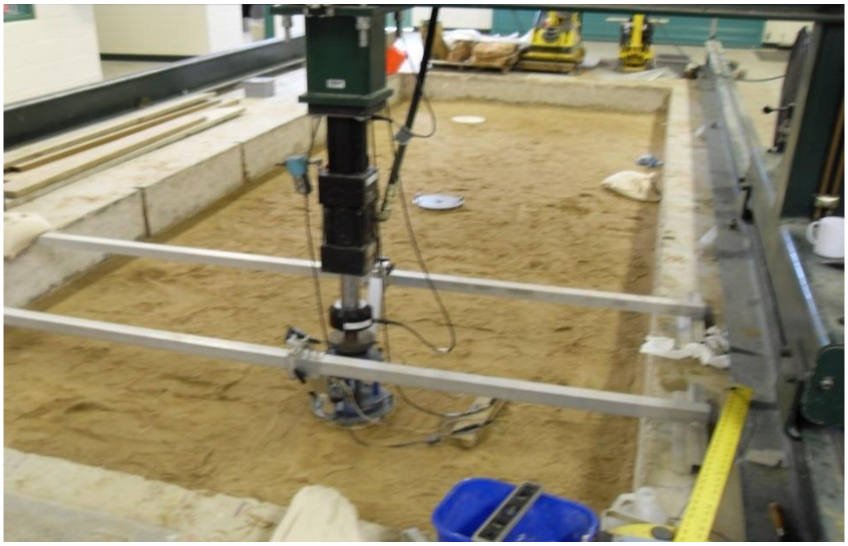
Figure 3.16: Photograph of east test pit and 12-in. static PLT

3.6.1 Test Pit East

Soil types and moisture conditions at the time of testing are provided in the following list. Initial testing was conducted first near optimum moisture content, and then moistures were adjusted to test the soil type below and above optimum levels, if possible. Moisture content information can be found in Appendix B.

- A-3 soil near optimum moisture
- A-3 soil slightly below optimum moisture
- Stabilized subgrade near optimum moisture
- Stabilized subgrade below optimum moisture

3.6.2 Test Pit West

Initial testing was conducted near optimum moisture content. Moistures were adjusted to test the soil type below and above optimum content, if possible. The A-2-4 soil type contained approximately 24% passing the No. 200 mesh sieve. Testing was conducted at the (approximately) optimum moisture content. For this material, a higher than optimum moisture content level was not reached even though the pit was flooded and the water level was brought up to 2 in. below the surface of the material. The pit was left in this condition for several weeks. After numerous discussions, it was determined with the concurrence of the FDOT that it was not practicable to achieve above-optimum moisture content for the A-2-4 soil with the high percentage of material passing the No. 200 mesh sieve. The approximate moisture content at the time of testing can be found in Appendix B.

- A-2-4 soil at optimum moisture

- A-2-4 soil at less than optimum moisture

Figure 3.17: Photograph of east test pit and static PLT

3.7 Static Plate Load Testing

Both the east and west soil test pits are configured to allow automated 12-in. diameter static plate load testing. Utilization of this system follows the completion of each series of portable equipment tests. A test series is deemed complete after each of the testing devices has completed testing at all predefined testing locations within the pit and at a specific moisture content. When testing material that is in the full (24-ft. length) pit, three PLTs are then run. In some instances, a test pit is divided in half, allowing two types of materials to be tested while still obtaining a valid statistical sample. When testing material in the half-pit configuration (12-ft. length), two PLTs are run. A summary of the static PLTs is provided in Appendix B.

3.8 Test Box Static Plate Load Testing

Phase I testing included the soil test box and 6-in. diameter plate load testing. This was done to compliment the test pit standard 12-in. PLTs and to compare the data from both tests to determine if statistically similar results can be achieved. Test box preparation is similar to the procedures described previously with the material placed and compacted near optimum moisture. For this series of tests, the MTS load frame and actuator system in the east test pit provide the loading and data collection. The soil level in the test pit is lowered to accommodate the application of the PLT apparatus directly on the test box material. Two tests are run for each soil type. Loading plate locations are orientated in opposite and diagonal quarters of the test box surface.

Six-in. diameter PLTs in test box:

- A-3 soil near optimum moisture
- A-2-4 high fines soil near optimum moisture
- Stabilized subgrade near optimum moisture
- Limerock base near optimum moisture
- A-2-4 low fines soil near optimum moisture

Figure 3.18: Photograph of 6-in. diameter PLT in test box placed in east test pit

3.9 Test Pit Results

Test results and analyses are presented in Appendix B.

Chapter 4: Phase II Test Pit Second Series

4.1 Equipment

The devices that were evaluated in Phase I and passed on to Phase II for data acquisition were:

- LWDs
- GeoGauge
- DCP
- CIST
- 12-in. static PLT
- NDG

Also included:

- Oven-dried moisture
- Testing at the SMO Soil Lab
    - Resilient modulus (M_R)
    - Gradation
    - Maximum density
    - LBR

4.2 Moisture

All tests are designed to be made on prepared, compacted material. Moisture is a crucial parameter that can affect properties and results of soils such as modulus, density, and stiffness. Soils are tested at two different moistures—optimum and below optimum. As shown in Section 3, the task of achieving an above-optimum moisture content for soils with a high content of material passing the 200-mesh sieve can be long and tedious. For A-3 soils, this is not the case. However, testing must commence as soon as possible after obtaining the desired moisture content as drain down, lowering the moisture content, begins.

4.3 Testing Sequence

Although every effort is made to minimize variability during placement and compaction of the materials in the test pits, test data indicate that variability exists throughout the test pit. In-place moisture is a key variable. Following compaction and throughout the equipment testing period, monitoring the water table elevation and sampling for moisture percentage verification are ongoing tasks. Although variability cannot be totally eliminated, the Single Spot Testing Sequence (SSTS) was designed to minimize the effects of material and moisture variability. An SSTS consists of testing each piece of portable equipment, as selected based on the data generated thus far. In an attempt to minimize disturbance of the soil surface, a 12-in. diameter static PLT is run at each location prior to the operation of the portable equipment. Following completion of the PLTs, the portable equipment listed in Section 4.1 is operated at each location per the applicable test method. Typically, during placement but before compaction, a soil sample of at least 30 lb. is obtained from each site for laboratory testing. Following completion of the portable equipment testing, a moisture sample is obtained from each test location and sealed in a moisture-proof container for oven drying to a constant mass. The SSTS concludes with the NDG determination of in-place density and moisture. The initial sample is used for laboratory resilient modulus (M_R), gradation, maximum density (proctor), and limerock bearing ratio (LBR) testing. This procedure ensures that inherent variation found throughout the test pit is minimized. Results from each piece of portable equipment are evaluated in terms of accuracy and precision based on statistical comparisons with the PLT, M_R, and NDG values obtained from the same area of the test pit and under the same conditions—namely, moisture content at time of testing. Four materials—stabilized subgrade, A-2-4 soil, limerock base, and A-3 soil—were used for SSTS evaluation.

4.4 Test Pit East

The soil types and moisture conditions at the time of testing are provided in the following list. Typically, testing is conducted first at optimum moisture content. The moisture is then adjusted in order to test the soil type at a moisture not near the optimum percentage. This east test pit was divided into two half sections by placing an east-west partition, making each section approximately 12 ft. long by 9 ft. wide.

- Stabilized subgrade between 2% and 4% below optimum moisture
- Stabilized subgrade at ±1% of optimum moisture
- Limerock base between 2% and 4% below optimum moisture
- Limerock base at ±1% of optimum moisture
- Stabilized subgrade at 9.5% moisture compared to optimum (11.0%) condition
- Stabilized subgrade at 6.5% moisture compared to optimum (11.0%) condition
- Limerock base at 9.8% moisture compared to optimum (12.0% average) condition
- Limerock base at 8.6% moisture compared to optimum (12.0% average) condition

4.5 Test Pit West

The soil types and moisture conditions at the time of testing are provided in the following list. As with the east test pit, testing typically is conducted first at optimum moisture content. The moisture is then adjusted in order to test the soil type at a moisture not near the optimum percentage. The west test pit was not partitioned. Soil placement and compaction were contiguous throughout the entire length of the pit.

- A-3 soil at ±2% of optimum moisture
- A-3 soil at approximately 7% less than optimum moisture
- A-2-4 soil at 9.2% (oven-dried) moisture compared to optimum (11.0%) condition
- A-2-4 soil at 6.1% (oven-dried) moisture compared to optimum (11.0%) condition
- A-3 soil at 9.1% moisture compared to optimum (11.9% average) condition
- A-3 soil at 3.4% moisture compared to optimum (11.9% average) condition

Additional moisture information is provided in Appendix C.

Figure 4.1: Photograph of east test pit south limerock base compaction

Figure 4.2: Photograph of east test pit north stabilized subgrade, NDG for in-place density

Figure 4.3: Photograph of east test pit looking south
(Test equipment from left: DCP [in case], LWD-1, CIST, NDG, GeoGauge)

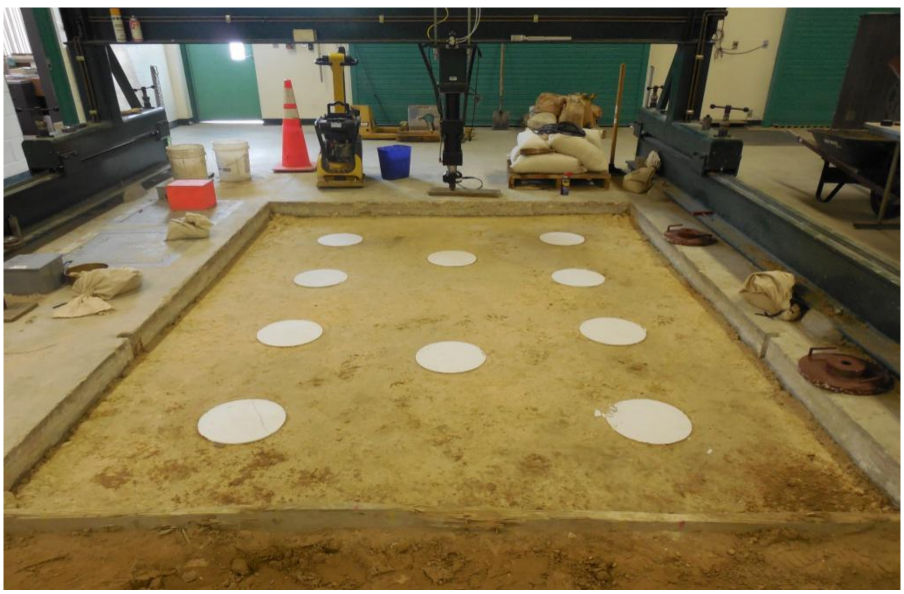

Figure 4.4: Photograph of east test pit limerock base ten PLT locations per half pit

Figure 4.5: Photograph of west test pit A-2-4 soil
(Locations for 24 PLTs: 12 per moisture percentage. NDG for in-place densities and moistures)

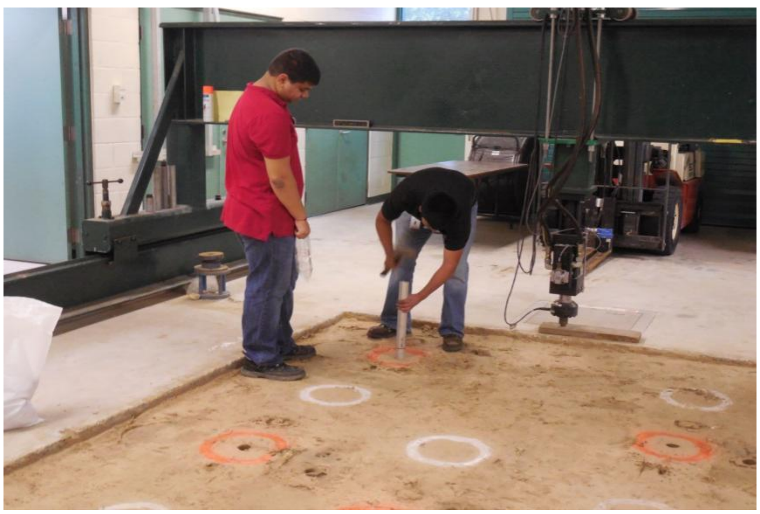

Figure 4.6: Photograph of east test pit limerock base ten moisture samples
(One per each SSTS location)

Figure 4.7: Photograph of oven drying for moisture content determination

4.6 Static Plate Load Testing

Both the east and west soil test pits are configured to allow automated static PLTs. Utilization of this system occurs prior to equipment testing. The west pit was not partitioned, allowing for 24 PLTs for each material. In this instance, the PLT locations are carefully located to allow for 12 PLTs at the initial moisture and an additional 12 PLTs at the second moisture. The sequence is PLTs, equipment testing, water table adjustment, PLTs at the new moisture percentage, and then equipment testing at the adjusted moisture percentage. The east pit, as indicated before, is divided in half. Ten PLTs are run prior to equipment testing. In order to test the material at a second moisture content, the material must be removed and replaced with similar material. Following successful compaction, based on the appropriate laboratory maximum density and determined by the NDG, another set of ten PLTs are run. This is followed by a complete series of equipment tests. In all cases, the location of the PLTs and equipment tests is critical to ensure that a previous test did not influence the results of subsequent testing.

4.7 Resilient Modulus and Moisture

Per AASHTO T 307, resilient modulus (M_R) is a measurement of the stiffness and strength of the subgrade soils (AASHTO, 1999). From the work of previous researchers and the general knowledge of soil mechanics, the need to control moisture content—making comparisons at the same moisture percentages—is critical to determining both accuracy and precision of the equipment being evaluated when compared to modulus values obtained from proven test procedures.

Gudishala (2004) validated relationships between M_R, dry density, and moisture content. Soils were tested at different combinations of moisture content and dry density. It was shown that the M_R followed a trend similar to the dry density, increasing to its maximum value near optimum moisture then dropping as the moisture content continued to increase. This research project was part of a multiyear study (Seyman, 2003; Nazzal, 2003; Abu-Farsakh et al., 2004; Mohammad et al., 2008).

The National Cooperative Highway Research Program's "Guide for Mechanistic-Empirical Design, Resilient Modulus as Function of Soil Moisture-Summary of Predictive Models" cites that moisture, along with other factors, affects the M_R of unbound materials. All other conditions being equal, the higher the moisture content, the lower the modulus. Unbound materials used in pavements typically are constructed with the moisture near the optimum, and then, with time, the moisture content reaches an equilibrium condition. It is recommended, in order to simulate this variation in the laboratory, to first compact the specimens at optimum moisture content and maximum dry density and then vary the moisture content (by soaking or drying) until the desired moisture content is achieved. Then, the M_R test should be performed (NCHRP, 2004).

Siekmeier et al. (2000) obtained Shelby tube samples from granular base material at the locations where the portable devices (DCP, LWD and SSG) were tested. Recognizing the importance of the moisture content prior to laboratory testing, the samples were returned to within 1% of the in situ moisture.

Hiltunen et al. (2011) determined that M_R is nonlinear with respect to effective confinement stress, loading strain, and moisture.

To date, there is limited literature showing correlations between the modulus values obtained from field equipment such as the LWD and the laboratory M_R (White et al., 2007).

4.8 Nuclear Density Gauge Testing

Both the ASTM D6938 and the AASHTO T 310 test methods provide information on precision using an NDG to determine in-place wet density. As a metric for equipment evaluation, the selected equipment for in-place soil evaluation should equal or exceed the precision and accuracy of the NDG (ASTM, 2010; AASHTO, 2013).

4.9 Data Retrieval

Every piece of equipment has a unique way of recording output. Appendices B, C, and E have graphs and summaries of the data collected and used for statistical analysis. Once the data were retrieved from each device, they were recorded immediately in a digital tablet. Recording data immediately after collection is done to ensure that all values are accounted for.

4.10 Data Analysis

Once all values for each soil type were collected, a statistical analysis was performed to determine the values that fell within a range of acceptance, which was set as $\pm 1.5*\sigma$ where σ is denoted as the standard deviation per device.

4.11 Method of Analysis

Testing the equipment for consistency was to ensure the accuracy of the results of the second portion of statistical analyses.

The first analysis was a direct correlation between values. A sample of this analysis is shown in Table 4.1.

Table 4.1: Correlation values calculated for A-3 at optimum moisture content

R - Values

	LWD-1	GeoGauge	DCP	PLT	NDG	Moisture
LWD-1	1.000	-0.434	0.207	-0.161	-0.416	-0.732
GeoGauge	-0.434	1.000	-0.075	-0.072	0.457	0.643
DCP	0.207	-0.075	1.000	-0.336	0.025	-0.429
PLT	-0.161	-0.072	-0.336	1.000	-0.179	-0.025
NDG	-0.416	0.457	0.025	-0.179	1.000	0.296
Moisture	-0.732	0.643	-0.429	-0.025	0.296	1.000

Table 4.1 shows the potential of a linear correlation corresponding to the given values. If the value is greater than 0, then a linear relationship may exist; but if it is relatively close to 0, another type of analysis must be done. From what can be seen in Table 4.1, a linear correlation is plausible. The next step of this analysis is to create a multiple regression model in which selected variables are defined.

Chapter 5: Phase III Field Testing

Phase III field testing is a continuation of the work performed in the previous two phases of this research project, Florida Department of Transportation (FDOT) Contract No. BDK75-977-72 and University of Florida (UF) Contract Nos. 00102168 & 00102731.

5.1 Site Selection

In November 2013, five ongoing construction projects located in the northeast quadrant of FDOT District 2 were selected for field testing. These projects provided a cross-section of different materials that typically are specified for roadway construction on FDOT projects and utilized by road building contractors following standard plans and FDOT specifications when under contract with the FDOT for such services.

The projects were inspected by UF and FDOT personnel. Arrangements were made with the Consultant Engineering and Inspection (CEI) representatives who coordinated project access and scheduling with the contractors.

Evaluations of the final group of portable in-place soil testing equipment selected from the Phase I and Phase II evaluations were in accordance with the approved testing plan. Field test data can be found in Appendices D and E.

5.2 Site Information

Information pertaining to the five projects (six sites) is provided in the following list. The list is in the order tested.

1. I-95 (SR-9) widening/overland bridges, Jacksonville. Ramp H, graded aggregate base (GAB) material was tested on December 18 and 19, 2013.
2. US-301 (SR-200) north of I-10, Callahan. A-2-4 embankment material for service road was tested on January 23 and 25, 2014.
3. SR-9B (I-295) new construction. A-3 embankment material was tested on April 5, 12, and 14, 2014.
4. CR-210 at US-1 new construction. Limerock stabilized subgrade material was tested on May 29, 2014.
5. SR-23 at Normandy Boulevard. Turn lane. Limerock base material was tested on July 8 and 9, 2014.
6. SR-23 at New World Avenue new construction. A-3 embankment material was tested on July 14 and 17, 2014.

5.3 Equipment

In this phase, the values generated by the selected portable equipment are compared to measured in-place values encountered on the various construction sites.

Selection of the field testing equipment utilized for Phase III includes:

- LWD-1
- LWD-2
- GeoGauge
- DCP
- CIST
- NDG
- PLT

5.4 Typical Field Site Setup

For each material, three sites are located approximately 100 ft. apart. (This will vary depending on site conditions.) One 12-in. diameter static PLT is the center of each site. The testing pattern for the portable equipment is a radial orientation at 90-degree increments around the PLT location. Each of the four quadrants contains three tests per piece of equipment for a total of 12 tests for each piece of equipment per test site. Thus, for each material, there are three PLT sites with 12 tests per site for each piece of portable equipment under evaluation.

Refer to Appendix D for detailed descriptions of the testing conducted at each site.

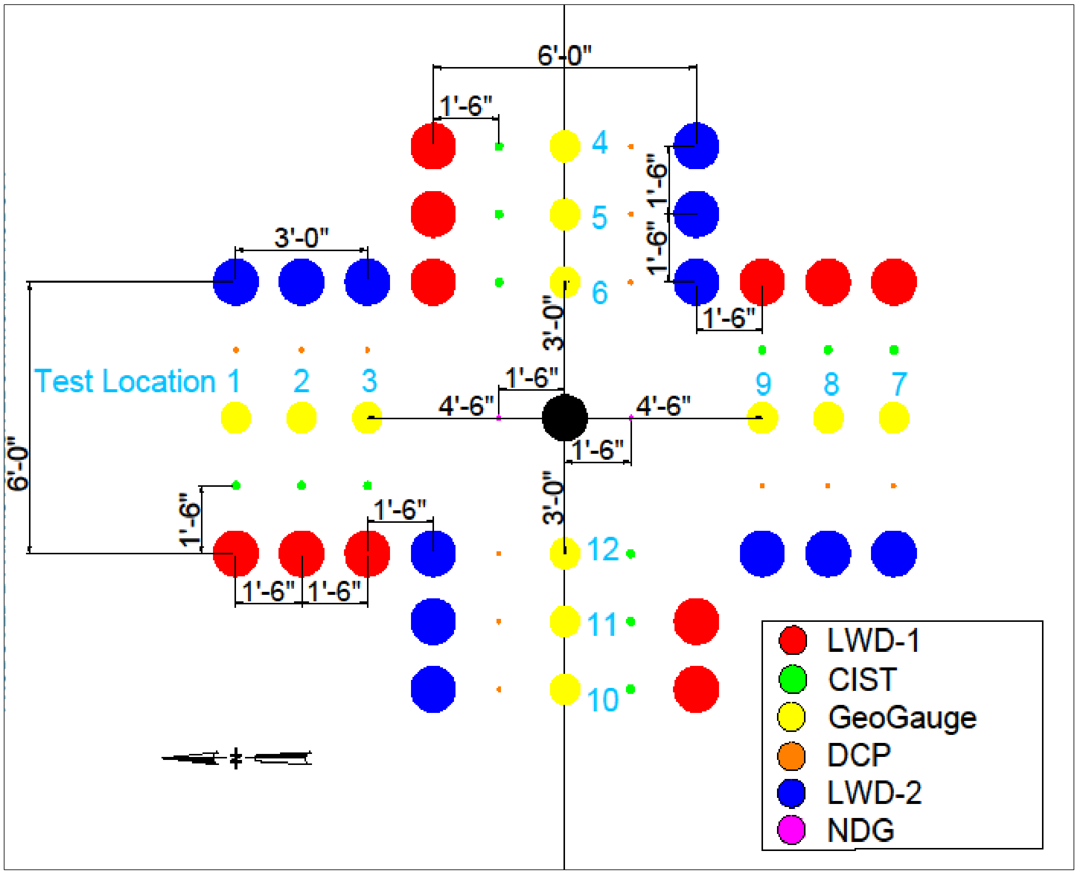

Figure 5.1: Orientation of each test site, typical three places

5.5 Project Site Number 1: I-95

On December 18, 2013, UF personnel initiated portable equipment testing. Testing concluded mid-morning the following day. Operational problems with one of the LWDs were encountered early, and a complete set of data were not attainable.

FDOT SMO personnel used an 18-wheel semi tanker filled with water for ballast for the 12-in. diameter static PLTs. SMO personnel also ran an NDG to determine in-place densities and moistures.

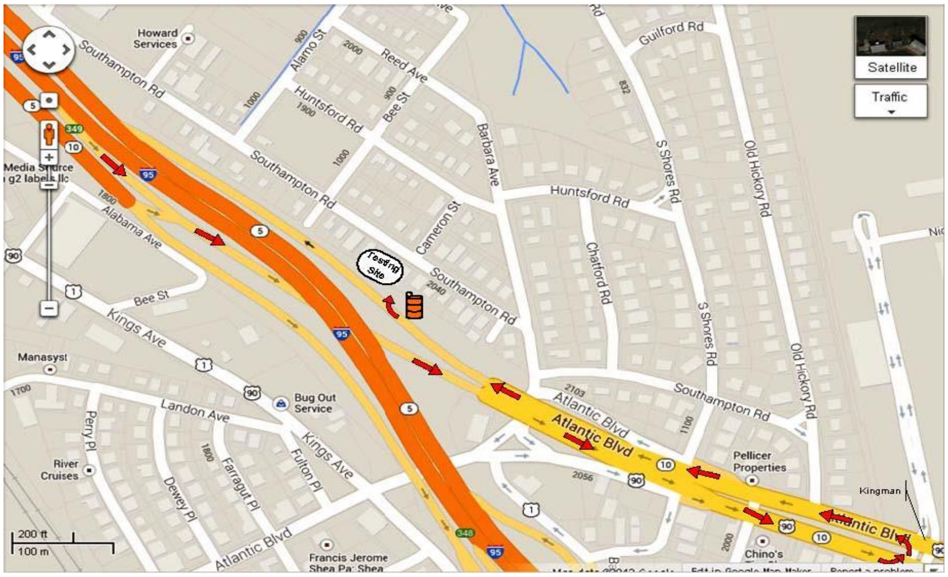

Figure 5.2: Project Site #1 I-95 - Location map southbound exit

The material made available for testing was graded aggregate (crushed concrete) base, 6 in. compacted thickness underlying an asphalt base course.

Figure 5.3: Project Site #1 I-95 - Photograph of graded aggregate (crushed concrete) used for base under asphalt base

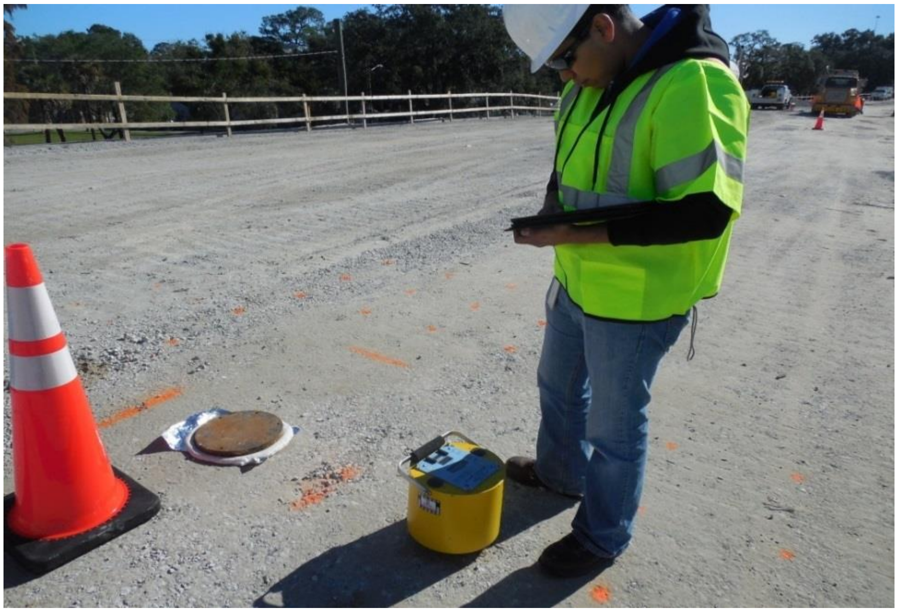

Figure 5.4: Project Site #1 I-95 - Photograph of GeoGauge and plate load testing

Figure 5.5: Project Site #1 I-95 - Photograph of fine sand used as bedding over rough surface texture

Figure 5.6: Project Site #1 I-95 - Photograph of tanker positioned over PLT location

Figure 5.7: Project Site #1 I-95 - Photograph of 12-in. diameter static PLT

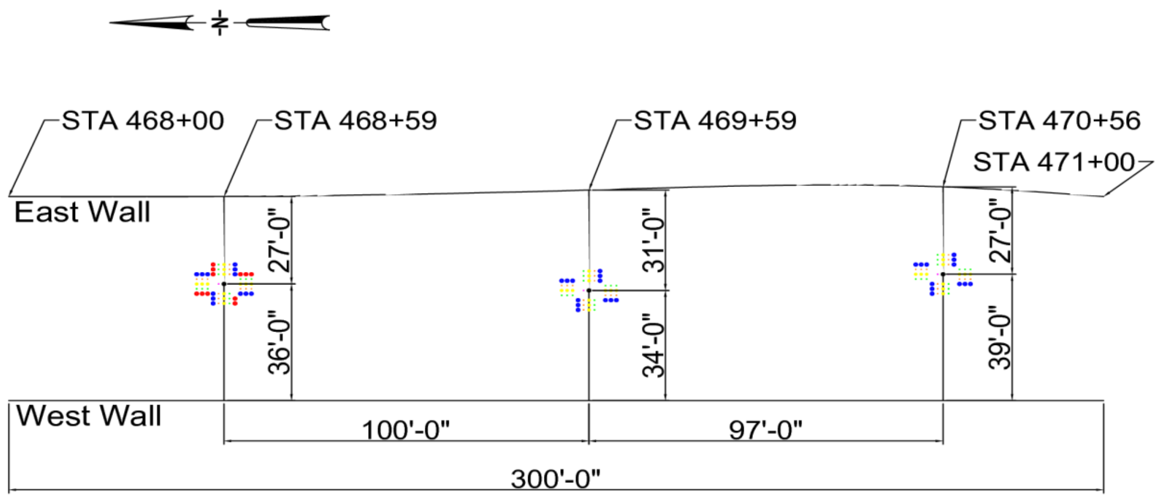

Figure 5.8: Project Site #1 I-95 - Test site location sketch

5.6 Project Site Number 2: US-301

The location of the second project was US-301 (SR-200), south of Callahan, Florida. A service road southeast of the railroad bridge overpass, east of the northbound travel lanes was the test site. Embankment fill material was placed 4 to 5 ft. thick. The material, brown to dark brown sand with wood fragments and pieces of white rock, was classified as A-2-4 soil.

PLTs were conducted by FDOT SMO personnel on January 23, 2014. Portable equipment testing was conducted by UF personnel on January 25, 2014.

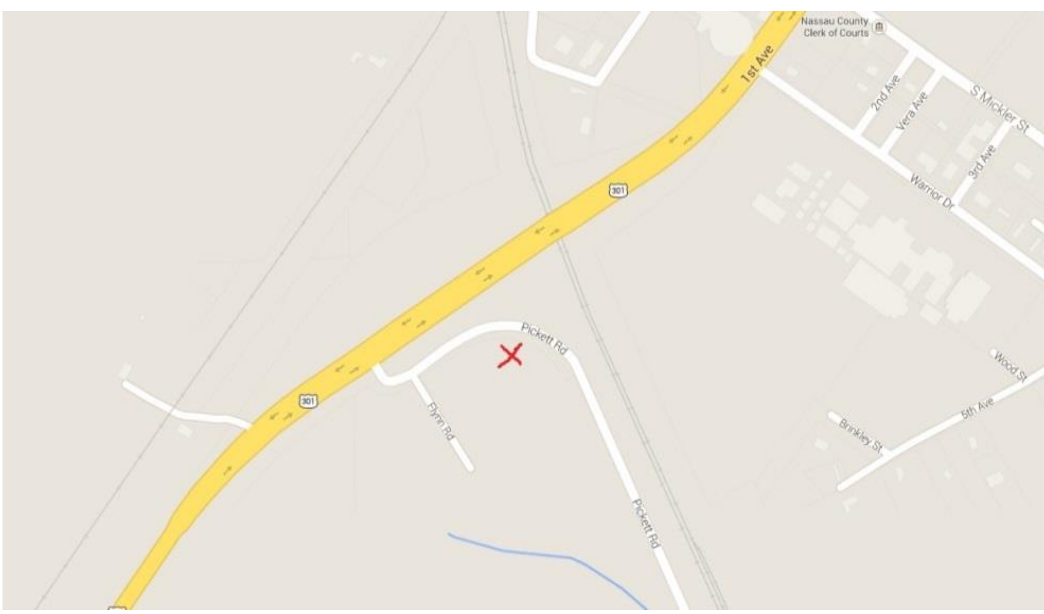

Figure 5.9: Project Site #2 US-301 - Location map east service road

Figure 5.10: Project Site #2 US-301 - Photograph of PLT tanker used for ballast

Figure 5.11: Project Site #2 US-301 - Photograph of LWDs by two different manufacturers

Figure 5.12: Project Site #2 US-301 - Photograph of LWDs background, GeoGauge foreground

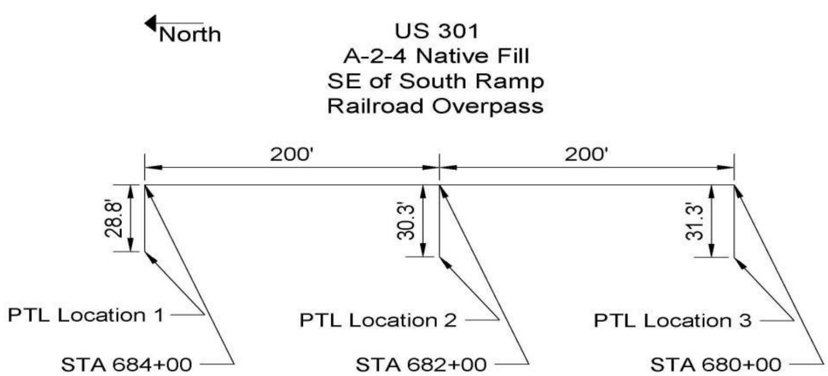

Figure 5.13: Project Site #2 US-301 - East service road test site location sketch

5.7 Project Site Number 3: SR-9B

The third test site was SR-9B (I-295), new construction. Testing at the project site was conducted on embankment material placed north of the US-1 overpass (MSE wall and bridge). Portable equipment testing on the first lift of the A-3 embankment soil was conducted by UF personnel on April 5 and 12, 2014. PLTs were conducted by FDOT SMO personnel on April 14, 2014.

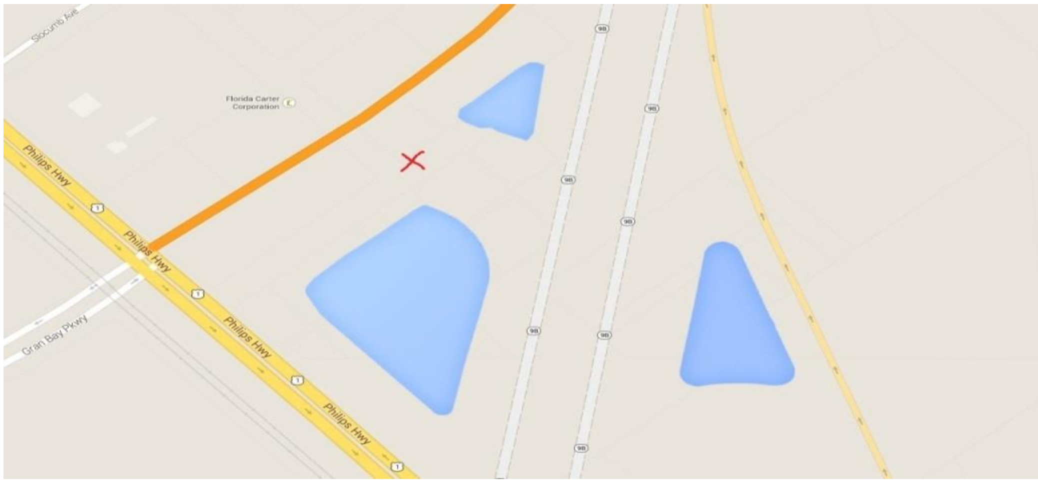

Figure 5.14: Project Site #3 SR-9B - Location map north of US-1 overpass

Figure 5.15: Project Site #3 SR-9B - Photograph of A-3 embankment material placement

Figure 5.16: Project Site #3 SR-9B - Photograph of GeoGauge calibration check

Figure 5.17: Project Site #3 SR-9B - Photograph of layout, typical at three stations (GeoGauge shown)

Figure 5.18: Project Site #3 SR-9B - Photograph of Station #2, looking north

Figure 5.19: Project Site #3 SR-9B - Photograph of CIST data collection

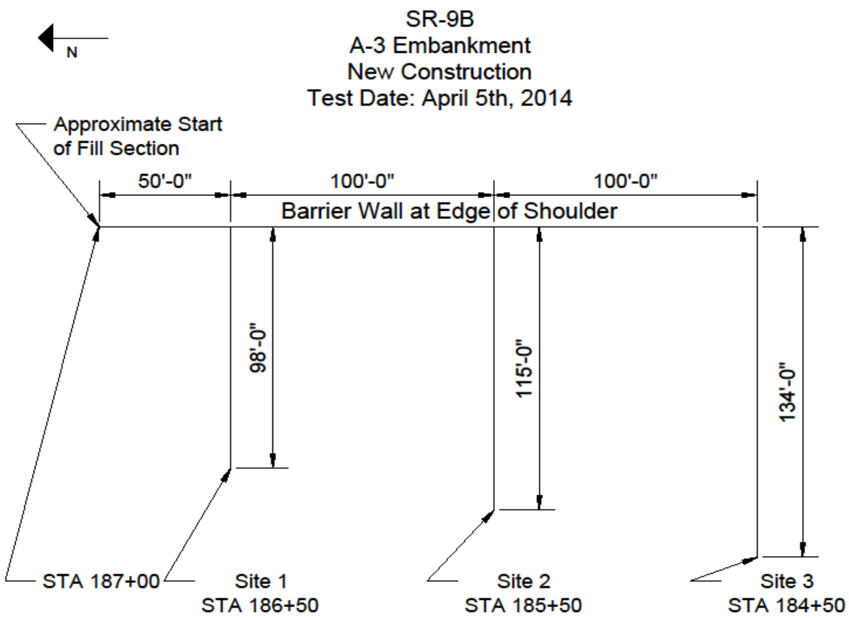

Figure 5.20: Project Site #3 SR-9B - Test site location sketch

5.7.1 Attempt #1

The first lift of A-3 embankment soil was placed, compacted, and tested to determine percentage of the maximum density, a minimum of 100% of the laboratory standard Proctor. Although the in-place density test indicated acceptance (see Appendix D, Quality Control Density Report), it was yielding (e.g., spongy) under foot. On April 5, 2014, portable equipment testing commenced but was terminated. The testing plan was revised. The PLTs were run on Thursday, April 10, 2014, and the portable equipment tests were run on Saturday, April 12, 2014.

5.7.2 Field Notes and Equipment Issues

From April 5, 2014:

- The LWD-2 handheld Trimble data logger was inoperable.
- The CIST would not provide output on the A-3, so no measurements were obtained. It was moved to undisturbed soil (unyielding), where it worked properly. The research team concluded that the consistency of the test strip material affected the device's accelerometer.
- The DCP magnetic ruler was not working. Penetrations were read manually.
- Values form the LWD-1 fluctuated greatly. The yielding soil was suspected.
- LWD-2 and DCP values were obtained from Station #1 only.
- GeoGauge values were obtained from Station #2 only. Significant variation between the 12 test sites at this test station location was observed.

- Shortly after the portable equipment testing concluded, the site received more than 2 in. of rainfall, delaying the PLTs. The testing schedule was revised. Portable equipment testing for all three stations now scheduled for April 12, 2014 and PLTs scheduled for April 14, 2014.

From April 12, 2014:

- The LWD-2 was not communicating with Trimble data logger. The password in the data logger was reset. All attempts to reinitiate communication between the components were unsuccessful.
- The DCP magnetic ruler worked properly.
- The CIST provided data.
- Samples were taken for moisture determination from all three stations and stored in moisture-proof containers. Oven-dried moistures were run at UF's Center for Transportation Training (CTT) lab the following day.

From April 14, 2014:

PLTs and NDG tests were to be run by the FDOT SMO personnel on all three sites. Plate load testing was conducted at two of the three sites. The tanker truck got stuck twice and had to be pulled by the contractor's front-end loader both times. Positioning on the third site was not attempted. NDG tests were run at all three sites.

5.7.3 Observations

The PLT data were as follows: Station 1 = 3,174.1 psi. Station 2 = 4,254.9 psi. The average of two stations = 3,714.5 psi.

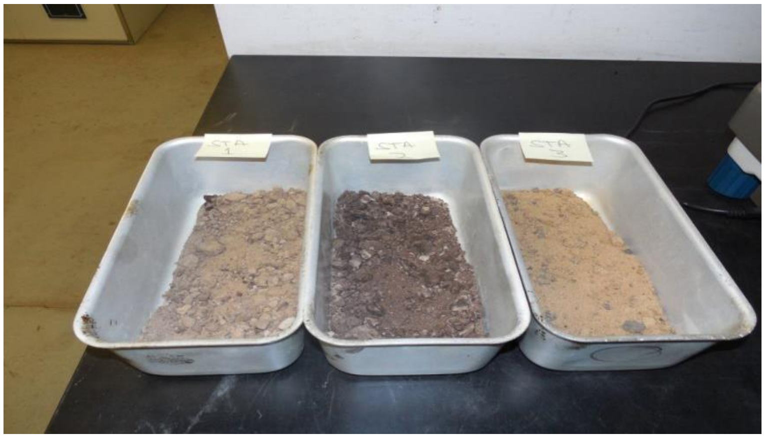

Figure 5.21: Project Site #3 SR-9B - Photograph of oven-dried samples
(From left to right: north, center, and south. Note different colors.)

From the SMO geotechnical laboratory report, although the material varied in color, laboratory tests indicated that the soil tested consistently in the A-3 soil classification. The percentage of material passing the 200 mesh sieve is used to illustrate this point. Percent -200 mesh by station (test site): 2.4%; 3.9%; and 4.0%.

The resilient modulus (M_R) data by station is as follows: M_R (psi) Station 1 = 14,629 and 14,258; Station 2 = 13,273 and 13,411; Station 3 = 12,850 and 13,422.

Station 1 average = 14,444 psi; Station 2 average = 13,342 psi; Station 3 average = 13,136 psi.

M_R overall average = 13,641 psi.

From the Mechanistic-Empirical Pavement Design Guide (MEPDG), the recommended resilient modulus at optimum moisture, AASHTO Soil Classification A-3, embankment and subgrade for flexible pavements is 16,500 psi. The laboratory M_R values at optimum moisture are reasonably close (overall average = 13,641 psi). The PLT values (average of two = 3,714.5 psi) at various moistures are significantly lower.

5.8 Project Site Number 4: CR-210

Project site Number 4 is the construction of additional lanes for CR-210 north and south of the US-1 overpass. This project is south of the SR-9B project. Although shell-rock was used for stabilized subgrade and base throughout the majority of the project, limited sections of limerock subgrade were available for testing. Testing of the limerock subgrade of the southbound lanes north of the US-1 overpass was conducted on May 29, 2014. This included portable equipment testing by UF personnel and PLT and NDG testing by FDOT personnel.

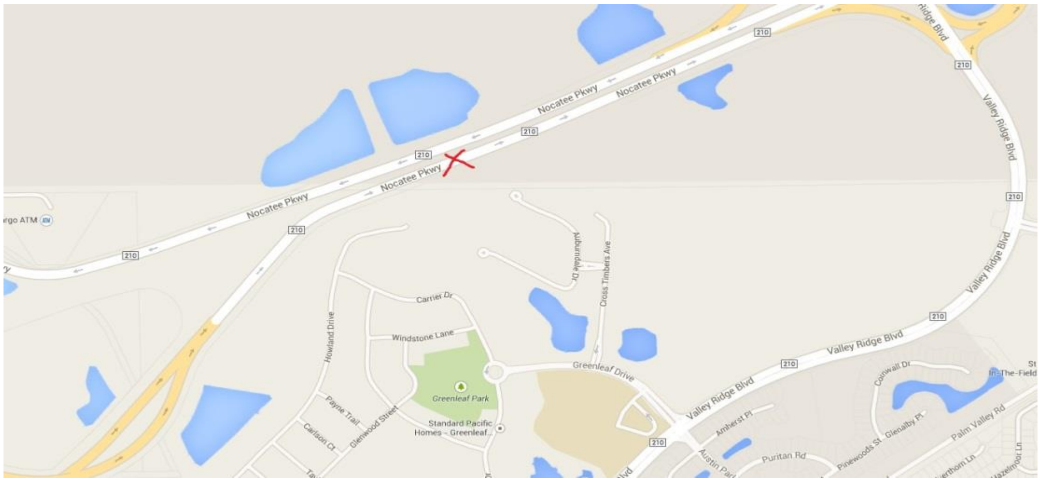

Figure 5.22: Project Site #4 CR-210 - Location map north of US-1 overpass

Figure 5.23: Project Site #4 CR-210 - Photograph of north station test location

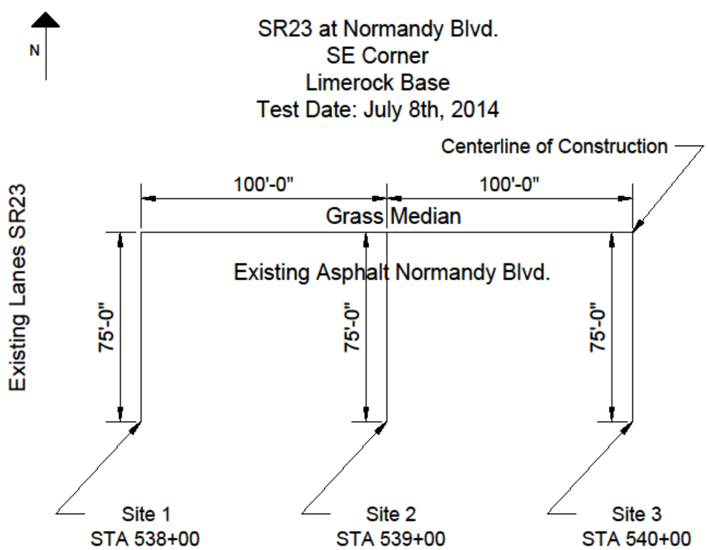

Figure 5.24: Project Site #4 CR-210 - Photograph of LWD-2, left, and LWD-1, right

Figure 5.25: Project Site #4 CR-210 - Photograph of 12 test locations per device **(**GeoGauge shown)

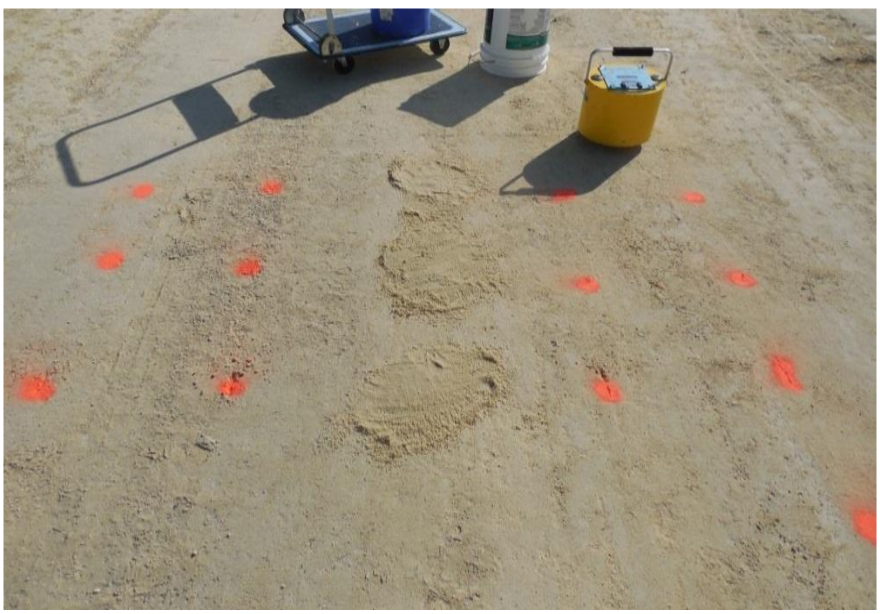

Figure 5.26: Project Site #4 CR-210 - Photograph of damp, fine sand used for bedding for GeoGauge due to the irregular surface of the limerock

Figure 5.27: Project Site #4 CR-210 - Photograph of PLT equipment

Two nuclear densities and moistures are run at each of the three sites.

Figure 5.28: Project Site #4 CR-210 - Photograph of DCP data collection

5.8.1 Field Notes and Equipment Issues

Due to the high level of densification, use of the DCP was suspended. It was felt that continued attempts to penetrate the limerock base material would cause damage to the DCP equipment.

On the second of three stations, the GeoGauge display began to give unintelligible output. Low battery strength was a possible cause. It was necessary to stop and get new batteries, and the six "D" cell batteries were replaced; however, this did not correct the problem. During this time, with the GeoGauge left uncovered, a brief rain shower occurred. Although the shower was brief and the sun quickly dried out the surface areas, moisture accumulated in the display windows of the instrument. Numerous attempts were made to fix the instrument but were unsuccessful. The GeoGauge had to be sent to the manufacturer for repair.

Both LWDs and the CIST worked properly throughout the testing event.

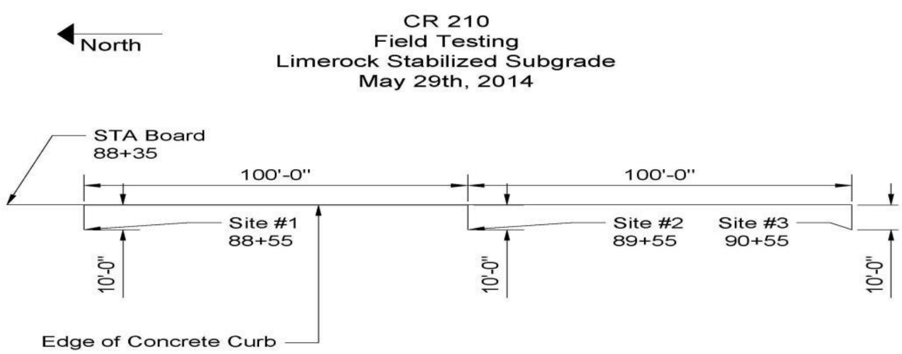

Figure 5.29: Project Site #4 CR-210 - Test location sketch

5.9 Project Site Number 5: SR-23 Limerock Base

Portable equipment testing was conducted by UF personnel on July 8, 2014. PLTs were conducted by FDOT SMO personnel on July 9, 2014.

Figure 5.30: Project Site #5 SR-23 limerock base - Location map southeast quadrant SR-23 and Normandy Boulevard

Figure 5.31: Project Site #5 SR-23 limerock base - Photograph of DCP (ineffective)

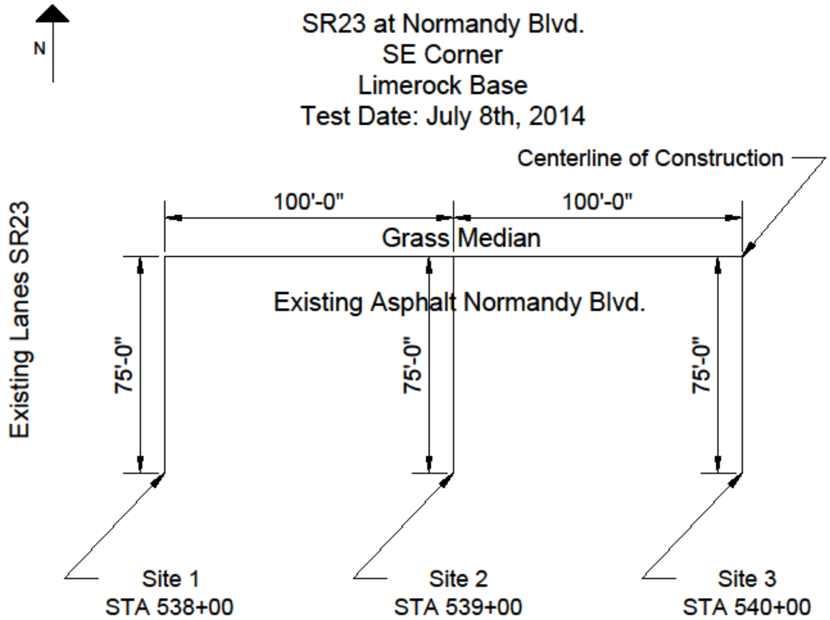

Figure 5.32: Project Site #5 SR-23 limerock base - Test site location sketch

5.10 Project Site Number 6: SR-23 A-3 Embankment

Portable equipment testing was conducted by UF personnel on July 14, 2014. PLTs were conducted by FDOT SMO personnel on July 17, 2014.

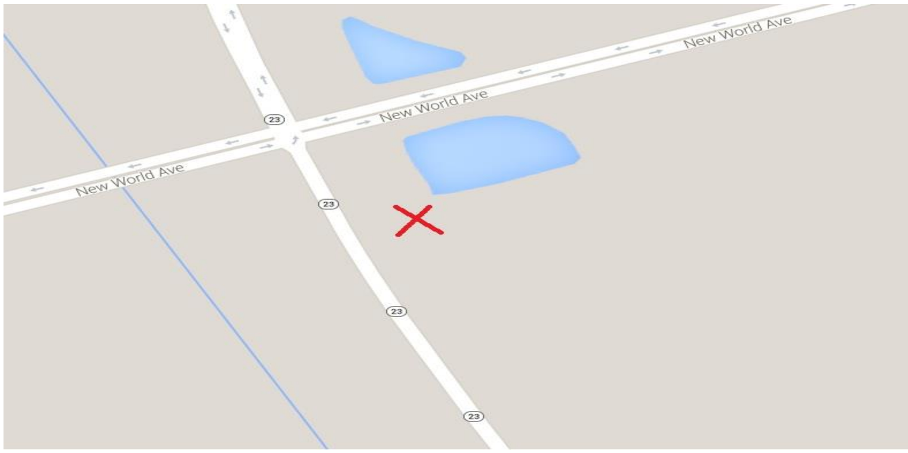

Figure 5.33: Project Site #6 SR-23 A-3 embankment - Location map southeast quadrant SR-23 and New World Avenue

Figure 5.34: Project Site #6 SR-23 A-3 embankment - Photograph of LWD-2 test

Figure 5.35: Project Site #6 SR-23 A-3 embankment - Photograph of PLT

Figure 5.36: Project Site #6 SR-23 A-3 embankment - Photograph of PLT

Figure 5.37: Project Site #6 SR-23 A-3 embankment - Photograph of PLT equipment

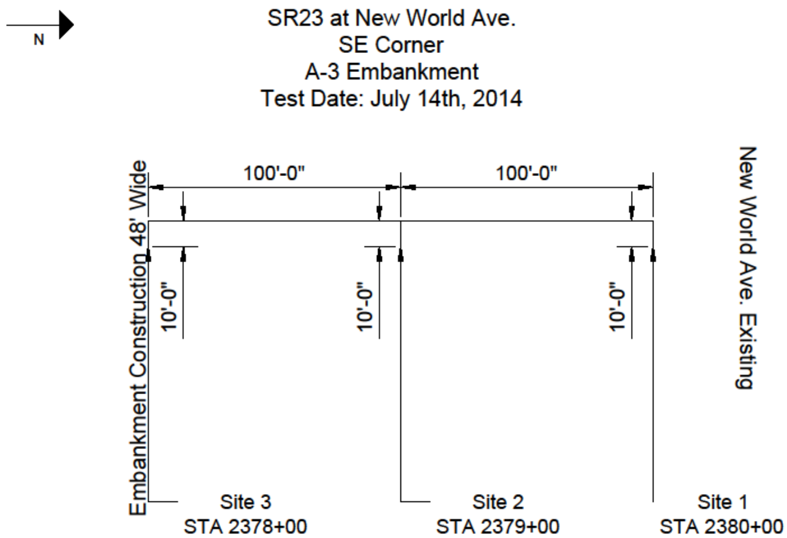

Figure 5.38: Project Site #6 SR-23 A-3 embankment - Test site location sketch

Chapter 6: Statistical Analysis

6.1 Phase I Test Box and Test Pits

The objective of Phase I was to develop a detailed assessment of portable equipment designed to provide in-place soil properties. Phase I equipment consisted of LWDs, GeoGauge, BCD, DSPA, CIST, and DCP. Devices showing the most promise in precision and reliability went forward in the research test plan. Analyses addressed measures of precision and repeatability. Specifically, coefficients of variance (COV) were calculated for each piece of equipment. A summary of the statistical analysis for the Phase I test results is given in Table 6.1.

6.1.1 Phase I Summary of Statistical Analysis Results

- The DSPA overall COV was 55.79%. This is calculated from average COVs from individual materials tested. The DSPA average COV for A-3 soil was 77.76%; average COV for A-2-4 high fines soil was 38.87%; and average COV for stabilized subgrade was 50.73%.
- The BCD overall COV was 48.69%. The BCD average COV for A-3 soil was 61.64%; average COV for A-2-4 high fines soil was 54.77%; and average COV for stabilized subgrade was 29.67%.
- The LWD-1 overall COV was 18.43%. The LWD-1 average COV for A-3 soil was 10.94%; average COV for A-2-4 high fines soil was 23.36%; and average COV for stabilized subgrade was 20.99%.
- The GeoGauge overall COV was 11.52%. The GeoGauge average COV for A-3 soil was 8.70%; average COV for A-2-4 high fines soil was 14.92%; and average COV for stabilized subgrade was 10.96%.
- The LWD-2 overall COV was 20.88%. The LWD-2 average COV for A-3 soil was 14.65%; and average COV for A-2-4 high fines soil was 23.36%.
- The DCP overall COV was 26.39%. The DCP average COV for A-3 soil was 31.86%; average COV for A-2-4 high fines soil was 26.83%; and average COV for stabilized subgrade was 20.47%.

6.1.2 Phase I Recommendations

Phase I results indicated higher variability associated with the DSPA and the BCD tests. Therefore, the DSPA and the BCD were recommended to be removed from the equipment list in the following phases.

Table 6.1: Phase I coefficients of variance

DSPA Coefficient of Variance (%)				Overall Average
Date	**A-3**	**A-2-4 High Fines**	**Stabilized Subgrade**	
3/15/2013	60.83	-	-	
4/3/2013	94.68	-	-	
4/30/2013	-	57.88	-	
5/6/2013	-	23.16	-	
5/13/2013	-	-	48.24	
5/16/2013	-	-	23.51	
5/22/2013	-	-	80.45	
5/29/2013	-	35.57	-	
Average	**77.76**	**38.87**	**50.73**	**55.79**
BCD Coefficient of Variance (%)				**Overall Average**
Date	**A-3**	**A-2-4 High Fines**	**Stabilized Subgrade**	
3/13/2013	61.64	-	-	
4/29/2013	-	49.09	-	
5/7/2013	-	57.35	-	
5/14/2013	-	-	20.90	
5/17/2013	-	-	45.43	
5/28/2013	-	-	22.69	
5/29/2013	-	57.86	-	
Average	**61.64**	**54.77**	**29.67**	**48.69**
Zorn LWD Coefficient of Variance (%)				**Overall Average**
Date	**A-3**	**A-2-4 High Fines**	**Stabilized Subgrade**	
3/13/2013	9.09	-	-	
4/1/2013	12.78	-	-	
4/30/2013	-	25.50	-	
5/7/2013	-	22.90	-	
5/14/2013	-	-	12.88	
5/16/2013	-	-	14.14	
5/22/2013	-	-	35.95	
5/29/2013	-	21.67	-	
Average	**10.94**	**23.36**	**20.99**	**18.43**

Table 6.1, continued

GeoGauge Coefficient of Variance (%)				Overall Average
Date	**A-3**	**A-2-4 High Fines**	**Stabilized Subgrade**	
3/15/2013	11.49	-	-	
4/3/2013	5.90	-	-	
4/29/2013	-	13.19	-	
5/6/2013	-	16.64	-	
5/13/2013	-	-	9.60	
5/16/2013	-	-	10.54	
5/22/2013	-	-	12.75	
Average	**8.70**	**14.92**	**10.96**	**11.52**
Dynatest LWD Coefficient of Variance (%)				**Overall Average**
Date	**A-3**	**A-2-4 High Fines**	**Stabilized Subgrade**	
4/1/2013	14.65	-	-	
4/30/2013	-	29.90	-	
5/7/2013	-	24.33	-	
Average	**14.65**	**27.12**		**20.88**
DCP Coefficient of Variance (%)				**Overall Average**
Date	**A-3**	**A-2-4 High Fines**	**Stabilized Subgrade**	
3/14/2013	31.15			
4/2/2013	32.57			
4/30/2013		25.08		
5/14/2013			31.01	
5/17/2013			17.33	
5/22/2013			13.08	
5/29/2013		28.58		
Average	**31.86**	**26.83**	**20.47**	**26.39**

6.2 Phase II Test Pits

Phase II focused on accuracy defined as the difference between measured values. The single spot testing sequence was developed to minimize the variability found in the test pit, notably density, moisture, and particle size. Phase II equipment consisted of LWDs, GeoGauge, CIST, and DCP. The output of each device under evaluation was compared to an adjacent 12-in. static PLT modulus and an NDG density test. A sample for moisture content determination also was obtained. Comparisons were made to laboratory M_R tests. Specifically, the Pearson correlation coefficients were calculated first, correlating equipment measurements with laboratory resilient modulus. Regression analyses were then conducted to obtain possible prediction expressions for resilient modulus values from equipment measurements. A summary of the statistical analysis for the Phase II test results follows.

6.2.1 Phase II Summary of Correlation Analysis Results

Based on the significance probability of correlations between equipment measurements and laboratory resilient modulus values (Table 6.2), the following correlation information can be obtained.

- When soils are tested below optimum moisture:
  - For A-2-4 low fines, GeoGauge measurements show correlation with the laboratory resilient modulus at the significance level of 0.1.
  - For A-3, no equipment measurements show correlation with the laboratory resilient modulus at the significance level of 0.1.
  - For stabilized subgrade, no equipment measurements show correlation with the laboratory resilient modulus at the significance level of 0.1.
  - For limerock base, DCP depth per blow measurements show correlation with the laboratory resilient modulus at the significance level of 0.1.

- When soils are tested near optimum moisture:
  - For A-2-4 low fines, NDG measurements show correlation with the laboratory resilient modulus at the significance level of 0.05.
  - For A-3, no equipment measurements show correlation with the laboratory resilient modulus at the significance level of 0.1.
  - For stabilized subgrade, DCP CBR measurements show correlation with the laboratory resilient modulus at the significance level of 0.05.
  - For limerock base, no equipment measurements show correlation with the laboratory resilient modulus at the significance level of 0.1.

- If moisture conditions are not specified:
  - For A-2-4 low fines, NDG measurements show correlation with the laboratory resilient modulus at the significance level of 0.1. DCP CBR measurements, CIST CIV measurements, PLT measurements, and DCP depth per blow measurements also show correlation with the laboratory resilient modulus at the significance level of 0.1.
  - For A-3, NDG measurements show correlation with the laboratory resilient modulus at the significance level of 0.05. DCP depth per blow measurements also show correlation with the laboratory resilient modulus at the significance level of 0.05. DCP CBR measurements, GeoGauge measurements, and PLT measurements show correlations with the laboratory resilient modulus at the significance level of 0.1.

- For stabilized subgrade, NDG measurements show correlation with the laboratory resilient modulus at the significance level of 0.05. PLT measurements, LWD-1 modulus measurements, LWD-1 deflection measurements, DCP depth per blow measurements, and CIST CIV measurements also show correlation with the laboratory resilient modulus at the significance level of 0.05.
- For limerock base, GeoGauge measurements show correlation with the laboratory resilient modulus at the significance level of 0.05. NDG measurements and PLT measurements also show correlation with the laboratory resilient modulus at the significance level of 0.05.

6.2.2 Phase II Correlation Analysis Interpretation

Table 6.2 summarizes the correlation analysis results, including both the Pearson correlation coefficients and the significance probabilities. Generally, the strength of the relationship is indicated by the correlation coefficient while the significance of the relationship is expressed in probability levels, which tells how unlikely a given correlation coefficient will occur given no relationship in the population. Therefore, the smaller the significance probability, the more significant the linear relationship; however, the larger the correlation coefficient, the stronger the relationship.

For example, in Table 6.2, 0.2749 is the calculated sample correlation between results obtained from LWD-1 measurements (in MPa) and the laboratory M_R for the A-2-4 low fines soil tested below the optimum moisture percentage. To test if the estimated correlation (0.2749) is significantly different from 0 (i.e., no correlation), we propose two hypotheses:

- H_0 (Null Hypothesis): The correlation in the population is 0.
- H_a (Alternative Hypothesis): The correlation in the population is different from 0.

Since the significance probability $p = 0.3872 > 0.10$, the null hypothesis of no correlation cannot be rejected at the significance level of 0.1. In other words, we cannot reject the null hypothesis that the correlation is 0 between results obtained from LWD-1 measurements (in MPa) and M_R for the A-2-4 low fines soil tested below the optimum moisture percentage. Therefore, we recommend “No” in Table 6.3 for using LWD-1 measurements to infer laboratory resilient modulus values for A-2-4 low fines below the optimum moisture percentage.

Similarly, in Table 6.2, since the significance probability is 0.0798 for the correlation between GeoGauge measurements and laboratory M_R for the A-2-4 low fines soil tested below the optimum moisture percentage, the null hypothesis of no correlation in the population of the two can be rejected at the significance level of 0.1. The Pearson correlation coefficient is 0.5247 in this case, which measures the strength of the correlation between the two. Therefore, we recommend “Yes” in Table 6.3 for using GeoGauge measurements to infer laboratory resilient modulus values for A-2-4 low fines near the optimum moisture percentage.

Table 6.2: Correlation between equipment measurements and laboratory resilient modulus summary

Equipment	Correlation Coefficient	Significance Probability	Correlation Coefficient	Significance Probability	Correlation Coefficient	Significance Probability	Correlation Coefficient	Significance Probability
Below Optimum Moisture	**A-2-4L (12 samples)**		**A-3 (12 samples)**		**Stab. Sub. (10 samples)**		**Limerock (10 samples)**	
CIST (CIV)	-0.0141	0.9653	0.3402	0.2793	0.1138	0.7542	0.1001	0.7831
DCP (CBR)	-0.0519	0.8728	0.0898	0.7813	0.0860	0.8132	0.4668	0.1738
DCP Depth/Blow (mm)	-0.0132	0.9675	-0.0221	0.9457	-0.2205	0.5404	-0.5800	0.0781*
GeoGauge (MPa)	0.5247	0.0798*	0.1519	0.6374	-0.5176	0.1254	0.5395	0.1075
LWD-1 (MPa)	0.2749	0.3872	0.2334	0.4652	-0.2978	0.4034	-0.0426	0.9070
LWD-1 Deflection (mm)	-0.2674	0.4009	-0.2889	0.3625	0.3204	0.3668	0.0422	0.9078
LWD-2 (MPa)	0.1731	0.5905						
LWD-2 Deflection (μm)	-0.2212	0.4897						
NDG (pcf)	0.4021	0.1950	0.3645	0.2441	-0.0297	0.9352	-0.2233	0.5351
PLT (MPa)	0.0118	0.9709	0.3086	0.3291	0.0357	0.9220	0.4364	0.2074
Near Optimum Moisture	**A-2-4L (12 samples)**		**A-3 (12 samples)**		**Stab. Sub. (10 samples)**		**Limerock (10 samples)**	
CIST (CIV)	0.0833	0.7969	0.1666	0.6047	-0.2173	0.5464	-0.2734	0.4446
DCP (CBR)	0.0734	0.8207	-0.3108	0.3254	0.6567	0.0391**	-0.3294	0.3527
DCP Depth/Blow (mm)	-0.0914	0.7775	0.3074	0.3310	-0.3801	0.2786	0.1242	0.7324
GeoGauge (MPa)	-0.0058	0.9858	0.0911	0.7783	0.4842	0.1562	-0.0901	0.8045
LWD-1 (MPa)	0.3111	0.3250	-0.2061	0.5205	-0.3507	0.3204	0.2361	0.5113
LWD-1 Deflection (mm)	-0.2791	0.3797	0.2822	0.3741	0.3214	0.3652	-0.1897	0.5996
NDG (pcf)	0.5968	0.0405**	-0.0895	0.7820	0.1260	0.7287	0.1552	0.6687
PLT (MPa)	-0.0125	0.9692	-0.2499	0.4334	0.0252	0.9448	0.0831	0.8194
Overall	**A-2-4L (24 samples)**		**A-3 (24 samples)**		**Stab. Sub. (20 samples)**		**Limerock (20 samples)**	
CIST (CIV)	-0.3536	0.0901*	0.0414	0.8477	-0.5386	0.0143**	0.3338	0.1504
DCP (CBR)	-0.3738	0.0720*	-0.3892	0.0601*	-0.4028	0.0783*	0.1469	0.5367
DCP Depth/Blow (mm)	0.3442	0.0996*	0.4340	0.0341**	-0.6230	0.0033**	-0.1851	0.4347
GeoGauge (MPa)	0.0190	0.9298	-0.3570	0.0868*	-0.0793	0.7395	0.6723	0.0012**
LWD-1 (MPa)	-0.1888	0.3769	-0.1351	0.5290	-0.6554	0.0017**	0.3025	0.1948
LWD-1 Deflection (mm)	0.2055	0.3353	0.1570	0.4637	0.6408	0.0023**	-0.2392	0.3097
NDG (pcf)	0.3880	0.0610*	0.4757	0.0188**	0.6885	0.0008**	0.6182	0.0037**
PLT (MPa)	-0.3507	0.0929*	-0.3536	0.0901*	-0.6870	0.0008**	0.4583	0.0421**

* The correlation is statistically significant at the significance level of 0.1.

** The correlation is statistically significant at the significance level of 0.05.

Table 6.3: Recommendations based on correlation analysis results

Moisture Condition	Soil Type	CIST (CIV)	DCP (CBR)	DCP Depth/Blow (mm)	GeoGauge (MPa)	LWD-1 (MPa)	LWD-1 Deflection (mm)	NDG (pcf)	PLT (MPa)
Below Optimum	A-2-4 low fines	No	No	No	Yes	No	No	No	No
	A-3	No	No	No	No	No	No	No	No
	Stabilized subgrade	No	No	No	No	No	No	No	No
	Limerock base	No	No	Yes	No	No	No	No	No
Near Optimum	A-2-4 low fines	No	No	No	No	No	No	Yes	No
	A-3	No	No	No	No	No	No	No	No
	Stabilized subgrade	No	Yes	No	No	No	No	No	No
	Limerock base	No	No	No	No	No	No	No	No
Overall	A-2-4 low fines	Yes	Yes	Yes	No	No	No	Yes	Yes
	A-3	No	Yes	Yes	Yes	No	No	Yes	Yes
	Stabilized subgrade	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes
	Limerock base	No	No	No	Yes	No	No	Yes	Yes

Note: The recommended pieces of equipment for different soil types or moisture conditions are bolded and highlighted. Priorities of data collection can be put on those highlighted pieces of equipment for further in-depth statistical analysis in future research.

6.2.3 Phase II Summary of Regression Analysis

Regression models have been developed to correlate portable equipment measurements with the laboratory M_R values. Table 6.4 summarizes the prediction expressions generated from regression models that have considered possible interactions between the equipment measurements and soil types or moisture conditions. Model development details can be found in Appendix F.

Table 6.4: Summary of prediction expressions from regression models with interaction

CIST	**Below Optimum (M_R in MPa; CIST in CIV)**	**Optimum (M_R in MPa; CIST in CIV)**
A-2-4 low fines	M_R = 78.215 + 0.256 x CIST	M_R = 85.821 – 0.307 x CIST
A-3	M_R = 81.521 + 0.256 x CIST	M_R = 89.127 – 0.307 x CIST
Stabilized subgrade	M_R = 89.320 + 0.256 x CIST	M_R = 96.925 – 0.307 x CIST
Limerock base	M_R = 167.840 + 0.256 x CIST	M_R = 175.445 – 0.307 x CIST
GeoGauge	**Below Optimum (M_R and GeoGauge in MPa)**	**Optimum (M_R and GeoGauge in MPa)**
A-2-4 low fines	M_R = 68.144 + 0.1204 x GeoGauge	M_R = 84.533 + 0.00204 x GeoGauge
A-3	M_R = 70.465 + 0.1204 x GeoGauge	M_R = 86.853 + 0.00204 x GeoGauge
Stabilized subgrade	M_R = 77.843 + 0.1204 x GeoGauge	M_R = 94.231 + 0.00204 x GeoGauge
Limerock base	M_R = 145.341 + 0.1204 x GeoGauge	M_R = 161.730 + 0.00204 x GeoGauge
PLT	**Below Optimum (M_R and PLT in MPa)**	**Optimum (M_R and PLT in MPa)**
A-2-4 low fines	M_R = 66.041 + 0.163 x PLT	M_R = 84.506 – 0.00475 x PLT
A-3	M_R = 65.423 + 0.206 x PLT	M_R = 83.888 + 0.0387 x PLT
Stabilized subgrade	M_R = 108.967 - 0.130 x PLT	M_R = 127.432 – 0.297 x PLT
Limerock base	M_R = 132.304 + 0.224 x PLT	M_R = 150.769 + 0.0562 x PLT
DCP	**Below Optimum (M_R in MPa; DCP in %)**	**Optimum (M_R in MPa; DCP in %)**
A-2-4 low fines	M_R = 91.836 – 0.288 x DCP	M_R = 104.096 – 0.813 x DCP
A-3	M_R = 73.195 + 0.778 x DCP	M_R = 85.455 + 0.253 x DCP
Stabilized subgrade	M_R = 88.100 + 0.132 x DCP	M_R = 100.361 – 0.393 x DCP
Limerock base	M_R = 154.080 + 0.456 x DCP	M_R = 166.340 – 0.069 x DCP
LWD-1	**Below Optimum (M_R and LWD-1 in MPa)**	**Optimum (M_R and LWD-1 in MPa)**
A-2-4 low fines	M_R = 71.771 + 0.1437 x LWD-1	M_R = 97.252 - 0.2873 x LWD-1
A-3	M_R = 75.975 + 0.1437 x LWD-1	M_R = 101.457 - 0.2873 x LWD-1
Stabilized subgrade	M_R = 82.148 + 0.1437 x LWD-1	M_R = 107.630 - 0.2873 x LWD-1
Limerock base	M_R = 164.020 + 0.1437 x LWD-1	M_R = 189.501 - 0.2873 x LWD-1
NDG	**Below Optimum (M_R in MPa; NDG in pcf)**	**Optimum (M_R in MPa; NDG in pcf)**
A-2-4 low fines	M_R = -124.993 + 1.904 x NDG	M_R = -47.066 + 1.194 x NDG
A-3	M_R = -123.573 + 1.904 x NDG	M_R = -45.647 + 1.194 x NDG
Stabilized subgrade	M_R = -121.445 + 1.904 x NDG	M_R = -43.519 + 1.194 x NDG
Limerock base	M_R = -45.476 + 1.904 x NDG	M_R = 32.450 + 1.194 x NDG

6.2.4 Phase II Recommendations

These results are based on the significance probabilities of correlations between equipment measurements and laboratory M_R values. The significance probabilities are calculated from limited data points (ten or 12 data points for each soil type below the optimum moisture or near the optimum moisture, and 20 or 24 data points for each soil type overall). Therefore, those results provide only potential trend information for future research. Specifically, it is recommended that data collection for in-depth statistical analysis is continued for those devices whose measurements have shown significant correlations with the laboratory M_R for certain soil types under certain moisture conditions. Such pieces of equipment are highlighted in both Table 6.2 and Table 6.3. Following the collection of enough data, stronger relationships could be expected between measurements from those devices and the laboratory M_R values. Priorities of data collection for further in-depth statistical analyses can be placed on those devices that are highlighted.

6.3 Phase III Field Testing

Due to limited data sets, the linear regression models obtained from Phase II data are likely to generate unacceptable errors if applied to in-place testing. Therefore, the interaction regression models generated from the Phase II data are used only for validation purposes in Phase III on this research project, comparing the predicted M_R values from the interaction regression models with the field testing results obtained from the selected projects (i.e., I-95, US-301, SR-9B, CR-210, SR-23/Normandy Boulevard, and SR-23/New World Avenue).

6.3.1 Phase III Summary of Statistical Analysis Results

Mean absolute deviation (MAD), root mean square error (RMSE), and mean absolute percent error (MAPE) were calculated for measures of the overall forecast accuracy of each regression model summarized in Table 6.4. The MAD measures the size of the error in units. The RSME is a measure of the differences between the value predicted and the value actually observed. It represents the sample standard deviation of the differences between predicted values and observed values. MAPE measures the size of the error in percentage terms. See Equations 6.1 through 6.3.

$$MAD = \frac{\sum_{i=1}^{n} |F_i - y_i|}{n} \qquad \text{(Eq. 6.1)}$$

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n} (F_i - y_i)^2}{n}} \qquad \text{(Eq. 6.2)}$$

$$MAPE = \frac{1}{n}\sum_{i=1}^{n}\left|\frac{F_i}{y_i} - 1\right| \qquad \text{(Eq. 6.3)}$$

where F_i = predicted M_R values based on equipment measurements; y_i = actual laboratory tested M_R values; and n = sample number.

Table 6.5 summarizes the validation results for application of the interaction models based on the in-place testing results on this project. Appendix F, Table F.6, contains the complete data set.

Table 6.5: Summary of interaction regression model validation

	GeoGauge	PLT	DCP	CIST	NDG	LWD-1
MAD	14.739	14.840	14.912	23.345	22.808	27.581
RMSE	19.785	21.205	19.290	29.751	29.352	35.116
MAPE	11.52%	11.77%	12.18%	18.09%	18.88%	20.33%

These observations are to be used with caution as the number of data sets is small, ranging from nine to 18 (see Table 6.6). For example, there are ten valid data points from DCP in Phase III, including one data point at SR-9B, three data points at SR-23 and New World, one data point at CR-210, two data points at SR-9B, and three data points at I-95. Again, see Table F.6 for details.

Table 6.6: Interaction regression model validation data points

Equipment	**DCP**	**GeoGauge**	**PLT**	**NDG**	**CIST**	**LWD-1**
Valid data points	10	17	16	18	15	9

6.3.2 Phase III Recommendations

It is recommended that additional studies be conducted to generate field-proven target values for the device(s) chosen. These target values would be obtained from materials that have met the current minimum in-place density specification requirements. To do so, side-by-side testing with the NDG and the chosen device(s) on base, subgrade, and embankment soils commonly used in Florida road construction would be necessary. Attention must be paid to moisture content at the time of testing. Particle size distribution is another key variable. Mn/DOT and INDOT both have employed this approach to develop target value data tables for devices such as the DCP and LWD. It is recommended that the expanded field data be combined with this project's statistical modeling to develop acceptance values and/or acceptance ranges for soil compaction when measured with the alternative devices.

In addition, with more data sets obtained in future research, more reliable regression models could be established and their effectiveness could be validated by applying those models to field testing results.

Chapter 7: Field Work with Other State Departments of Transportation

Other state DOTs were contacted regarding portable (non-NDG) equipment currently being used or evaluated for use in the soil compaction acceptance decision. Field visits to construction projects under the jurisdiction of other DOTs assisted the research team in their determination of the types of equipment and the methodology that would be most beneficial to this research project.

7.1 Indiana Department of Transportation

7.1.1 Site Visit

On May 22 and 23, 2014, Wayne Rilko visited with the Indiana Department of Transportation (INDOT). The meeting was held at Office of Geotechnical Services, Indianapolis, IN.

Nayyar Siddiki, P.E., was the primary contact person. Mr. Siddiki has been instrumental in the implementation of the DCP and LWD as in-place soil testing equipment used for acceptance by the INDOT. Day 1 was spent in the office. Mr. Siddiki gave presentations regarding the use of the equipment, test procedures, and specifications. Day 2 was split between the office and the field, where the DCP and LWD were demonstrated on INDOT construction projects. Wayne Rilko gave a presentation to INDOT personnel pertaining to this research project.

7.1.2 Background

INDOT has developed and implemented specifications for compaction control of various materials based on the results of DCP and LWD tests. The DCP is used for clayey, silty, or sandy soils, as well as granular soils with aggregate sizes smaller than ¾ in. and structural backfill sizes 1 in., ½ in., and Nos. 4 and 30. The LWD is used for granular soils with aggregate sizes greater than ¾ in.; coarse aggregate sizes Nos. 43, 53, and 73; and structural backfill sizes 2 in. and 1.5 in.

7.1.3 DCP

INDOT divides their soil materials (-¾ in.) into three categories—sandy soils, silty soils, and clayey soils. This is done by plotting the maximum dry density versus the optimum moisture.

From extensive data compilation, a minimum blow count for 6-in. and/or 12-in. lift thicknesses at 95% and/or 100% maximum density for various soils was established.

The moisture of the in-place material is determined. INDOT typically uses the cook-off method where the soil is placed in a pan, which is heated on a gas-fired camp stove. This method was selected due to the high clay content found in many of their soils.

A moisture range for each category has been established. For example, the moisture range for clayey material is -2% to +2% of optimum. Silty and sandy soils are -3% to optimum. Granular soils are -6% to optimum. If the in-place moisture is out of range, the DCP acceptance test will not be run. One moisture content test is required for each day that density or strength measurements are taken. The sample for moisture content is required to be representative of the entire depth of the compaction lift being tested.

As a field check, a one-point proctor can be run (see Section 7.1.6). Along with the field moisture percentage, the material category can be verified. If the material is out of range, a sample is taken back to the laboratory for a full analysis.

7.1.4 LWD

For controlling the compaction of aggregate materials and chemically modified soils, INDOT specifies performing LWD tests 2 ft. from each edge of the construction area and at the midpoint of the site width. To accept the compacted layer, the average value of the maximum deflection obtained in the three LWD tests is to be equal to or less than the maximum allowable deflection determined from a test section. The moisture content of the aggregate is to be within -6% of the optimum moisture content. This may change to -3% to optimum. One moisture content test of the compacted aggregates per day is required. The frequency for acceptance for compaction control testing is one test per 800 tons of compacted aggregate. To determine the maximum allowable deflection, a test section approximately 100 ft. long by the width of the layer is constructed. The test section is constructed with the contractor's equipment available for the project. LWD tests are conducted on the test section. The roller is operated in the vibratory mode. After the completion of four roller passes on the test section, ten random LWD tests are conducted at the approximate locations shown in Figure 7.1. After the completion of five roller passes, a second set of ten random LWD tests are conducted at approximately the same locations as the previous set of LWD tests. If the difference between the average values of the maximum deflections of the LWD tests conducted after the fourth and fifth passes is equal to or less than 0.02 mm, the compaction will be considered to have peaked and the average of the ten LWD values after the fifth pass will be used as the maximum allowable LWD deflection. However, if the difference between the average deflection values of LWD tests is greater than 0.02 mm, an additional roller pass in the vibratory mode is applied and an additional ten LWD tests are performed. This procedure is continued until the difference between the average maximum deflection values of consecutive roller passes is equal to or less than 0.02 mm. The maximum allowable deflection will be the lowest average maximum deflection value of the LWD tests performed on the last roller pass.

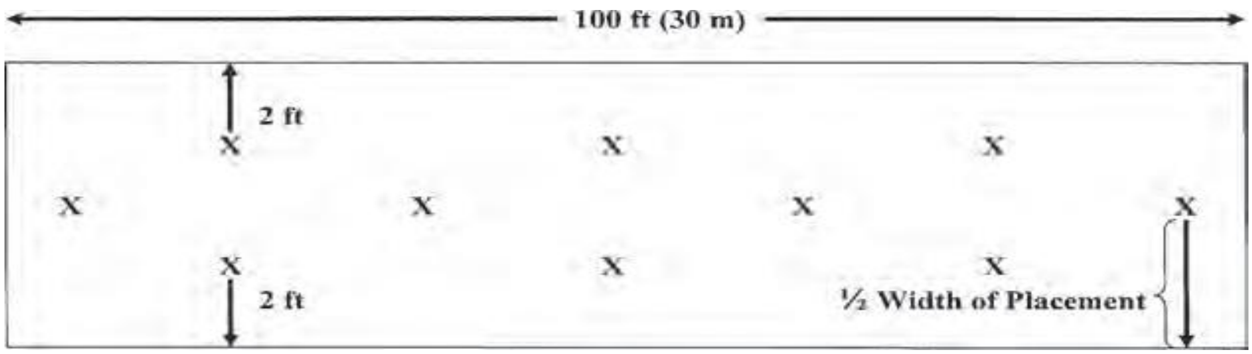

Figure 7.1: INDOT LWD test pad for maximum allowable deflection determination

The 0.02 mm maximum deflection is a recent change. Soheil Nazarian, Ph.D., P.E. made the suggestion to revise from 0.01 mm to 0.02 mm.

INDOT requires that the LWD used in compaction control tests have a metal loading plate with a diameter of 11.8 in., an accelerometer attached to the center of the loading plate for measuring the maximum vertical deflection, and falling mass of 22 lbs. (10 kg). The maximum force applied by the LWD should be 1,589.4 lbf (7.07 kN).

INDOT's guidelines also state that the test area must be leveled so that the entire undersurface of the LWD load plate is in contact with the material being tested. Loose and protruding materials are to be removed. Any unevenness is to be filled with fine sand. The LWD load plate also should be rotated approximately 45 degrees back and forth to seat it. The LWD test includes conducting six drops. The first three are considered seating drops. The measurements of the last three drops are averaged and reported as the LWD deflection value. Additional compaction of the tested material is required if the change in deflection for any two consecutive LWD drops is 10% or greater. The LWD plate is not to move laterally with successive drops.

The acceptance criteria are based on deflection. INDOT has amassed enough data on certain materials commonly used in base construction to determine a maximum allowable deflection value for these materials.

Table 7.1: INDOT LWD maximum allowable deflection table

Material Type	Maximum Allowable Deflection (mm)
Lime modified soil	0.30
Cement modified soil	0.27
Aggregates over lime modified soil	0.30
Aggregates over cement modified soil	0.27

Materials not listed in Table 7.1 will require construction of a test pad (Figure 7.1) to determine the maximum allowable deflection.

7.1.5 Field Observations

INDOT noted that the main limitation in the implementation LWD specifications was the difficulty of using the LWD in confined areas and small projects for which control strips cannot be constructed. In addition, the DCP posed problems (lack of confinement) for use with sandy materials. The agency is working to resolve these issues.

The majority of the testing is conducted by INDOT employees, although some DCP and LWD testing currently is provided by CEI firms. Contractor test results are not used in the acceptance decision.

Photos from field testing performed on May 23, 2104, are provided in Figures 7.2 and 7.3.

Figure 7.2: Photograph of INDOT LWD deflection measurement on #53 aggregate base

Figure 7.3: Photograph of INDOT #53 aggregate base

7.1.6 One-Point Proctor

In accordance with INDOT Field Determination of Maximum Dry Density and Optimum Moisture Content of Soil ITM No. 512-14T:

> Samples representing each soil type on the contract are required to be tested in a laboratory for maximum dry density and optimum moisture content in accordance with AASHTO T 99. The soil classification and DCP count are determined from these tests. As soils may have a wide variation in properties on a given contract, determination of the correct DCP criteria or the maximum dry density and optimum moisture content in a laboratory for every possible soil combination is difficult.

A procedure has been developed for field determination of the maximum dry density and optimum moisture content of the soils encountered at each location on the contract. The procedure consists of compacting a soil specimen in accordance with AASHTO T 272, which specifies a procedure for a rapid determination of the maximum density and optimum moisture content of a soil sample utilizing a family of curves and a one-point determination. Charts have been developed. The first is a family of curves based on previously acquired soil data. The maximum wet density vs. moisture is plotted. This value is transferred to a plot of maximum wet density vs. maximum dry density. This value is then transferred to a third plot to determine optimum moisture content. Knowing the maximum dry density and the optimum moisture content, the DCP blow count may then be determined and/or adjusted, if necessary.

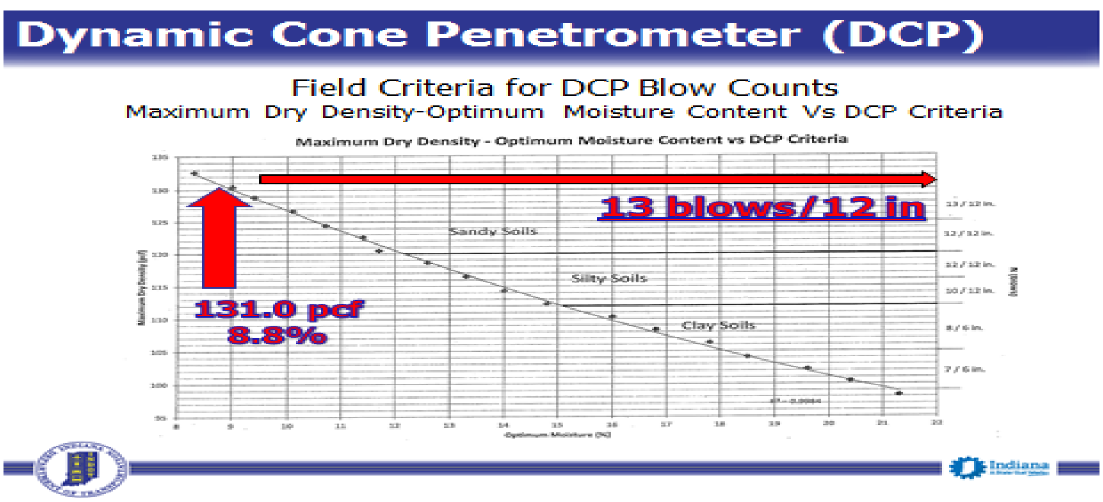

Figure 7.4: INDOT field criteria for DCP blow counts

The intersecting value is the required number of DCP blows for a penetration of 6 in. for clayey soils and a penetration of 12 in. for silty and sandy soils. The charts referenced previously are for clayey, silty, or sandy (not granular) soils.

7.2 Minnesota Department of Transportation

7.2.1 Site Visit

Wayne Rilko visited with the Minnesota Department of Transportation (Mn/DOT) personnel at the Materials and Research Office, Maplewood, MN, during the week of August 25, 2014. Mn/DOT has been a leader in the implementation of the DCP and LWD for in-place soil testing for acceptance.

Representing Mn/DOT were John Siekmeier, P.E., Senior Geotechnical Research Engineer; Terry Beaudry, Grading and Base Engineer; Rebecca Embacher, Advanced Materials and Technology; and Tim Anderson, Pavement Design Engineer.

Kevin McLain, P.E., R.G., Geotechnical Director, Missouri Department of Transportation (MoDOT), Jefferson City, MO, also was in attendance.

Rebecca Embacher gave a presentation on the LWD pilot testing protocol she authored for establishing repeatability. Briefly, the LWD is placed on a series of polychloroprene bearing pads. The conformity of deflection (average pad deflection) is measured. This procedure is to be performed:

- On receipt of a newly purchased device.
- Prior to recommissioning a device after calibration by a testing institute.
- Annually.
- When measurements are no longer repeatable.
- When measurements are questionable.

John Siekmeier and Terry Beaudry elaborated on many changes proposed by Mn/DOT management.

At the time of the site visit (August 2014), only two Districts, D1 (northeast) and D4 (northwest) were using the LWD. The Materials and Research Office was under the assumption that all districts were using the LWDs purchased for QA field testing. Direction from Mn/DOT management includes writing and/or revising the specifications rather than relying on project-specific special provisions for LWD usage.

Quality Management, shifting acceptance testing from Mn/DOT (QA) to the contractor (QC), will be implemented within the next year. An exact date has yet to be determined.

Quality Management field testing will be conducted by one of the following methods:

- DCP
- LWD
- Specified density (sand cone or NDG)
- Test rolling

- Quality Compaction (informal test rolling provision)

Mn/DOT verification testing will be conducted at a frequency of one test for every four QC tests run by the contractor. Details pertaining to what test methods will be used on what types of projects have yet to be determined. The transition to Quality Management will affect this process. The DCP currently is used statewide. There are no plans to change this, but the DCP is one of several methods allowed under Quality Management.

Typical production (field procedures) for compaction control using the DCP are as follows: A particle size analysis (gradation) is performed in the laboratory to determine a grading number. A field moisture test is conducted, usually with a portable stove. The DCP test is performed to determine a seating depth (depth after two blows) and a penetration depth (depth after an additional three blows). The test procedure is conducted in accordance with ASTM D6951.

Typical production (field procedures) for compaction control using the LWD are as follows: A table containing maximum deflection (mm) values is used to determine acceptance. Although there are provisions for construction of a test section to determine a maximum deflection value for a specific soil, this is rarely done. The test procedure is conducted in accordance with ASTM D2835.

Day two of the visit was at the Mn/DOT District 4 Construction Office located in Detroit Lakes, MN. Wayne Rilko, Kevin McLain, and John Siekmeier met with Trudy Kordosky, P.E., Resident Engineer, and her staff. District 4 continues to use LWDs for acceptance testing. District personnel have generated enough data to compile a table of acceptable (not to exceed) deflection values. Following the office meeting, Ms. Kordosky and two field technicians accompanied us to an ongoing construction project, a bypass lane on a state route. Class 5 base material had been placed and compacted. A Kessler DCP and Zorn LWD were used. The technicians will provide feedback regarding the use of the Olson LWD. From the set of tests run on the base material, the Olson and Zorn LWDs compared favorably to each other.

Figure 7.5: Photograph of LWDs
(From left to right: Modified Prima 100, Zorn Model ZF 2000, and Olson Prototype)

Figure 7.5 shows a Prima 100 with modified (shortened) handle, Zorn Model ZF 2000, and Olson prototype. All units are using an 8-in. diameter plate. The Prima handle was shortened to allow for upright transport on the passenger side floorboard of a pickup truck. The 8-in. diameter plate rather than the standard 12-in. diameter plate was chosen for weight reduction. Mn/DOT staff encountered problems with the Prima unit connecting with the Bluetooth PDA. Similar issues were encountered with the FDOT Prima unit. Currently, all LWDs used by Mn/DOT are manufactured by Zorn.

Figure 7.6: Photograph of Mn/DOT DCP on Class 5 aggregate base

Figure 7.7: Photograph of Mn/DOT LWDs and DCP on Class 5 aggregate base

Figure 7.8: Photograph of Mn/DOT LWDs on Class 5 aggregate base

Figure 7.9: Photograph of Olson prototype LWD on Class 5 aggregate base

Figure 7.10: Photograph of Olson prototype LWD data collector

Patrick Miller, P.E., Sr. Project Engineer with Olson Engineering, Wheat Ridge, CO, has been contacted. Olson has offered to loan one of their LWDs for our evaluation.

Mr. McLain has started similar research in conjunction with his position with MoDOT as well as for his Ph.D. dissertation. He has been attending online graduate engineering courses originating from Iowa State University, Ames, Iowa. Dr. David White is a member of his dissertation committee.

7.3 Lessons Learned from Other State DOTs

7.3.1 Indiana Department of Transportation

INDOT has developed clear and concise test methods for the DCP and the LWD—ITM 506 and ITM 508, respectively. For the DCP, DOT personnel have compiled enough soils data that—based on the maximum dry density and the optimum moisture content obtained from either laboratory or field Proctor tests—a maximum blow count for various soil types has been established.

For the LWD, a limited number of soils have enough test data to establish target values for maximum allowable deflections. In the event that the soil being tested is not listed on the maximum allowable deflection table, a test section is constructed.

Ongoing work includes verifying moisture content ranges at the time of testing for both the DCP and the LWD. For example, results from the soils laboratory indicated that the allowable moisture range for clay material being tested with the DCP should be ±2% of optimum. If the field moisture is out of this range, the DCP test would be invalid. Similar work is being done for LWD testing.

7.3.2 Minnesota Department of Transportation

The DCP is used throughout the state. The blow count is based on a grading number, which is determined by the results of a sieve analysis. The LWD currently is not used throughout the state. LWD acceptance is based on maximum deflection.

Directions from management include:

- Utilization of the research conducted by Mn/DOT for establishing LWD target values for various materials and moisture contents. *"Using the Dynamic Cone Penetrometer and Light Weight Deflectometer for Construction Quality Assurance,"* (Siekmeier et al., 2009). Similar tables from the previously referenced project also are included in NCHRP Synthesis 456, *"Non-Nuclear Methods for Compaction Control of Unbound Materials"* (Nazzal, 2014).
- Continuation of the Intelligent Compaction (IC) program based on the research conducted by Mn/DOT and others. The portable devices that measure in-place stiffness are a key component for verifying the IC roller stiffness values. Dr. David White (Iowa State University) and Dr. Michael Mooney (Colorado School of Mines) have worked with Mn/DOT on numerous projects.

Other changes:

- Effective November 1, 2014, Mn/DOT will use M-E design criteria for both flexible and rigid pavement designs.

- Mn/DOT is in the process of transitioning from their Laboratory Information Management System (LIMS) to AASHTOWare. Due to field data entry needs, the Materials and Research Office is involved in this development.
- An Olson LWD prototype, manufactured in Colorado and based on technology from the Colorado School of Mines, was available at the Materials Office. It was transported to District 4 for our use and left there for use and further evaluation by the District 4 QA technicians.

Chapter 8: Proposed Target Values and Test Methods

8.1 Target Values of Other State DOTs

One objective of this research was to establish target values for various soils based on the test results generated by the selected portable equipment. Information garnered from other state DOTs—both through the literature review and field visits with Mn/DOT and INDOT personnel—is presented as background.

8.1.1 Mn/DOT DCP Target Values Test Procedure Using Grading Number

The Mn/DOT DCP procedure for testing of granular materials is based on the amount of penetration from the three blows after the distance for the two seating blows is subtracted.

A grading number (GN) is calculated using the percentage passing the designated sieves.

GN = (1" + 3/4" + 3/8" + #4 + #10 + #40 + #200) / 100 (Eq. 8.1)

The GN is used in conjunction with the moisture content (%) of the material tested. The combination of these two values is used to determine if the seating value [Penetration Reading (2 blows) - Initial Reading] is less than the maximum allowable in either mm or mm/blow. The Penetration Index (DPI) then is calculated using the following formula:

DPI = [Penetration Reading (5 blows) – Penetration Reading (2 blows)] / 3 (Eq. 8.2)

This value is compared to the maximum allowable DPI. If either the seating value or the DPI exceeds the values listed in the table, the DCP test fails.

Tables 8.1 and 8.2 illustrate the GN method. Table 8.1 provides a typical sieve analysis showing the percentage passing selected sieves. The GN for this material is 4.7.

Table 8.1: Mn/DOT sieve analysis for DCP grading number

Sieve	% Passing
1 in.	100
3/4 in.	100
3/8 in.	95
# 4	80
# 10	50
# 40	30
# 200	15.0
Grading Number	**4.7**

Table 8.2: Mn/DOT DCP penetration requirements.

Grading Number	MC (% Dry)	Maximum Allowable Seating (mm) *	Maximum Allowable DPI (mm/blow)	Grading Number	MC (% Dry)	Maximum Allowable Seating (mm)	Maximum Allowable DPI (mm/blow)
3.1 – 3.5	< 5.0	40	10	4.6 – 5.0	< 5.0	65	15
	5.0 – 8.0	40	12		5.0 – 8.0	75	19
	> 8.0	40	16		> 8.0	85	23
3.6 – 4.0	< 5.0	40	10	5.1 – 5.5	< 5.0	85	17
	5.0 – 8.0	45	15		5.0 – 8.0	95	21
	> 8.0	55	19		> 8.0	105	25
4.1 – 4.5	< 5.0	50	13	5.6 – 6.0	< 5.0	100	19
	5.0 – 8.0	60	17		5.0 – 8.0	115	24
	> 8.0	70	21		> 8.0	125	28

For this example, assume a moisture content (MC) at the time of testing of 7.5%. The GN is 4.7. Referring to Table 8.2, penetration requirements, the maximum allowable seating (mm) is 75 mm. The maximum allowable penetration index (DPI) = 19 mm.

The maximum allowable seating is based on the penetration after two blows. The maximum allowable penetration is based on three additional blows.

This procedure is described in Section 8.3.1 as Procedure B. As an alternative, Procedure A is based on using target values obtained from the testing of similar materials.

8.1.2 INDOT DCP Target Values Using Blows per Soil Type and Layer Thickness

INDOT's approach differs from that of Mn/DOT. INDOT, through extensive soil testing, has established a minimum number of DCP blows per soil layer based on the type of soil, plastic index, and maximum dry density. The minimum number of blows for penetration of 6-in. and/or 12-in. thick lifts is given in Table 8.3.

Table 8.3: INDOT DCP soils vs. blows relationship.

DCP Criteria for Compaction

Plastic Index , Max. Dry Density VS DCP blows Relation

S.No	Textural Classification	Plastic Index	Max. Dry Density pcf	DCP criteria for 6 in. lift	DCP criteria for 12 in. thick (2 lift of 6 in.) (95 % Compaction)	DCP Criteria for 0 to 12 in. thick (100% Compaction)
A	Clay Soils					
	Clay Soils	greater than 20	less than 105 pcf	7		
	Clay Soils	8 to 20	105 to 112 pcf	8		
B	Silty Soils					
	Silty Soils	4 to 8	113 to 116 pcf		10	
	Silty Soils	less than 4	117 to 120 pcf		12	
C	Sandy Soils					
	Sandy Soils	less than 8	121 to 125 pcf		12	
	Sandy Soils	less than 4	greater than 125 pcf		13	
D	Granular Soils					
	Structure Backfill					
	Structure backfill # 30				7	10
	Structure backfill # 4				9	12
	Structure backfill # 1/2 in.				11	14
	Structure backfill # 1 in.				18	-

8.1.3 MoDOT DCP Maximum Penetration Specification

MoDOT specifications require Type 7 aggregate base under both roadway and shoulders to be compacted to achieve an average DCP penetration index value of less than or equal to 0.4 in. per blow through the base lift thickness, as determined by a standard DCP device with a 17.6-lb. hammer meeting the requirements of ASTM D6951.

8.1.4 Mn/DOT Target Value Research

Mn/DOT has conducted numerous studies correlating soil properties, densities, and portable equipment output. Summary tables from “Using the Dynamic Cone Penetrometer and Lightweight Deflectometer for Construction Quality Assurance” are provided in Tables 8.4 and 8.5 (Siekmeier et al., 2009). These target values are being re-evaluated (October 2014) for incorporation in the Mn/DOT Standard Specifications, Mn/DOT Table S-xx.1. Similar tables from the previously referenced project also are included in NCHRP Synthesis 456, “Non-Nuclear Methods for Compaction Control of Unbound Materials” (Nazzal, 2014). These tables are included in Appendix H of this report but are provided here for reference.

Table 8.4: Mn/DOT target values granular materials

Mn/DOT DCP AND LWD TARGET VALUES FOR GRANULAR MATERIALS

Grading Number (GN)	Moisture Content (%)	Target DPI (mm/drop)	Target DPI Modulus CSIR (MPa)	Target LWD Modulus Dynatest (MPa)	Target LWD Modulus Zorn (MPa)	Target LWD Deflection Zorn (mm)
3.1–3.5						
	5–7	10	97	120	80	0.38
	7-9	12	80	100	67	0.45
	9-11	16	59	75	50	0.60
3.6–4.0						
	5–7	10	97	120	80	0.38
	7-9	15	63	80	53	0.56
	9-11	19	49	63	42	0.71
4.1–4.5						
	5–7	13	73	92	62	0.49
	7-9	17	55	71	47	0.64
	9-11	21	44	57	38	0.79
4.6–5.0						
	5–7	15	63	80	53	0.56
	7-9	19	49	63	42	0.71
	9-11	23	40	52	35	0.86
5.1–5.5						
	5–7	17	55	71	47	0.64
	7-9	21	44	57	38	0.79
	9-11	25	37	48	32	0.94
5.6–6.0						
	5–7	19	49	63	42	0.71
	7-9	24	38	50	33	0.90
	9-11	28	32	43	29	1.05

Table 8.5: Mn/DOT target values fine-grained soils

Mn/DOT TARGET DPI and LWD DEFLECTION VALUES FOR FINE-GRAINED SOILS

Plastic Limit (%)	Estimated Optimum Moisture	Field Moisture as a Percentage of Optimum Moisture	DCP Target DPI at Field Moisture (mm/drop)	Zorn Deflection Target at Field Moisture (%)	Zorn Target at Field Moisture Maximum (mm)
Nonplastic	10–14	70–74	12	0.5	1.1
		75–79	14	0.6	1.2
		80–84	16	0.7	1.3
		85–89	18	0.8	1.4
		90–94	22	1	1.6
15–19	10–14	70–74	12	0.5	1.1
		75–79	14	0.6	1.2
		80–84	16	0.7	1.3
		85–89	18	0.8	1.4
		90–94	22	1	1.6
20–24	15–19	70–74	18	0.8	1.4
		75–79	21	0.9	1.6
		80–84	24	1	1.7
		85–89	28	1.2	1.9
		90–94	32	1.4	2.1
25–29	20–24	70–74	24	1	1.7
		75–79	28	1.2	1.9
		80–84	32	1.4	2.1
		85–89	36	1.6	2.3
		90–94	42	1.8	2.6
30–34	25–29	70–74	30	1.3	2
		75–79	34	1.5	2.2
		80–84	38	1.7	2.4
		85–89	44	1.9	2.7
		90–94	50	2.2	3

8.1.5 Mn/DOT Proposed LWD Specification Target Values

Based on previous research and data obtained during construction testing, Mn/DOT's proposed revisions (October 2014) for LWD target values are shown as follows. The minimum allowable elastic modulus (MPa) values for granular and clay and clay loam materials have been deleted (strike through). The MPa value for base materials remains. Field inspectors record the modulus values obtained from each LWD test. Acceptance, however, is based on the maximum allowable deflection (mm) values for all materials.

(2105) LWD QUALITY COMPACTION METHOD (TARGET VALUE)

Modified Spec.

The LWD is used in lieu of testing compaction compliance by use of a sand cone, nuclear gauge, or DCP. The target values are only for a Zorn 2000 or 3000 LWD and would need to be adjusted if a different device were used.

Table 8.6: Mn/DOT LWD target values

Mn/DOT Table S-xx.1 LWD Target Values			
Specification	**Material Type**	**Maximum Allowable Deflection (mm)**	**Minimum Allowable Elastic Modulus (MPa)**
2105 or 2106	Granular	0.78	~~40~~
	Clay and Clay Loam	1.47	~~20~~
2211	Base	0.55	50

8.1.6 INDOT LWD Specification Target Values

INDOT's LWD specification was discussed in Section 8.1.4. From Table 8.1, the maximum allowable LWD deflection target values are as follows: lime modified soil < 0.30 mm; cement modified soil < 0.27 mm; aggregates over lime modified soil < 0.30 mm; and aggregates over cement modified soil < 0.27 mm.

8.2 Phase II DCP and LWD Target Values

8.2.1 Target Values from Phase II Test Pit Data

Target values and their ranges based on the 95% confidence intervals were generated for the DCP and LWD from the Phase II test pit data. These values are provided in Tables 8.7 and 8.8.

DCP target values are provided for low fines A-2-4 and A-3 soils. Target values are in depth per blow (mm) as well as minimum blows for penetrating 6-in. and 12-in. thick compacted soil layers. The LWD target values are maximum allowable deflection (mm). In addition to the target value, the 95% confidence values (upper and lower) are given. This approach is similar to percent within limits (PWL) currently used by the FDOT for asphalt acceptance.

Both the DCP and LWD measurements of the stabilized subgrade were affected by the larger limerock particles found throughout the layer. Although the layer was compacted to meet the minimum density requirement (see Appendix D, Figures D.9 and D.10), it was observed during testing that the larger limerock particles could move or be dislodged. This is a concern whenever granular particles are not confined, allowing them to move under test. This affects not only the DCP penetration depth per blow but also the LWD deflection measurements.

Table 8.7: DCP target values from Phase II test pit data

Phase II Test Pit Data	**Below Optimum (95% Confidence Interval)**				**At Optimum (95% Confidence Interval)**			
DCP Target Values	**< Opt. Target**	**< Opt. Lower CI**	**< Opt. Upper CI**	**Moisture (%)**	**@Opt. Target**	**@Opt. Lower CI**	**@Opt. Upper CI**	**Moisture (%)**
A-2-4 low fines								
DCP depth/blow (mm)	6.99	6.57	7.41	5.67~6.8	8.83	8.29	9.38	8.2~10.0
in./blow	0.275	0.259	0.292	5.6~6.8	0.348	0.326	0.369	
blows/6 in.	22	23	21		17	18	16	8.3~10.0
blows/12 in.	44	46	41		35	37	33	
A-3								
DCP depth/blow (mm)	8.91	8.11	9.71	4.4~4.9	11.63	10.90	12.35	8.2~10.5
in./blow	0.351	0.319	0.382	4.4~4.9	0.458	0.429	0.486	
blows/6 in.	17	19	16		13	14	12	8.2~10.5
blows/12 in.	34	38	31		26	28	25	
Limerock base								
DCP depth/blow (mm)	5.63	4.91	6.35	10.0~10.7	5.74	4.68	6.80	11.8~13.7
in./blow	0.222	0.193	0.250	10.0~10.7	0.226	0.184	0.268	
blows/6 in.	27	31	24		27	33	22	11.8~13.7
blows/12 in.	54	62	48		53	65	45	
Stabilized subgrade								
DCP depth/blow (mm)	14.73	12.63	16.83	7.1~8.7	10.95	9.66	12.24	8.2~10.1
in./blow	0.580	0.497	0.663	7.1~8.7	0.431	0.380	0.482	
blows/6 in.	10	12	9		14	16	12	8.2~10.1
blows/12 in.	21	24	18		28	32	25	

Table 8.8: LWD target values from Phase II test pit data

LWD-1 Target Values								
LWD-1 deflection (mm)	Target	Lower CI	Upper CI	Moisture (%)	Target	Lower CI	Upper CI	Moisture (%)
A-2-4 low fines	0.34	0.31	0.38	5.6~6.8	0.45	0.42	0.48	8.3~10.0
A-3	0.40	0.37	0.43	4.4~4.9	0.43	0.40	0.46	8.2~10.5
Limerock base	0.23	0.19	0.28	10.0~10.7	0.26	0.23	0.29	11.8~13.7
Stabilized subgrade	0.39	0.33	0.44	7.1~8.7	0.55	0.46	0.65	8.2~10.1

8.2.2 Other States' DCP Target Values Compared to Phase II Test Pit Data

Other state DOTs' DCP target values were compared to the Phase II test pit data (see Table 8.9). Complete data sets from other state DOTs can be found in Appendix G of this report.

Table 8.9: Other states' DCP target values compared to Phase II test pit data

DCP Target Value Summary				
Mn/DOT	**mm/blow**	**in/blow**	**blows/6 in.**	**blows/12 in.**
Mn/DOT				
Mn/DOT Research granular (range)	10 to 28			
Mn/DOT Research fine-grained (range)	12 to 50			
Mn/DOT Grading Number (range)	10 to 28			
INDOT				
6 in. clay (range)			7 to 8	
12 in. silt (range)				10 to 12
12 in. sand (range)				12 to 13
12 in. granular (range)				7 to 18
MoDOT Specification		0.4		
Phase II test pits				
A-2-4 low fines @< optimum	7	0.275	22	44
A-2-4 low fines @ near optimum	9	0.348	17	35
A-3 @< optimum	9	0.351	17	34
A-3 @ near optimum	12	0.458	13	26
Limerock base @< optimum	6			
Limerock base @ near optimum	6			
Stabilized subgrade @< optimum	15			
Stabilized subgrade @ near optimum	11			

8.2.3 Other States' LWD Target Values Compared to Phase II Test Pit Data

Other state DOTs' LWD target values were compared to the Phase II test pit data (see Table 8.10). Complete data sets from other state DOTs can be found in Appendix G of this report.

Table 8.10: Other states' LWD target values compared to Phase II test pit data

LWD Target Value Summary	
	mm deflection
Mn/DOT	
Mn/DOT research granular (range)	0.38 to 1.05
Mn/DOT research fine-grained (range)	1.1 to 3.0
Mn/DOT proposed spec. granular	0.78
Mn/DOT proposed spec. clay & clay loam	1.47
Mn/DOT proposed spec. base	0.55
INDOT	
Lime modified soils	0.30
Cement modified soils	0.27
Aggregates over lime modified soils	0.30
Aggregates over cement modified soils	0.27
Phase II test pit data	
A-2-4 low fines @ < optimum	0.34
A-2-4 low fines @ near optimum	0.45
A-3 @ < optimum	0.40
A-3 @ near optimum	0.43
Limerock base @ < optimum	0.23
Limerock base @ near optimum	0.26
Stabilized subgrade @ < optimum	0.39
Stabilized subgrade @ near optimum	0.55

8.2.4 High Fines A-2-4

A-2-4 soil with high fines (24% passing the 200 mesh sieve) was tested in Phase I and the initial series of Phase II. Section 3.6.2 mentions the difficulty in adjusting the moisture content of this material. Based on this experience, the A-2-4 high fines material was not included in the Phase II SSTS, where measures were taken to minimize variability to generate target values that would be compared to future test series. The results are presented in Tables 8.11 and 8.12. Those tests were conducted near optimum moisture or slightly below optimum moisture, and thus moisture range is included without distinction between two moisture conditions as shown in Tables 8.7 and 8.8.

Table 8.11: DCP target values for high fines A-2-4 soil from test pit data

A-2-4 High Fines	Target	95% Confidence Interval		Moisture (%)*
DCP		Lower CI	Upper CI	
DCP depth/blow (mm)	11.27	9.90	12.64	7.53~10.61
inches/blow	0.44	0.39	0.50	7.53~10.61
blows/6 inches	14	15	12	7.53~10.61
blows/12 inches	27	31	24	7.53~10.61

*Optimum moisture = 11.0%

Table 8.12: LWD target values for high fines A-2-4 soil from test pit data

A-2-4 High Fines	Target	95% Confidence Interval		Moisture (%)*
		Lower CI	Upper CI	
LWD-1 deflection (mm)	0.30	0.27	0.32	7.10~11.41
LWD-2 deflection (mm)	0.13	0.12	0.15	7.10~11.41

*Optimum moisture = 11.0%

8.3 Test Method Development

ASTM test methods were examined for the DCP—D6951 Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications—and for the LWD—E2835 Standard Test Method for Measuring Deflections Using a Portable Impulse Plate Load Test Device and E2583 Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD). This examination revealed that additional information is needed for developing test procedures that fully address utilization of these devices in the soil compaction acceptance decision.

Modification to the Florida Sampling and Testing Methods (FSTM)—often referred to as Florida Methods (FM)—will be required to supplement the ASTM procedures. The purposes for developing FMs are to standardize the FDOT's sampling and testing methods and to provide a compilation of these methods for inclusion in the Standard Specifications for Road and Bridge Construction.

Historically, FMs have been developed uniquely when no recognized standard method is available to meet the intended purpose or because an existing published method needs to be extensively revised to accommodate a specific FDOT need.

Knowledge of specification development for M-E acceptance testing procedures was gained by visiting with the INDOT in May 2014 and the Mn/DOT in August 2014. The language included in the draft test methods for the DCP and the LWD is based largely on similar documents currently in use by these DOTs. Full text versions of their applicable procedures and specifications can be found in Appendix H of this report.

8.3.1 Dynamic Cone Penetrometer

As seen in Section 8.1.1, DCP usage by Mn/DOT for testing granular materials requires the calculation of a GN by determining the percentage of material passing the designated sieves.

The GN is used in conjunction with the moisture content (%) of the material tested. The combination of these two values is used to determine if the seating value and/or the penetration index exceeds the maximum allowable values listed in the applicable table. If either the seating value or the penetration index exceeds the values listed in the table, the DCP test fails.

This method could be used, at least initially, due to the limited amount of data pertaining to the DCP. The number of blows (two for seating and three for DPI) is of concern and may have to be adjusted based on subsequent field testing. It is Procedure B in the proposed FM.

Alternatively, Procedure A—using the target values generated from this research by testing similar materials in the test pit—also could be used as a starting point. Using the DCP and A-2-4 material near optimum moisture, a partial table of DCP values is provided in Table 8.11. The table shows penetration (in.) per blow as well as the number of blows required to penetrate 6-in. and 12-in. thick lifts. Upper and lower confidence intervals also are given. The dry density was approximately 108 pcf. Low fines includes materials with up to 15% passing the 200 mesh sieve.

Table 8.13: DCP target penetration values for A-2-4 soil

A-2-4 Low Fines	**@Opt. Target**	**@Opt. Lower CI**	**@Opt. Upper CI**
in./blow	0.348	0.326	0.369
blows/6 in.	17	18	16
blows/12 in.	35	37	33

These values are higher than those listed on INDOT's table. INDOT's closest soil would be clayey with a 105-112 pcf dry density and a plasticity index range from eight to 20. Blow counts range from seven to eight for a 6-in. lift, but this is based on a soil with slightly different properties but a similar dry density. A full list of DCP penetration target values and confidence intervals can be found in Table 8.7. Comparisons of the target values from this research project and those used by other state DOTs can be found in Table 8.9.

Due to the limited amount of data available, it is likely that soils not included in Table 8.7 will be encountered. In this instance, the procedure for the construction of a test section or sections consisting of the material(s) being used is included in the proposed DCP FM. This test section is based on that used by INDOT for testing with the LWD. INDOT has sufficient soil data for soil testing with the DCP, making test sections unnecessary. The proposed DCP FM also includes language for use of the DCP along with density control measured with the NDG.

Ten tests will be performed on the test section at the approximate locations depicted in Figure 8.1.

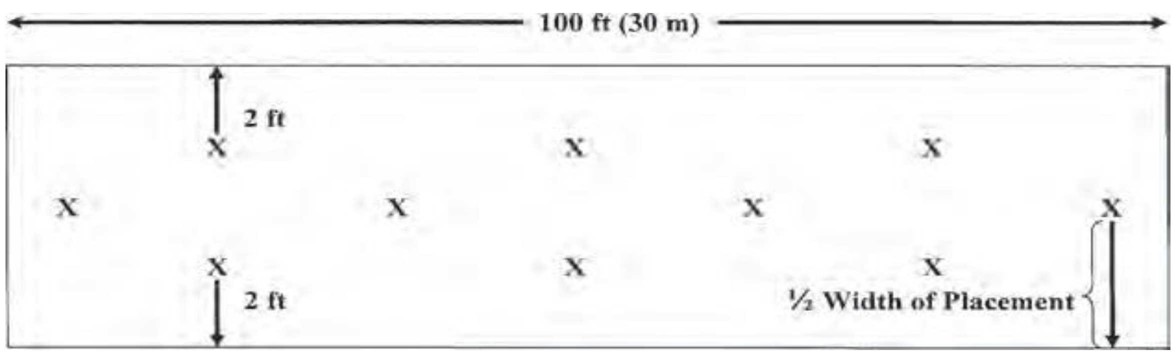

Figure 8.1: INDOT test section for LWD (for reference as DCP test section)

8.3.2 Florida Method of Test for Standard Test Method for Use of the Dynamic Cone Penetrometer

(Numbering in this section applies only to this section, Standard Test Method for Use of the Dynamic Cone Penetrometer.)

DRAFT

Florida Method of Test
for
Standard Test Method for Use of the Dynamic Cone Penetrometer
Designation: FM D6951

FM D6951 is identical to ASTM D6951 except for the following provisions.

1. Scope

1.1 This test method covers the measurement of the penetration rate of the dynamic cone penetrometer (DCP) with an 8-kg [17.6-lb] hammer through undisturbed soil or compacted materials, or both.

2. Reference Documents

2.1 ASTM Standards

ASTM D6951 Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

2.2 AASHTO/ASTM Standards

AASHTO T 99 / ASTM D698 - Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in. Drop

2.3 Florida Sampling and Testing Methods

FM 1-T238. Density of Soils and Bituminous Concrete Mixtures in Place by the Nuclear Method

FM 5-507 Determination of Moisture Content by means of a Calcium Carbide Gas Pressure Moisture Tester

FM 1-T180 Moisture Density Relations of Soils using a 10-lb. Rammer and an 18-in. Drop

2.4 Manufacturer's User Guide

Follow the instructions contained in the manufacturer's user guide.

3. Procedure A Maximum Allowable Penetration

The maximum allowable penetration will be in accordance with either of the following criteria:

A. Developmental Specification 120-10.2 Acceptance Criteria Dynamic Cone Penetrometer *(Additional field data is needed to expand the table include in the proposed Developmental Specification.)*
B. Test section for each material type

For materials not included in the table referenced previously, a test section shall be constructed to determine the maximum allowable penetration.

3.1. Test Sections

Test sections shall be constructed in the presence of the Engineer with the equipment provided by the Contractor.

3.2 Test Section with DCP

A test section shall be constructed and DCP testing will be performed to determine the minimum number of blows/layer. The roller shall be operated in the vibratory mode and initially four passes shall be placed on the test section. The average penetration of the ten random tests will be determined after completion of the four passes. One additional pass of the roller in the vibratory mode shall be made, and ten DCP tests will be taken at the same locations. If the difference between the average DCP test values obtained from passes four and five is equal to or less than *(needs to be established)* mm, the compaction will be considered to have peaked, and the average of the ten DCP values at five passes will be used as the maximum allowable penetration. If the difference between the average DCP test values is greater than *(needs to be established)* mm, an additional roller pass in the vibratory mode shall be placed, and ten DCP tests will be taken at the same locations. This procedure will continue until the difference between the average of the ten DCP tests of consecutive roller passes is equal to or less than *(needs to be established)* mm. The maximum allowable penetration will be the lowest average of the ten DCP test values.

3.3 DCP Testing with Density Control

DCP testing will be performed concurrently with density testing performed in accordance with FM 1-T238 Density of Soils and Bituminous Concrete Mixtures in Place by the Nuclear Method.

The density shall meet the requirements of the applicable specification based on obtaining a minimum QC density of the specified percentage of either the standard or modified Proctor.

Refer to Developmental Specification 120.10 Acceptance Program for Independent Assurance (IA) requirements for Initial Equipment Comparison.

3.4 DCP Blows per Layer

The full thickness of each material layer shall be compacted to achieve an acceptable DCP blow count (blows/layer). For test purposes, a layer (lift thickness) is defined as a 6-in. compacted thickness. Other thicknesses require prior written approval of the Engineer.

4. Procedure B Penetration Index

1. To properly seat the DCP (coned tip), two hammer blows are required. Carefully raise the sliding weighted hammer until it meets the handle, then release the hammer under its own weight. Repeat this process one more time for a total of two complete blows. If the seating process causes initial penetration exceeding 40 mm (1.6 in.), move the test site at least 12 in. from the previous test location and reseat the cone. If the second test site still does not meet the seating criteria, DCP testing for acceptance cannot proceed. The area being tested must be recompacted.

2. Record the penetration measurement after seating using the graduated rule on the DCP. DCPs with a "magnetic ruler" have been shown to be an acceptable alternative. The measurement is taken to the nearest 2.5 mm (0.1 in.).

3. Carefully raise the hammer until it meets the handle, then release the hammer under its own weight. Repeat this process two more times for a total of three times.

4. Record the final penetration measurement using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.).

5. Subtract the measurement in Step 2 from the measurement in Step 4 and then divide the difference of the measurements by the number of blows (n) required for testing. If necessary, convert from in. to mm. Round off all test results to the nearest mm or one-tenth of an inch.

Seat = Penetration Reading (2 blows) - Initial Reading (Eq. 8.3)

Calculate the DCP Penetration Index (PI) by using the following formula:

DCP PI = {Penetration Reading (n blows) – Penetration Reading (2 blows)} / n (Eq. 8.4)
Where: n = total number of blows

(The actual number of blows for initial penetration and for final penetration may vary depending on soil properties. This needs to be established through continued field testing.)

6. Upon conclusion of the DCP test, obtain a sample of the material being tested and determine the moisture content by using a speedy moisture tester in accordance with FM 5-507 Determination of Moisture Content by means of a Calcium Carbide Gas Pressure Moisture Tester.

7. Record the moisture content

5. Moisture

Moisture requirements accompany this procedure. For a valid DCP test, allowable moisture ranges are compared to the optimum moisture percentage as determined by the applicable Proctor for the particular soil type.

6. Maintenance and Handling

- Due to the fact that the DCP is driven into the ground, sometimes into very hard soil layers, regular maintenance and care are required. To ensure that the DCP operates properly, the following guidelines must be followed.
- Monitor the condition of the connection bolt. Extra bolts should be kept in the DCP carrying case. These bolts frequently can become stripped or broken and may need to be replaced during testing.
- Keep the upper shaft clean. Lubricate very lightly with oil if binding develops. Frequently wipe both shafts clean with a soft cloth during use.
- Monitor the DCP for excessive wear on any of the components and make repairs as needed. Because the DCP is a standardized testing device, its overall weight and dimensions must not change from specifications.
- The cone tip should be replaced when the diameter of its widest section is reduced by more the 10% (2 mm [0.08 in.]) or the cone's surface is gouged by rocks. Inspect the cone tip before and after each test. Regardless, the cone tip should be replaced at least once a year.
- Never extract the DCP from the test hole by forcefully striking the hammer against the handle. Striking the handle causes accelerated wear and may lead to broken welds and connections. At least once a year, all welds on the DCP should be inspected critically for hairline or larger cracks.
- Do not lay the device on the ground when not in use. The DCP should be kept in its carrying case to avoid bending the shafts. Straightness of the shafts is extremely important. The hammer cannot free fall if the shafts are bent. The straightness of the shafts should be measured critically and reviewed annually.

7. Safety

- Always use caution to avoid pinching fingers between the hammer and the anvil. During testing, use the handle to hold shafts plumb. Do not hold the DCP near the anvil area.
- It is important to lift the hammer slowly and drop it cleanly, allowing at least two seconds to elapse between drops. Lifting and dropping too rapidly may affect results because the hammer's full energy may not be allowed to transfer to the lower shaft. This will cause incorrect test results.

(End, Standard Test Method for Use of the Dynamic Cone Penetrometer.)

8.3.3 Lightweight Deflectometer

Both Mn/DOT and INDOT use the LWD to measure maximum allowed deflection rather than modulus. From conversations with personnel from both departments, the use of deflection values as opposed to modulus values is based on the ability of the technicians and contractors to understand the meaning of the test results. A soil layer with a higher level of compaction should deflect less under load. If the test indicates that the deflection is less than the target value for that particular soil, the test passes. If it deflects more than the target value, continue the rolling operation and retest. According to Mn/DOT personnel, technicians also record modulus for design comparison.

Mn/DOT standardized the LWD mass at 10 kg (22.0 lbs.) with the drop height at 50 cm (19.7 in.) and the plate diameter at 20 cm (7.9 in.). This configuration was selected for ease of use and to ensure an appropriate influence depth to test for a lift of compacted pavement foundation material (Davich et al., 2006). Because the buffer type affects the force delivered to the ground, Mn/DOT specifies that a force of 6.28 kN be delivered to the ground. This equates to a stress of 0.2 MPa for a 20-cm diameter plate (Siekmeier et al., 2009). INDOT uses a 10-kg (22.0 lbs.) mass with drop height at 72 cm (28.3 in.) and plate diameter at 20 cm (7.9 in.).

The LWD's standard drop heights were used throughout the project. They are: LWD-1, 720 mm; and LWD-2, 850 mm. Both were equipped with 300-mm (12-in.) diameter load plates.

Due to the limited amount of data available, it is likely that soils other than those included in Table 8.8 will be encountered. In this instance, the procedure for the construction of a test section or sections consisting of the material(s) being used is included in the proposed test method. This test section is based on the test section used by INDOT for testing with the LWD (see Figure 8.1).

The research team recommends acceptance based on deflection, but modulus values also should be recorded. These values could prove to be valuable for future pavement design purposes.

8.3.4 Florida Method of Test for Standard Test Method for Measuring Deflections Using a Portable Impulse Plate Load Test Device

(Numbering in this section applies only to this section, Standard Test Method for Measuring Deflection Using a Portable Impulse Plate Load Test Device.)

DRAFT

Florida Method of Test
for
Measuring Deflection Using a Portable Impulse Plate Load Test Device
Designation: FM E2835

FM E2835 is identical to ASTM E2835 except for the following provisions.

1. Scope

1.1 This method uses a Portable Impulse Plate Load Test Device and covers the measurement of deflection of the load plate rather than the deflection of the surface of the layer being tested. The load is a force pulse generated by a falling mass dropped onto a spring assembly that transmits the load pulse to a plate resting on the material being tested. *LWDs manufactured by Zorn Instruments are in this category.*
1.2 For lightweight deflectometers where the load is a force pulse generated by a falling weight dropped on a buffer system that transmits the load pulse through a plate resting on the material being tested, refer to ASTM E2583 – 07 (2011). *LWDs manufactured by Dynatest Incorporated, Inc. are in this category.*

2. Reference Documents

2.1 ASTM Standards

ASTM E2835 - 11 Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load Test Device
ASTM E2583 – 07 (2011) Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD)

2.2 AASHTO / ASTM Standards

AASHTO T 99 / ASTM D698 - Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in. Drop

AASHTO T 27 / ASTM C136 - Sieve Analysis of Fine and Coarse Aggregates

2.3 Florida Sampling and Testing Methods

FM 1-T238. Density of Soils and Bituminous Concrete Mixtures in Place by the Nuclear Method
FM 5-507 Determination of Moisture Content by Means of a Calcium Carbide Gas Pressure Moisture Tester
FM 1-T011 Total Amount of Material Finer Than 0.075 mm (No. 200) Sieve in Aggregate
FM 1-T088 Particle Size Analysis of Soils
FM 1-T089 Determining the Liquid Limit of Soils
FM 1-T090 Determining the Plastic Limit and Plasticity Index of Soils
FM 1-T180 Moisture Density Relations of Soils Using a 10-lb. Rammer and an 18-in. Drop

2.4 Manufacturer's User Guide

Follow the instructions contained in the manufacturer's user guide.

3. Definitions

Deflection Test Measurement. The average deflection measured from the fourth, fifth, and sixth drops in the testing sequence. The first, second, and third drops in the testing sequence are seating drops.

Seating Drops. The first, second, and third drops in the testing sequence and are not used for acceptance.

LWD Target Value. The Maximum Allowable Deflection (or Minimum Allowable Elastic Modulus) values allowed for a given soil or material type.

Maximum Allowable Deflection. The maximum settlement allowed beneath the loading plate.

4. Procedure

1. Rotate the loading plate approximately 45 degrees back and forth to seat the plate. The plate should not move laterally with successive drops of the falling weight.
2. Place the force generating device onto the loading plate. Hold the guide rod perpendicular to the loading plate.
3. Conduct three seating drops by raising the falling weight to the release mechanism, allowing the hammer to fall freely, and catching the falling weight after the weight rebounds from striking the plate.

4. Following the three seating drops, conduct three drops of the falling weight and record the data for each drop. A test is considered invalid if the operator does not catch the falling weight after the weight rebounds from the load plate or the load plate moves laterally. A new test area is required at least 2 ft. away from the original area of testing when the test is invalid. If the change in deflection is 10% or greater for any two consecutive drops, the material shall require additional compaction or aeration and steps 4.1, 4.2, and 4.3 shall be repeated.
5. Record the test drop deflection measurements on the data collection form.

5. Maximum Allowable Deflection

The maximum allowable deflection will be in accordance with either of the following criteria:

A. Developmental Specification 160-4.2.3.1 (Stabilizing) or 200-7.2.1 (Rock Base) Acceptance Criteria Lightweight Deflectometer. *(Additional field data is needed to expand the table included in the proposed Developmental Specifications.)*

B. Test section for each material type.

For materials not included previously, a test section shall be constructed to determine the maximum allowable deflection.

6. Test Sections

Test sections shall be constructed in the presence of the Engineer with the equipment provided by the Contractor to determine the maximum allowable deflection.

6.1 Test Section with LWD

A test section shall be constructed and LWD testing will be performed to determine the maximum allowable deflection. The roller shall be operated in the vibratory mode and initially four passes shall be placed on the test section. The average deflection of the ten random tests will be determined after completion of the four passes. One additional pass of the roller in the vibratory mode shall be made, and ten LWD tests will be taken at the same locations. If the difference between the average LWD test values obtained from passes four and five is equal to or less than 0.02 mm, the compaction will be considered to have peaked and the average of the ten LWD values at five passes will be used as the maximum allowable deflection. If the difference between the average LWD test values is greater than 0.02 mm, an additional roller pass in the vibratory mode shall be placed and ten LWD tests will be taken at the same locations. This procedure will continue until the difference of the average of the ten LWD tests between consecutive roller passes is equal to or less than 0.02 mm. The maximum allowable deflection will be the lowest average of the ten LWD test values.

6.2 LWD Testing with Density Control

In the test section, LWD testing will be performed concurrently with density testing performed in accordance with FM 1-T238 Density of Soils and Bituminous Concrete Mixtures in Place by the Nuclear Method. The density shall meet the requirements of the applicable specification based on obtaining a minimum QC density of the specific percentage of either the standard or modified Proctor. The maximum allowable deflection will be the average of the ten LWD test values.

7. Compaction Acceptance with Lightweight Deflectometer

The maximum allowable deflection will be determined from the test section or as specified. The optimum moisture content and gradation will be determined by performing AASHTO T 99 Standard Proctor; FM 1-T180 Modified Proctor; FM 1-T011 Percent Passing No. 200 Sieve; AASHTO T 27 Sieve Analysis; FM 1-T88 Particle Size Analysis; FM 1-T089 Liquid Limit; and FM 1-T090 Plasticity Index on representative samples of each type of material.

The moisture content of the material being placed shall be controlled within -3 percentage points of the optimum moisture content and the optimum moisture content prior to placement. The frequency of the moisture tests for the material being placed will be a minimum of two moisture tests each day materials are placed. Moisture testing will be performed in accordance with FM 5-507.

Acceptance of the compaction will be determined by averaging three LWD tests obtained at a random station. The average deflection shall be equal to or less than the maximum allowable deflection allowed in the specifications or determined by the test section. The frequency of testing will be one test for each LOT. If the average deflection is not equal to or less than the maximum allowable deflection, a moisture test will be performed. Additional LWD tests will be taken at the same locations after 24 hours, and the material will be accepted if the LWD tests are equal to or less than the maximum allowable deflection.

7.1 LWD Testing Constraints

- (A) Do not test within 2 ft. of the water table
- (B) The LWD deflection method cannot be used for embankment thicknesses:
  - (1) less than 6 in. compacted thickness, or
  - (2) greater than 12 in. compacted thickness

7.2 Construction Requirements

Compact the entire lift to achieve the LWD target value. Ensure that the same LWD target value parameter for each soil type is used throughout the entire project. Re-evaluate the selected LWD target value and contact the Engineer when failing results consistently occur.

7.3 Moisture Requirements

Moisture percentage shall be determined in accordance with FM 5-507. An NDG moisture content determination in accordance with FM 1-T238 may be used for limerock, cemented coquina, and shell-rock base material with the use of an approved NDG.

8. Safety

- Always use caution to avoid pinching fingers or extremities with falling weight. During testing, use the handle to hold shaft plumb.
- It is important to lift the weight slowly and drop it cleanly. Failure to do so will cause incorrect test results.

(End, Standard Test Method for Measuring Deflection Using a Portable Impulse Plate Load Test Device.)

Chapter 9: Specification Development

State DOT specifications contain the requirements for compaction control used in the acceptance decision process. These specifications can be either method or procedure, or end-product type specifications. Method or procedure specifications specify the type, weight, and number of passes of compaction equipment as well as the lift thickness or material volume. End-product specifications, often referred to as performance specifications, require the contractor to compact the soil layer to achieve a target value, typically a percentage of the maximum density determined by a laboratory Proctor test.

9.1 Current Practice (Typical)

The NDG is the most commonly used device to ensure that the appropriate (minimum) in-place density is achieved. Densities—both laboratory and in-place—are functions of moisture content at the time of compaction. Most FDOT specifications do not include moisture content or a moisture content range. Field moisture measurement typically is determined by use of the speedy moisture tester, although base material moisture can be determined with the use of an approved NDG.

9.2 Modulus/Stiffness-Based Specification

Several requirements must be satisfied in order to transition from density-based specifications to modulus/stiffness-based specifications:

1. The proposed specification(s) shall be based on the field (in-place) measurement of modulus or provide a conversion to a modulus/stiffness value.
2. The type of device may be dependent on the type of material tested; e.g., +3/4 in. nominal maximum aggregate size (NMAS) or -3/4 in. NMAS.
3. In-place moisture content at the time of testing shall be determined.
4. Knowledge of in-place moduli would be beneficial if future design criteria are based on modulus values.
5. The specification(s) shall provide well defined methods applicable to the materials commonly encountered on FDOT construction projects.
6. Although the DCP does not measure stiffness/modulus, it is included as it could be used in conjunction with equipment that measures stiffness/modulus such as the LWD.
7. Although a particular device from a specific manufacturer cannot be specified, consideration should be given to devices that are commercially available and represented by a sales and service network, preferably located in the United Sates, and have been determined acceptable based on this research.

8. Devices that have met the criteria to be included in the test methods published by the American Society for Testing Materials (ASTM) should be given greater consideration than devices without such methods.
9. To provide continuity validation, it is anticipated that a significant period of time be allocated for transition from the conventional moisture-density specifications to modulus-based specifications.
10. Field measurements shall be able to distinguish between acceptable and unacceptable states of compaction by being sensitive, accurate, and precise.

From our results, moduli from laboratory tests can differ significantly when compared to in situ results. These differences can be due to sampling disturbance, differences in the state-of-stress between the specimen and in-place material, long-term time effects, and inherent errors in the field and laboratory test procedures (Anderson and Woods, 1975).

9.2.1 Modulus/Stiffness-Based Design

The FDOT has not adopted the ME-PDG criteria for flexible pavement design. However, recent research, such as "Comparison of Resilient Modulus Values used in Pavement Design," recognized the need to equate moduli from in situ testing to laboratory modulus values. Although the focus of this research was the design of flexible pavement overlays, knowledge of in-place soil properties, such as modulus, would be of similar value for new construction design (Oh and Fernando, 2011).

During the 2014 site visit, Mn/DOT personnel stated that although in-place soil testing done with the LWD is based on deflection, modulus values also are reported for possible use in future designs.

The Maryland Department of Transportation State Highway Administration collected data to create a table of recommended moduli for unbound materials, "Catalog of Material Properties for Mechanistic-Empirical Pavement Design" (Schwartz and Li, 2011).

Table 9.1: Maryland DOT typical modulus values for design

Material	**Modulus (psi)**		
Base/Subbase Materials	**Minimum**	**Typical**	**Maximum**
Graded Aggregate Base	15,000	25,000	45,000
Gravel	10,000	15,000	30,000
Soil Contaminated Aggregate Base	3,000	10,000	20,000
Capping Borrow	10,500	10,500	10,500

Table 9.1, continued

Subgrade Soils	**Typical**
Silts and Clays (w/ high compressibility)	1,000–2000
Fine Grained Soils with Silts and Clays (w/ low compressibility)	2,000–3,000
Poorly Graded Sands	3,000–4,500
Gravelly Soils, Well Graded Sands, and Sand/Gravel Mixtures	4,500–10,000

With an accurate method for measuring in-place moduli with a portable device, a similar table of moduli values based on typical Florida materials could be compiled. Correlations of the actual in-place measurements and future design criteria could be made.

9.3 Proposed Developmental Specifications

Developmental Specifications are specifications developed around a new process, procedure, or material approved for limited use by the State Specifications and Estimates Office. These specifications are signed and sealed by the professional engineer responsible for authorizing their use and monitoring their performance in the field. A Developmental Specification must be requested from the District Specifications Office on a project-by-project basis.

Developmental Specifications are provided for Section 120 Excavation and Embankment (Soil Compaction Testing with the Dynamic Cone Penetrometer); Section 160 Stabilizing (Stabilized Subgrade Compaction Testing with the Lightweight Deflectometer); and Base Courses Section 200 Rock Base (Base Compaction with the Lightweight Deflectometer).

Equipment and target values vary from section to section.

(Numbering in this section applies only to this section and is based on the Standard Specifications for Road and Bridge Construction.)

EARTHWORK AND RELATED OPERATIONS
DEVELOPMENTAL SPECIFICATION SECTION 120
EXCAVATION AND EMBANKMENT

SOIL COMPACTION TESTING WITH THE DYNAMIC CONE PENETROMETER

The following SUBARTICLES 120-9.1, 120-10.1.1, 120-10.1.2, 120-10.1.4.2, 120-10.1.5, 120-10.1.6, 120-10.2, 120-10.3.1, and 120-10.4.2 are deleted and the following substituted:

120-9 Compaction Requirements.
120-9.1 Moisture Content: Compact the materials at a moisture content such that the specified compaction can be attained. If necessary to attain the specified compaction, add water to the material or lower the moisture content by manipulating the material or allowing it to dry, as appropriate.

120-10 Acceptance Program.
120-10.1 General Requirements:

120-10.1.1 Initial Equipment Inspection: Before initial production, perform an inspection of all Quality Control, Verification, and Independent Assurance DCP equipment. Repair and replace any Quality Control DCP that does meet the ASTM D6951 and FM D6951 requirements. Calibrate all Quality Control DCPs annually.

120-10.1.2 Initial Production Lot: Before construction of any other LOT, prepare a 500-ft. initial control section consisting of one full LOT in accordance with the approved QC Plan for the project. Notify the Engineer at least 24 hours prior to production of the initial control section. Perform all QC tests required in 120-10.1.4. When the initial QC test results pass specifications, the Engineer will perform a Verification test to verify compliance with the specifications. Do not begin constructing another LOT until successfully completing the initial production LOT. The Engineer will notify the Contractor of the initial production lot approval within three working days after receiving the Contractor's QC data when test results meet the following conditions:

QC tests must meet the specifications.

Verification test must meet the specifications.

If Verification test result fails the requirements of 120-10.2, correct the areas of noncompliance. The QC and Verification tests will be then repeated. The Engineer will reject the Contractor's QC Plan after three unsuccessful Verification attempts. Submit a revised QC Plan to the Engineer for approval.

120-10.1.4.2 Compaction Requirements: Ensure compliance to the requirements of 120-10.2 by DCP testing in accordance with FM D6951. Determine the in-place moisture content for each test. Use FM 1-T 238, FM 5-507 (Determination of Moisture Content by Means of a Calcium Carbide Gas Pressure Moisture Tester) or ASTM D4643 (Laboratory Determination of Moisture Content of Granular Soils by use of a Microwave Oven) for moisture determination.

120-10.1.5 Department Verification: The Engineer will conduct Verification tests in order to accept all materials and work associated with 120-10.1.4. The Engineer will verify the QC results if they meet the Verification Comparison Criteria; otherwise, the Engineer will implement Resolution procedures. The Engineer will select test locations, including Station, Offset, and Lift, using a random number generator based on the LOTs under consideration. Each Verification test evaluates all work represented by the QC testing completed in those LOTs. In addition to the Verification testing, the Engineer may perform additional Independent Verification (IV) testing. The Engineer will evaluate and act on the IV test results in the same manner as Verification test results. When the project requires less than four QC tests per material type, the Engineer reserves the right to accept the materials and work through visual inspection.

120-10.1.6 Reduced Testing Frequency: Reduced testing frequency will not be permitted with the use of non-nuclear density devices.

120-10.2 Acceptance Criteria:
Dynamic Cone Penetrometer: Obtain a maximum allowable penetration value. *Additional testing on other materials is necessary.*

Dynamic Cone Penetrometer						
A-2-4 with up to 15% fines	**< Opt. Moisture Target***	**< Opt. Moisture Lower CI**	**< Opt. Moisture Upper CI**	**@Opt. Moisture Target**	**@Opt. Moisture Lower CI**	**@Opt. Moisture Upper CI**
in./blow	0.275	0.259	0.292	0.348	0.326	0.369
blows/6 in.	22	23	21	17	18	16
blows/12 in.	44	46	41	35	37	33
A-3						
in./blow	0.351	0.319	0.382	0.458	0.429	0.486
blows/6 in.	17	19	16	13	14	12
blows/12 in.	34	38	31	26	28	25

*For A-2-4, -3% to optimum. For A-3, -5% to optimum.

120-10.3 Additional Requirements:
120-10.3.1 Frequency: Conduct QC sampling and testing at a minimum frequency listed in the following table. The Engineer will perform Verification sampling and tests at a minimum frequency listed in the following table.

Test Name	Quality Control	Verification	Verification of Shoulder-Only Areas, Bike/Shared Use Paths, and Sidewalks
Standard Proctor Maximum Density	One per soil type	One per soil type	One per soil type
Compaction	One per LOT	One per four LOTS and for wet conditions, the first lift not affected by water	One per two LOTs
Soil Classification	One per Standard Proctor Maximum Density	One per Standard Proctor Maximum Density	One per Standard Proctor Maximum Density

120-10.4.2 Compaction Testing: When a Verification or Independent Verification test fails the Acceptance Criteria, retest the site within a 5-ft. radius and the following actions will be taken:
1. If the QC retest meets the Acceptance Criteria and meets the 120-10.1.1 criteria when compared with the Verification or Independent Verification test, the Engineer will accept those LOTs.
2. If the QC retest does not meet the Acceptance Criteria and compares favorably with the Verification or Independent Verification test, rework and retest the LOT. The Engineer will re-verify those LOTs.
3. If the QC retest and the Verification or Independent Verification test do not compare favorably, complete a new comparison analysis as defined in 120-10.1.1. Once acceptable comparison is achieved, retest the LOTs. The Engineer will perform new verification testing. Acceptance testing will not begin on a new LOT until the Contractor has a DCP that meets the comparison requirements. Record QC test results in the log book on approved Department forms provided by the Engineer. Submit the original, completed density log book to the Engineer at final acceptance.

DEVELOPMENTAL SPECIFICATION SECTION 160
STABILIZING

SUBGRADE COMPACTION TESTING WITH THE LIGHTWEIGHT DEFLECTOMETER

The following SUBARTICLES 160-4.1, 160-4.2.3.1, 160-4.2.4, and 160-4.4.4 are deleted and the following substituted:

160-4 Acceptance Program.
160-4.1 General Requirements: Meet the requirements of 120-10, except use 160-4.2 instead of 120-10.2, 160-4.3 instead of 120-10.3, and 160-4.4 instead of 120-10.4.
160-4.2.3 Compaction Requirements:
160-4.2.3.1 General: Within the entire limits of the width and depth of the areas to be stabilized, other than as provided in 160-4.2.3.2, obtain a minimum lightweight deflectometer (LWD) deflection as determined by FM E2835.

	Below Optimum Moisture (95% Confidence Interval)			**At Optimum Moisture (95% Confidence Interval)**		
Accelerometer-type LWD Deflection (mm)	**Target***	**Lower CI**	**Upper CI**	**Target**	**Lower CI**	**Upper CI**
Stabilized Subgrade	0.39	0.33	0.44	0.55	0.46	0.65

*-1% to optimum.

160-4.2.4 Frequency: Conduct QC LWD testing at a minimum frequency of one maximum allowable deflection test per LOT. The Engineer will perform Verification LWD testing at a minimum frequency of one per four LOTs.
160-4.4.4 Compaction: When a Verification or Independent Verification LWD test does not meet 160-4.2.3 (Acceptance Criteria), retest at a site within a 5-ft. radius of the Verification test location and observe the following:
1. If the QC retest meets the Acceptance Criteria and compares favorably with the Verification or Independent Verification test, the Engineer will accept the LOTs in question.
2. If the QC retest does not meet the Acceptance Criteria and compares favorably with the Verification or Independent Verification test, rework and retest the material in that LOT. The Engineer will re-verify the LOTs in question.
3. If the QC retest and the Verification or Independent Verification test do not compare favorably, complete a new equipment comparison analysis as defined in 120-10.1.2. Once acceptable comparison is achieved, retest the LOTs. The Engineer will perform new verification testing. Acceptance testing will not begin on a new LOT until the Contractor has an LWD that meets the comparison requirements.

BASE COURSES
DEVELOPMENTAL SPECIFICATION SECTION 200
ROCK BASE

BASE COMPACTION TESTING WITH THE LIGHTWEIGHT DEFLECTOMETER

The following SUBARTICLES 200-7.1, 200-7.2.1, 200-7.2.2, and 200-7.4.3 are deleted and the following substituted:

200-7 Acceptance Program.
200-7.1 General Requirements: Meet the requirements of 120-10, except use 200-7.2 instead of 120-10.2, 200-7.3 instead of 120-10.3, and 200-7.4 instead of 120-10.4.
200-7.2 Acceptance Criteria:
200-7.2.1 Compaction: Within the entire limits of the width and depth of the base, obtain a maximum lightweight deflectometer (LWD) deflection as shown in the following table.

	Below Optimum Moisture (95% Confidence Interval)			At Optimum Moisture (95% Confidence Interval)		
Accelerometer-type LWD Deflection (mm)	**Target***	**Lower CI**	**Upper CI**	**Target**	**Lower CI**	**Upper CI**
Limerock Base	0.23	0.19	0.27	0.26	0.23	0.29

*-2% to optimum.
200-7.2.2 Frequency: Conduct QC LWD testing at a minimum frequency of one maximum allowable deflection test per LOT. The Engineer will perform Verification LWD testing at a minimum frequency of one per four LOTs.
200-7.4.3 Compaction: When a Verification or Independent Verification LWD test does not meet the requirements of 200-7.2.1 (Acceptance Criteria), retest at a site within a 5-ft. radius of the Verification test location and observe the following:
1. If the Quality Control retest meets the Acceptance Criteria and compares favorably with the Verification or Independent Verification test, the Engineer will accept the LOTs in question.
2. If the Quality Control retest does not meet the Acceptance Criteria and compares favorably with the Verification or Independent Verification test, rework and retest the material in that LOT. The Engineer will re-verify the LOTs in question.
3. If the Quality Control retest and the Verification or Independent Verification test do not compare favorably, complete a new equipment comparison analysis as defined in 120-10.1.1. Once acceptable comparison is achieved, retest the LOTs. The Engineer will perform new verification testing. Acceptance testing will not begin on a new LOT until the Contractor has an LWD that meets the comparison requirements.
(End, Developmental Specification Sections 120, 160, and 200.)

Chapter 10: Portable Equipment

10.1 Equipment Advantages and Disadvantages

Comprehensive testing of several portable devices included the BCD, CIST, DSPA, DCP, GeoGauge, and LWDs of both the geophone and accelerometer type. Following the Phase I data collection, statistical analyses were conducted. Referencing the COV results in Table 6.1, the BCD and DSPA had the second-highest and highest COVs, respectively. From an operator's perspective, the BCD required the most human interaction—placing a uniform amount of pressure on the device's handles—compared to all other devices. The DSPA required a laptop computer for data gathering. Interpretation of the data often was necessary, making use by a non-technical inspector questionable. The DSPA is significantly more expensive than all other devices tested, with the exception of the geophone-type LWD. For these reasons, the BCD and DSPA were eliminated from further evaluation.

The first geophone-type LWD under evaluation, LWD-P, was configured with a Bluetooth-connected personal data assistant (PDA) data collector. The arrangement was unreliable. Although this unit was a prototype, it was discovered that our experience was typical of other users with newer production models. This LWD was replaced with a newer version geophone-type LWD, LWD-2. A Bluetooth connectivity feature still was utilized, but the data collector was replaced with a more rugged, field-type handheld unit. Although this unit was more reliable than the prototype configuration, it was not as user friendly as one might expect for field equipment used by inspectors who may not be very tech-savvy. Also, the operator had to be careful not to take the data collector out of Bluetooth range from the deflectometer. If this occurred, several steps were necessary to reinitiate communication between the deflectometer and the data collector.

The accelerometer-type LWD, LWD-1, was connected to its data collector with a cable and standard ¼-in. phone plugs, allowing for a trouble-free connection between the deflectometer and the data collector. Both LWD-1 and LWD-2 were prone to erratic measurements when used on soils that exceeded the optimum moisture content. This was the case even when in-place dry densities met the minimum 100% of the laboratory standard Proctor value. For all LWDs, a smooth surface is necessary. Although LWD-1 is constructed with stainless steel components, continued use in Florida conditions caused oxidation (rust) on the shaft.

The DCP used for this project was equipped with a "magnetic ruler." This option allowed for operation by a single technician as opposed to the typical two-technician team—one to raise and lower the weight and one to read and record the penetration depths. Even with this option, the DCP was the least expensive of all devices. Lack of confinement in the upper-most portion of the lift being tested, especially for the first few blows/drops of embankment materials, is of concern. The DCP was ineffective on most stabilized subgrade and base materials (stiff materials typically drier than optimum moisture) due to the amount of effort (blows per 6-inch or 12-inch layer).

The cone tip hanging up on one or more coarse particles such as those found in subgrade and base layers could influence the results. In many instances, testing on these layers was suspended in order to avoid damaging the equipment.

A new CIST was purchased for this project. It was easy to use, and the data collector was an improvement over the older CIST units owned by the FDOT. Care must be taken to prevent the cable running from the accelerometer to the data collector from being damaged by the falling accelerometer. Similar to the LWDs, soils with moisture contents significantly higher than optimum—even those meeting the minimum dry densities—presented problems. In the case of the CIST, the accelerometer penetrates the soil to the depth that the handle on the accelerometer hits the guide tube, thus preventing even further penetration into the soil being tested. From Tables 6.2 and 6.3, the CIST is a good choice for testing A-2-4 low fines soil and limerock stabilized subgrade.

The GeoGauge is easy to use, but proper placement on the test site is imperative. As with the LWDs, a smooth surface is necessary. For a field device, the GeoGauge is more susceptible to moisture (rainfall) than one would expect. Exposure to a brief rain shower required the unit to be sent back to the manufacturer for repair. The GeoGauge is a good choice for testing A-3 soil and limerock base.

The CIST, accelerometer-type LWD, and GeoGauge were comparably priced with one another and also with the NDG. Approximate costs are provided in Table 10.2.

Based on both the statistical analyses and the hands-on experience with the devices, it is recommended that the DCP be considered for testing embankment materials and LWD-1 be considered for testing stabilized subgrade and rock base in the field pilot testing program.

10.2 Nuclear Density Gauge Precision

From ASTM D6938 Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth) and AASHTO T 310 Standard Specification for In-Place Density and Moisture Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth), the criteria for judging the acceptability of wet density test results obtained by these test methods are given in Table 10.1. The figure in column three represents the standard deviations that have been found to be appropriate for the materials tested in column one. The figures given in column four are the limits that should not be exceeded by the difference between the results of two properly conducted tests. The figures given are based on an inter-laboratory study in which five test sites containing soils with wet densities as shown in column two were tested by eight different devices and operators. The wet density of each test site was determined three times by each device (ASTM, 2010; AASHTO, 2013).

Table 10.1: ASTM/AASHTO nuclear density gauge results of statistical analysis

Results of Statistical Analysis (Wet Density)

Direct Transmission:

Material	Average kg/m3 or (lbm/ft 3)	Repeatability Standard Deviation kg/m 3 or (lbm/ft3)	Reproducibility Standard Deviation kg/m 3 or (lbm/ft3)	95 % Repeatability Limit on the Difference Between Two Test Results kg/m3 or (lbm/ft3)	95 % Reproducibility Limit on the Difference Between Two Test Results kg/m3 or (lbm/ft3)
ML	2084 (130.1)	7.4 (0.46)	12.3 (0.77)	21 (1.3)	34 (2.1)
CL	1837 (114.7)	5.4 (0.34)	10.6 (0.66)	15 (0.9)	30 (1.9)
SP	1937 (120.9)	4.2 (0.26)	11.0 (0.68)	12 (0.7)	31 (1.9)

Backscatter:

Material	Average kg/m3 or (lbm/ft 3)	Repeatability Standard Deviation kg/m 3 or (lbm/ft3)	Reproducibility Standard Deviation kg/m 3 or (lbm/ft3)	95 % Repeatability Limit on the Difference Between Two Test Results kg/m3 or (lbm/ft3)	95 % Reproducibility Limit on the Difference Between Two Test Results kg/m3 or (lbm/ft3)
ML	1997 (124.6)	16.0 (1.00)	32.0 (2.00)	45 (2.8)	90 (5.6)

Referencing Table 10.1, this research began a process similar to that of the ASTM and AASHTO for determining repeatability and reproducibility. As additional data become available, it is suggested that this process continue. This data will be valuable in determining comparison criteria. As indicated in the Development Specifications (Section 120.10.1.1), perform a comparison test using the Quality Control, Verification and Independent Assurance equipment; ensure that the difference does not exceed the established comparison criteria; and repair or replace any (device) that does not compare favorably.

10.3 Nuclear Gauge Cost Analysis

A benefit-cost analysis is a systematic approach to estimating the strengths and weaknesses of alternatives and how well each alternative satisfies functional requirements. The results of the analysis are used to determine which option(s) will provide the best approach in labor, time, and cost savings. Often referred to as a benefit-cost ratio, the analysis is a summary of the overall economic value of a project or proposal. Both the benefits and the costs are expressed in monetary terms.

The ratio is the monetary gain realized by performing a project versus the expense incurred to execute the project. The higher the ratio, the better the investment. Costs and benefits can be both quantitative and qualitative as some benefits and costs cannot be measured exclusively in financial terms (David *et al.*, 2013).

A benefit-cost analysis:

- Determines the useful life of the project or the number of years over which the benefits and costs of the project need to be evaluated.
- Estimates in physical units all benefits and costs of the project for each year of its useful life, irrespective of whether they are monetary or non-monetary in nature.
- Converts physical units of benefits and costs into dollars, using appropriate prices and values.
- Recommends implementation of the project if benefits exceed the costs (Sharma, undated).

Table 10.2 provides current approximate purchase prices. The speedy moisture tester also is included. It is the current device and method for determining moisture in the field. Regardless of the device(s) selected, a field moisture determination will be necessary.

Table 10.2: Equipment purchase price

Equipment Purchase Price (approximate)	
Lightweight deflectometer accelerometer-type	$7,500
Lightweight deflectometer geophone-type	$22,500
Dynamic cone penetrometer	$2,002
GeoGauge w/verifier mass	$7,050
Briaud compaction device	current price N/A
Dirt seismic pavement analyzer w/laptop and case	$27,050
Clegg impact soil tester 4.5 kg	$3,950
Nuclear density gauge	$7,000
Speedy moisture tester	$1,557

10.4 Nuclear Density Gauge Requirements

NDGs require licensure for ownership, usage, transport, and storage. The license, Use of Sealed Sources in Portable Gauging Devices, is issued by the State of Florida Department of Health, Bureau of Radiation Control. In accordance with Regulatory Guide 6.20, Revision 5, Issuance Date August 2007, radioactive materials license fees are as follows:
Section (III) l. (I) portable gauging devices, application fee $726; annual fee $1,769; and annual reclamation fee $57.95. A new license is valid for five years. Portable gauges fall under

Specific Licensing criteria. As such, there are no additional fees for additional devices as long as they are stored at the same site.

Each user is required to be monitored for photon X-ray, gamma ray, beta particle, and neutron radiations. A typical dosimeter service is about $147 per year per person. This includes the dosimeter, analysis of dosage, and reporting. Shipping, receiving, and return are additional. The annual occupational dose and the monitoring period dose are determined from analysis of the individual dosimeters. Radiation exposure is based on exposure over one's lifetime. Dosage is carried forward when an individual changes employers.

Gauge disposal, for estimating purposes, is $1,500 to $1,600, depending on location, type of source, and origin of source. This is based on gauges with U.S. sources and disposal in the U.S. Calibration is required annually, which costs about $300 excluding shipping. Rates vary based on the type of gauge and measurement depth (1-in. increments or 2-in. increments). Leak tests are required either annually or semiannually, depending on the gauge. A leak test kit is approximately $30. The leak test is usually done onsite by the radiation safety officer (RSO).

Chapter 64E-5 of the Florida Administrative Code states, "Radioactive materials shall be used by individuals who are qualified by training and experience to protect public health, safety and the environment." Only personnel who have received proper training are permitted to operate, transport, or handle equipment containing radioactive materials or equipment producing ionizing radiation.

Per the Florida Department of Health, Bureau of Radiation Control, each gauge operator is required to complete an eight-hour training session and successfully complete a written or online examination. Gauge operators also are required to complete a HazMat refresher course every three years.

An RSO is required. This person has the authority to administer a radiation safety program. In order to do so, the RSO must have sufficient training and experience with radioactive materials. Basic RSO training consists of an eight-hour course and exam. Longer, more in-depth courses are available. Online courses also are available. The basic classroom RSO training costs approximately $400.

As shown in Table 10.3, the total cost for operating a single NDG throughout its typical service life of 15 years is $45,236. Other than the initial purchase price and annual calibration, all other costs are unique to the NDG due to its nuclear materials.

Table 10.3: Nuclear density gauge operating costs
(Based on today's dollars without consideration of discounting or the time value of money)

Nuclear Density Gauge Operating Costs															
Item Year	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
Purchase	7000														
License application and renewal	726				NC					NC					NC
License annual fee	1769	1769	1769	1769	1769	1769	1769	1769	1769	1769	1769	1769	1769	1769	1769
License annual reclamation fee	58	58	58	58	58	58	58	58	58	58	58	58	58	58	58
Safety/HazMat training	150														
RSO training	400														
Monitoring (dosimeter)	147	147	147	147	147	147	147	147	147	147	147	147	147	147	147
Calibration	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300
Leak test	30	30	30	30	30	30	30	30	30	30	30	30	30	30	30
Survey Meter	400														
HazMat training (renewal)			100			100			100			100			100
Disposal															1500
SUBTOTALS	**$10,980**	**$2,304**	**$2,404**	**$2,304**	**$2,304**	**$2,404**	**$2,304**	**$2,304**	**$2,404**	**$2,304**	**$2,304**	**$2,404**	**$2,304**	**$2,304**	**$3,904**
TOTAL								**$45,236**							

10.5 Net Present Worth (NPW)

To calculate the net present worth (NPW), the following assumptions are made:

- The useful life of all devices is 15 years.
- The interest rate is 10%.
- The NDG and LWD are calibrated annually, $300 each device per calibration.
- The DCP does not require calibration.
- The DCP requires disposable tips for stiffer materials, approximately $2 each, estimated 150 tests per year.
- Both the LWD and DCP will be needed in order to test all materials (embankment, subgrade, base).

The values in Table 10.3 are used to calculate the NPW for the NDG. The NPWs for the LWD and DCP include the purchase prices shown in Table 10.2 plus $300 per year for LWD calibration and $300 per year for DCP disposable tips.

NPW NDG = -$26,001
NPW LWD = -$9,100
NPW DCP = -$4,102

An NPW savings of $12,799 is realized when the NDG is replaced by the LWD and the DCP.

Chapter 11: Final Conclusions

The Phase I coefficient of variance (COV) analyses found that the GeoGauge had the lowest COV followed by the accelerometer-type LWD, the geophone-type LWD, the DCP, and the BCD, with the SPA having the highest overall COV.

In Phase II, measurements obtained from the selected equipment were compared to the modulus values obtained by the PLT, the laboratory M_R, and the NDG measurements. Correlations were developed through statistical analysis. Target values were developed for various soils for verification on similar soils that were field tested in Phase III.

Phase III, field testing, was performed on A-3 and A-2-4 embankments, limerock stabilized subgrade, limerock base, and graded aggregate base found on Florida Department of Transportation construction projects.

The Phase II and Phase III results provided potential trend information for future research—specifically, data collection for in-depth statistical analysis for correlations with the laboratory M_R for specific soil types under specific moisture conditions.

Based on the statistical analyses and the experience gained from extensive use of the equipment, the combination of the DCP and the LWD was selected for in-place soil testing for compaction control acceptance. Test methods and developmental specifications were written for the DCP and the LWD. The developmental specifications include target values for the compaction control of embankment, subgrade, and base materials.

Chapter 12: Future Research

Per the current FDOT acceptance requirements, although the compaction of a lift of material has met the current acceptance criteria—a minimum percentage of the laboratory Proctor dry density—there was no measurement of the soil properties.

Task #1: Additional target values for soils used for the construction of embankment, subgrade, and base need to be established.

In order to implement non-density devices and methods for soil compaction control for acceptance, additional field testing must be established to determine target values for soils encountered throughout the state. Both the Mn/DOT and INDOT have done considerable work for the use of the DCP and the LWD in lieu of density testing. However, both states recognize the need for continued data compilation and refinement. On a national level, the following recommendation comes from the NCHRP's Synthesis 20-05 (Synthesis of Information Related to Highway Problems), Topic 44-10 (Final Synthesis) Non-Nuclear Methods for Compaction Control of Unbound Materials, Suggestions for Future Research:

> A database for target values of in situ stiffness/strength measurements needs to be established for different soil types and moisture contents to facilitate the use of these devices in compaction control specifications. This database should be verified for local materials in each state before it is used in quality control. It is recommended that DOTs start with the DCP or LWD because the use of these devices has been successfully implemented in some states (Nazzal, 2014).

To begin this process, the proposed Florida Methods for the DCP and LWD containing the target values derived from this research project—as well as the proposed Developmental Specifications for Embankment, Subgrade, and Rock Base—should be utilized. For full implementation and replacement of the NDG, enough data should be generated to prove (if possible) that the accuracy and precision of the replacement device(s) are equal to or better than the current NDG method.

Task #2: Examine the performance of pavement where the underlying layer or layers were evaluated per in-place modulus/stiffness measurements as opposed to in-place density measurements.

In recent years, many state DOTs have experienced an increase in the severity and extent of permanent deformation (rutting) in hot mix asphalt (HMA) pavements. Rutting is caused by a combination of densification and shear-related deformation and may occur in any layer of a pavement structure. It develops with an increasing number of load applications.

When failure occurs in the HMA surface layer, depressions in the wheel paths are accompanied by upheaval along the sides of the wheel paths. When failure occurs in the supporting layers, there are depressions in the wheel paths, but upheaval along the sides of the wheel paths is minimal to none (White et al., 2002).

From NCHRP Synthesis 20-05 (Synthesis of Information Related to Highway Problems), Topic 44-10 (Final Synthesis) Non-Nuclear Methods for Compaction Control of Unbound Materials, Suggestions for Future Research:

> There is a need to fully understand the effects of using stiffness- and strength-based compaction control specifications on a pavement structure's longevity. This can be done by comparing the performance of similar pavement structures where conventional stiffness- and strength-based compaction control specifications have been used. Future studies might investigate the relationship between the in situ stiffness measurements of unbound pavement materials and subgrade soils and ultimate pavement performance (Nazzal, 2014).

It is proposed that the Heavy Vehicle Simulator (HVS) and the outdoor test pits located at the FDOT's SMO be utilized for this task. A plan would be developed to compact underlying materials side-by-side based on modulus/stiffness and density. These materials would be overlaid with an asphaltic concrete course that would be subjected to the forces generated by the HVS. Rutting would be measured. Comparisons would be made between the section compacted per modulus/stiffness measurements vs. the section compacted per density measurements.

Topic Panel 44-10 members include David J. Horhota, Florida DOT, and John A. Siekmeier, Minnesota DOT. NCHRP Committee for Project 20-05 members include Brian A. Blanchard, Florida DOT.

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Appendix A: Additional Literature Review

A.1 Background

Previous and ongoing research concludes that no one device in its current state provides the precision and repeatability required for testing all types of soils used in all types of construction. Section 2.2 of this report highlights the research conducted by state DOTs. Following is a brief overview of additional research by others.

A.2 Department of Transportation Research

A.2.1 Florida Department of Transportation Research

A summary of FDOT research pertaining to non-nuclear device in-place soil testing can be found in Section 2.2.5 of this report.

A.2.2 Other State Departments of Transportation Research

A summary of Minnesota, Indiana, Missouri, and Nebraska DOT research pertaining to non-nuclear device in-place soil testing can be found in Sections 2.2.1 through 2.2.4 of this report.

A.3 National Research

A.3.1 Strategic Highway Research Program and Federal Highway Administration

The National Research Council's Strategic Highway Research Program (SHRP) in 1993 and the Federal Highway Administration (FHWA) in 1997 funded development and testing of a seismic pavement (property) tester. Much of the data generated for the latter project originated in Florida. The emphasis was on development of a method whereby the design, construction, and pavement management aspects could be interrelated (Nazarian et al., 1993; Nazarian et al., 1999; Mahavadi, 1998; Gucunski, 2002; FHWA, 1997).

A.3.2 Federal Highway Administration and U.S. Department of Defense

The SSG also gained national attention in the 1990s. In this instance the emphasis was on the elimination of equipment requiring nuclear isotopes for operation. The FHWA joined with the U.S. Department of Defense to cosponsor a study to investigate the possible use of military technology to solve this problem. FHWA researchers partnered with industry in the redesign of an existing military device that used acoustic and seismic detectors to locate buried land mines. The result of this cooperative development is the SSG (FHWA, 2001; Adams, 2001).

A.3.3 Federal Highway Administration and University of New Mexico

In 2001 the University of New Mexico in cooperation with the FHWA evaluated the SSG. Their conclusions were twofold. First, the SSG results relating moisture, density, and stiffness were found to be consistent with earlier research. Second, it was determined that a laboratory target value was necessary to determine the degree of compaction in the field. Various trials were run placing the SSG in a 6-in. diameter Proctor mold filled with compacted soil. The ability to obtain consistent target values for stiffness in the laboratory proved to be elusive. There was agreement with other researchers who stated that future specifications for compaction control using this technology may require the control of moisture to allow for accurate stiffness monitoring via the SSG (Lenke et al., 2001).

A.3.4 Federal Highway Administration and Virginia Department of Transportation

In 2010, the Virginia Department of Transportation in conjunction with the FHWA studied the LWD, GeoGauge, and DCP in an effort to measure their suitability in measuring in situ pavement layer moduli. A high spatial variability was found for the stiffness modulus values measured by all three devices. There were no significant correlations among the results obtained with the devices. The effect of dry density was not evident, but moisture content showed a significant influence on the measured stiffness with all three devices, especially the LWD. The LWD was not recommended for use in construction quality control until further research is conducted to determine the causes of the high spatial variability and the effect of moisture on the LWD-measured modulus. The study further recommends that additional well controlled laboratory testing be performed to evaluate the effect of moisture on LWD-modulus measurements and that field studies are conducted to verify the findings (Hossain, 2010).

A.3.5 National Cooperative Research Program

Recognizing the vast amount of research conducted and the general lack of consensus on the subject, the National Cooperative Research Program (NCHRP) commissioned a synthesis, "Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design." This project, completed in 2008, examined both laboratory and in situ test methods for determining resilient modulus, as well as correlations that relate resilient moduli properties with basic soil properties and compaction conditions. In conclusion, most of the studies developed correlations accurately predicting moduli properties. However, correlations with high R^2 values from the statistical regression analysis have provided poor predictions of resilient properties when attempted on other soils. Several DOTs, including the FDOT, participated in this project (Puppala, 2008).

A.4 University-Based Research

A.4.1 Louisiana State University

In 2003 and 2004 the Louisiana State University (LSU) and the Louisiana Research Center (LRC) conducted several studies. Field and laboratory tests included evaluation of the GeoGauge, LWD, and DCP in conjunction with the PLT and FWD test results. California Bearing Ratio (CBR) laboratory tests also were conducted on samples collected during the field and laboratory testing programs. A statistical analysis revealed good correlations between the measurements obtained from the three investigated devices and those obtained from the "standard" tests. In 2008, additional statistical analyses were conducted on the data generated by the earlier field and laboratory studies. These later analyses were for the development of models to estimate the M_R of cohesive and granular soils from the test results of the in situ testing devices (Seyman, 2003; Nazzal, 2003; Abu-Farsakh et al., 2004; Gudishala, 2004; Mohammad et al., 2008).

A.4.2 Iowa State University

Iowa State University has been a participant on several state DOT research projects, including Mn/DOT, as well as various FHWA research efforts. In 2009 their researchers provided a review of the basic operating principles of LWDs from different manufacturers. Correlations between the LWD elastic modulus (E_{LWD}) and moduli determined from other testing devices were compiled. Comparison test measurements for LWDs with different plate diameters, plate contact stresses, buffer stiffnesses and measurement techniques, and correlations with static PLT measurements were reported (Vennapusa and White, 2009; Vennapusa et al., 2012; White et al., 2007).

A.4.3 Colorado School of Mines

The Colorado School of Mines also has conducted research on both the state and federal levels. Research conducted in 2010 presented results from LWD testing with radial sensors that measure the deflection bowl on one- and two-layer test beds. The LWD with radial sensors demonstrated the ability to accurately back-calculate layered moduli. The back-calculated moduli closely matched laboratory-determined moduli from triaxial testing. The measurement depth for the LWD with radial sensors was found to be 1.8 times plate diameter compared to a measurement depth of 1.0 to 1.5 times plate diameter for conventional LWDs. The LWD with radial sensors was able to measure deeper than conventional LWD testing because the radial geophones measure vertical surface deflections caused almost entirely by deeper material. Compared to other configurations, the 12- and 24-in. radial sensor configuration is recommended because it produced the most accurate moduli back-calculation results and captured the deflections critical to the back-calculation process (Senseney and Mooney, 2010; Mooney et al., 2008; Stamp and Mooney, 2008).

A.5 Industry-Sponsored Research

Although the vast majority of research was agency funded, a privately funded study is noted. In 2005 the Gas Research Institute examined nuclear density testing along with several alternative methods for soil compaction control. The soil compaction supervisor, GeoGauge, CIST, and DCP were evaluated. The DCP and the CIST had the highest overall performance. The DCP yielded a better correlation with relative density in silty-clay soils than in sandy and stone-base soils. Correlations between the CIST Impact Value (IV) and soil relative compaction were better in silty-clay than in sand and stone. The GeoGauge was sensitive to seating above the soil and had poor correlations in sand and stone materials. Moisture probes designed to determine in-place moisture were evaluated based on comparisons to laboratory obtained optimum moisture content. None of the moisture probes received a satisfactory recommendation with comments ranging from not extremely accurate to not particularly durable (Farrag et al., 2005).

A.6 Seismic Methods

The Spectral Analysis of Surface Waves (SASW) test method can provide a stiffness profile from a ground surface test site. SASW test results have been compared to other tests including the cone penetrometer test (CPT), dilatometer test (DMT), ground-penetrating radar (GPR), pressuremeter, dynamic and static plate load tests (PLT), resilient modulus (M_R), Dynaflect, and falling weight deflectometer (FWD). The overall results showed the SASW method as a versatile, accurate, and reliable test and its use should be continued to improve procedures and explore new applications (Horhota, 1996).

Seismic testing both in the laboratory and in the field is rapid and quite repeatable. For the base and subgrade, there is good agreement between the seismic moduli measured in the field and that measured in the laboratory as long as the laboratory specimens are prepared at the density and moisture content of the field materials. Seismic moduli are sensitive to variations in moisture content and dry density. Moduli measured with seismic methods are higher than those obtained from other testing methods such as the M_R and FWD tests. Large variability in the base and subgrade moduli was observed and found to be related to the test location in most cases (Nazarian et al., 1993; Nazarian et al., 1999; Mahavadi, 1998; Gucunski and Maher, 2002; Nazarian et al., 1999).

A.7 Portable Equipment Correlations

Correlation of the output from portable field testing equipment to a measured soil value, such as Young’s modulus, also has been studied by others. Previous researchers have compiled numerous correlation equations. Vennapusa and White (2009) listed 34 equations by others attempting to correlate various LWDs to PLT and FWD values. They added six additional correlation equations to the list. Danish researchers have derived LWD correlation equations based on the surface modulus, E_o (Hejlesen and Baltzer, 2008). Moody et al. (2010) have done extensive work with the Dynatest LWD software, LWDMod, where moduli were back-calculated from measured deflections and compared to laboratory triaxial moduli. Correlations between the DCP Penetration Index (DPI) and the California Bearing Ratio (CBR) have been derived (Gill, 2010).

A.8 Athletic Field Hardness Testing with the CIST

The CIST is also used to determine the hardness on all types of athletic surfaces, including baseball, football, soccer, and horse racing, and composed of natural grass, artificial turf, and infill surfaces. The condition of the playing surface is important for both turf care and playability (Turf-Tec, 2014).

Field density—or how hard the ground is—plays a sizable role in head injuries. A properly maintained playing surface can help reduce head injury risk. The National Football League (NFL) now requires field managers to measure surface hardness before every game. The NFL field testing program requires that the surface hardness of both natural and synthetic turf fields be measured with the CIST. Fields must be tested in multiple locations prior to every game and must be below 100 Gmax (Bradley, 2014).

Appendix B: Phase I Test Pit Results

B.1 Briaud Compaction Device (BCD)

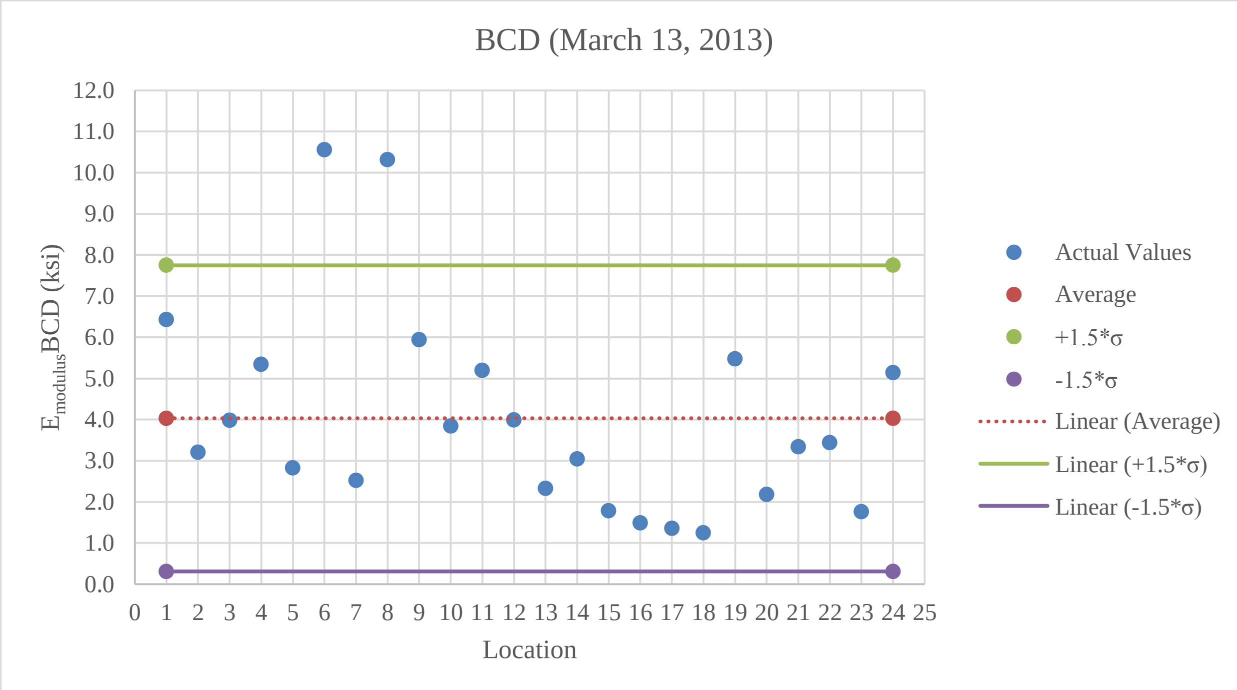

Figure B.1: BCD testing on A-3 at optimum moisture content
(In kilopounds per square inch, *ksi*)

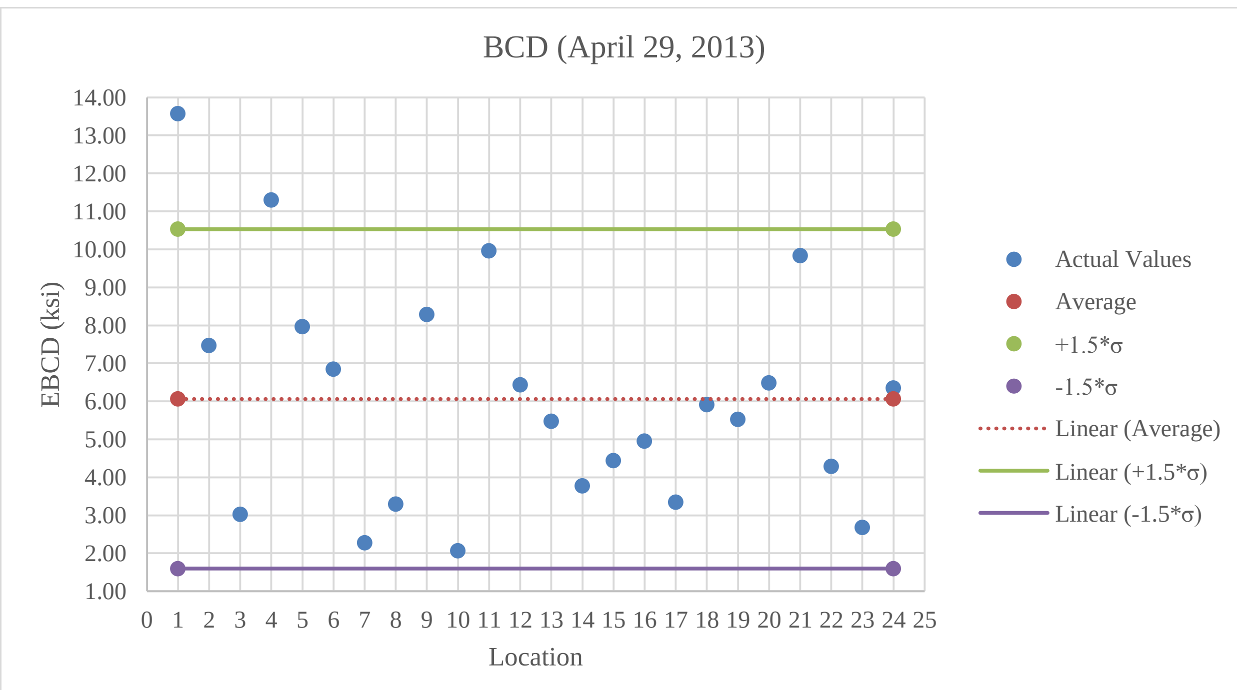

Figure B.2: BCD testing on A-2-4 at optimum moisture content

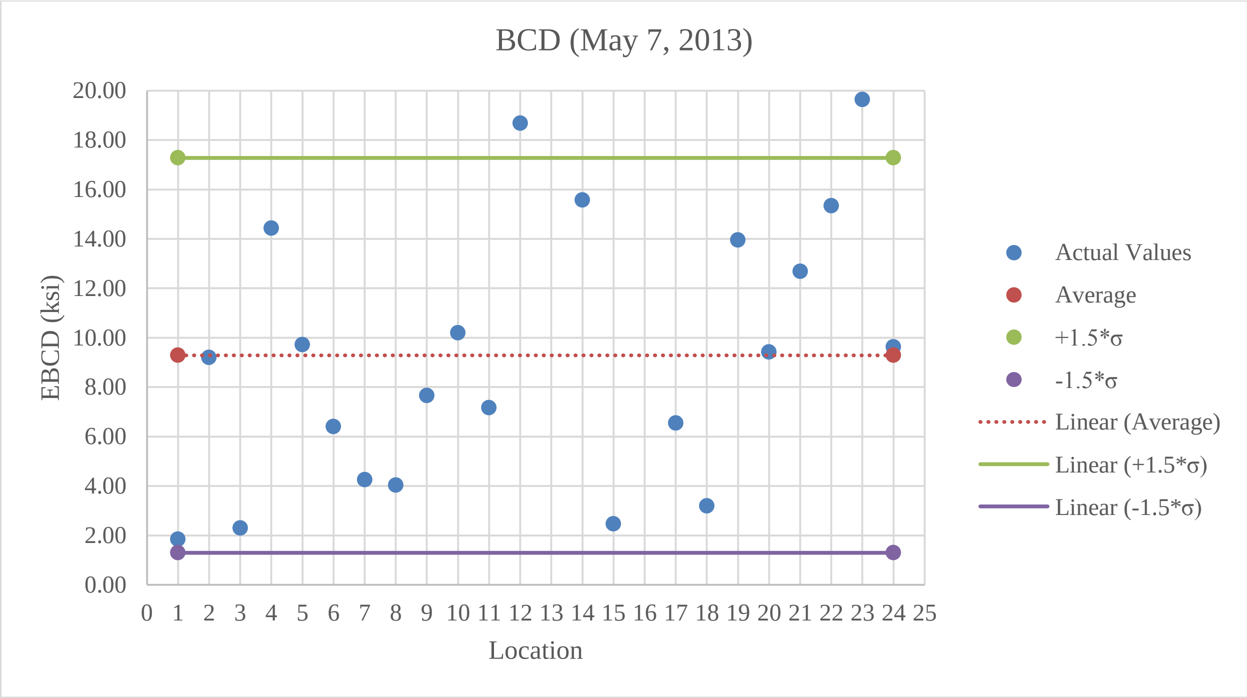

Figure B.3: BCD testing on A-2-4 below optimum moisture content

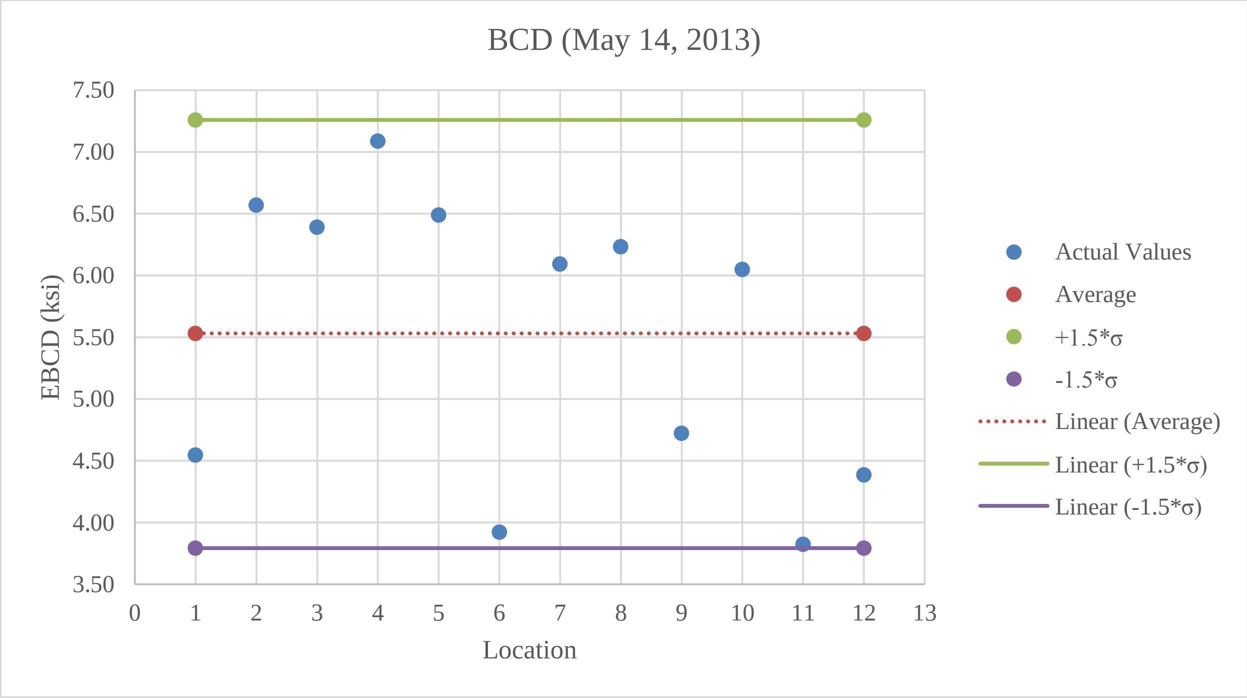

Figure B.4: BCD testing on stabilized subgrade at optimum moisture content

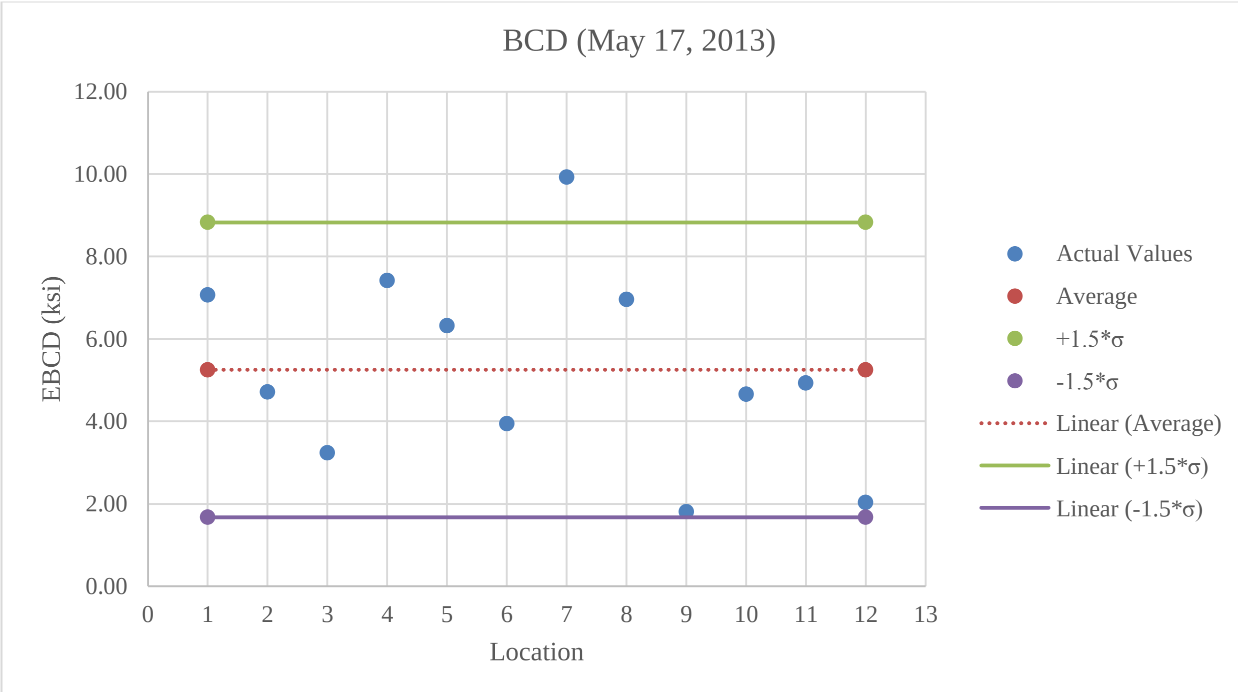

Figure B.5: BCD testing on stabilized subgrade below optimum moisture content

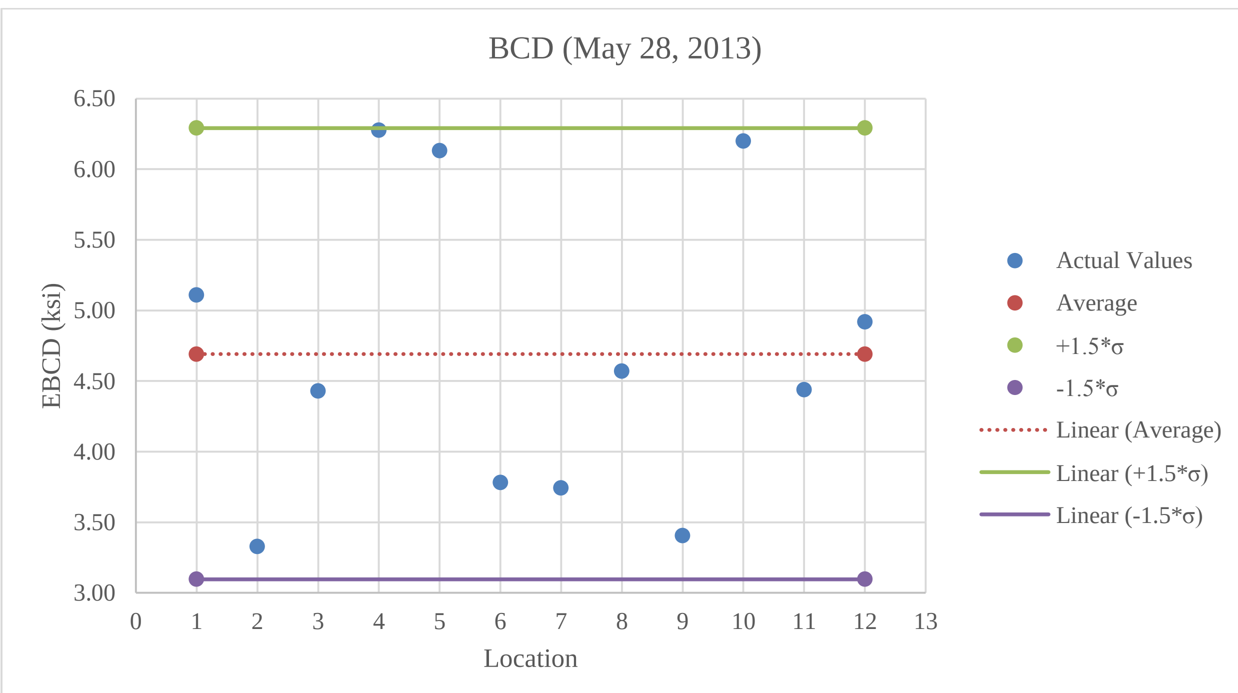

Figure B.6: BCD testing on stabilized subgrade above optimum moisture content

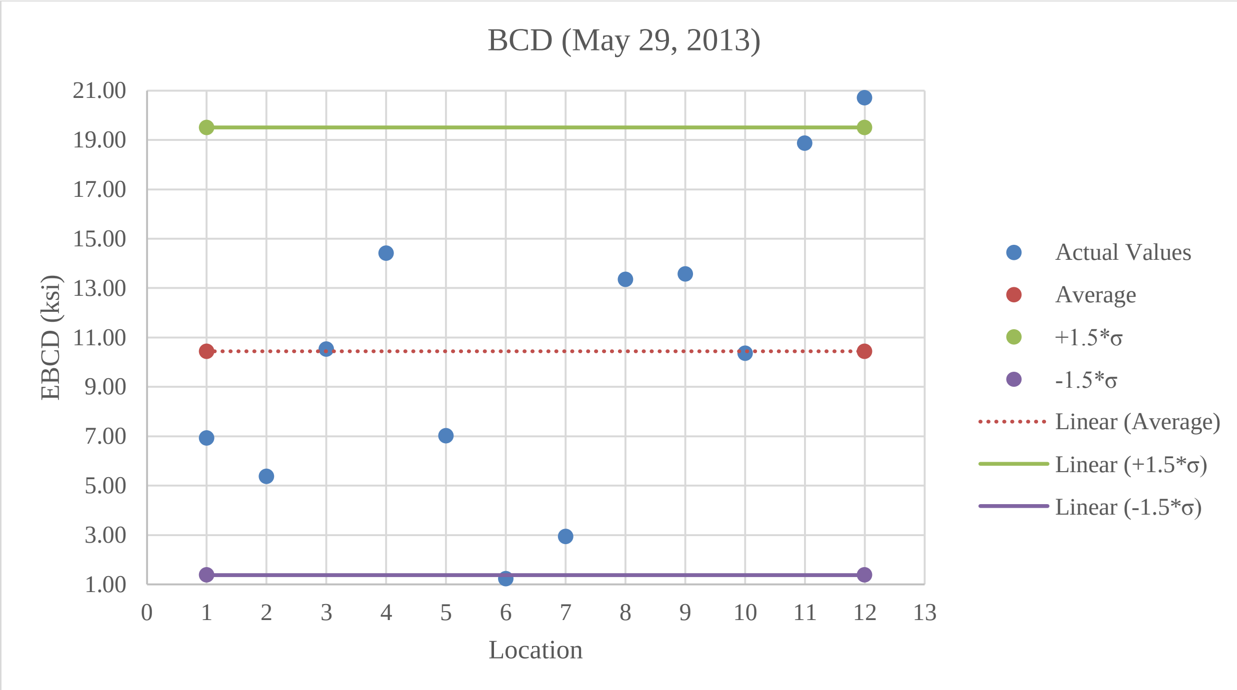

Figure B.7: BCD testing on A-2-4 below optimum moisture content

B.2 Dirt Seismic Pavement Analyzer (DSPA)

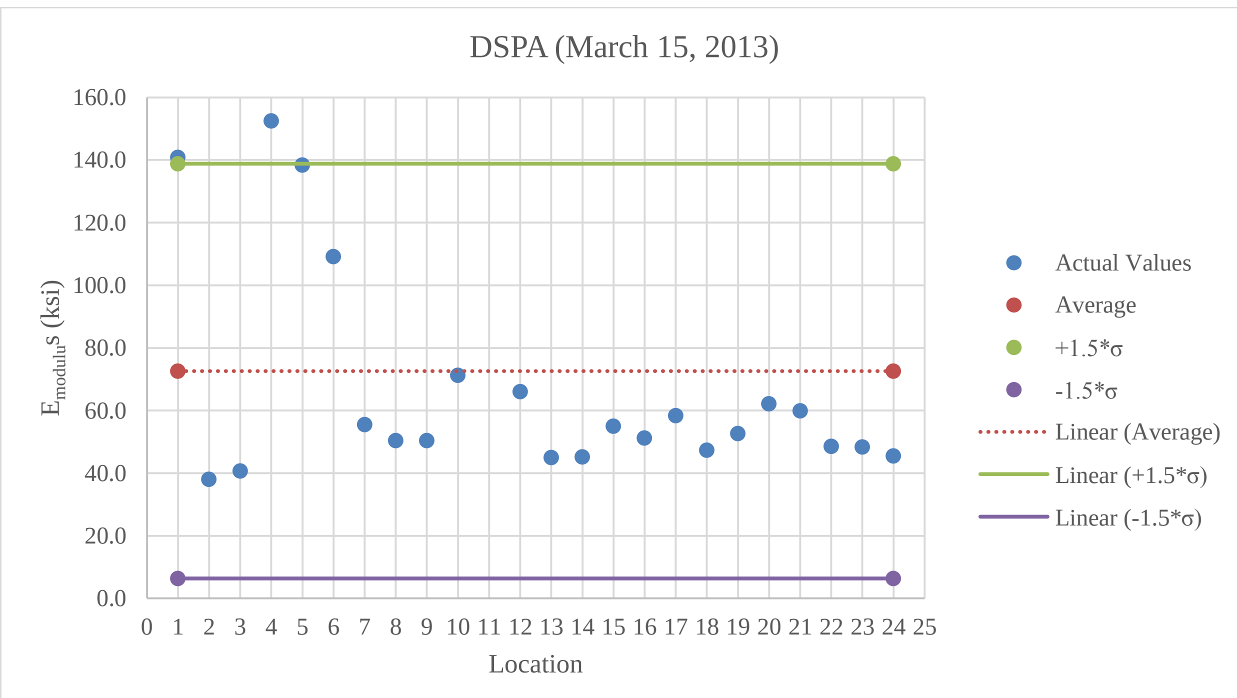

Figure B.8: DSPA testing on A-3 at optimum moisture content

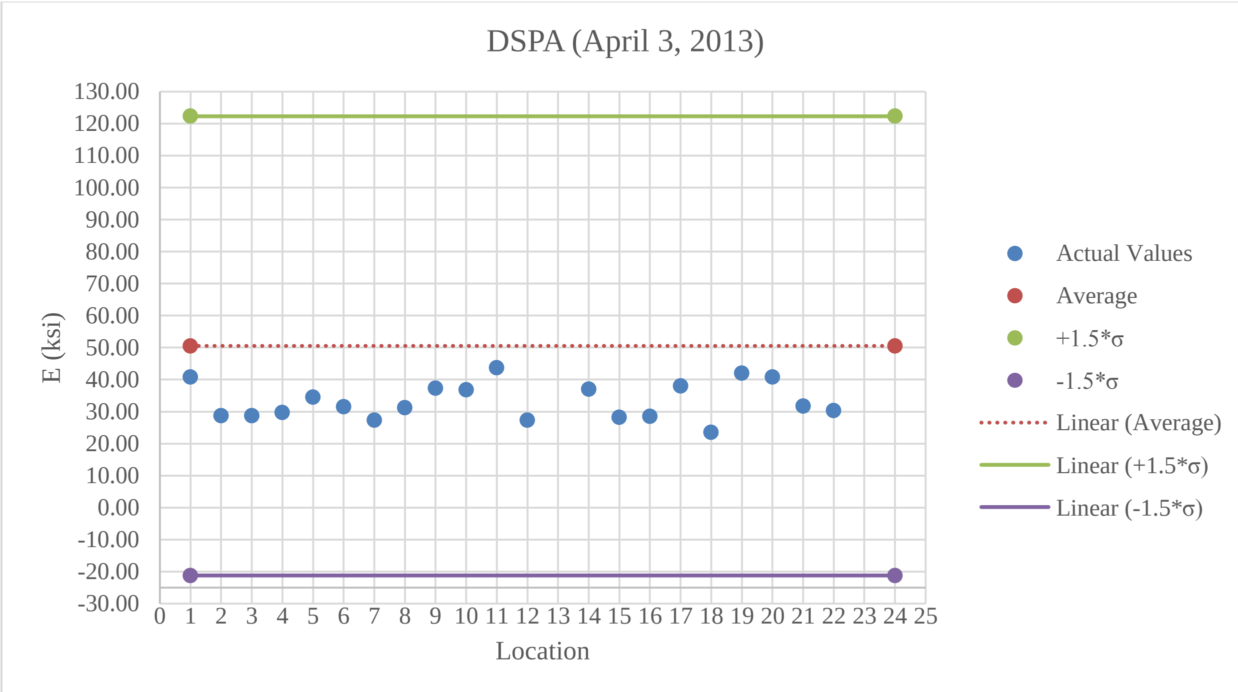

Figure B.9: DSPA testing on A-3 below optimum moisture content

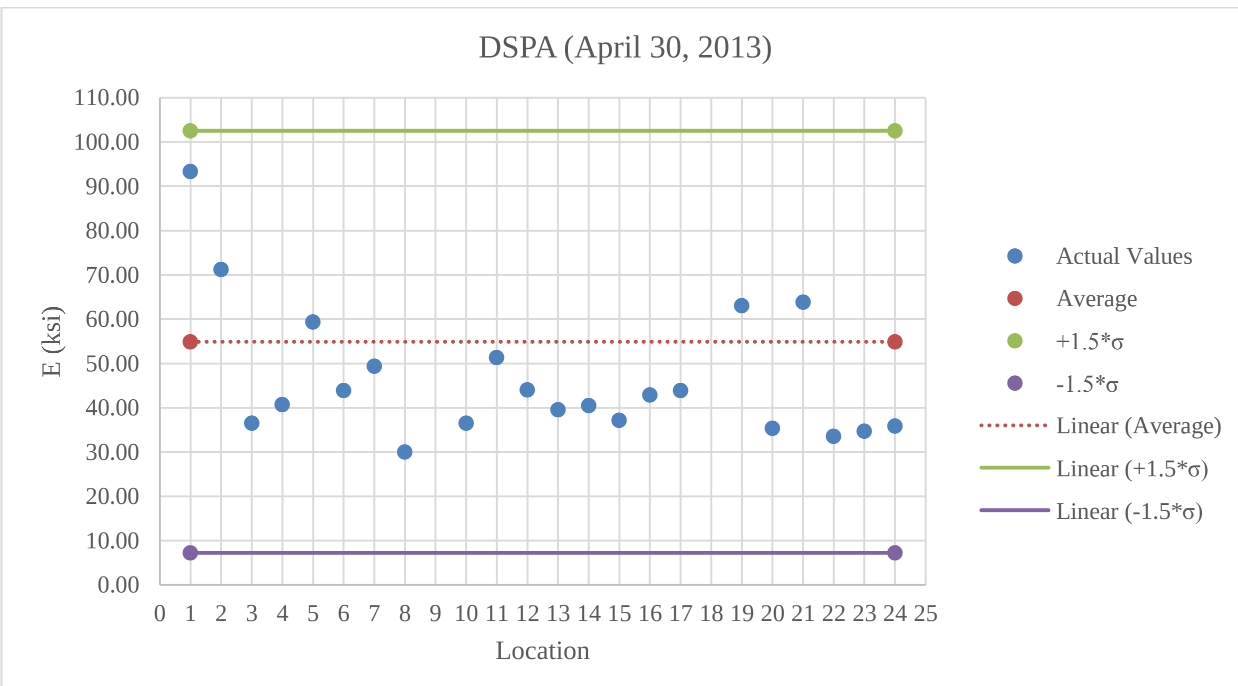

Figure B.10: DSPA testing on A-2-4 at optimum moisture content

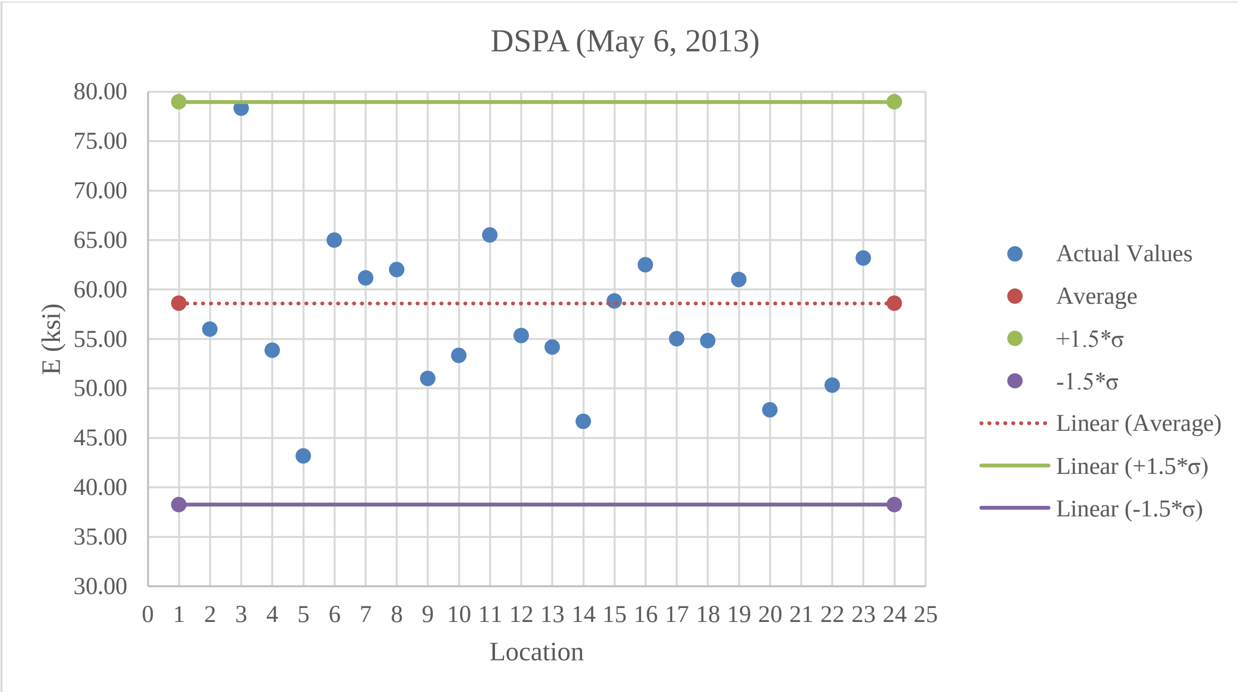

Figure B.11: DSPA testing on A-2-4 below optimum moisture content

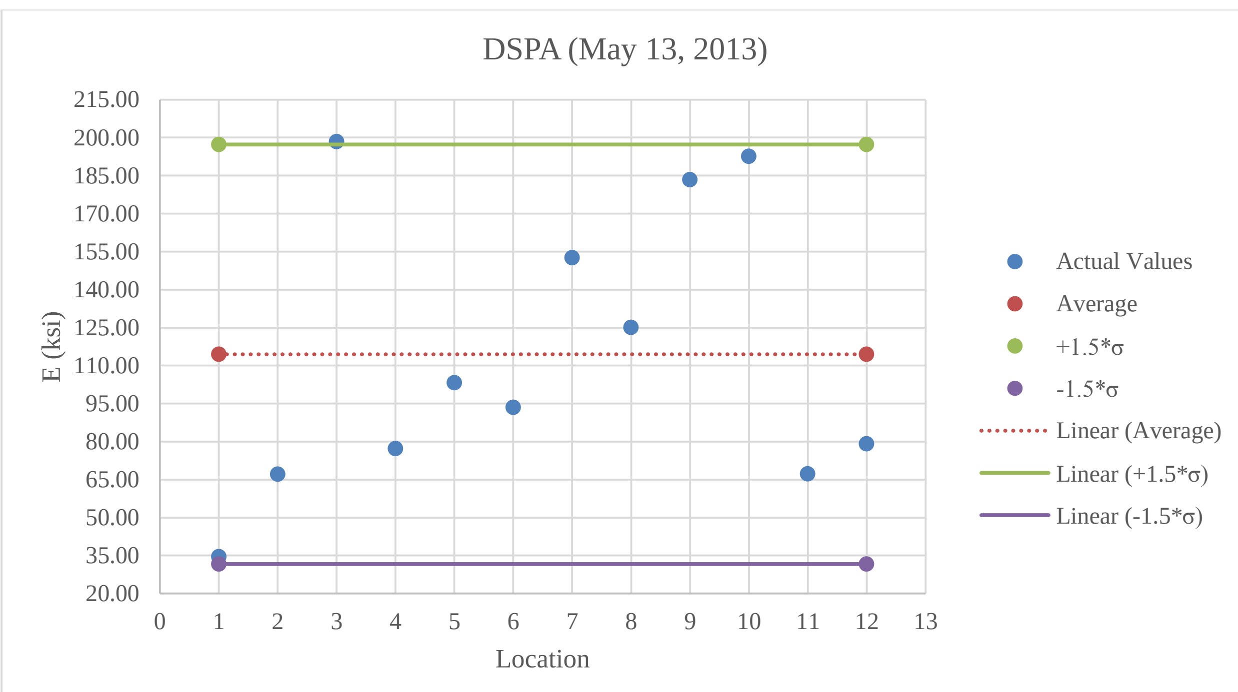

Figure B.12: DSPA testing on stabilized subgrade at optimum moisture content

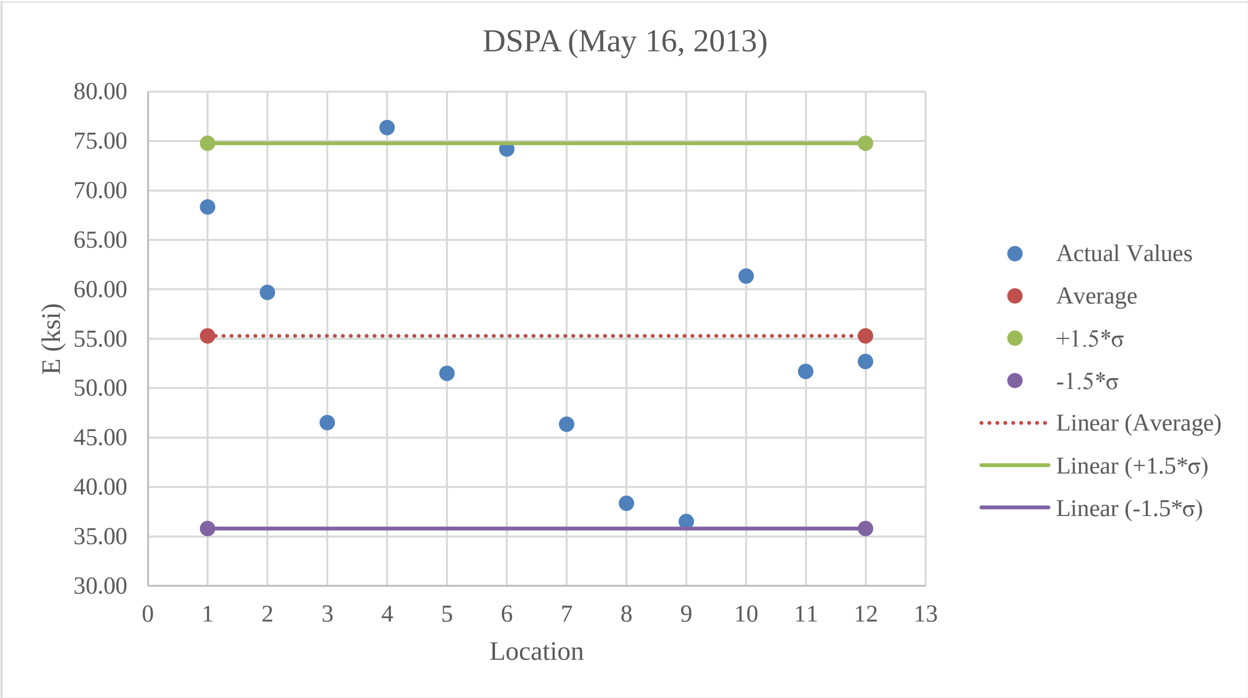

Figure B.13: DSPA testing on stabilized subgrade below optimum moisture content

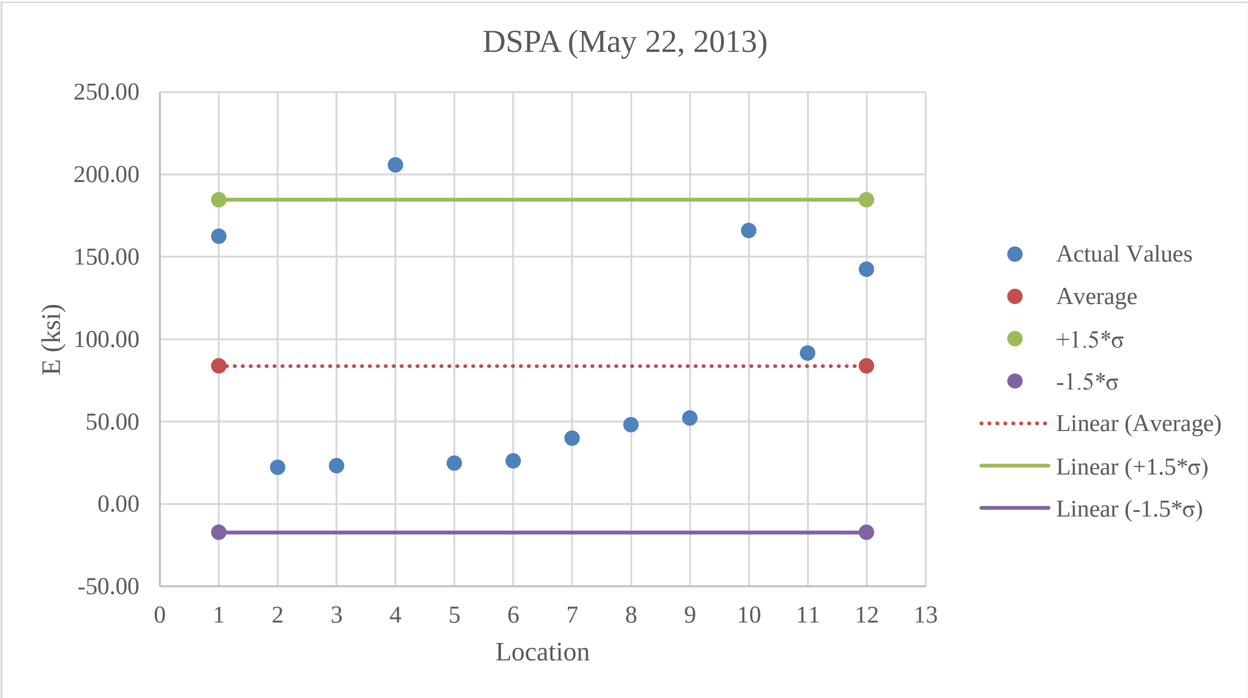

Figure B.14: DSPA testing on stabilized subgrade above optimum moisture content

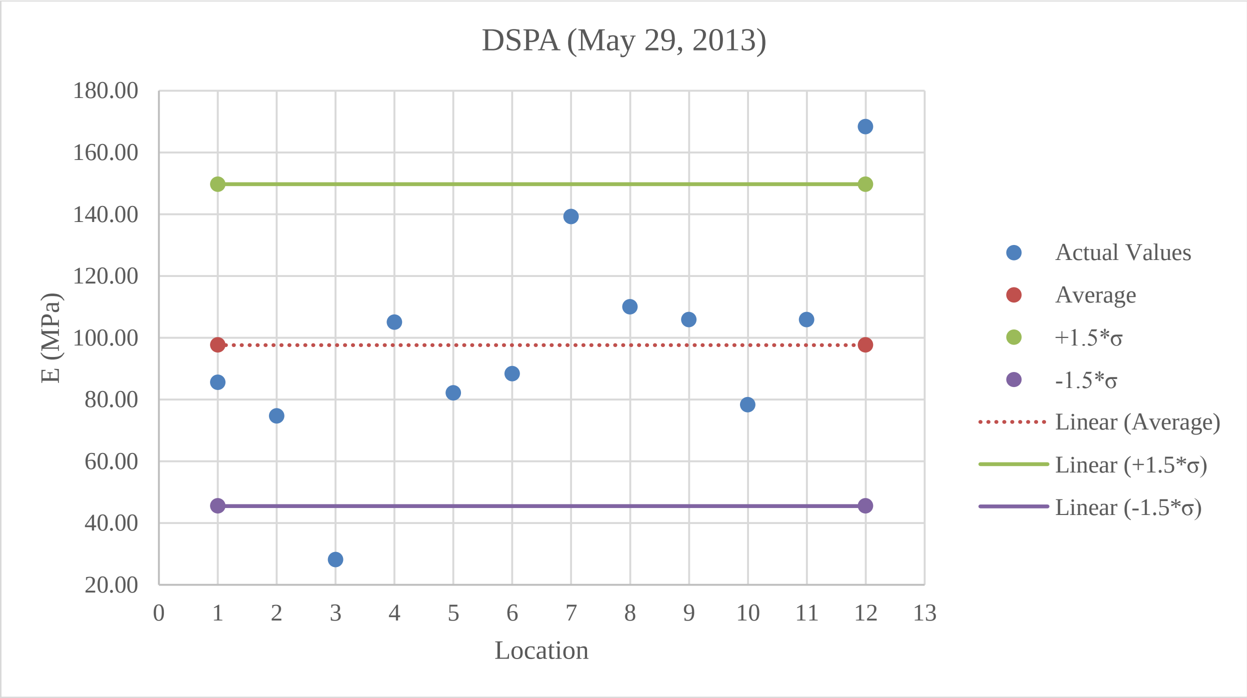

Figure B.15: DSPA testing on A-2-4 below optimum moisture content

B.3 Dynamic Cone Penetrometer (DCP)

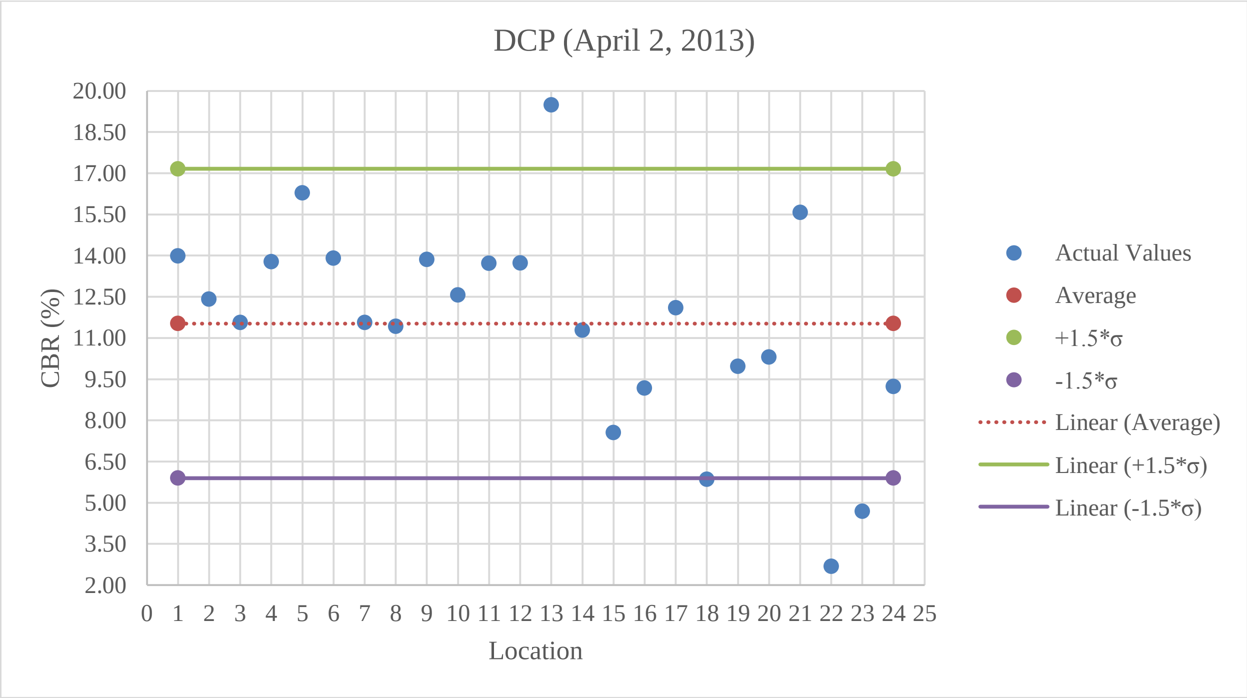

Figure B.16: DCP testing on A-3 below optimum moisture content

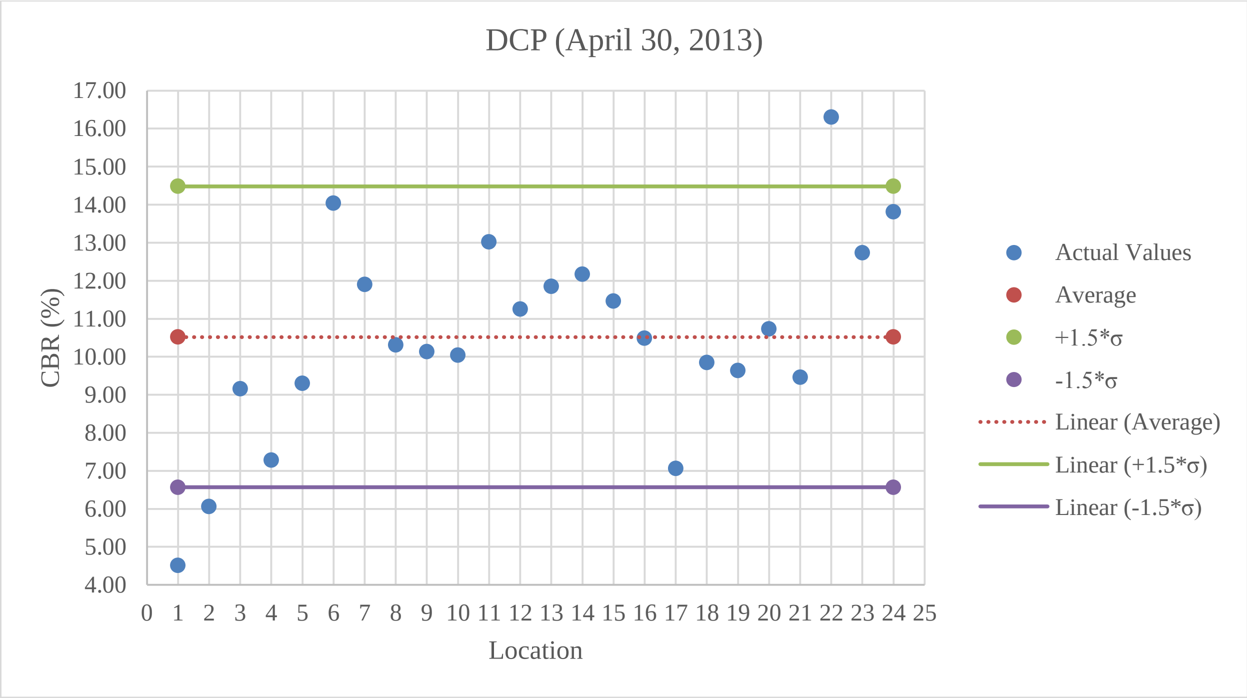

Figure B.17: DCP testing on A-2-4 at optimum moisture content

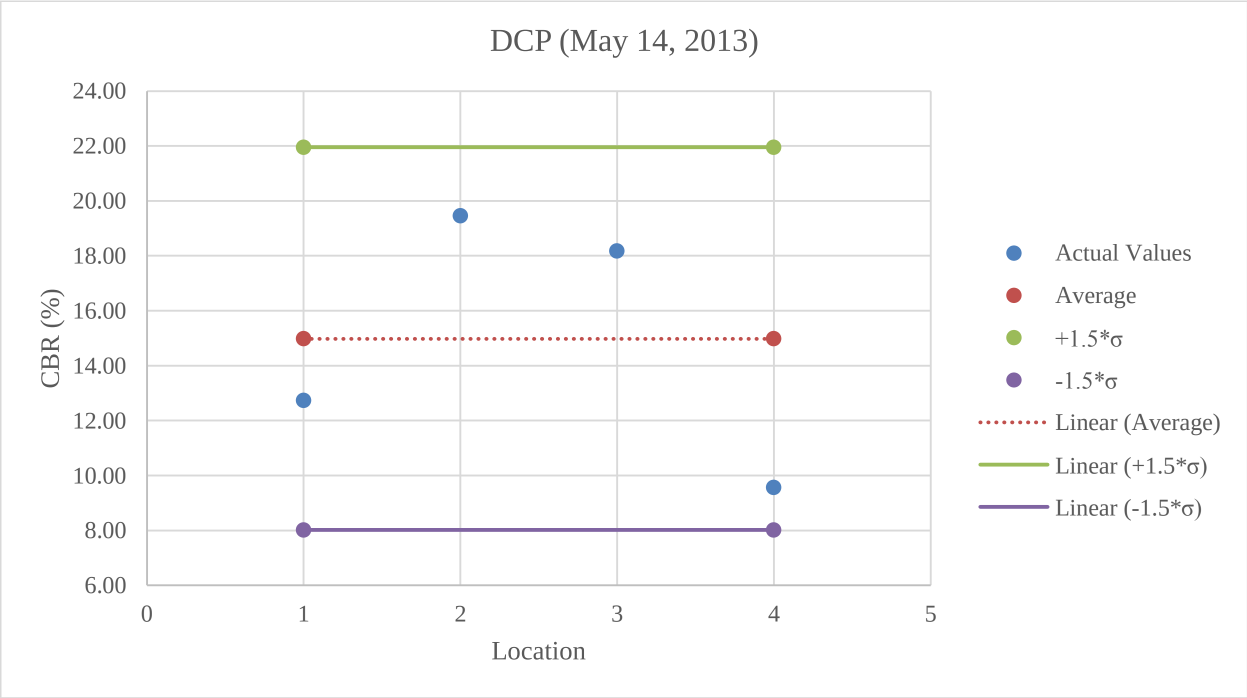

Figure B.18: DCP testing on stabilized subgrade at optimum moisture content

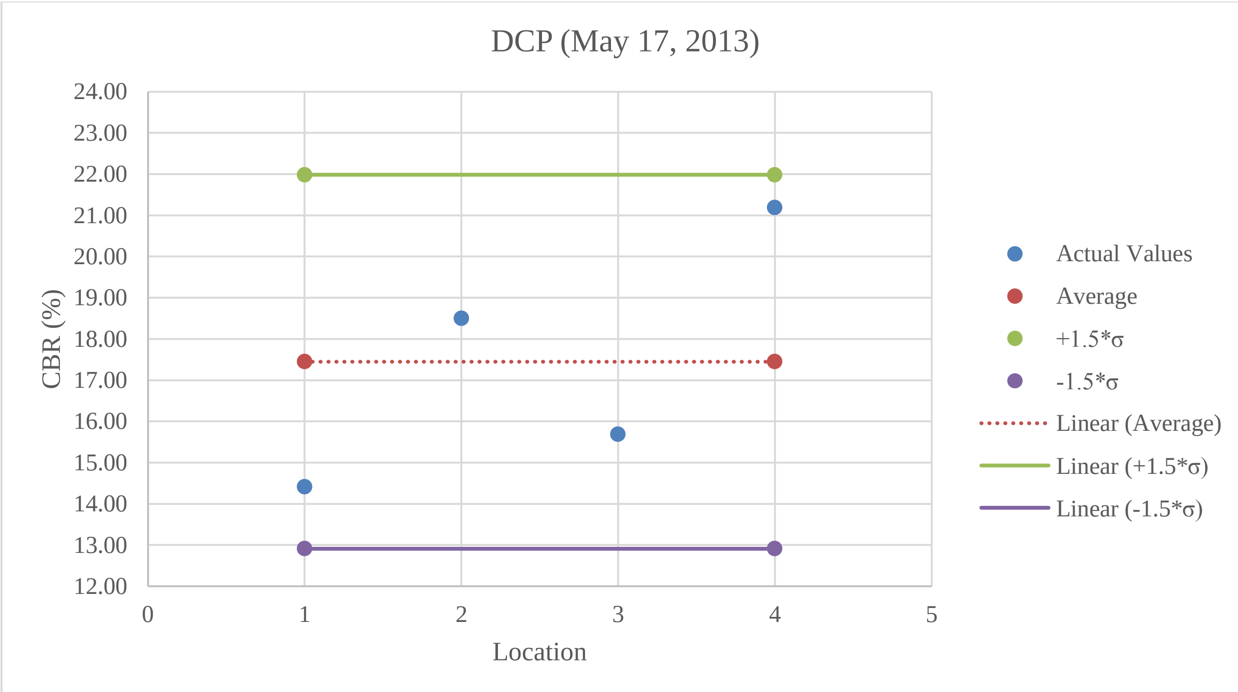

Figure B.19: DCP testing on stabilized subgrade below optimum moisture content

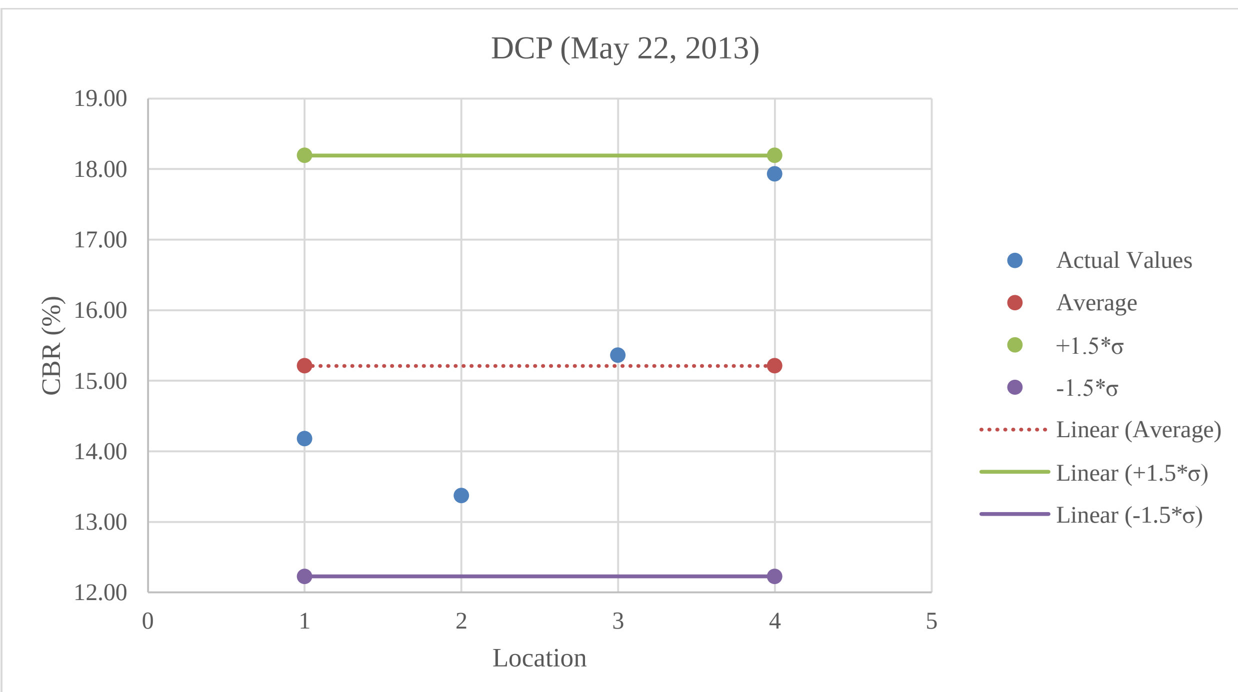

Figure B.20: DCP testing on stabilized subgrade above optimum moisture content

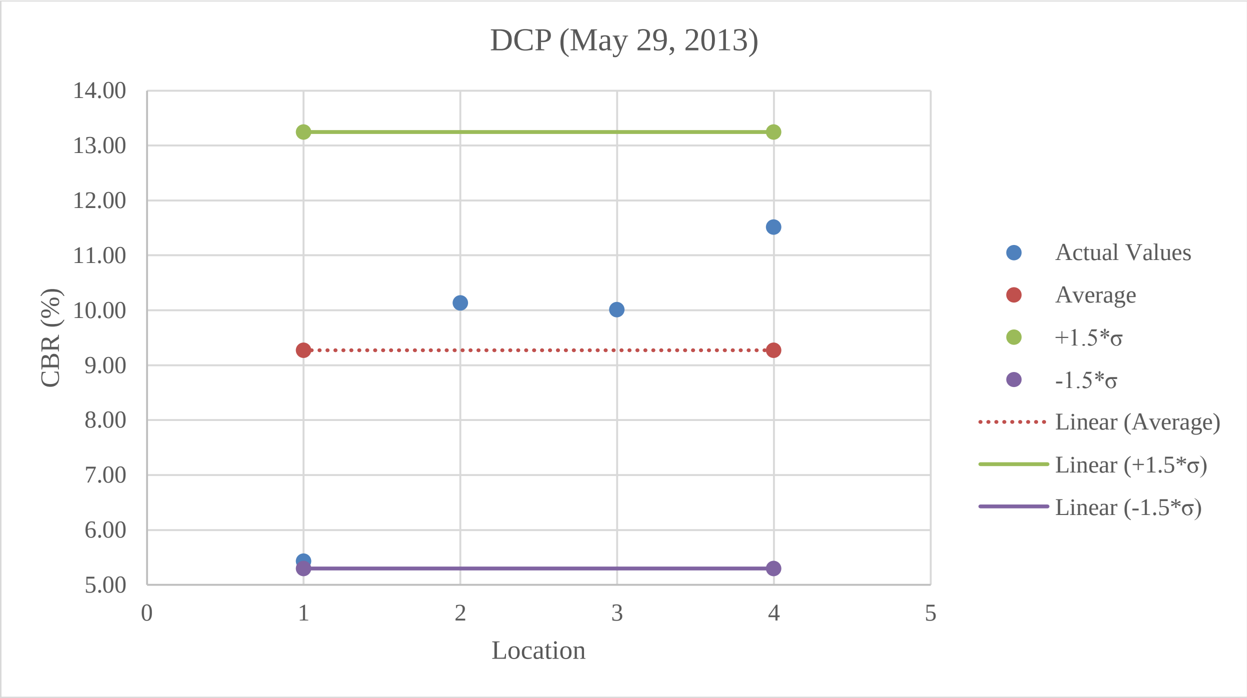

Figure B.21: DCP testing on A-2-4 below optimum moisture content

B.4 GeoGauge

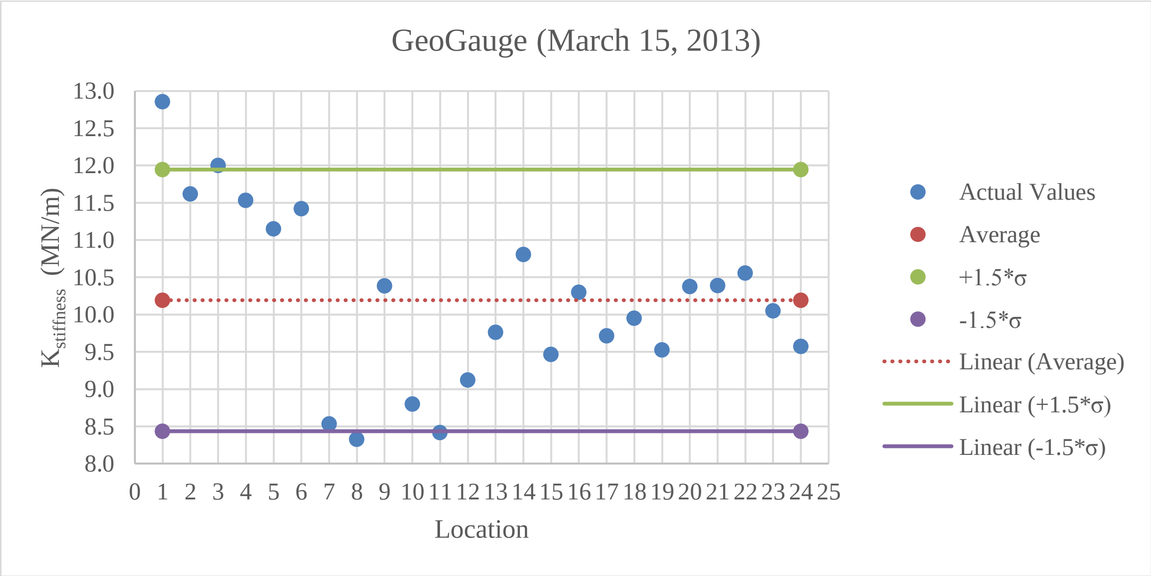

Figure B.22: GeoGauge testing on A-3 at optimum moisture content

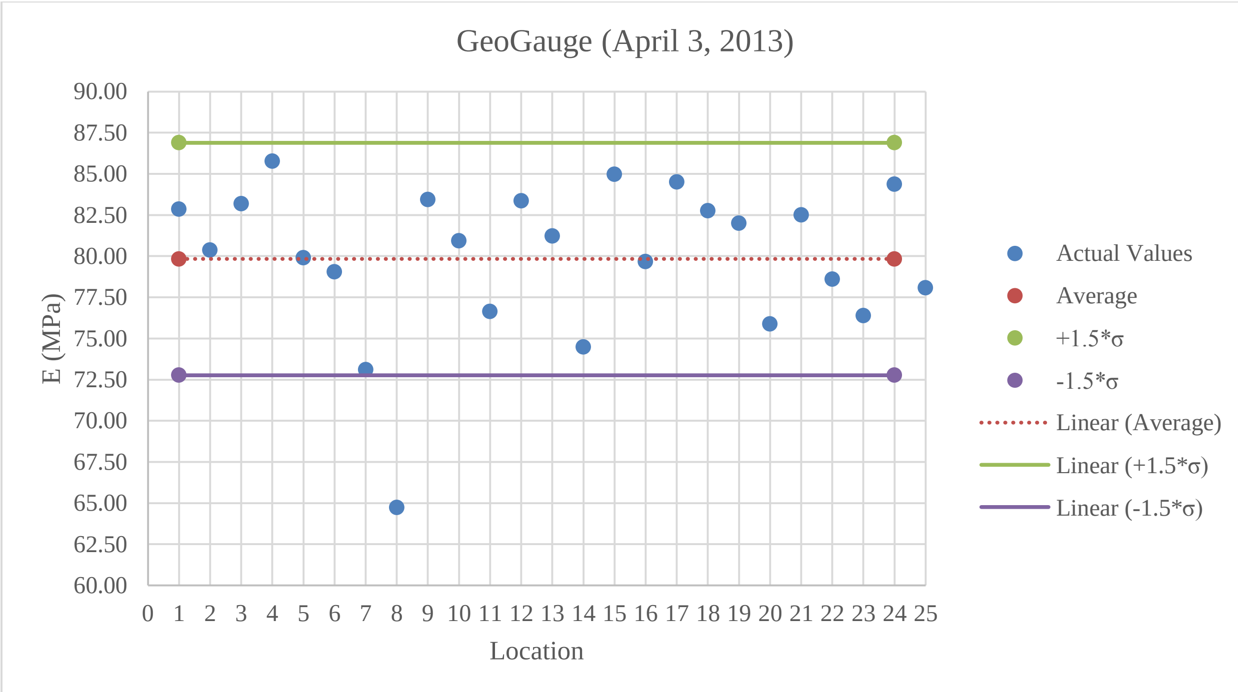

Figure B.23: GeoGauge testing on A-3 below optimum moisture content

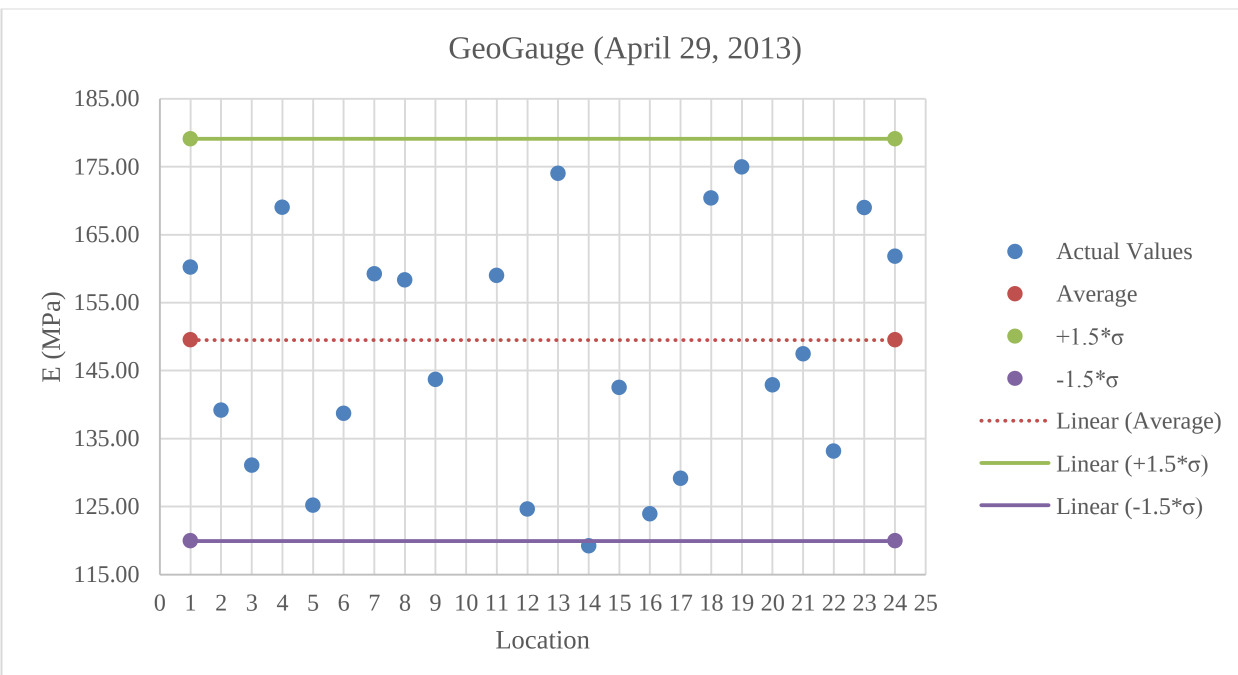

Figure B.24: GeoGauge testing on A-2-4 at optimum moisture content

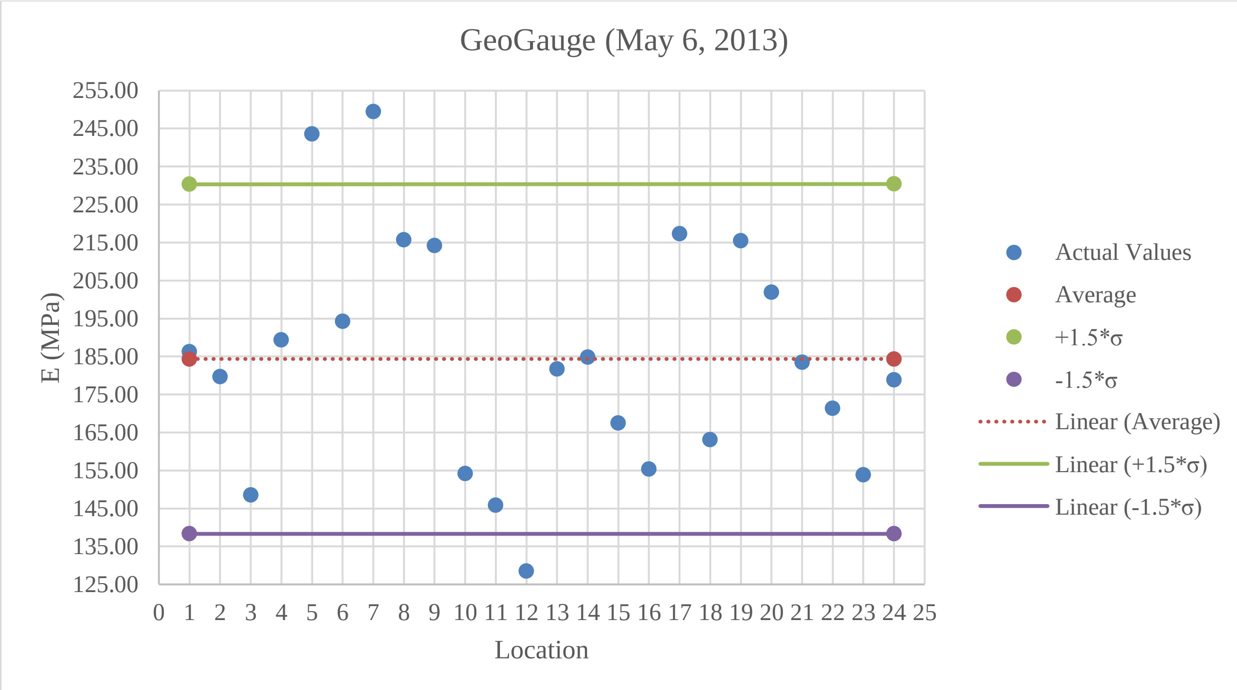

Figure B.25: GeoGauge testing on A-2-4 below optimum moisture content

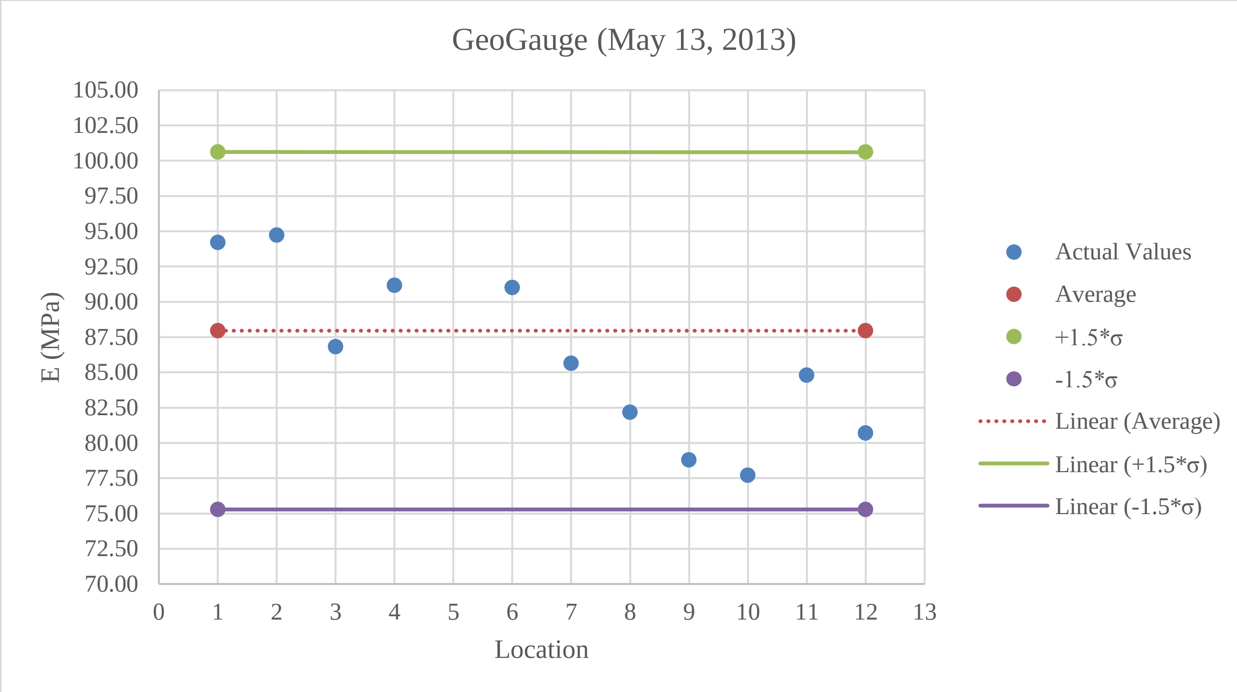

Figure B.26: GeoGauge testing on stabilized subgrade at optimum moisture content

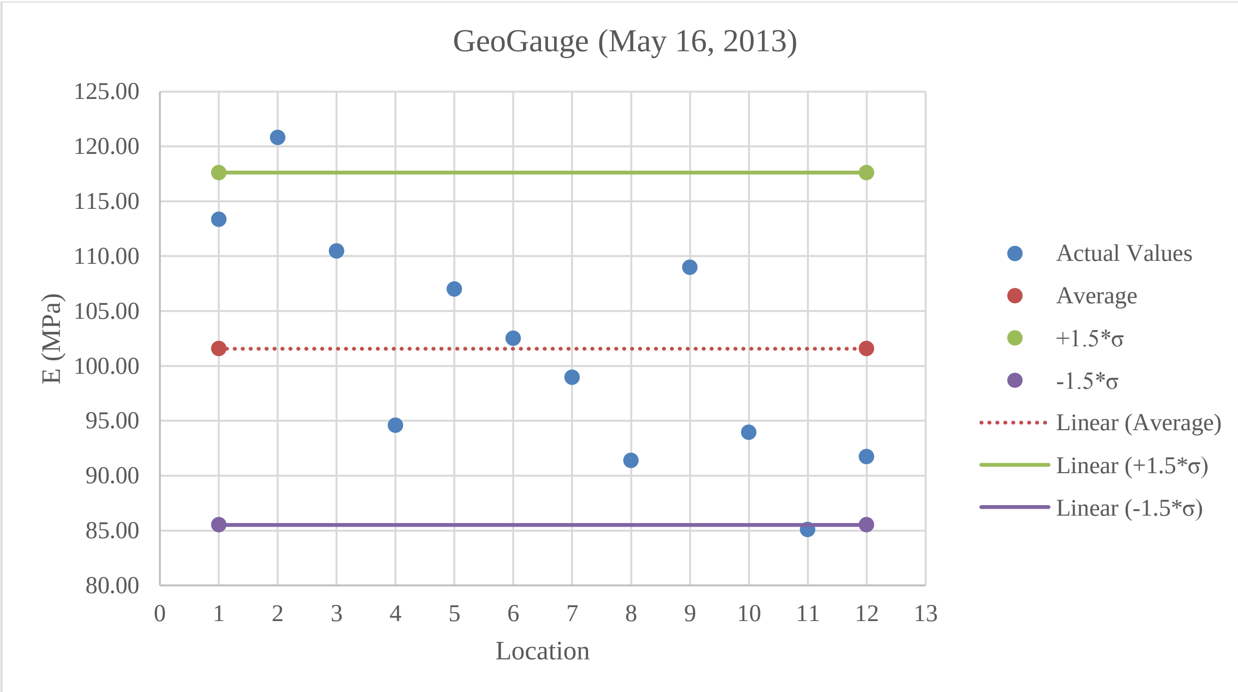

Figure B.27: GeoGauge testing on stabilized subgrade below optimum moisture content

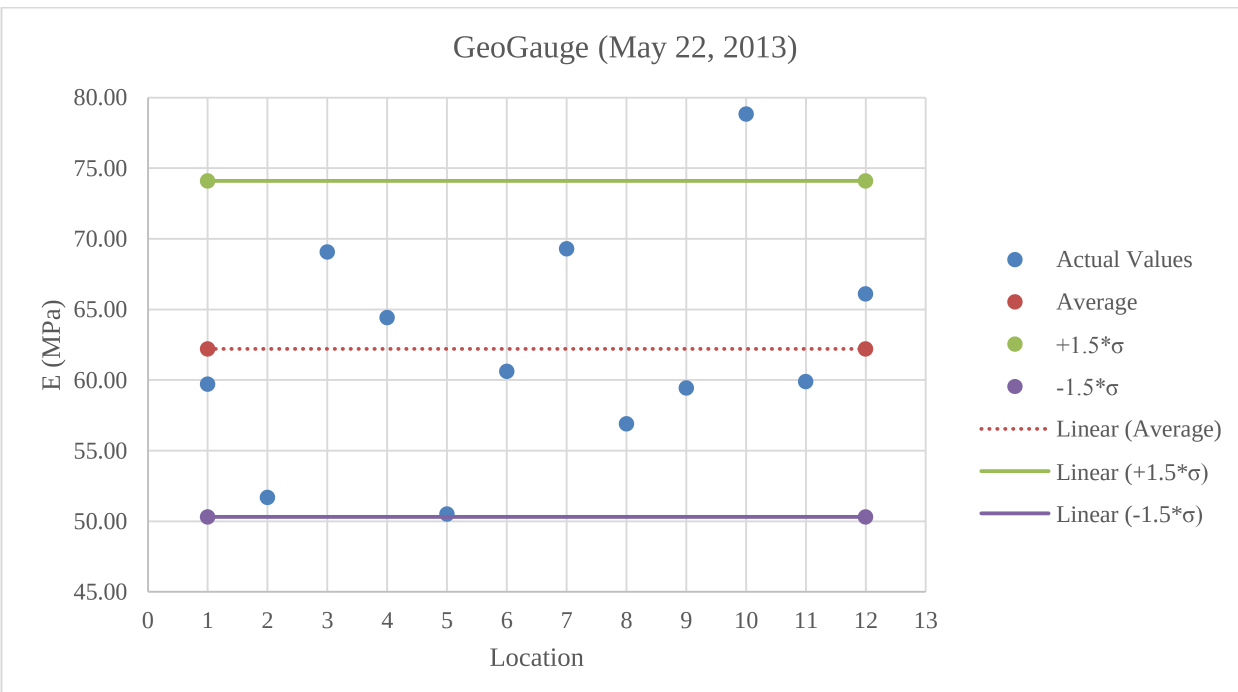

Figure B.28: GeoGauge testing on stabilized subgrade above optimum moisture content

B.5 LWD-1

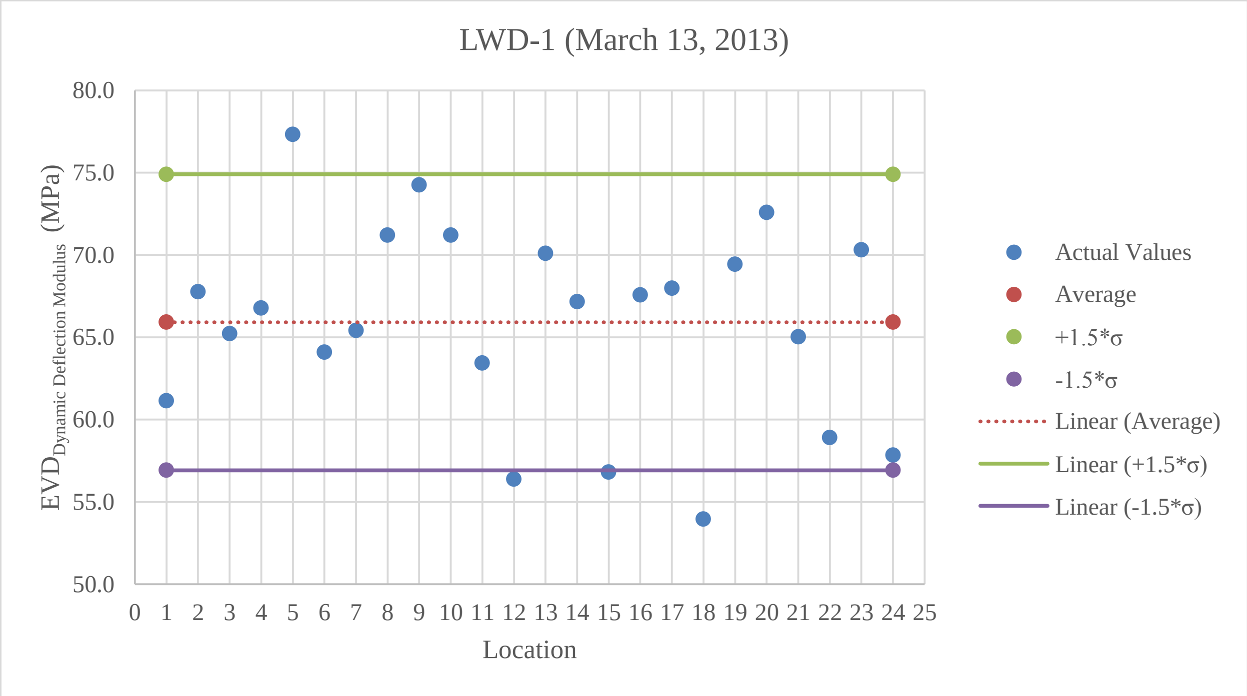

Figure B.29: LWD-1 testing on A-3 at optimum moisture content

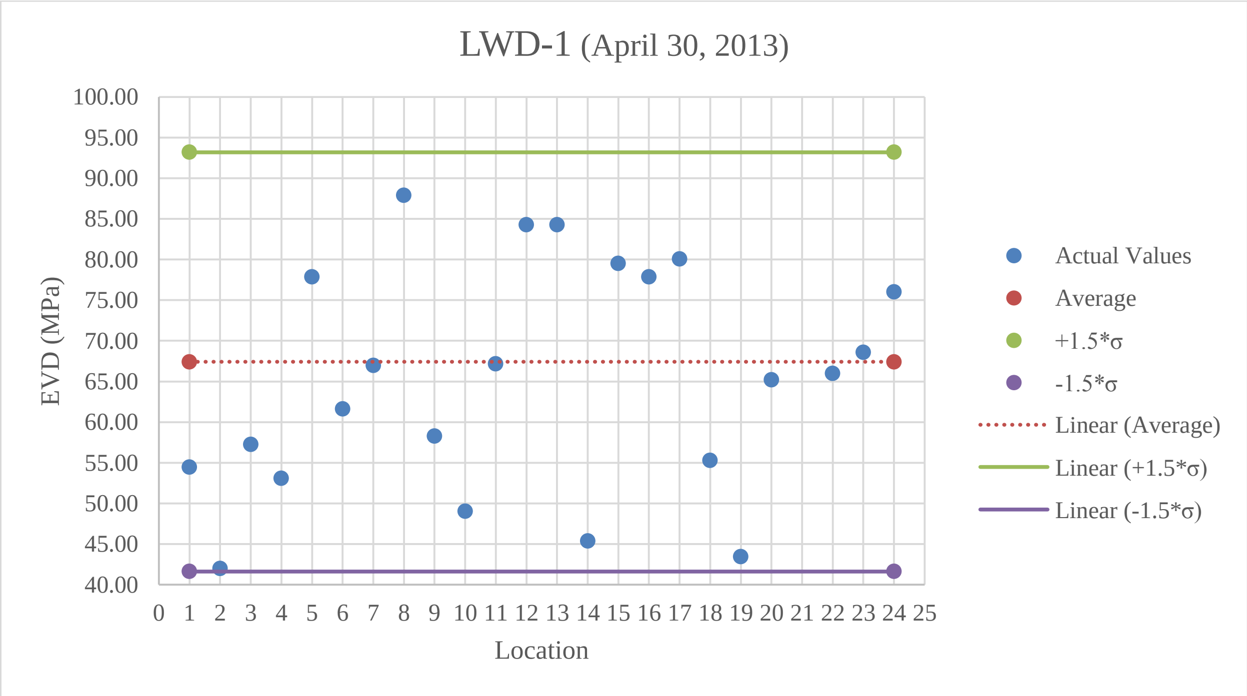

Figure B.30: LWD-1 testing on A-2-4 at optimum moisture content

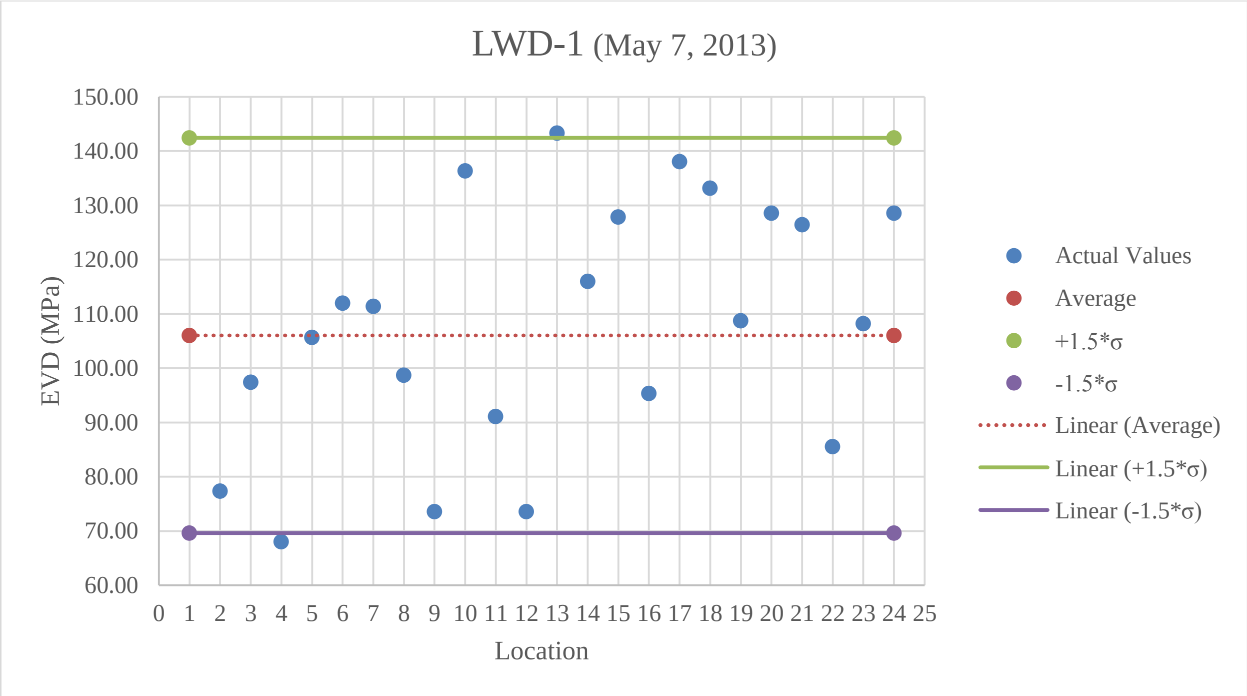

Figure B.31: LWD-1 testing on A-2-4 below optimum moisture content

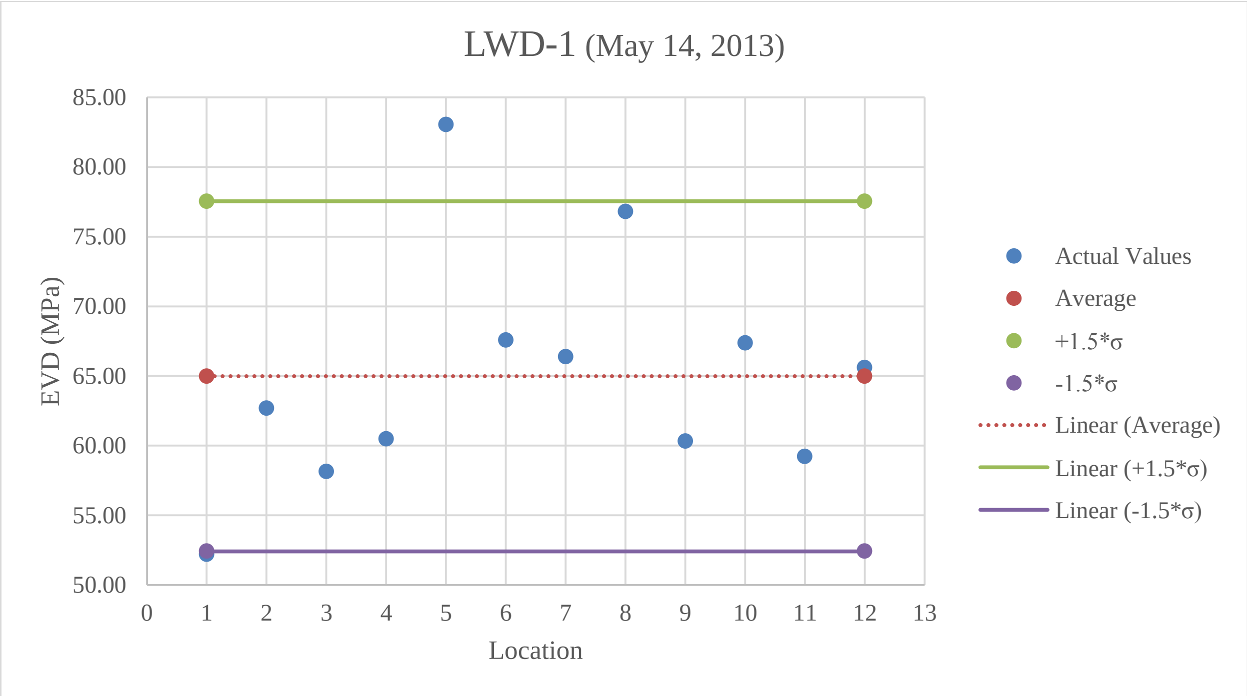

Figure B.32: LWD-1 testing on stabilized subgrade at optimum moisture content

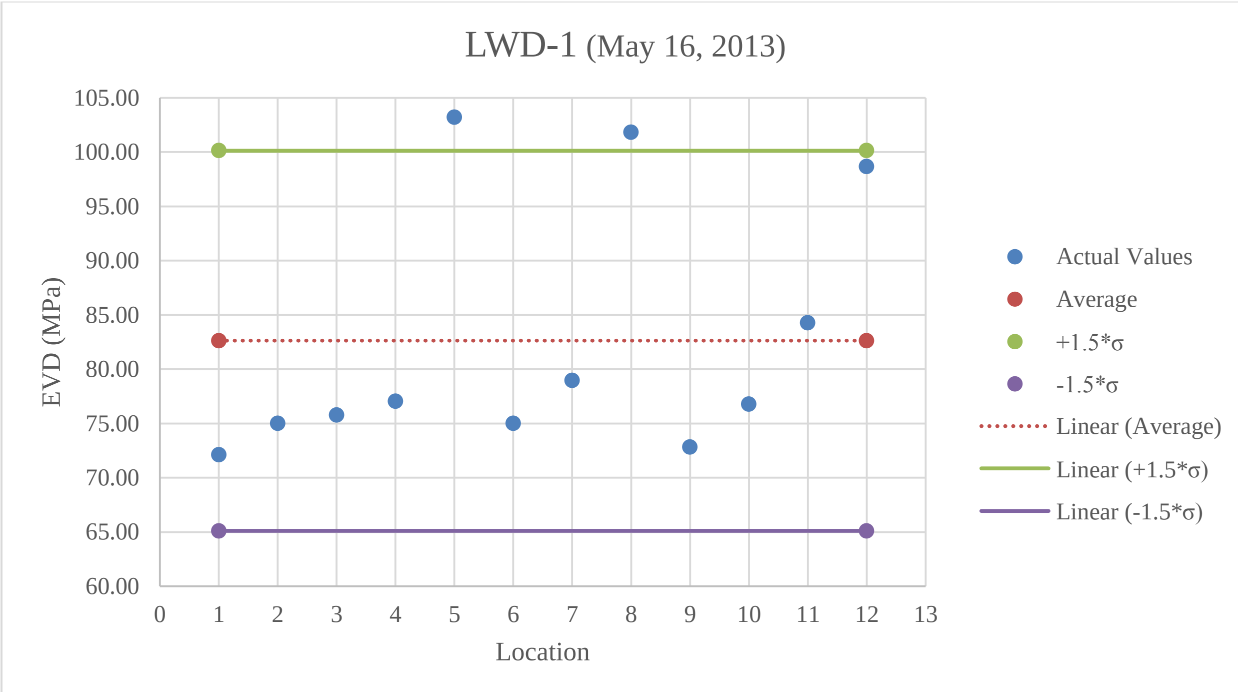

Figure B.33: LWD-1 testing on stabilized subgrade below optimum moisture content

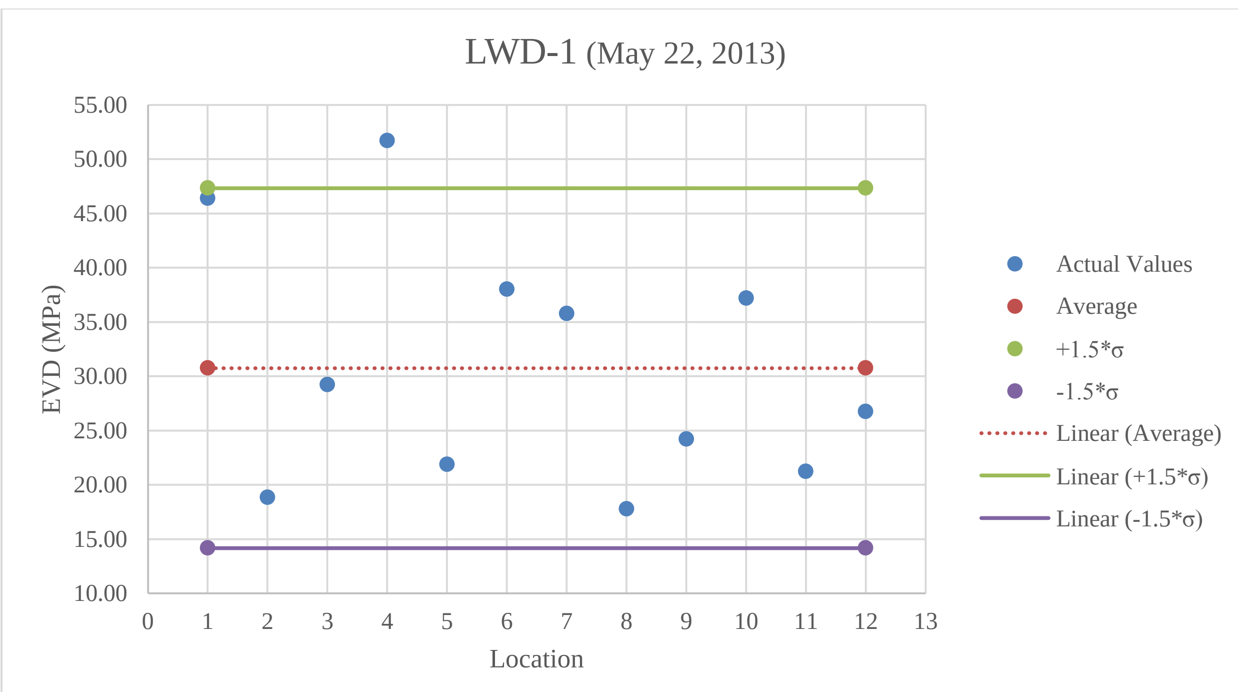

Figure B.34: LWD-1 testing on stabilized subgrade above optimum moisture content

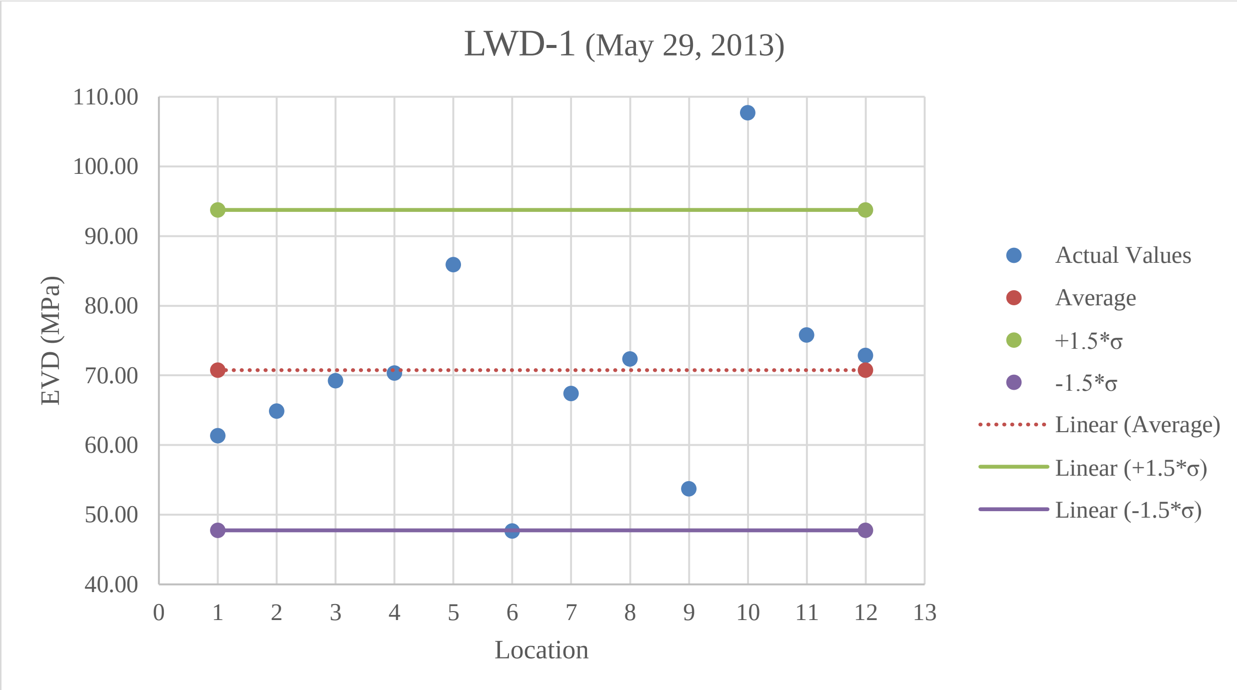

Figure B.35: LWD-1 testing on A-2-4 below optimum moisture content

B.6 LWD-2

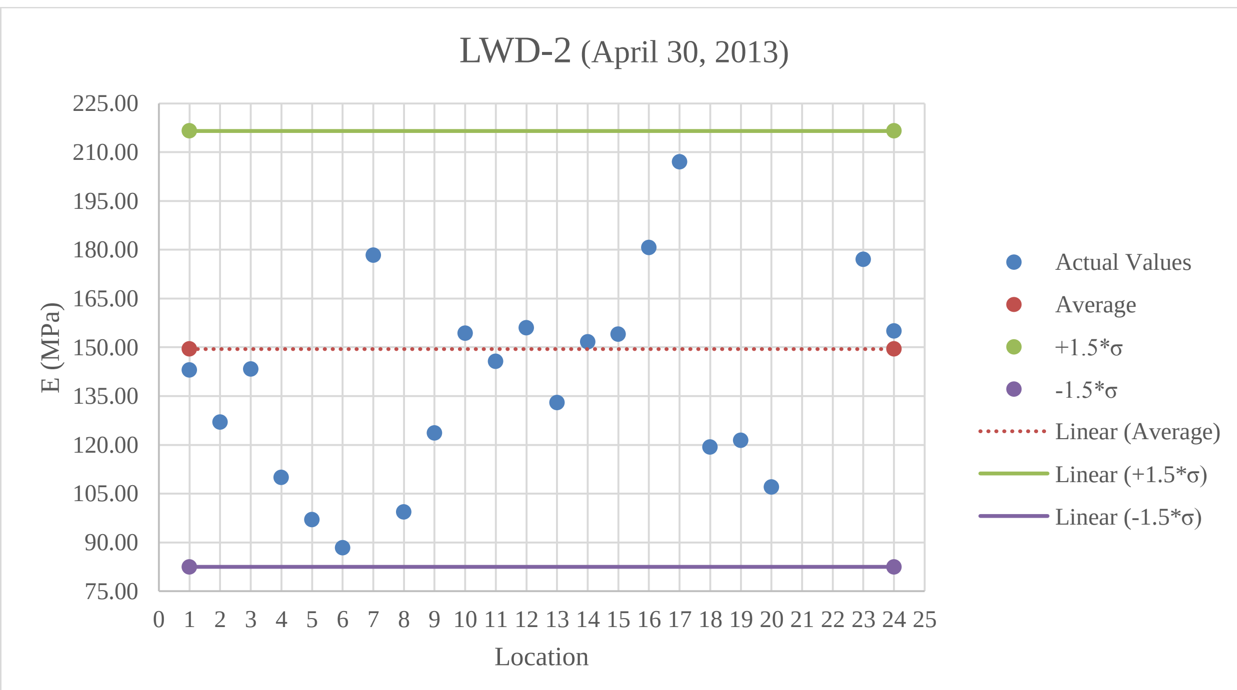

Figure B.36: LWD-2 testing on A-2-4 at optimum moisture content

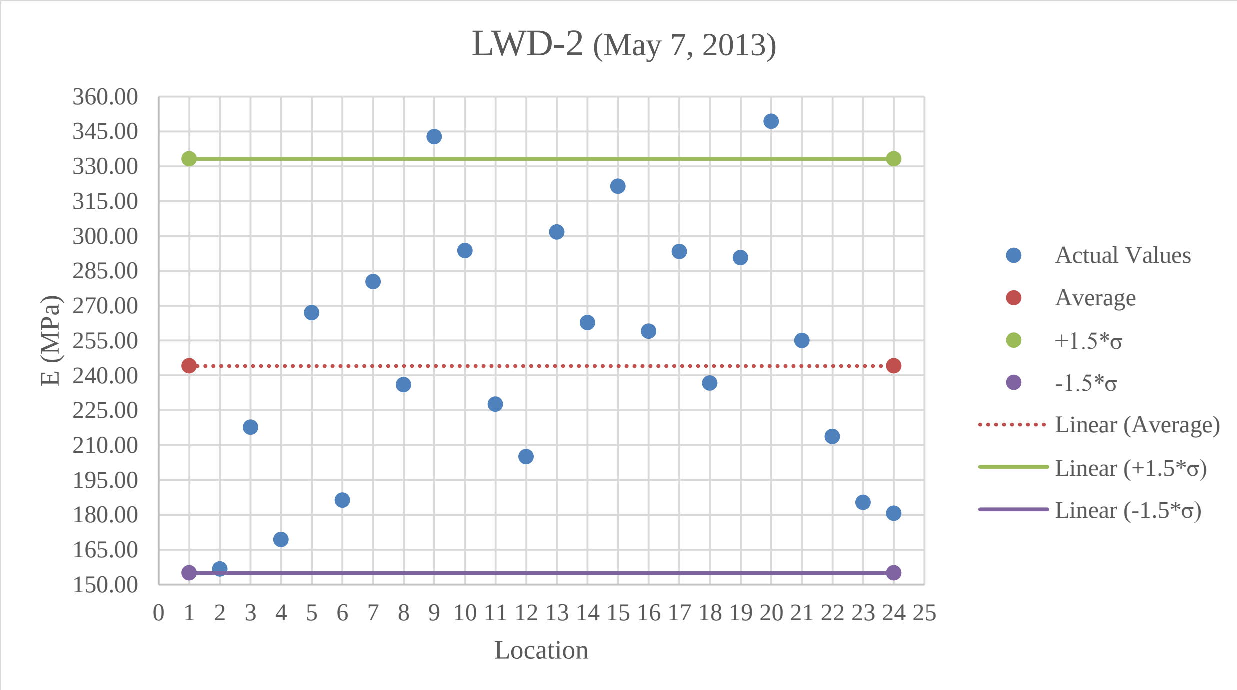

Figure B.37: LWD-2 testing on A-2-4 below optimum moisture content

B.7 Static Plate Load Tests (PLT)

Table B.1: Summary of 6-in. static PLTs in test box

PLATE LOAD TESTING	MATERIAL	MOISTURE	TEST BOX 6-IN. STATIC	TYPE	MODULUS (PSI)		MODULUS (PSI)
Date: 6.04	A-3	Optimum	Test Box	6-in.	10,415		10,148
6.06	A-2-4	Above	Test Box	6-in.	948		979
6.11	A-2-4	Optimum	Test Box	6-in.	5,502		5502
6.13	Subgrade	Optimum	Test Box	6-in.	8,451		10,582
6.20	Limerock base	Optimum	Test Box	6-in.	11,103		10,080
7.02	A-2-4 low fines	Optimum	Test Box	6-in.	10,384		10,947

Table B.2: Summary of static PLTs

PLATE LOAD TESTING		TEST PIT 12-IN. STATIC					
DATE	MATERIAL	MOISTURE	LOCATION	TYPE	SOUTH MODULUS (PSI)	MIDDLE MODULUS (PSI)	NORTH MODULUS (PSI)
3.18	A-3	Optimum	East Pit	12-in.	13,031	13,732	12,862
4.04	A-3	Optimum	East Pit	12-in.	13,579	11,643	12,425
5.02	A-2-4	Optimum	West Pit	12-in.	5,509	7,625	9,753
5.13	A-2-4	Optimum	West Pit	12-in.	11,612	11,531	13,674
5.20	Subgrade	Drained	East Pit	12-in.	17,274	---- ---	18,146
5.23	Subgrade	Optimum	East Pit	12-in.	15,821	---- ---	15,987
5.29	Subgrade	Soaked 5K	East Pit	12-in.	9,946		
5.29	Subgrade	Soaked 2K	East Pit	12-in.	13,377	13,800	
5.30	A-2-4	Drained	West Pit	12-in.	20,054	25,399	21,002

Appendix C: Phase II Single Spot Testing Sequence Results

C.1 Results for Devices of A-3 Material at Optimum Moisture Content

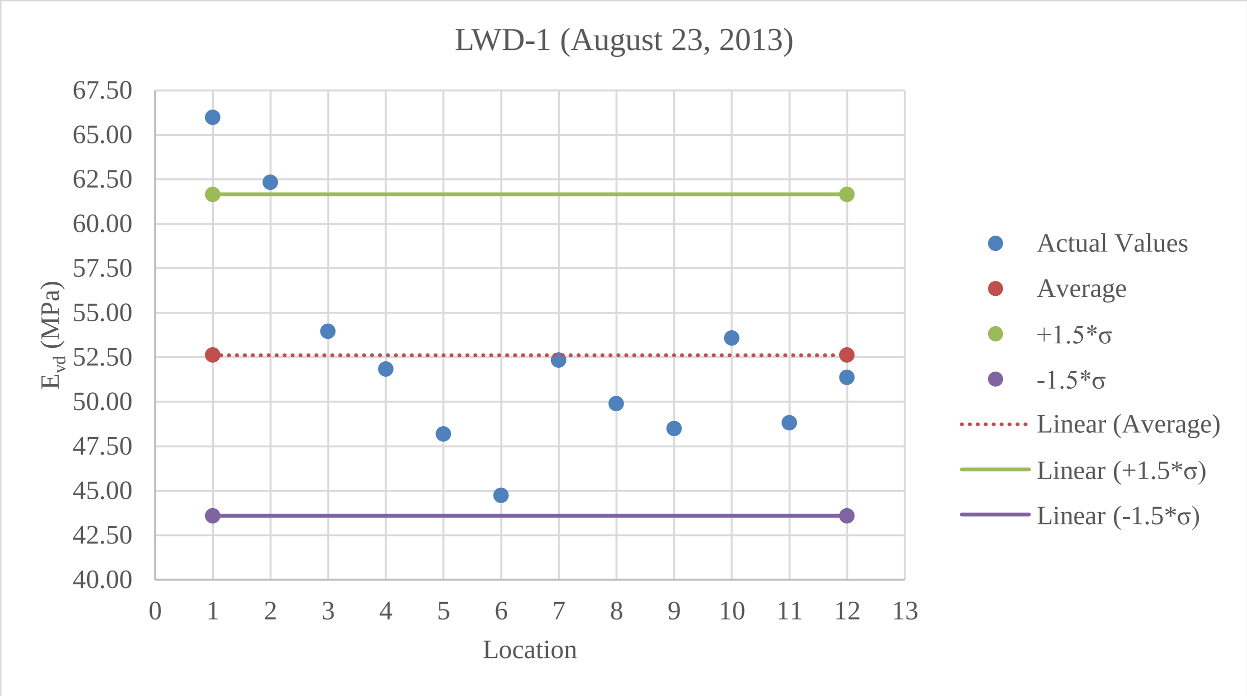

Figure C.1: Plot of LWD-1 values for A-3 at optimum moisture content

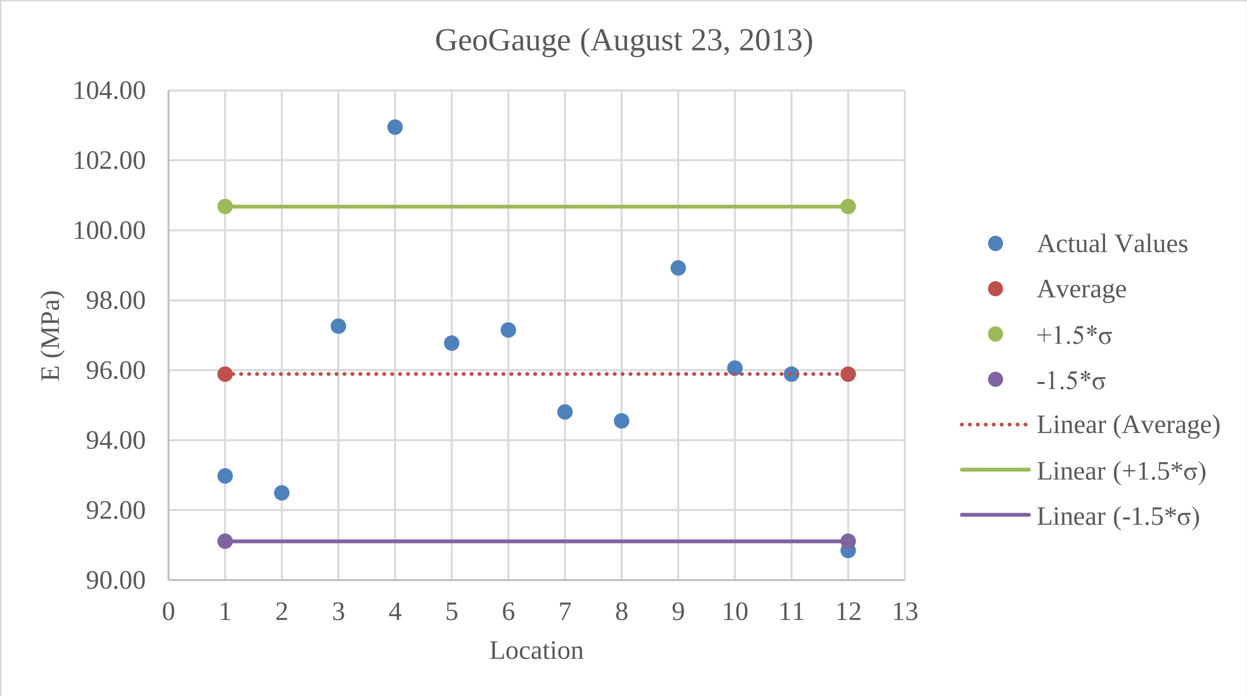

Figure C.2: Plot of GeoGauge values for A-3 at optimum moisture content

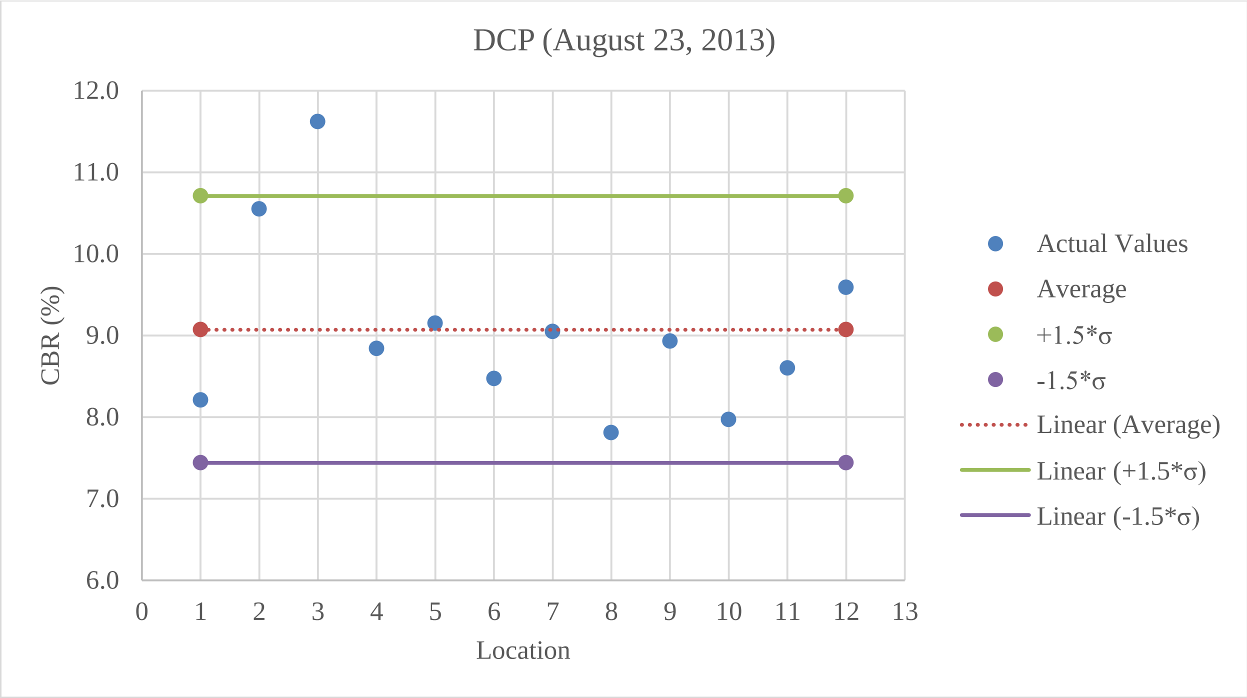

Figure C.3: Plot of DCP values for A-3 at optimum moisture content

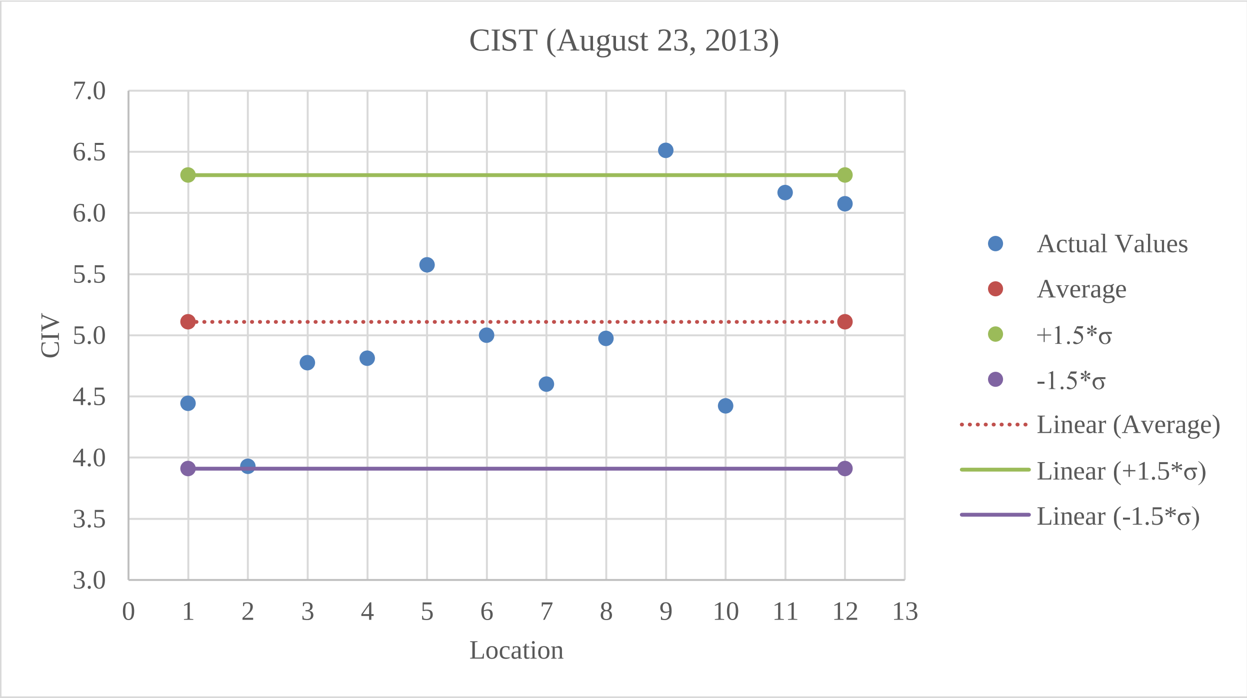

Figure C.4: Plot of CIST values for A-3 at optimum moisture content

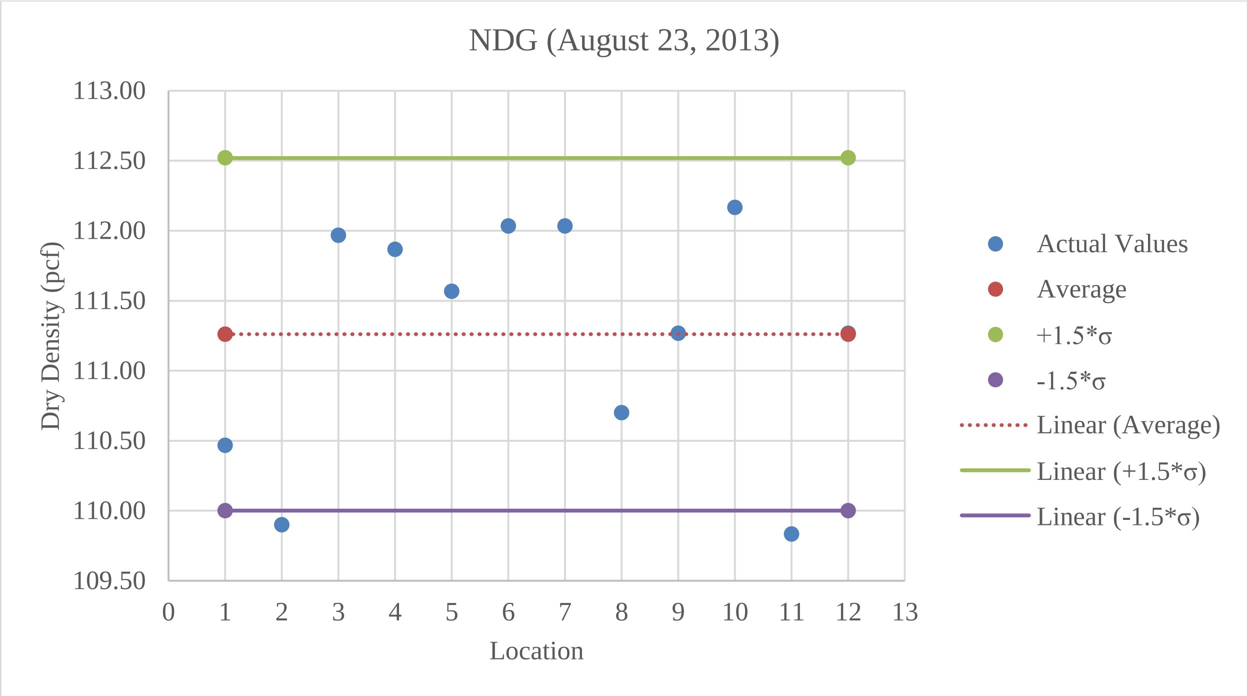

Figure C.5: Plot of NDG values for A-3 at optimum moisture content

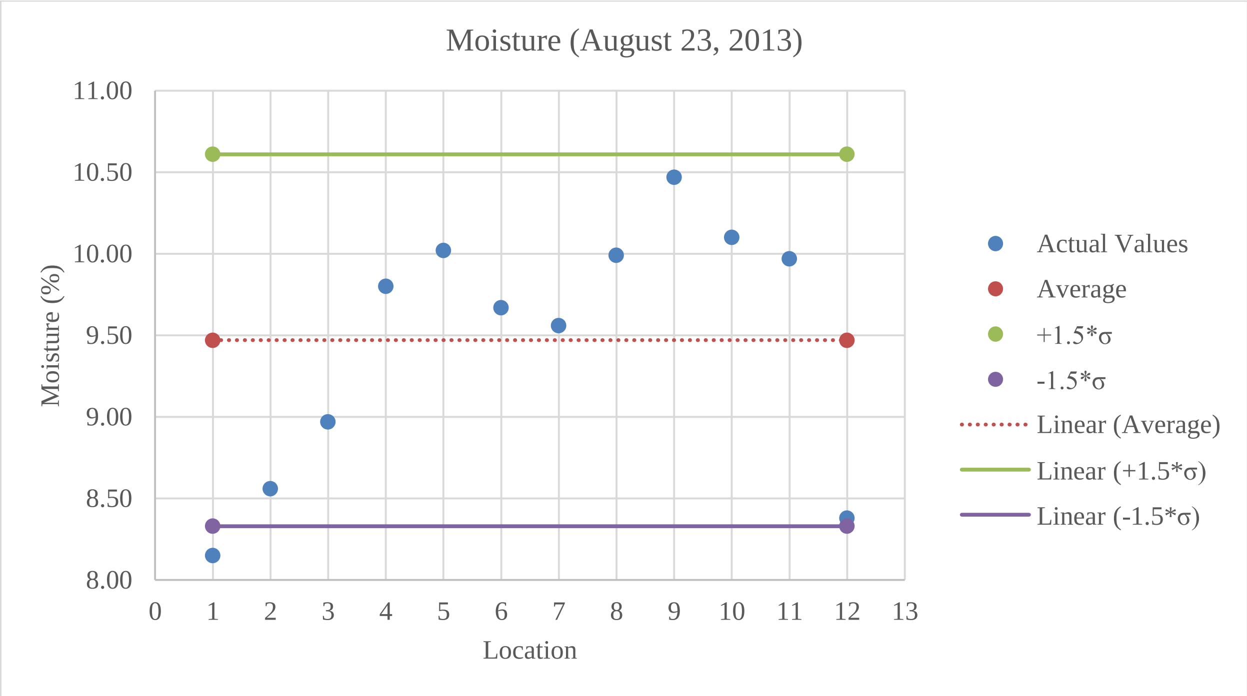

Figure C.6: Plot of moisture values for A-3

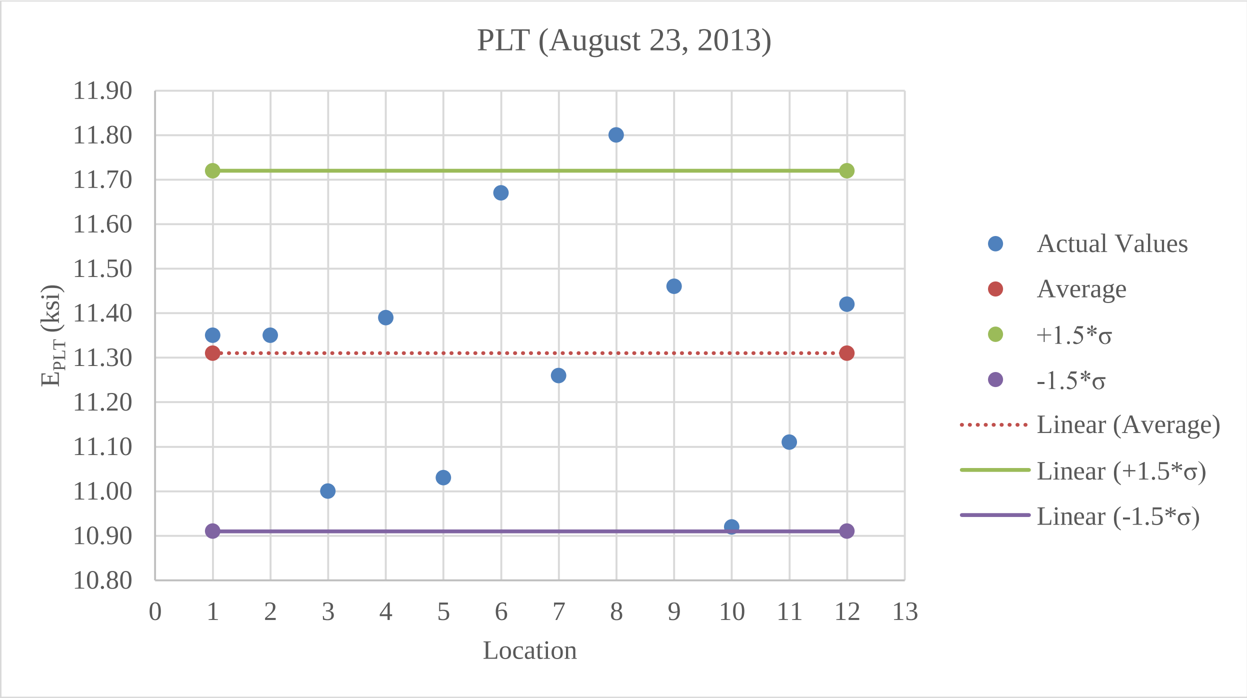

Figure C.7: Plot of PLT values for A-3 at optimum moisture content

C.2 Results for Devices of A-3 Material below Optimum Moisture Content

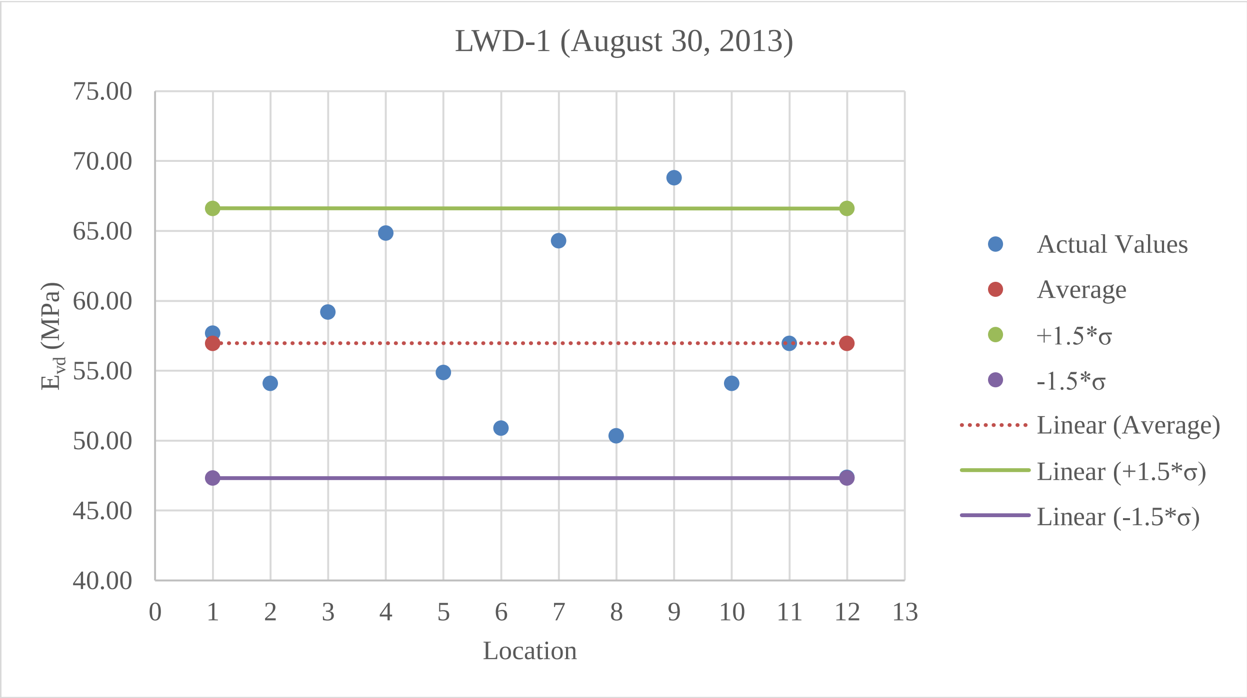

Figure C.8: Plot of LWD-1 values for A-3 at 7% less than optimum moisture content

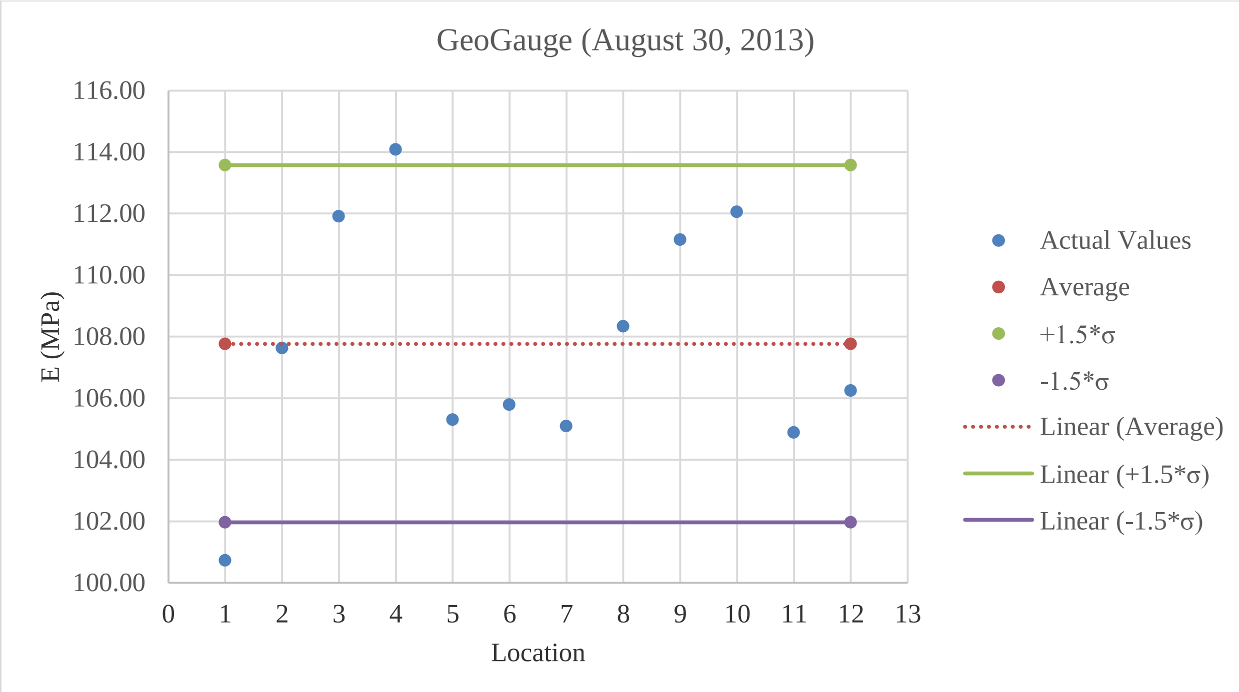

Figure C.9: Plot of GeoGauge values for A-3 at 7% less than optimum moisture content

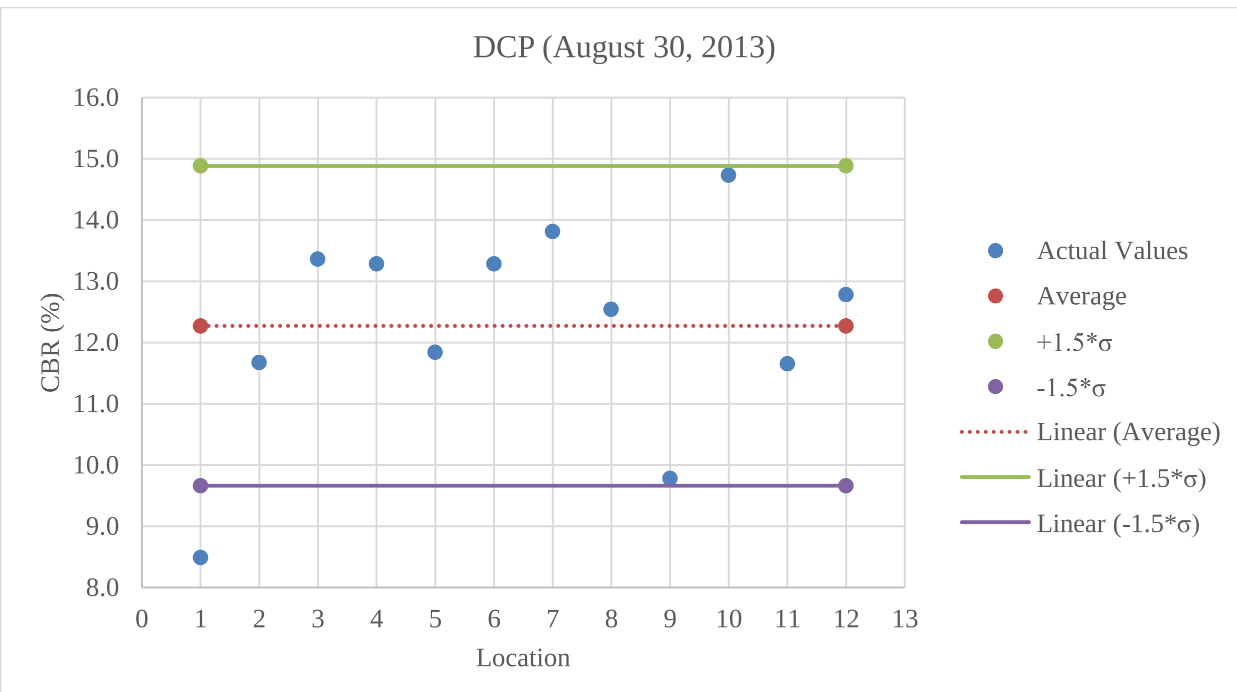

Figure C.10: Plot of DCP values for A-3 at 7% less than optimum moisture content

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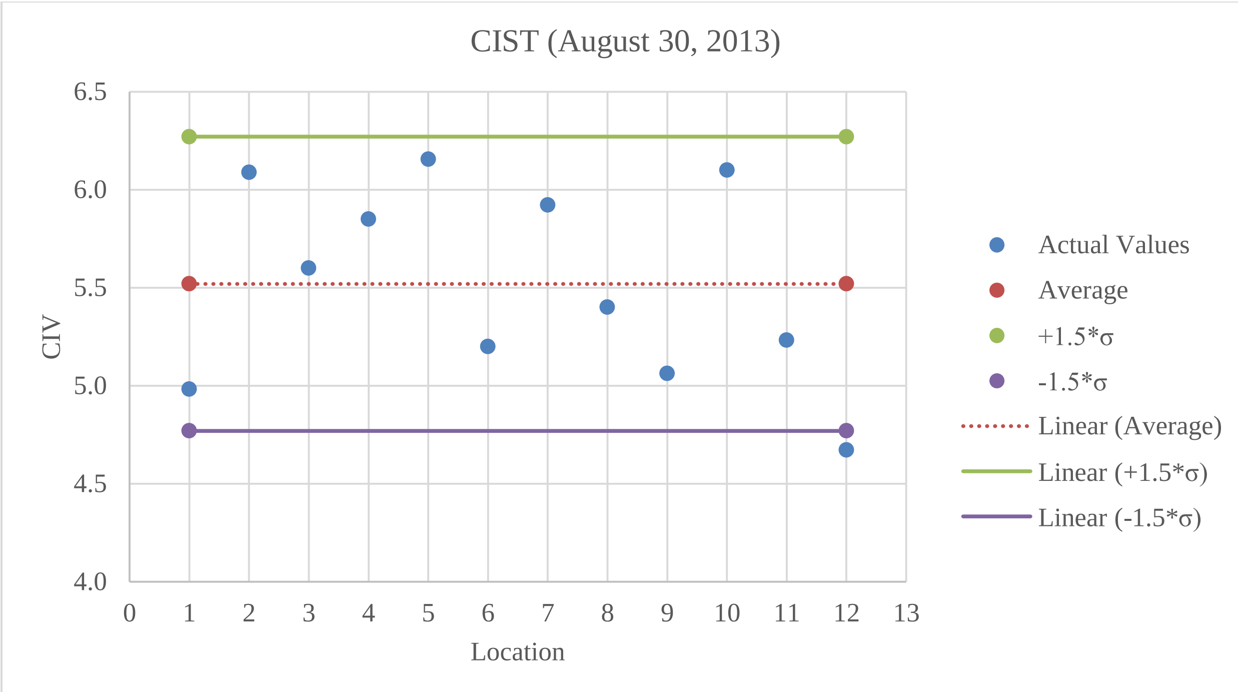

Figure C.11: Plot of CIST values for A-3 at 7% less than optimum moisture content

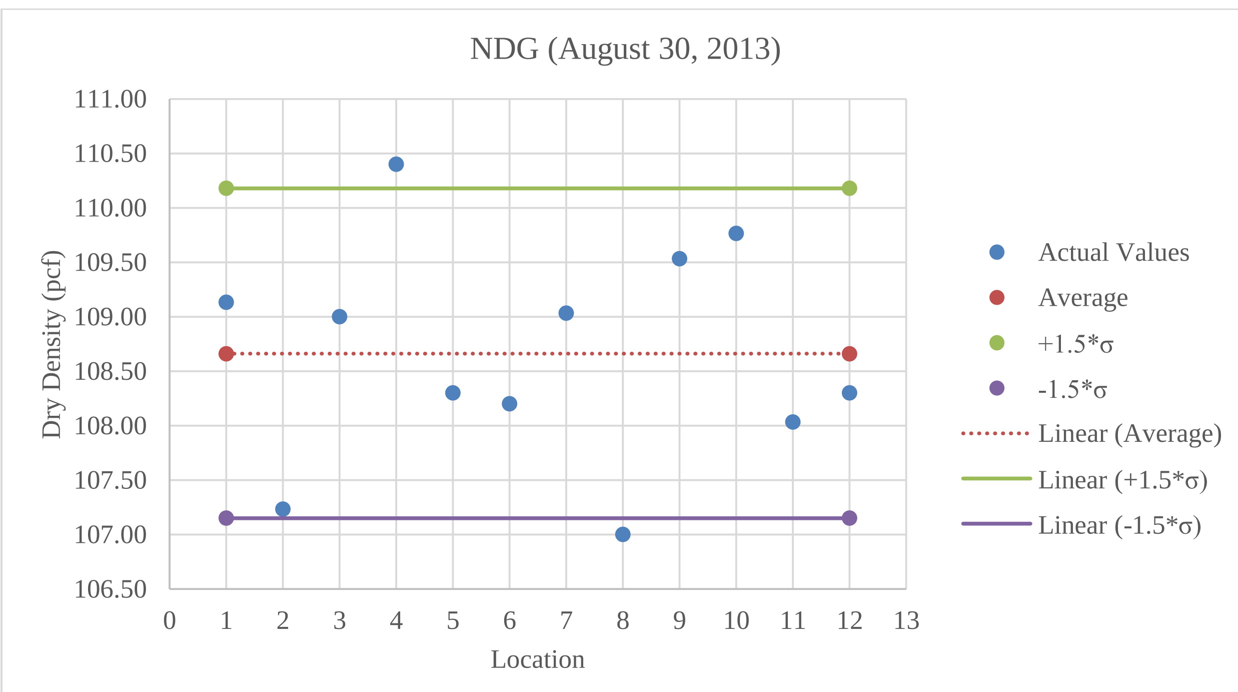

Figure C.12: Plot of NDG values for A-3 at 7% less than optimum moisture content

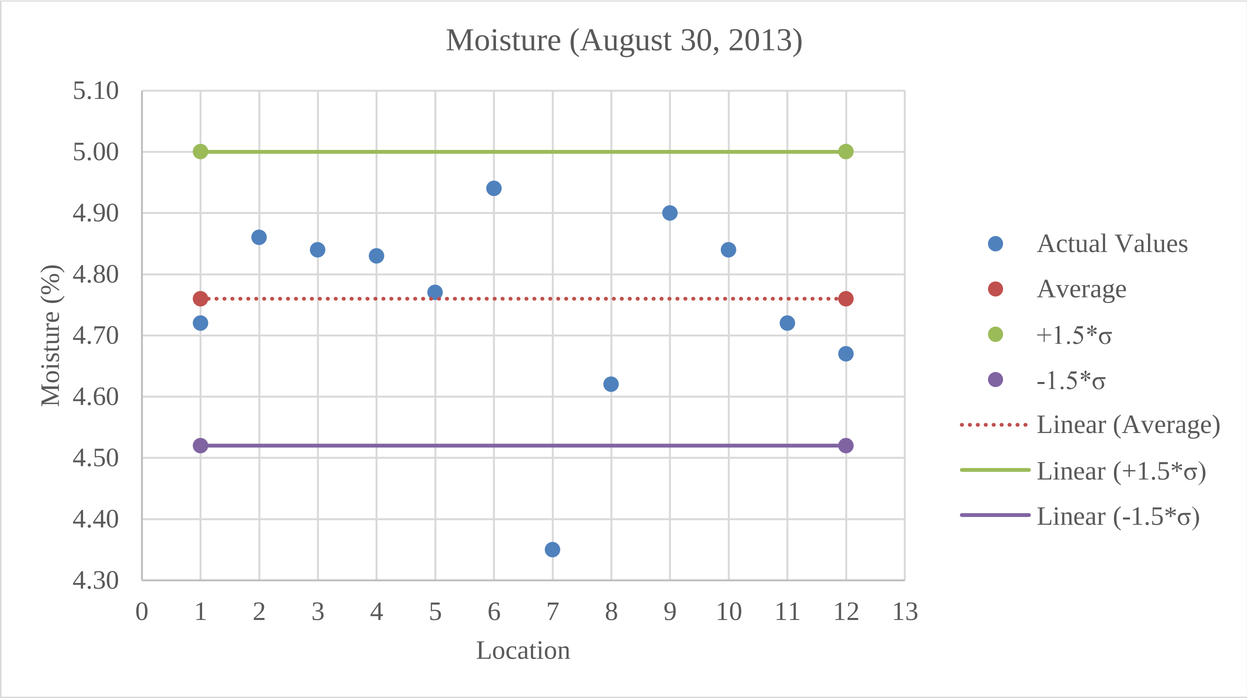

Figure C.13: Plot of moisture values for A-3

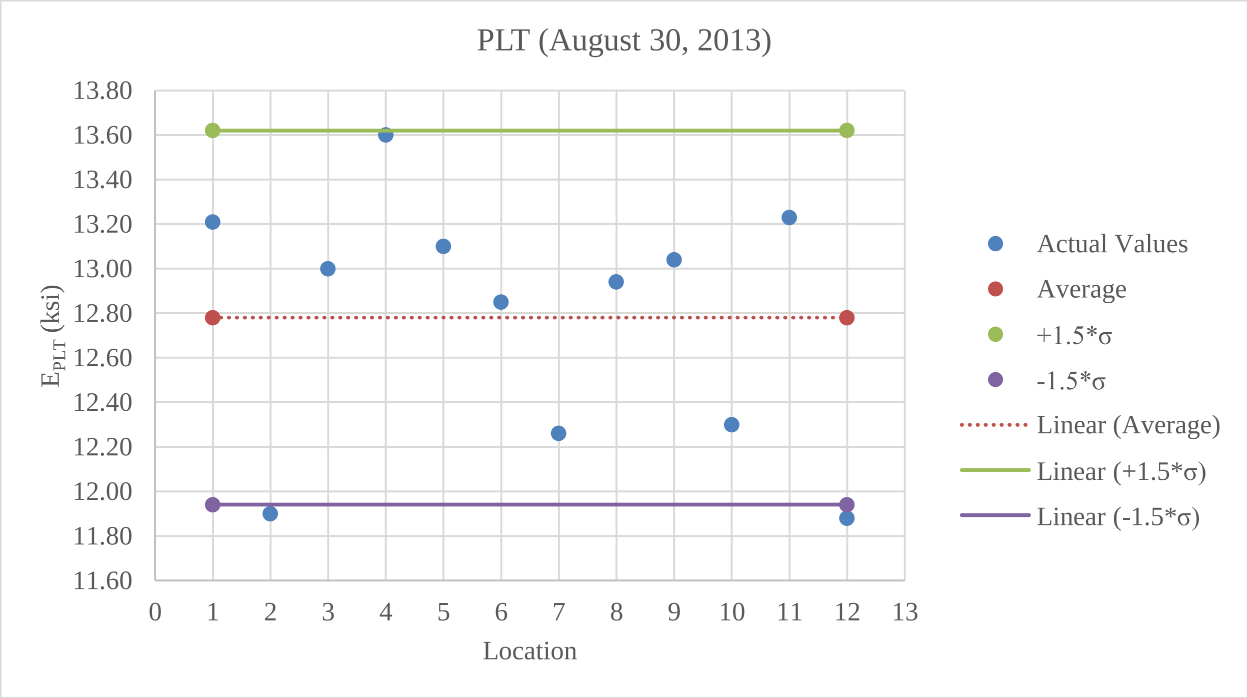

Figure C.14: Plot of PLT values for A-3 at 7% less than optimum moisture content

C.3 Results for Devices with A-2-4 Low Fines Material at Optimum Moisture Content

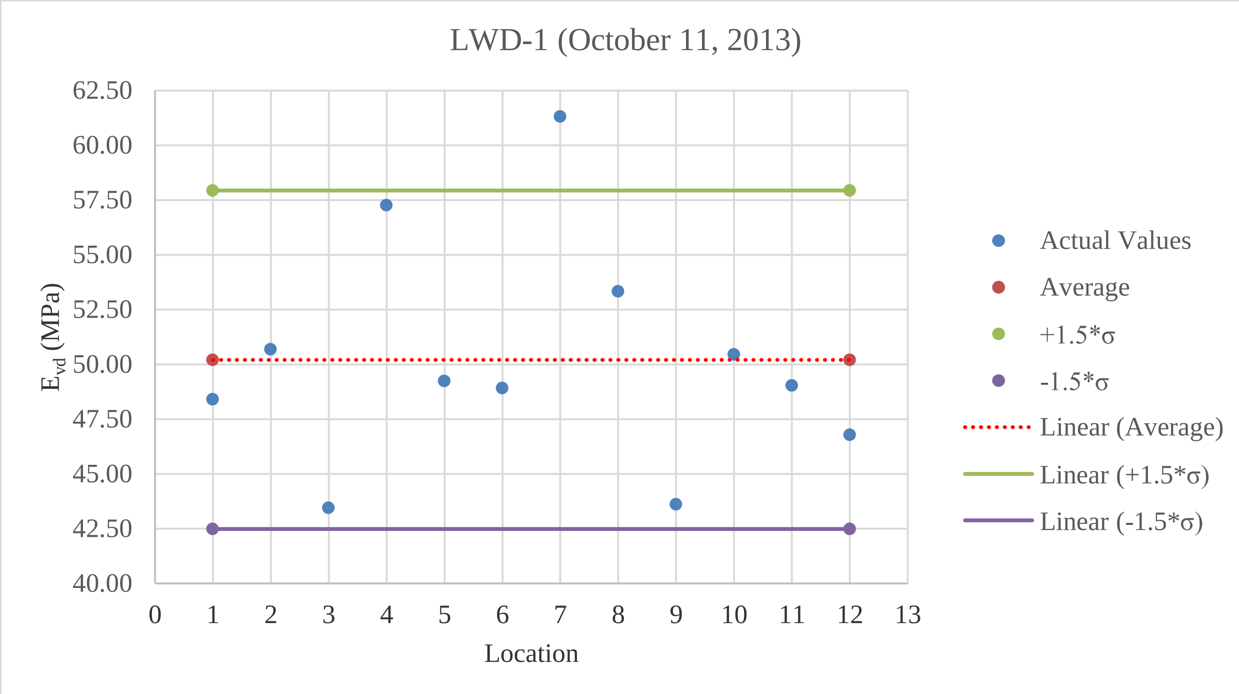

Figure C.15: Plot of LWD-1 values for A-2-4 low fines at optimum moisture content

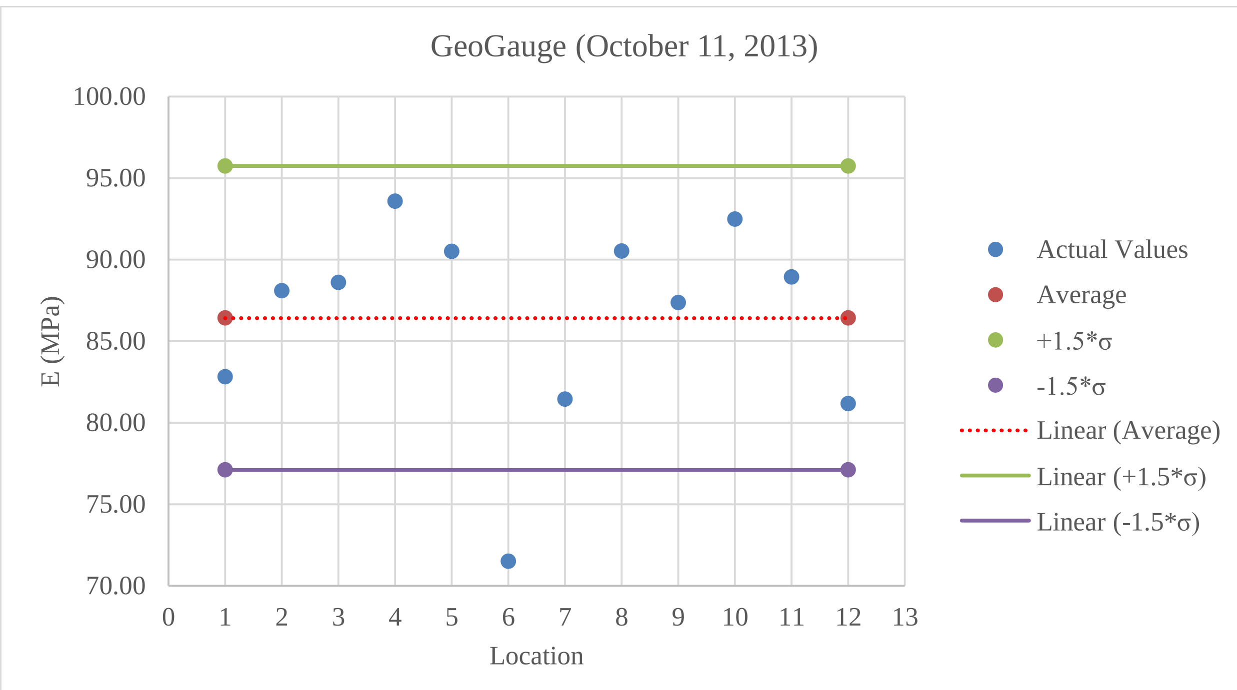

Figure C.16: Plot of GeoGauge values for A-2-4 low fines at optimum moisture content

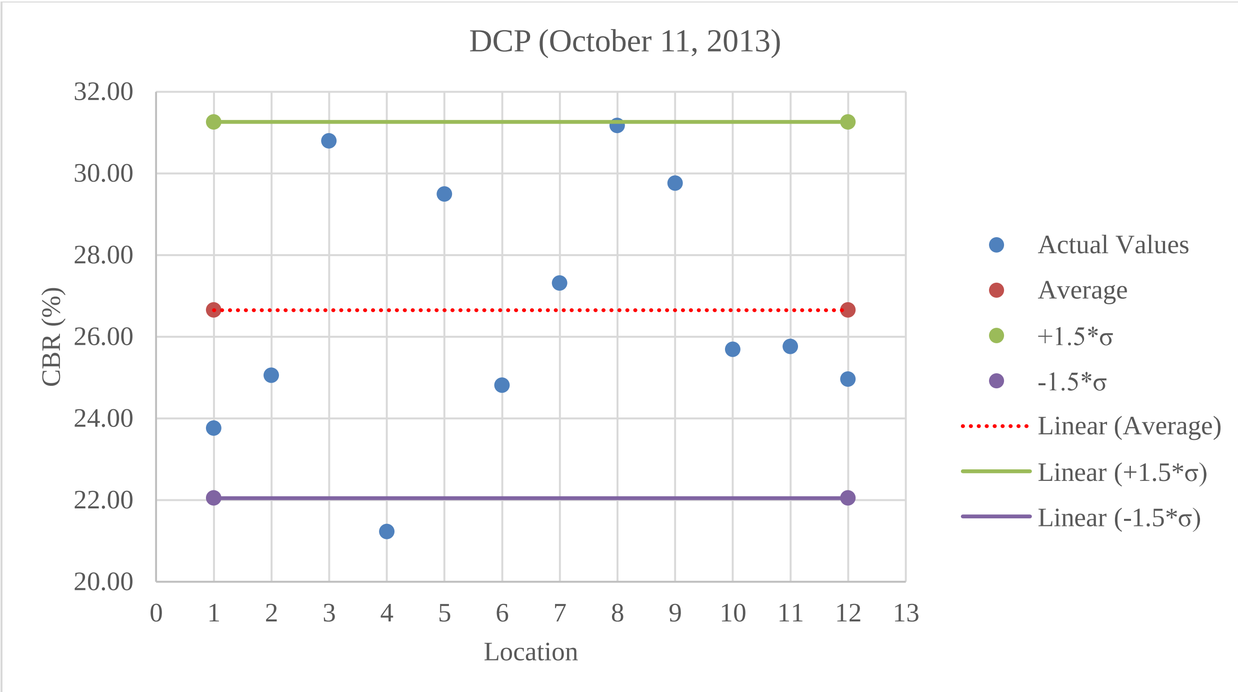

Figure C.17: Plot of DCP values for A-2-4 low fines at optimum moisture content

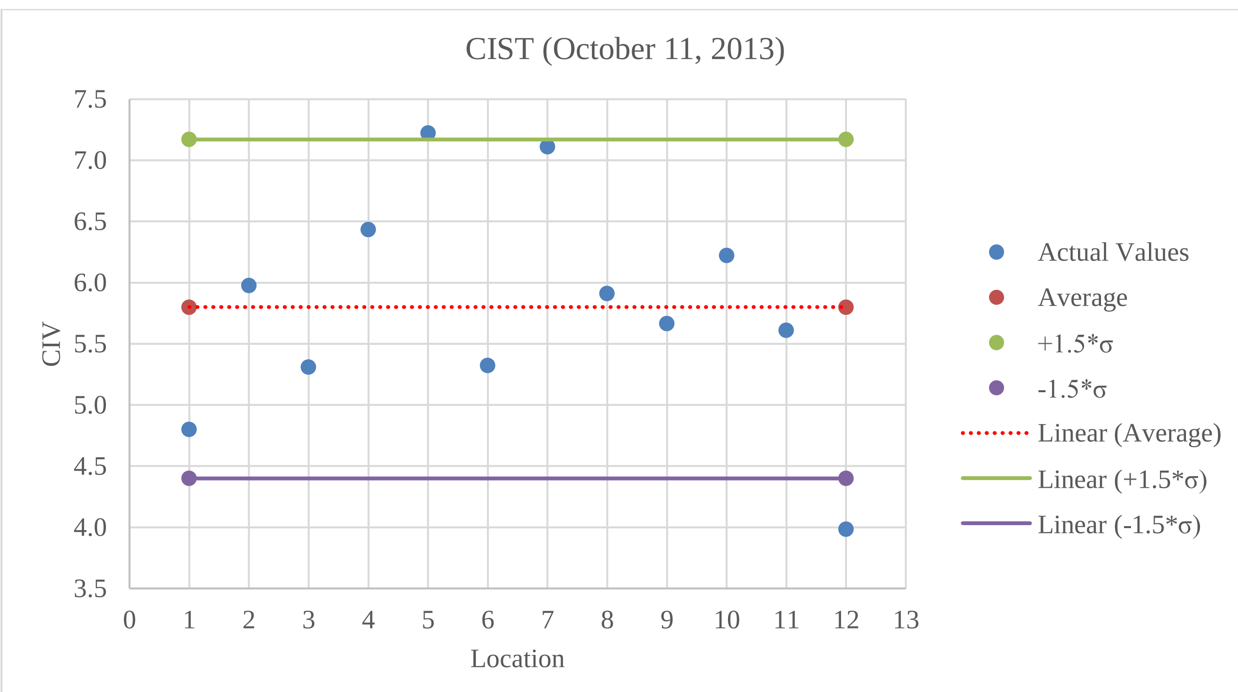

Figure C.18: Plot of CIST values for A-2-4 low fines at optimum moisture content

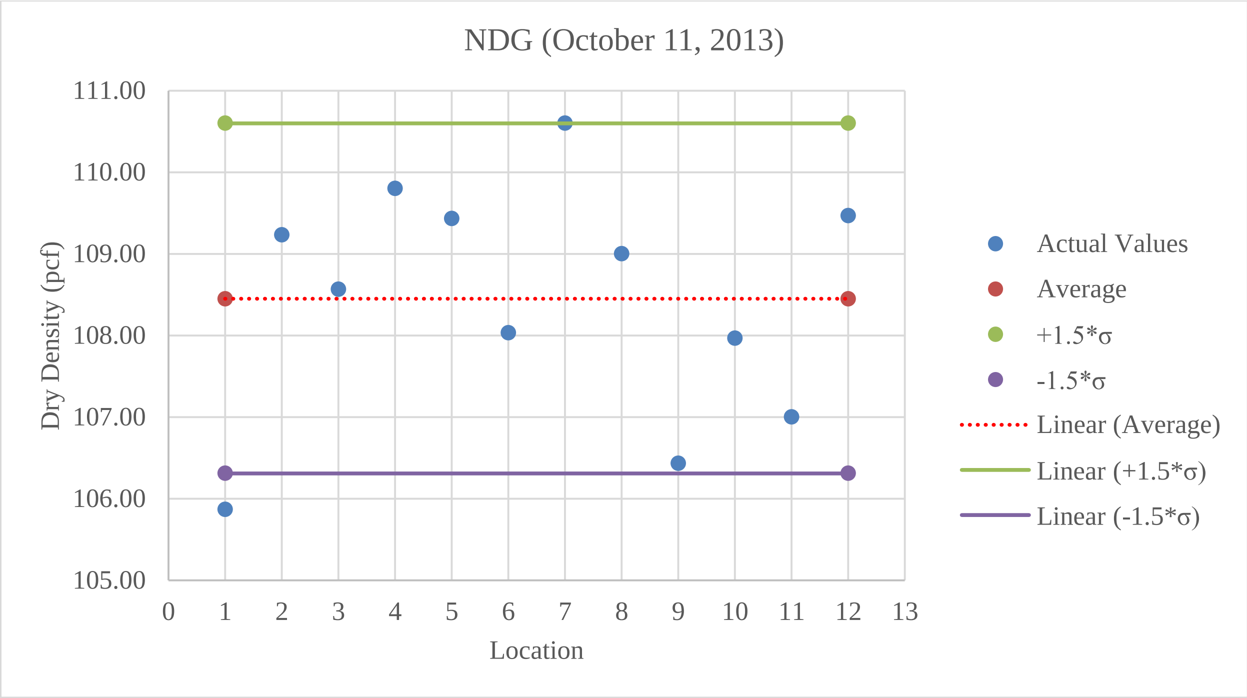

Figure C.19: Plot of NDG values for A-2-4 low fines at optimum moisture content

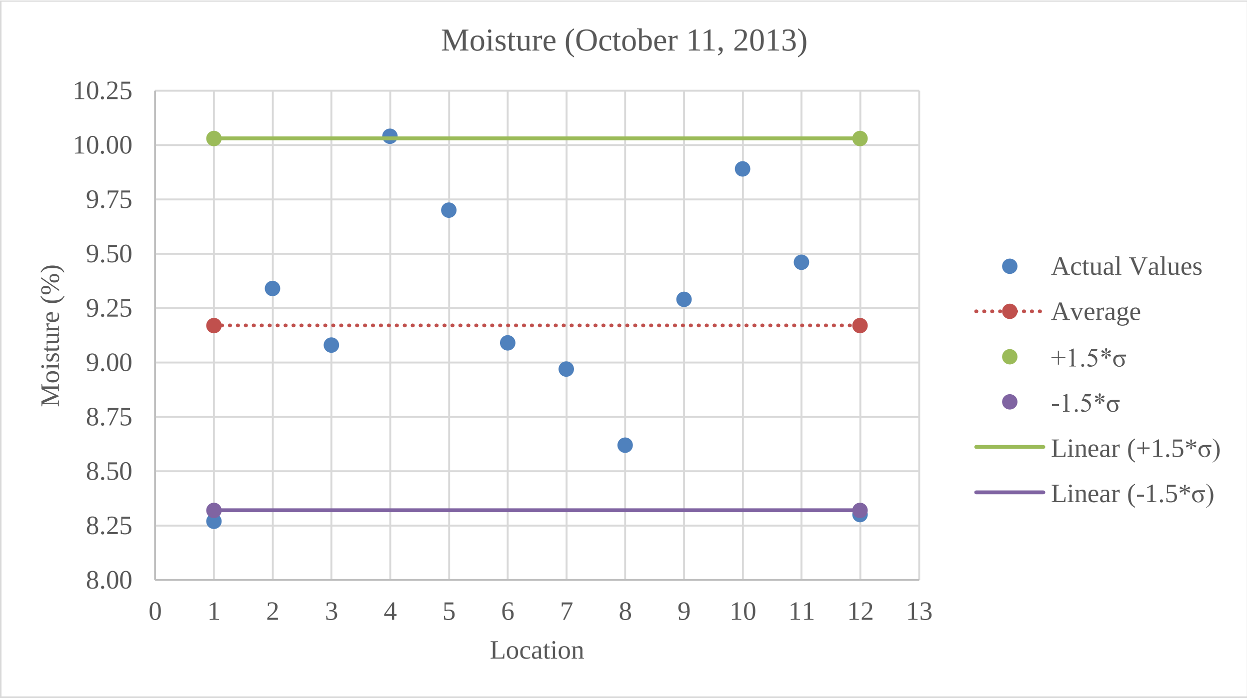

Figure C.20: Plot of moisture values for A-2-4 low fines

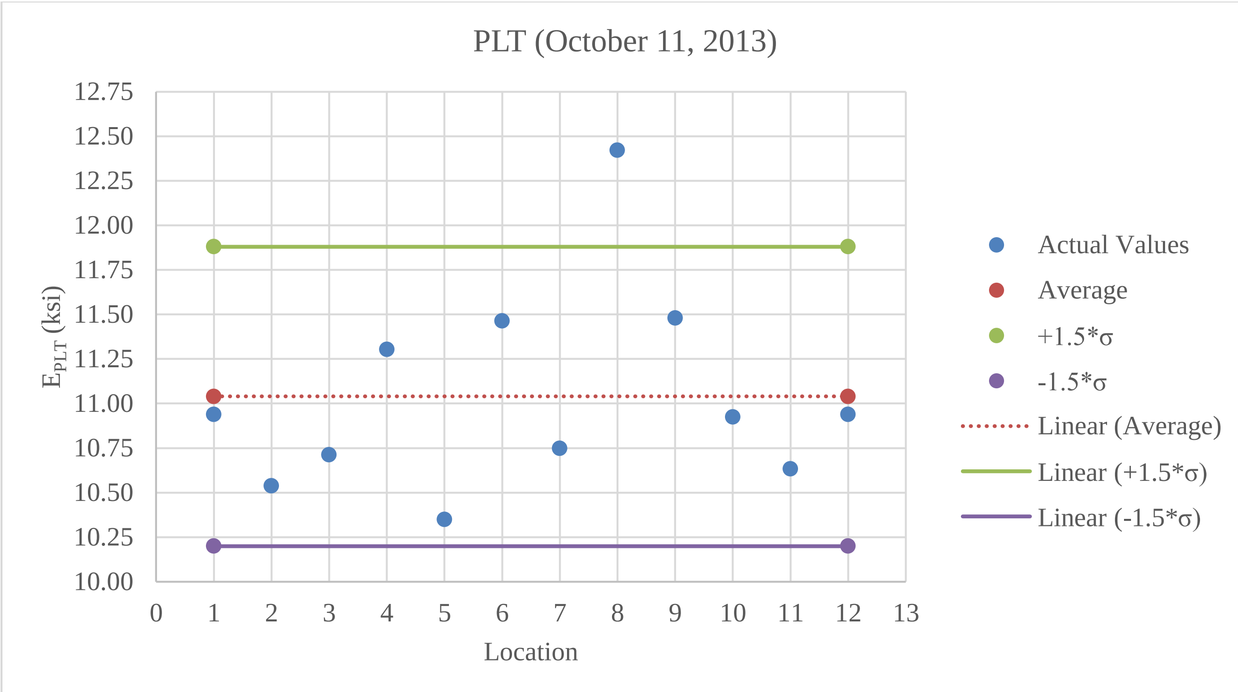

Figure C.21: Plot of PLT values for A-2-4 low fines at optimum moisture content

C.4 Results for Devices with A-2-4 Low Fines Material below Optimum Moisture Content

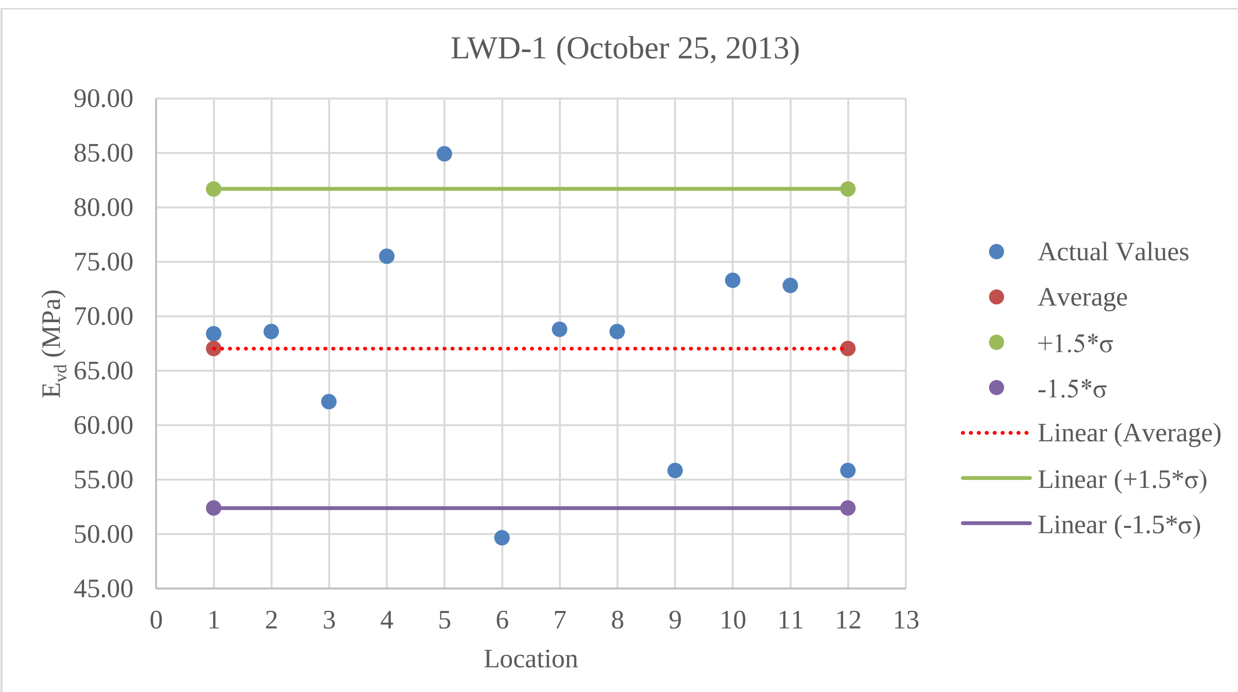

Figure C.22: Plot of LWD-1 values for A-2-4 low fines at 6% less than optimum moisture content

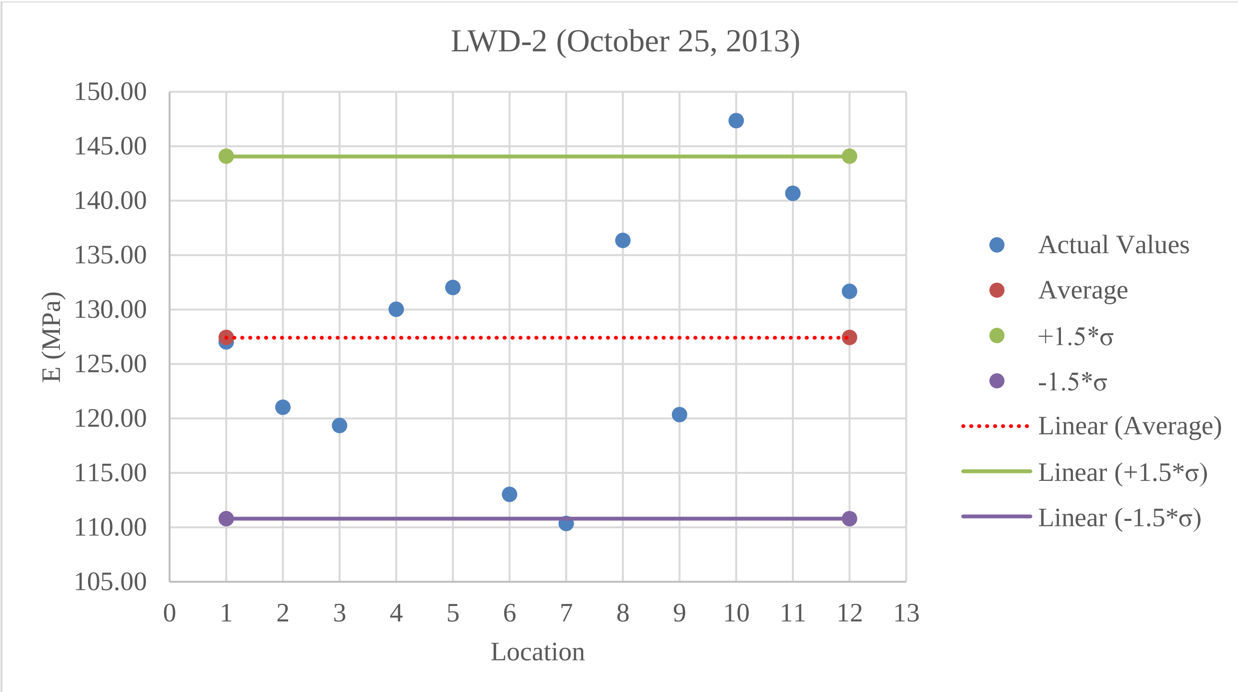

Figure C.23: Plot of LWD-2 values for A-2-4 low fines at 6% less than optimum moisture content

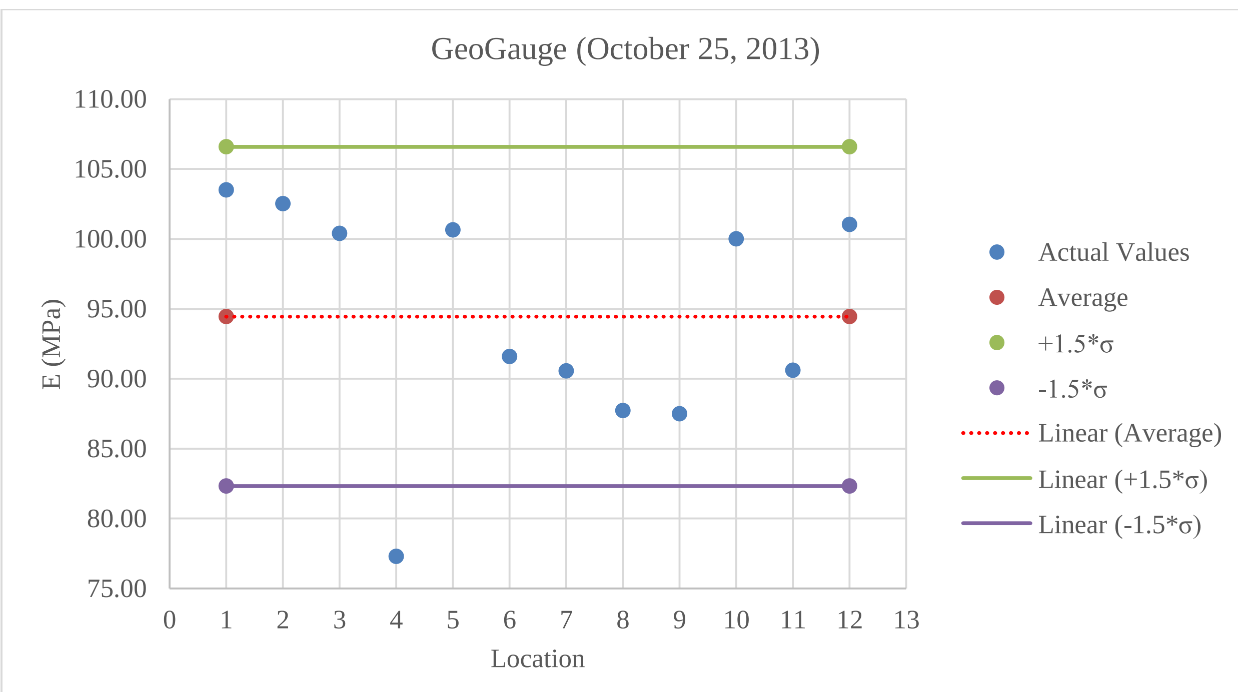

Figure C.24: Plot of GeoGauge values for A-2-4 low fines at 6% less than optimum moisture content

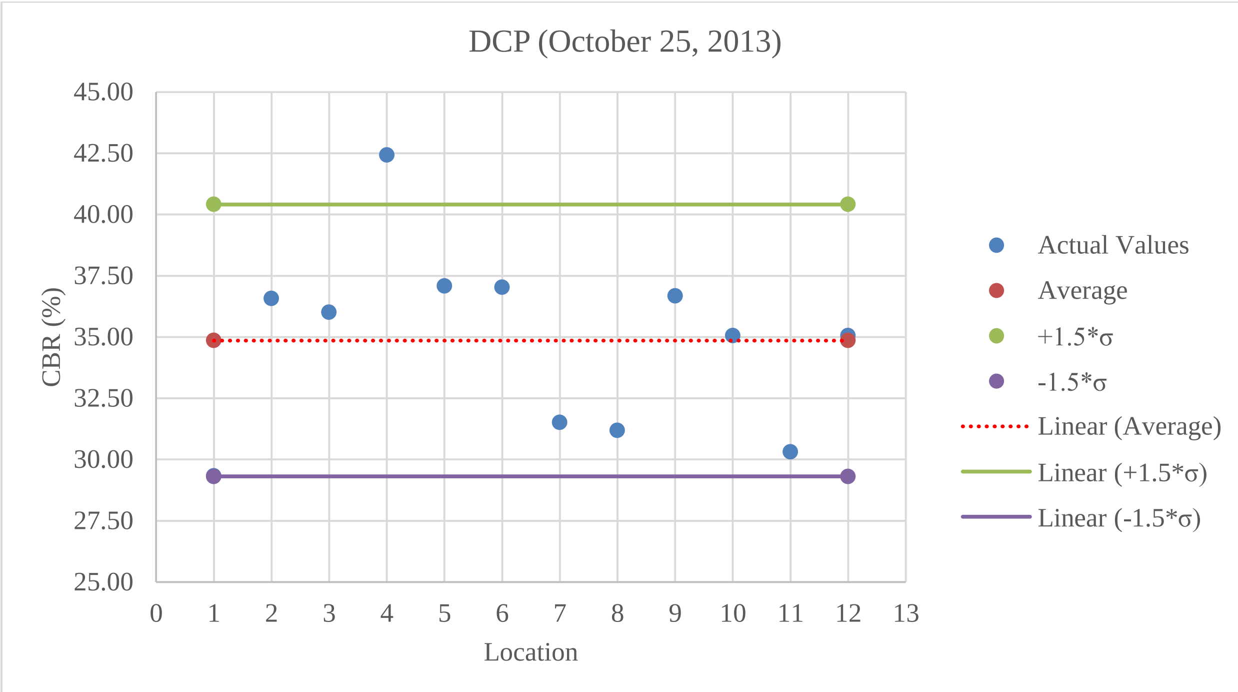

Figure C.25: Plot of DCP values for A-2-4 low fines at 6% less than optimum moisture content

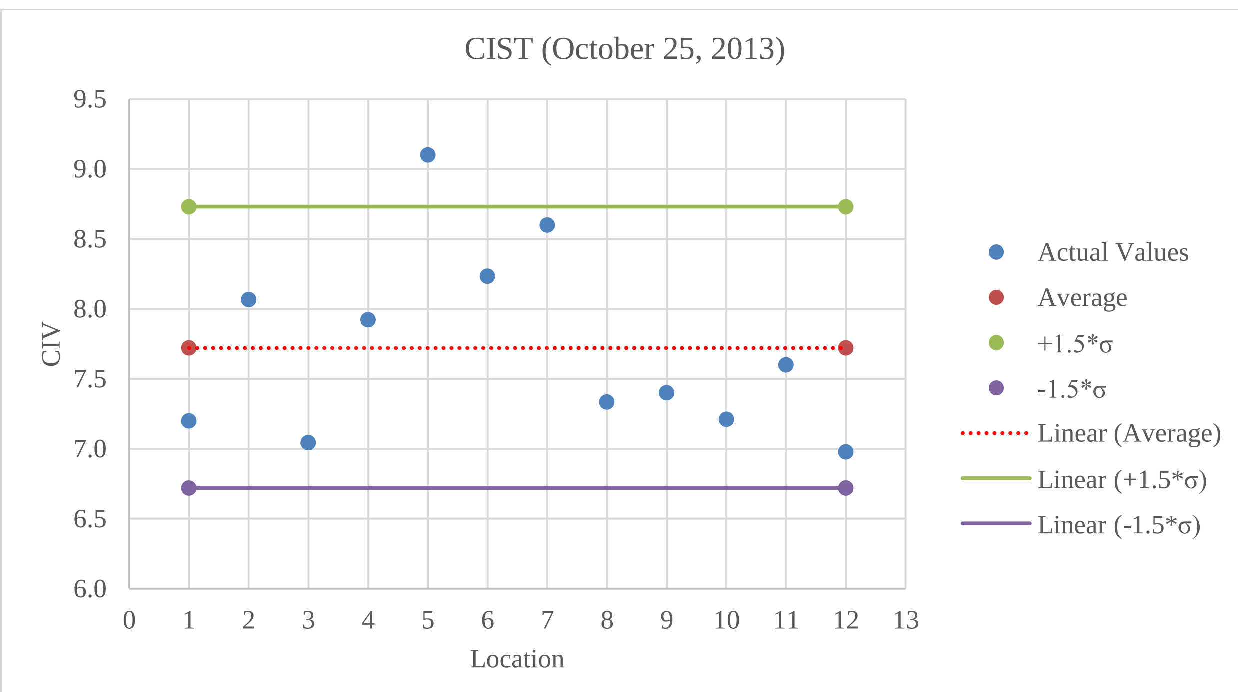

Figure C.26: Plot of CIST values for A-2-4 low fines at 6% less than optimum moisture content

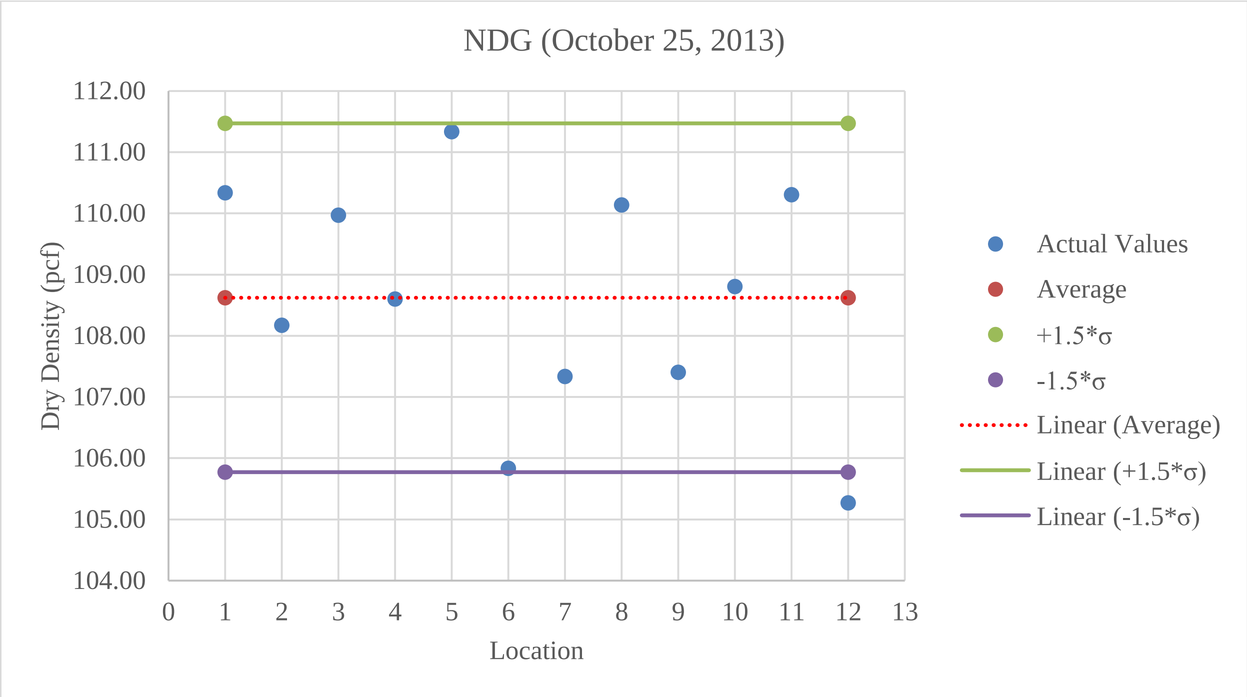

Figure C.27: Plot of NDG values for A-2-4 low fines at 6% less than optimum moisture content

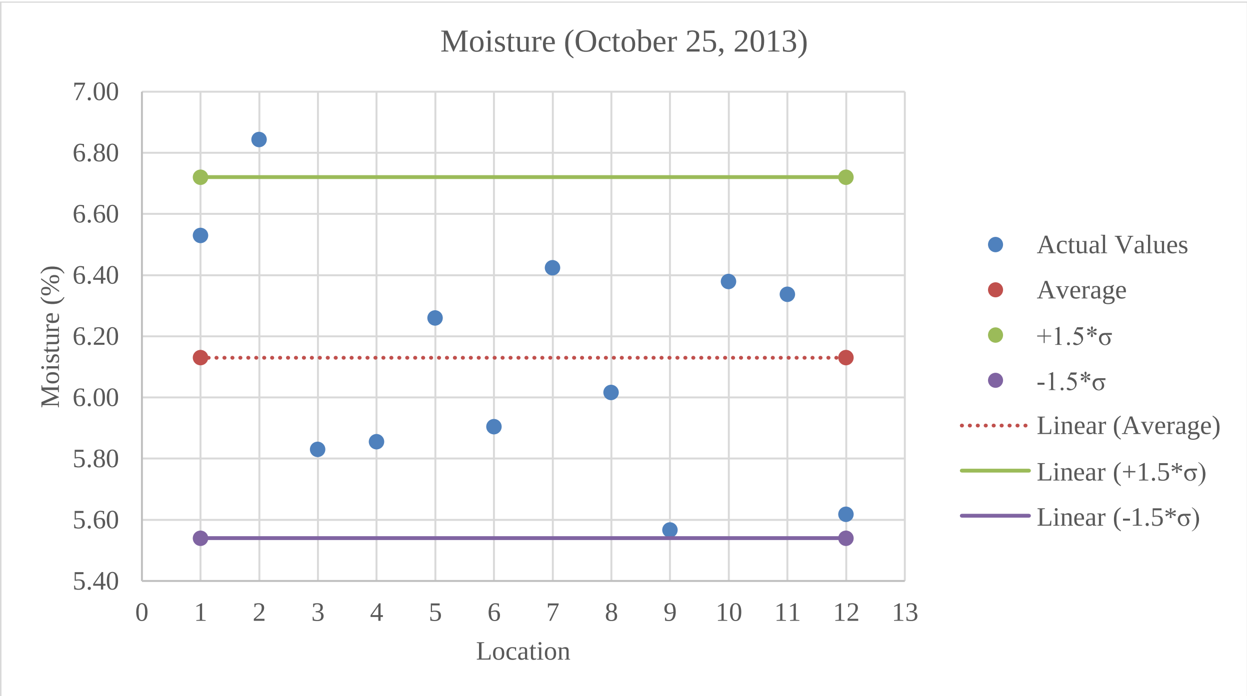

Figure C.28: Plot of moisture values for A-2-4 low fines

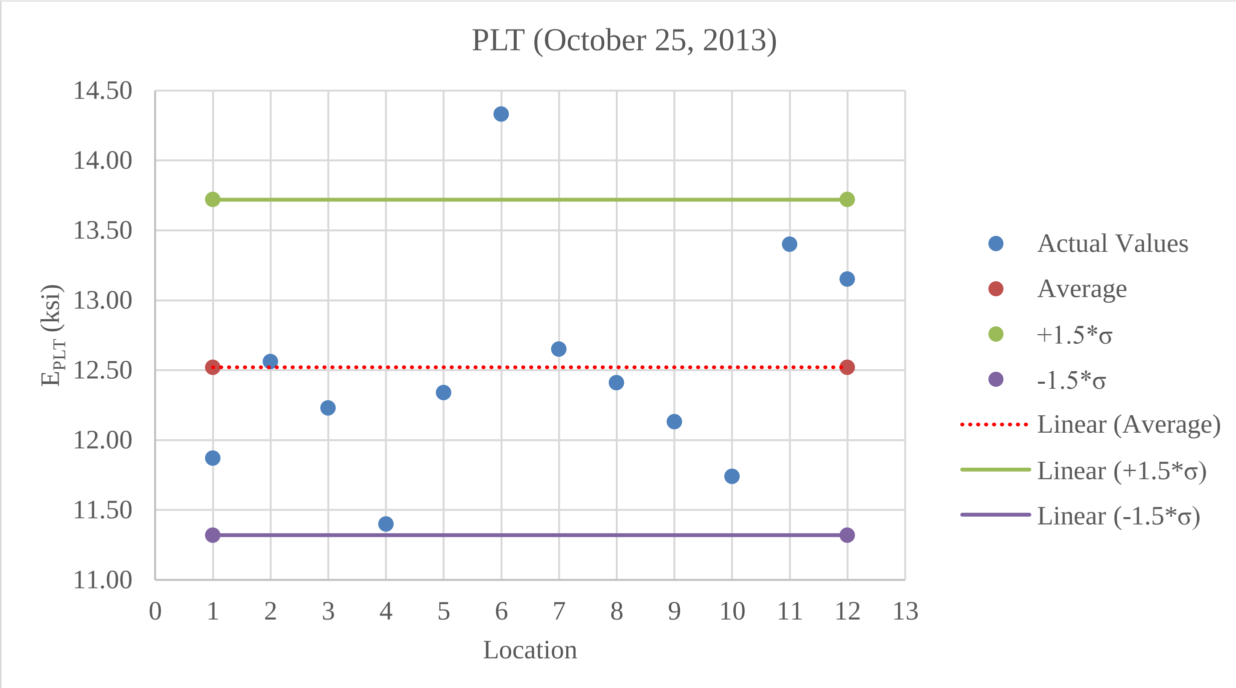

Figure C.29: Plot of PLT values for A-2-4 low fines at 6% less than optimum moisture content

C.5 Results for Devices with Limerock Material at Optimum Moisture Content

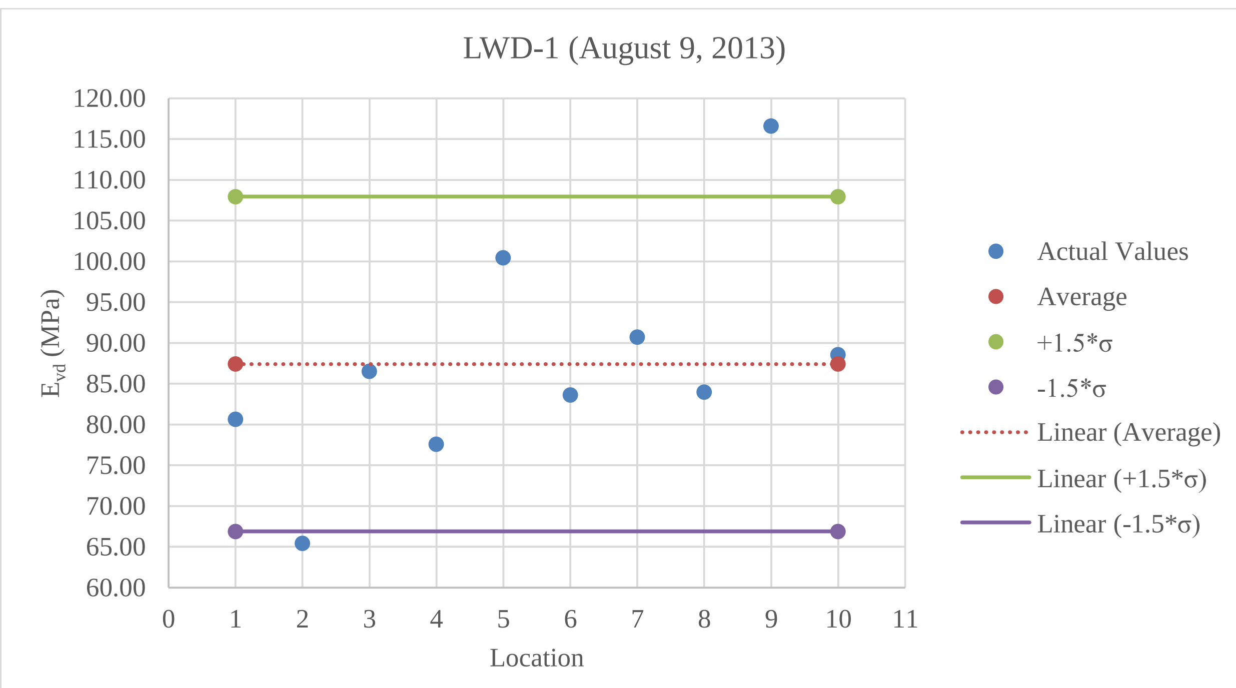

Figure C.30: Plot of LWD-1 values for limerock at optimum moisture content

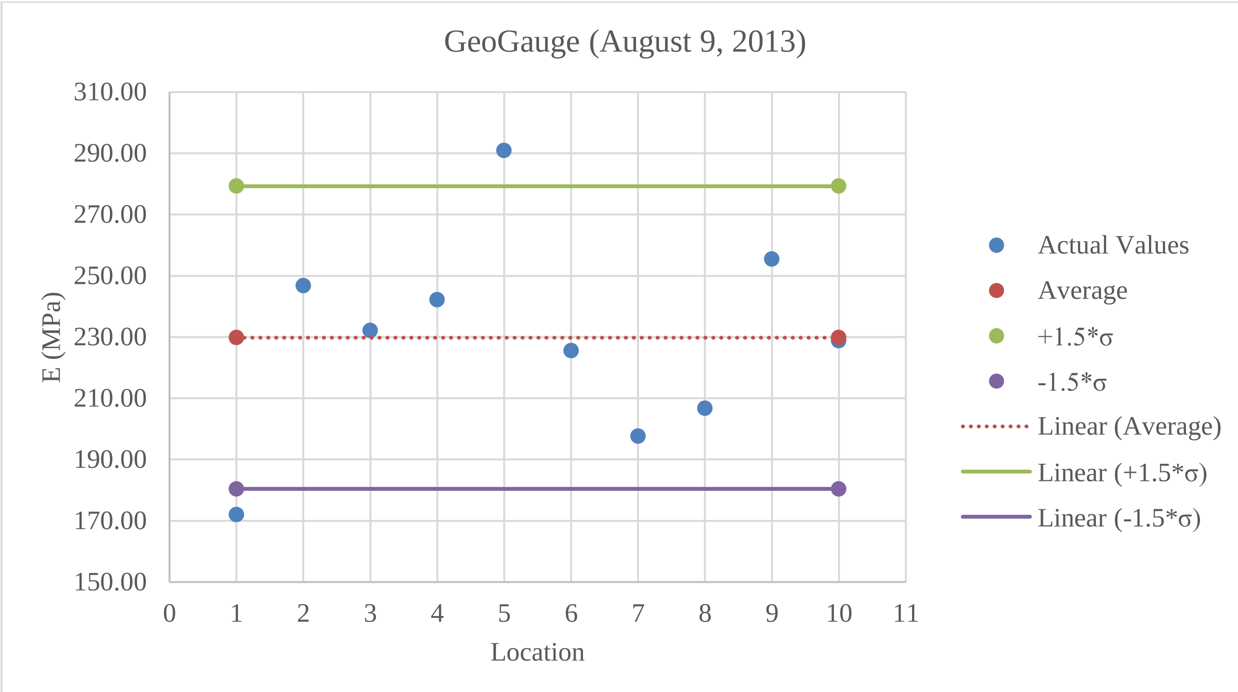

Figure C.31: Plot of GeoGauge values for limerock at optimum moisture content

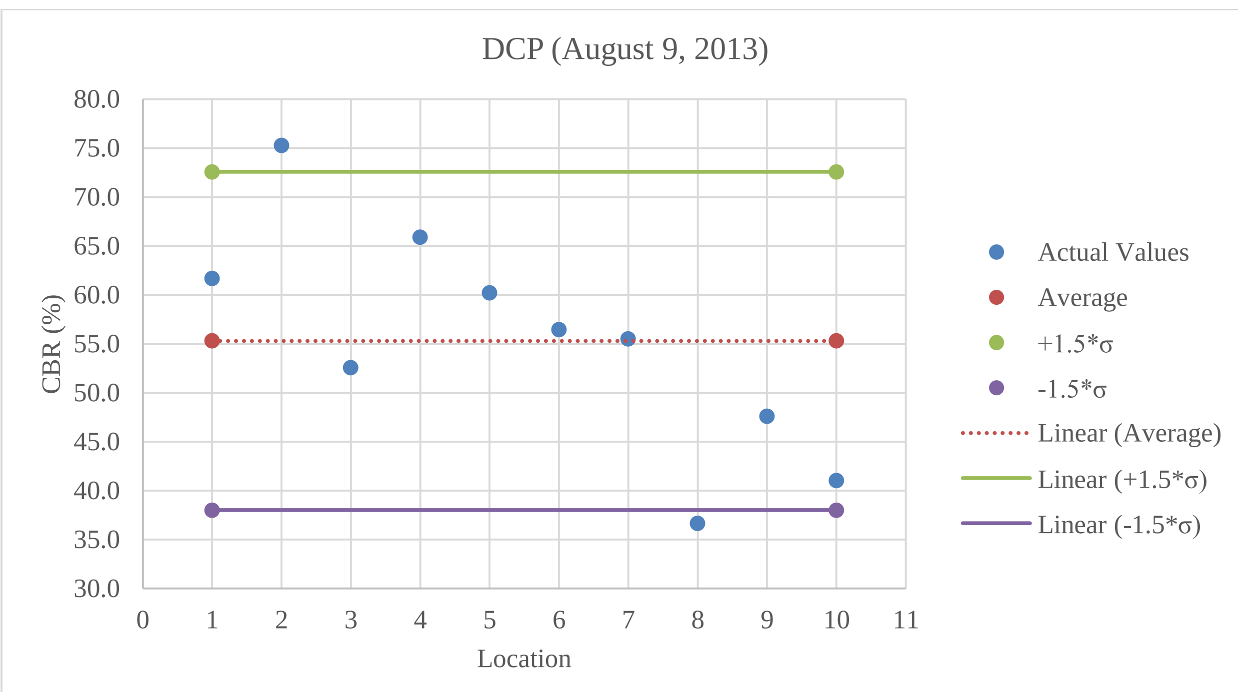

Figure C.32: Plot of DCP values for limerock at optimum moisture content

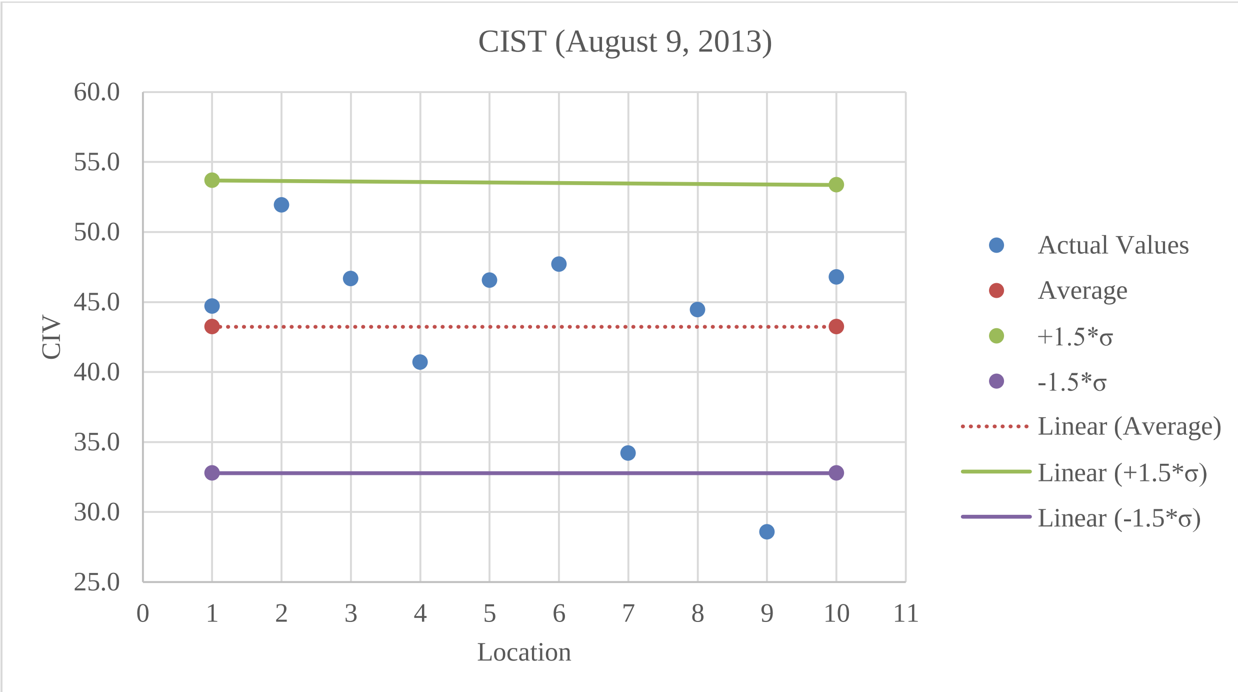

Figure C.33: Plot of CIST values for limerock at optimum moisture content

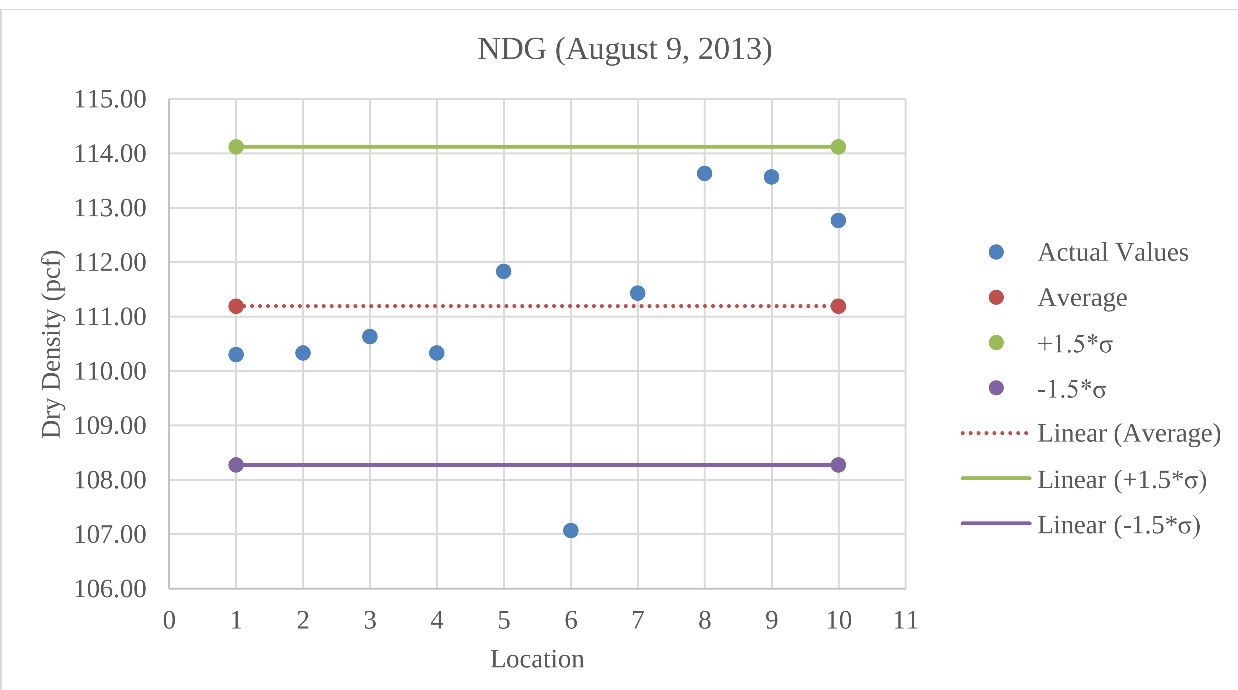

Figure C.34: Plot of NDG values for limerock at optimum moisture content

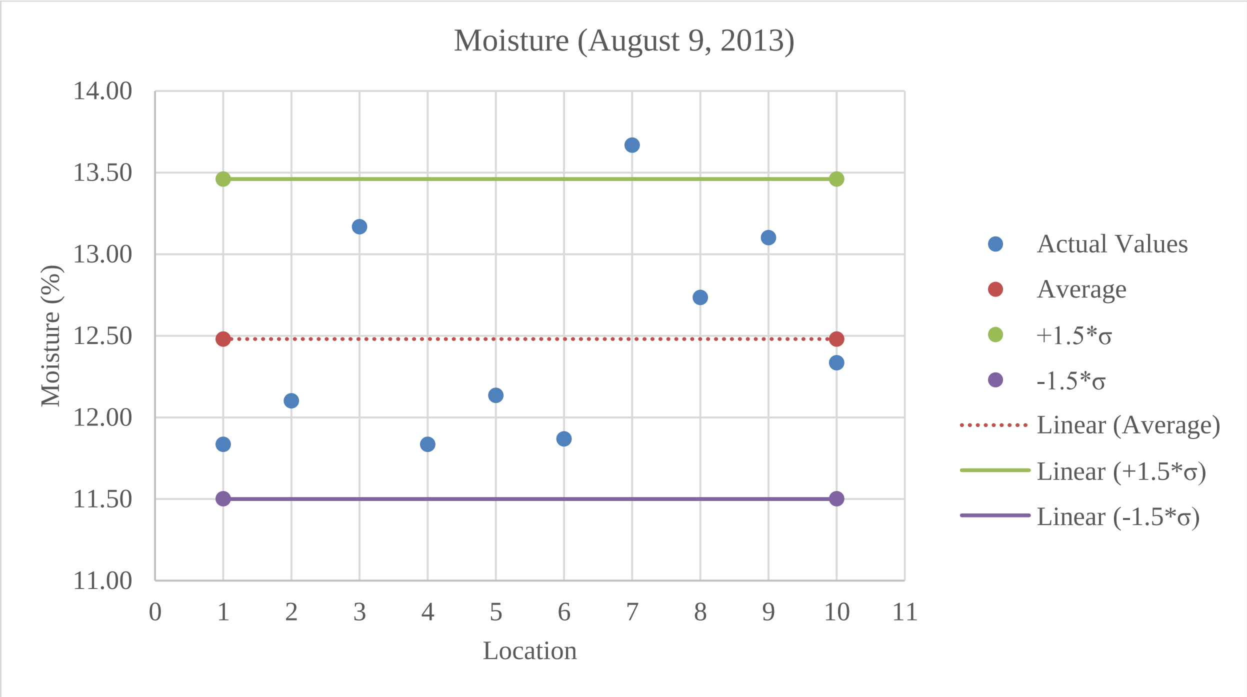

Figure C.35: Plot of moisture values for limerock

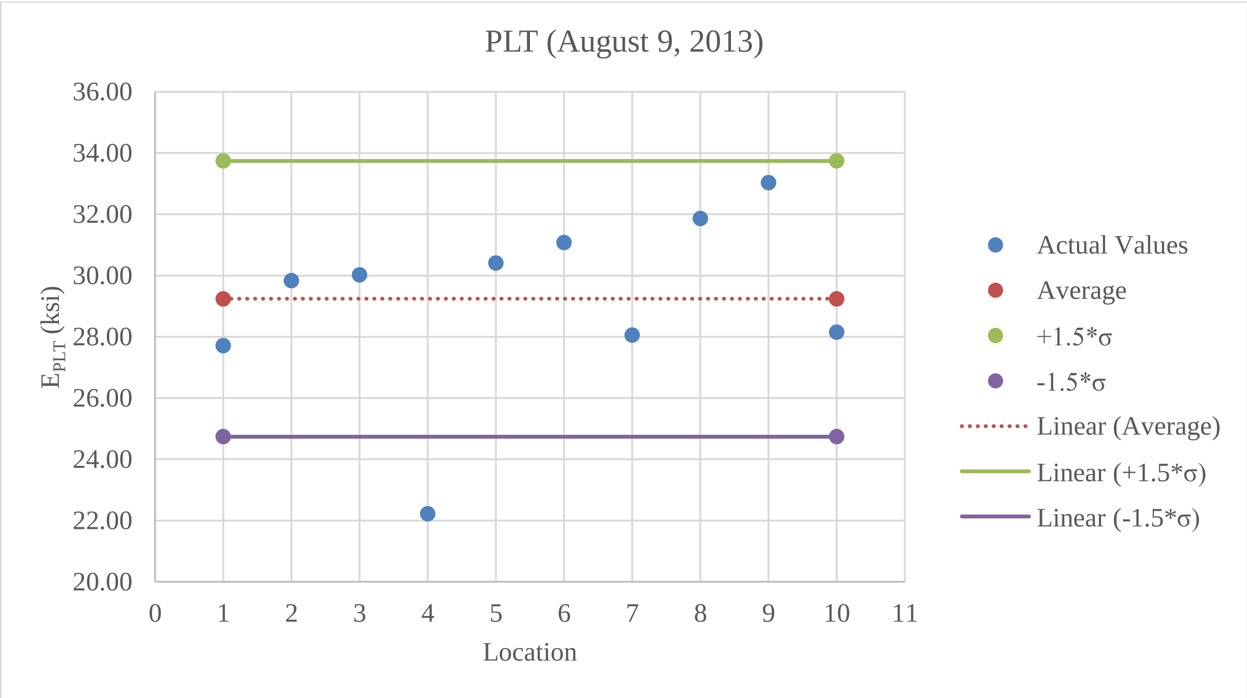

Figure C.36: Plot of PLT values for limerock at optimum moisture content

C.6 Results for Devices with Limerock Material below Optimum Moisture Content

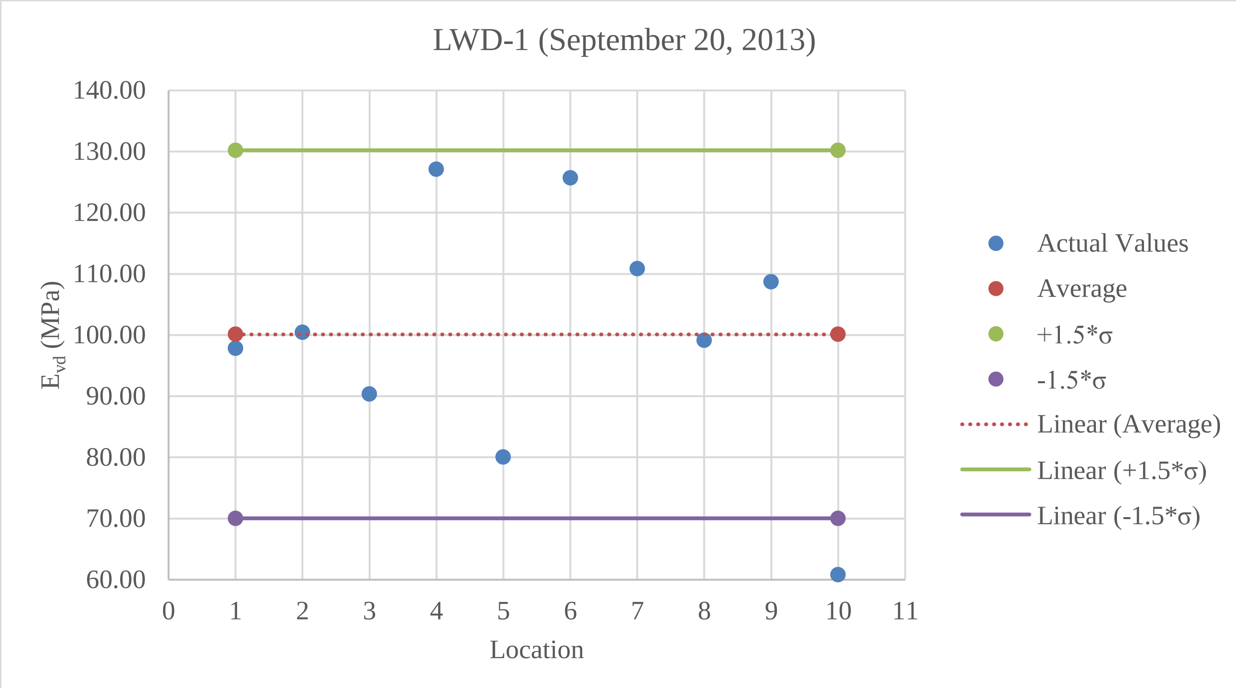

Figure C.37: Plot of LWD-1 values for limerock at 3% less than optimum moisture content

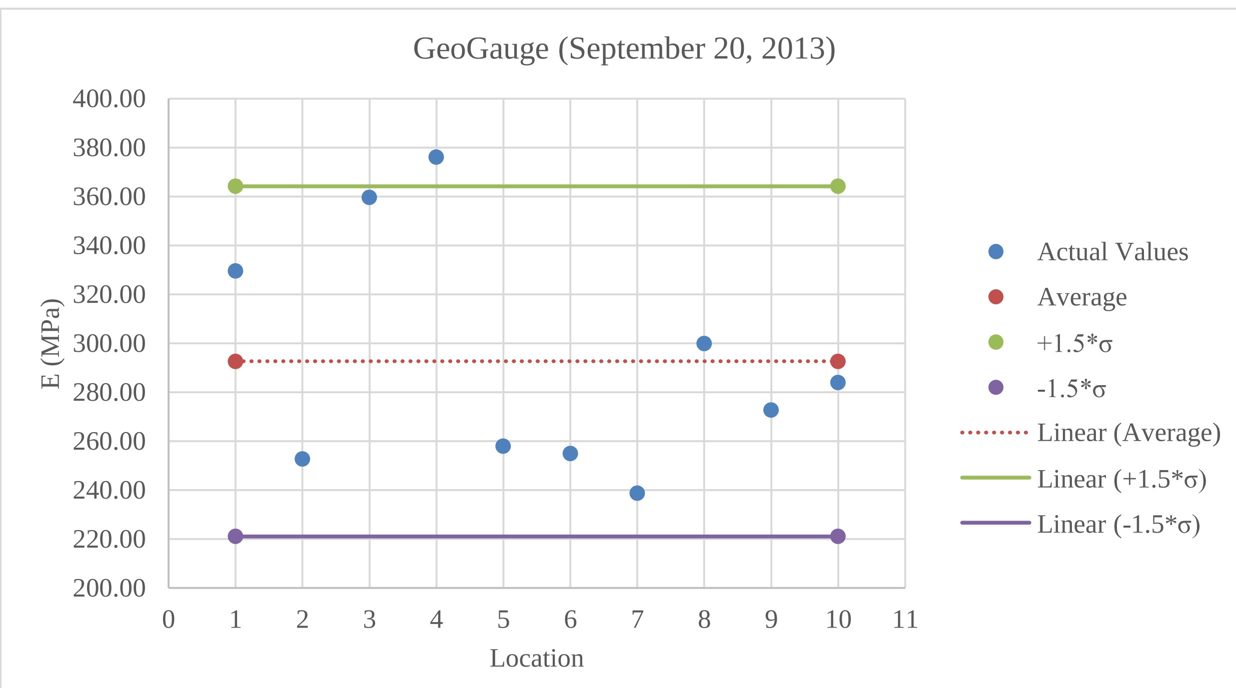

Figure C.38: Plot of GeoGauge values for limerock at 3% less than optimum moisture content

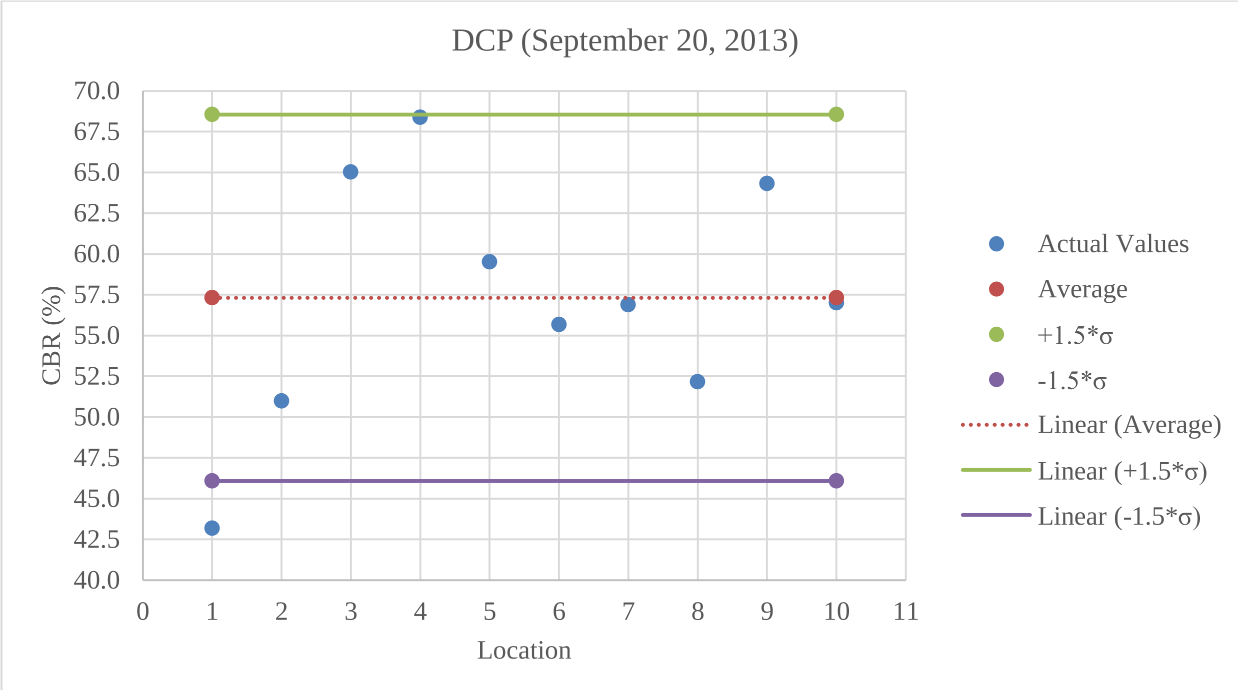

Figure C.39: Plot of DCP values for limerock at 3% less than optimum moisture content

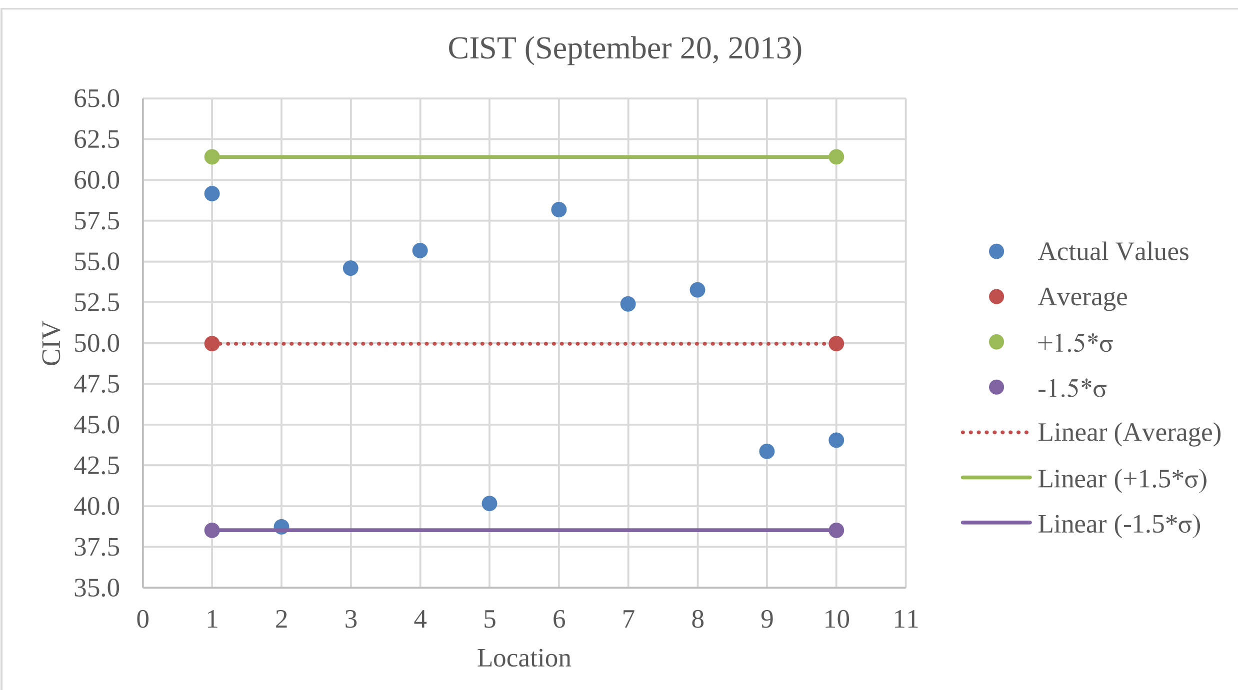

Figure C.40: Plot of CIST values for limerock at 3% less than optimum moisture content

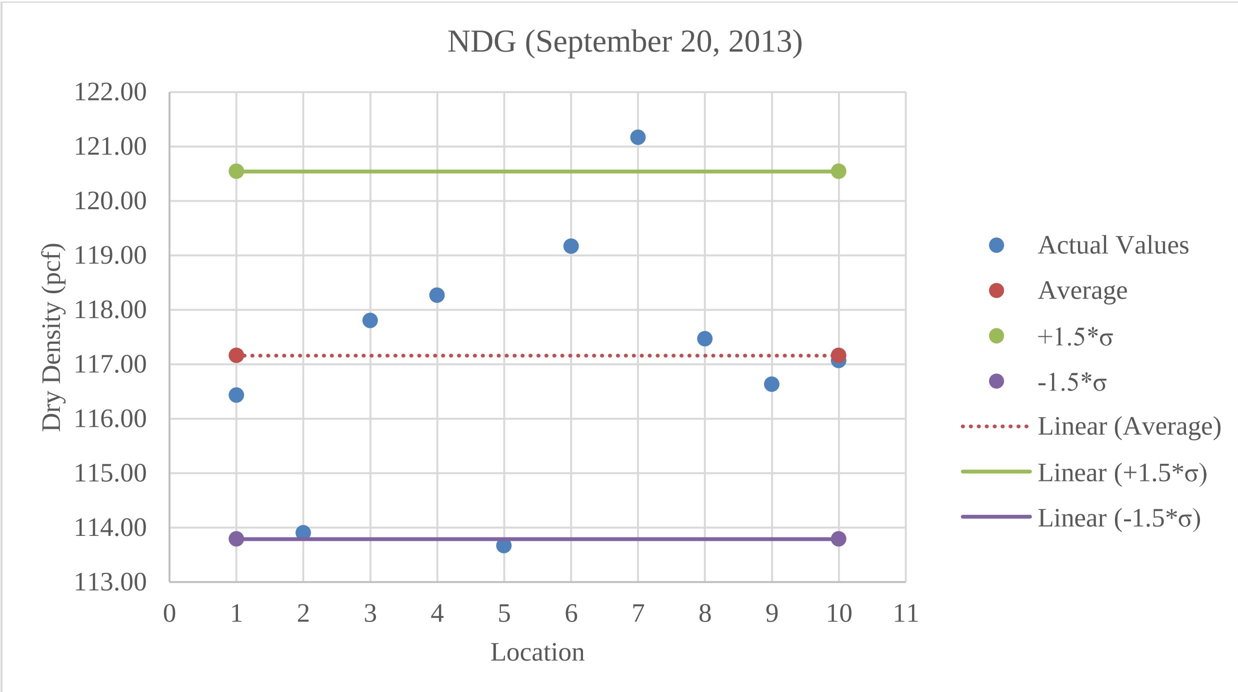

Figure C.41: Plot of NDG values for limerock at 3% less than optimum moisture content

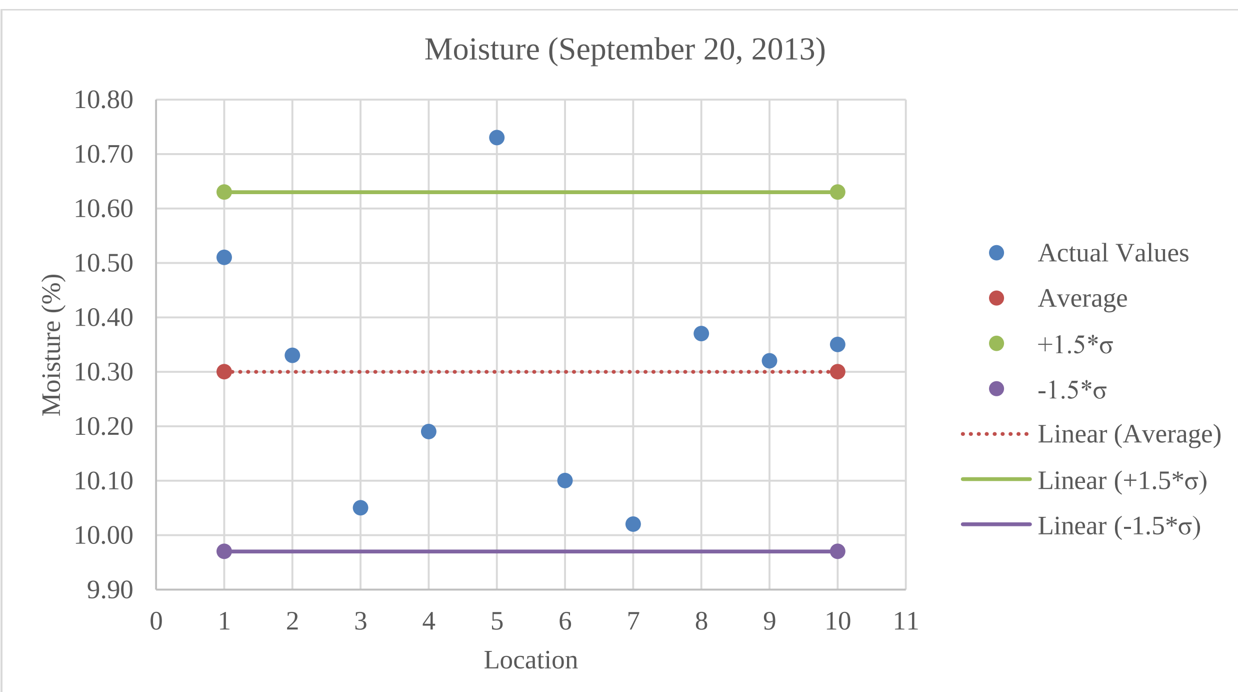

Figure C.42: Plot of moisture values for limerock

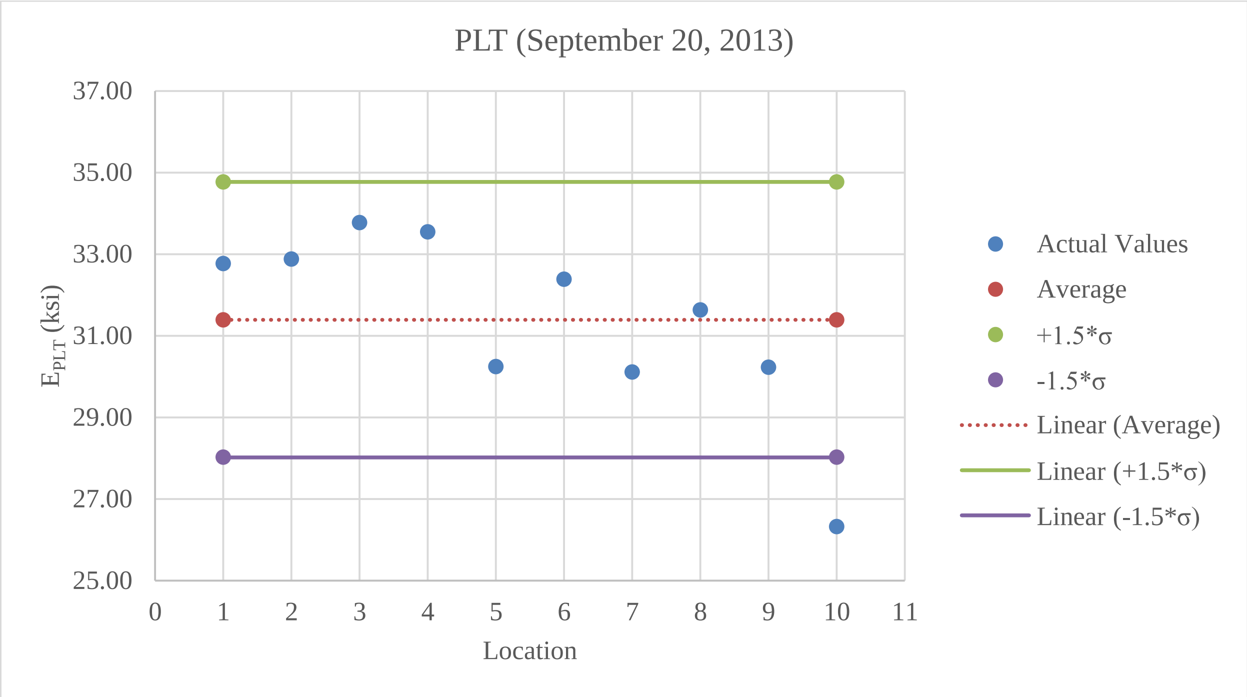

Figure C.43: Plot of PLT vales for limerock at 3% less than optimum moisture content

C.7 Results for Devices with Stabilized Subgrade Material at Optimum Moisture Content

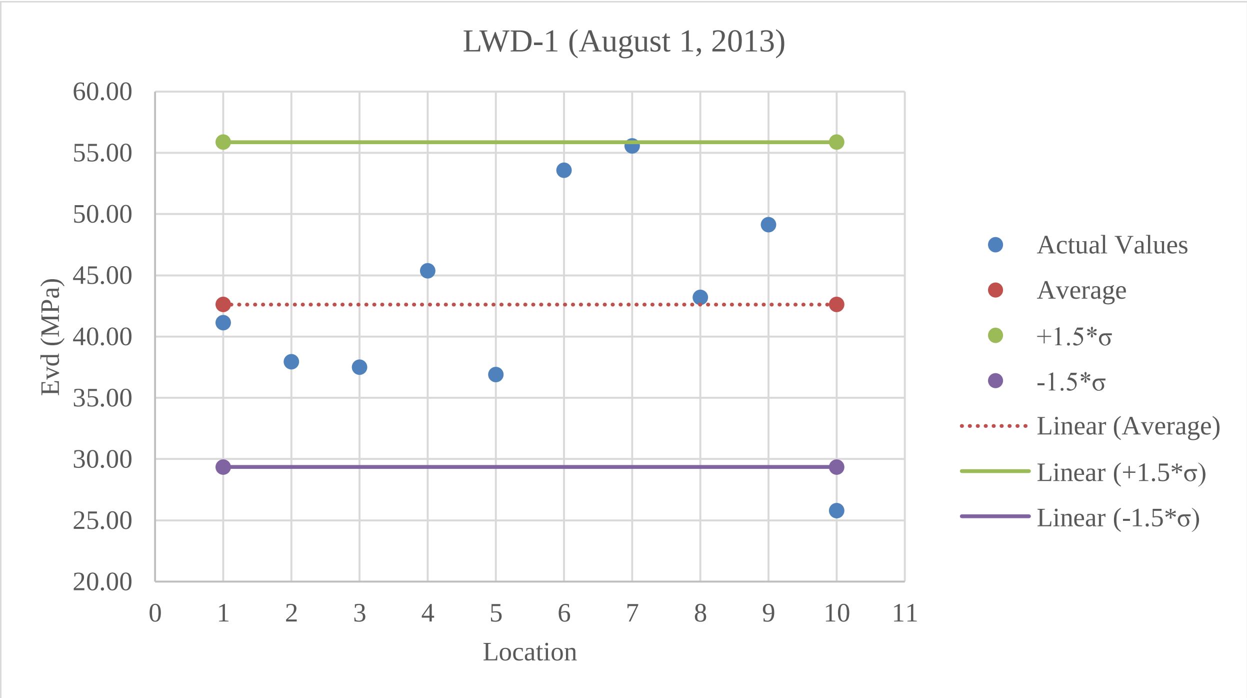

Figure C.44: Plot of LWD-1 values for stabilized subgrade at optimum moisture content

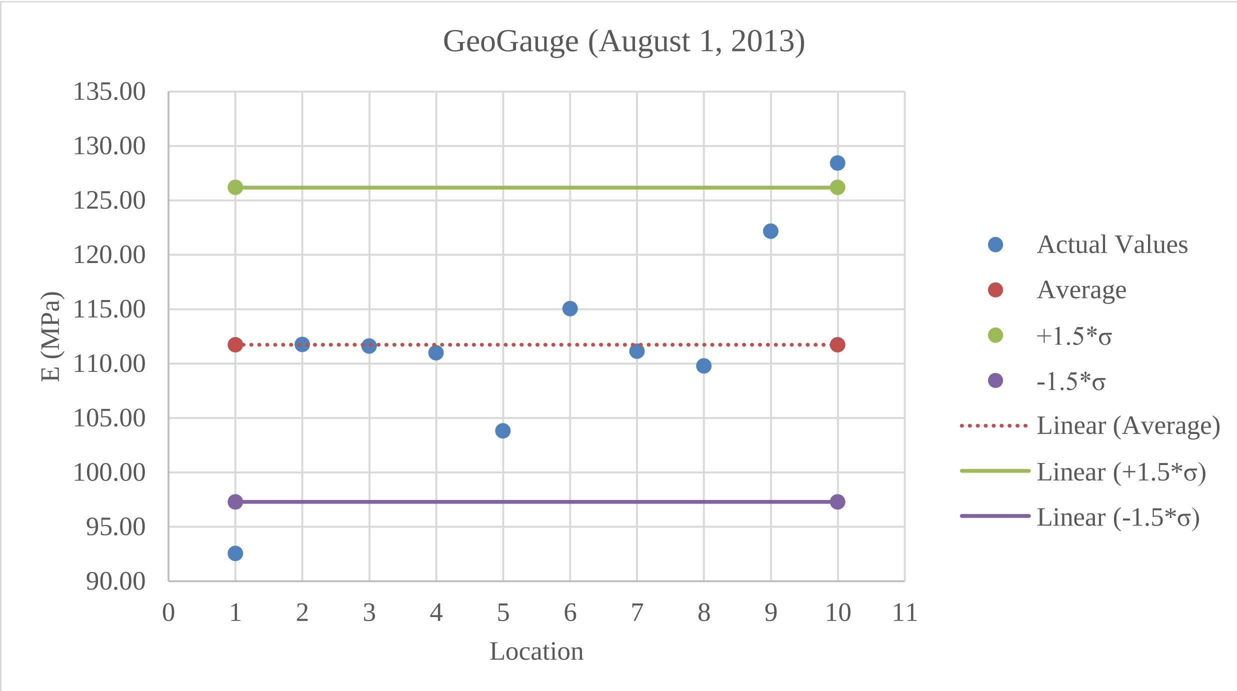

Figure C.45: Plot of GeoGauge values for stabilized subgrade at optimum moisture content

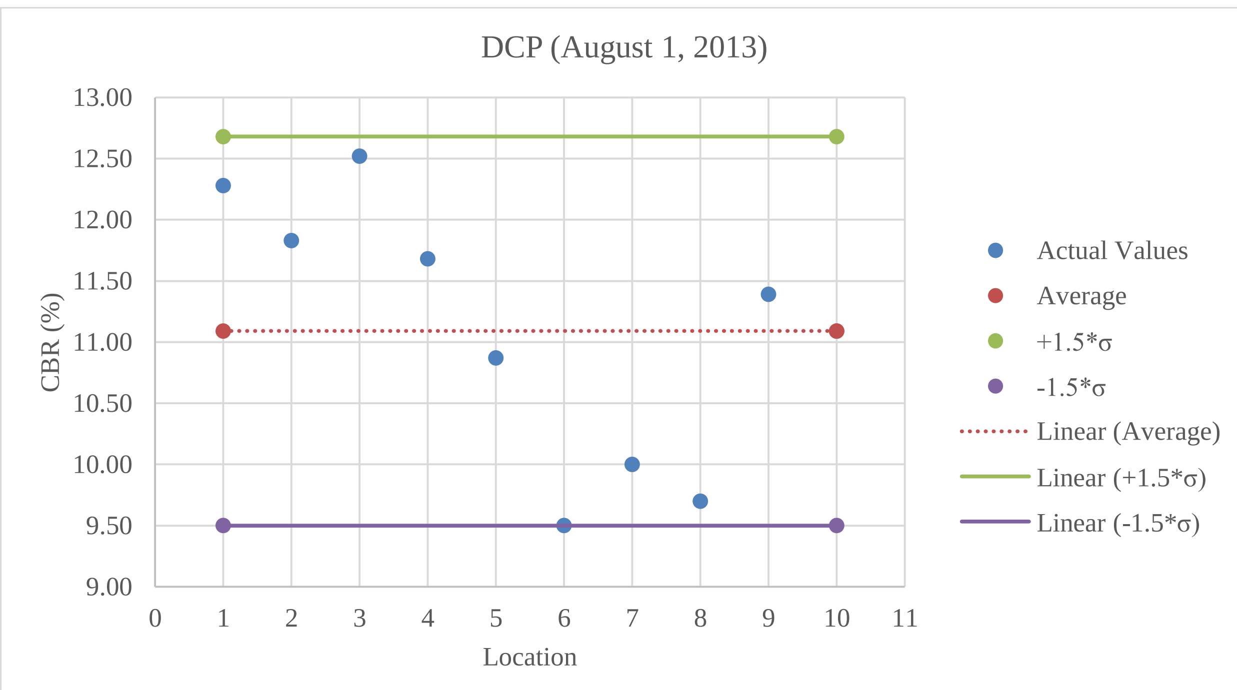

Figure C.46: Plot of DCP values for stabilized subgrade at optimum moisture content

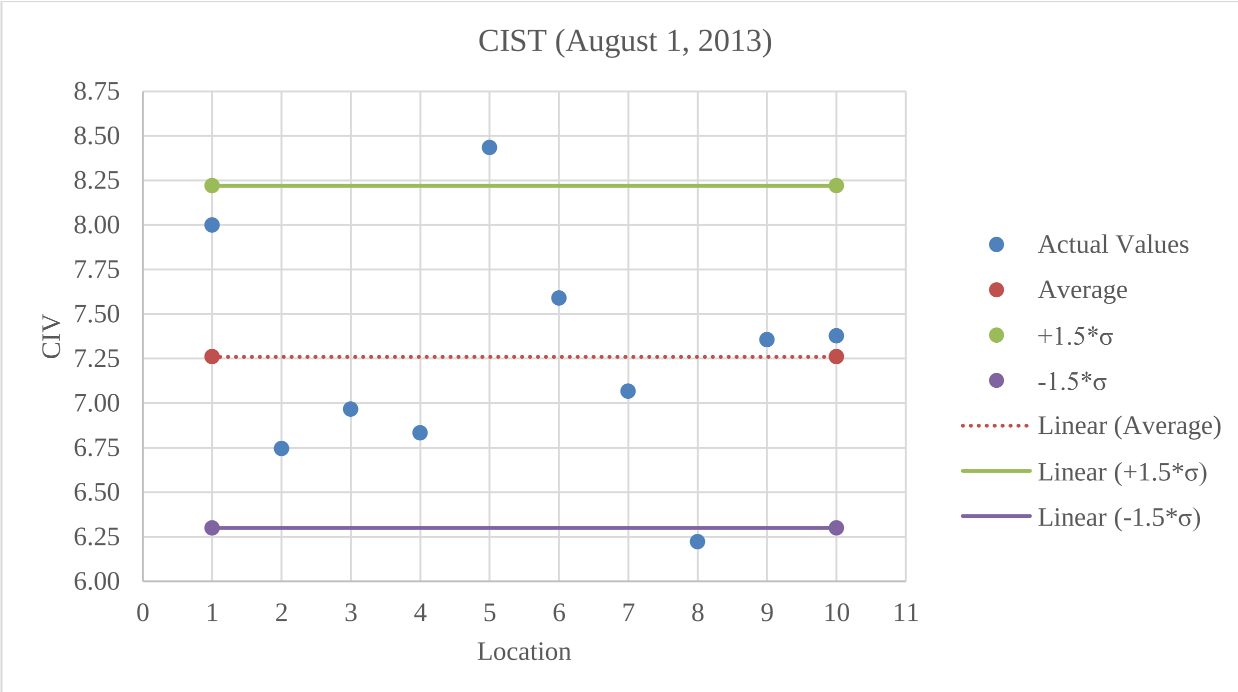

Figure C.47: Plot of CIST values for stabilized subgrade at optimum moisture content

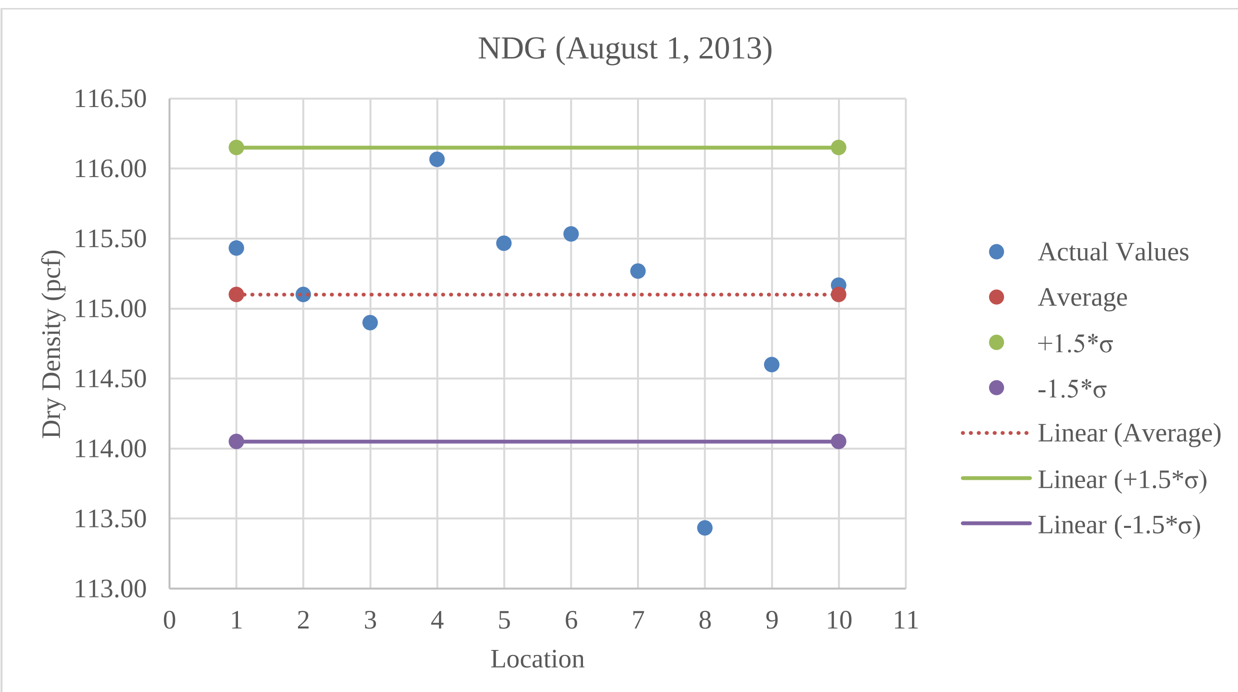

Figure C.48: Plot of NDG values for stabilized subgrade at optimum moisture content

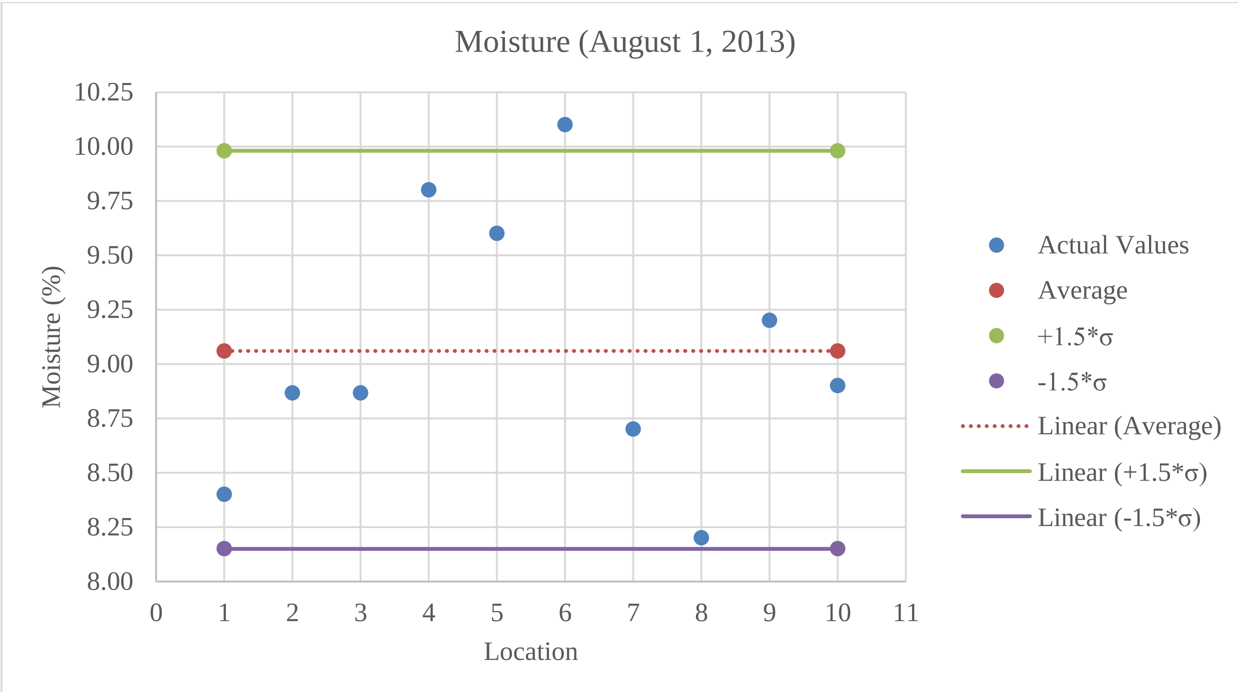

Figure C.49: Plot of moisture values for stabilized subgrade

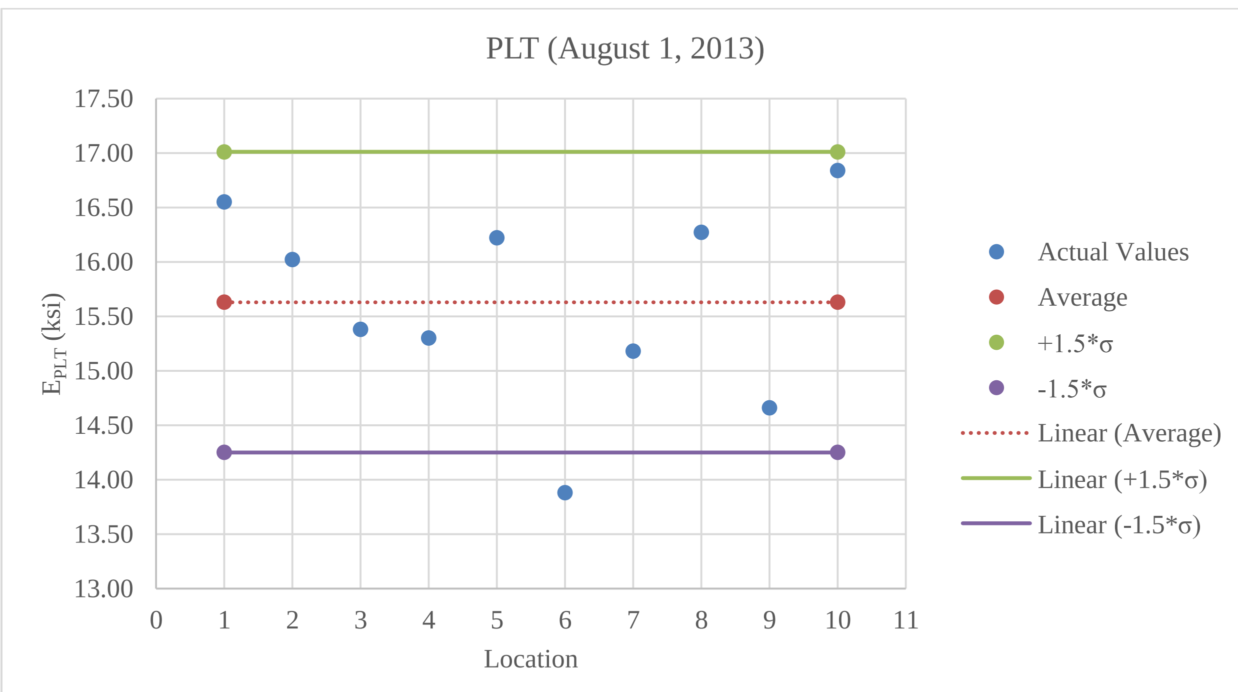

Figure C.50: Plot of PLT values for stabilized subgrade at optimum moisture content

C.8 Results for Devices with Stabilized Subgrade Material below Optimum Moisture Content

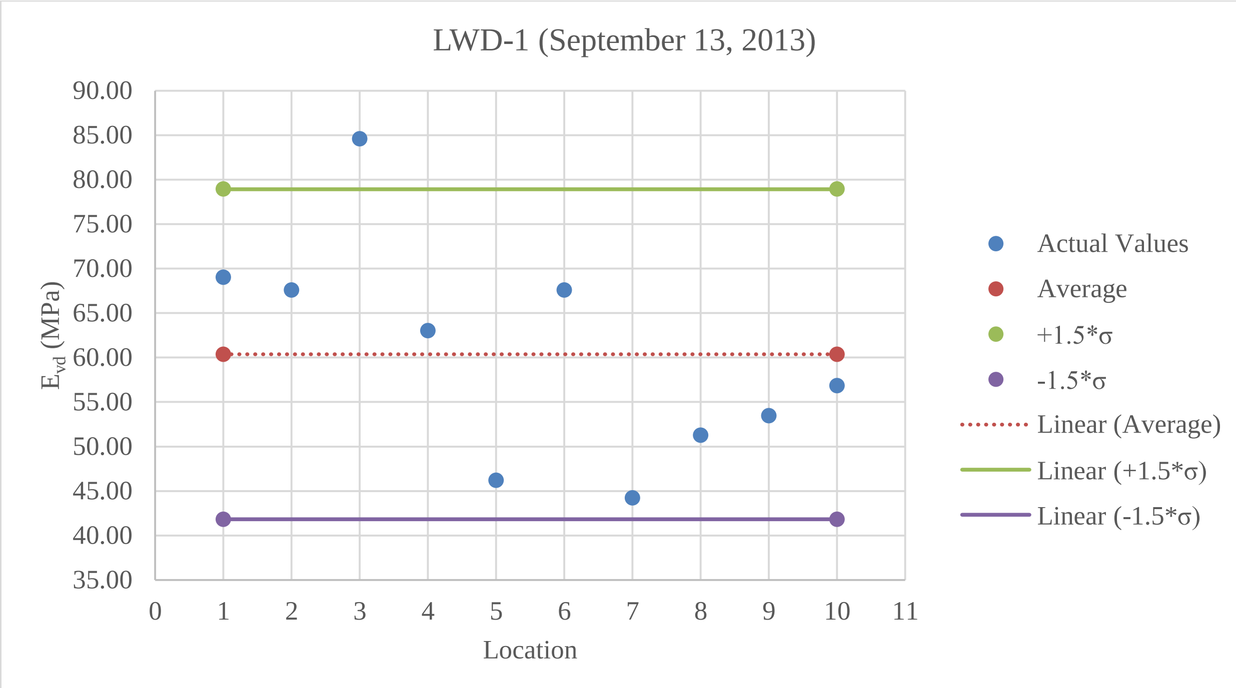

Figure C.51: Plot of LWD-1 values for stabilized subgrade below optimum moisture content

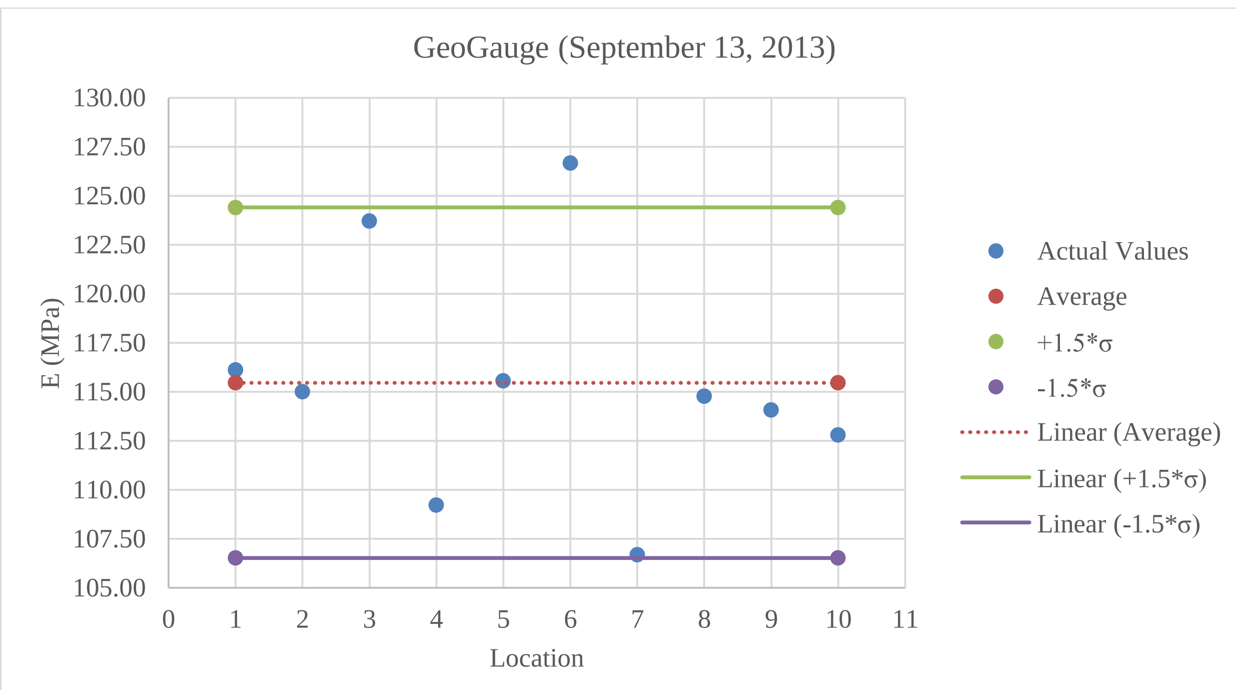

Figure C.52: Plot of GeoGauge values for stabilized subgrade below optimum moisture content

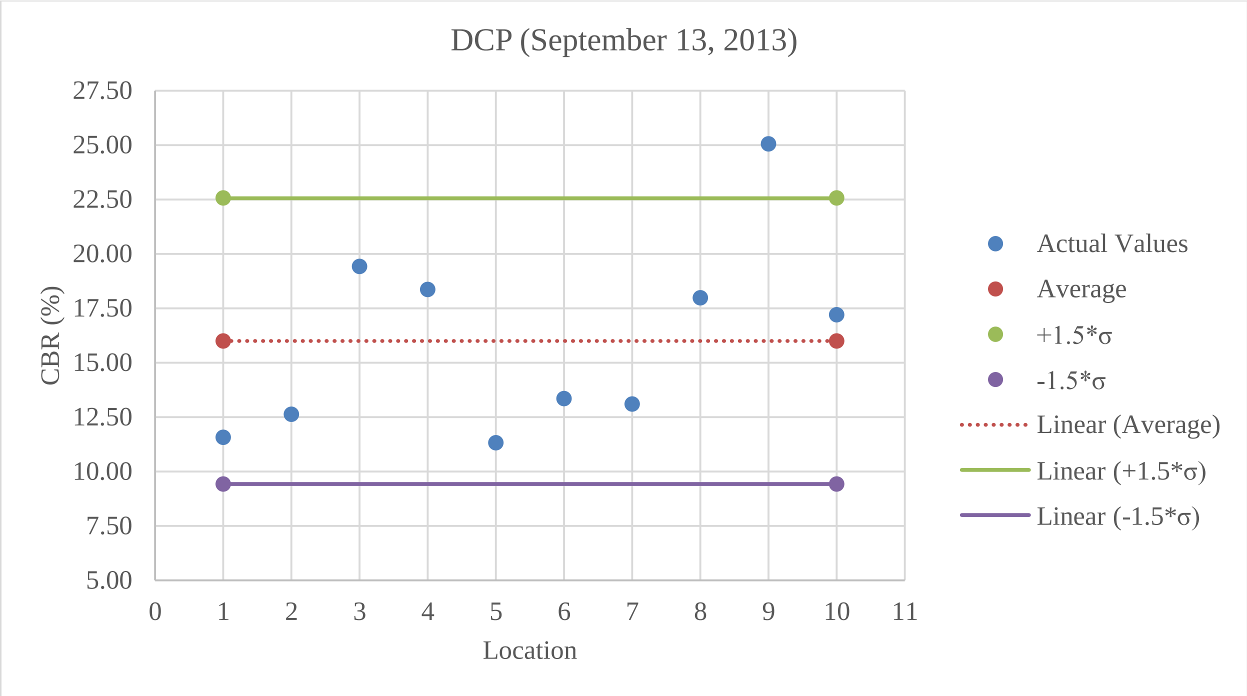

Figure C.53: Plot of DCP values for stabilized subgrade below optimum moisture content

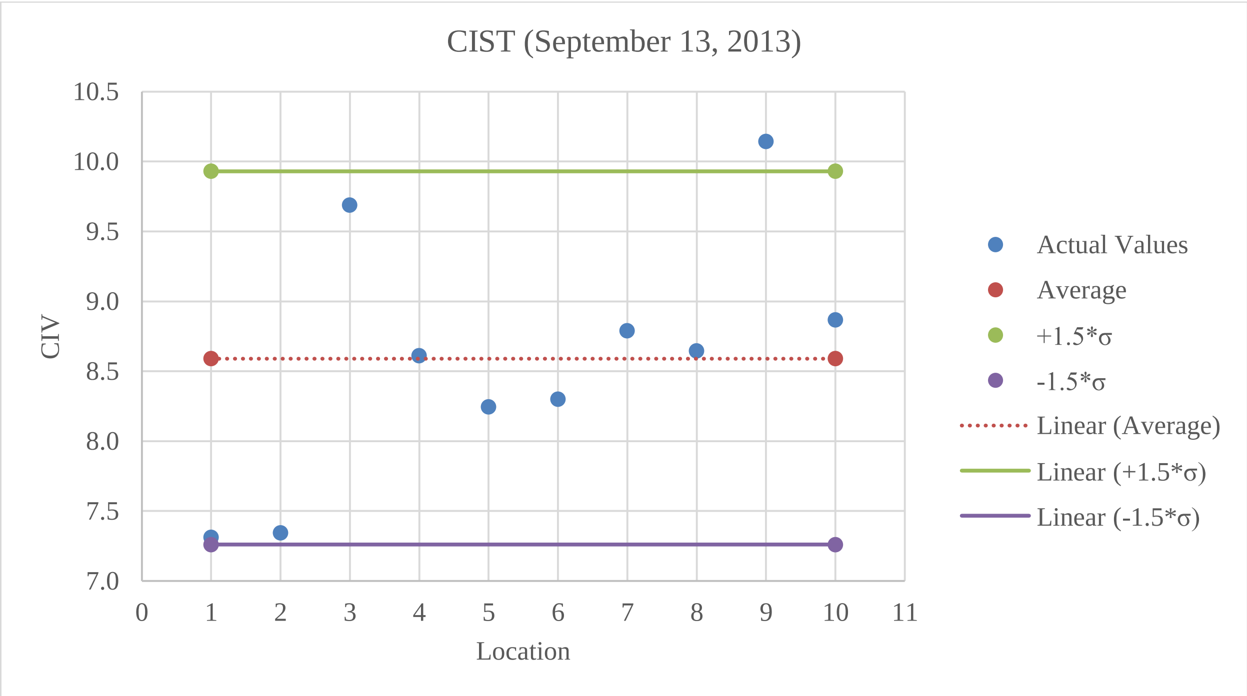

Figure C.54: Plot of CIST values for stabilized subgrade below optimum moisture content

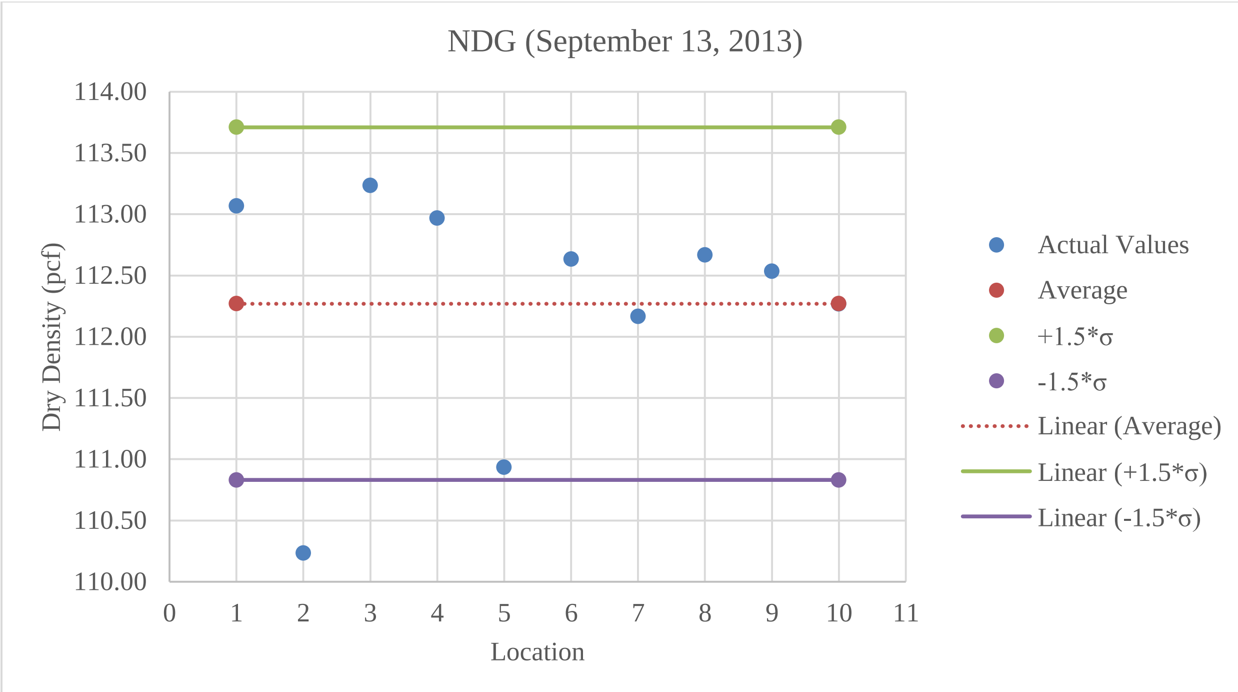

Figure C.55: Plot of NDG values for stabilized subgrade below optimum moisture content

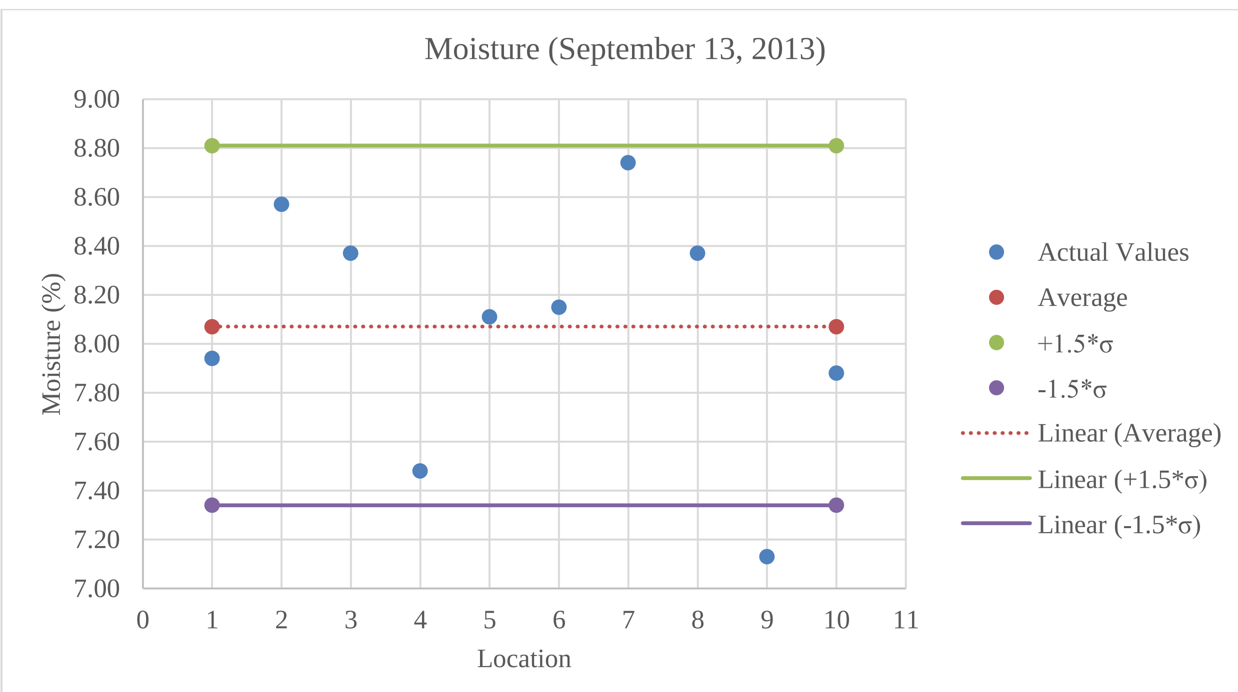

Figure C.56: Plot of moisture values for stabilized subgrade

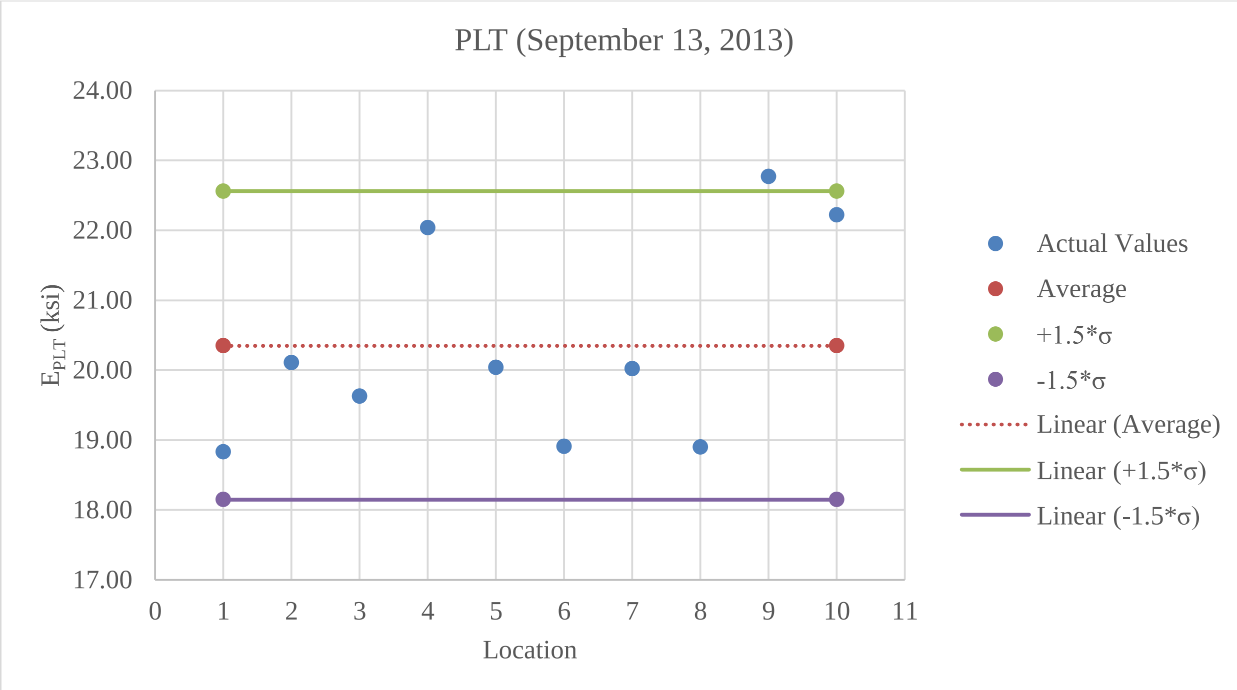

Figure C.57: Plot of PLT values for stabilized subgrade below optimum moisture content

C.9 Correlation between Each Pair of Results

Table C.1: Correlation values for A-3 near optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	NDG	Moisture
LWD-1	1.000	-0.434	0.207	-0.161	-0.416	-0.732
GeoGauge	-0.434	1.000	-0.075	-0.072	0.457	0.643
DCP	0.207	-0.075	1.000	-0.336	0.025	-0.429
PLT	-0.161	-0.072	-0.336	1.000	-0.179	-0.025
NDG	-0.416	0.457	0.025	-0.179	1.000	0.296
Moisture	-0.732	0.643	-0.429	-0.025	0.296	1.000

Table C.2: Correlation values for A-3 below optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	NDG	Moisture
LWD-1	1.000	0.331	-0.226	0.431	0.657	-0.043
GeoGauge	0.331	1.000	0.483	0.111	0.448	0.405
DCP	-0.226	0.483	1.000	-0.321	0.091	-0.136
PLT	0.431	0.111	-0.321	1.000	0.340	0.244
NDG	0.657	0.448	0.091	0.340	1.000	0.141
Moisture	-0.043	0.405	-0.136	0.244	0.141	1.000

Table C.3: Correlation values for A-2-4 low fines near optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	NDG	Moisture
LWD-1	1.000	0.074	-0.314	0.098	0.621	0.189
GeoGauge	0.074	1.000	0.171	-0.055	0.087	0.547
DCP	-0.314	0.171	1.000	0.191	0.013	-0.146
PLT	0.098	-0.055	0.191	1.000	-0.099	-0.254
NDG	0.621	0.087	0.013	-0.099	1.000	0.171
Moisture	0.189	0.547	-0.146	-0.254	0.171	1.000

Table C.4: Correlation values for A-2-4 low fines below optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	NDG	Moisture	LWD-2
LWD-1	1.000	0.012	-0.009	-0.535	0.755	0.527	0.503
GeoGauge	0.012	1.000	-0.324	0.070	0.123	0.416	0.066
DCP	-0.009	-0.324	1.000	-0.197	-0.267	-0.390	-0.141
PLT	-0.535	0.070	-0.197	1.000	-0.459	-0.096	-0.298
NDG	0.755	0.123	-0.267	-0.459	1.000	0.390	0.430
Moisture	0.527	0.416	-0.390	-0.096	0.390	1.000	0.090
LWD-2	0.503	0.066	-0.141	-0.298	0.430	0.090	1.000

Table C.5: Correlation values for limerock near optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	Moisture	NDG
LWD-1	1.000	0.351	-0.494	0.460	0.464	0.505
GeoGauge	0.351	1.000	0.185	0.172	-0.114	0.133
DCP	-0.494	0.185	1.000	-0.415	-0.444	-0.585
PLT	0.460	0.172	-0.415	1.000	0.345	0.247
Moisture	0.464	-0.114	-0.444	0.345	1.000	0.484
NDG	0.505	0.133	-0.585	0.247	0.484	1.000

Table C.6: Correlation values for limerock below optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	Moisture	NDG
LWD-1	1.000	0.095	0.170	0.631	-0.496	0.462
GeoGauge	0.095	1.000	0.292	0.488	-0.128	0.085
DCP	0.170	0.292	1.000	-0.003	-0.351	0.200
PLT	0.631	0.488	-0.003	1.000	-0.247	0.002
Moisture	-0.496	-0.128	-0.351	-0.247	1.000	-0.803
NDG	0.462	0.085	0.200	0.002	-0.803	1.000

Table C.7: Correlation values for stabilized subgrade near optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	Moisture	NDG
LWD-1	1.000	-0.148	-0.474	-0.786	0.221	0.047
GeoGauge	-0.148	1.000	-0.222	-0.235	0.003	-0.156
DCP	-0.474	-0.222	1.000	0.318	-0.121	0.282
PLT	-0.786	-0.235	0.318	1.000	-0.298	-0.185
Moisture	0.221	0.003	-0.121	-0.298	1.000	-0.039
NDG	0.047	-0.156	0.282	-0.185	-0.039	1.000

Table C.8: Correlation values for stabilized subgrade below optimum moisture.

R - Values

	LWD-1	GeoGauge	DCP	PLT	Moisture	NDG
LWD-1	1.000	0.651	0.073	-0.236	0.033	0.328
GeoGauge	0.651	1.000	-0.042	-0.482	0.094	0.179
DCP	0.073	-0.042	1.000	0.630	-0.617	0.449
PLT	-0.236	-0.482	0.630	1.000	-0.700	-0.020
Moisture	0.033	0.094	-0.617	-0.700	1.000	-0.344
NDG	0.328	0.179	0.449	-0.020	-0.344	1.000

Appendix D: Phase III Field Test Data

D.1 Field Test Information

D.1.1 Site Selection

On November 21, 2013, Jose Hernando with the FDOT State Materials Office (SMO), Jesse Sutton with the FDOT District Two Materials Office, and Wayne Rilko with UF inspected each of the five project sites chosen for field testing. Arrangements were made to meet with Consultant Engineering and Inspection (CEI) representatives for project access and scheduling with the contractors.

D.1.2 Site Information

Information pertaining to the five project sites is provided as follows. The projects are listed in the order tested.

1. I-95 (SR-9) widening/overland bridges, Jacksonville. Graded aggregate base (GAB), Specification 204, is used for base. Six-in.-thick GAB is under asphalt base, which is directly under the concrete pavement. (Ten-in.-thick GAB is used for base under the structural asphalt and asphalt friction courses on the side streets.) Testing on the GAB, Ramp H Northbound Collector Distributor, was conducted on December 18 and 19, 2013.

2. US-301 (SR-200) north of I-10. Seventeen miles, new lanes, from two to four. The project site is located in the northern section of the project, south of Callahan and southeast of the railroad bridge overpass. Embankment construction is in accordance with Specification 120. Testing on the A-2-4 embankment material used to construct the service road east of the northbound lanes south of the overpass was conducted on January 23 and 25, 2014.

3. SR-9B (I-295) new construction. Embankment placement, Specification 120. The project site is located north of the US-1 overpass (MSE wall and bridge.) Testing on the first lift of A-3 embankment was conducted on April 5, 12, and 14, 2014. (W. Rilko did intelligent compaction research on the previous project, the northern section of SR-9B, in May 2010. The embankment material was a yellow, very clean A-3. This fill differs in color, ranging from light to dark brown.)

4. CR-210 at US-1. This project is south of the SR-9B project. Although shell-rock was used for stabilized subgrade and base throughout the majority of the project, limited sections of limerock stabilized subgrade, Specification 160, were available for testing. Testing of the stabilized subgrade, the southbound lanes north of the US-1 overpass was conducted on May 29, 2014.

5. SR-23. New lanes, from two to four and numerous overpasses at existing intersections. Limerock base, Specification 200, used to construct a temporary turn lane in the southeast quadrant of the intersection of SR-23 and Normandy Boulevard was tested on July 8 and 9, 2014.

Permanent limerock base placement, however, was not underway in time to be included in this project.

6. SR-23. A-3 embankment, Specification 120, used for the construction of new lanes at the intersection of SR-23 and New World Avenue (northbound, south of New World) was tested on July 14 and 17, 2014.

D.2 Project Site Number 1 I-95

On December 10, 2013, Wayne Rilko met with CEI personnel assigned to the first project selected for testing, SR-9 (I-95) Overland Bridge Replacement, Jacksonville, Florida, to confirm the details and scheduling of the proposed testing. On December 12, 2013, Jose Hernando visited the site to determine the logistics for mobilization of the Plate Load Testing (PLT) equipment.

On December 18, 2013, UF personnel initiated portable equipment testing. Testing concluded mid-morning the following day. Equipment technicians were Jay Patel, Luis Avila, and Wayne Rilko. Portable equipment included two lightweight deflectometers (LWD-1 and LWD-2), dynamic cone penetrometer (DCP), Clegg impact soil tester (CIST), and GeoGauge. Operational problems with one of the LWDs were encountered early, and a complete set of data was not attainable.

Also on December 18, 2013, FDOT SMO personnel used the eighteen-wheel semi tanker filled with water for ballast for the 12-in. diameter static PLTs. SMO personnel also ran an NDG to determine in-place densities and moistures. These results are included in Appendix D. SMO personnel included Todd Britton, Kyle Sheppard, Bruce Swidarski, and Dalton Stevens. Jose Hernando and Dino Jameson with the SMO were also on site.

D.2.1 Earthwork Density Reports

All Earthwork Density Reports are by either Quality Control or Verification personnel assigned to the project.

Florida Department of Transportation

675-020-27
Materials – 01/13

CONTRACTOR - QC DENSITY RECORDS
Earthwork Density Report

Page 3

Project FIN: 213304-3-52-01
Type of Const: Subgrade / BASE
FROM STATION 467+05 TO STATION 462+05

LOT No.	RF	Date	TIN	Gauge Serial No.	STD. Dns/Mst Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns/Mst Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens.	Status / Disp.
1-2		5/19/14	B65043866	34686	2214 / 673	133 / SB0010	1	465+55	20' Lt of ℄ NBCD	1/1	6"	1409 / 116	144.4	6.9	135.1	102	P/V

Disposition Letter Code: V – LOTs Verified by Verification Test R – LOTs Verified by Resolution Procedure N – Lots Not Verified and Resolution initiated

Figure D.1: I-95 quality control earthwork density report

Florida Department of Transportation

675-020-27
Materials – 01/13

CONTRACTOR - QC DENSITY RECORDS
Earthwork Density Report

Page 3

Project FIN: 213304-3-52-01
Type of Const: Subgrade BASE
FROM STATION 462+05 TO STATION 458+50

LOT No.	RF	Date	TIN	Gauge Serial No.	STD. Dns/Mst Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns/Mst Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens.	Status / Disp.
1-3		5/19/14	B65043866	34686	2214 / 673	137 / SB0019	1	460+15	25' Lt of ℄ NBCD	1/1	6"	1436 / 108	143.6	6.3	135.1	102	P/V

Disposition Letter Code: V – LOTs Verified by Verification Test R – LOTs Verified by Resolution Procedure N – Lots Not Verified and Resolution initiated

Figure D.2: I-95 quality control earthwork density report

FLORIDA DEPARTMENT OF TRANSPORTATION

675-020-28
Materials – 10/11

Verification - Earthwork Density Report

Project FIN: 213304-35201

Type of Const: BASE (CAB) (IJ LECU CS) MSE WALL

FROM STATION 467+02

TO STATION 473+50

Page

LOT No.	Date	TIN	Gauge Serial No.	STD. Dns./Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns./Mst. Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens	LOTs Accepted
1-4	1-30-14	W10047571	26883	2057 / 641	113 / CB003Q	1	468+40	40' L CONST NBCP	1/1	6"	1765 / 191	127.3	12.1	113.6	101	1-4, 1-5

Figure D.3: I-95 verification earthwork density report

FLORIDA DEPARTMENT OF TRANSPORTATION

675-020-28
Materials – 10/11

Verification - Earthwork Density Report

Project FIN: 213504-35201

Type of Const: SUBGRADE / BASE

FROM STATION 467+05

TO STATION 462+05

Page 2

LOT No.	Date	TIN	Gauge Serial No.	STD. Dns./Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns./Mst. Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens	LOTs Accepted
1-2	5-19-14	W10047571	26883	2047 / 641	133 / SB001Q	1	463+45	24' L B/L NBCP	1/1	6"	1193 / 115	144.8	7.2	135.1	102	1-2, 1-5

Figure D.4: I-95 verification earthwork density report

D.2.2 Nuclear Density Gauge Data

All nuclear density gauge data is by FDOT SMO personnel assigned to the project.

Table D.1: I-95 NDG densities and moistures

Phase 3 Field Densities				
19-Dec-13				
Site #1				
I-95 US 1 Southbound Exit				
Graded Aggregate Base				
	Orientation	Dry Density (pcf)	Wet Density (pcf)	Moisture (%)
PLT #1 (South)	South	122.4	111.5	9.8
		123.8	113.2	9.3
	Average	123.1	112.4	9.6
	North	120.3	109.9	9.5
		126.9	114.7	10.6
	Average	123.6	112.3	10.1
	North Site Average	123.4	112.3	9.8
PLT #2 (Center)	South	126.2	114.1	10.6
		123.0	112.5	9.4
	Average	124.6	113.3	10.0
	North	122.9	111.8	9.9
		118.6	107.6	10.2
	Average	120.8	109.7	10.1
	Center Site Average	122.7	111.5	10.0
PLT #1 (North)	South	124.2	114.2	8.8
		119.5	109.6	9.1
	Average	121.9	111.9	9.0
	North	116.2	107.3	8.4
		122.9	113.5	8.2
	Average	119.6	110.4	8.3
	North Site Average	120.7	111.2	8.6

D.2.3 Plate Load Test Data

All plate load test data is by FDOT SMO personnel assigned to the project.

Table D.2: I-95 PLT #1

PLATE BEARING TEST
STATIC LOADING

Project: US1 GAB
Station or Lab No.:
Layer Tested: base
Thickness:
Description: GAB

Test No.: 1
Plate Diameter: 12
Degree Saturation:
Date: 12/18/2013
Tested By:

Max. Density (AASHO T-)

Optimum Moisture (AASHO T-)

LBR
At Optimum
Soaked

Dry Density @ Test: pcf

Moisture @ Test: %

p = Stress @ 0.05" Defl.
= 126.5 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05
= 17917.8 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	757	6.69	3	0.007	0.005	0.005	0.006
	986	8.72	3	0.010	0.007	0.008	0.008
	1354	11.97	3	0.013	0.012	0.014	0.013
	1919	16.97	3	0.017	0.016	0.018	0.017
	2452	21.68	3	0.021	0.020	0.023	0.021
	4264	37.70	3	0.030	0.031	0.035	0.032
	0	0.00	3	0.017	0.020	0.021	0.020
Seating Load #2	1066	9.43	3	0.006	0.005	0.004	0.005
	2132	18.85	3	0.011	0.011	0.008	0.010
	3198	28.28	3	0.015	0.015	0.012	0.014
	4264	37.70	3	0.019	0.018	0.016	0.018
	5330	47.13	3	0.024	0.023	0.021	0.023
	7036	62.21	3	0.032	0.032	0.030	0.031
	0	0.00	3	0.012	0.015	0.012	0.013
Seating Load #3	1599	14.14	3	0.007	0.007	0.005	0.006
	3198	28.28	3	0.013	0.012	0.010	0.012
	4797	42.41	3	0.017	0.016	0.014	0.016
	6369	56.31	3	0.021	0.020	0.018	0.020
	7995	70.69	3	0.028	0.027	0.024	0.026
	9594	84.83	3	0.034	0.033	0.031	0.033
	0	0.00	3	0.008	0.010	0.008	0.009
Final Load	2132	18.85	3	0.010	0.009	0.008	0.009
	4264	37.70	3	0.016	0.015	0.013	0.015
	6396	56.55	3	0.021	0.021	0.018	0.020
	8528	75.40	3	0.026	0.026	0.025	0.026
	10660	94.26	3	0.034	0.034	0.031	0.033
	12792	113.11	3	0.042	0.041	0.039	0.041
	15457	136.67	3	0.056	0.052	0.050	0.053
	0	0.00	3	0.017	0.019	0.017	0.018

Table D.3: I-95 PLT #2

PLATE BEARING TEST
STATIC LOADING

Project: US1 GAB
Station or Lab No.:
Layer Tested: base
Thickness:
Description: GAB

Test No.: 2
Plate Diameter: 12
Degree Saturation:
Date: 12/18/2013
Tested By:

Max. Density
(AASHO T- ______)

Optimum Moisture
(AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test:
______ pcf

Moisture @ Test:
______ %

p = Stress @ 0.05" Defl.
= 148.0 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05
= 20958.5 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	853	7.54	3	0.007	0.009	0.003	0.006
	1599	14.14	3	0.010	0.010	0.005	0.008
	2665	23.56	3	0.002	0.006	0.012	0.007
	4264	37.70	3	0.004	0.001	0.023	0.009
	6396	56.55	3	0.014	0.010	0.036	0.020
	8528	75.40	3	0.023	0.020	0.048	0.030
	0	0.00	3	0.007	0.006	0.023	0.012
Seating Load #2	2132	18.85	3	0.004	0.003	0.010	0.006
	4264	37.70	3	0.008	0.008	0.017	0.011
	6396	56.55	3	0.013	0.013	0.023	0.016
	8528	75.40	3	0.019	0.019	0.030	0.023
	10660	94.26	3	0.028	0.028	0.039	0.032
	0	0.00	3	0.007	0.009	0.010	0.009
Seating Load #3	2665	23.56	3	0.005	0.004	0.012	0.007
	5330	47.13	3	0.010	0.010	0.197	0.072
	7995	70.69	3	0.016	0.016	0.027	0.020
	10660	94.26	3	0.021	0.021	0.036	0.026
	12259	108.39	3	0.026	0.027	0.043	0.032
	0	0.00	3	0.006	0.005	0.009	0.007
Final Load	3198	28.28	3	0.005	0.006	0.015	0.008
	6396	56.55	3	0.012	0.013	0.024	0.016
	9594	84.83	3	0.018	0.019	0.032	0.023
	12792	113.11	3	0.026	0.027	0.042	0.032
	15990	141.38	3	0.037	0.038	0.055	0.044
	18122	160.23	3	0.045	0.047	0.065	0.052
	0	0.00	3	0.014	0.015	0.020	0.016

Table D.4: I-95 PLT #3

PLATE BEARING TEST
STATIC LOADING

Project: US1 GAB
Station or Lab No.:
Layer Tested: base
Thickness:
Description: GAB

Test No.: 3
Plate Diameter: 12
Degree Saturation:
Date: 12/18/2013
Tested By:

Max. Density
(AASHO T-)

Optimum Moisture
(AASHO T-)

LBR
At Optimum
Soaked

Dry Density @ Test:
pcf

Moisture @ Test:
%

p = Stress @ 0.05" Defl.
= 92.0 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 13032.7 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	853	7.54	3	0.009	0.003	0.011	0.008
	1599	14.14	3	0.018	0.008	0.019	0.015
	2665	23.56	3	0.028	0.016	0.030	0.024
	3198	28.28	3	0.031	0.018	0.033	0.027
	3731	32.99	3	0.036	0.022	0.038	0.032
	0	0.00	3	0.026	0.018	0.025	0.023
Seating Load #2	1066	9.43	3	0.002	0.003	0.006	0.004
	2132	18.85	3	0.006	0.008	0.010	0.008
	3198	28.28	3	0.009	0.014	0.016	0.013
	4264	37.70	3	0.014	0.023	0.022	0.020
	5330	47.13	3	0.019	0.031	0.025	0.025
	6396	56.55	3	0.023	0.036	0.034	0.031
	0	0.00	3	0.009	0.023	0.014	0.016
Seating Load #3	1599	14.14	3	0.004	0.006	0.009	0.006
	3198	28.28	3	0.007	0.010	0.015	0.011
	4797	42.41	3	0.012	0.015	0.021	0.016
	6396	56.55	3	0.130	0.019	0.028	0.059
	7995	70.69	3	0.224	0.026	0.036	0.095
	8795	77.76	3	0.027	0.029	0.040	0.032
	0	0.00		0.008	0.014	0.014	0.012
Final Load	2132	18.85	3	0.005	0.005	0.012	0.007
	4264	37.70	3	0.009	0.010	0.019	0.012
	6396	56.55	3	0.013	0.014	0.024	0.017
	8528	75.40	3	0.020	0.020	0.033	0.024
	10660	94.26	3	0.027	0.305	0.042	0.124
	12792	113.11	3	0.035	0.039	0.050	0.042
	14924	131.96	3	0.047	0.051	0.062	0.053
	0	0.00	3	0.016	0.021	0.022	0.020

D.2.4 Laboratory Data

All laboratory data unless otherwise noted is by FDOT SMO personnel assigned to the project.

Table D.5: I-95 laboratory data

STATE MATERIALS OFFICE
UF Research, I-95
Graded Aggregate Base Material
Mr Summary

Sample Number	Station	Offset	Depth	LIMS Number	Proctor Dry Unit Mass (pcf)	Proctor Opt. Moisture (%)	M(r) Dry Unit Mass (pcf)	M(r) Opt. Moisture (%)	Mr @ bulk stress (θ) = 30 psi
1					110.5	14.4	109.6	13.5	20,182
							108.8	12.8	20,390
2					111.3	14.6	110.9	13.0	20,035
							111.3	12.9	22,059
3					111.3	14.7	109.5	12.8	19,686
							111.2	12.4	19,541

** θ = sum of principal stresses, $\sigma 1 + \sigma 2 + \sigma 3$

D.3 Project Site Number 2 US-301

The location of the second project was US-301 (SR-200), south of Callahan, Florida. A service road southeast of railroad bridge overpass, east of the northbound travel lanes was the test site. Embankment fill material was placed 4 to 5 ft. thick. The material, brown to dark brown sand with wood fragments and pieces of white rock, was classified as A-2-4 soil.

Numerous attempts (January 3, 10, and 17, 2014) were scheduled for testing but were postponed due to rain and/or wet conditions. The project site was inspected by Wayne Rilko on January 9 and 23, 2014. PLTs were conducted by FDOT SMO personnel Todd Britton, Bruce Swidarski, Travis Stevens, Thad Bryant, and Kyle Sheppard on January 23, 2014. Portable equipment was conducted by UF personnel Jay Patel, Luis Avila, and Wayne Rilko on January 25, 2014.

The temperature at the start of the portable equipment testing was 38˚F. Light showers gave way to a cloudy, cold, windy day. The lower temperatures did not appear to affect the operation of any of the testing devices.

D.3.1 Earthwork Density Reports

675-020-28
Materials – 02/10

Verification - Earthwork Density Report

Page 404

Project FIN: 429551-152-01
Type of Const: Embankment

FROM STATION 680+00 TO STATION 685+00 * 1-404 first lift out of wet conditions

LOT No.	Date	TIN	Gauge Serial No.	STD. Dns./Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns./Mst. Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens.	LOTs Accepted
1-404	11/28/12	DY0015373	39657	2766 / 693	108.0 / E031Q	1	681+90	20' RT € SR 200	2/9	6"	2907 / 169	122.3	12.2	109.0	101	1-404 *
2-404	12/5/12	#53079370	39591	2639 / 714	99.0 / E032Q	2	682+58	28' Rt € SR 200	3/9	6"	2942 / 227	119.5	15.6	103.4	104	2-404, 3-404, 4-404, 5-404, 6-404, 7-404 BG
7-404	03-29-13	H53079370	39591	2590 / 714	106 / E022Q	3	684+25	39' Rt € SR 200	8/2	6"	3117 / 163	116.0	9.4	106.0	100	8-404

Figure D.5: US-301 verification earthwork density report

675-020-27
Materials – 02/10

CONTRACTOR - QC DENSITY RECORDS
Earthwork Density Report

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Project FIN: 429551-1-52-01
Type of Const: EMBANKMENT

FROM STATION 680+00 TO STATION 685+00

LOT No.	RF	Date	TIN	Gauge Serial No.	STD. Dns./Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns./Mst Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens.	Status / Disp.
1-404		11-28-12	B30072867	16399	1685 / 633	108 / E031Q	1	681+48	27' R€	2/9	6"	1755 / 183	122.7	11.1	110.4	102	P/V BK
2-404		12-5-12	B30072867	16399	1691 / 641	99 / E032Q	2	683+14	38' R€	3/9	6"	1919 / 195	118.5	16.6	101.6	103	P/V
3-404		12-11-12	B30072867	16399	1675 / 643	99 / E032Q	3	684+30	32' R€	4/9	6"	2191 / 140	111.9	11.5	100.3	101	P/V
4-404		1-17-13	B30072867	16399	1675 / 643	99 / E032Q	4	683+07	28' R€	5/9	6"	2156 / 176	112.5	13.3	99.3	100	P/V
5-404		1-25-13	B30072867	16399/646	1667 ~~1666~~ / ~~642~~	99 / E032Q	5	680+85	34' R€	6/9	6"	2209 / 157	111.2	11.6	99.6	101	P/V BG
6-404		1-29-13	B30072867	16399	1672 / 639	101 / E021Q	6	682+47	30' R€	7/9	6"	1786 / 250	121.1	18.8	102.0	101	P/V
7-404		3-29-13	B30072867	529	2523 / 855	106 / E022Q	7	684+71	33' R€	8/9	6"	3130 / 195	119.3	10.6	108.0	102	P/V BP
8-404		10-14-13	B30072867	16399	1860 / 645	99 / E037Q	8	684+95	36' R€	9/9	6"	2599 / 112	107.0	7.6	99.4	100	P/V

Disposition Letter Code	V – LOTs Verified by Verification Test	R – LOTs Verified by Resolution Procedure	N – LOTs Not Verified and Resolution initiated

Figure D.6: US-301 quality control earthwork density report

D.3.2 Nuclear Density Gauge Data

Table D.6: US-301 NDG densities and moistures

Phase 3 Field Densities				
01.25.2014				
Site #2 US 301				
Southeast of railroad overpass				
A-2-4 embankment				
	Orientation	Wet Density (pcf)	Dry Density (pcf)	Moisture (%)
PLT #1 (South) STA 680	South	121.3	108.9	11.4
		119.9	106.0	13.1
	Average	120.6	107.5	12.3
	North	121.5	108.5	12.0
		119.9	106.4	12.6
	Average	120.7	107.5	12.3
	North Site Average	120.7	107.5	12.3
PLT #2 (Center) STA 682	South	122.4	106.9	14.4
		123.4	109.5	12.7
	Average	122.9	108.2	13.6
	North	120.3	105.9	13.7
		123.8	108.5	14.1
	Average	122.1	107.2	13.9
	Center Site Average	122.5	107.7	13.7
PLT #1 (North) STA 684	South	122.6	110.0	11.4
		123.0	109.3	12.5
	Average	122.8	109.7	12.0
	North	122.3	106.0	15.4
		122.5	108.3	13.1
	Average	122.4	107.2	14.3
	North Site Average	122.6	108.4	13.1

D.3.3 Plate Load Test Data

Table D.7: US-301 PLT #1
Station 680

PLATE BEARING TEST
STATIC LOADING

Project: US301 Bryceville
Station or Lab No.: Station 680
Layer Tested: embankment
Thickness:
Description: UF Research

Test No.: 1
Plate Diameter: 12
Degree Saturation:
Date:
Tested By: Todd,Bruce,Travis,Thad, & Kyle

Max. Density
(AASHO T- ____)

Optimum Moisture
(AASHO T- ____)

LBR
At Optimum ____
Soaked ____

Dry Density @ Test:
____ pcf

Moisture @ Test:
____ %

p = Stress @ 0.05" Defl.
= 33.4 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 4732.6 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	500	4.42	3	0.018	0.003	0.032	0.018
	600	5.31	3	0.024	0.008	0.038	0.023
	700	6.19	3	0.027	0.012	0.043	0.027
	800	7.07	3	0.032	0.016	0.048	0.032
	0	0.00	3	0.030	0.016	0.041	0.029
Seating Load #2	400	3.54	3	0.001	0.000	0.005	0.002
	840	7.43	3	0.007	0.000	0.016	0.007
	1200	10.61	3	0.017	0.005	0.029	0.017
	1600	14.15	3	0.033	0.019	0.047	0.033
	0	0.00	3	0.025	0.018	0.033	0.025
Seating Load #3	600	5.31	3	0.002	-0.001	0.008	0.003
	1200	10.61	3	0.006	-0.001	0.017	0.007
	1800	15.92	3	0.013	0.004	0.029	0.015
	2400	21.22	3	0.027	0.016	0.045	0.029
	2600	22.99	3	0.032	0.021	0.053	0.035
	0	0.00	3	0.021	0.010	0.029	0.020
Final Load	600	5.31	3	0.003	-0.001	0.008	0.003
	1200	10.61	3	0.006	-0.001	0.015	0.007
	2000	17.68	3	0.012	0.003	0.026	0.014
	2800	24.76	3	0.024	0.012	0.041	0.026
	3600	31.83	3	0.042	0.029	0.061	0.044
	4000	35.37	3	0.052	0.037	0.070	0.053
	0	0.00	3	0.031	0.026	0.038	0.032

Table D.8: US-301 PLT #2
Station 682

PLATE BEARING TEST
STATIC LOADING

Project: US301 Bryceville
Station or Lab No.: Station 682
Layer Tested: embankment
Thickness:
Description: UF Research

Test No.: 1
Plate Diameter: 12
Degree Saturation:
Date:
Tested By: Todd,Bruce,Travis,Thad, & Kyle

Max. Density (AASHO T-)

Optimum Moisture (AASHO T-)

LBR
At Optimum
Soaked

Dry Density @ Test: pcf

Moisture @ Test: %

p = Stress @ 0.05" Defl.
= 26.1 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 3698.6 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	300	2.65	3	0.018	-0.004	0.019	0.011
	500	4.42	3	0.032	-0.004	0.035	0.021
	700	6.19	3	0.041	-0.003	0.044	0.027
	800	7.07	3	0.046	0.002	0.049	0.033
	0	0.00	3	0.041	0.003	0.042	0.029
Seating Load #2	300	2.65	3	0.002	0.000	0.003	0.002
	500	4.42	3	0.005	0.000	0.006	0.004
	700	6.19	3	0.008	0.000	0.011	0.007
	1000	8.84	3	0.018	0.002	0.023	0.014
	1500	13.26	3	0.042	0.020	0.048	0.037
	0	0.00	3	0.031	0.019	0.034	0.028
Seating Load #3	400	3.54	3	0.004	0.001	0.003	0.003
	800	7.07	3	0.008	0.001	0.006	0.005
	1200	10.61	3	0.015	0.001	0.011	0.009
	1600	14.15	3	0.026	0.004	0.023	0.018
	2000	17.68	3	0.041	0.015	0.048	0.035
	0	0.00	3	0.028	0.011	0.034	0.024
Final Load	600	5.31	3	0.007	0.002	0.006	0.005
	1300	11.49	3	0.014	0.002	0.015	0.010
	2000	17.68	3	0.028	0.006	0.032	0.022
	2700	23.87	3	0.051	0.023	0.058	0.044
	3300	29.18	3	0.074	0.042	0.081	0.066
	0	0.00	3	0.046	0.031	0.048	0.042

Table D.9: US-301 PLT #3
Station 684

PLATE BEARING TEST
STATIC LOADING

Project: US301 Bryceville
Station or Lab No.: Station 684
Layer Tested: embankment
Thickness:
Description: UF Research

Test No.: 1
Plate Diameter: 12
Degree Saturation:
Date:
Tested By: Todd,Bruce,Travis,Thad, & Kyle

Max. Density
(AASHO T- ________)

Optimum Moisture
(AASHO T- ________)

LBR
At Optimum ________
Soaked ________

Dry Density @ Test:
________ pcf

Moisture @ Test:
________ %

p = Stress @ 0.05" Defl.
= 23.3 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 3302.5 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	300	2.65	3	0.004	0.000	0.003	0.002
	600	5.31	3	0.009	0.000	0.009	0.006
	900	7.96	3	0.021	0.007	0.018	0.015
	1200	10.61	3	0.041	0.020	0.039	0.033
	0	0.00	3	0.034	0.019	0.028	0.027
Seating Load #2	300	2.65	3	0.008	0.000	0.003	0.004
	600	5.31	3	0.009	0.000	0.008	0.006
	900	7.96	3	0.013	0.001	0.013	0.009
	1200	10.61	3	0.022	0.004	0.021	0.016
	1500	13.26	3	0.040	0.019	0.039	0.033
	0	0.00	3	0.031	0.015	0.026	0.024
Seating Load #3	400	3.54	3	0.001	0.002	0.004	0.002
	800	7.07	3	0.009	0.002	0.011	0.007
	1200	10.61	3	0.015	0.002	0.016	0.011
	1600	14.15	3	0.029	0.007	0.029	0.022
	2000	17.68	3	0.050	0.026	0.051	0.042
	0	0.00	3	0.035	0.017	0.033	0.028
Final Load	400	3.54	3	0.004	0.002	0.005	0.003
	900	7.96	3	0.011	0.002	0.012	0.008
	1200	10.61	3	0.015	0.002	0.015	0.010
	1600	14.15	3	0.022	0.000	0.022	0.015
	2200	19.45	3	0.043	0.011	0.039	0.031
	2600	22.99	3	0.062	0.029	0.052	0.048
	2700	23.87	3	0.070	0.035	0.067	0.057
	0	0.00	3	0.052	0.026	0.046	0.042

D.3.4 Laboratory Data

Table D.10: US-301 laboratory data

STATE MATERIALS OFFICE
Internal Research, UF Research
US 301 Field Samples
Mr Summary

Sample Number	Station	Offset	Depth	LIMS Number	Std Proctor Dry Unit Mass (pcf)	Std Proctor Opt. Moisture (%)	M(r) Dry Unit Mass (pcf)	M(r) Opt. Moisture (%)	M(r) @ bulk stress (θ) = 11 psi
Loc 1	684				107.9	12.1%	107.7	10.8%	12,370
					107.9	12.1%	107.6	11.0%	11,977
Loc 2	682				107.8	12.5%	107.3	10.6%	12,425
					107.8	12.5%	106.7	10.9%	11,919
Loc 3	680				110.7	11.8%	109.2	10.8%	14,216
					110.7	11.8%	109.0	11.1%	13,360

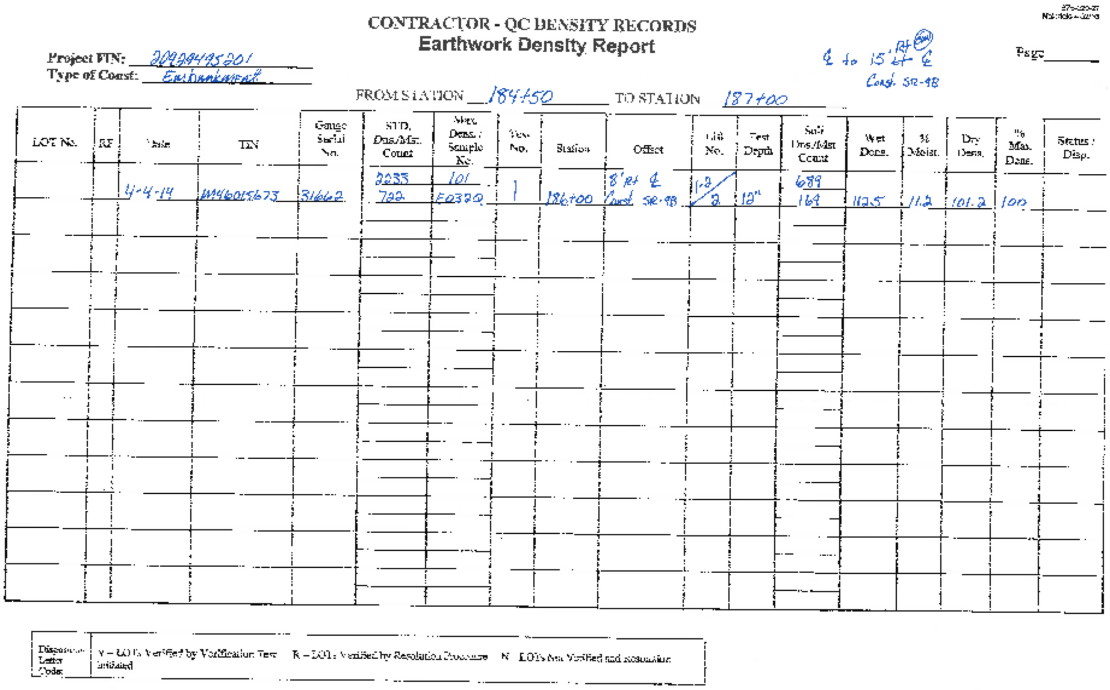
** θ = sum of principal stresses, σ1 + σ2 + σ3

D.4 Project Site Number 3 SR- 9B

The third test site was SR 9-B (I-295), new construction. Testing at the project site was conducted on embankment material placed north of the US 1 overpass (MSE wall and bridge.) On April 4, 2014, on a 300-plus-ft.-long “test strip,” the first lift of A-3 embankment soil was placed, compacted, and tested to determine percentage of the maximum density, a minimum of 100% of the laboratory standard Proctor. Although the density test indicated acceptance (see D.3.1, Contractor QC Density Records) it was yielding (e.g., spongy) under foot.

On April 5, 2014 portable equipment testing was conducted by UF personnel Jay Patel, Luis Avila, and Wayne Rilko. Testing commenced at 8:05 AM and was discontinued at approximately 1:00 PM due to the condition of the soil. As the PLT equipment would not be available until Thursday of the following week, the moisture could change significantly during the time elapsed between portable equipment testing and PLT testing. The testing plan was revised to conduct the PLT tests on Thursday April 10, 2014 and the portable equipment tests on Saturday April 12, 2014. The two dates were the closest times when the two test crews (UF and FDOT SMO) were available. Samples, two soil bags from each station, were taken for resilient modulus and gradation tests.

D.4.1 Earthwork Density Report



CONTRACTOR - QC DENSITY RECORDS
Earthwork Density Report

Project FIN: 20928495201
Type of Const: Embankment

℄ to 15' Rt ℄ Const. SR-9B

Page ______

FROM STATION 184+50 TO STATION 187+00

LOT No.	RF	Date	TIN	Gauge Serial No.	STD. Dns/Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	LR No.	Test Depth	Soil Dns/Mst Count	Wet Dens.	% Moist.	Dry Dens.	% Max. Dens.	Status / Disp.
		4-4-14	M46015673	31662	2235 / 722	101 / E0329	1	186+00	8' Rt ℄ Const SR-9B	1-2 / 2	12"	689 / 164	112.5	11.2	101.2	100	

Disposition Letter Code: V – LOTs Verified by Verification Test R – LOTs Verified by Resolution Procedure N – LOTs Not Verified and resolution initiated

Figure D.7: SR-9B quality control earthwork density report

D.4.2 Nuclear Density Gauge Data

Table D.11: SR-9B NDG densities and moistures

Phase 3 Field Densities				
04.14.2014				
Site #3 SR 9B				
A-3 Embankment				
	Orientation	Wet Density (pcf)	Dry Density (pcf)	Moisture (%)
PLT #1 (North)	Test 1	114.6	102.7	11.6
	Average	114.6	102.7	11.6
	Test 2	114.4	103.4	10.6
	Average	114.4	103.4	10.6
	North Site Average	114.5	103.1	11.1
PLT #2 (Center)	Test 1	111.9	105.6	5.9
	Average	111.9	105.6	5.9
	Test 2	111.3	104.1	6.9
	Average	111.3	104.1	6.9
	Center Site Average	111.6	104.9	6.4
PLT #1 (South)	Test 1	104.3		3.3
	Average	104.3	100.9	3.3
	Test 2	104.9	101.0	3.8
	Average	104.9	101.0	3.8
	North Site Average	104.6	101.0	3.6

D.4.3 Plate Load Test Data

Table D.12: SR-9B PLT #1

PLATE BEARING TEST
STATIC LOADING

Project: Jacksonville 9B
Station or Lab No.:
Layer Tested: Embankment
Thickness: 12"
Description: UF Study

Test No.: 1
Plate Diameter: 12"
Degree Saturation:
Date: 4/14/2014
Tested By: Todd Britton
Kyle Sheppard

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	500	4.42		0.006	0.088	0.000	0.032
	1000	8.84		0.018	0.021	0.000	0.013
	1500	13.26		0.038	0.040	0.022	0.033
	0	0.00		0.031	0.041	0.022	0.031
Seating Load #2	500	4.42		0.002	-0.002	-0.002	0.000
	1000	8.84		0.008	0.002	0.001	0.004
	1500	13.26		0.025	0.022	0.011	0.020
	1700	15.03		0.043	0.041	0.028	0.037
	0	0.00		0.032	0.039	0.024	0.032
Seating Load #3	500	4.42		0.003	-0.003	-0.001	0.000
	1000	8.84		0.007	0.001	0.000	0.003
	1500	13.26		0.014	0.009	0.003	0.009
	2000	17.68		0.032	0.030	0.015	0.025
	2200	19.45		0.042	0.040	0.025	0.036
	0	0.00		0.027	0.037	0.020	0.028
Final Load	500	4.42		0.003	-0.001	-0.001	0.000
	1000	8.84		0.007	0.002	0.001	0.003
	1500	13.26		0.011	0.006	0.002	0.006
	2000	17.68		0.021	0.019	0.007	0.016
	2500	22.10		0.043	0.045	0.026	0.038
	2650	23.43		0.058	0.060	0.039	0.052
	0	0.00		0.040	0.053	0.030	0.041

Max. Density
(AASHO T- ______)

Optimum Moisture
(AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test:
______ pcf

Moisture @ Test:
______ %

p = Stress @ 0.05" Defl.
= 22.4 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05
= 3174.1 psi

Table D.13: SR-9B PLT #2

PLATE BEARING TEST
STATIC LOADING

Project:	Jacksonville 9B	Test No.:	2
Station or Lab No.:		Plate Diameter:	12"
Layer Tested:	Embankment	Degree Saturation:	
Thickness:	12"	Date:	4/14/2014
Description:	UF Study	Tested By:	Todd Britton
			Kyle Sheppard

Max. Density
(AASHO T- ______)

Optimum Moisture
(AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test:
______ pcf

Moisture @ Test:
______ %

p = Stress @ 0.05" Defl.
= 30.0 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05
= 4254.9 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	500	4.42		0.003	0.003	0.002	0.003
	1000	8.84		0.009	0.014	0.008	0.010
	1500	13.26		0.022	0.029	0.020	0.024
	1800	15.92		0.030	0.037	0.028	0.032
	0	0.00		0.018	0.028	0.020	0.022
Seating Load #2	500	4.42		0.004	0.005	0.005	0.004
	1000	8.84		0.008	0.010	0.009	0.009
	1500	13.26		0.013	0.016	0.014	0.014
	2000	17.68		0.025	0.029	0.024	0.026
	2250	19.89		0.031	0.035	0.029	0.032
	0	0.00		0.014	0.020	0.015	0.016
Seating Load #3	500	4.42		0.003	0.004	0.004	0.004
	1000	8.84		0.008	0.008	0.007	0.007
	1650	14.59		0.013	0.014	0.011	0.013
	2000	17.68		0.019	0.022	0.018	0.020
	2500	22.10		0.031	0.033	0.027	0.030
	0	0.00		0.012	0.016	0.013	0.014
Final Load	1000	8.84		0.008	0.007	0.006	0.007
	1500	13.26		0.012	0.011	0.009	0.011
	2000	17.68		0.019	0.019	0.015	0.017
	2500	22.10		0.026	0.026	0.021	0.024
	3000	26.53		0.040	0.039	0.032	0.037
	3500	30.95		0.059	0.058	0.047	0.055
	0	0.00		0.033	0.035	0.026	0.031

D.4.4 Laboratory Data

Table D.14: SR-9B laboratory data

STATE MATERIALS OFFICE

IR - Internal Research

00 - No District, 00 - No County, SR-9B

04/07/2014

Sample Number	T-88 % Passing 3/4"	T-88 % Passing 1/2"	T-88 % Passing 3/8"	T-88 % Passing #4	T-88 % Passing #10	T-88 % Passing #40	T-88 % Passing #60	T-88 % Passing #100	T-88 % Passing #200	T-88 % Sand	T-88 AASHTO Class.	T-88 Unified Class.
PLT Location 1	100.0	100.0	100.0	100.0	100.0	99.5	93.2	27.7	2.4	97.6	A-3	SP
PLT Location 2	100.0	100.0	100.0	100.0	99.9	97.7	90.4	42.0	3.9	96.1	A-3	SP
PLT Location 3	100.0	100.0	100.0	100.0	99.9	98.9	89.7	20.6	4.0	96.0	A-3	SP

	Proctor Dry Unit Mass (pcf)	Proctor Opt. Moisture (%)	M(r) Dry Unit Mass (pcf)	M(r) Opt. Moisture (%)	Mr @ bulk stress (θ) = 30 psi
PLT Location 1	101.2	11.2%	98.8	11.7	14,629
	101.2	11.2%	98.1	11.5	14,258
PLT Location 2	101.2	11.2%	99.3	12.0	13,273
	101.2	11.2%	99.1	12.0	13,411
PLT Location 3	101.2	11.2%	98.1	11.7	12,850
	101.2	11.2%	97.7	11.9	13,422

D.5 Project Site Number 4 CR-210

Project site Number 4 is the construction of additional lanes for CR-210 north and south of the US-1 overpass. This project is south of the SR-9B project. Although shell-rock was used for stabilized subgrade and base throughout the majority of the project, limited sections of limerock subgrade were available for testing. Testing of the limerock subgrade on southbound lanes north of the US-1 overpass was conducted on May 29, 2014. Both portable equipment testing and PLTs were run.

Portable equipment testing was conducted by UF personnel Jay Patel, Luis Avila, and Wayne Rilko. The PLTs were conducted by FDOT SMO personnel Todd Britton, Bruce Swidarski, Travis Stevens, and Kyle Sheppard.

D.5.1 Earthwork Density Reports

CONTRACTOR - QC DENSITY RECORDS

Quality Control - Summary of Proctor Samples

Project FIN 210420-9-52-01 Type of Construction 020L Page 1 of

QC Sample Information				Verification Data		
Sample No.	Material Description	Max. Density	Opt. Moist.	LOTs Represented	Sample No.	Max. Density
S001Q	Brown Fine Sand w/ Limerock + RAP (LBR-37)	107.0	13.0	1 + 2	S001V	106.0@13.0
~~S001V~~	~~Brown Fine Sand w/ Limerock + RAP~~					
~~S001Q~~	(LBR-24)	~~108.0~~	~~14.0~~			
S002Q	Brown Fine Sand w/ Shellrock (LBR-51)	112.0	11.0	3 + 4		
S003Q	Brown Fine Sand w/ Shellrock (LBR-65)	114.0	10.0	5		
S004Q	Brown Fine Sand w/ Shellrock (LBR-44)	108.0	12.0	6+7		
S005Q	Brown Fine Sand w/ Shellrock (LBR-37)	108.0	9.0	8		
S006Q	Brn Fine Sand w/ Shellrock (LBR-46)	110.0	12.0	9+10	S006Q	111
S007Q	Brn Fine Sand w/ Shellrock (LBR 50)	111.0	10.0	11 + 12		
S008Q	Brn Fine Sand w/ Shellrock (LBR 103)	120.0	10.0	13 + 14		
S009Q	Brn Fine Sand w/ Shellrock (LBR 60)	113.0	11.0	15 + 16		
S010Q	Brn Fine Sand w/ Shellrock (LBR-58)	112.0	11.0	17 + 18	S010V	113.0
S011Q	Brn Fine Sand w/ Shellrock (LBR-44)	110.0	12.0	19		
S012Q	Brn Fine Sand w/ Shellrock (LBR-69)	119.0	10.0	20 - 21		
S013Q	Brn Fine Sand w/ Shellrock (LBR-66)	116.0	11.0	22		

Figure D.8: CR-210 quality control summary of proctor samples

CONTRACTOR - QC DENSITY RECORDS

Earthwork Density Report

Project FIN: 21042095201

Type of Const: Subgrade / compacted

*6 4-13, 5-13 Limerock in Lieu of 12" Stab. subgrade JRP

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675-020-27 Materials – 02/10

FROM STATION 87+00 TO STATION 92+00

LOT No.	RF	Date	TIN	Gauge Serial No.	STD. Dns./Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns./Mst Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens.	Status / Disp.
1-13		12-5-13	P3504367!	34686	2214 / 681	113.0 / S009Q	1	89+00	20' LT ℄ CONST	1-2/2	12"	633 / 80	116.8	5.3	110.9	98	P/V
2-13		12-06-13	P35043671	34686	2203 / 670	113.0 / S009Q	2	89+50	25' LT ℄ CONST	1/1	4"	3067 / 129	121.6	9.5	111.1	98	P/V
3-13		12-06-13	P35043671	34686	2203 / 670	113.0 / S009Q	3	90+25	55' RT ℄ CONST	1/1	4"	23+24	121.7	9.0	111.7	99	P/V
4-13		5-14-14	P35043671	30166	2237 / 589	117.0 / B002Q	4	91+00	17' LT ℄ CONST	1/1	6"	1786 / 163	131.6	12.9	116.6	100	P/V
5-13		5-14-14	P35043671	30166	2237 / 589	117.0 / B002Q	5	90+80	36' LT ℄ CONST	1/1	6"	1812 / 156	130.5	12.3	116.2	99	P/V

Disposition Letter Code: V – LOTs Verified by Verification Test R – LOTs Verified by Resolution Procedure N – LOTs Not Verified and Resolution initiated

Figure D.9: CR-210 quality control earthwork density report

FLORIDA DEPARTMENT OF TRANSPORTATION

675-030-28

Verification - Earthwork Density Report

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Project FIN: 210420-9-52-01 & 210420-5-56-01
Type of Const: STAB. SUBGRADE + CURB PAD FROM STATION 87+00 TO STATION 92+00

LOT No.	Date	TIN	Gauge Serial No.	STD Dns./Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns./Mst. Count	Wet Dens	% Moist.	Dry Dens.	% Max Dens	LOTs Accepted
★ [illegible]-13	12-6-13	R10052179	22575	2272 / 592	113 / S009Q	1	88+50	45' LT BL OF CONS	1	4"	2127 / 110	120.1	7.9	111.3	98	2-13 3-13
						6" IN LIEU OF 12" STABILIZED										
1-13	5-14-14	R10052179	22575	2271 / 597	117 / B002Q	1	91+20	35' LT CL OF CONST	1	6"	1787 / 152	127.8	11.1	115	98	4-13 1-37
						CURB PAD										
2-13	5-14-14	R10052179	22575	2271 / 597	117 / B002Q	2	91+00	15' LT CL OF CONST	1	6"	1781 / 147	128.2	11.4	115.1	98	5-13 2-31

★ 4" CURB PAD

Figure D.10: CR-210 verification earthwork density report

D.5.2 Nuclear Density Gauge Data

Table D.15: CR-210 NDG densities and moistures

Phase 3 Field Densities				
05.29.2014				
Site #4 CR 210				
Limerock Subgrade				
	Orientation	**Wet Density (pcf)**	**Dry Density (pcf)**	**Moisture (%)**
PLT #1 (North)	East	112.2	110.5	10.6
		124.8	113.4	10.1
	Average	118.5	112.0	10.4
	West	124.3	113.1	9.9
		122.3	110.8	10.4
	Average	123.3	112.0	10.2
	North Site Average	**120.9**	**112.0**	**10.3**
PLT #2 (Center)	East	130.7	115.9	9.0
		123.4	113.3	9.0
	Average	127.1	114.6	9.0
	West	127.9	118.2	8.3
		132.2	123.2	7.4
	Average	130.1	120.7	7.9
	Center Site Average	**128.6**	**117.7**	**8.4**
PLT #1 (South)	East	121.6	115.1	5.7
		124.0	117.1	5.9
	Average	122.8	116.1	5.8
	West	125.4	119.0	5.4
		126.2	119.2	5.9
	Average	125.8	119.1	5.7
	South Site Average	**124.3**	**117.6**	**5.7**

D.5.3 Plate Load Test Data

Table D.16: CR-210 PLT #1

PLATE BEARING TEST
STATIC LOADING

Project: SR 210 Jacksonville
Station or Lab No.: 1
Layer Tested: base
Thickness: 8"
Description: UF Study

Test No.: 1
Plate Diameter: 12
Degree Saturation:
Date: 5/29/2014
Tested By: Britton, Swidarski
Sheppard, Stevens

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	1000	8.84	3	0.006	0.003	0.010	0.006
	2000	17.68	3	0.013	0.006	0.020	0.013
	3000	26.53	3	0.019	0.011	0.028	0.019
	4000	35.37	3	0.029	0.026	0.036	0.030
	0	0.00	3	0.018	0.017	0.024	0.020
Seating Load #2	1500	13.26	3	0.003	0.004	0.006	0.004
	3000	26.53	3	0.008	0.009	0.012	0.010
	4500	39.79	3	0.012	0.014	0.017	0.014
	6000	53.05	3	0.017	0.019	0.023	0.020
	8000	70.74	3	0.025	0.027	0.032	0.028
	9000	79.58	3	0.029	0.030	0.036	0.032
	0	0.00	3	0.010	0.014	0.014	0.012
Seating Load #3	3000	26.53	3	0.010	0.010	0.012	0.011
	6000	53.05	3	0.017	0.018	0.019	0.018
	9000	79.58	3	0.024	0.024	0.027	0.025
	12000	106.10	3	0.032	0.032	0.037	0.034
	0	0.00	3	0.009	0.010	0.010	0.010
Final Load	4000	35.37	3	0.016	0.016	0.015	0.016
	8000	70.74	3	0.023	0.024	0.024	0.023
	12000	106.10	3	0.030	0.030	0.032	0.031
	16000	141.47	3	0.041	0.041	0.046	0.042
	19000	168.00	3	0.051	0.052	0.059	0.054
	0	0.00	3	0.016	0.017	0.019	0.017

Max. Density
(AASHO T-)

Optimum Moisture
(AASHO T-)

LBR
At Optimum
Soaked

Dry Density @ Test:
pcf

Moisture @ Test:
%

p = Stress @ 0.05" Defl.
= 180.7 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 25583.1 psi

Table D.17: CR-210 PLT #2

PLATE BEARING TEST
STATIC LOADING

Project: SR 210 Jacksonville
Station or Lab No.: 2
Layer Tested: base
Thickness: 8"
Description: UF Study

Test No.: 2
Plate Diameter: 12
Degree Saturation:
Date: 5/29/2014
Tested By: Britton, Swidarski
Sheppard, Stevens

Max. Density
(AASHO T- ______)

Optimum Moisture
(AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test:
______ pcf

Moisture @ Test:
______ %

p = Stress @ 0.05" Defl.
= 173.7 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 24599.6 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	1000	8.84	3	0.007	0.002	0.006	0.005
	2000	17.68	3	0.014	0.009	0.013	0.012
	3000	26.53	3	0.019	0.014	0.019	0.017
	4000	35.37	3	0.024	0.019	0.025	0.023
	5000	44.21	3	0.029	0.024	0.031	0.028
	6000	53.05	3	0.035	0.030	0.037	0.034
	0	0.00	3	0.023	0.022	0.022	0.023
Seating Load #2	2000	17.68	3	0.006	0.003	0.006	0.005
	4000	35.37	3	0.010	0.008	0.011	0.010
	6000	53.05	3	0.016	0.014	0.017	0.015
	8000	70.74	3	0.024	0.024	0.026	0.025
	10000	88.42	3	0.032	0.032	0.035	0.033
	0	0.00	3	0.015	0.015	0.014	0.015
Seating Load #3	3000	26.53	3	0.007	0.007	0.008	0.007
	6000	53.05	3	0.013	0.013	0.015	0.014
	9000	79.58	3	0.018	0.020	0.021	0.020
	12000	106.10	3	0.028	0.030	0.031	0.030
	12500	110.52	3	0.032	0.033	0.035	0.034
	0	0.00	3	0.011	0.012	0.011	0.011
Final Load	4000	35.37	3	0.010	0.008	0.010	0.009
	8000	70.74	3	0.017	0.015	0.018	0.017
	12000	106.10	3	0.024	0.023	0.027	0.025
	16000	141.47	3	0.037	0.037	0.040	0.038
	20000	176.84	3	0.055	0.053	0.058	0.055
	0	0.00	3	0.023	0.021	0.022	0.022

Table D.18: CR-210 PLT #3

PLATE BEARING TEST
STATIC LOADING

Project: SR 210 Jacksonville
Station or Lab No.: 3
Layer Tested: base
Thickness: 8"
Description: UF Study

Test No.: 3
Plate Diameter: 12
Degree Saturation:
Date: 5/29/2014
Tested By: Britton, Swidarski
Sheppard, Stevens

Max. Density
(AASHO T- ______)

Optimum Moisture
(AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test:
______ pcf

Moisture @ Test:
______ %

p = Stress @ 0.05" Defl.
= 235.1 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 33295.9 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	1200	10.61	3	0.001	0.003	0.004	0.003
	2000	17.68	3	0.003	0.006	0.008	0.006
	3000	26.53	3	0.007	0.011	0.014	0.011
	4000	35.37	3	0.010	0.015	0.018	0.014
	6000	53.05	3	0.015	0.023	0.023	0.020
	9000	79.58	3	0.025	0.034	0.038	0.032
	0	0.00	3	0.017	0.018	0.022	0.019
Seating Load #2	2000	17.68	3	0.000	0.004	0.004	0.003
	4000	35.37	3	0.001	0.008	0.008	0.006
	6000	53.05	3	0.004	0.013	0.012	0.010
	8000	70.74	3	0.007	0.017	0.016	0.013
	12000	106.10	3	0.018	0.028	0.027	0.024
	15000	132.63	3	0.026	0.038	0.037	0.034
	0	0.00	3	0.010	0.011	0.012	0.011
Seating Load #3	4000	35.37	3	0.001	0.008	0.009	0.006
	8000	70.74	3	0.007	0.015	0.015	0.012
	12000	106.10	3	0.013	0.022	0.022	0.019
	16000	141.47	3	0.020	0.030	0.030	0.027
	18000	159.15	3	0.026	0.036	0.036	0.032
	0	0.00	3	0.005	0.007	0.007	0.006
Final Load	5000	44.21	3	0.008	0.012	0.011	0.010
	10000	88.42	3	0.014	0.020	0.018	0.018
	15000	132.63	3	0.021	0.027	0.026	0.024
	20000	176.84	3	0.031	0.037	0.036	0.035
	26000	229.89	3	0.046	0.054	0.056	0.052
	0	0.00	3	0.016	0.014	0.015	0.015

D.5.4 Laboratory Data

Table D.19: CR-210 laboratory data

STATE MATERIALS OFFICE

IR - Internal Research

02 - District 2 - Lake City, 00 - No County, CR-210

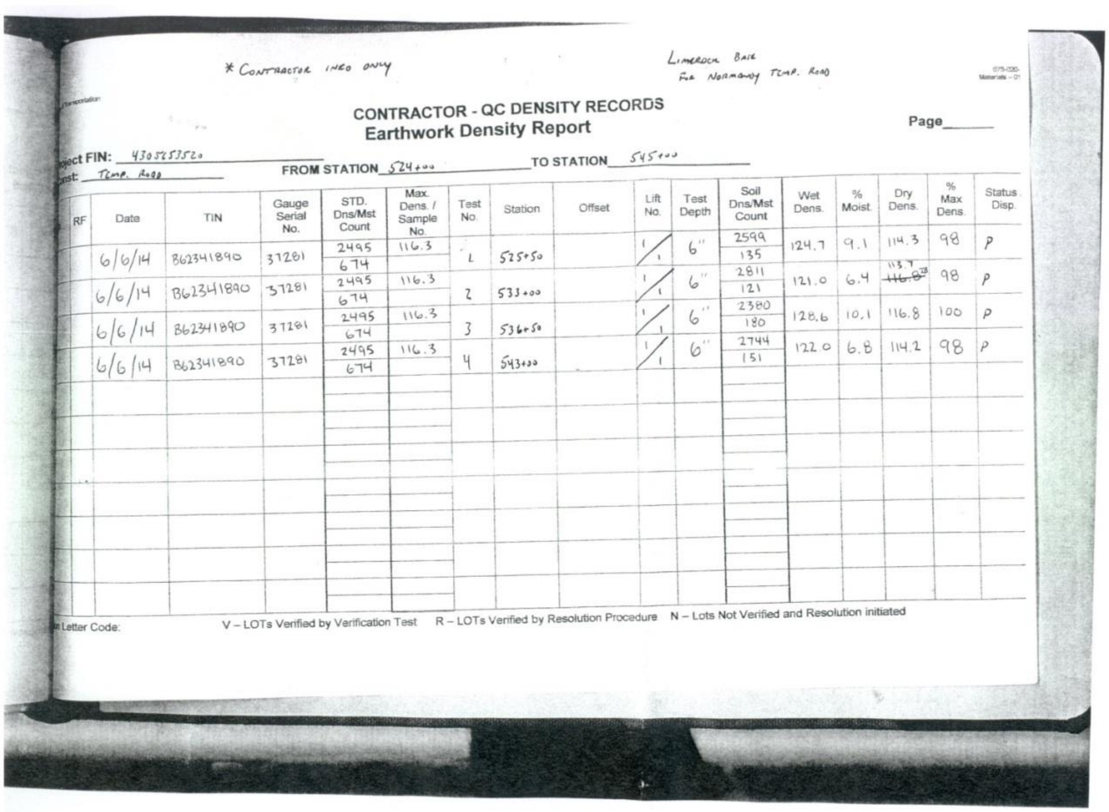
05/30/2014

Sample Number	Location				T-88 % Passing 3/4"	T-88 % Passing 1/2"	T-88 % Passing 3/8"	T-88 % Passing #4	T-88 % Passing #10	T-88 % Passing #40	T-88 % Passing #60	T-88 % Passing #100	T-88 % Passing #200	T-88 % Sand	T-88 % Silt	T-88 % Clay
1	Stockpile				74.6	63.8	57.7	49.0	41.8	34.1	28.7	22.1	15.5	84.5	12.7	2.8
			T-88 AASHTO Class.	T-88 Unified Class.	T-89&90 LL (%)	T-89&90 PL (%)	T-89&90 PI (%)	T-180 Max. Density (pcf)	T-180 Opt. Moisture (%)	T-307 K1	T-307 K2	T-307 11 psi (Embankment)	T-307 15 psi (Subgrade)	T-307 30 psi (Base)	T-307 Density (pcf)	T-307 Moisture (%)
			A-1-b	GM	np	np	np	118.6	11	3233.4927	0.5778306	12,608	17,735	22,363	119.4	10.4

D.6 Project Site Number 5 SR-23 Limerock Base

On July 7, 2014, Jesse Sutton, District Materials Earthwork Engineer, FDOT District 2, was contacted. He provided contact information for the CEI representatives for SR-23. Later that day, Wayne Rilko met with the CEI personnel to discuss project scope and to request access to the project. The CEIs received permission from the contractor to proceed. Portable equipment testing was conducted by UF personnel Luis Avila and Wayne Rilko the following morning, July 8, 2014. PLTs were conducted by FDOT SMO personnel on July 9, 2014 by Todd Britton, Bruce Swidarski, Travis Stevens, and Kyle Sheppard. The project location was the southeast quadrant of SR-23 and Normandy Boulevard.

D.6.1 Earthwork Density Reports



* Contractor info only

Limerock Base
For Normandy Temp. Road

CONTRACTOR - QC DENSITY RECORDS
Earthwork Density Report

Page______

…ject FIN: 430525352 0
…nst: Temp. Road
FROM STATION 524+00 TO STATION 545+00

RF	Date	TIN	Gauge Serial No.	STD. Dns/Mst Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns/Mst Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens.	Status Disp.
	6/6/14	B62341890	37281	2495 / 674	116.3	1	525+50		1/1	6"	2599 / 135	124.7	9.1	114.3	98	P
	6/6/14	B62341890	37281	2495 / 674	116.3	2	533+00		1/1	6"	2811 / 121	121.0	6.4	~~116.8~~ 113.7	98	P
	6/6/14	B62341890	37281	2495 / 674	116.3	3	536+50		1/1	6"	2380 / 180	128.6	10.1	116.8	100	P
	6/6/14	B62341890	37281	2495 / 674	116.3	4	543+00		1/1	6"	2744 / 151	122.0	6.8	114.2	98	P

…Letter Code: V – LOTs Verified by Verification Test R – LOTs Verified by Resolution Procedure N – Lots Not Verified and Resolution initiated

Figure D.11: SR-23 quality control earthwork density report

D.6.2 Nuclear Density Gauge Data

Table D.20: SR-23 NDG densities and moistures

Phase 3 Field Densities				
07.09.2014				
Site #5 SR 23				
Limerock Base				
	Orientation	**Wet Density (pcf)**	**Dry Density (pcf)**	**Moisture (%)**
PLT #1 (West)	**Test 1**	116.4	107.7	8.2
		116.1	107.7	8.4
	Average	116.3	107.7	8.3
	Test 2	118.1	109.1	8.3
		116.7	107.5	8.5
	Average	117.4	108.3	8.4
	West Site Average	**116.8**	**108.0**	**8.4**
PLT #2 (Center)	**Test 1**	121.5	111.3	9.2
		123.9	113.0	9.6
	Average	122.7	112.2	9.4
	Test 2	124.3	114.2	8.8
		123.6	113.4	8.9
	Average	124.0	113.8	8.9
	Center Site Average	**123.3**	**113.0**	**9.1**
PLT #1 (East)	**Test 1**	124.6	114.7	8.6
		121.8	112.6	8.2
	Average	123.2	113.65	8.4
	Test 2	124.0	114.5	8.3
		122.5	112.9	8.4
	Average	123.3	113.7	8.4
	East Site Average	**123.2**	**113.7**	**8.4**

D.6.3 Plate Load Test Data

Table D.21: SR-23 PLT #1

PLATE BEARING TEST
STATIC LOADING

Project: SR-23 Jacksonville
Station or Lab No.: 538
Layer Tested: limerock base
Thickness: 6.5"
Description: UF Study

Test No.: 1
Plate Diameter: 12"
Degree Saturation:
Date: 7/9/2014
Tested By: KS,TB,BS,TS

Max. Density (AASHO T-)

Optimum Moisture (AASHO T-)

LBR
At Optimum
Soaked

Dry Density @ Test: pcf

Moisture @ Test: %

p = Stress @ 0.05" Defl. = 157.6 psi

a = Radius of Plate 6 in.

E = 1.18 pa / 0.05

= 22312.8 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	1000	8.84	3	0.003	0.008	0.003	0.004
	2000	17.68	3	0.009	0.014	0.007	0.010
	3000	26.53	3	0.011	0.019	0.098	0.043
	4000	35.37	3	0.014	0.025	0.013	0.017
	6000	53.05	3	0.023	0.036	0.021	0.027
	7000	61.89	3	0.024	0.042	0.027	0.031
	0	0.00	3	0.018	0.025	0.011	0.018
Seating Load #2	3000	26.53	3	0.005	0.011	0.007	0.008
	6000	53.05	3	0.011	0.019	0.014	0.015
	9000	79.58	3	0.020	0.030	0.025	0.025
	10500	92.84	3	0.026	0.037	0.030	0.031
	0	0.00	3	0.013	0.017	0.010	0.013
Seating Load #3	3000	26.53	3	0.006	0.008	0.073	0.029
	6000	53.05	3	0.012	0.016	0.014	0.014
	9000	79.58	3	0.021	0.026	0.023	0.023
	12000	106.10	3	0.028	0.035	0.031	0.031
	0	0.00	3	0.013	0.009	0.008	0.010
Final Load	4000	35.37	3	0.008	0.015	0.011	0.011
	8000	70.74	3	0.014	0.023	0.018	0.018
	12000	106.10	3	0.021	0.033	0.027	0.027
	16000	141.47	3	0.041	0.053	0.044	0.046
	17000	150.31	3	0.046	0.059	0.049	0.051
	0	0.00	3	0.016	0.020	0.015	0.017

Table D.22: SR-23 PLT #2

PLATE BEARING TEST
STATIC LOADING

Project: SR-23 Jacksonville
Station or Lab No.: 538
Layer Tested: limerock base
Thickness: 6.5"
Description: UF Study

Test No.: 2
Plate Diameter: 12"
Degree Saturation:
Date: 7/9/2014
Tested By: KS,TB,BS,TS

Max. Density
(AASHO T- ______)

Optimum Moisture
(AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test:
______ pcf

Moisture @ Test:
______ %

p = Stress @ 0.05" Defl.
= 174.7 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 24742.3 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	1500	13.26	3	0.006	0.004	0.008	0.006
	3000	26.53	3	0.013	0.012	0.015	0.013
	4500	39.79	3	0.018	0.016	0.021	0.018
	6000	53.05	3	0.024	0.021	0.027	0.024
	8000	70.74	3	0.031	0.029	0.034	0.031
	0	0.00	3	0.018	0.018	0.018	0.018
Seating Load #2	2000	17.68	3	0.003	0.003	0.004	0.003
	4000	35.37	3	0.008	0.007	0.009	0.008
	6000	53.05	3	0.012	0.011	0.012	0.012
	8000	70.74	3	0.017	0.015	0.015	0.016
	10000	88.42	3	0.024	0.022	0.021	0.022
	12000	106.10	3	0.033	0.030	0.029	0.030
	0	0.00	3	0.011	0.011	0.007	0.009
Seating Load #3	3000	26.53	3	0.004	0.005	0.006	0.005
	6000	53.05	3	0.011	0.011	0.012	0.011
	9000	79.58	3	0.018	0.018	0.018	0.018
	12000	106.10	3	0.025	0.024	0.024	0.024
	14500	128.21	3	0.034	0.032	0.032	0.033
	0	0.00	3	0.007	0.008	0.006	0.007
Final Load	4000	35.37	3	0.010	0.007	0.009	0.009
	8000	70.74	3	0.017	0.014	0.016	0.016
	12000	106.10	3	0.025	0.022	0.024	0.024
	16000	141.47	3	0.038	0.034	0.036	0.036
	20000	176.84	3	0.057	0.053	0.054	0.054
	0	0.00	3	0.019	0.018	0.017	0.018

Table D.23: SR-23 PLT #3

PLATE BEARING TEST
STATIC LOADING

Project: SR-23 Jacksonville
Station or Lab No.: 539
Layer Tested: limerock base
Thickness: 6.5"
Description: UF Study

Test No.: 3
Plate Diameter: 12"
Degree Saturation:
Date: 7/9/2014
Tested By: KS,TB,BS,TS

Max. Density (AASHO T- ________)

Optimum Moisture (AASHO T- ________)

LBR
At Optimum ________
Soaked ________

Dry Density @ Test: ________ pcf

Moisture @ Test: ________ %

p = Stress @ 0.05" Defl.
= 235.7 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 33375.5 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	1500	13.26	3	0.008	-0.001	0.007	0.005
	3000	26.53	3	0.011	0.003	0.012	0.009
	4500	39.79	3	0.017	0.008	0.016	0.014
	6000	53.05	3	0.021	0.012	0.021	0.018
	7500	66.31	3	0.025	0.015	0.025	0.022
	11000	97.26	3	0.035	0.025	0.035	0.032
	0	0.00	3	0.029	0.015	0.019	0.021
Seating Load #2	4000	35.37	3	0.007	0.000	0.007	0.005
	8000	70.74	3	0.013	0.006	0.013	0.010
	12000	106.10	3	0.021	0.012	0.021	0.018
	16000	141.47	3	0.035	0.025	0.034	0.031
	0	0.00	3	0.021	0.008	0.011	0.013
Seating Load #3	5000	44.21	3	0.003	0.007	0.008	0.006
	10000	88.42	3	0.011	0.017	0.017	0.015
	15000	132.63	3	0.019	0.028	0.025	0.024
	18500	163.58	3	0.024	0.037	0.032	0.031
	0	0.00	3	0.011	0.011	0.008	0.010
Final Load	5000	44.21	3	0.003	0.009	0.009	0.007
	10000	88.42	3	0.010	0.018	0.016	0.015
	15000	132.63	3	0.016	0.026	0.023	0.022
	20000	176.84	3	0.025	0.034	0.032	0.030
	25000	221.05	3	0.037	0.049	0.049	0.045
	27500	243.15	3	0.044	0.060	0.055	0.053
	0	0.00	3	0.017	0.021	0.017	0.018

D.6.4 Laboratory Data

Table D.24: SR-23 laboratory data
Refer to Limerock Base

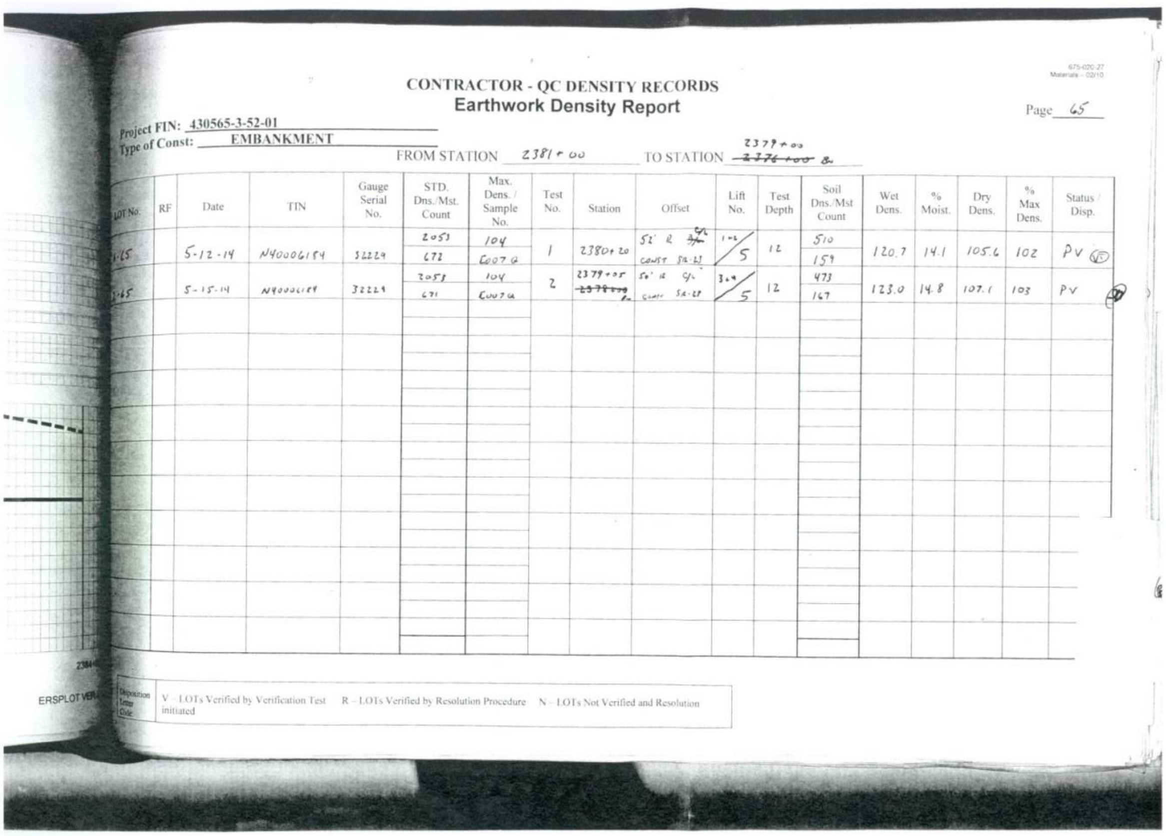
STATE MATERIALS OFFICE
TP - Test Pit
00 - No District, 00 - No County, SR-23
07/17/2014

Sample Number	Station	% Passing 3/4"	% Passing 1/2"	% Passing 3/8"	% Passing #4	% Passing #10	% Passing #40	% Passing #60	% Passing #100	% Passing #200	% Sand	% Silt	% Clay	AASHTO Class.	Unified Class.	Max. Density (pcf)	Opt. Moisture (%)	11 psi (Embankment)	30 psi (Base)
A-3 2678+00	2678+00	100.0	100.0	100.0	100.0	99.9	98.7	93.6	60.3	10.2	89.8	6.9	3.3	A-3	SP-SM	108.4	12.9	12,243	
A-3 2680+00	2680+00	100.0	100.0	100.0	100.0	100.0	98.8	94.2	52.2	9.8	90.2	6.9	2.9	A-3	SP-SM	107.5	12.7	12,563	
Limerock Base		97.5	89.6	84.6	73.2	60.0	41.4	34.8	26.8	18.8	81.2	14.1	4.7	A-1-b	SM	112.0	13.2		22,107

D.7 Project Site Number 6 SR-23 A-3 Embankment

Portable equipment testing was conducted by UF personnel Luis Avila and Wayne Rilko on July 14, 2014. PLTs were conducted by FDOT SMO personnel on July 17, 2014 by Kyle Sheppard, Travis Stevens, and Jason Noll. The project site location was the southeast quadrant SR 23 and New World Avenue.

D.7.1 Earthwork Density Reports



675-020-27
Materials – 02/10

CONTRACTOR - QC DENSITY RECORDS
Earthwork Density Report

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Project FIN: 430565-3-52-01
Type of Const: EMBANKMENT

FROM STATION 2381+00 TO STATION ~~2376+00~~ 2379+00

LOT No.	RF	Date	TIN	Gauge Serial No.	STD. Dns./Mst. Count	Max. Dens. / Sample No.	Test No.	Station	Offset	Lift No.	Test Depth	Soil Dns./Mst Count	Wet Dens.	% Moist.	Dry Dens.	% Max Dens.	Status / Disp.
1-65		5-12-14	N40006154	32229	2053 / 672	104 / C007 Q	1	2380+20	52' R ~~3/~~ C/L CONST SR-23	1-2 / 5	12	510 / 159	120.7	14.1	105.6	102	PV
2-65		5-15-14	N40006154	32229	2058 / 671	104 / C007 Q	2	2379+05 ~~2378+00~~	50' R C/L CONST SR-23	3-4 / 5	12	473 / 167	123.0	14.8	107.1	103	PV

Disposition Letter Code	V – LOTs Verified by Verification Test R – LOTs Verified by Resolution Procedure N – LOTs Not Verified and Resolution initiated

Figure D.12: SR-23 quality control earthwork density report

D.7.2 Nuclear Density Gauge Data

Table D.25: SR-23 NDG densities and moistures

Phase 3 Field Densities				
07.14.2014				
Site #6 SR 23				
A-3 Embankment				
	Orientation	**Wet Density (pcf)**	**Dry Density (pcf)**	**Moisture (%)**
PLT #1 (South)	Test 1	116.6	102.3	13.9
STA 2378+00	Test 2	117.6	103.2	13.9
	Average	117.1	102.8	13.9
	Test 1	116.3	102.9	13.0
	Test 2	117.4	103.3	13.6
	Average	116.9	103.1	13.3
	North Site Average	**117.0**	**102.9**	**13.6**
PLT #2 (Center)	Test 1	116.3	104.5	11.3
STA 2379+00	Test 2	115.5	102.3	12.8
	Average	115.9	103.4	12.1
	Test 1	117.5	103.8	13.2
	Test 2	116.5	104.5	11.5
	Average	117.0	104.2	12.4
	Center Site Average	**116.5**	**103.8**	**12.2**
PLT #1 (North)	Test 1	124.1	110.4	12.4
STA 2378+00	Test 2	114.4	99.0	15.5
	Average	119.3	104.7	14.0
	Test 1	115.8	101.4	14.2
	test 2	115.7	101.9	14.0
	Average	115.8	101.7	14.1
	North Site Average	**117.5**	**103.2**	**14.0**

D.7.3 Plate Load Test Data

Table D.26: SR-23 PLT #1

PLATE BEARING TEST
STATIC LOADING

Project:	New World Ave	Test No.:	1
Station or Lab No.:	2378	Plate Diameter:	12"
Layer Tested:	A-3	Degree Saturation:	
Thickness:		Date:	7/17/2014
Description:	UF Study	Tested By:	KS,JN,TS

Max. Density (AASHO T- ______)

Optimum Moisture (AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test: ______ pcf

Moisture @ Test: ______ %

p = Stress @ 0.05" Defl.
= 43.8 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 6199.0 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	500	4.42	3	0.018	0.018	0.020	0.019
	750	6.63	3	0.025	0.032	0.032	0.029
	900	7.96	3	0.032	0.041	0.040	0.038
	0	0.00	3	0.033	0.041	0.037	0.037
Seating Load #2	500	4.42	3	0.003	0.002	0.003	0.003
	750	6.63	3	0.006	0.005	0.005	0.005
	1500	13.26	3	0.012	0.012	0.012	0.012
	2000	17.68	3	0.019	0.026	0.023	0.022
	2500	22.10	3	0.030	0.040	0.035	0.035
	0	0.00	3	0.030	0.034	0.023	0.029
Seating Load #3	1000	8.84	3	0.008	0.008	0.008	0.008
	1500	13.26	3	0.009	0.010	0.010	0.010
	2000	17.68	3	0.012	0.015	0.014	0.014
	2500	22.10	3	0.019	0.021	0.020	0.020
	3000	26.53	3	0.024	0.030	0.028	0.027
	3500	30.95	3	0.032	0.040	0.037	0.036
	0	0.00	3	0.032	0.031	0.021	0.028
Final Load	1000	8.84	3	0.005	0.005	0.007	0.006
	2000	17.68	3	0.010	0.012	0.013	0.012
	3000	26.53	3	0.018	0.022	0.022	0.020
	4000	35.37	3	0.028	0.035	0.035	0.032
	5000	44.21	3	0.044	0.055	0.052	0.050
	0	0.00	3	0.037	0.039	0.029	0.035

Table D.27: SR-23 PLT #2

PLATE BEARING TEST
STATIC LOADING

Project: New Worl Ave
Station or Lab No.: 2379
Layer Tested: A-3
Thickness:
Description: UF Study

Test No.: 2
Plate Diameter: 12"
Degree Saturation:
Date: 7/17/2014
Tested By: KS,TS,JN

Max. Density
(AASHO T-)

Optimum Moisture
(AASHO T-)

LBR
At Optimum
Soaked

Dry Density @ Test:
pcf

Moisture @ Test:
%

p = Stress @ 0.05" Defl.
= 62.6 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05
= 8870.9 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	800	7.07	3	0.008	0.004	0.005	0.006
	1500	13.26	3	0.014	0.009	0.010	0.011
	2000	17.68	3	0.018	0.013	0.013	0.015
	3000	26.53	3	0.032	0.024	0.024	0.026
	3500	30.95	3	0.038	0.029	0.029	0.032
	0	0.00	3	0.025	0.018	0.013	0.019
Seating Load #2	1000	8.84	3	0.010	0.003	0.006	0.006
	2000	17.68	3	0.017	0.009	0.012	0.013
	2500	22.10	3	0.020	0.012	0.014	0.015
	3000	26.53	3	0.026	0.016	0.018	0.020
	4000	35.37	3	0.038	0.026	0.028	0.031
	0	0.00	3	0.022	0.013	0.011	0.015
Seating Load #3	1100	9.73	3	0.006	0.005	0.006	0.006
	2200	19.45	3	0.013	0.011	0.013	0.012
	3500	30.95	3	0.023	0.020	0.021	0.021
	4000	35.37	3	0.026	0.022	0.024	0.024
	4800	42.44	3	0.034	0.030	0.031	0.032
	0	0.00	3	0.015	0.013	0.011	0.013
Final Load	2000	17.68	3	0.020	0.009	0.012	0.014
	3000	26.53	3	0.026	0.015	0.016	0.019
	4000	35.37	3	0.032	0.020	0.022	0.025
	5000	44.21	3	0.040	0.027	0.030	0.032
	6000	53.05	3	0.050	0.037	0.040	0.042
	6800	60.13	3	0.059	0.045	0.050	0.051
	0	0.00	3	0.031	0.021	0.021	0.024

Table D.28: SR-23 PLT #3

PLATE BEARING TEST
STATIC LOADING

Project: New World Ave
Station or Lab No.: 2380
Layer Tested: A-3
Thickness:
Description: UF Study

Test No.: 3
Plate Diameter: 12"
Degree Saturation:
Date: 7/17/2014
Tested By: KS,JN,TS

Max. Density
(AASHO T- ______)

Optimum Moisture
(AASHO T- ______)

LBR
At Optimum ______
Soaked ______

Dry Density @ Test:
______ pcf

Moisture @ Test:
______ %

p = Stress @ 0.05" Defl.
= 55.0 psi

a = Radius of Plate
6 in.

E = 1.18 pa / 0.05

= 7782.6 psi

Mode	Applied Load (lbs)	Applied Load (psi)	Duration min.	Deflections (in.) Dial #1	Dial #2	Dial #3	Avg. Dial
Seating Load #1	1000	8.84	3	0.006	0.004	0.009	0.006
	1500	13.26	3	0.012	0.009	0.014	0.012
	2000	17.68	3	0.020	0.018	0.022	0.020
	2500	22.10	3	0.028	0.028	0.029	0.028
	2800	24.76	3	0.034	0.035	0.035	0.035
	0	0.00	3	0.033	0.025	0.019	0.025
Seating Load #2	1500	13.26	3	0.009	0.008	0.010	0.009
	2500	22.10	3	0.016	0.015	0.016	0.016
	3500	30.95	3	0.025	0.030	0.029	0.028
	4000	35.37	3	0.030	0.037	0.035	0.034
	0	0.00	3	0.026	0.023	0.015	0.021
Seating Load #3	2000	17.68	3	0.015	0.013	0.015	0.014
	2500	22.10	3	0.018	0.017	0.019	0.018
	3000	26.53	3	0.022	0.021	0.023	0.022
	3500	30.95	3	0.026	0.025	0.026	0.026
	4000	35.37	3	0.032	0.031	0.031	0.031
	0	0.00	3	0.023	0.014	0.010	0.016
Final Load	1500	13.26	3	0.012	0.009	0.012	0.011
	3000	26.53	3	0.020	0.020	0.021	0.020
	4500	39.79	3	0.030	0.032	0.032	0.031
	5500	48.63	3	0.041	0.045	0.044	0.043
	6000	53.05	3	0.046	0.055	0.052	0.051
	0	0.00	3	0.028	0.028	0.020	0.025

D.7.4 Laboratory Data

Table D.29: SR-23 laboratory data
Refer to A-3 (embankment)

STATE MATERIALS OFFICE
TP - Test Pit
00 - No District, 00 - No County, SR-23
07/17/2014

Sample Number	Station	% Passing 3/4"	% Passing 1/2"	% Passing 3/8"	% Passing #4	% Passing #10	% Passing #40	% Passing #60	% Passing #100	% Passing #200	% Sand	% Silt	% Clay	AASHTO Class.	Unified Class.	Max. Density (pcf)	Opt. Moisture (%)	11 psi (Embankment)	30 psi (Base)
A-3 2678+00	2678+00	100.0	100.0	100.0	100.0	99.9	98.7	93.6	60.3	10.2	89.8	6.9	3.3	A-3	SP-SM	108.4	12.9	12,243	
A-3 2680+00	2680+00	100.0	100.0	100.0	100.0	100.0	98.8	94.2	52.2	9.8	90.2	6.9	2.9	A-3	SP-SM	107.5	12.7	12,563	
Limerock Base		97.5	89.6	84.6	73.2	60.0	41.4	34.8	26.8	18.8	81.2	14.1	4.7	A-1-b	SM	112.0	13.2		22,107

Appendix E: Phase III Field Test Results

E.1 I-95 (Site 1)

E.1.1 First Plate Load Test Location

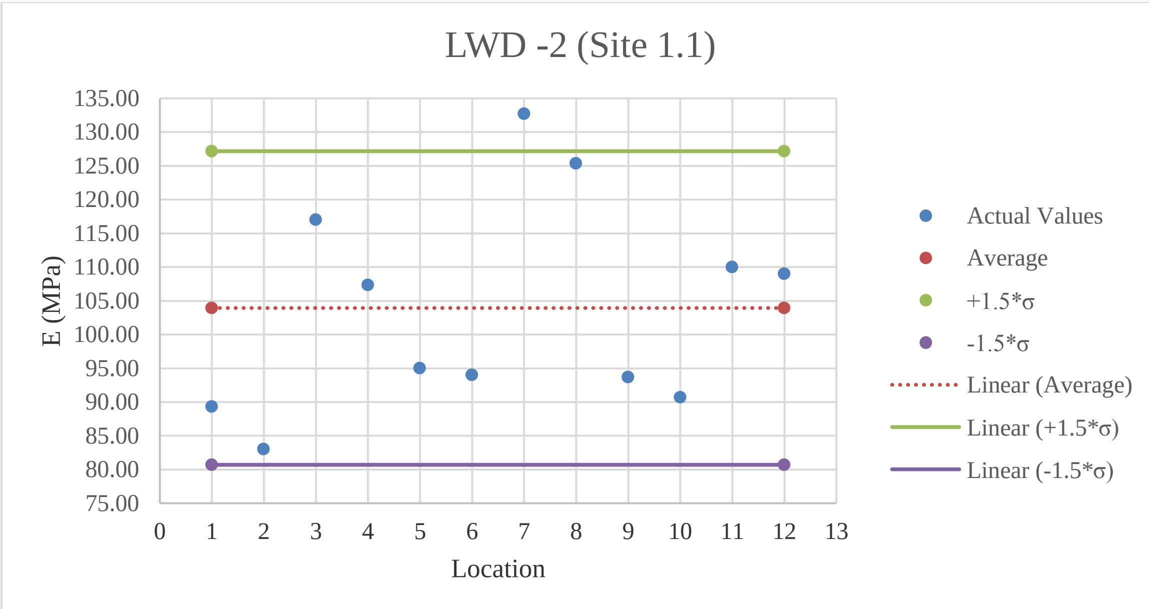


Figure E.1: LWD-2 testing results on GAB material

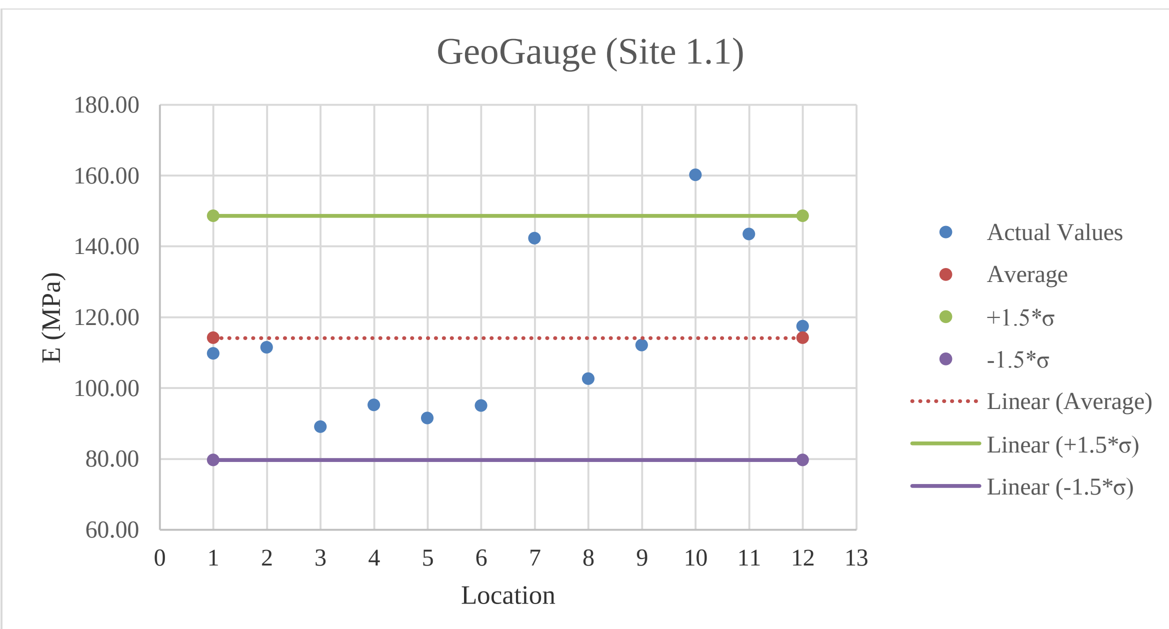


Figure E.2: GeoGauge testing results on GAB material

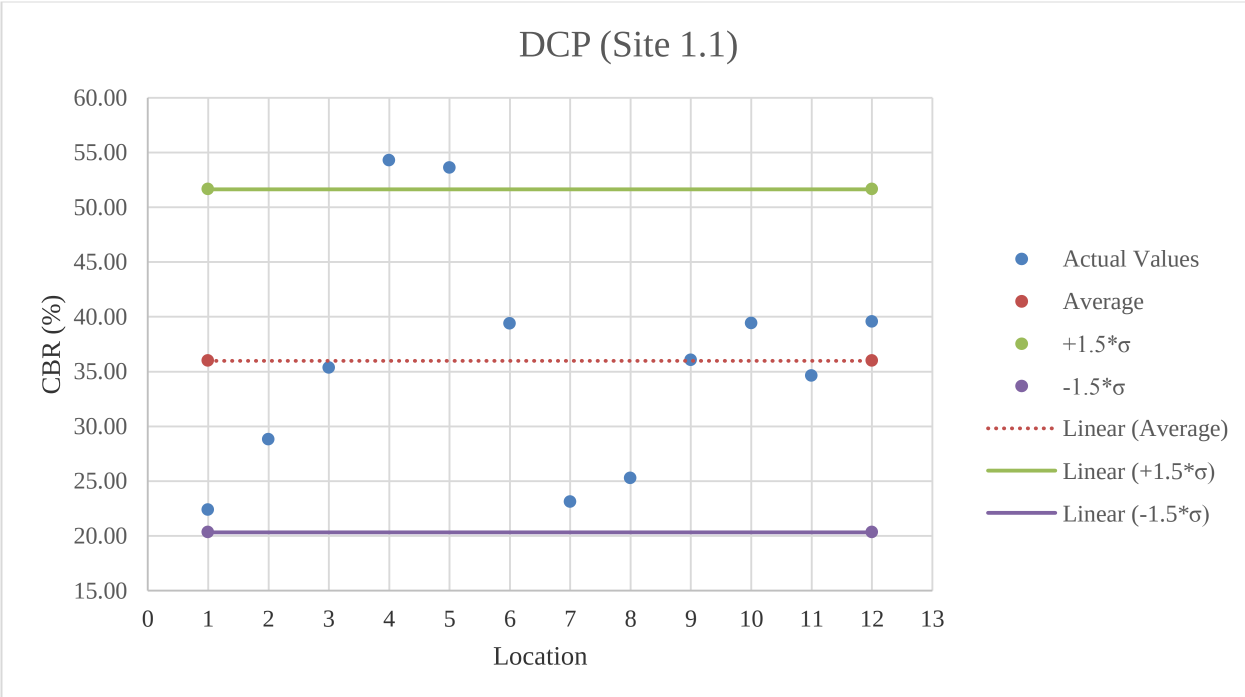


Figure E.3: DCP testing results on GAB material

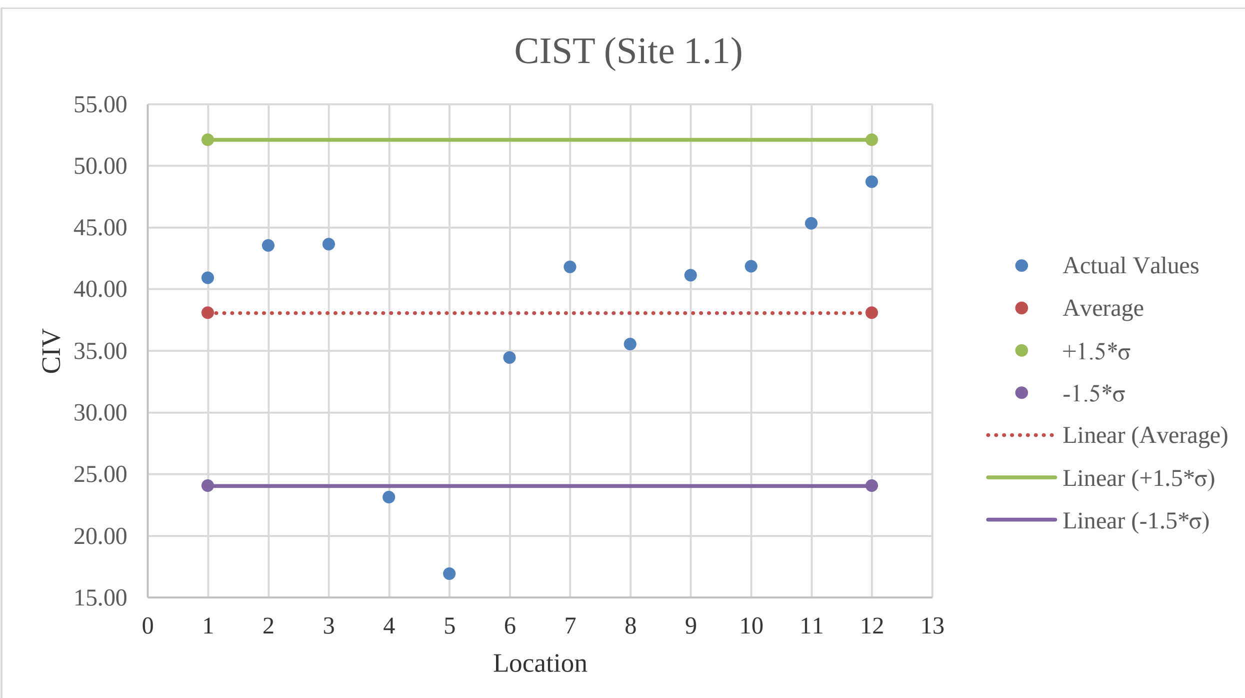


Figure E.4: CIST testing results on GAB material

E.1.2 Second Plate Load Test Location

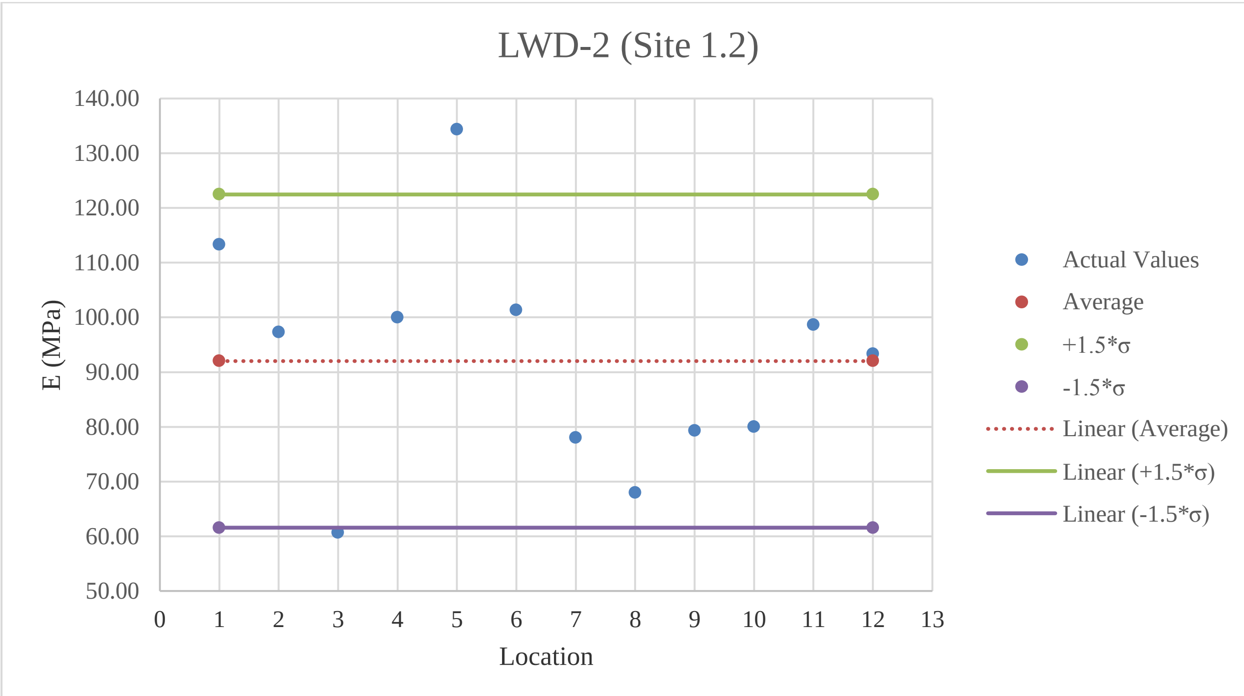


Figure E.5: LWD-2 testing results on GAB material

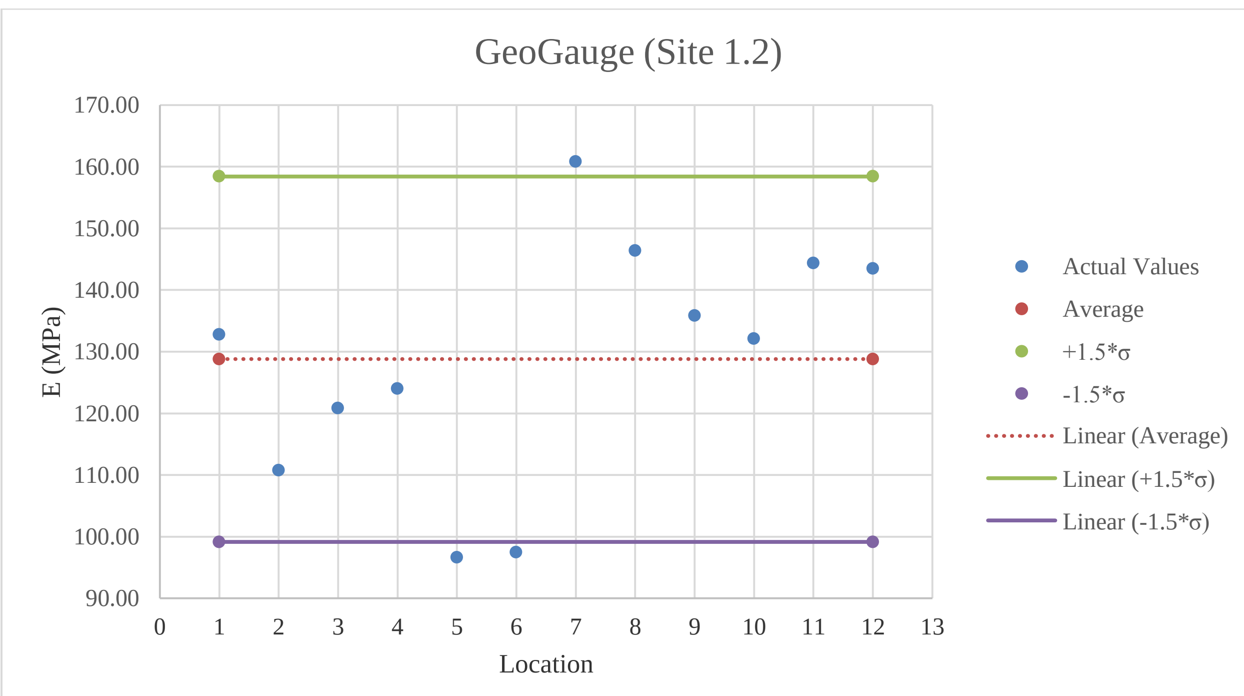


Figure E.6: GeoGauge testing results on GAB material

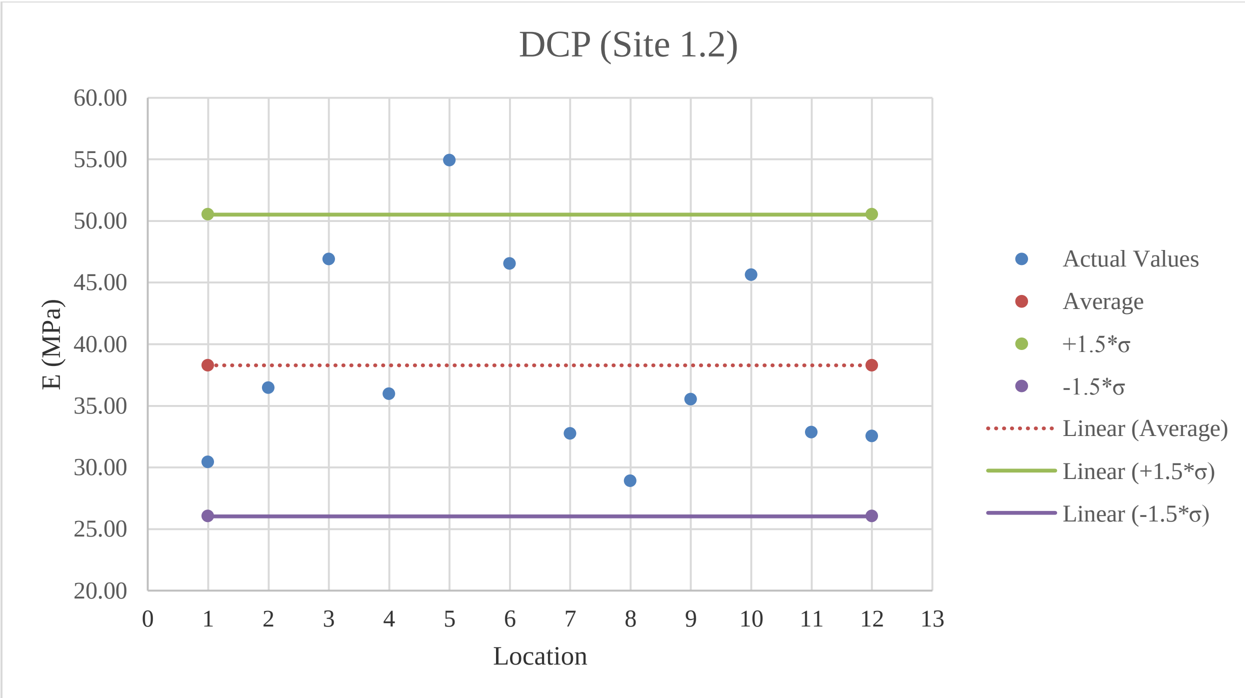


Figure E.7: DCP testing results on GAB material

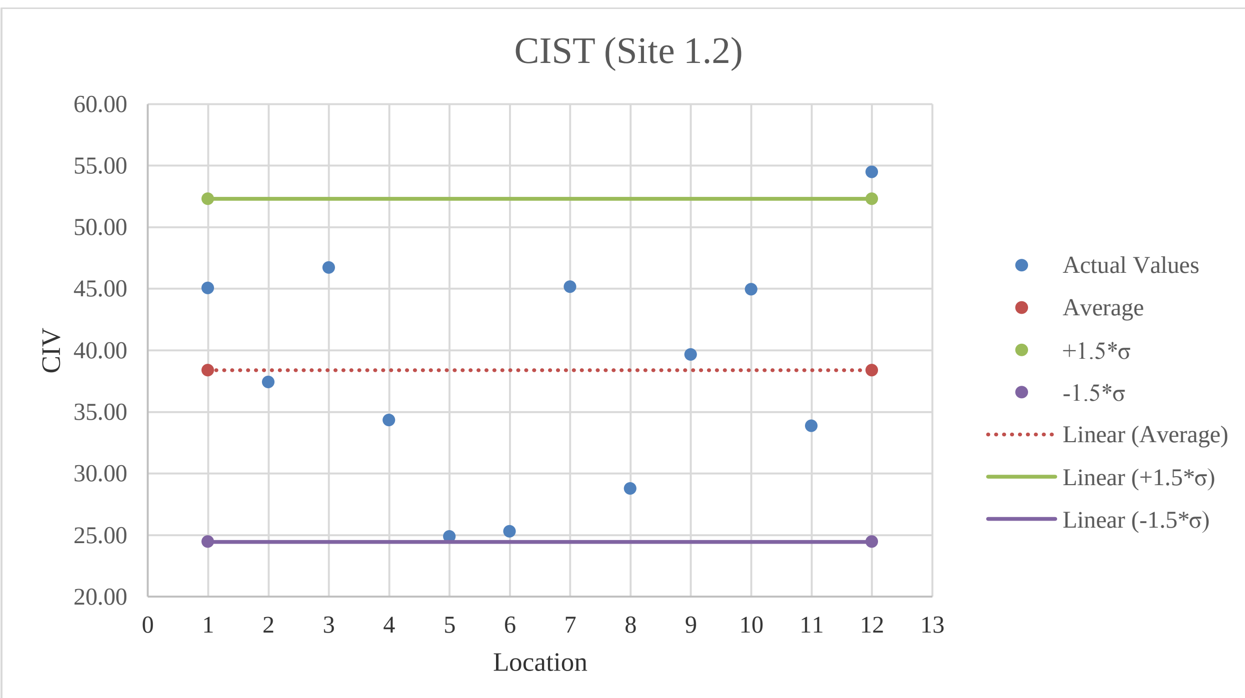


Figure E.8: CIST testing results on GAB material

E.1.3 Third Plate Load Test Location

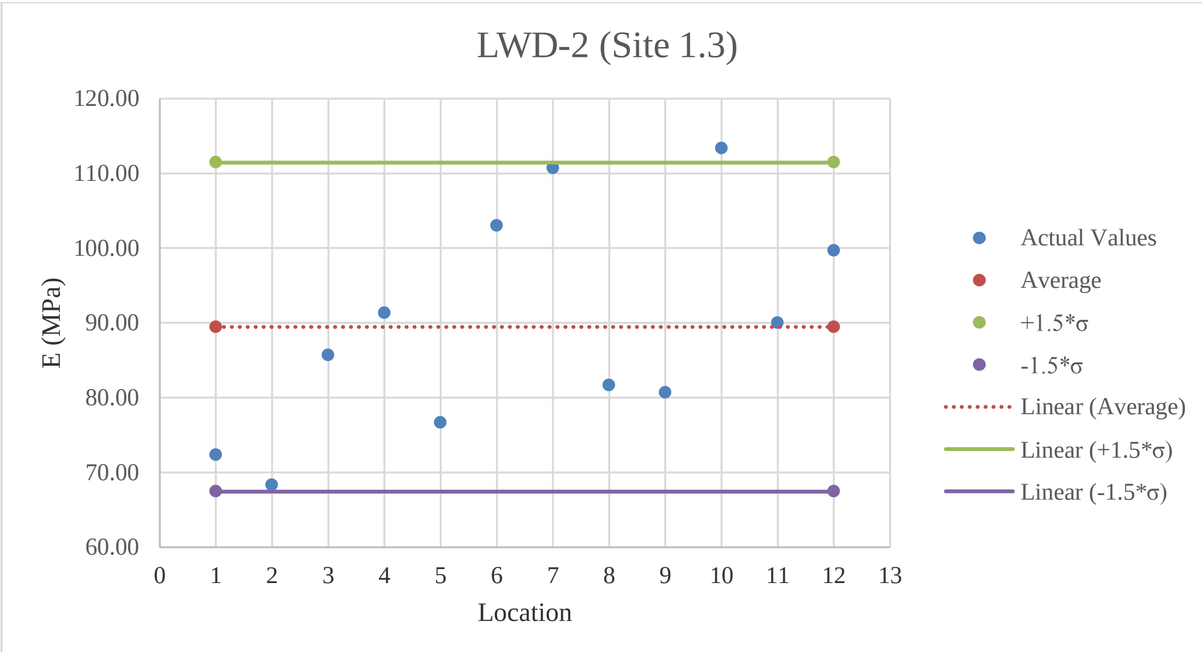


Figure E.9: LWD-2 testing results on GAB material

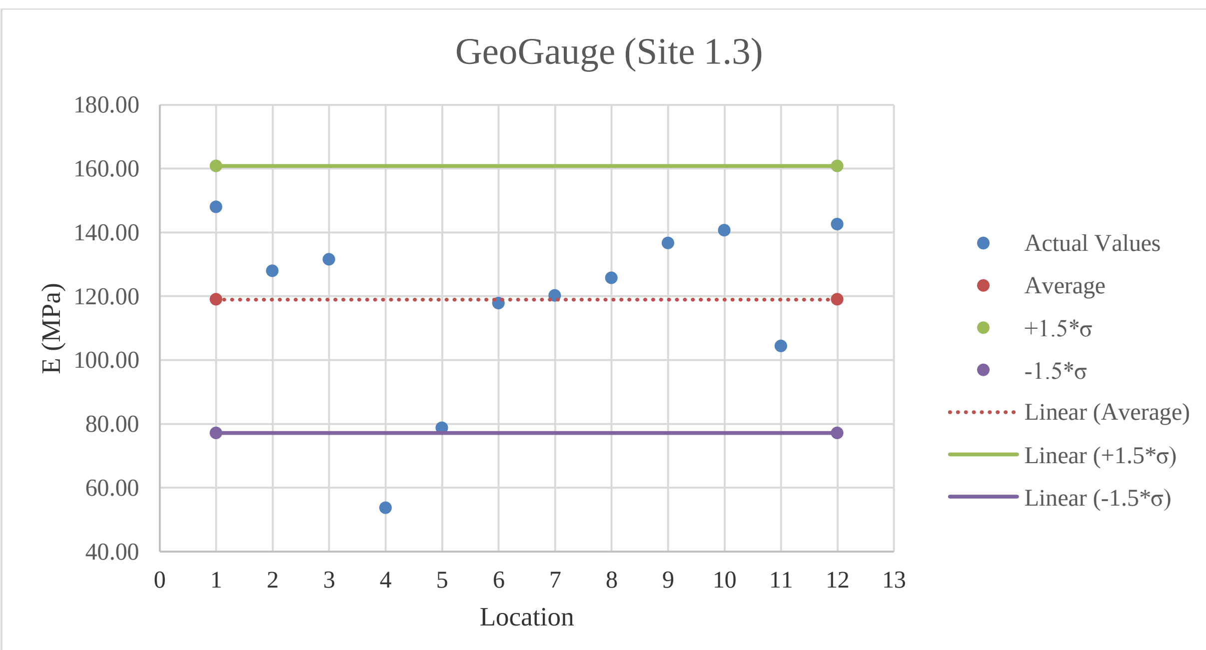


Figure E.10: GeoGauge testing results on GAB material

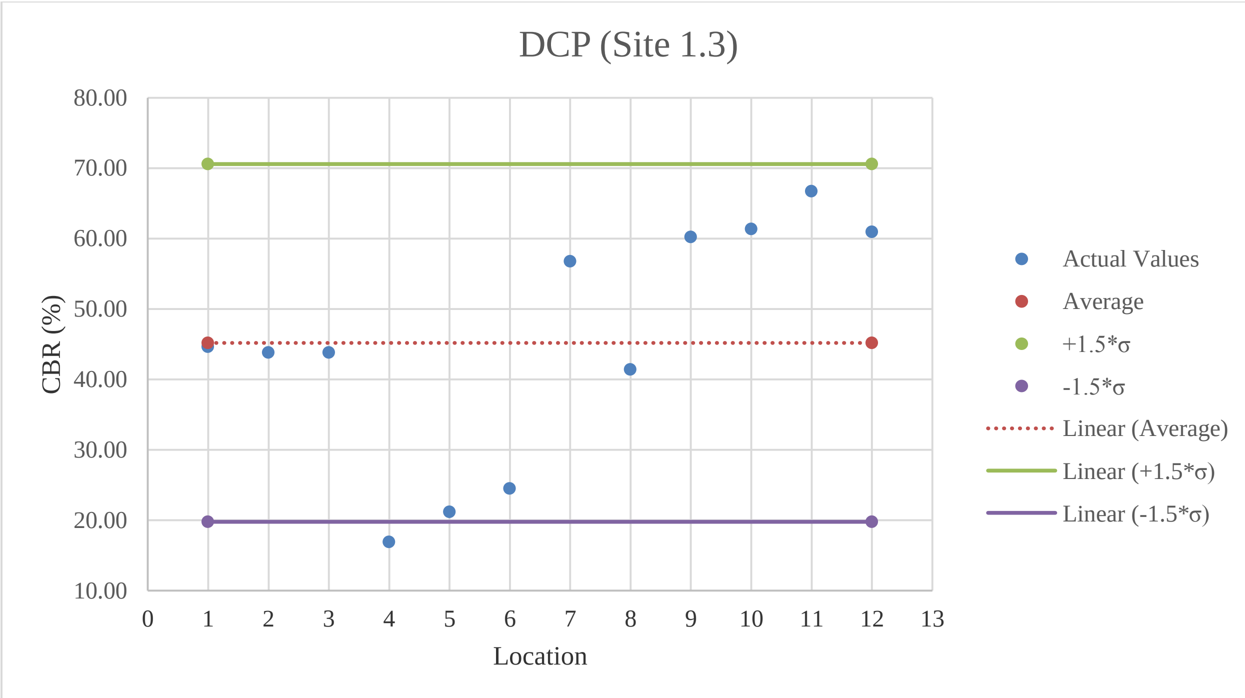


Figure E.11: DCP testing results on GAB material

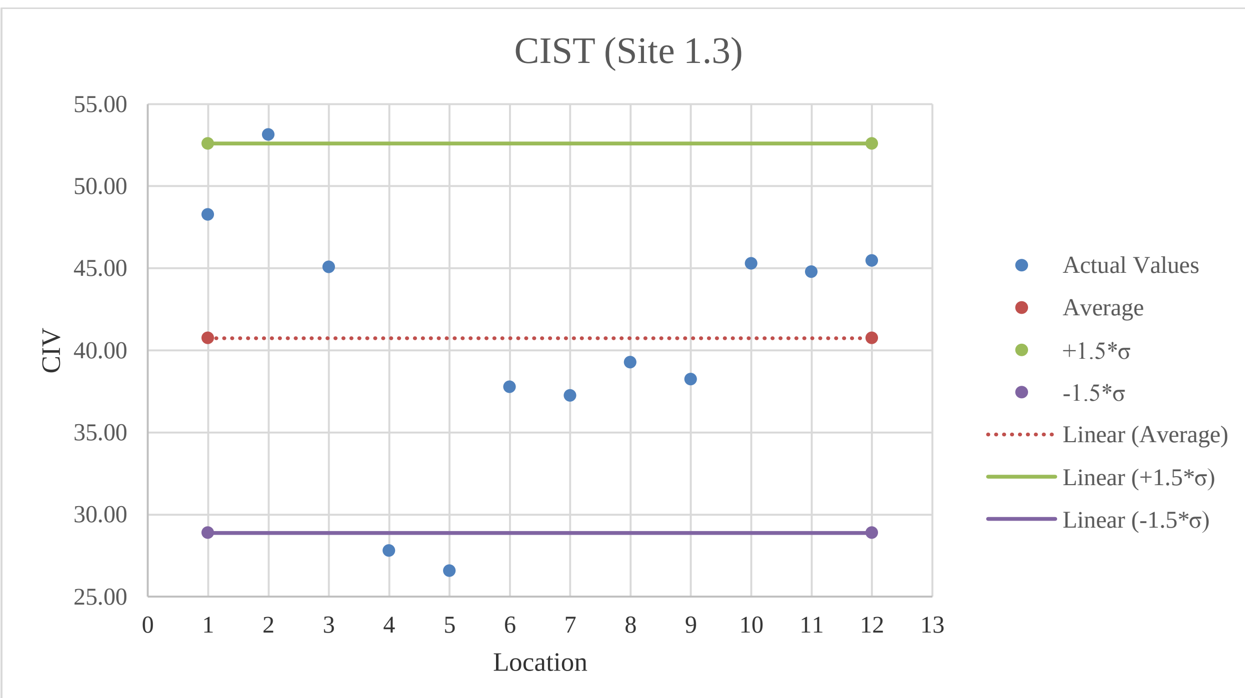


Figure E.12: CIST testing results on GAB material

E.2 US-301 (Site 2)

E.2.1 First Plate Load Test Location

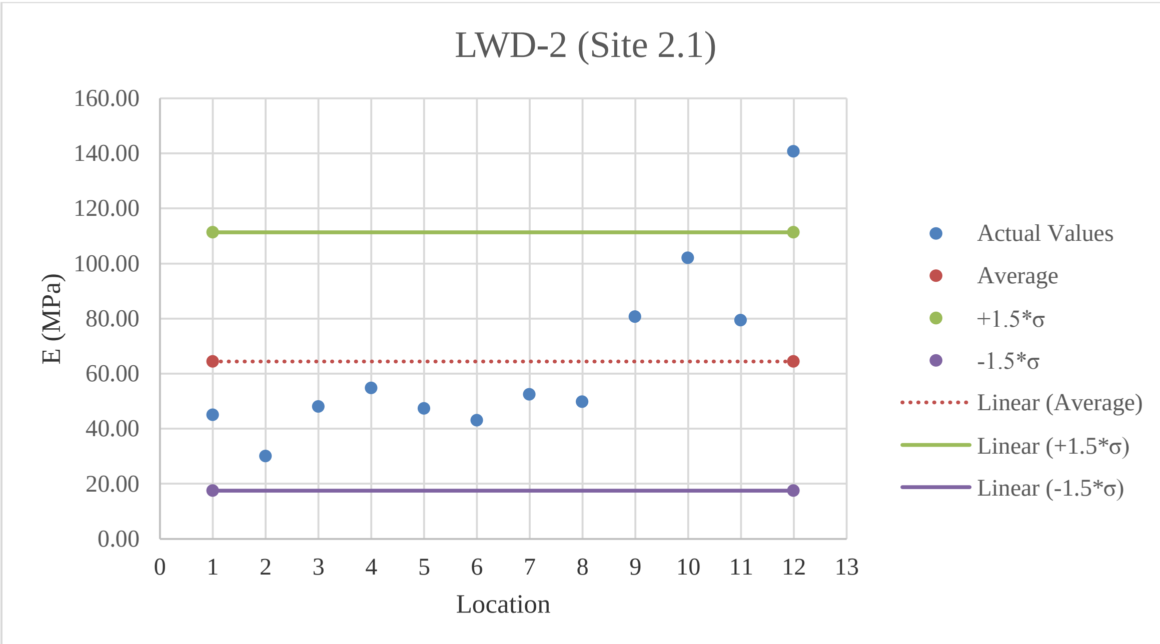


Figure E.13: LWD-2 testing results on A-2-4 material

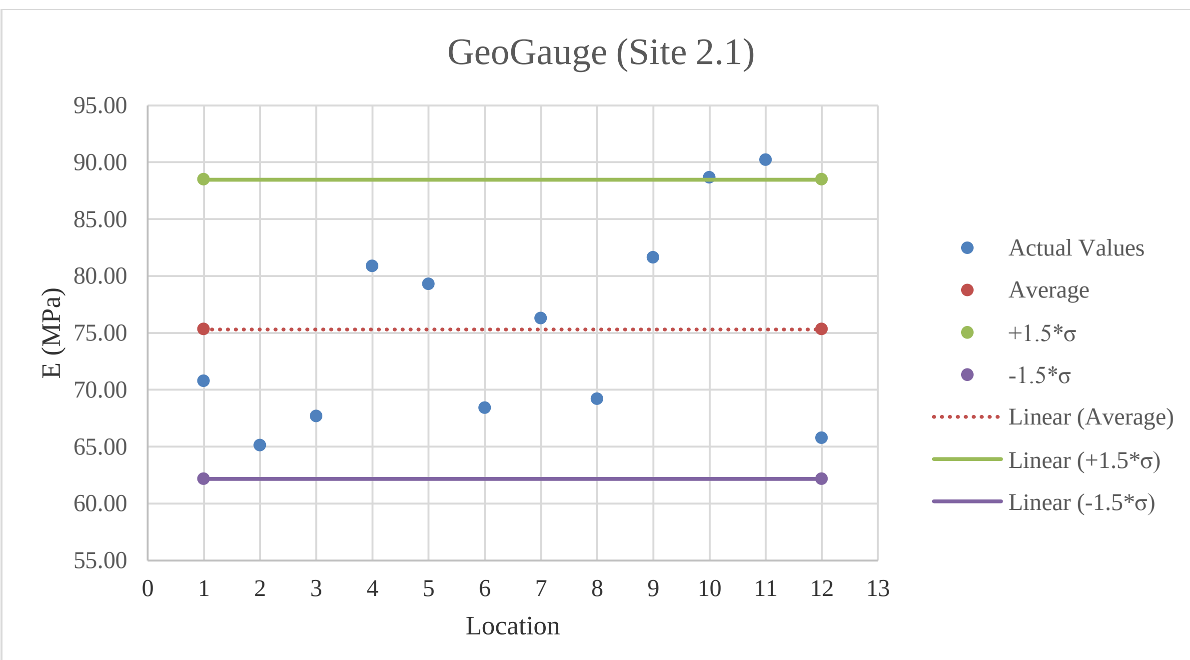


Figure E.14: GeoGauge testing results on A-2-4 material

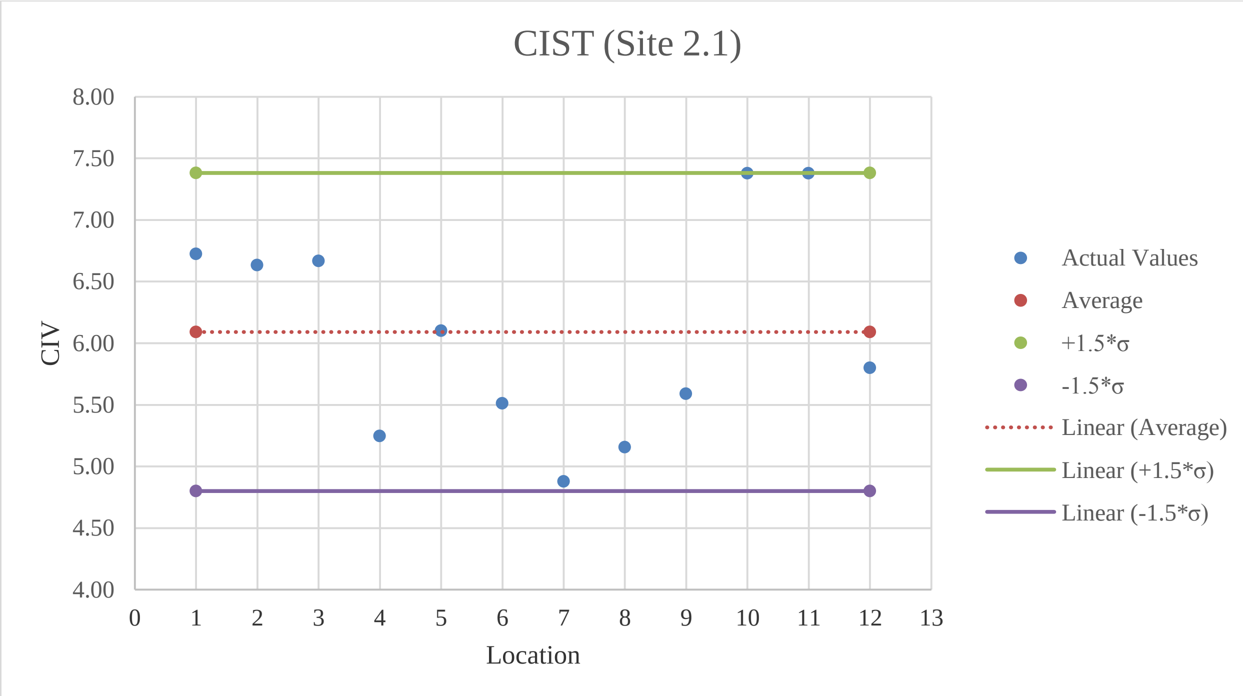


Figure E.15: CIST testing results on A-2-4 material

E.2.2 Second Plate Load Test Location

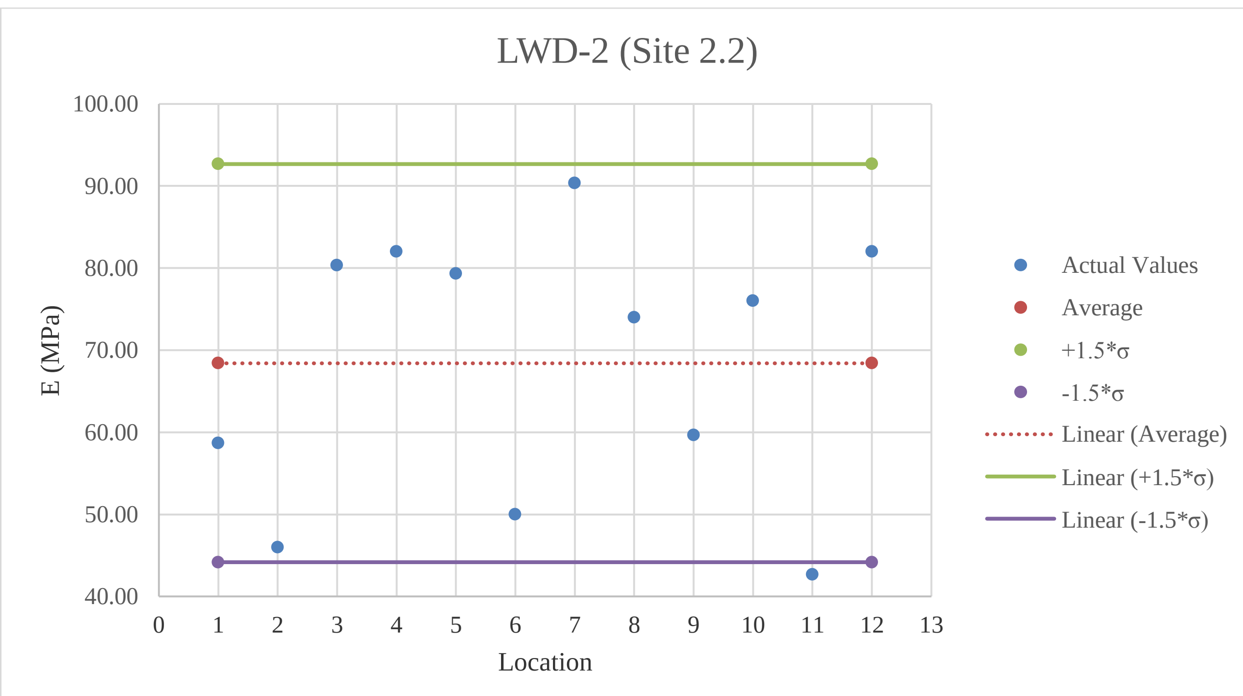


Figure E.16: LWD-2 testing results on A-2-4 material

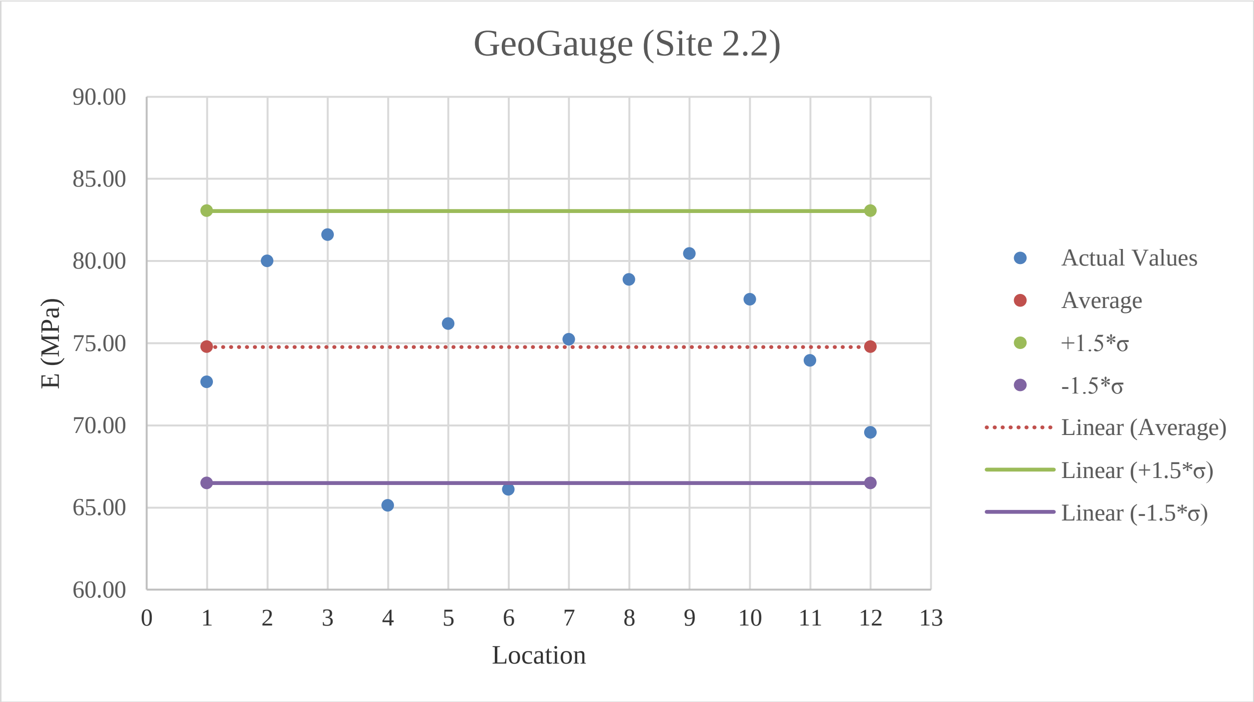


Figure E.17: GeoGauge testing results on A-2-4 material

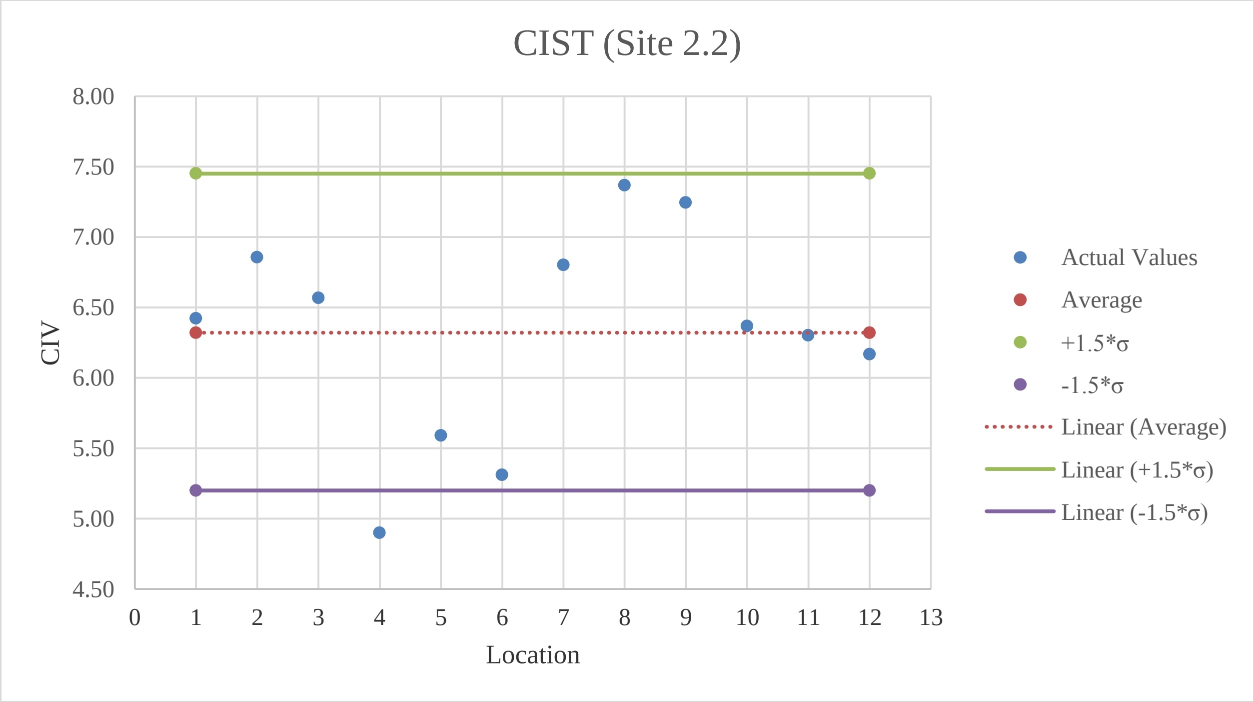


Figure E.18: CIST testing results on A-2-4 material

E.2.3 Third Plate Load Test Location

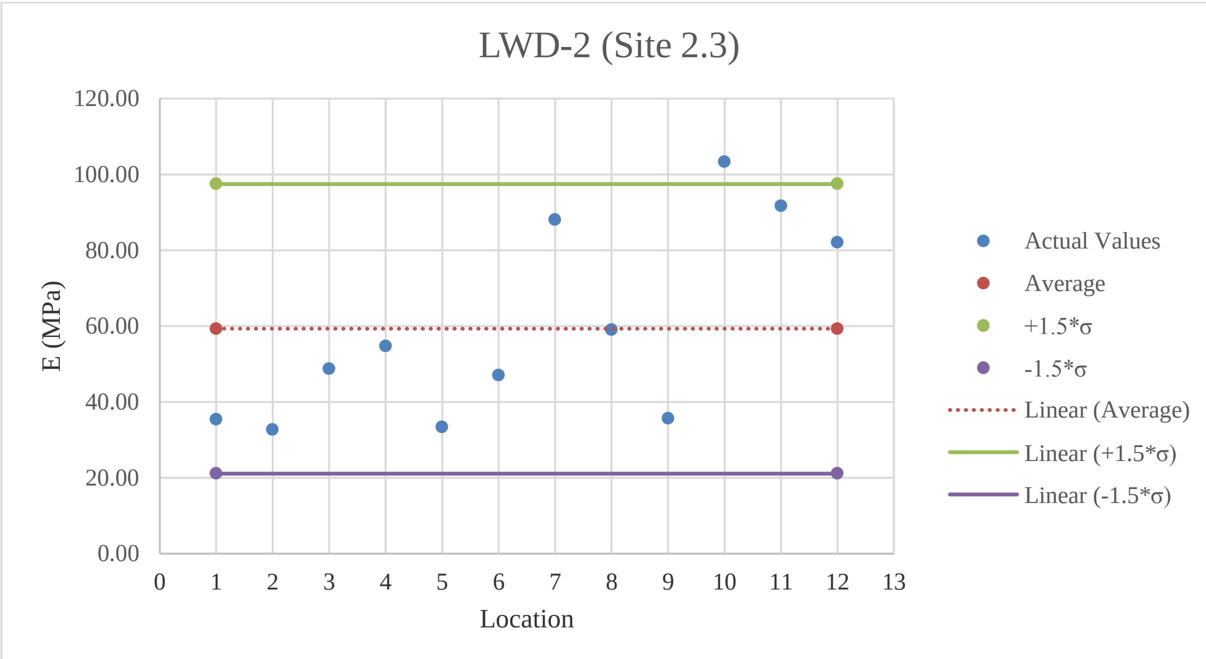


Figure E.19: LWD-2 testing results on A-2-4 material

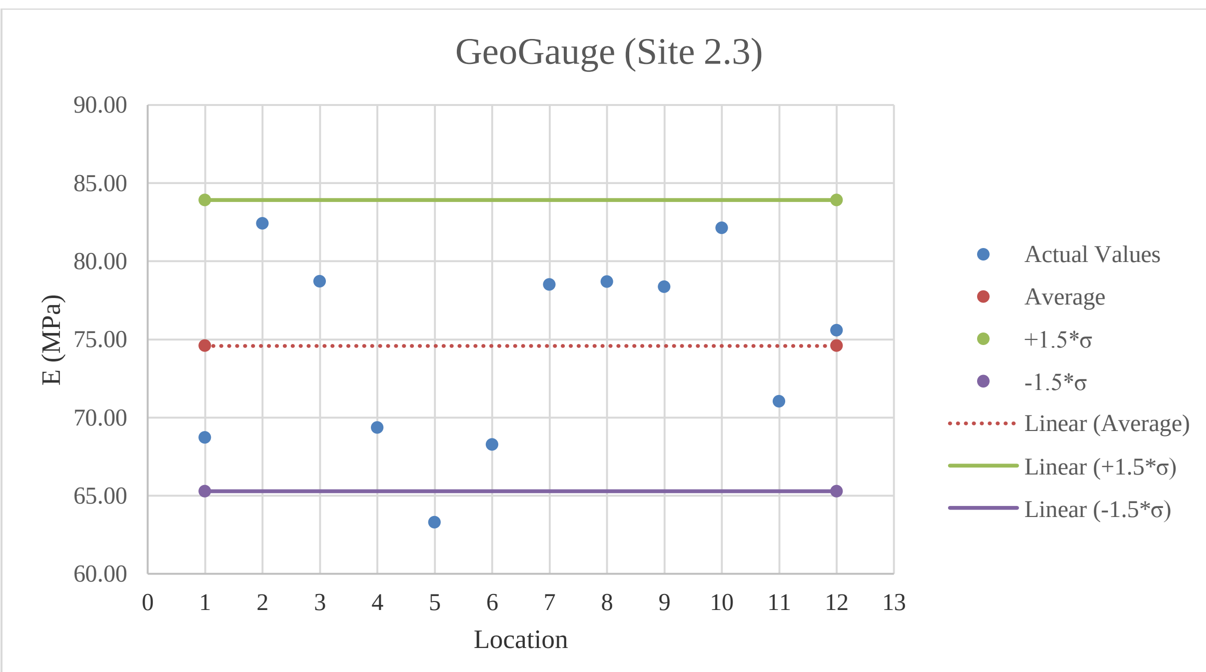


Figure E.20: GeoGauge testing results on A-2-4 material

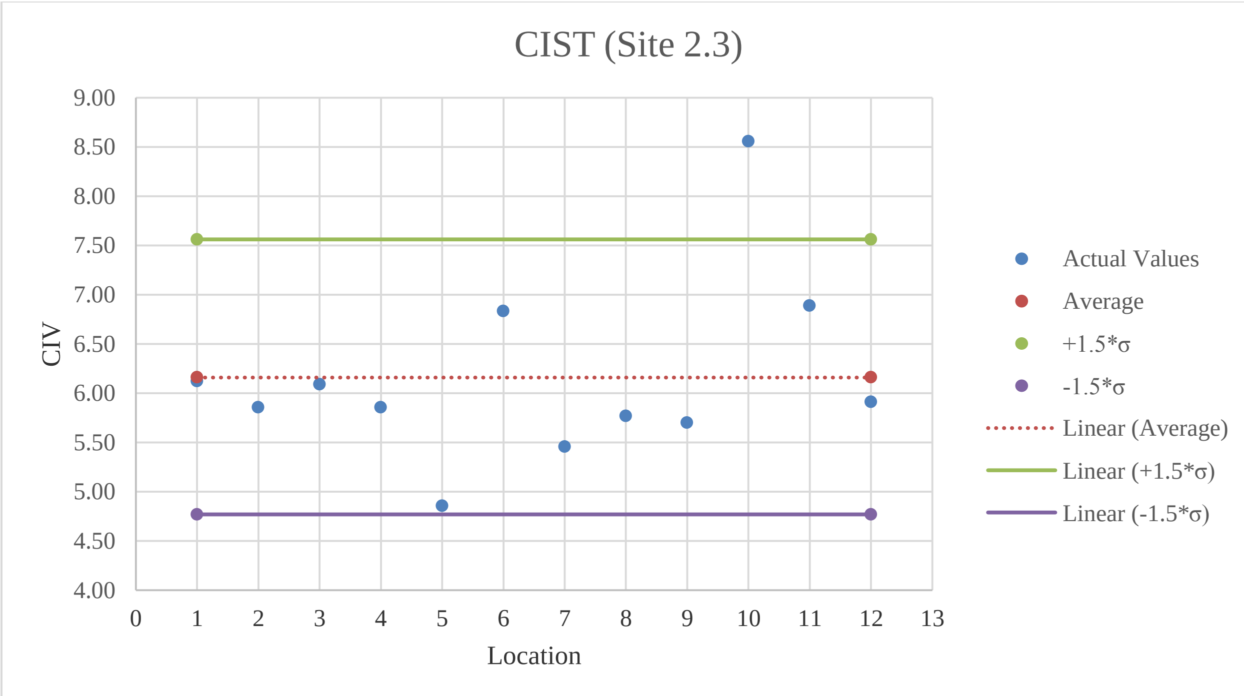


Figure E.21: CIST testing results on A-2-4 material

E.3 SR-9B (Site 3)

E.3.1 First Plate Load Test Location

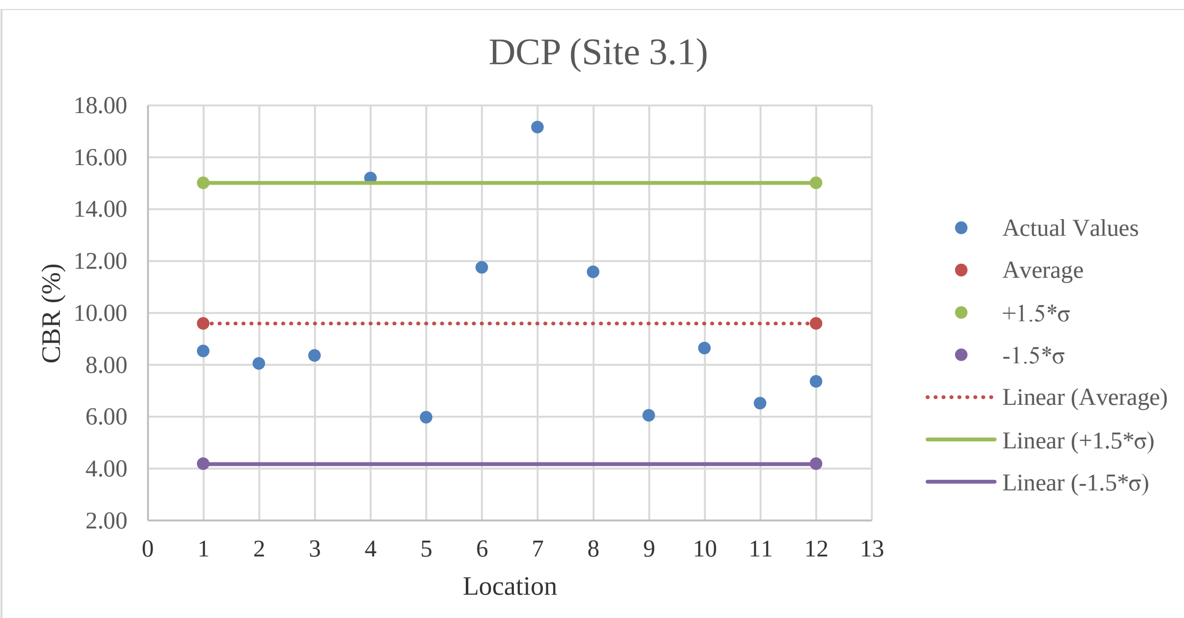


Figure E.22: DCP testing results on A-3 material

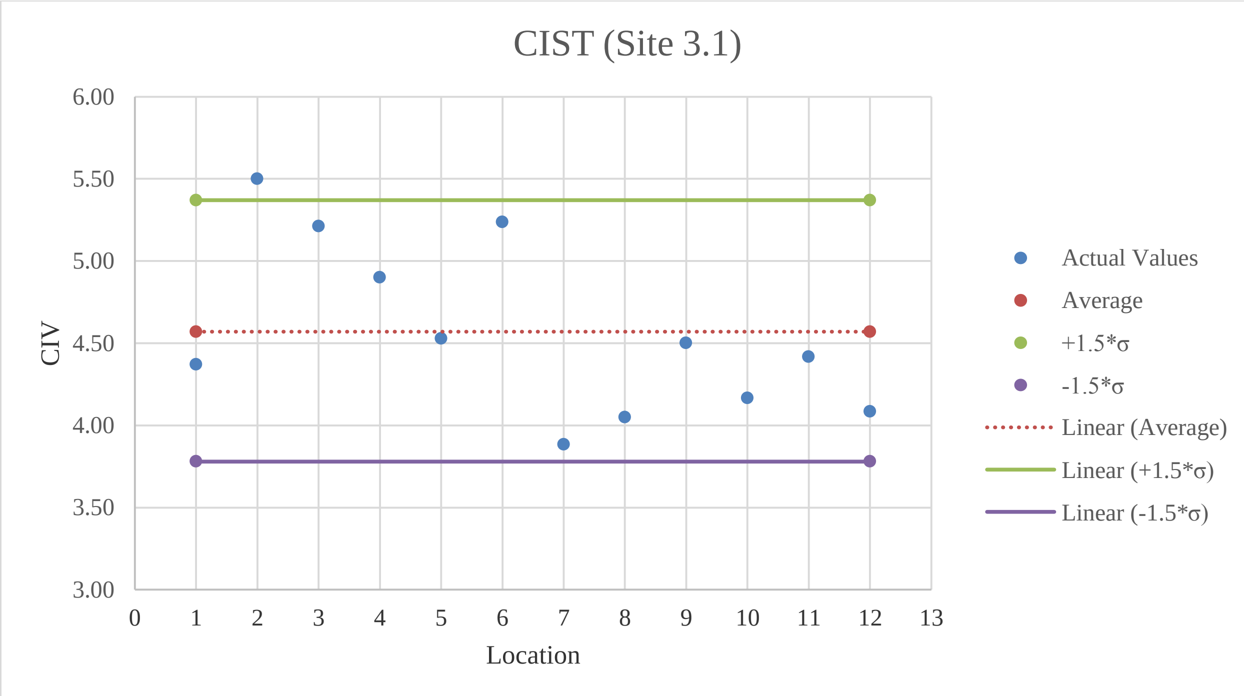


Figure E.23: CIST testing results on A-3 material

E.3.2 Second Plate Load Test Location

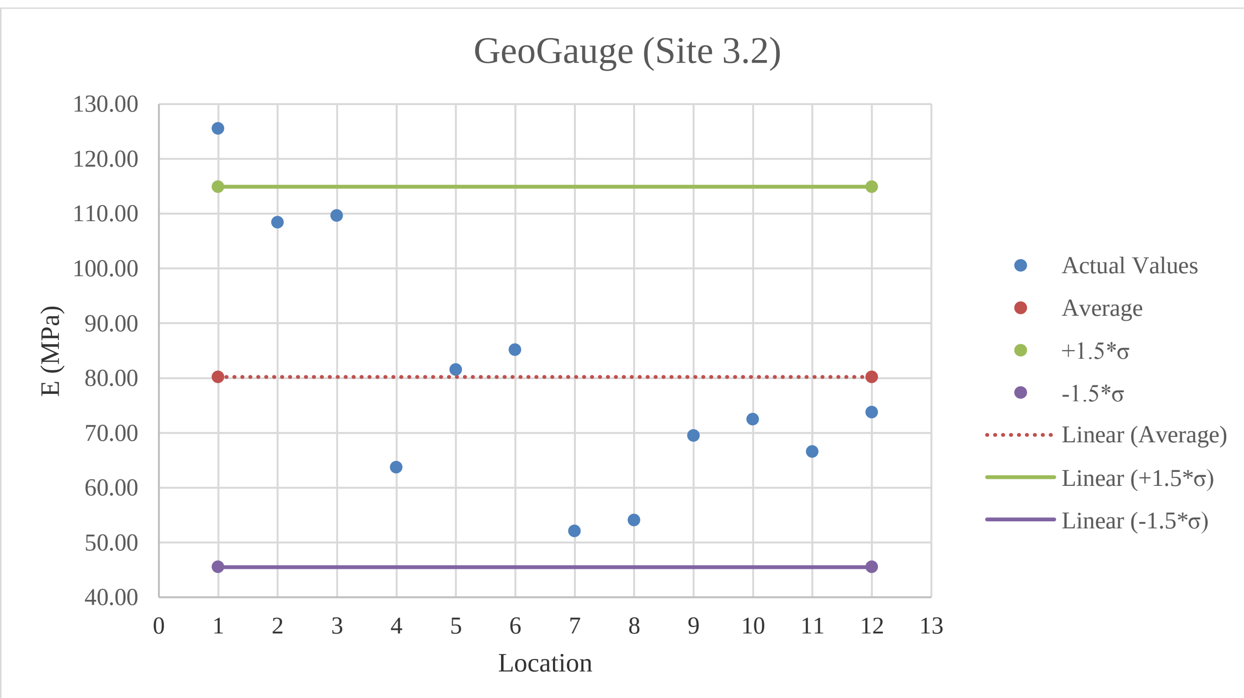


Figure E.24: GeoGauge testing results on A-3 material

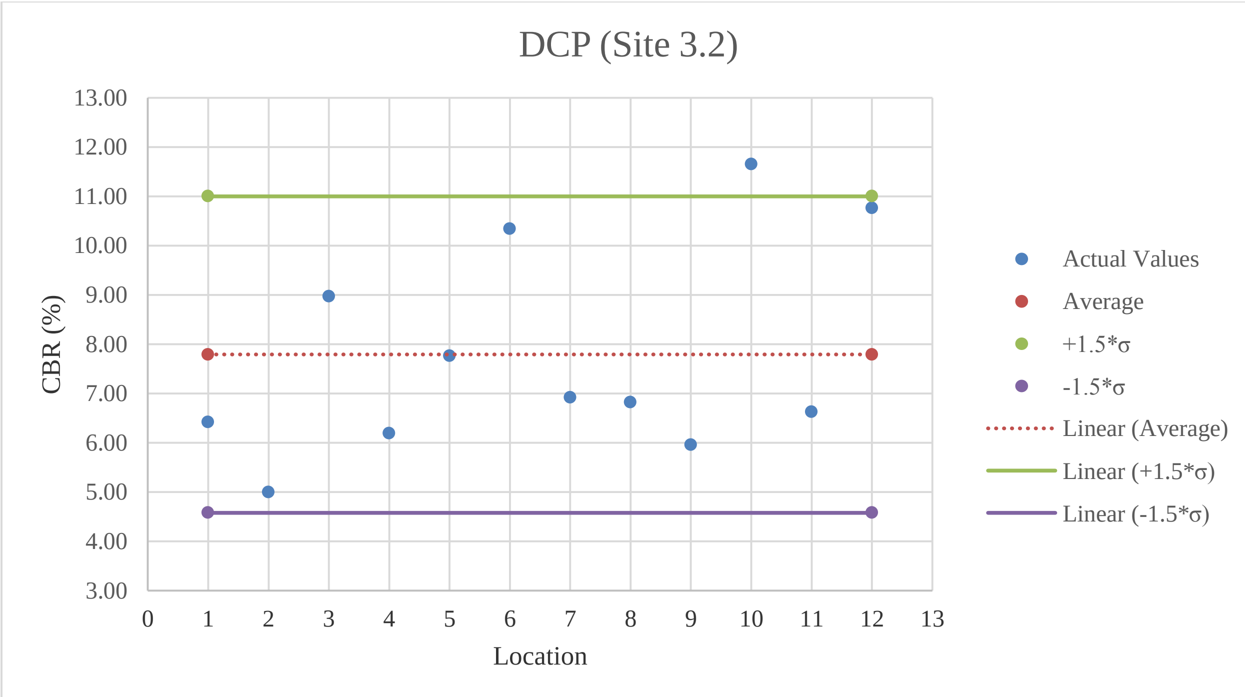


Figure E.25: DCP testing results on A-3 material

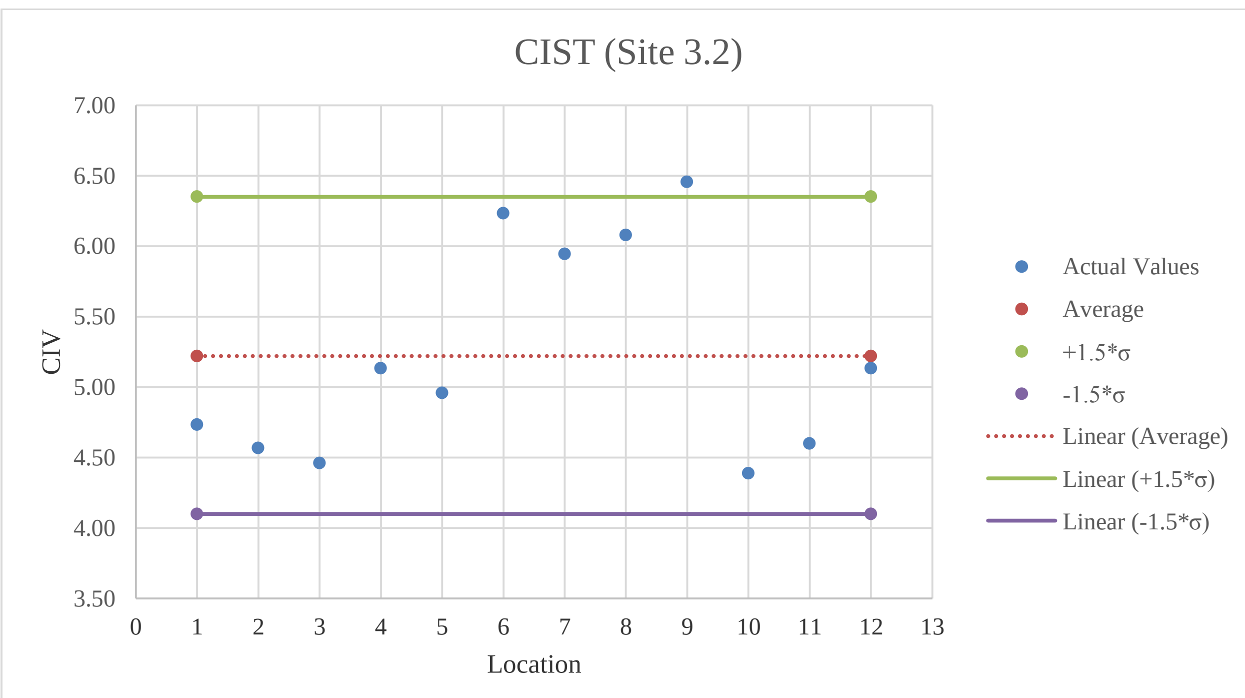


Figure E.26: CIST testing results on A-3 material

E.3.3 Third Plate Load Test Location

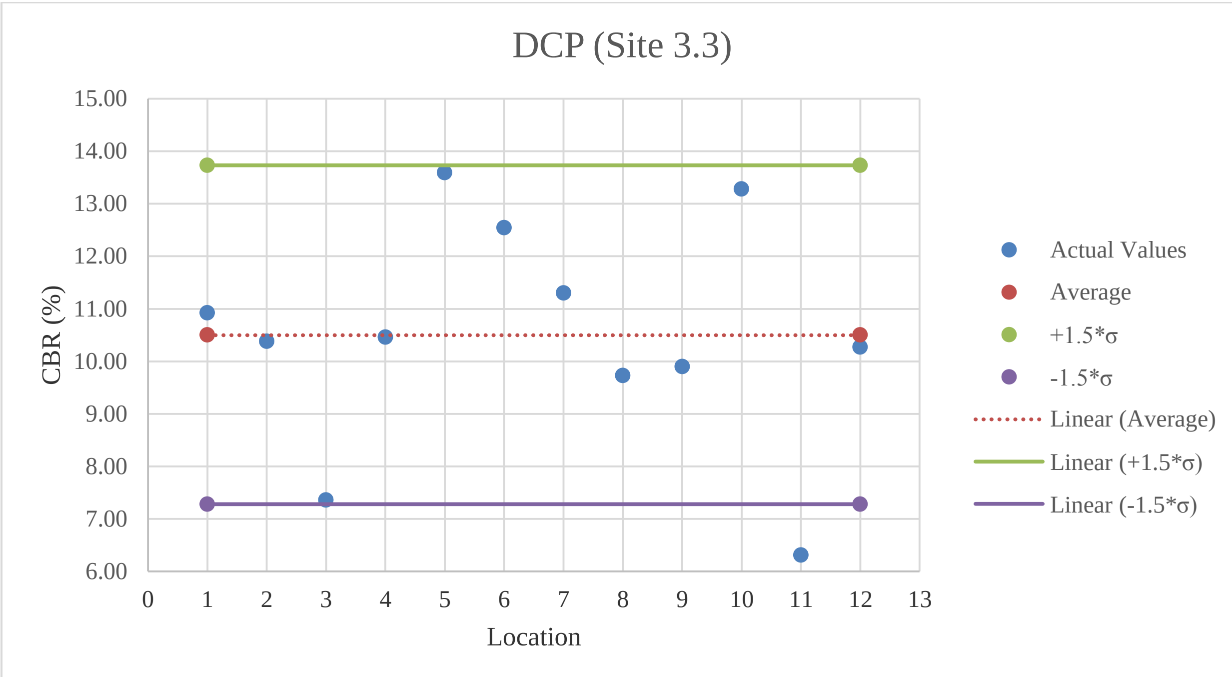


Figure E.27: DCP testing results on A-3 material

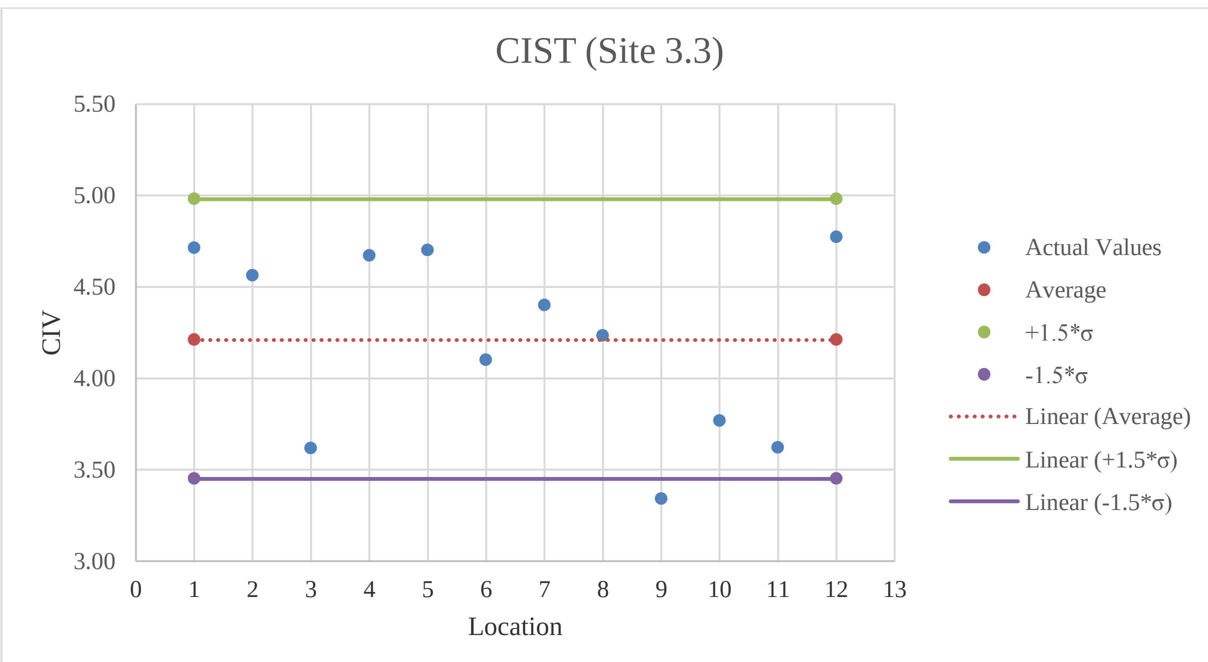


Figure E.28: CIST testing results on A-3 material

E.4 CR-210 (Site 4)

E.4.1 First Plate Load Test Location

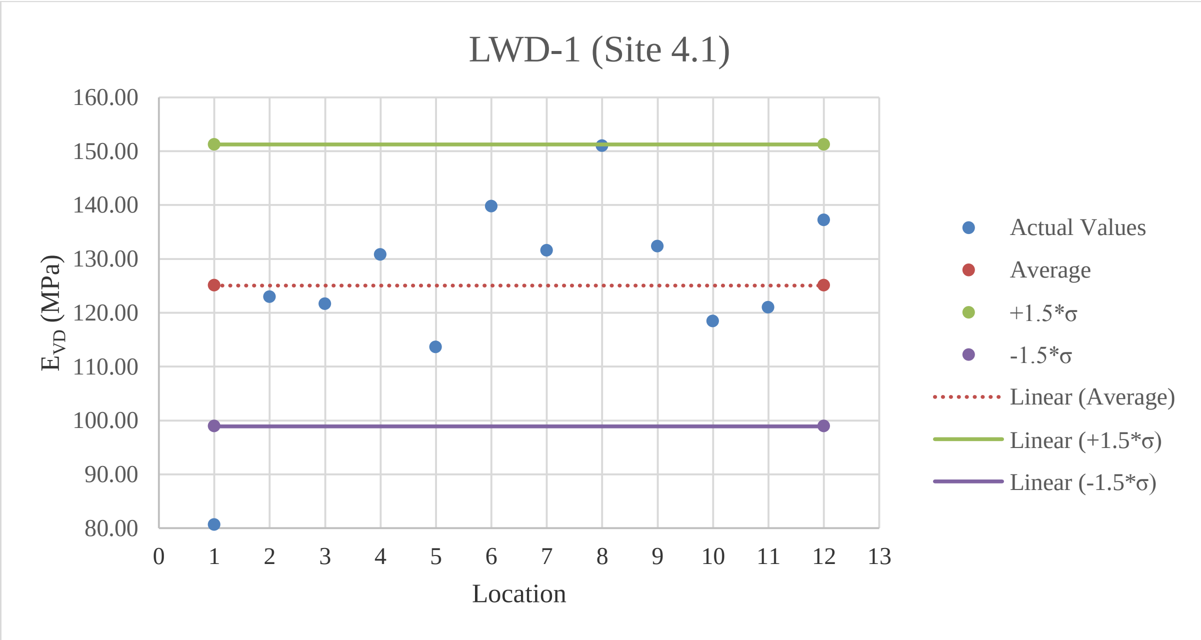


Figure E.29: LWD-1 testing results on stabilized subgrade

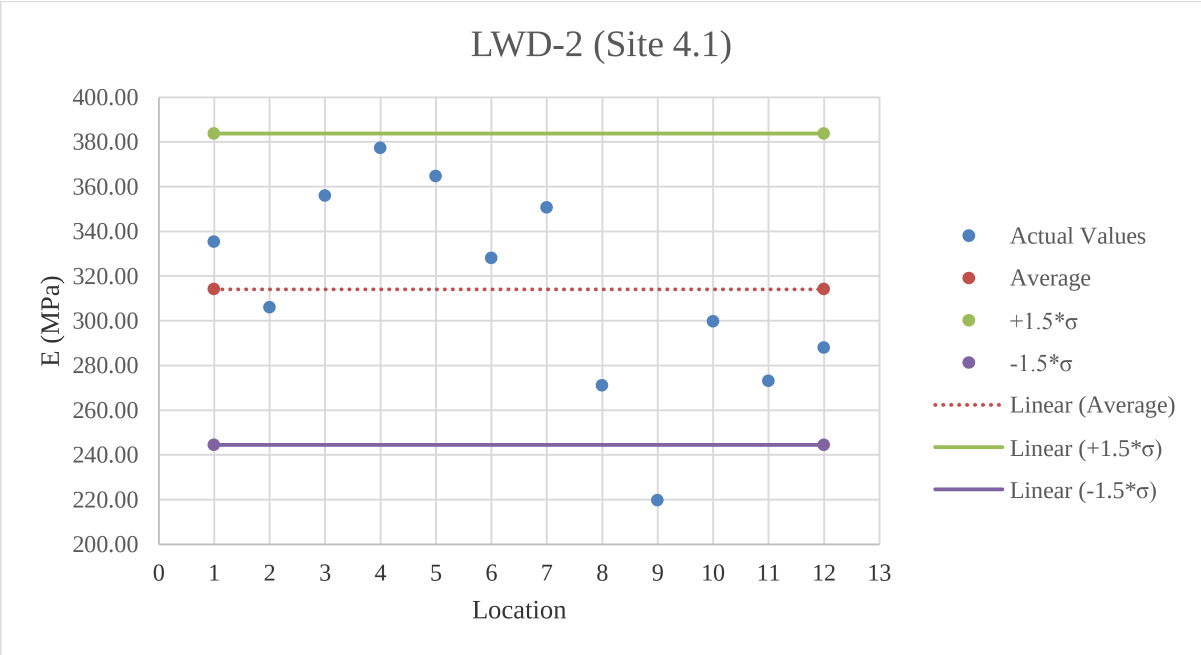


Figure E.30: LWD-2 testing results on stabilized subgrade

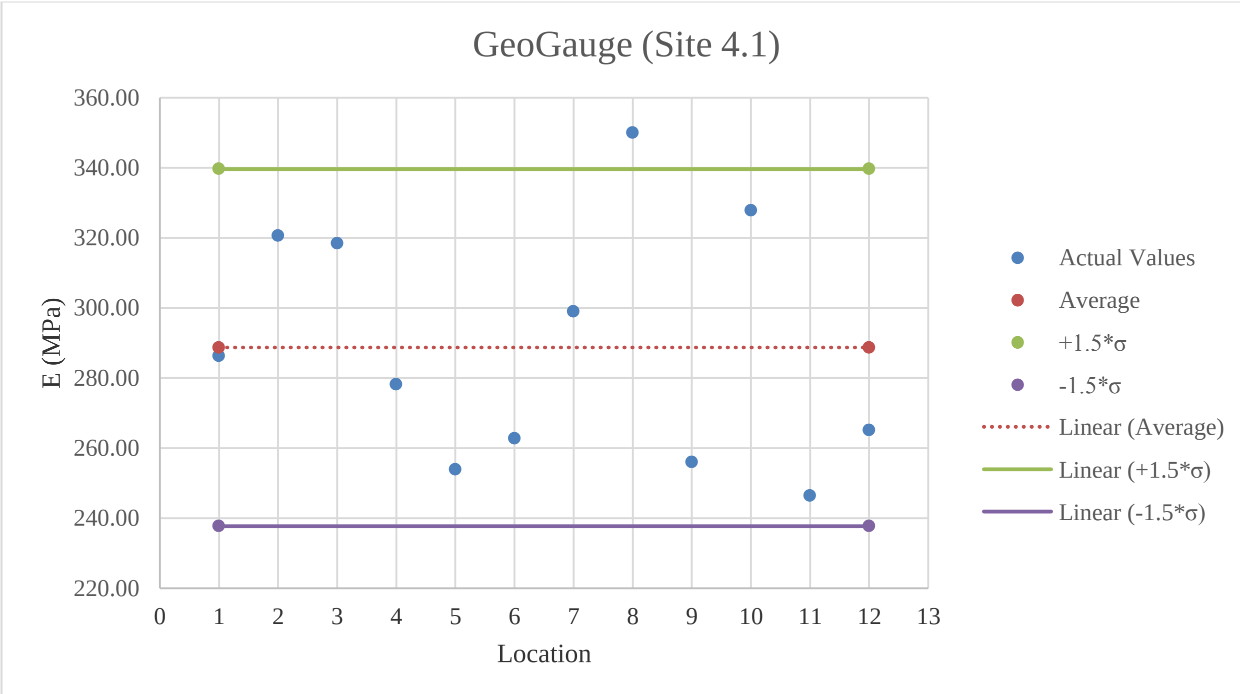


Figure E.31: GeoGauge testing results on stabilized subgrade

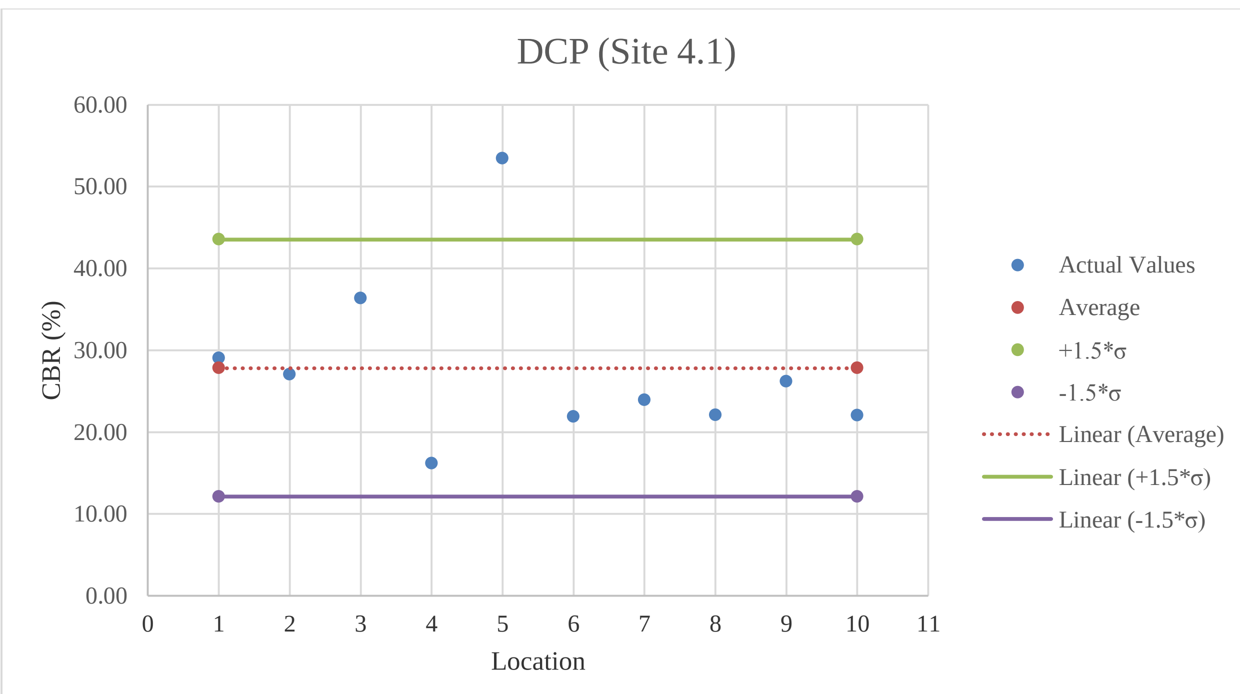


Figure E.32: DCP testing results on stabilized subgrade

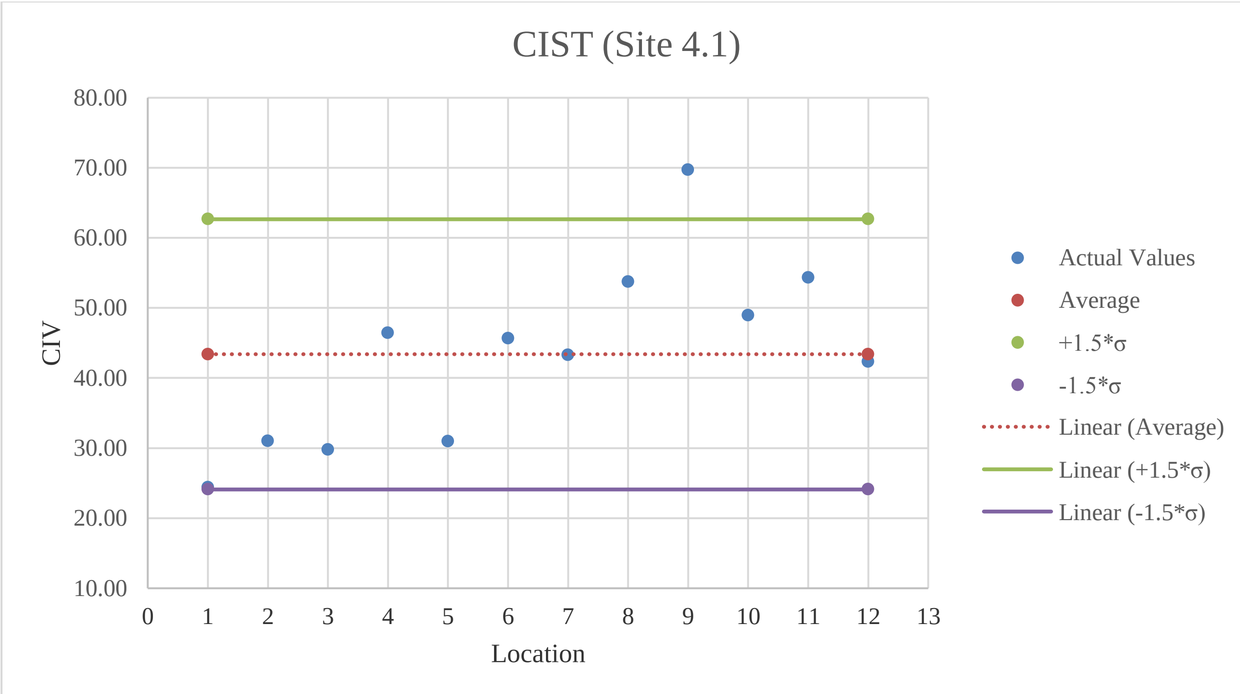


Figure E.33: CIST testing results on stabilized subgrade

E.4.2 Second Plate Load Test Location

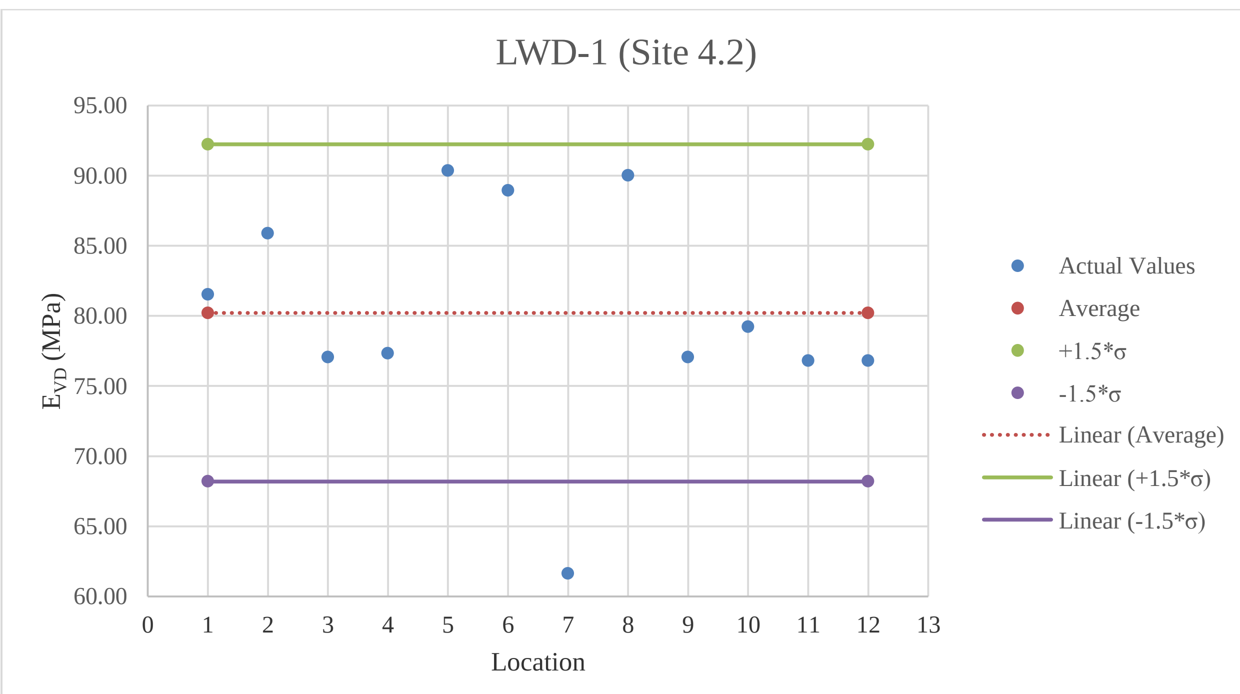


Figure E.34: LWD-1 testing results on stabilized subgrade

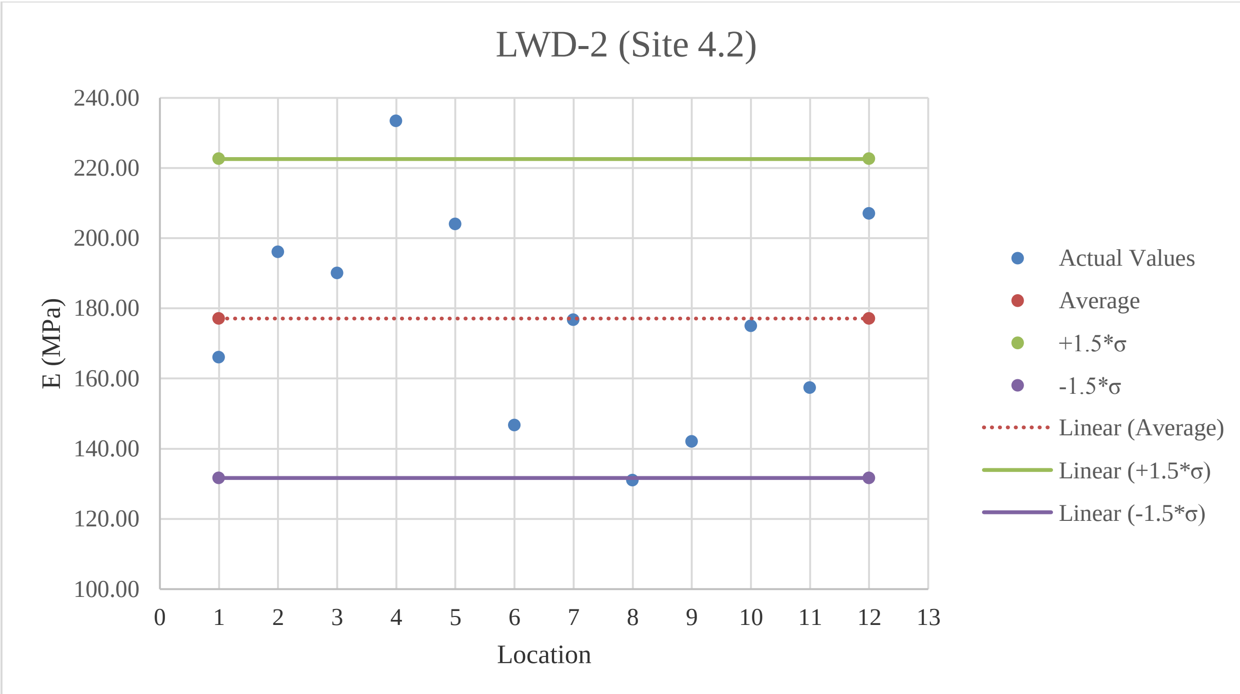


Figure E.35: LWD-2 testing results on stabilized subgrade

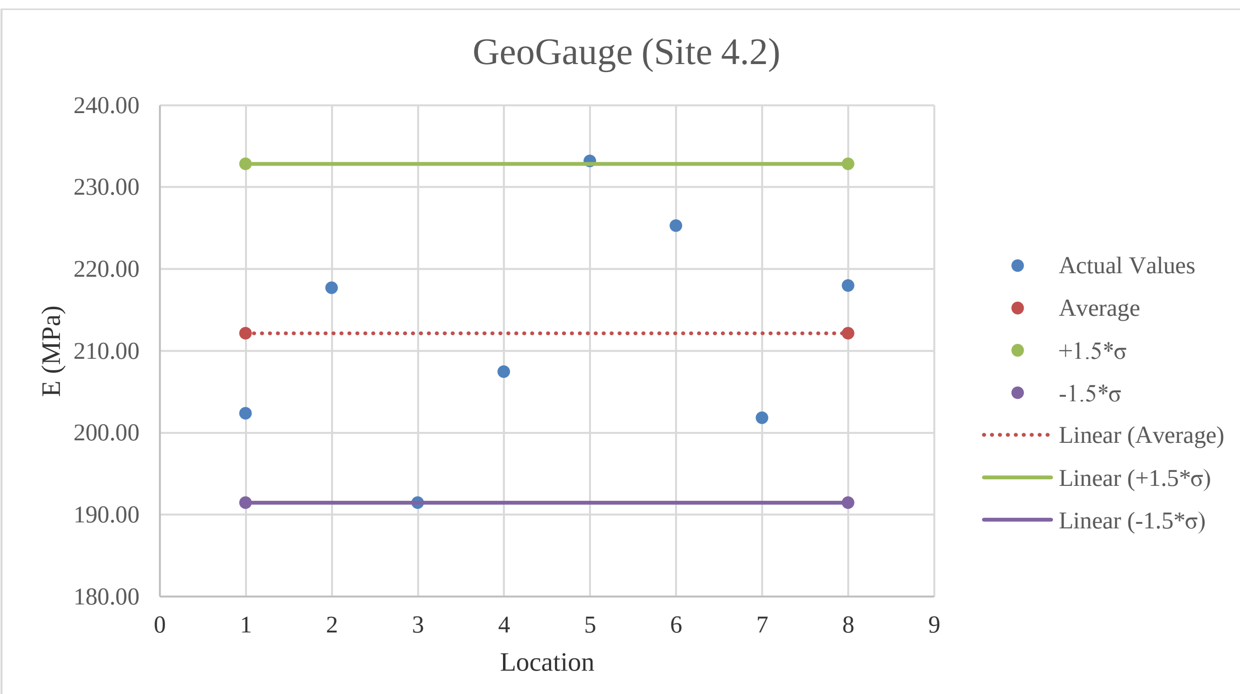


Figure E.36: GeoGauge testing results on stabilized subgrade

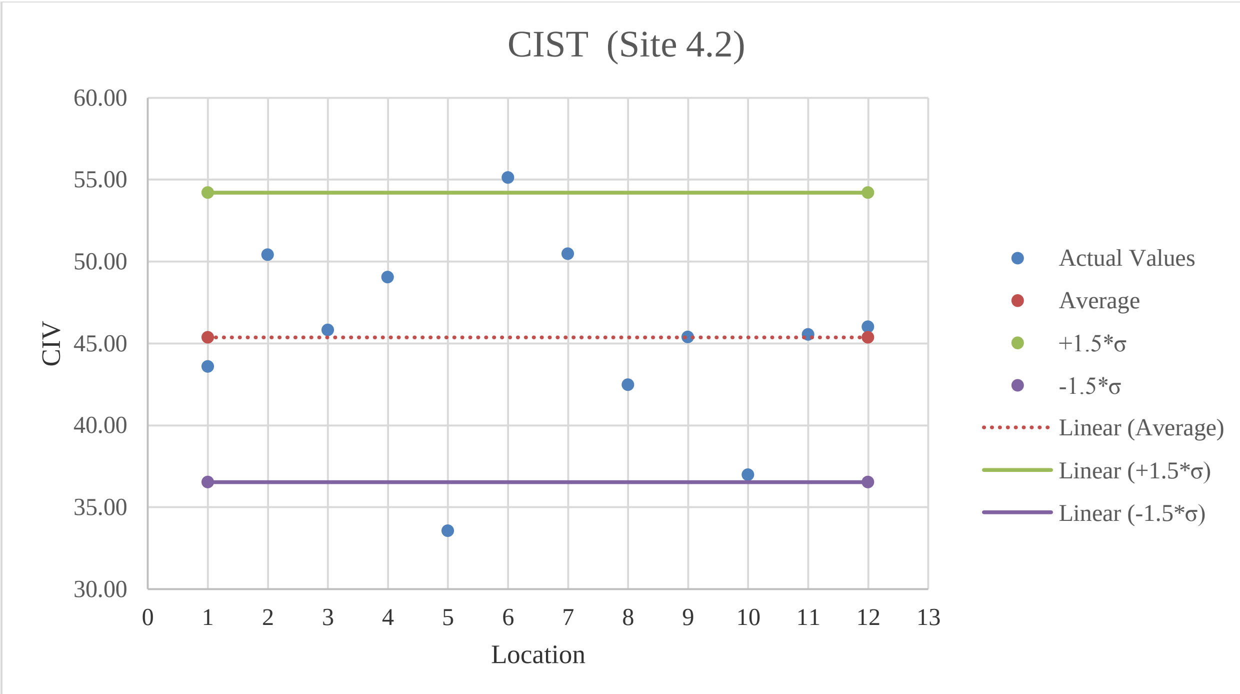


Figure E.37: CIST testing results on stabilized subgrade

E.4.3 Third Plate Load Test Location

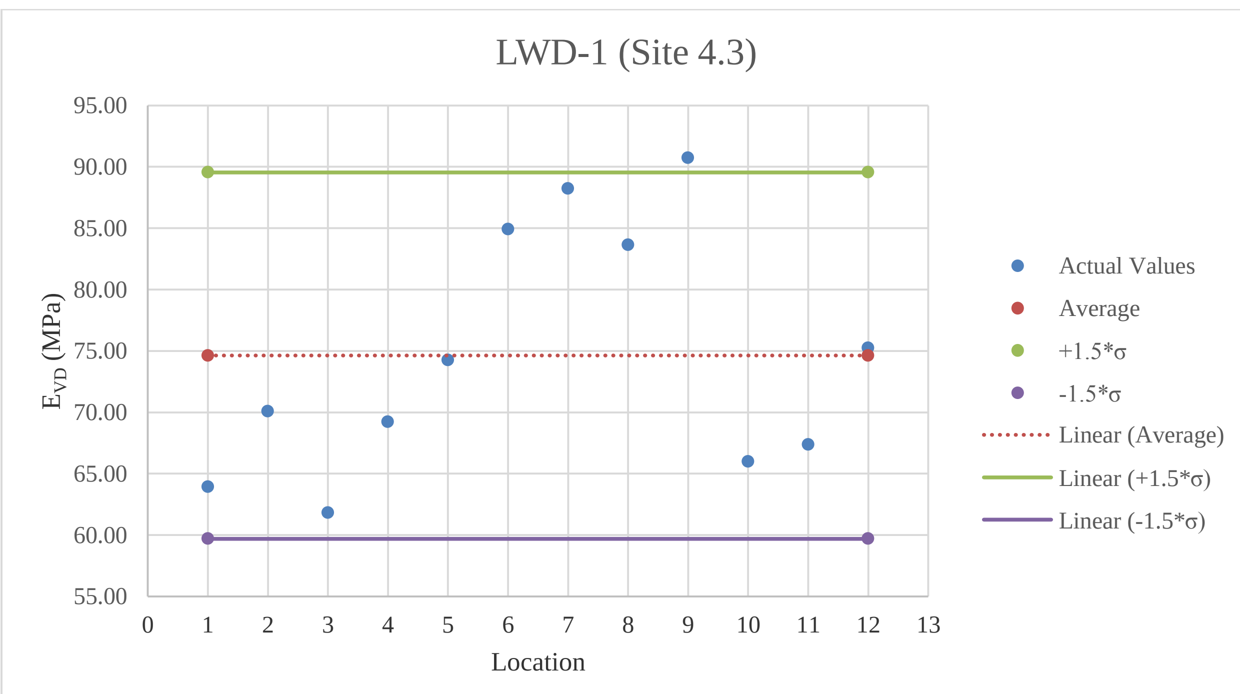


Figure E.38: LWD-1 testing results on stabilized subgrade

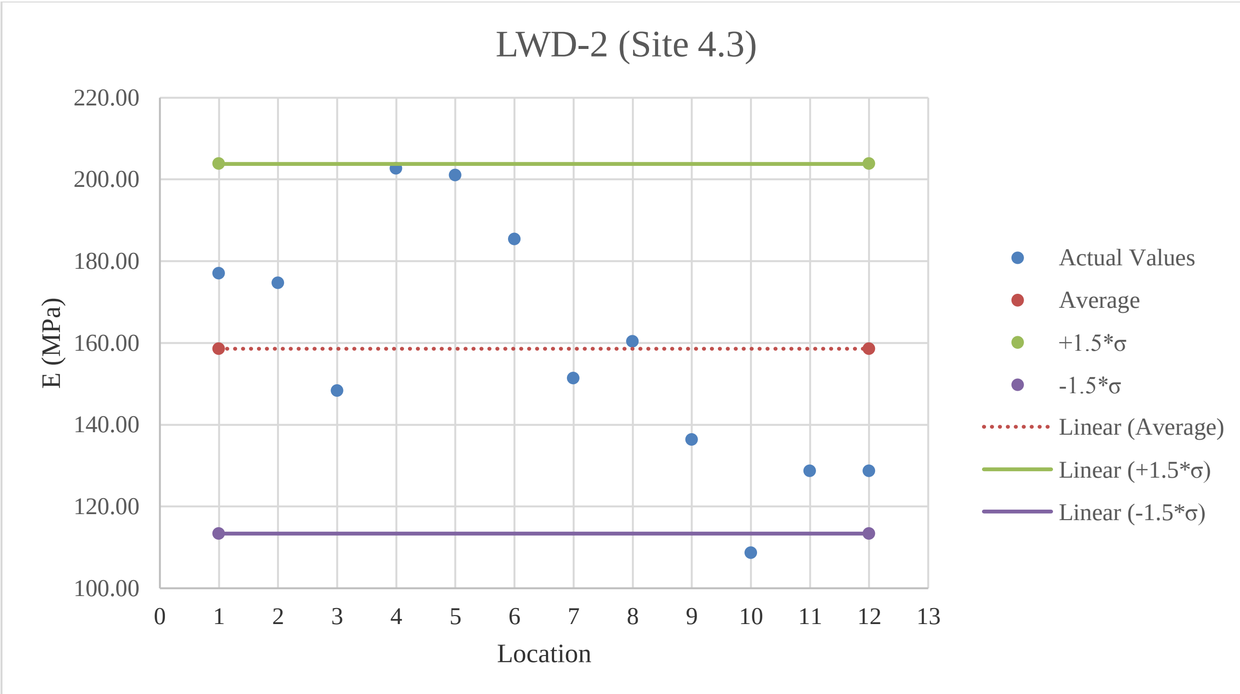


Figure E.39: LWD-2 testing results on stabilized subgrade

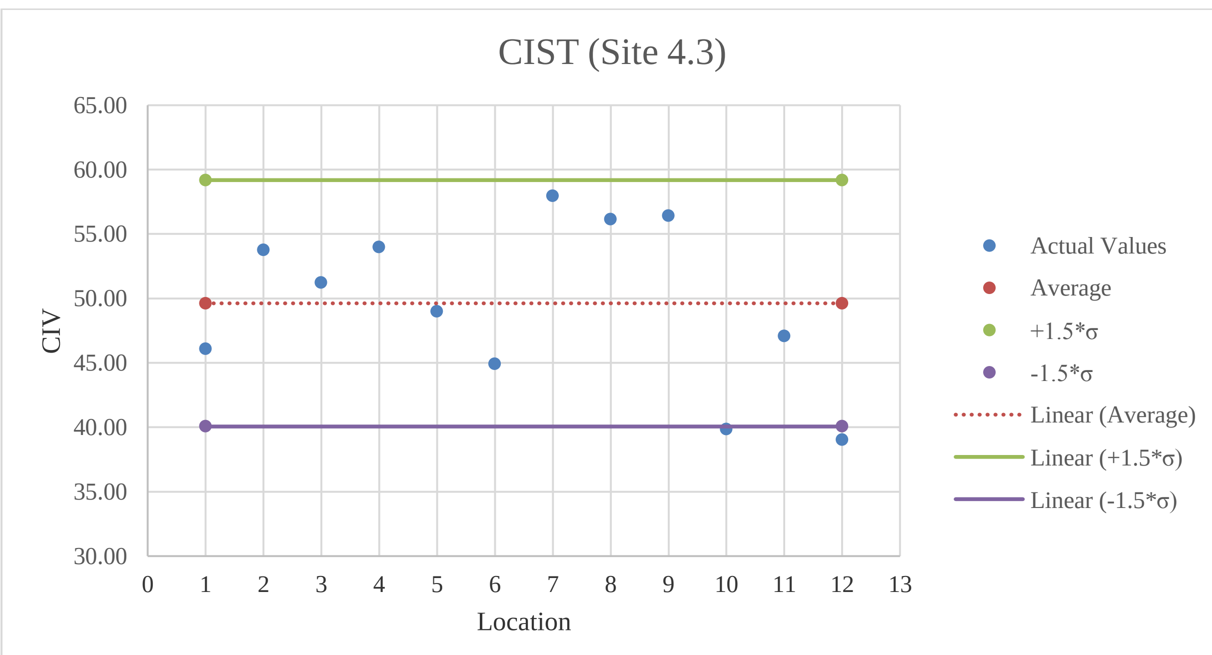


Figure E.40: CIST testing results on stabilized subgrade

E.5 SR-23 and Normandy Boulevard (Site 5)

E.5.1 First Plate Load Test Location

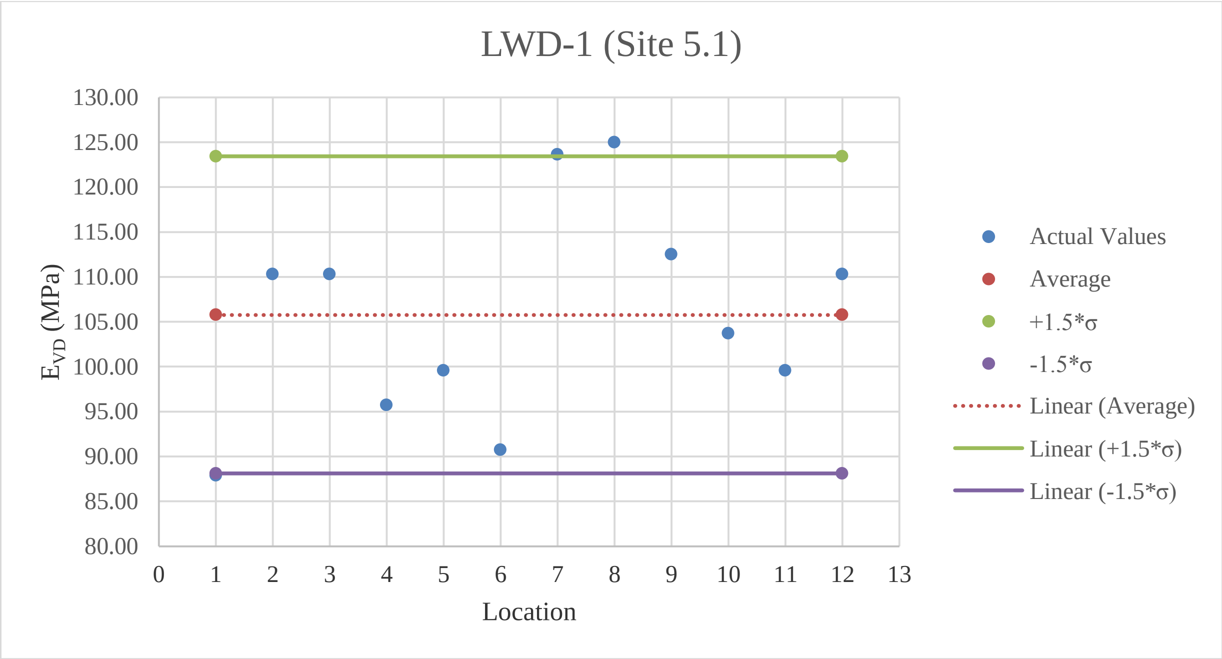


Figure E.41: LWD-1 testing results on limerock material

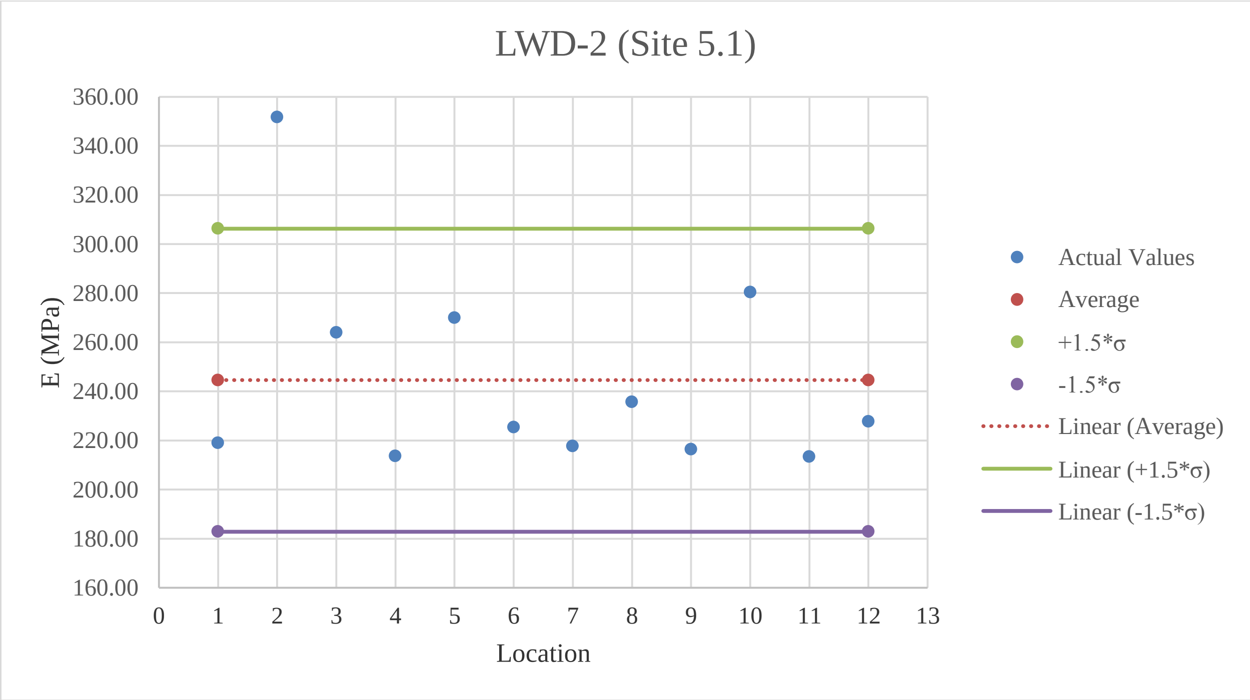


Figure E.42: LWD-2 testing results on limerock material

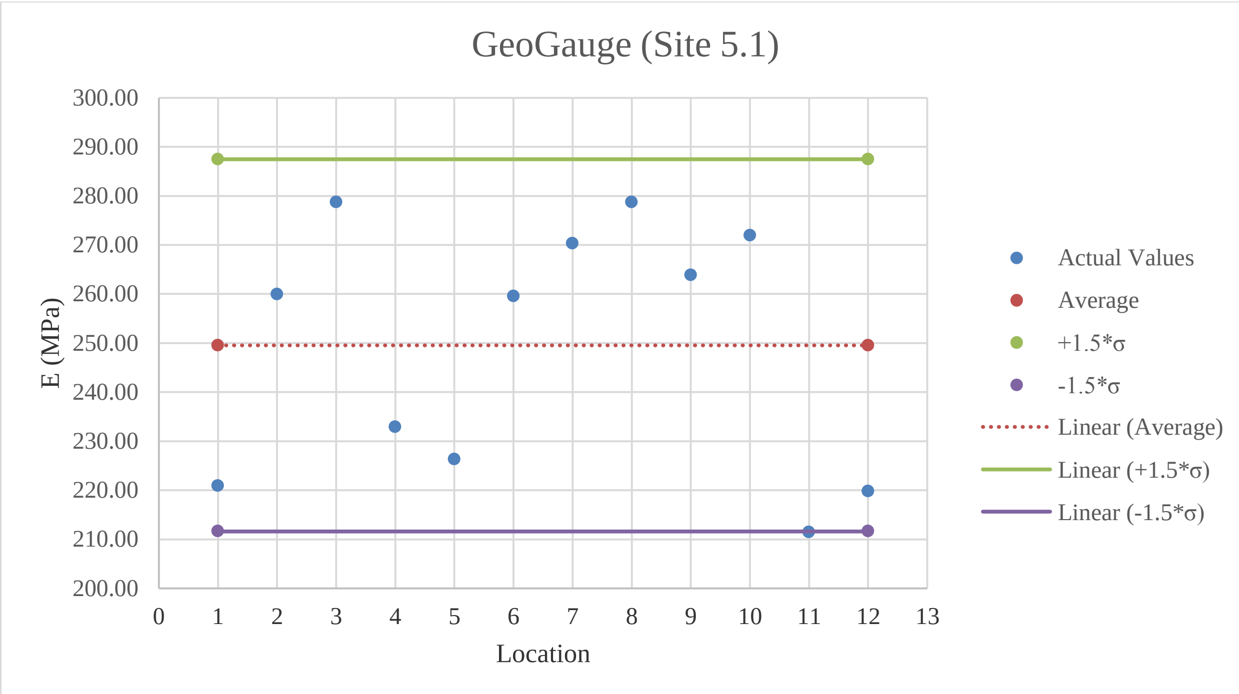


Figure E.43: GeoGauge testing results on limerock material

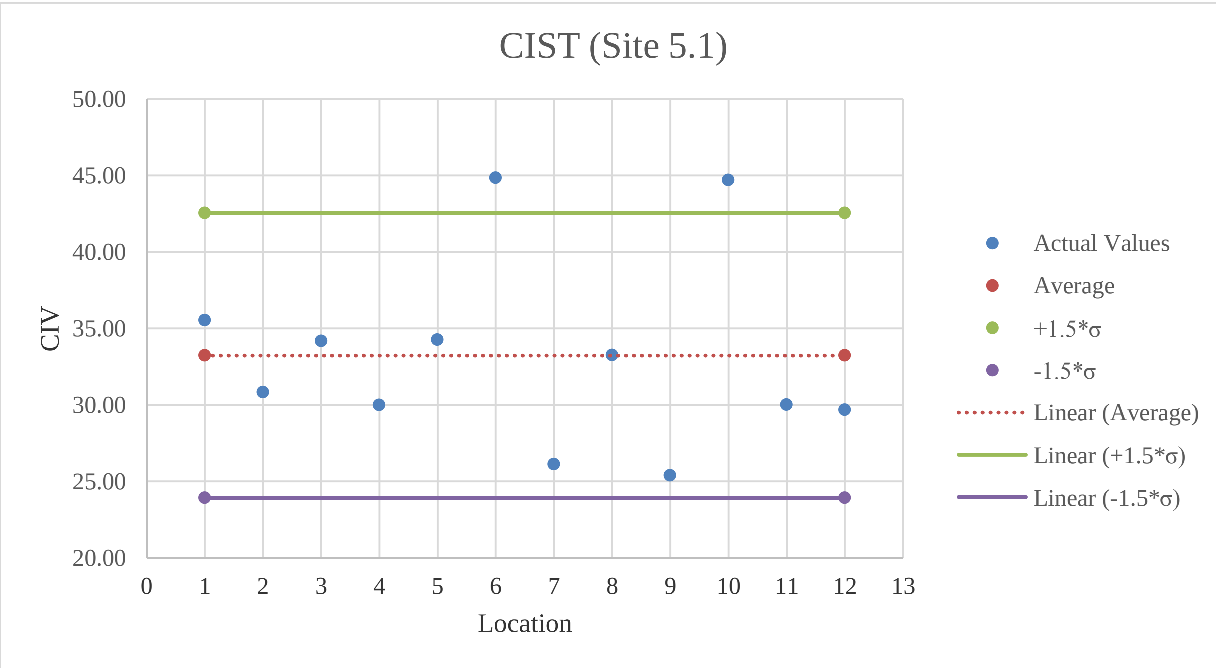


Figure E.44: CIST testing results on limerock material

E.5.2 Second Plate Load Test Location

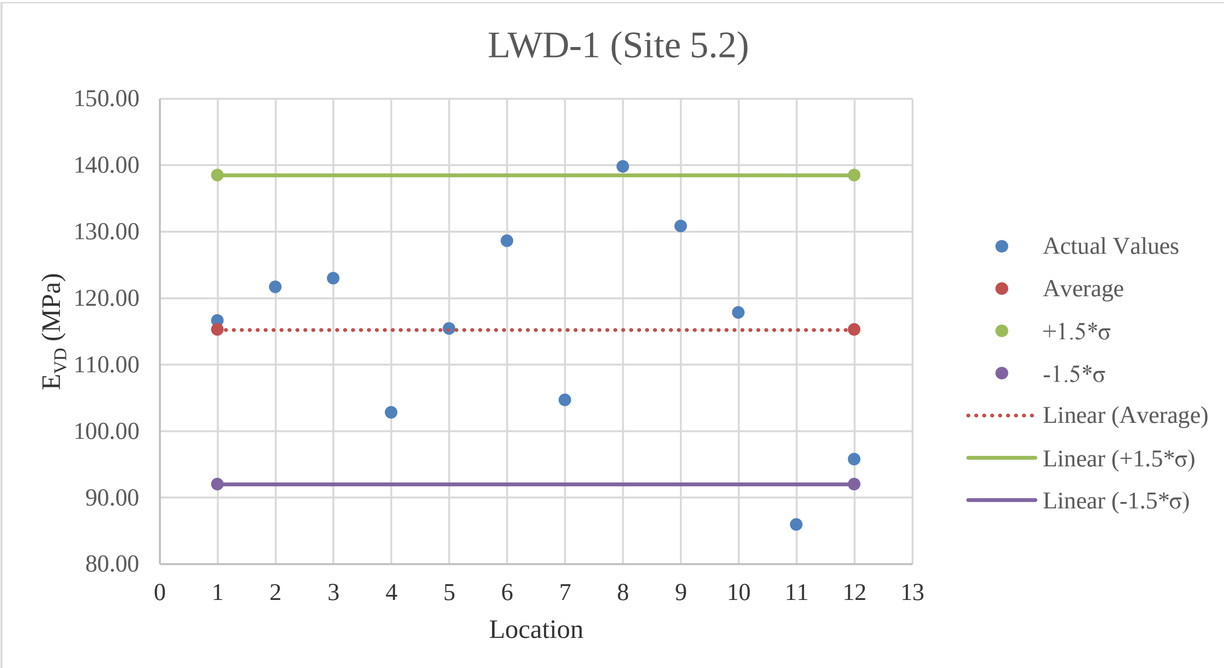


Figure E.45: LWD-1 testing results on limerock material

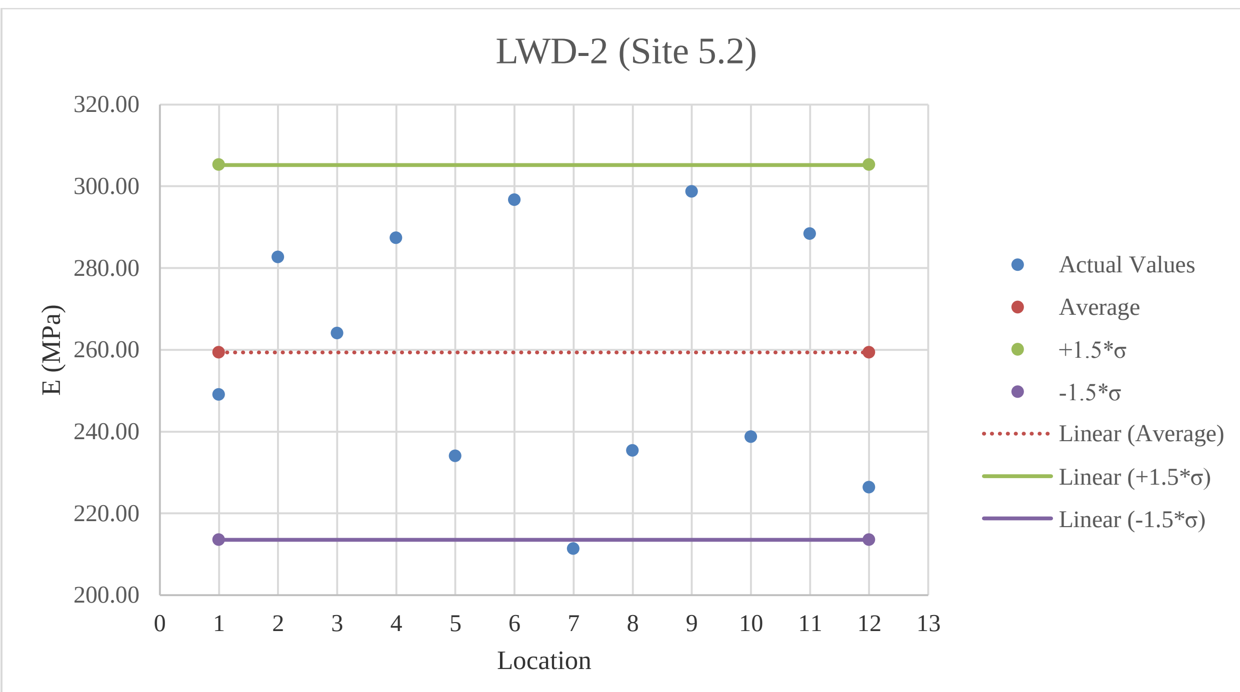


Figure E.46: LWD-2 testing results on limerock material

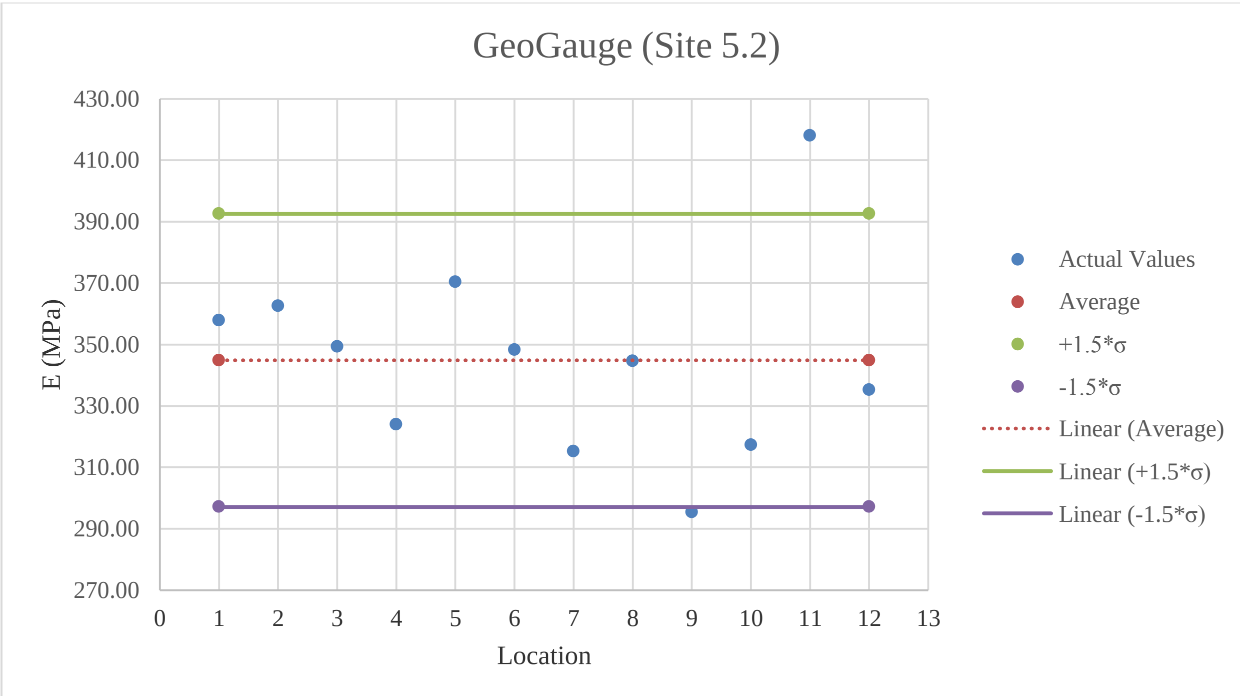


Figure E.47: GeoGauge testing results on limerock material

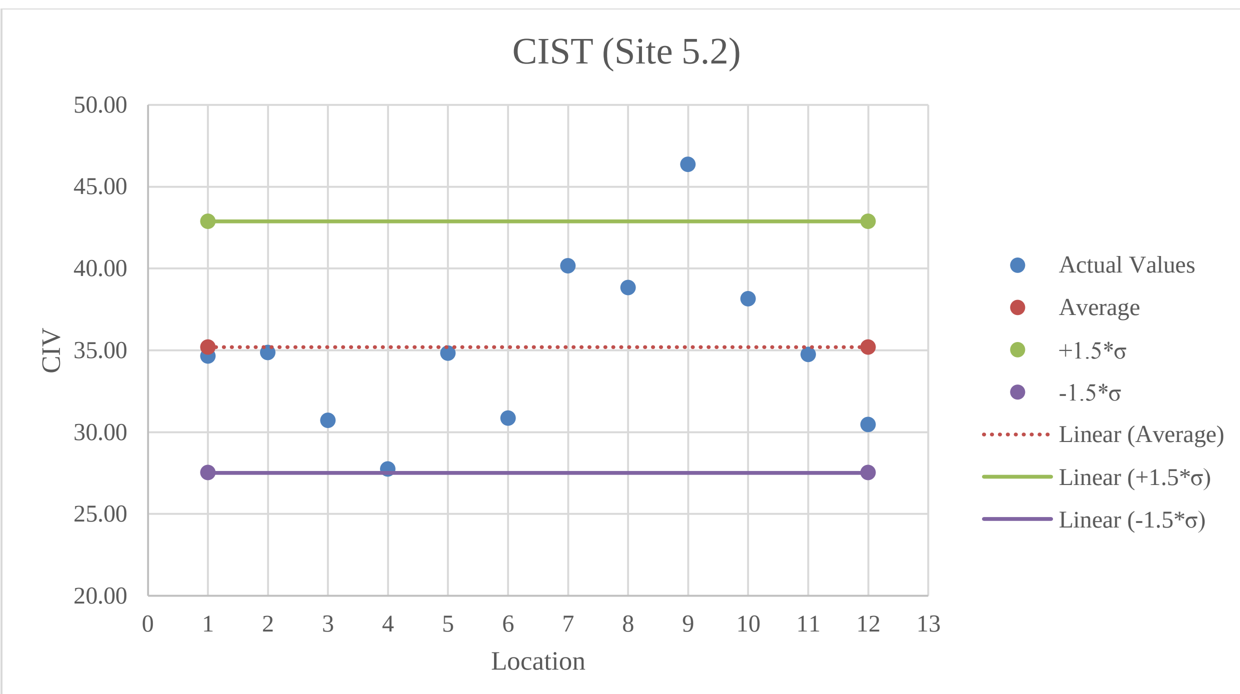


Figure E.48: CIST testing results on limerock material

E.5.3 Third Plate Load Test Location

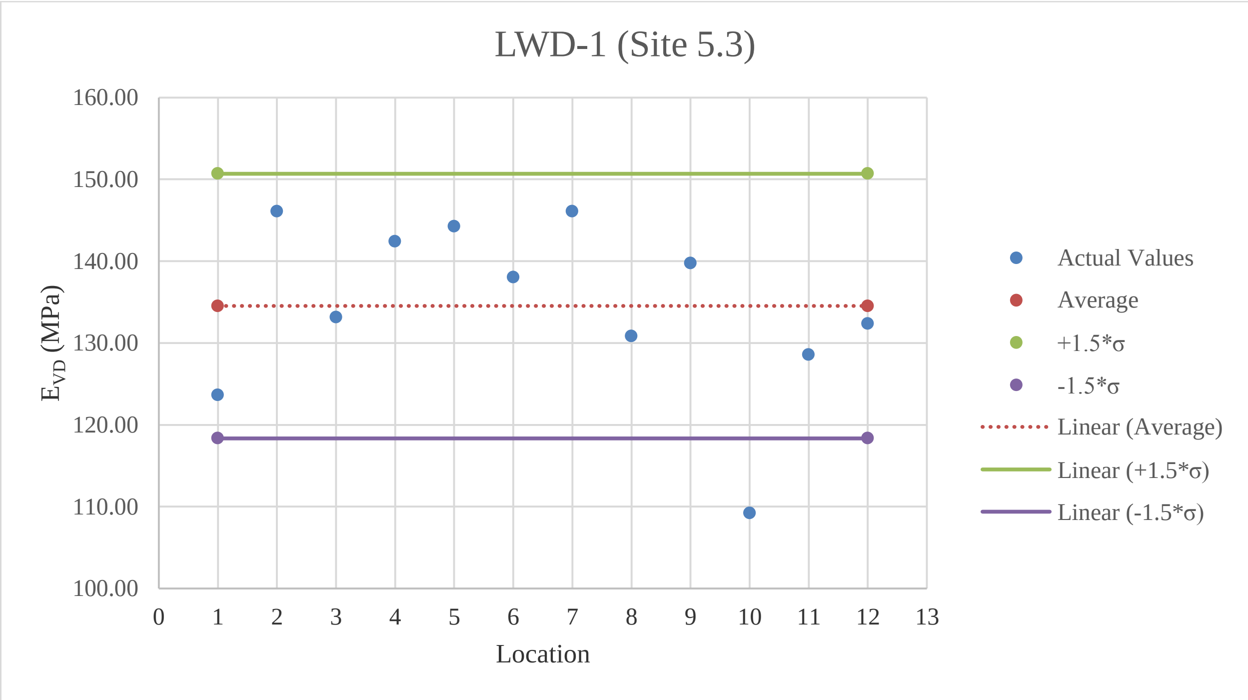


Figure E.49: LWD-1 testing results on limerock material

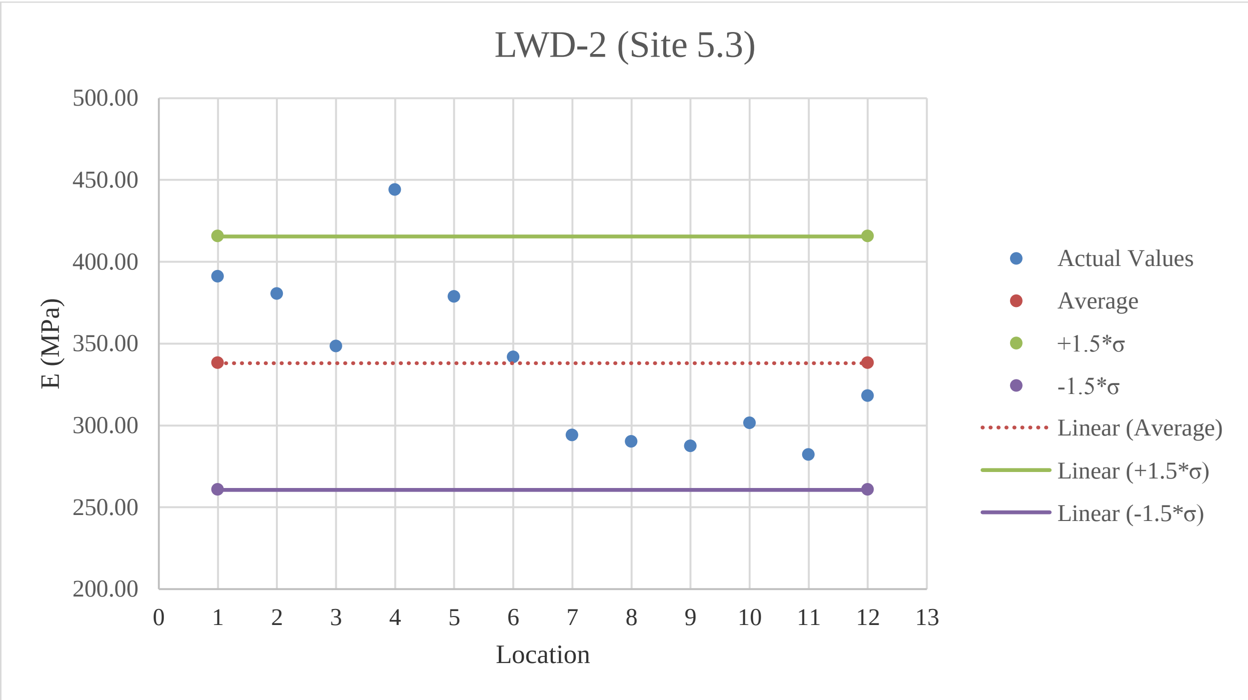


Figure E.50: LWD-2 testing results on limerock material

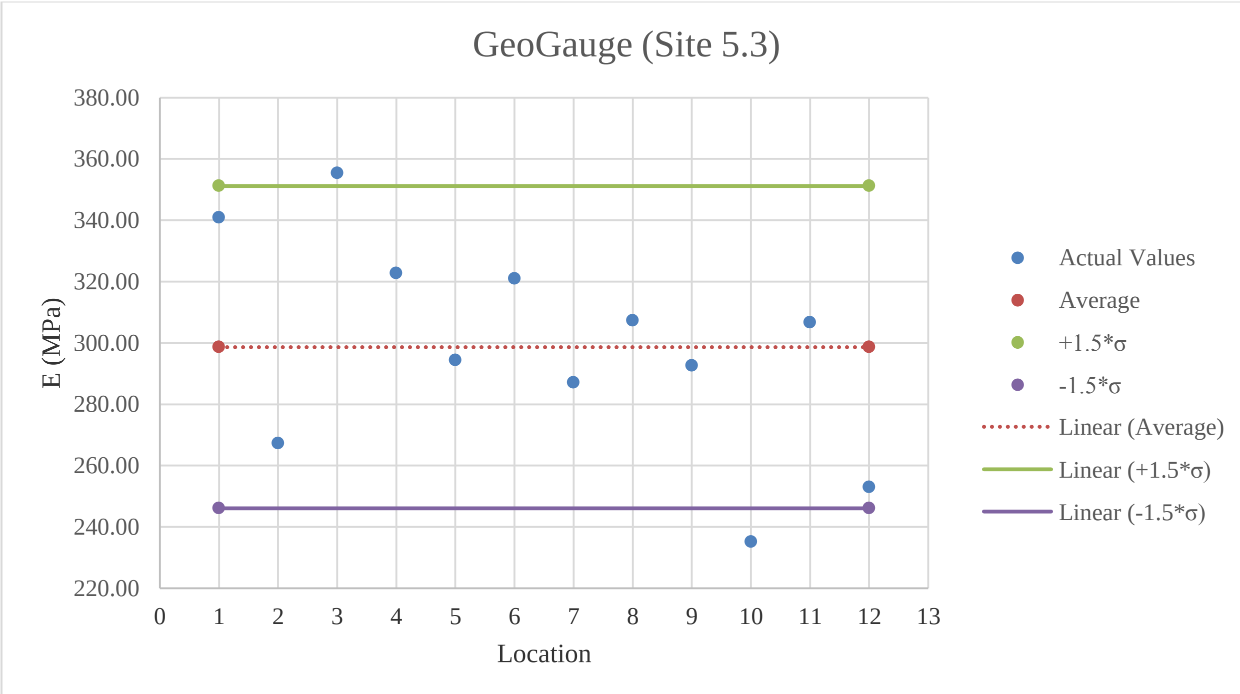


Figure E.51: GeoGauge testing results on limerock material

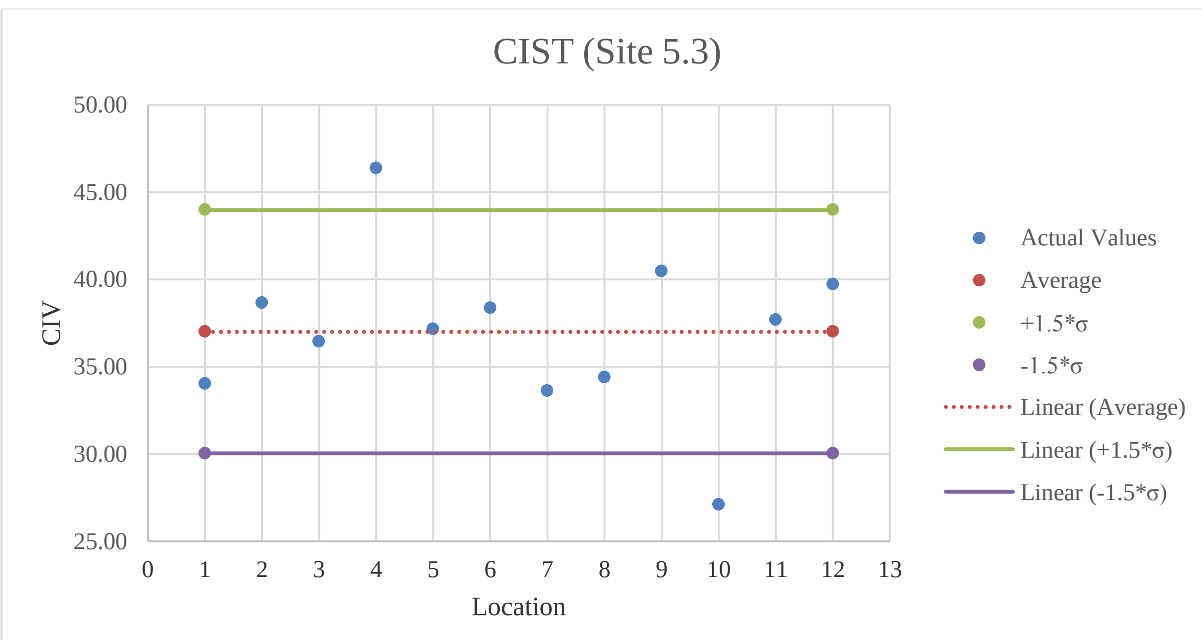


Figure E.52: CIST testing results on limerock material

E.6 SR-23 and New World Avenue (Site 6)

E.6.1 First Plate Load Test Location

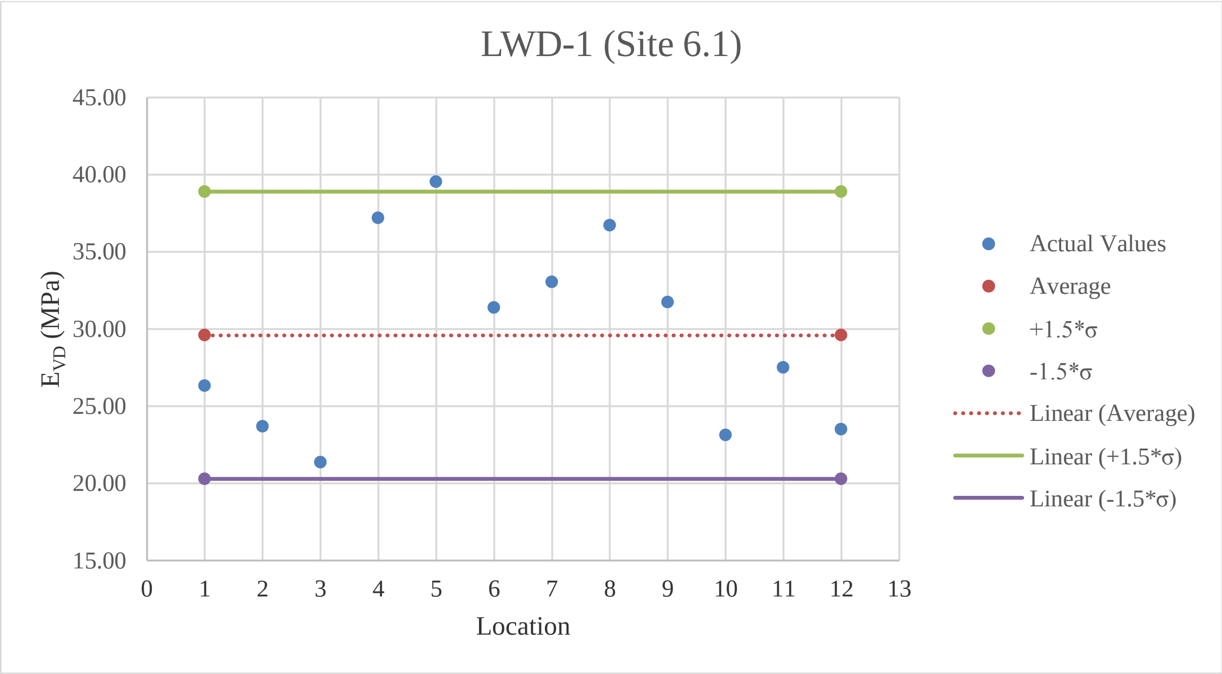


Figure E.53: LWD-1 testing results on A-3 material

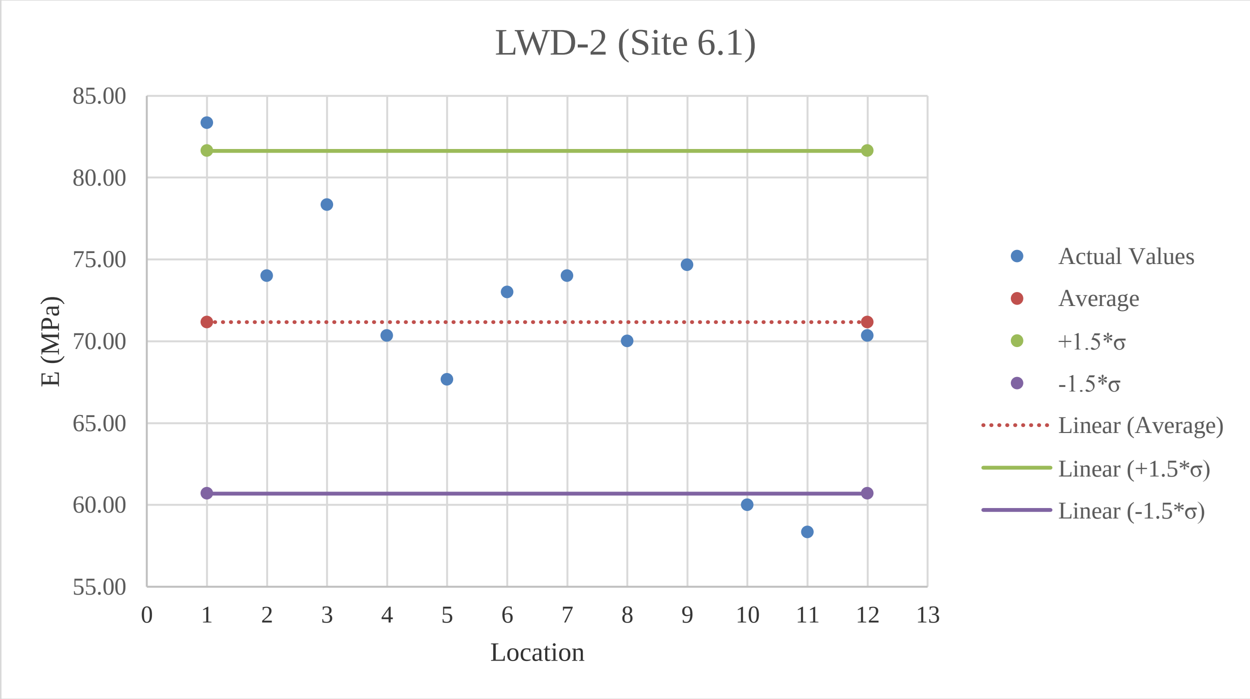


Figure E.54: LWD-2 testing results on A-3 material

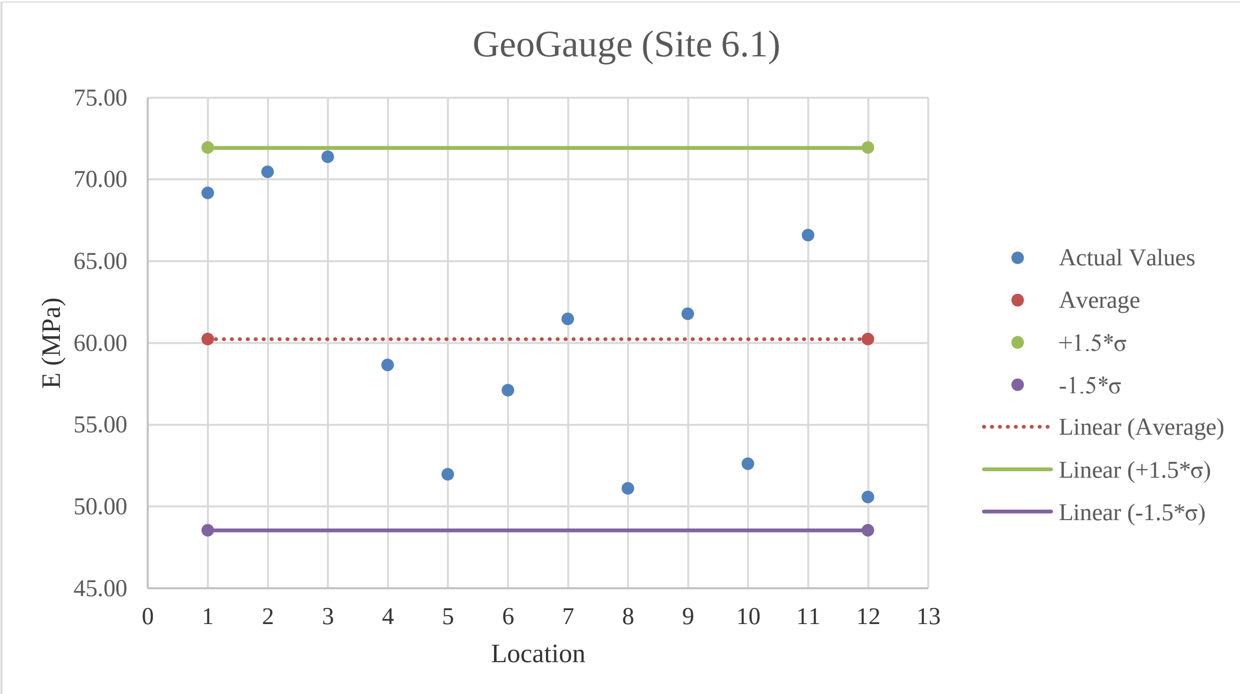


Figure E.55: GeoGauge testing results on A-3 material

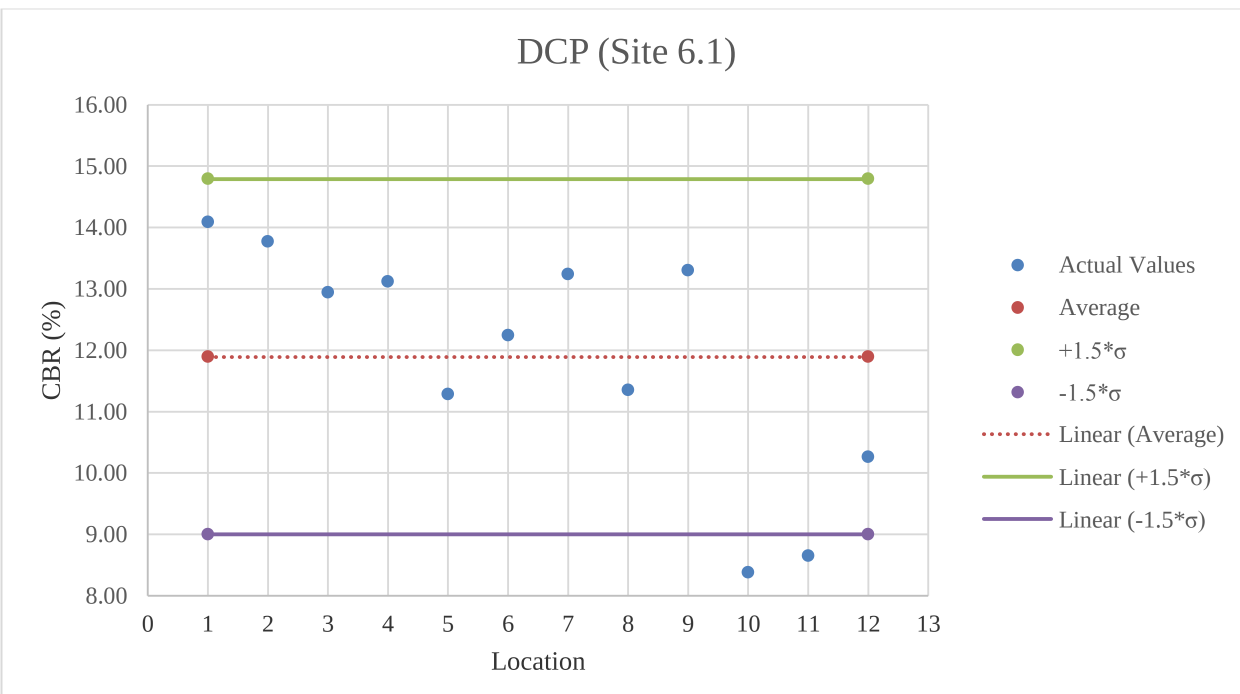


Figure E.56: DCP testing results on A-3 material

E.6.2 Second Plate Load Test Location

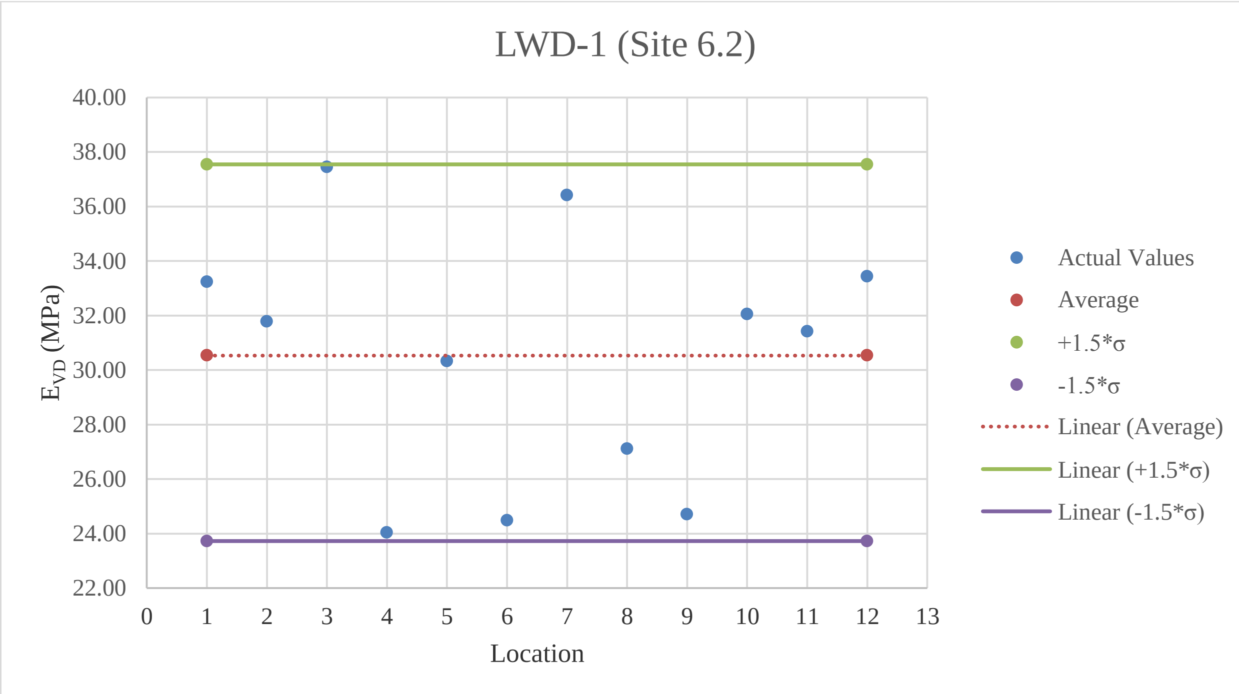


Figure E.57: LWD-1 testing results on A-3 material

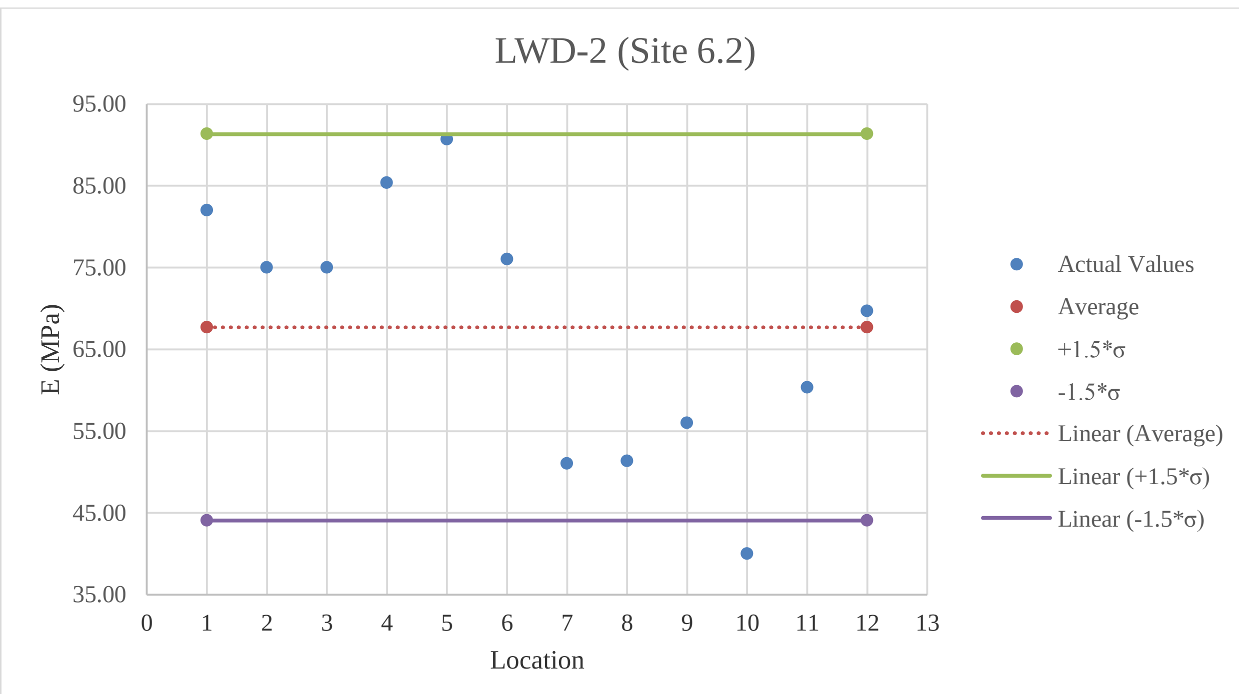


Figure E.58: LWD-2 testing results on A-3 material

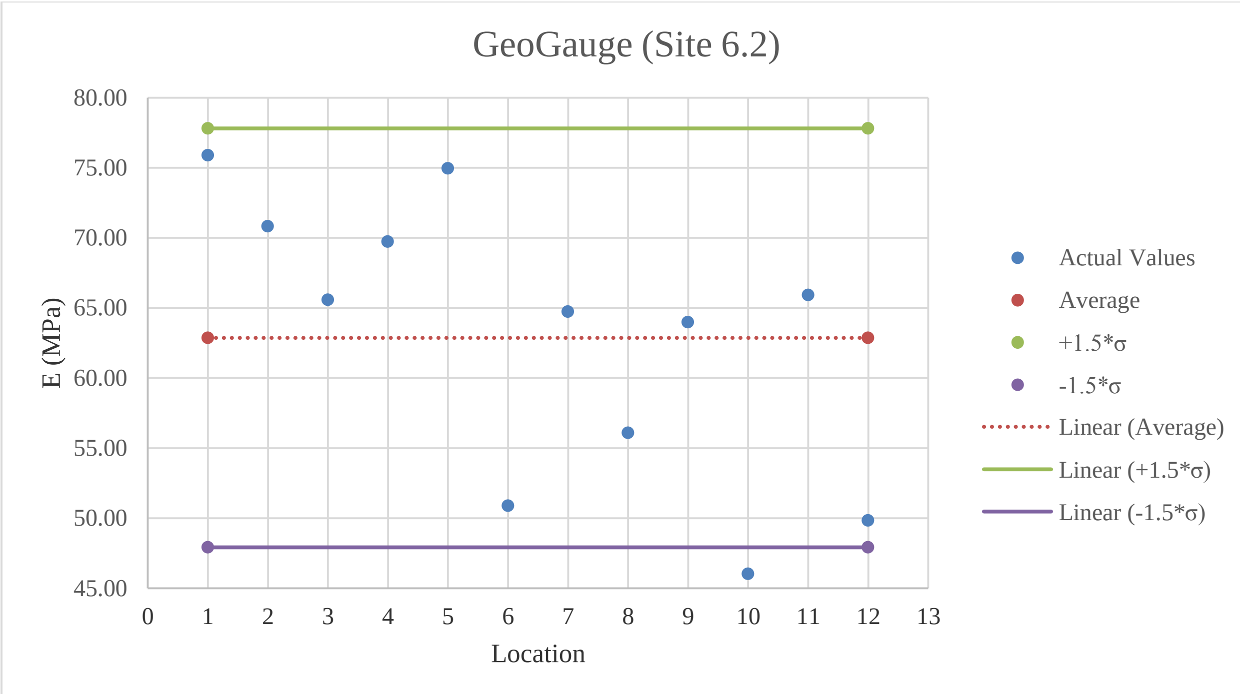


Figure E.59: GeoGauge testing results on A-3 material

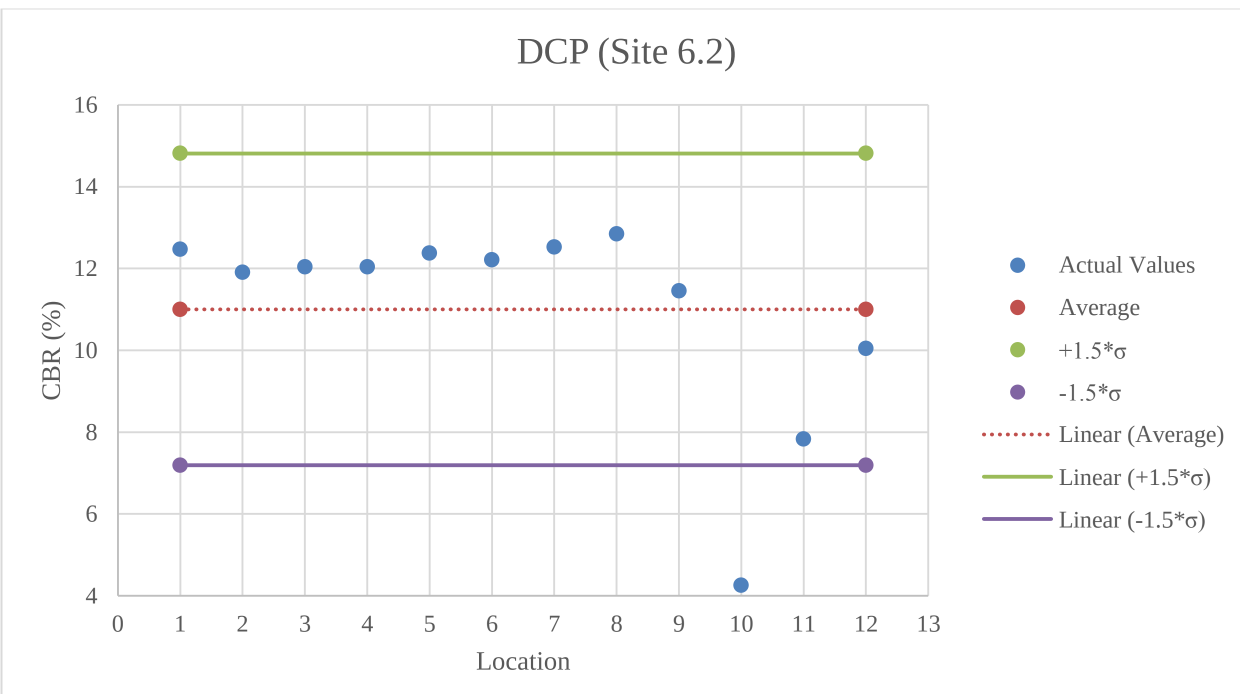


Figure E.60: DCP testing results on A-3 material

E.6.3 Third Plate Load Test Location

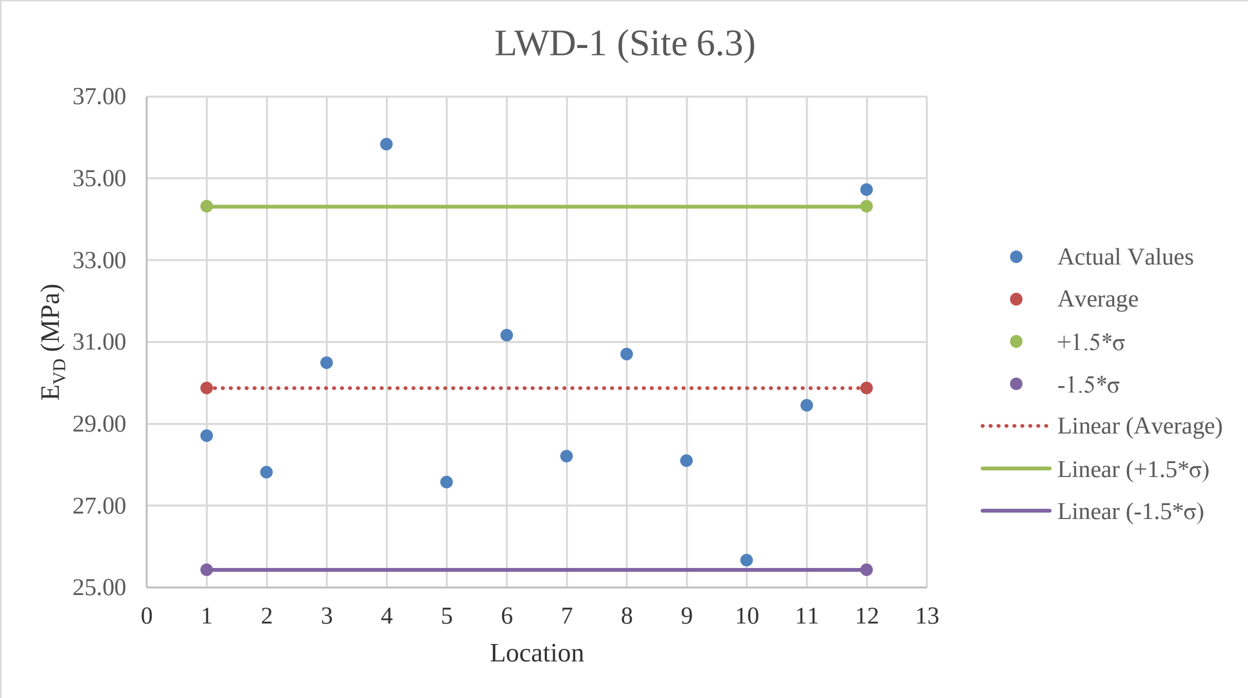


Figure E.61: LWD-1 testing results on A-3 material

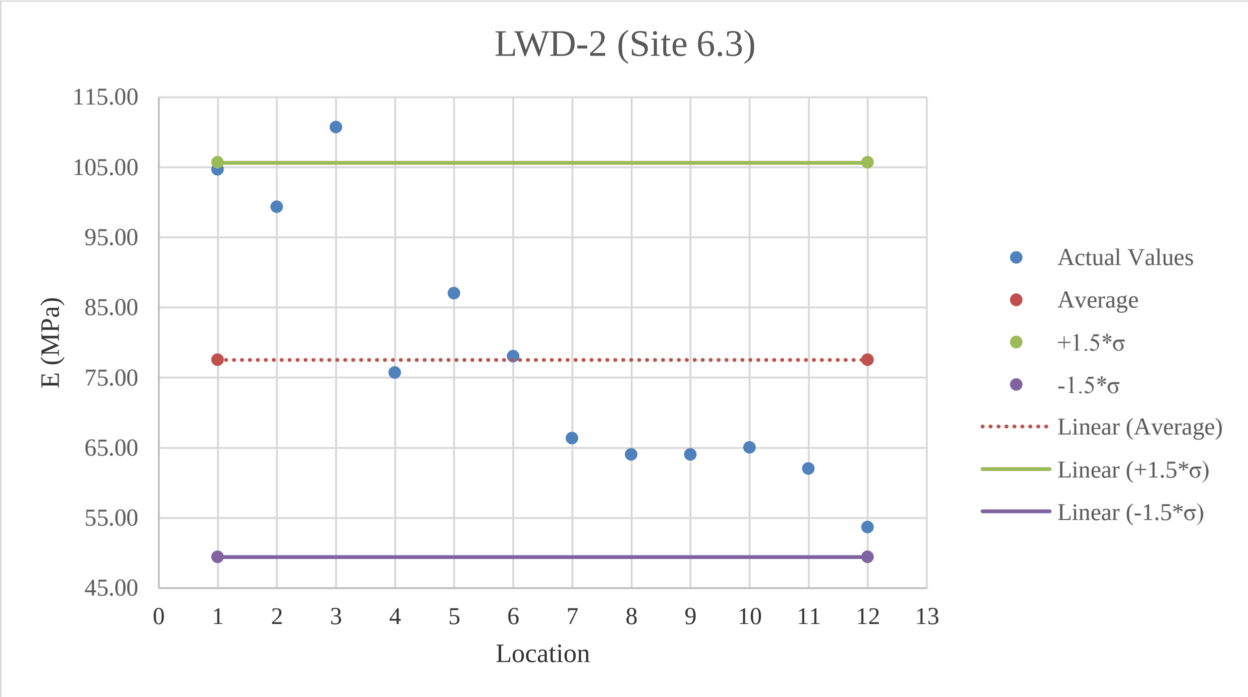


Figure E.62: LWD-2 testing results on A-3 material

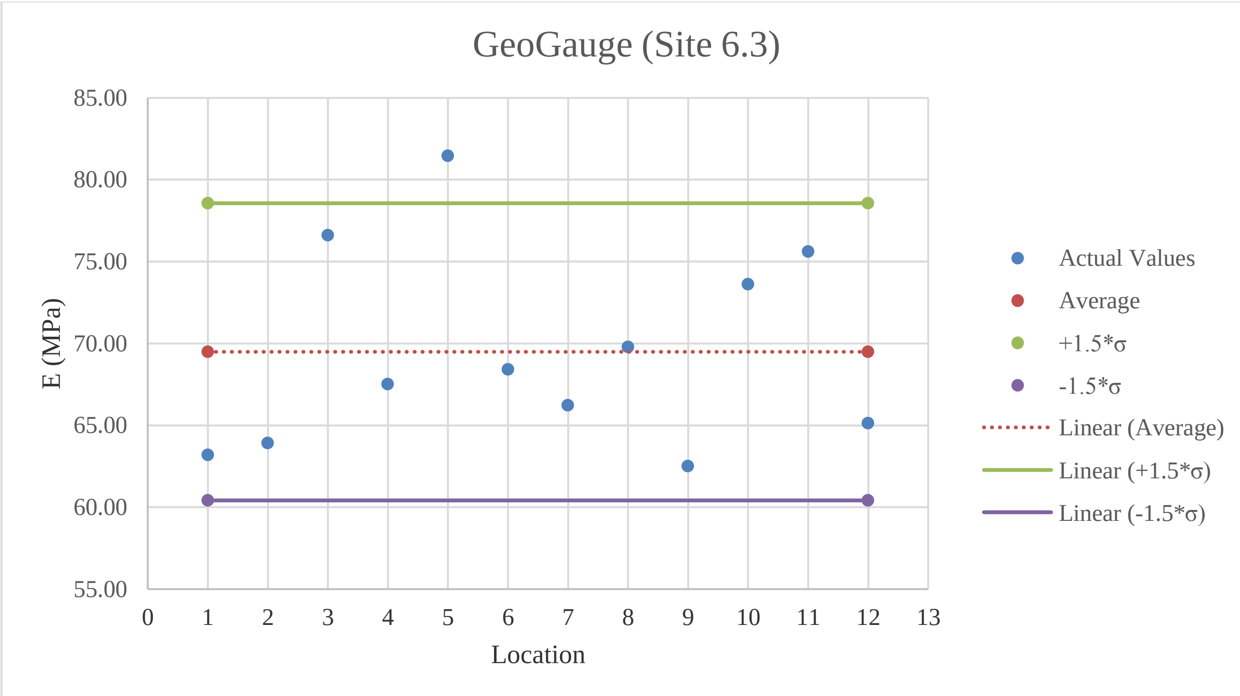


Figure E.63: GeoGauge testing results on A-3 material

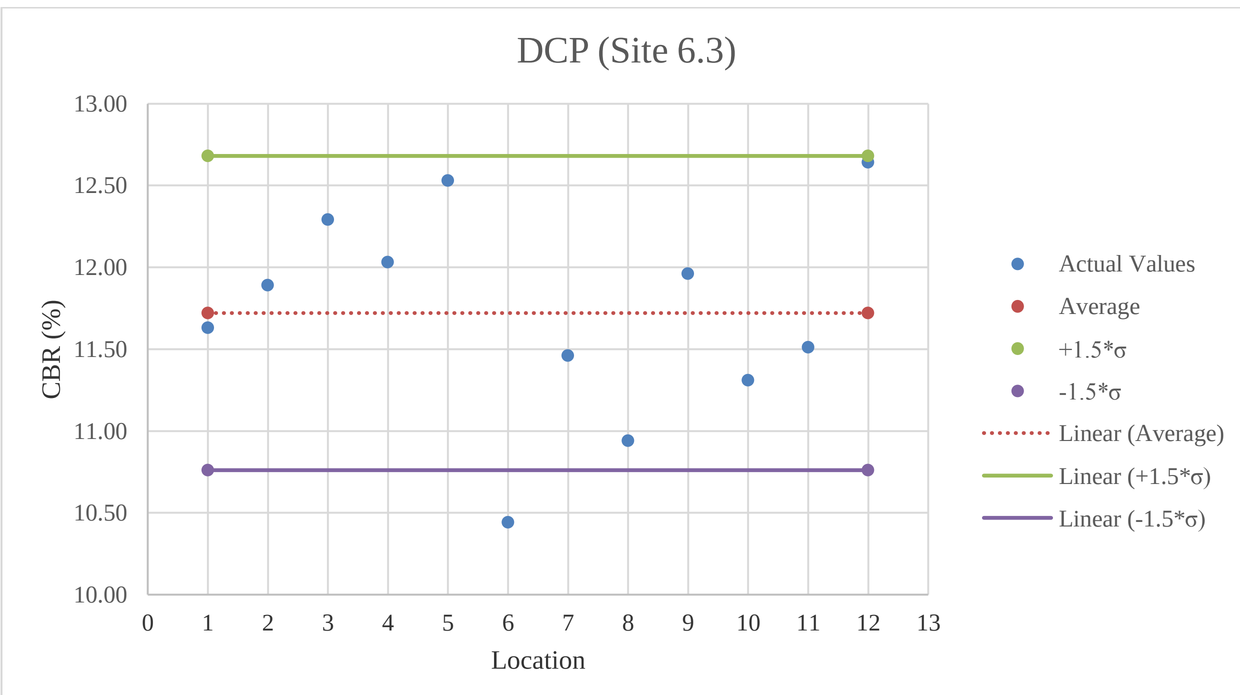


Figure E.64: DCP testing results on A-3 material

Appendix F: Correlations between Equipment Measurements and Laboratory Resilient Modulus

F.1 Correlation Analysis

F.1.1 Correlation Analysis Conducted

In order to fully compare stiffness/modulus values obtained from the measurements of different pieces of equipment with the laboratory resilient modulus (M_R) values, correlations between the two sets of values were calculated for different soil types at (near) or below the optimum moisture content. Therefore, the following correlation tables (Tables F.1-F.3) were generated:

1. Correlations between equipment measurements and the laboratory M_R for different soil types tested below the optimum moisture.
2. Correlations between equipment measurements and the laboratory M_R for different soil types tested near the optimum moisture.
3. Correlations between equipment measurements and the laboratory M_R for different soil types, regardless of the soil's moisture condition when tested.

F.1.2 Interpretation of Correlation Strength and Significance

For example, Table F.1 Columns 1-4 show correlations between the equipment measurements and the laboratory M_R for the A-2-4 low fines soil tested below the optimum moisture. Generally, the Pearson correlation coefficient (r) measures the strength of the *linear* relationship between the two variables. The relationship is assessed for its significance as well as its strength. In Table F.1, Column 1 lists equipment used in the test pit; Column 2 shows the total data point sets that were obtained; Column 3 lists the Pearson correlation coefficient between the equipment measurements and the laboratory M_R; Column 4 shows the significance probability (p-value), which is the probability of obtaining, by chance alone, a correlation with a greater absolute value than the computed value if no linear relationship exists between the two variables (i.e., equipment measurements vs. laboratory M_R).

Interpretations of correlation strength and significance should be noted:

1. The strength of the relationship is indicated by the correlation coefficient.
2. The significance of the relationship is expressed in probability levels, which tell how unlikely a given correlation coefficient will occur given no relationship in the population.
3. Therefore, the smaller the significant probability, the more significant the relationship.
4. However, the larger the correlation coefficient, the stronger the relationship.

F.1.3 Observation

Table F.1 shows correlations between equipment measurements and the laboratory M_R for different soil types tested below the optimum moisture. Table F.2 shows correlations between equipment measurements and the laboratory M_R for different soil types tested near the optimum moisture. Table F.3 shows correlations between equipment measurements and the laboratory M_R for different soil types tested at all moisture conditions. The largest correlation coefficient and the smallest p-value are bolded. Significant p-values (at the level of 0.05) also are highlighted with an asterisk.

For example, in Table F.1, the sample correlation between the results obtained from the LWD-1 measurements (in MPa) and the laboratory M_R for the A-2-4 low fines soil tested below the optimum moisture is 0.2749. To test if the estimated correlation (0.2749) is significantly different from 0 (i.e., no correlation), we propose two hypotheses:

1. H_0 (Null Hypothesis): The correlation in the population is 0.
2. H_a (Alternative Hypothesis): The correlation in the population is different from 0.

Since the significance probability $p = 0.3872 > 0.10$, the null hypothesis of no correlation cannot be rejected at the significance level of 0.1. In other words, we cannot reject the null hypothesis that the correlation is 0 between results obtained from the LWD-1 measurements (in MPa) and the laboratory M_R for the A-2-4 low fines soil tested below the optimum moisture.

Similarly, since the significance probability is 0.0798 for the correlation between GeoGauge measurements and the laboratory M_R for the A-2-4 low fines soil tested below the optimum moisture, the null hypothesis of no correlation in the population of the two can be rejected at the significance level of 0.1. The Pearson correlation coefficient is 0.5247 in this case, which measures the strength of the correlation between the two.

Observations are summarized with each table (Tables F.1-F.3).

F.1.4 Further Discussion

Tables F.1 and F.2 show only a few statistically significant correlations between the equipment measurements and the laboratory M_R under different moisture conditions. Contributions to inconsistency—other than inherent reasons due to soil types and moisture conditions—include:

1. Sample size. Correlations in Tables F.1 and F.2 are obtained with small samples (less than 15 data sets) and thus might be unreliable. Schönbrodt and Perugini (2013) suggested a sample size of 250 to achieve stable estimates for correlations. In practical use, a sample size of 80 usually is a rule of thumb.
2. Scatterplots check. Figures F.1-F.8 plot the bivariate fits of the laboratory M_R and the equipment measurements for each soil type under the two moisture conditions. These scatterplots show consistency with the above linear correlation analysis in Tables F.1 and F.2. The linear trends in most of the figures seem unclear and could be due to the small sample sizes (only tenor 12 data points for each bivariate fit) for each soil type under different moisture conditions.

Table F.1: Correlation between equipment measurements and laboratory resilient modulus for different soil types tested below optimum moisture

Soil Type	Mixed			Non-cohesive			Subgrade			Base		
Sample Soil	**A-2-4 Low Fines**			**A-3**			**Stabilized Subgrade**			**Limerock Base**		
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**	**(11)**	**(12)**	**(13)**
Equipment	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**
CIST (CIV)	12	-0.0141	0.9653	12	0.3402	0.2793	10	0.1138	0.7542	10	0.1001	0.7831
DCP (CBR)	12	-0.0519	0.8728	12	0.0898	0.7813	10	0.0860	0.8132	10	0.4668	0.1738
DCP Depth/Blow (mm)	12	-0.0132	0.9675	12	-0.0221	0.9457	10	-0.2205	0.5404	10	**-0.5800**	**0.0781**
LWD-2 (MPa)	12	0.1731	0.5905									
LWD-2 Deflection (µm)	12	-0.2212	0.4897									
GeoGauge (MPa)	12	**0.5247**	**0.0798**	12	0.1519	0.6374	10	-0.5176	0.1254	10	**0.5395**	**0.1075**
NDG (pcf)	12	0.4021	0.1950	12	0.3645	0.2441	10	-0.0297	0.9352	10	-0.2233	0.5351
PLT (MPa)	12	0.0118	0.9709	12	0.3086	0.3291	10	0.0357	0.9220	10	0.4364	0.2074
LWD-1 (MPa)	12	0.2749	0.3872	12	0.2334	0.4652	10	-0.2978	0.4034	10	-0.0426	0.9070
LWD-1 Deflection (mm)	12	-0.2674	0.4009	12	-0.2889	0.3625	10	0.3204	0.3668	10	0.0422	0.9078

Observations: See Sections F.1.2 and F.1.3 for interpretation of the values contained in Table F.1.

When tests are conducted below the optimum moisture, the ten or 12 data sets for each soil type indicate that different pieces of equipment show different strengths of correlation with the laboratory M_R; however, none of these relationships are statistically significant at the level of 0.05. Three correlations are statistically significant at the level of 0.1.

For mixed soils (A-2-4 low fines), the GeoGauge measurements show fair correlation (>0.5) with the laboratory M_R at the significance level of 0.1. None of the other pieces of equipment show statistically significant correlation with the laboratory M_R at the level of 0.1.

For non-cohesive soils (A-3), none of the equipment show statistically significant correlation with the laboratory M_R at the level of 0.1.

For stabilized subgrade materials, none of the equipment show statistically significant correlation with the laboratory M_R at the level of 0.1.

For base materials (limerock), the GeoGauge (roughly) and DCP (depth per blow) measurements show fair correlation (>0.5) with the laboratory M_R at the significance level of 0.1. None of the other equipment show statistically significant correlation with the laboratory M_R at the level of 0.1.

Table F.2: Correlation between equipment measurements and laboratory resilient modulus for different soil types tested near optimum moisture

Soil Type	Mixed			Non-cohesive			Subgrade			Base		
Sample Soil	**A-2-4 Low Fines**			**A-3**			**Stabilized Subgrade**			**Limerock Base**		
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**	**(11)**	**(12)**	**(13)**
Equipment	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**
CIST (CIV)	12	0.0833	0.7969	12	0.1666	0.6047	10	-0.2173	0.5464	10	-0.2734	0.4446
DCP (CBR)	12	0.0734	0.8207	12	-0.3108	0.3254	10	**0.6567**	**0.0391***	10	-0.3294	0.3527
DCP Depth/Blow (mm)	12	-0.0914	0.7775	10	0.3074	0.3310	10	-0.3801	0.2786	10	0.1242	0.7324
GeoGauge (MPa)	12	-0.0058	0.9858	12	0.0911	0.7783	10	0.4842	0.1562	10	-0.0901	0.8045
NDG (pcf)	12	**0.5968**	**0.0405***	12	-0.0895	0.7820	10	0.1260	0.7287	10	0.1552	0.6687
PLT (MPa)	12	-0.0125	0.9692	12	-0.2499	0.4334	10	0.0252	0.9448	10	0.0831	0.8194
LWD-1 (MPa)	12	0.3111	0.3250	12	-0.2061	0.5205	10	-0.3507	0.3204	10	0.2361	0.5113
LWD-1 Deflection (mm)	12	-0.2791	0.3797	12	0.2822	0.3741	10	0.3214	0.3652	10	-0.1897	0.5996

Observations: See Sections F.1.2 and F.1.3 for interpretation of the values contained in Table F.2.

When tests are conducted near optimum moisture, the ten or 12 data sets for each soil type indicate that different pieces of equipment show different strengths of correlation with the laboratory M_R; however, only two of these relationships are statistically significant at the level of 0.05.

For mixed soils (A-2-4 low fines), the NDG measurements show fair correlation (around 0.6) with the laboratory M_R at the significance level of 0.05. None of the other equipment show statistically significant correlation with laboratory M_R at the level of 0.1.

For non-cohesive soils (A-3), none of the equipment show statistically significant correlation with laboratory M_R at the level of 0.1.

For stabilized subgrade materials, the DCP measurements show fair correlation (around 0.65) with the laboratory M_R at the significance level of 0.05. None of the other equipment show statistically significant correlation with the laboratory M_R at the level of 0.1.

For base materials, none of the equipment show statistically significant correlation with the laboratory M_R at the level of 0.1.

Table F.3: Correlation between equipment measurements and laboratory resilient modulus for different soil types

Soil Type	Mixed			Non-cohesive			Subgrade			Base		
Sample Soil	**A-2-4 Low Fines**			**A-3**			**Stabilized Subgrade**			**Limerock Base**		
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**	**(11)**	**(12)**	**(13)**
Equipment	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**	**Count**	**Correlation**	**Signif Prob**
CIST (CIV)	24	-0.3536	0.0901	24	0.0414	0.8477	20	-0.5386	0.0143*	20	0.3338	0.1504
DCP (CBR)	24	-0.3738	0.0720	24	-0.3892	0.0601	20	-0.4028	0.0783	20	0.1469	0.5367
DCP Depth/Blow (mm)	24	0.3442	0.0996	24	0.4340	0.0341*	20	-0.6230	0.0033*	20	-0.1851	0.4347
GeoGauge (MPa)	24	0.0190	0.9298	24	-0.3570	0.0868	20	-0.0793	0.7395	20	**0.6723**	**0.0012***
NDG (pcf)	24	**0.3880**	**0.0610**	24	**0.4757**	**0.0188***	20	**0.6885**	**0.0008***	20	0.6182	0.0037*
PLT (MPa)	24	-0.3507	0.0929	24	-0.3536	0.0901	20	-0.6870	0.0008*	20	0.4583	0.0421*
LWD-1 (MPa)	24	-0.1888	0.3769	24	-0.1351	0.5290	20	-0.6554	0.0017*	20	0.3025	0.1948
LWD-1 Deflection (mm)	24	0.2055	0.3353	24	0.1570	0.4637	20	0.6408	0.0023*	20	-0.2392	0.3097

Observations: See Sections F.1.2 and F.1.3 for interpretation of the values contained in Table F.3.

Without categorization of moisture conditions, the 20 or 24 data sets for each soil type indicate that different pieces of equipment show different strengths of correlation with the laboratory M_R and only some of such relationships are statistically significant at the significance level of 0.05.

For mixed soils (A-2-4 low fines), the NDG, DCP, CIST, and PLT measurements show weak to fair correlation (absolute values between 0.3 and 0.4) with the laboratory M_R at the significance level of 0.1. The correlations between the LWD-1 and GeoGauge measurements and the laboratory M_R are not statistically significant.

For non-cohesive soils (A-3), the NDG and DCP (depth per blow) measurements show fair correlation (between 0.4 and 0.5) with the laboratory M_R at the significance level of 0.05. The GeoGauge, DCP (CBR), and PLT measurements show weak to fair correlation (absolute values between 0.3 and 0.4) with the laboratory M_R at the significance level of 0.1. The correlations between the LWD-1 and CIST measurements and the laboratory M_R are not statistically significant.

For stabilized subgrade materials, the NDG, PLT, LWD-1, CIST, and DCP (depth per blow) measurements show fair correlation (absolution values between 0.5 and 0.7) with the laboratory M_R at the significant level of 0.05. The DCP (CBR) measurements show fair correlation (around -0.4) with the laboratory M_R at the significance level of 0.1. The correlation between the GeoGauge measurements and the laboratory M_R is not statistically significant.

For base materials, the GeoGauge, NDG, and PLT measurements show fair correlation (between 0.4 and 0.7) with the laboratory M_R at the significance level of 0.05. None of the other equipment show statistically significant correlation with the laboratory M_R.

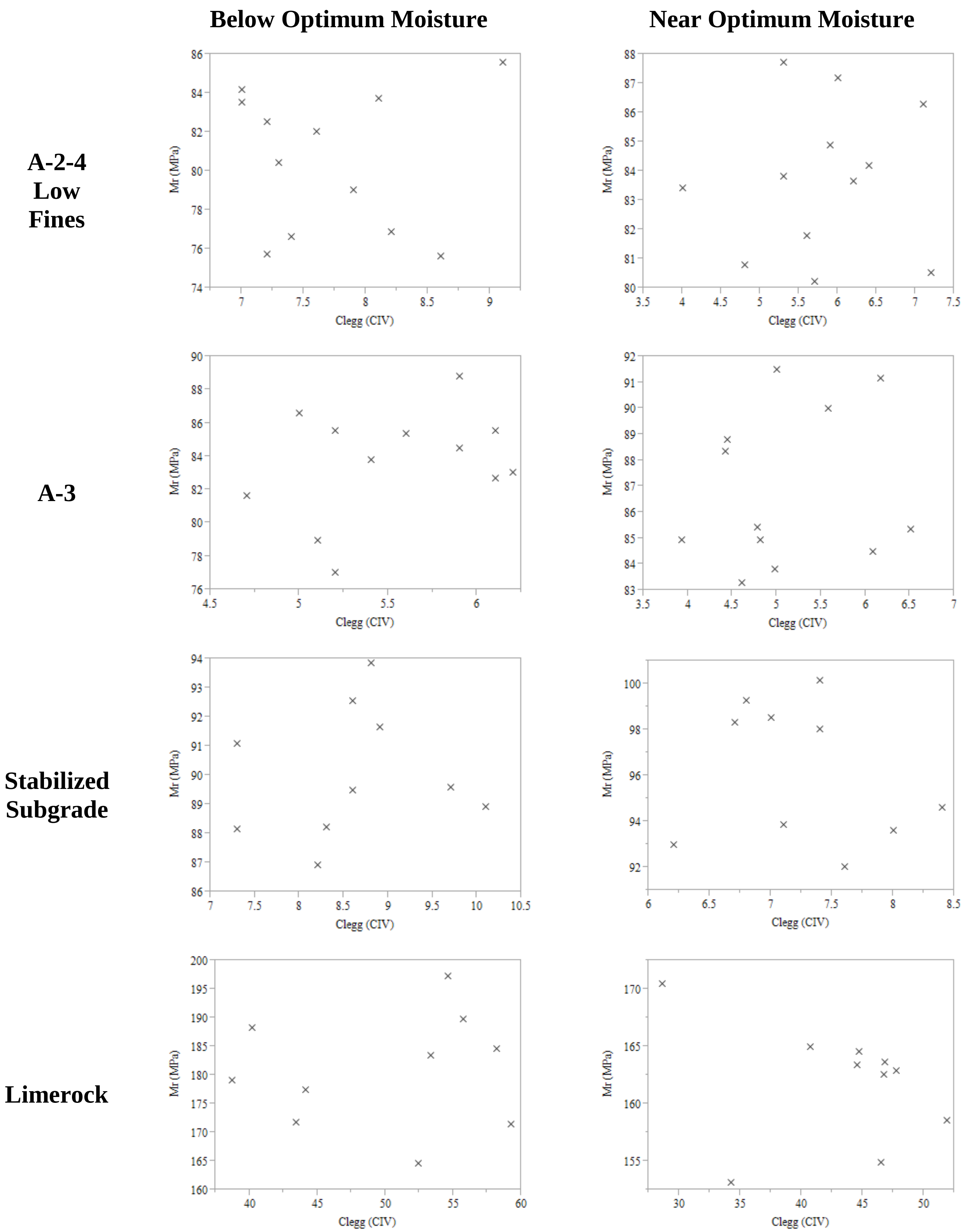


Figure F.1: Scatterplot of laboratory resilient modulus vs. CIST measurements (CIV)

(See Section F.1.4 for discussion)

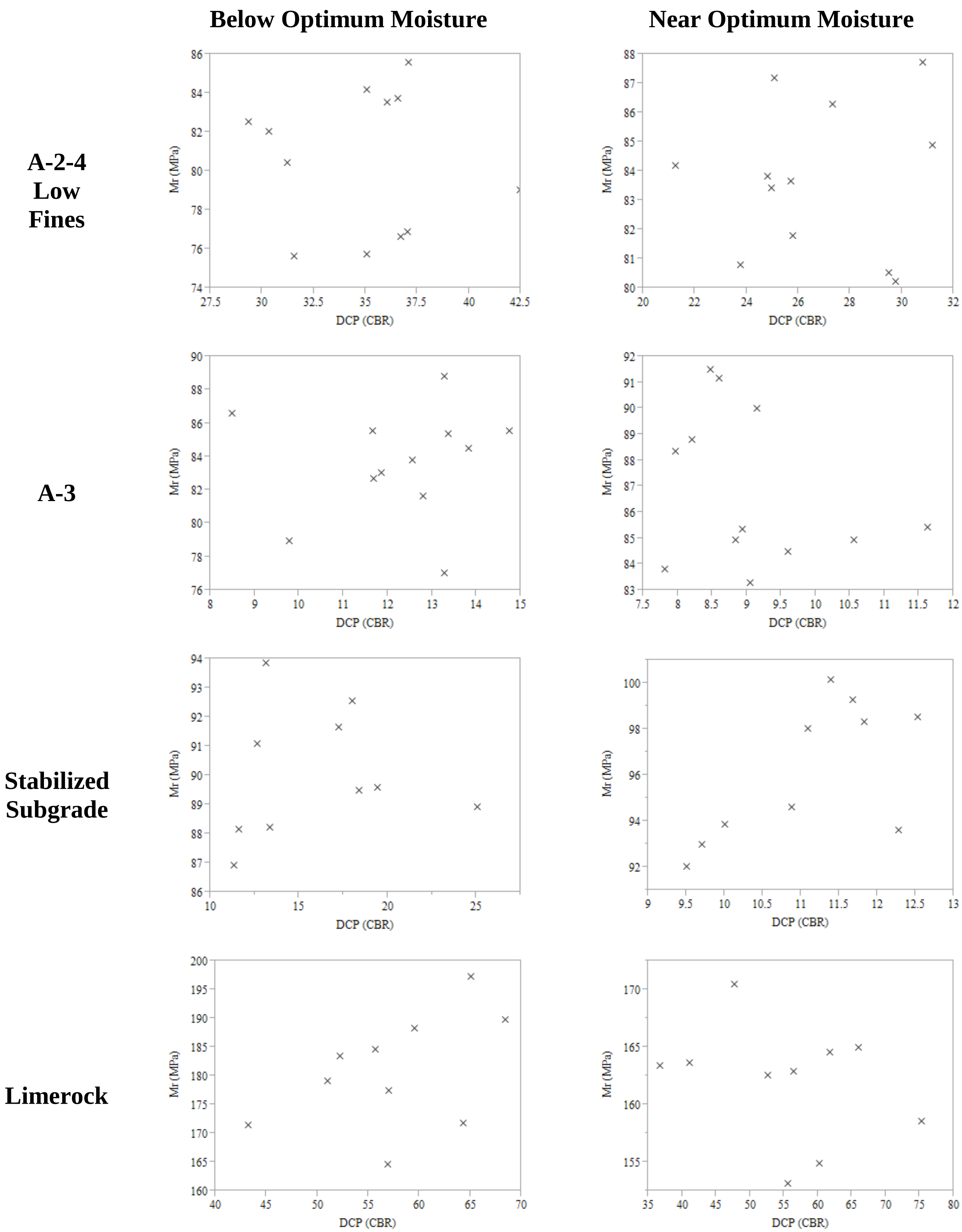


Figure F.2: Scatterplot of laboratory resilient modulus vs. DCP measurements (CBR)

(See Section F.1.4 for discussion)

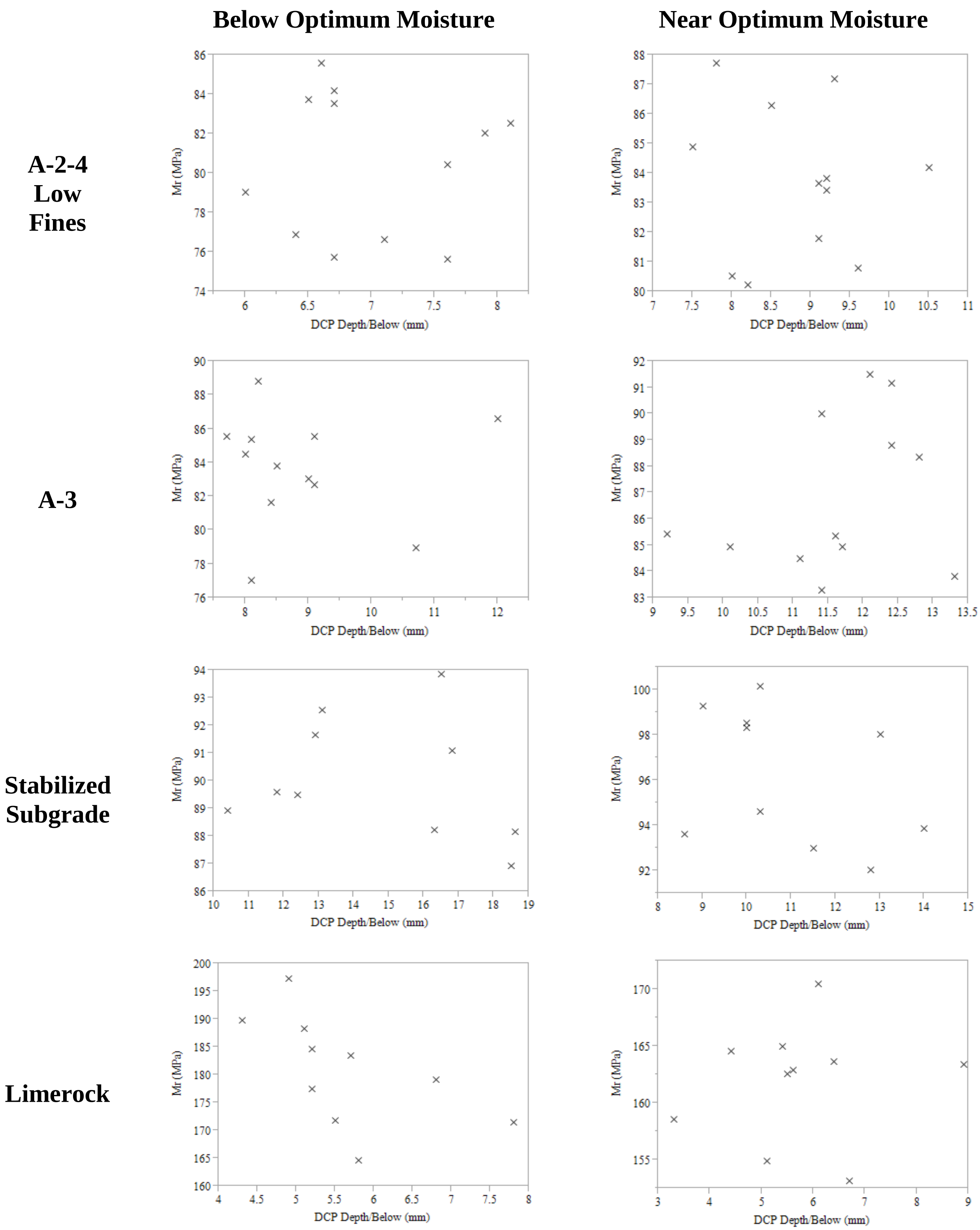


Figure F.3: Scatterplot of laboratory resilient modulus vs. DCP (depth per blow) measurements (mm)

(See Section F.1.4 for discussion)

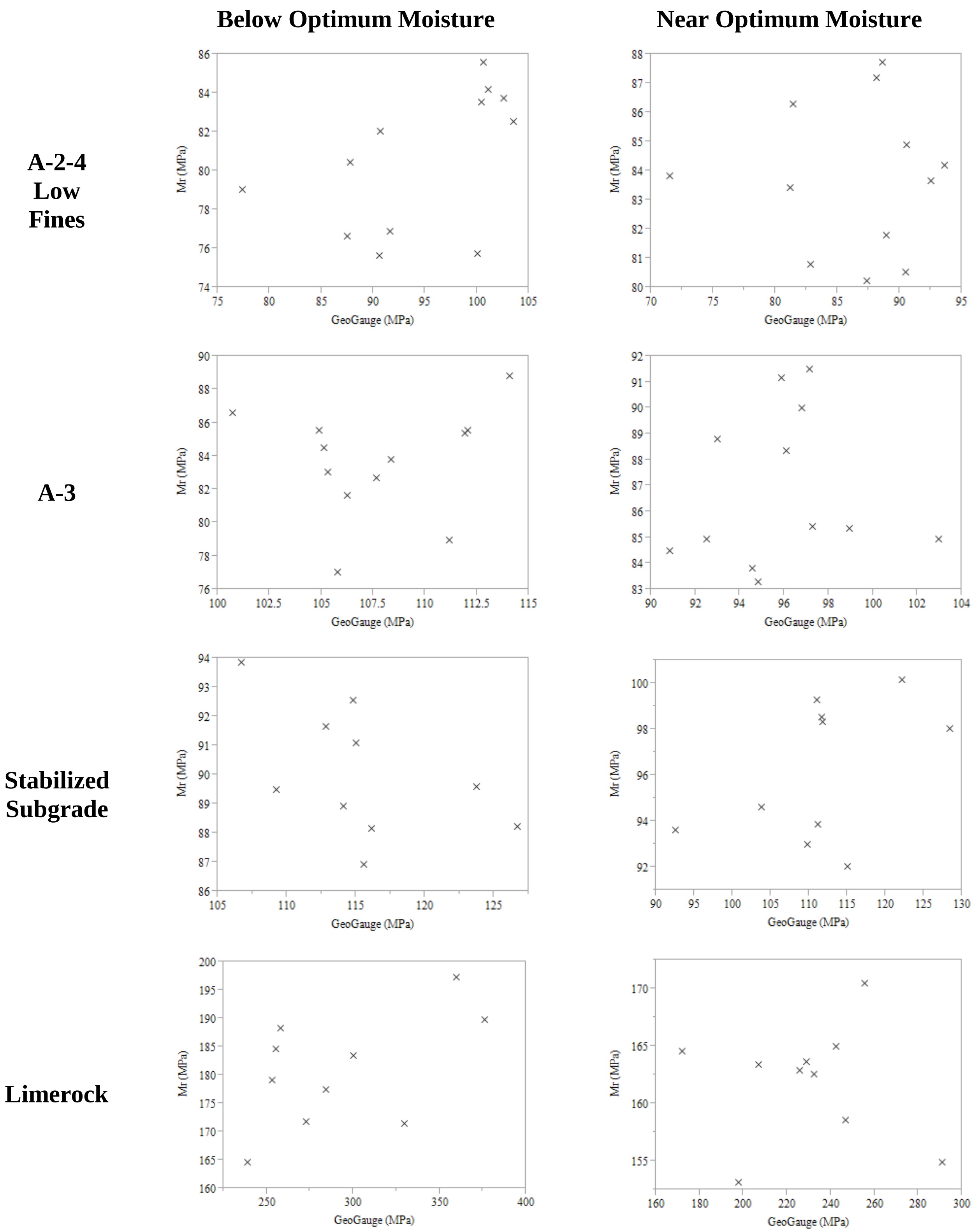


Figure F.4: Scatterplot of laboratory resilient modulus vs. GeoGauge measurements (MPa)

(See Section F.1.4 for discussion)

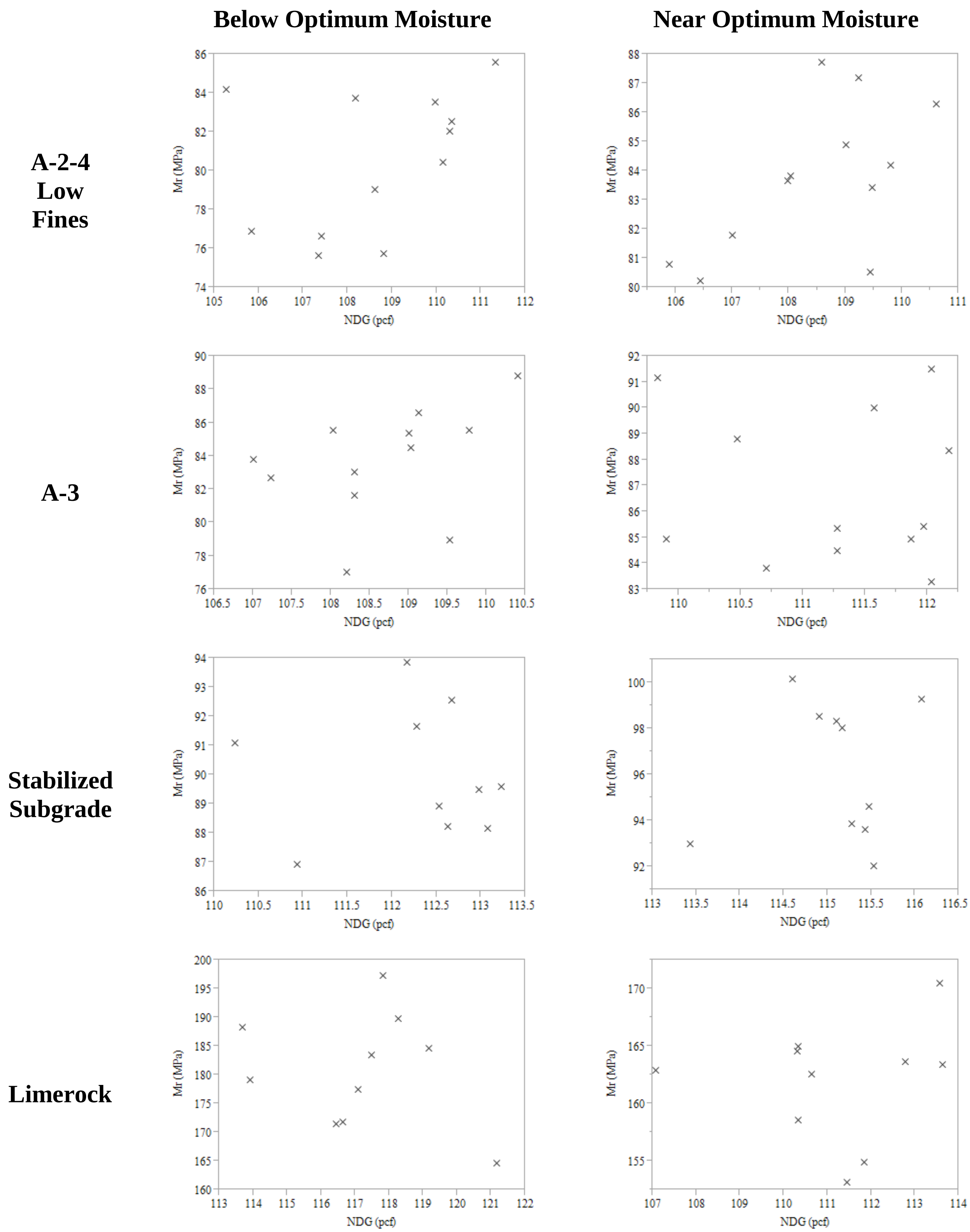


Figure F.5: Scatterplot of laboratory resilient modulus vs. NDG measurements (pcf)

(See Section F.1.4 for discussion.)

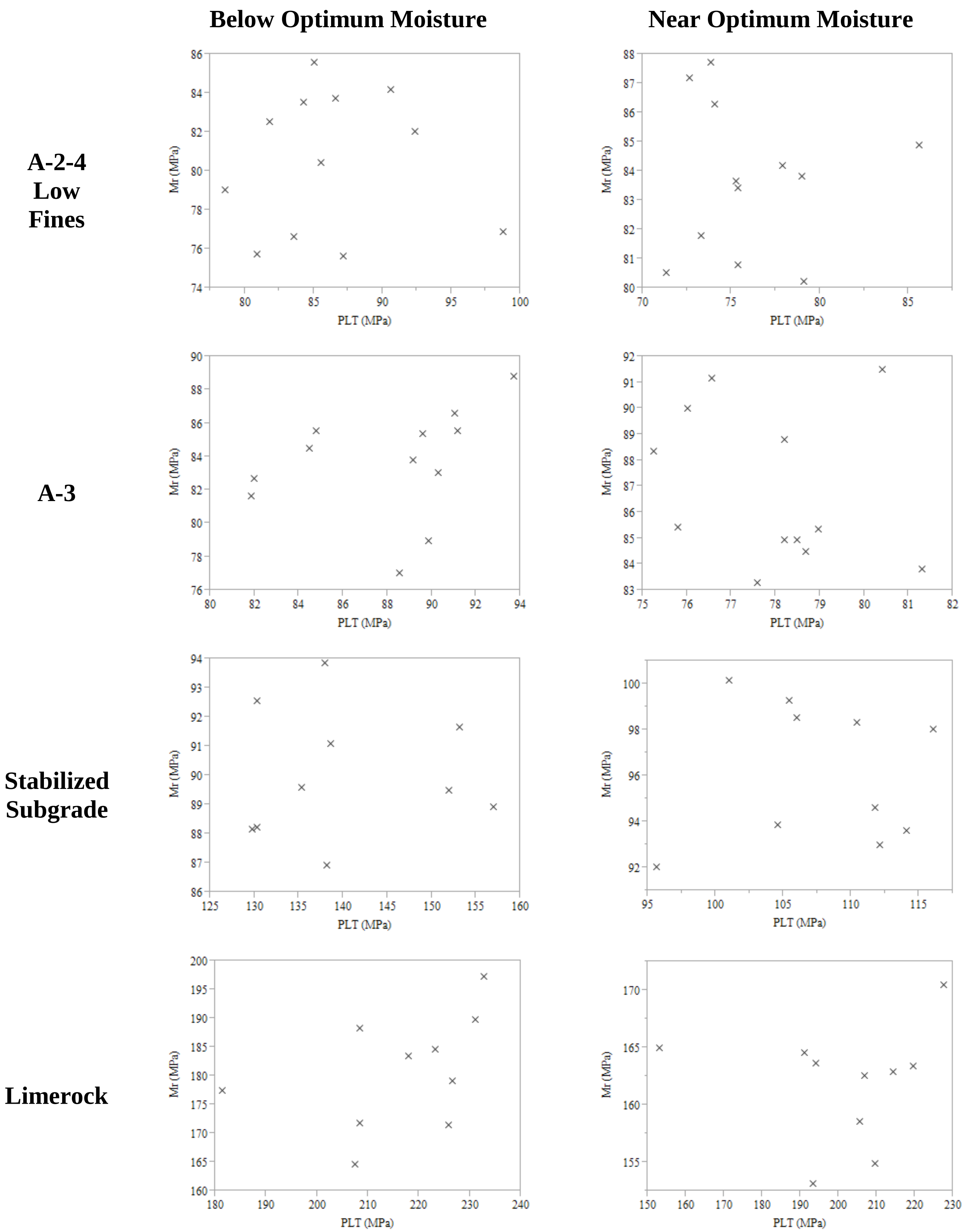


Figure F.6: Scatterplot of laboratory resilient modulus vs. PLT measurements (MPa)

(See Section F.1.4 for discussion)

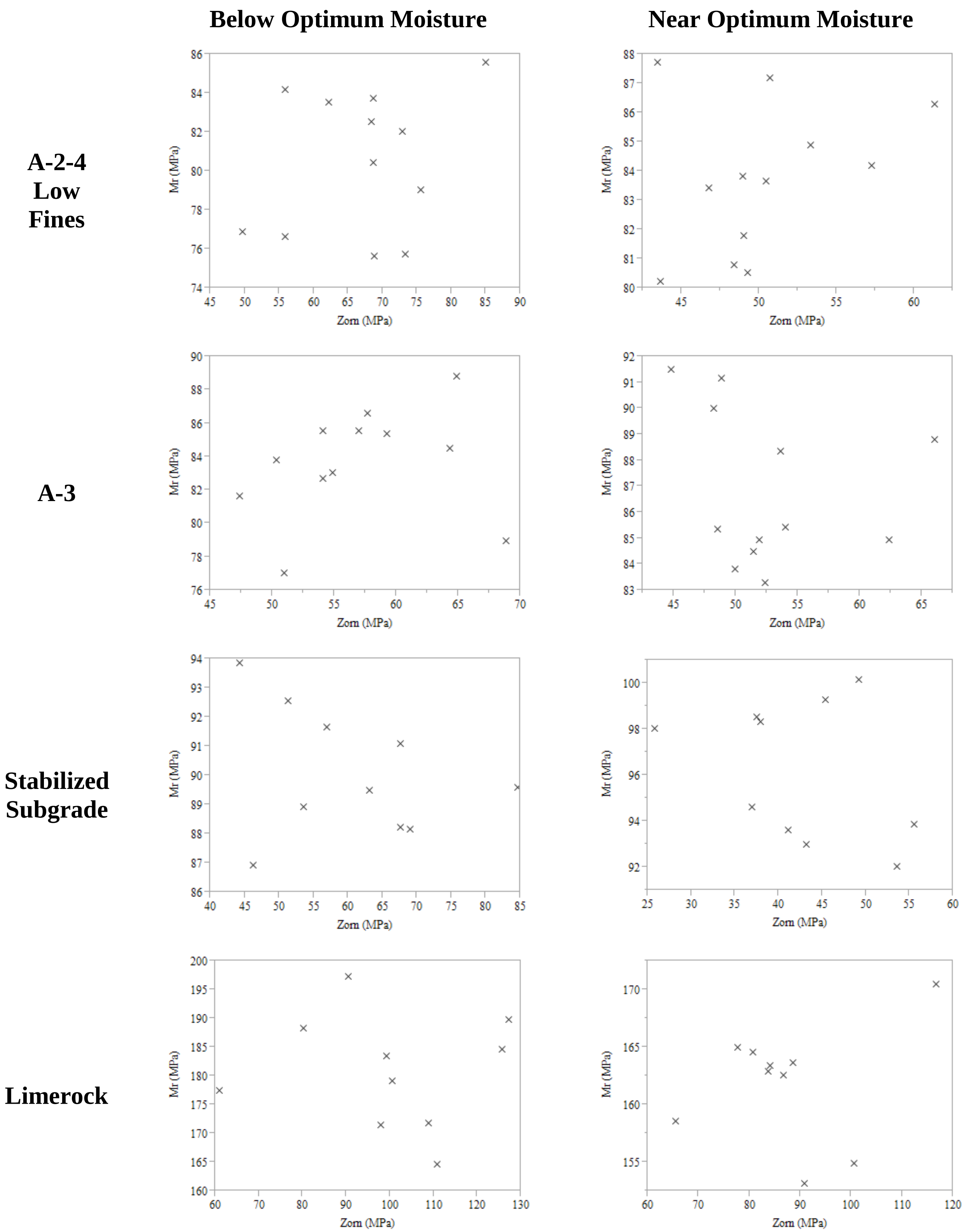


Figure F.7: Scatterplot of laboratory resilient modulus vs. LWD-1 measurements (MPa)

(See Section F.1.4 for discussion)

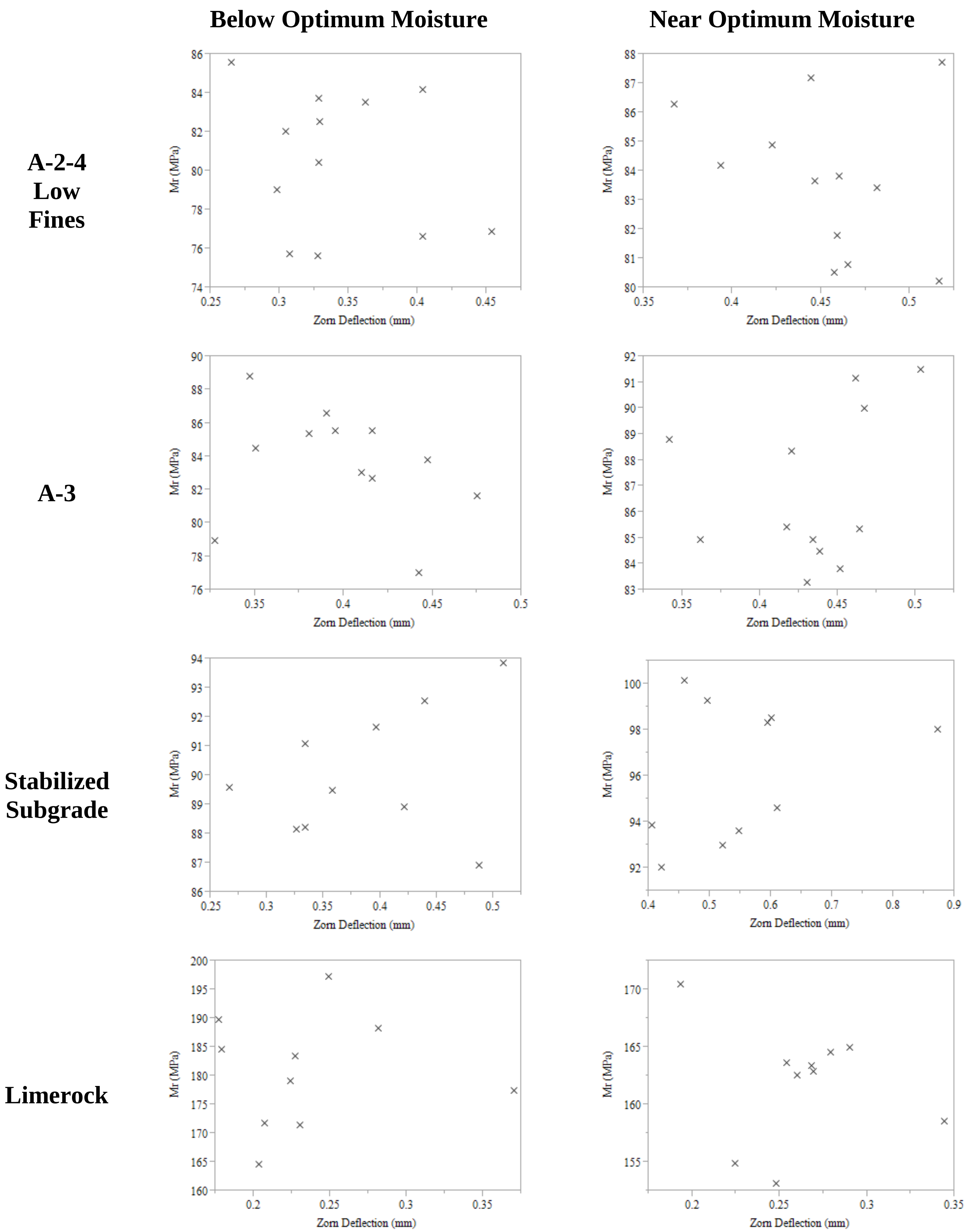


Figure F.8: Scatterplot of lab resilient modulus vs. LWD-1 deflection measurements (mm)

(See Section 1.4 for discussion)

F.2 Regression Analyses Overview

F.2.1 Regression Data Source

The data for regression analyses was obtained from the Single Spot Testing Sequence (SSTS) performed during the months of August, September, and October 2013.

F.2.2 Variables Selection

Testing activities were performed for four soil types (A-2-4 low fines, A-3, stabilized subgrade, and limerock base) under two different moisture conditions: (1) when soils are tested near the optimum moisture; and (2) when soils are tested below the optimum moisture. The correlation analyses discussed previously indicate that soil types and moisture conditions have an impact on the correlations between the equipment measurements and the laboratory M_R values. Therefore, the following variables are selected for the multiple regression analyses:

1. Dependent Variable: Laboratory resilient modulus values (M_R)
2. Independent Variable 1: Measurements from the equipment (i.e., CIST, GeoGauge, PLT, DCP, LWD-1, and NDG)
3. Independent Variable 2: Soil types (categorical variable; i.e., A-2-4 low fines, A-3, stabilized subgrade, and limerock base)
4. Independent Variable 3: Moisture conditions (categorical variable; i.e., below optimum and [near] optimum)

F.2.3 Model Development

For each piece of equipment, a regression model is first developed without interaction between the equipment measurements and soil types or moisture conditions. In such a model, the difference between the mean resilient modulus of soil types under either moisture condition is the same for all equipment measurements. In other words, the slope for the equipment measurements in the model is the same for each soil type under either moisture condition.

This model assumes no interaction between variables. However, this is neither necessarily nor always the case. In order to determine whether interactions between the equipment measurements and soil types or moisture conditions have an impact on the model, the effect test for interaction is conducted with the null hypothesis that there is no interaction. If there is evidence of an interaction (p-value <0.05 for the effect test), an interaction model is developed further. If there is not strong evidence of an interaction (p-value >0.05 for the effect test), the model is used without interactions.

Prediction expressions are provided for each model. Model details include: (1) Actual by Predicted Plot, (2) Summary of Fit, (3) Analysis of Variance, (4) Parameter Estimates, (5) Effect Tests, and (6) Residual by Predicted Plot.

F.2.4 Model Usage

Due to limited data sets, the linear regression models obtained from the Phase II data are likely to generate unacceptable errors if applied to in-place testing. Therefore, the regression models are used only for validation purposes for this research project, where the predicted laboratory M_R values from the regression models are compared with the field testing results on selected projects (i.e., I-95, US-301, SR-9B, CR-210, SR-23/Normandy Boulevard, and SR-23/New World Avenue).

With more data sets obtained from future research, more reliable regression models could be established and their effectiveness could be validated by applying those models to field testing results.

F.3 Regression Analyses for CIST

F.3.1 Regression Model without Interaction

F.3.1.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.969861	527.7426	<0.0001*

Note: (1) The R Square value indicates that 97.0% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 527.7 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.3.1.2 Prediction Expression

	Below Optimum (M_R in MPa; CIST in CIV)	Optimum (M_R in MPa; CIST in CIV)
A-2-4 low fines	M_R = 79.271 + 0.429 x CIST	M_R = 79.167 + 0.429 x CIST
A-3	M_R = 83.017 + 0.429 x CIST	M_R = 82.914 + 0.429 x CIST
Stabilized subgrade	M_R = 89.760 + 0.429 x CIST	M_R = 89.657 + 0.429 x CIST
Limerock base	M_R = 151.461 + 0.429 x CIST	M_R = 151.358 + 0.429 x CIST

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) The model assumes no interaction between equipment measurements and soil types or moisture conditions. In other words, the difference between the mean resilient modulus of soil types under either moisture condition is the same for all equipment measurements. Therefore, the slope for equipment measurements in the model is the same for each soil type under either moisture condition.

F.3.2 Effect Test for Interaction

Source	**Nparm**	**DF**	**Sum of Squares**	**F Ratio**	**Prob > F**
Soil Type	3	3	2477.1633	38.5555	<0.0001*
Condition	1	1	22.5918	1.0549	0.3076
CIST (CIV)	1	1	1.2561	0.0587	0.8093
CIST (CIV)*Soil Type	3	3	49.2258	0.7662	0.5165
CIST (CIV)*Condition	1	1	1576.7201	73.6220	<0.0001*

Conclusion: There is evidence of an interaction between the CIST measurements and moisture conditions with a p-value <0.0001. However, there is not strong evidence of an interaction between the CIST measurements and soil types.

F.3.3 Regression Model with Interaction

F.3.3.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.985431	913.1249	<0.0001*

Note: (1) The R Square value indicates that 98.5% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 913.1 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.3.3.2 Prediction Expression

	Below Optimum (M_R in MPa; CIST in CIV)	Optimum (M_R in MPa; CIST in CIV)
A-2-4 low fines	M_R = 78.215 + 0.256 x CIST	M_R = 85.821 – 0.307 x CIST
A-3	M_R = 81.521 + 0.256 x CIST	M_R = 89.127 – 0.307 x CIST
Stabilized subgrade	M_R = 89.320 + 0.256 x CIST	M_R = 96.925 – 0.307 x CIST
Limerock base	M_R = 167.840 + 0.256 x CIST	M_R = 175.445 – 0.307 x CIST

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) Interaction between equipment measurements and moisture conditions has been added to the model. In other words, the difference between the mean resilient modulus under the two moisture conditions is not the same for equipment measurements. Therefore, the slope for the equipment measurements in the model is different under the two moisture conditions.

F.4 Regression Analyses for GeoGauge

F.4.1 Regression Model without Interaction

F.4.1.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.978414	743.3509	<0.0001*

Note: (1) The R Square value indicates that 97.8% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 743.4 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.4.1.2 Prediction Expression

	Below Optimum (M_R and GeoGauge in MPa)	Optimum (M_R and GeoGauge in MPa)
A-2-4 low fines	M_R = 66.145 + 0.164 x GeoGauge	M_R = 68.359 + 0.164 x GeoGauge
A-3	M_R = 67.404 + 0.164 x GeoGauge	M_R = 69.618 + 0.164 x GeoGauge
Stabilized subgrade	M_R = 73.328 + 0.164 x GeoGauge	M_R = 75.542 + 0.164 x GeoGauge
Limerock base	M_R = 127.353 + 0.164 x GeoGauge	M_R = 129.567 + 0.164 x GeoGauge

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) The model assumes no interaction between equipment measurements and soil types or moisture conditions. In other words, the difference between the mean resilient modulus of soil types under either moisture condition is the same for all equipment measurements. Therefore, the slope for the equipment measurements in the model is the same for each soil type under either moisture condition.

F.4.2 Effect Test for Interaction

Source	Nparm	DF	Sum of Squares	F Ratio	Prob > F
Soil Type	3	3	5247.4600	94.6295	<0.0001*
Condition	1	1	3.8632	0.2090	0.6488
GeoGauge (MPa)	1	1	53.5785	2.8986	0.0926
GeoGauge (MPa)*Soil Type	3	3	29.2847	0.5281	0.6643
GeoGauge (MPa)*Condition	1	1	953.4958	51.5843	<0.0001*

Conclusion: There is evidence of an interaction between the GeoGauge measurements and moisture conditions with a p-value <0.0001. However, there is not strong evidence of an interaction between the GeoGauge measurements and soil types.

F.4.3 Regression Model with Interaction

F.4.3.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.987386	1056.758	<0.0001*

Note: (1) The R Square value indicates that 98.7% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 1056.8 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.4.3.2 Prediction Expression

	Below Optimum (M_R and GeoGauge in MPa)	Optimum (M_R and GeoGauge in MPa)
A-2-4 low fines	M_R = 68.144 + 0.1204 x GeoGauge	M_R = 84.533 + 0.00204 x GeoGauge
A-3	M_R = 70.465 + 0.1204 x GeoGauge	M_R = 86.853 + 0.00204 x GeoGauge
Stabilized subgrade	M_R = 77.843 + 0.1204 x GeoGauge	M_R = 94.231 + 0.00204 x GeoGauge
Limerock base	M_R = 145.341 + 0.1204 x GeoGauge	M_R = 161.730 + 0.00204 x GeoGauge

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) Interaction between equipment measurements and moisture conditions has been added to the model. In other words, the difference between the mean resilient modulus under the two moisture conditions is not the same for the equipment measurements. Therefore, the slope for the equipment measurements in the model is different under the two moisture conditions.

F.5 Regression Analyses for PLT

F.5.1 Regression Model without Interaction

F.5.1.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.968185	499.0733	<0.0001*

Note: (1) The R Square value indicates that 96.8% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 499.1 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.5.1.2 Prediction Expression

	Below Optimum (M_R and PLT in MPa)	Optimum (M_R and PLT in MPa)
A-2-4 low fines	M_R = 79.771 + 0.0328 x PLT	M_R = 79.140 + 0.0328 x PLT
A-3	M_R = 82.842 + 0.0328 x PLT	M_R = 82.211 + 0.0328 x PLT
Stabilized subgrade	M_R = 89.357 + 0.0328 x PLT	M_R = 88.726 + 0.0328 x PLT
Limerock base	M_R = 164.862 + 0.0328 x PLT	M_R = 164.231 + 0.0328 x PLT

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) The model assumes no interaction between equipment measurements and soil types or moisture conditions. In other words, the difference between the mean resilient modulus of soil types under either moisture condition is the same for all equipment measurements. Therefore, the slope for equipment measurements in the model is the same for each soil type under either moisture condition.

F.5.2 Effect Test for Interaction

Source	**Nparm**	**DF**	**Sum of Squares**	**F Ratio**	**Prob > F**
Soil Type	3	3	3237.3307	53.8616	<0.0001*
Condition	1	1	24.4486	1.2203	0.2727
PLT (MPa)	1	1	2.4132	0.1205	0.7295
PLT (MPa)*Soil Type	3	3	450.5084	7.4954	0.0002*
PLT (MPa)*Condition	1	1	1085.0686	54.1590	<0.0001*

Conclusion: There is evidence of an interaction between the PLT measurements and moisture conditions with a p-value <0.0001. There also is evidence of an interaction between the PLT measurements and soil types with a p-value of 0.0002.

F.5.3 Regression Model with Interaction

F.5.3.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.9866	638.1108	<0.0001*

Note: (1) The R Square value indicates that 98.7% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 638.1 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.5.3.2 Prediction Expression

	Below Optimum (M_R and PLT in MPa)	Optimum (M_R and PLT in MPa)
A-2-4 low fines	M_R = 66.041 + 0.163 x PLT	M_R = 84.506 – 0.00475 x PLT
A-3	M_R = 65.423 + 0.206 x PLT	M_R = 83.888 + 0.0387 x PLT
Stabilized subgrade	M_R = 108.967 - 0.130 x PLT	M_R = 127.432 – 0.297 x PLT
Limerock base	M_R = 132.304 + 0.224 x PLT	M_R = 150.769 + 0.0562 x PLT

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) Interaction between the equipment measurements and moisture conditions has been added to the model. Interaction between the equipment measurements and soil types has also been added to the model. In other words, the difference between the mean resilient modulus under the two moisture conditions is not the same for equipment measurements, and the difference between the mean resilient modulus for the four soil types is not the same for equipment measurements either. Therefore, the slope for the equipment measurements in the model is different for each soil type under either moisture condition.

F.6 Regression Analyses for DCP

F.6.1 Regression Model without Interaction

F.6.1.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.968169	498.8270	<0.0001*

Note: (1) The R Square value indicates that 96.8% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 498.8 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.6.1.2 Prediction Expression

	Below Optimum (M_R in MPa; DCP in %)	Optimum (M_R in MPa; DCP in %)
A-2-4 low fines	M_R = 84.733 – 0.0615 x DCP	M_R = 83.280 – 0.0615 x DCP
A-3	M_R = 86.629 – 0.0615 x DCP	M_R = 85.176 – 0.0615 x DCP
Stabilized subgrade	M_R = 94.663 – 0.0615 x DCP	M_R = 93.210 – 0.0615 x DCP
Limerock base	M_R = 175.581 – 0.0615 x DCP	M_R = 174.128 – 0.0615 x DCP

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) The model assumes no interaction between the equipment measurements and soil types or moisture conditions. In other words, the difference between the mean resilient modulus of soil types under either moisture condition is the same for all equipment measurements. Therefore, the slope for the equipment measurements in the model is the same for each soil type under either moisture condition.

F.6.2 Effect Test for Interaction

Source	**Nparm**	**DF**	**Sum of Squares**	**F Ratio**	**Prob > F**
Soil Type	3	3	11243.111	177.4947	<0.0001*
Condition	1	1	49.266	2.3333	0.1307
DCP (CBR)	1	1	0.029	0.0014	0.9705
DCP (CBR)*Soil Type	3	3	273.085	4.3112	0.0072*
DCP (CBR)*Condition	1	1	1654.584	78.3626	<0.0001*

Conclusion: There is evidence of an interaction between the DCP measurements and moisture conditions with a p-value <0.0001. There also is evidence of an interaction between the DCP measurements and soil types with a p-value of 0.0072.

F.6.3 Regression Model with Interaction

F.6.3.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.985878	605.0407	<0.0001*

Note: (1) The R Square value indicates that 98.6% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 605.0 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.6.3.2 Prediction Expression

	Below Optimum (M_R in MPa; DCP in %)	Optimum (M_R in MPa; DCP in %)
A-2-4 low fines	M_R = 91.836 – 0.288 x DCP	M_R = 104.096 – 0.813 x DCP
A-3	M_R = 73.195 + 0.778 x DCP	M_R = 85.455 + 0.253 x DCP
Stabilized subgrade	M_R = 88.100 + 0.132 x DCP	M_R = 100.361 – 0.393 x DCP
Limerock base	M_R = 154.080 + 0.456 x DCP	M_R = 166.340 – 0.069 x DCP

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) Interaction between equipment measurements and moisture conditions has been added to the model. Interaction between the equipment measurements and soil types has also been added to the model. In other words, the difference between the mean resilient modulus under the two moisture conditions is not the same for the equipment measurements, and the difference between the mean resilient modulus for the four soil types is not the same for equipment measurements either. Therefore, the slope for the equipment measurements in the model is different for each soil type under either moisture condition.

F.7 Regression Analyses for LWD-1

F.7.1 Regression Model without Interaction

F.7.1.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.968095	497.6194	<0.0001*

Note: (1) The R Square value indicates that 96.8% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 497.6 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.7.1.2 Prediction Expression

	Below Optimum (M_R and LWD-1 in MPa)	Optimum (M_R and LWD-1 in MPa)
A-2-4 low fines	M_R = 82.652 + 0.0007 x LWD-1	M_R = 81.496 + 0.0007 x LWD-1
A-3	M_R = 85.787 + 0.0007 x LWD-1	M_R = 84.631 + 0.0007 x LWD-1
Stabilized subgrade	M_R = 93.646 + 0.0007 x LWD-1	M_R = 92.490 + 0.0007 x LWD-1
Limerock base	M_R = 171.905 + 0.0007 x LWD-1	M_R = 170.749 + 0.0007 x LWD-1

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) The model assumes no interaction between the equipment measurements and soil types or moisture conditions. In other words, the difference between the mean resilient modulus of soil types under either moisture condition is the same for all equipment measurements. Therefore, the slope for the equipment measurements in the model is the same for each soil type under either moisture condition.

F.7.2 Effect Test for Interaction

Source	**Nparm**	**DF**	**Sum of Squares**	**F Ratio**	**Prob > F**
Soil Type	3	3	28958.353	345.5843	<0.0001*
Condition	1	1	90.473	3.2391	0.0758
LWD-1 (MPa)	1	1	47.828	1.7123	0.1945
LWD-1 (MPa)*Soil Type	3	3	55.959	0.6678	0.5743
LWD-1 (MPa)*Condition	1	1	1069.353	38.2845	<0.0001*

Conclusion: There is evidence of an interaction between the LWD-1 measurements and moisture conditions with a p-value <0.0001. There is no strong evidence of an interaction between the LWD-1 measurements and soil types.

F.7.3 Regression Model with Interaction

F.7.3.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.980839	691.0475	<0.0001*

Note: (1) The R Square value indicates that 98.1% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 691.0 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.7.3.2 Prediction Expression

	Below Optimum (M_R and LWD-1 in MPa)	Optimum (M_R and LWD-1 in MPa)
A-2-4 low fines	M_R = 71.771 + 0.1437 x LWD-1	M_R = 97.252 - 0.2873 x LWD-1
A-3	M_R = 75.975 + 0.1437 x LWD-1	M_R = 101.457 - 0.2873 x LWD-1
Stabilized subgrade	M_R = 82.148 + 0.1437 x LWD-1	M_R = 107.630 - 0.2873 x LWD-1
Limerock base	M_R = 164.020 + 0.1437 x LWD-1	M_R = 189.501 - 0.2873 x LWD-1

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) Interaction between equipment measurements and moisture conditions has been added to the model. In other words, the difference between the mean resilient modulus under the two moisture conditions is not the same for the equipment measurements. Therefore, the slope for the equipment measurements in the model is different under the two moisture conditions.

F.8 Regression Analyses for NDG

F.8.1 Regression Model without Interaction

F.8.1.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.978721	754.2973	<0.0001*

Note: (1) The R Square value indicates that 97.9% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 754.3 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.8.1.2 Prediction Expression

	Below Optimum (M_R in MPa; NDG in pcf)	Optimum (M_R in MPa; NDG in pcf)
A-2-4 low fines	M_R = -101.100 + 1.693 x NDG	M_R = -102.176 + 1.693 x NDG
A-3	M_R = -100.377 + 1.693 x NDG	M_R = -101.452 + 1.693 x NDG
Stabilized subgrade	M_R = -98.826 + 1.693 x NDG	M_R = -99.902 + 1.693 x NDG
Limerock Base	M_R = -21.368 + 1.693 x NDG	M_R = -22.443 + 1.693 x NDG

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) The model assumes no interaction between the equipment measurements and soil types or moisture conditions. In other words, the difference between the mean resilient modulus of soil types under either moisture condition is the same for all equipment measurements. Therefore, the slope for the equipment measurements in the model is the same for each soil type under either moisture condition.

F.8.2 Effect Test for Interaction

Source	**Nparm**	**DF**	**Sum of Squares**	**F Ratio**	**Prob > F**
Soil Type	3	3	49587.243	561.1686	<0.0001*
Condition	1	1	18.097	0.6144	0.4355
NDG (pcf)	1	1	486.283	16.5095	0.0001*
NDG (pcf)*Soil Type	3	3	107.800	1.2200	0.3081
NDG (pcf)*Condition	1	1	97.215	3.3005	0.0731

Conclusion: There is no strong evidence of an interaction between the NDG measurements and either soil types or moisture conditions at the significance level of 0.05. At the level of 0.1, however, there is evidence of an interaction between the NDG measurements and moisture conditions. For comparison purposes, a model with interaction between the two also was developed.

F.8.3 Regression Model with Interaction

F.8.3.1 Model Summary

R Square	F Ratio	Prob > F (p-value)
0.979376	641.0660	<0.0001*

Note: (1) The R Square value indicates that 97.9% of variation in the response can be attributed to the model rather than to random error. (2) The F Ratio is used to test whether the model differs significantly from a model where all predicted values are the response mean. (3) The p-value indicates the probability of obtaining an F Ratio as large as 641.1 is less than 0.01%, given that all parameters except the intercept are zero. Therefore, there is evidence that there is at least one significant effect in the model.

F.8.3.2 Prediction Expression

	Below Optimum (M_R in MPa; NDG in pcf)	Optimum (M_R in MPa; NDG in pcf)
A-2-4 low fines	M_R = -124.993 + 1.904 x NDG	M_R = -47.066 + 1.194 x NDG
A-3	M_R = -123.573 + 1.904 x NDG	M_R = -45.647 + 1.194 x NDG
Stabilized subgrade	M_R = -121.445 + 1.904 x NDG	M_R = -43.519 + 1.194 x NDG
Limerock Base	M_R = -45.476 + 1.904 x NDG	M_R = 32.450 + 1.194 x NDG

Note: (1) The prediction expression shows the equation used to predict the response (i.e., laboratory M_R), based on equipment measurements, soil types, and moisture conditions. (2) Interaction between the equipment measurements and moisture conditions has been added to the model. In other words, the difference between the mean resilient modulus under the two moisture conditions is not the same for the equipment measurements. Therefore, the slope for the equipment measurements in the model is different under the two moisture conditions.

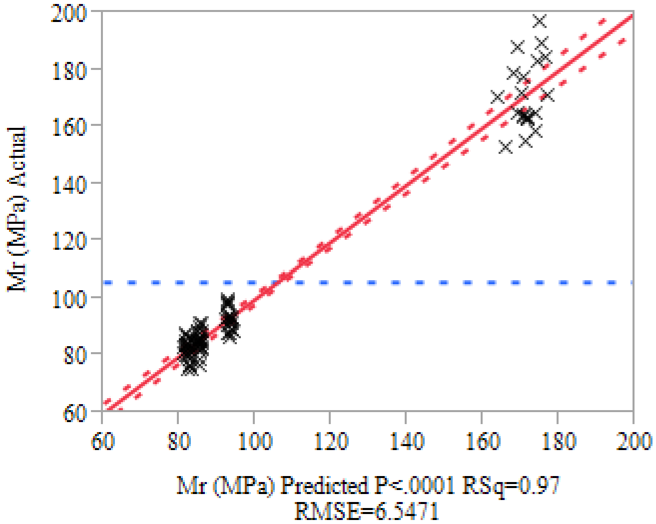
F.9 Regression Model Details

Details for each regression model (with or without interactions) are provided in the following subsections. Model details include: (1) Actual by Predicted Plot, (2) Summary of Fit, (3) Analysis of Variance, (4) Parameter Estimates, and (5) Residual by Predicted Plot.

F.9.1 Regression Models for CIST

F.9.1.1 Model without Interaction

(1) Actual by Predicted Plot



The Actual by Predicted Plot shows the actual (observed) M_R values against the predicted M_R values. As the predicted values come closer to the actual values, the points on the scatterplot fall closer around the red line.

(2) Summary of Fit

R Square	0.969861
R Square Adj	0.968023
Root Mean Square Error	6.547122
Mean of Response	105.757
Observations (or Sum Wgts)	88

R Square estimates the proportion of variation in the response that can be attributed to the model rather than to random error. R Square Adj adjusts R Square for the number of parameters in the model. Root Mean Square Error estimates the standard deviation of the random error. Mean of Response shows the overall mean of the response values. Observations (or Sum Wgts) gives the number of observations used in the model.

(3) Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	5	113107.93	22621.6	527.7426
Error	82	3514.91	42.9	**Prob > F**
C. Total	87	116622.84		<0.0001*

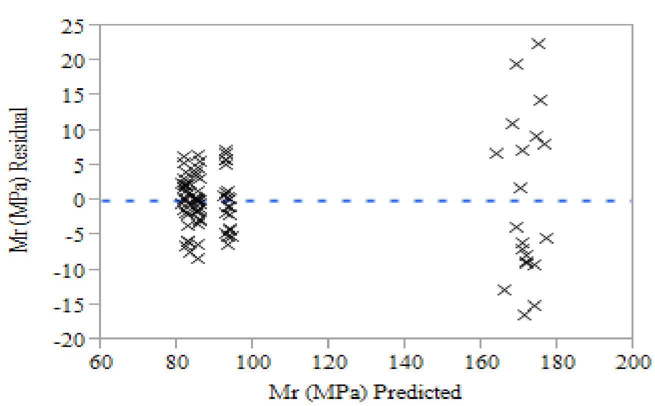
The Analysis of Variance provides the calculations for comparing the fitted model to a model where all predicted values equal the response mean. The Prob > F value measures the probability of obtaining an F Ratio as large as what is observed, given that all parameters except the intercept are zero. Small values of Prob > F indicate that the observed F Ratio is unlikely. Such values are considered evidence that there is at least one significant effect in the model.

(4) Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	100.82559	3.331145	30.27	<0.0001*
CIST (CIV)	0.4287674	0.195591	2.19	0.0312*
Soil Type [A-2-4 low fines]	-21.60653	2.265055	-9.54	<0.0001*
Soil Type [A-3]	-17.86017	2.508846	-7.12	<0.0001*
Soil Type [limerock base]	50.583905	5.990809	8.44	<0.0001*
Condition [below optimum]	0.051762	0.738705	0.07	0.9443

The Parameter Estimates provides the estimates of the model parameters and, for each parameter, gives a t test for the hypothesis that it equals zero. The t Ratio tests whether the true value of the parameter is zero. Prob > |t| lists the p-value for the test that the true parameter value is zero, against the two-sided alternative that it is not.

(5) Residual by Predicted Plot

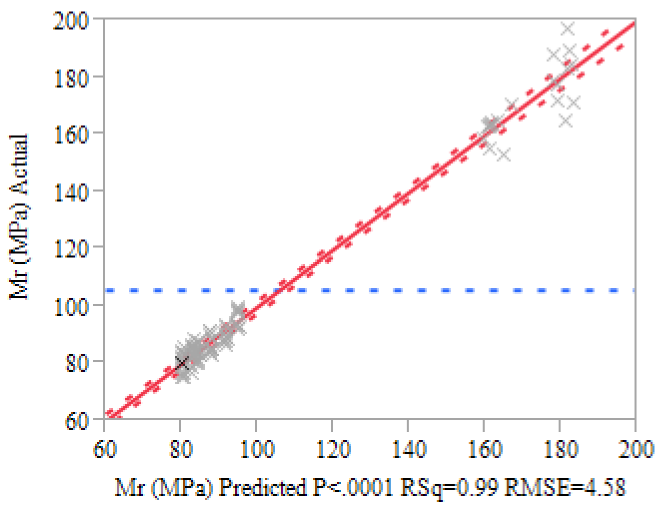


The Residual by Predicted Plot shows the residuals plotted against the predicted values of laboratory M_R. Residual values scattered randomly about zero are desired.

The regression reports in the subsequent models are similar to the model discussed and will not be noted again.

F.9.1.2 Interaction Model

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.985431
R Square Adj	0.984352
Root Mean Square Error	4.579987
Mean of Response	105.757
Observations (or Sum Wgts)	88

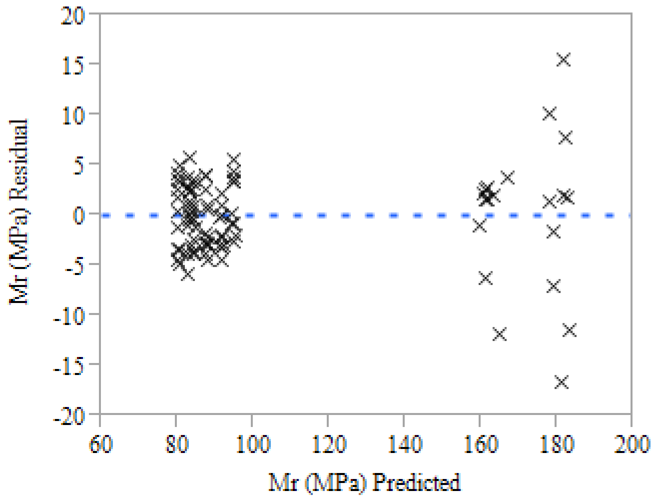
(3) Analysis of Variance

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**
Model	6	114923.77	19154.0	913.1249
Error	81	1699.08	21.0	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	108.02674	2.455448	43.99	<0.0001*
Soil Type [A-2-4 low fines]	-26.00876	1.653636	-15.73	<0.0001*
Soil Type [A-3]	-22.70268	1.830592	-12.40	<0.0001*
Soil Type [limerock base]	63.615759	4.418691	14.40	<0.0001*
Condition [below optimum]	0.6142455	0.52028	1.18	0.2412
CIST (CIV)	-0.025765	0.145284	-0.18	0.8597
Condition [below optimum] x ((CIST (CIV)-15.6852))	0.2816191	0.030268	9.30	<0.0001*

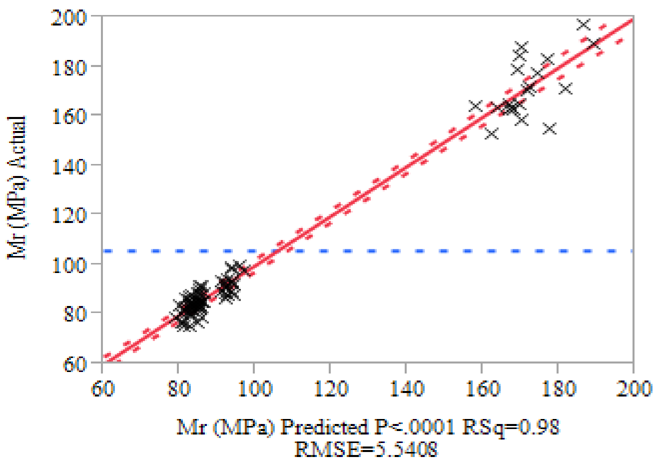
(5) Residual by Predicted Plot



F.9.2 Regression Models for GeoGauge

F.9.2.1 Model without Interaction

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.978414
R Square Adj	0.977098
Root Mean Square Error	5.540784
Mean of Response	105.757
Observations (or Sum Wgts)	88

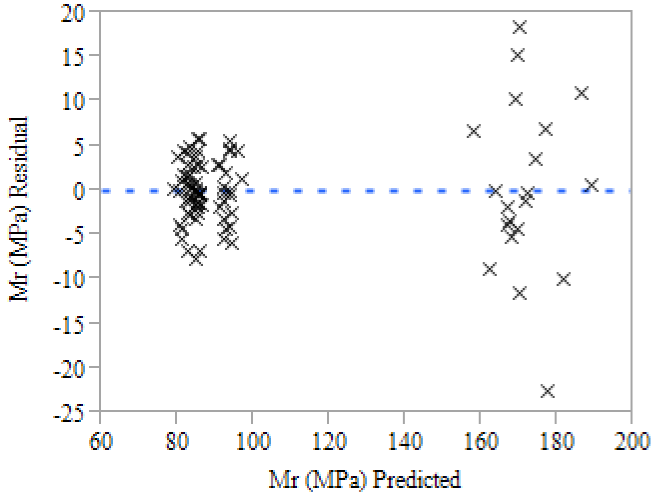
(3) Analysis of Variance

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**
Model	5	114105.42	22821.1	743.3509
Error	82	2517.42	30.7	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

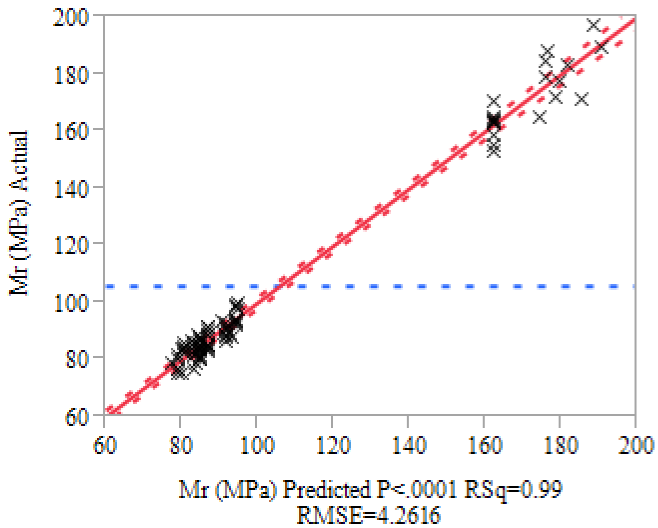
Term	**Estimate**	**Std Error**	**t Ratio**	**Prob>|t|**
Intercept	84.664415	3.7684	22.47	<0.0001*
Soil Type [A-2-4 low fines]	-17.41239	1.675492	-10.39	<0.0001*
Soil Type [A-3]	-16.15328	1.4459	-11.17	<0.0001*
Soil Type [limerock base]	43.795153	3.309311	13.23	<0.0001*
Condition [below optimum]	-1.106942	0.649357	-1.70	0.0920
GeoGauge (MPa)	0.1643546	0.02625	6.26	<0.0001*

(5) Residual by Predicted Plot



F.9.2.2 Interaction Model

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.987386
R Square Adj	0.986452
Root Mean Square Error	4.261593
Mean of Response	105.757
Observations (or Sum Wgts)	88

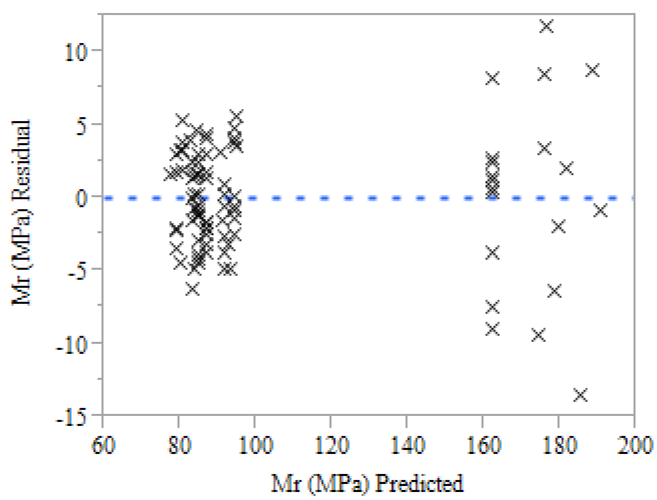
(3) Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	6	115151.79	19192.0	1056.758
Error	81	1471.06	18.2	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	98.642498	3.433935	28.73	<0.0001*
Soil Type [A-2-4 low fines]	-22.3039	1.440821	-15.48	<0.0001*
Soil Type [A-3]	-19.98321	1.221199	-16.36	<0.0001*
Soil Type [limerock base]	54.892915	2.935328	18.70	<0.0001*
Condition [below optimum]	-0.047134	0.518591	-0.09	0.9278
GeoGauge (MPa)	0.0612445	0.024334	2.52	0.0138*
((GeoGauge (MPa)-137.619)) x Condition [below optimum]	0.059175	0.007796	7.59	<0.0001*

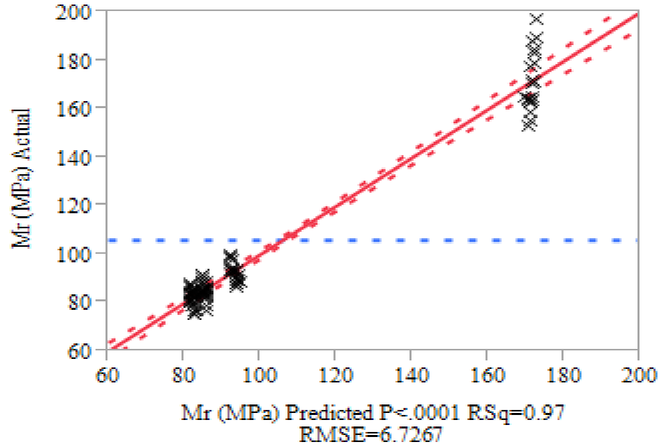
(5) Residual by Predicted Plot



F.9.3 Regression Models for PLT

F.9.3.1 Model without Interaction

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.968185
R Square Adj	0.966245
Root Mean Square Error	6.726726
Mean of Response	105.757
Observations (or Sum Wgts)	88

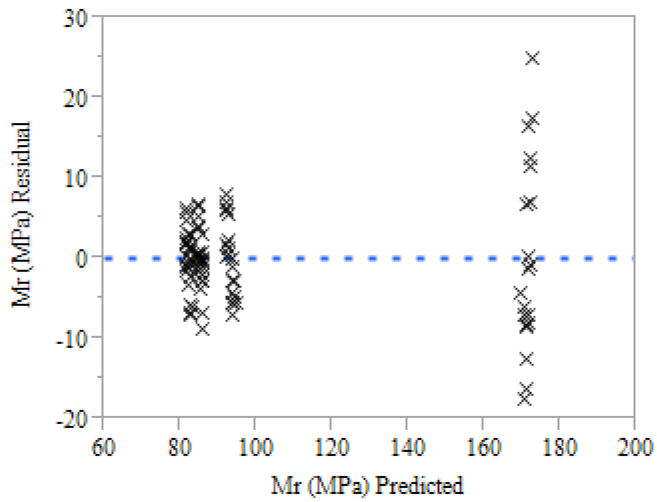
(3) Analysis of Variance

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**
Model	5	112912.44	22582.5	499.0733
Error	82	3710.40	45.2	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

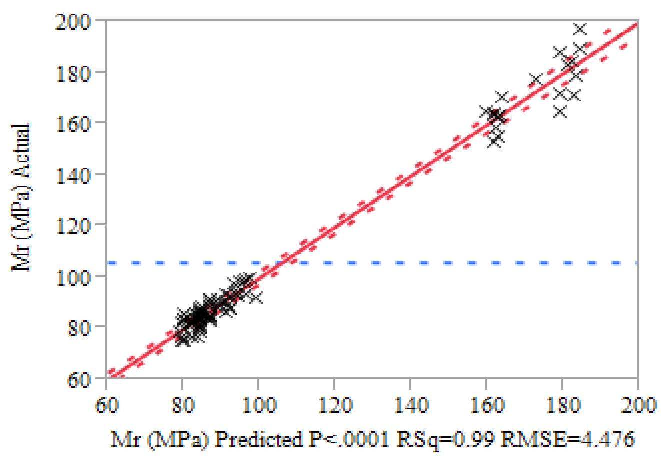
Term	**Estimate**	**Std Error**	**t Ratio**	**Prob>|t|**
Intercept	103.89243	8.48408	12.25	<0.0001*
Soil Type [A-2-4 low fines]	-24.43731	3.171132	-7.71	<0.0001*
Soil Type [A-3]	-21.36583	3.055753	-6.99	<0.0001*
Soil Type [limerock base]	60.653989	5.899412	10.28	<0.0001*
Condition [below optimum]	0.3155149	0.906123	0.35	0.7286
PLT (MPa)	0.0327785	0.068046	0.48	0.6313

(5) Residual by Predicted Plot



F.9.3.2 Interaction Model

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.9866
R Square Adj	0.985054
Root Mean Square Error	4.476031
Mean of Response	105.757
Observations (or Sum Wgts)	88

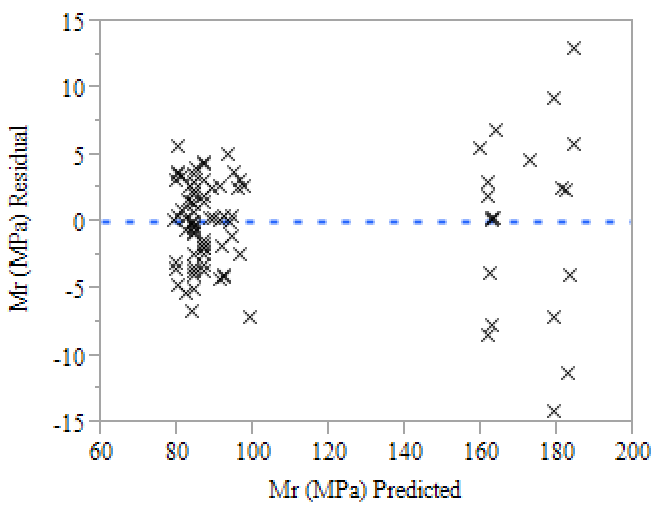
(3) Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	9	115060.13	12784.5	638.1108
Error	78	1562.72	20.0	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	102.41621	8.58638	11.93	<0.0001*
Soil Type [A-2-4 low fines]	-21.48136	4.857998	-4.42	<0.0001*
Soil Type [A-3]	-16.86332	5.810835	-2.90	0.0048*
Soil Type [limerock base]	52.117868	5.043581	10.33	<0.0001*
Condition [below optimum]	0.8568534	0.775662	1.10	0.2727
PLT (MPa)	0.0320249	0.092274	0.35	0.7295
((PLT (MPa)-120.399)) x Soil Type [A-2-4 low fines]	0.047025	0.115607	0.41	0.6853
((PLT (MPa)-120.399)) x Soil Type [A-3]	0.0905078	0.145506	0.62	0.5357
((PLT (MPa)-120.399)) x Soil Type [limerock base]	0.1079582	0.097979	1.10	0.2739
((PLT (MPa)-120.399)) x Condition [below optimum]	0.0838089	0.011388	7.36	<0.0001*

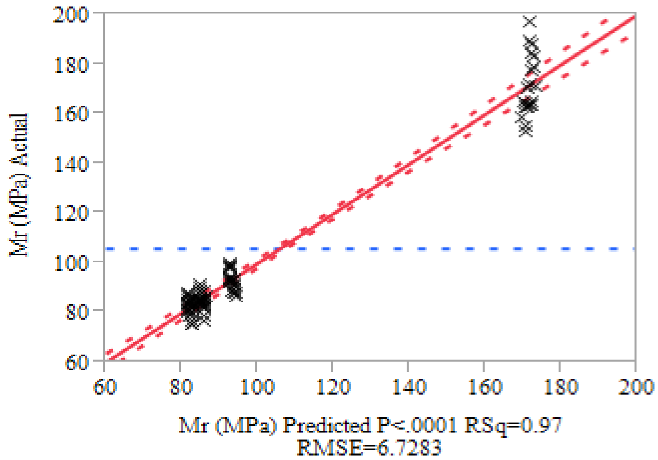
(5) Residual by Predicted Plot



F.9.4 Regression Models for DCP

F.9.4.1 Model without Interaction

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.968169
R Square Adj	0.966228
Root Mean Square Error	6.728334
Mean of Response	105.757
Observations (or Sum Wgts)	88

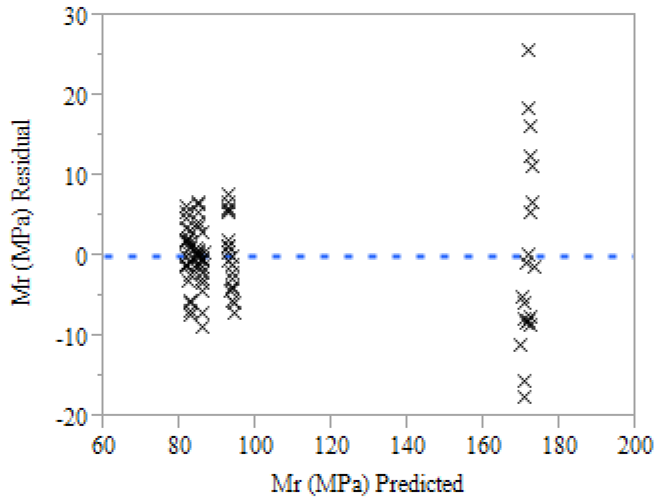
(3) Analysis of Variance

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**
Model	5	112910.67	22582.1	498.8270
Error	82	3712.18	45.3	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

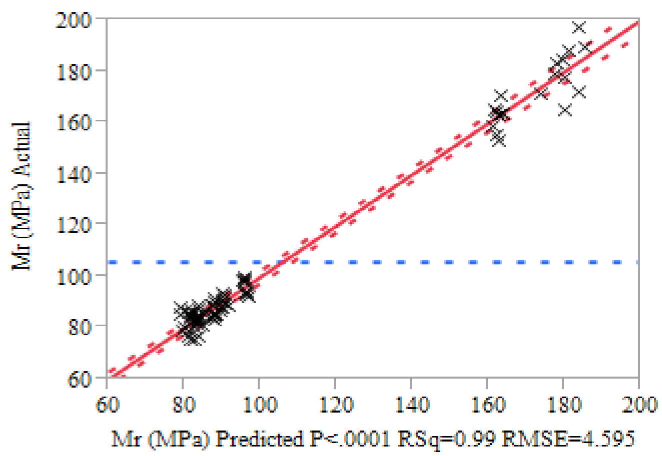
Term	**Estimate**	**Std Error**	**t Ratio**	**Prob>|t|**
Intercept	109.67513	3.962284	27.68	<0.0001*
Soil Type [A-2-4 low fines]	-25.66888	1.277184	-20.10	<0.0001*
Soil Type [A-3]	-23.77233	2.689208	-8.84	<0.0001*
Soil Type [limerock base]	65.179444	4.191936	15.55	<0.0001*
Condition [below optimum]	0.7265006	0.788814	0.92	0.3598
DCP (CBR)	-0.061499	0.140081	-0.44	0.6618

(5) Residual by Predicted Plot



F.9.4.2 Interaction Model

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.985878
R Square Adj	0.984249
Root Mean Square Error	4.595046
Mean of Response	105.757
Observations (or Sum Wgts)	88

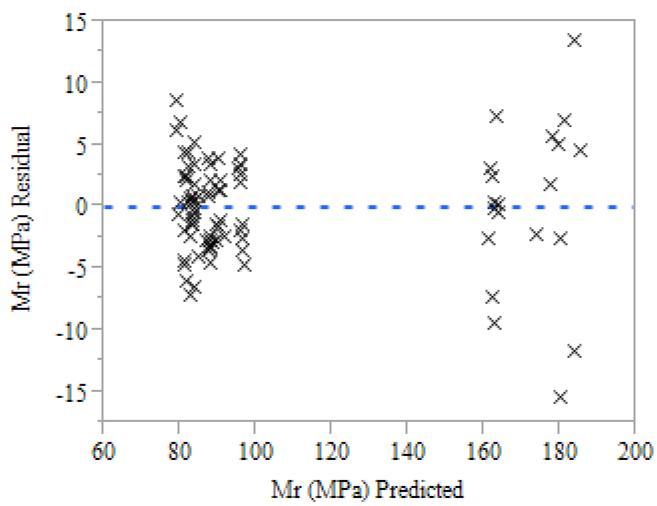
(3) Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	9	114975.92	12775.1	605.0407
Error	78	1646.93	21.1	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

Term	**Estimate**	**Std Error**	**t Ratio**	**Prob>|t|**
Intercept	107.93288	3.305194	32.66	<0.0001*
Soil Type [A-2-4 low fines]	-25.1158	3.054418	-8.22	<0.0001*
Soil Type [A-3]	-14.78738	6.718527	-2.20	0.0307*
Soil Type [limerock base]	57.352119	3.74917	15.30	<0.0001*
Condition [below optimum]	1.0015204	0.655657	1.53	0.1307
DCP (CBR)	0.0071068	0.191387	0.04	0.9705
((DCP (CBR)-27.1685)) x Soil Type [A-2-4 low fines]	-0.557562	0.201488	-2.77	0.0071*
((DCP (CBR)-27.1685)) x Soil Type [A-3]	0.5087187	0.389767	1.31	0.1957
((DCP (CBR)-27.1685)) x Soil Type [limerock base]	0.1867767	0.20173	0.93	0.3574
((DCP (CBR)-27.1685)) x Condition [below optimum]	0.2625527	0.029659	8.85	<0.0001*

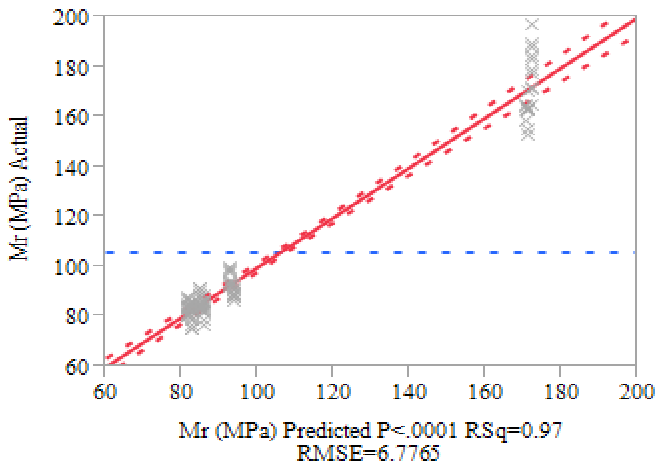
(5) Residual by Predicted Plot



F.9.5 Regression Models for LWD-1

F.9.5.1 Model without Interaction

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.968095
R Square Adj	0.966149
Root Mean Square Error	6.736232
Mean of Response	105.757
Observations (or Sum Wgts)	88

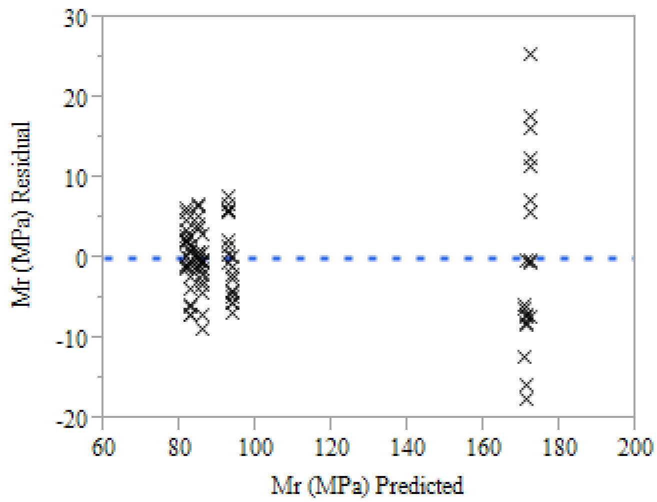
(3) Analysis of Variance

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**
Model	5	112901.94	22580.4	497.6194
Error	82	3720.90	45.4	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

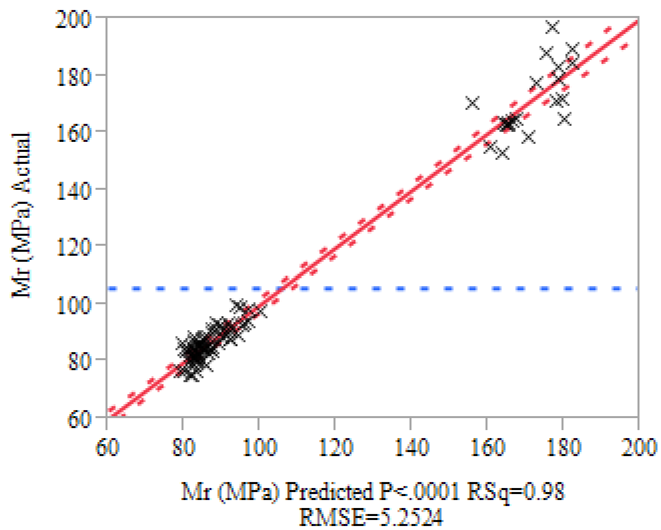
Term	**Estimate**	**Std Error**	**t Ratio**	**Prob>|t|**
Intercept	107.91954	4.365443	24.72	<0.0001*
Soil Type [A-2-4 low fines]	-25.84533	1.275691	-20.26	<0.0001*
Soil Type [A-3]	-22.71088	1.377475	-16.49	<0.0001*
Soil Type [limerock base]	63.407376	2.325313	27.27	<0.0001*
Condition [below optimum]	0.5779429	0.833199	0.69	0.4899
LWD-1 (MPa)	0.0006962	0.066584	0.01	0.9917

(5) Residual by Predicted Plot



F.9.5.2 Interaction Model

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.980839
R Square Adj	0.979419
Root Mean Square Error	5.25244
Mean of Response	105.757
Observations (or Sum Wgts)	88

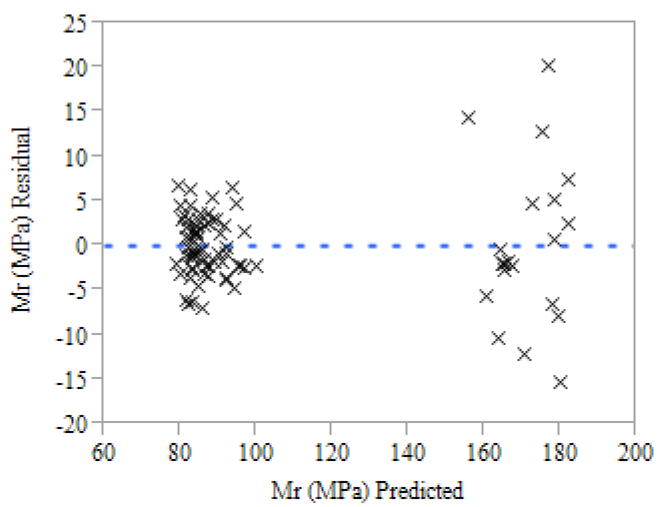
(3) Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Ratio
Model	6	114388.21	19064.7	691.0475
Error	81	2234.64	27.6	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	111.21925	3.433424	32.39	<0.0001*
Soil Type [A-2-4 low fines]	-26.70753	1.001606	-26.66	<0.0001*
Soil Type [A-3]	-22.50331	1.07443	-20.94	<0.0001*
Soil Type [limerock base]	65.541106	1.836272	35.69	<0.0001*
Condition [below optimum]	1.0382924	0.65269	1.59	0.1156
LWD-1 (MPa)	-0.071839	0.05285	-1.36	0.1778
((LWD-1 (MPa)-63.9389)) x Condition [below optimum]	0.2155246	0.029364	7.34	<0.0001*

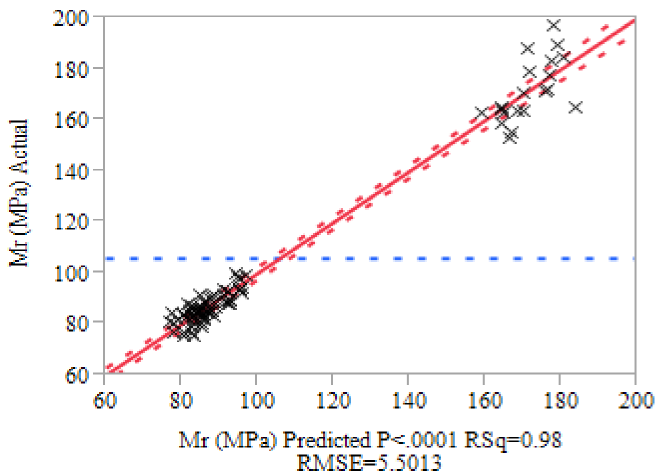
(5) Residual by Predicted Plot



F.9.6 Regression Models for NDG

F.9.6.1 Model without Interaction

(1) Actual by Predicted Plot



(2) Summary of Fit

RSquare	0.978721
RSquare Adj	0.977423
Root Mean Square Error	5.501295
Mean of Response	105.757
Observations (or Sum Wgts)	88

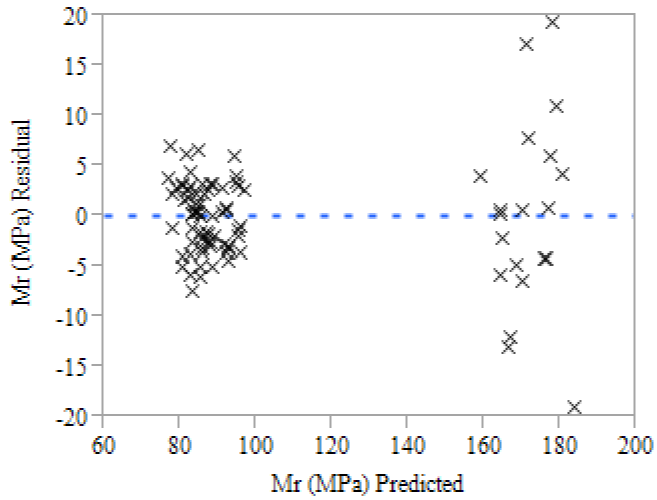
(3) Analysis of Variance

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**
Model	5	114141.18	22828.2	754.2973
Error	82	2481.67	30.3	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

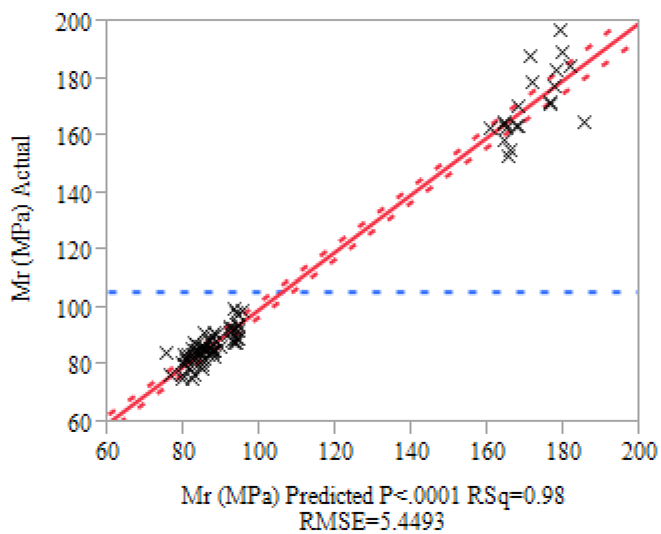
Term	**Estimate**	**Std Error**	**t Ratio**	**Prob>|t|**
Intercept	-80.95546	29.52923	-2.74	0.0075*
Soil Type [A-2-4 low fines]	-20.68253	1.276438	-16.20	<0.0001*
Soil Type [A-3]	-19.95905	1.078491	-18.51	<0.0001*
Soil Type [limerock base]	59.049984	1.253553	47.11	<0.0001*
Condition [below optimum]	0.5377275	0.586481	0.92	0.3619
NDG (pcf)	1.6930169	0.264575	6.40	<0.0001*

(5) Residual by Predicted Plot



F.9.6.2 Interaction Model

(1) Actual by Predicted Plot



(2) Summary of Fit

R Square	0.979376
R Square Adj	0.977848
Root Mean Square Error	5.449284
Mean of Response	105.757
Observations (or Sum Wgts)	88

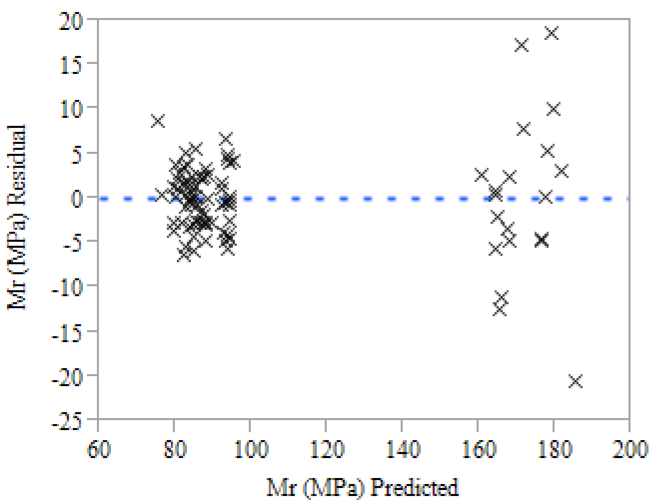
(3) Analysis of Variance

Source	**DF**	**Sum of Squares**	**Mean Square**	**F Ratio**
Model	6	114217.57	19036.3	641.0660
Error	81	2405.27	29.7	**Prob > F**
C. Total	87	116622.84		<0.0001*

(4) Parameter Estimates

Term	**Estimate**	**Std Error**	**t Ratio**	**Prob>|t|**
Intercept	-64.9085	30.91367	-2.10	0.0389*
Soil Type [A-2-4 low fines]	-21.12094	1.293577	-16.33	<0.0001*
Soil Type [A-3]	-19.70151	1.080294	-18.24	<0.0001*
Soil Type [limerock base]	58.395698	1.306988	44.68	<0.0001*
Condition [below optimum]	0.5415263	0.580942	0.93	0.3540
NDG (pcf)	1.5489264	0.277043	5.59	<0.0001*
((NDG (pcf)-111.375)) x Condition [below optimum]	0.3547173	0.221148	1.60	0.1126

(5) Residual by Predicted Plot



F.10 Model Validation

The suggested interaction regression models are validated based on data obtained from field testing performed on selected projects during December 2013 (I-95); January 2014 (US-301); April 2014 (SR-9B); May 2014 (CR-210); and July 2014 (SR-23/Normandy Boulevard and SR-23/New World Avenue). Mean absolute deviation (MAD), root mean square error (RMSE), and mean absolute percent error (MAPE) are calculated for measures of the overall forecast accuracy of each model by use of the following equations:

$$MAD = \frac{\sum_{i=1}^{n} |F_i - y_i|}{n} \qquad RMSE = \sqrt{\frac{\sum_{i=1}^{n} (F_i - y_i)^2}{n}} \qquad MAPE = \frac{1}{n}\sum_{i=1}^{n} \left| \frac{F_i}{y_i} - 1 \right| \quad \text{(Eqs. F.1, F.2, and F.3)}$$

where F_i – Predicted M_R values based on equipment measurements; y_i – Actual laboratory tested M_R values; and n – Sample number.

Generally, the MAD measures the size of the error in units, the RMSE represents the sample standard deviation of the differences between predicted values and observed values, and the MAPE measures the size of the error in percentage terms. The results shown in Table F.4 indicate that the prediction expressions from GeoGauge, PLT, and DCP measurements work well with the in-place testing results on this project.

Table F.4: Summary of interaction regression model validation

	GeoGauge	PLT	DCP	CIST	NDG	LWD-1
MAD	14.739	14.840	14.912	23.345	22.808	27.581
RMSE	19.785	21.205	19.290	29.751	29.352	35.116
MAPE	11.52%	11.77%	12.18%	18.09%	18.88%	20.33%

A summary of the prediction expressions is provided in Table F.5. These expressions are used in the model validation analysis.

Table F.5: Summary of prediction expressions from interaction regression models

CIST	**Below Optimum (M_R in MPa; CIST in CIV)**	**Optimum (M_R in MPa; CIST in CIV)**
A-2-4 low fines	M_R = 78.215 + 0.256 x CIST	M_R = 85.821 – 0.307 x CIST
A-3	M_R = 81.521 + 0.256 x CIST	M_R = 89.127 – 0.307 x CIST
Stabilized subgrade	M_R = 89.320 + 0.256 x CIST	M_R = 96.925 – 0.307 x CIST
Limerock base	M_R = 167.840 + 0.256 x CIST	M_R = 175.445 – 0.307 x CIST
GeoGauge	**Below Optimum (M_R and GeoGauge in MPa)**	**Optimum (M_R and GeoGauge in MPa)**
A-2-4 low fines	M_R = 68.144 + 0.1204 x GeoGauge	M_R = 84.533 + 0.00204 x GeoGauge
A-3	M_R = 70.465 + 0.1204 x GeoGauge	M_R = 86.853 + 0.00204 x GeoGauge
Stabilized subgrade	M_R = 77.843 + 0.1204 x GeoGauge	M_R = 94.231 + 0.00204 x GeoGauge
Limerock base	M_R = 145.341 + 0.1204 x GeoGauge	M_R = 161.730 + 0.00204 x GeoGauge
PLT	**Below Optimum (M_R and PLT in MPa)**	**Optimum (M_R and PLT in MPa)**
A-2-4 low fines	M_R = 66.041 + 0.163 x PLT	M_R = 84.506 – 0.00475 x PLT
A-3	M_R = 65.423 + 0.206 x PLT	M_R = 83.888 + 0.0387 x PLT
Stabilized subgrade	M_R = 108.967 - 0.130 x PLT	M_R = 127.432 – 0.297 x PLT
Limerock base	M_R = 132.304 + 0.224 x PLT	M_R = 150.769 + 0.0562 x PLT
DCP	**Below Optimum (M_R in MPa; DCP in %)**	**Optimum (M_R in MPa; DCP in %)**
A-2-4 low fines	M_R = 91.836 – 0.288 x DCP	M_R = 104.096 – 0.813 x DCP
A-3	M_R = 73.195 + 0.778 x DCP	M_R = 85.455 + 0.253 x DCP
Stabilized subgrade	M_R = 88.100 + 0.132 x DCP	M_R = 100.361 – 0.393 x DCP
Limerock base	M_R = 154.080 + 0.456 x DCP	M_R = 166.340 – 0.069 x DCP
LWD-1	**Below Optimum (M_R and LWD-1 in MPa)**	**Optimum (M_R and LWD-1 in MPa)**
A-2-4 low fines	M_R = 71.771 + 0.1437 x LWD-1	M_R = 97.252 - 0.2873 x LWD-1
A-3	M_R = 75.975 + 0.1437 x LWD-1	M_R = 101.457 - 0.2873 x LWD-1
Stabilized subgrade	M_R = 82.148 + 0.1437 x LWD-1	M_R = 107.630 - 0.2873 x LWD-1
Limerock base	M_R = 164.020 + 0.1437 x LWD-1	M_R = 189.501 - 0.2873 x LWD-1
NDG	**Below Optimum (M_R in MPa; NDG in pcf)**	**Optimum (M_R in MPa; NDG in pcf)**
A-2-4 low fines	M_R = -124.993 + 1.904 x NDG	M_R = -47.066 + 1.194 x NDG
A-3	M_R = -123.573 + 1.904 x NDG	M_R = -45.647 + 1.194 x NDG
Stabilized subgrade	M_R = -121.445 + 1.904 x NDG	M_R = -43.519 + 1.194 x NDG
Limerock base	M_R = -45.476 + 1.904 x NDG	M_R = 32.450 + 1.194 x NDG

Table F.6: Details of interaction regression model validation

In-Place Tests Conducted				Lab Test	Equipment Measurements					Predicted M_R (MPa) Values from Equipment Measurements					
Date	Location	Moisture Condition	Soil Type	Lab M_R (MPa)	LWD-1 (MPa)	GeoGauge (MPa)	DCP (CBR)	Clegg (CIV)	LWD-2 (MPa)	LWD-1	GeoGauge	DCP	LWD-2	PLT	NDG
1/23/2014	US-301	At (near) Optimum	A-2-4	94.999	N/A	75.312	N/A	6.092	64.389		84.687		83.951	84.351	81.289
1/23/2014	US-301	At (near) Optimum	A-2-4	83.865	N/A	74.774	N/A	6.333	68.417		84.686		83.877	84.385	81.528
1/23/2014	US-301	At (near) Optimum	A-2-4	83.875	N/A	74.588	N/A	6.175	59.278		84.685		83.925	84.398	82.364
4/12/2014	SR-9B	At (near) Optimum	A-3	99.516	N/A	99.185	9.593	4.575	N/A		87.055	87.882	87.722	84.735	77.395
7/14/2014	SR-23NWA	At (near) Optimum	A-3	85.519	29.591	60.228	11.885	N/A	71.166	92.956	86.976	88.462		85.542	77.216
7/14/2014	SR-23NWA	At (near) Optimum	A-3	85.519	30.534	62.860	10.997	N/A	67.694	92.685	86.981	88.237		86.255	78.290
7/14/2014	SR-23NWA	At (near) Optimum	A-3	85.519	29.865	69.486	11.719	N/A	77.528	92.877	86.995	88.420		85.965	77.574
5/29/2014	CR-210	At (near) Optimum	Stabilized Subgrade	154.081	125.079	288.686	27.818	43.375	314.112	71.695	162.319	164.421	162.129	160.682	166.118
4/12/2014	SR-9B	Below Optimum	A-3	91.926	N/A	87.828	7.785	5.225	N/A		81.039	79.252	82.859	71.466	76.061
4/12/2014	SR-9B	Below Optimum	A-3	90.507	N/A	99.185	10.503	4.208	N/A		82.407	81.367	82.598		68.636
12/18/2013	I-95	Below Optimum	GAB	139.771	N/A	114.154	35.983	38.050	103.917		159.085	170.488	177.581	160.063	189.478
12/18/2013	I-95	Below Optimum	GAB	145.014	N/A	128.796	38.267	38.392	92.027		160.848	171.530	177.668	164.802	188.145
12/18/2013	I-95	Below Optimum	GAB	135.137	N/A	118.954	45.167	40.750	89.445		159.663	174.676	178.272	152.458	184.337
7/8/2014	SR-23NB	Below Optimum	Limerock Base	152.428	105.764	249.531	N/A	33.217	244.556	179.218	175.385		176.343	166.764	159.870
7/8/2014	SR-23NB	Below Optimum	Limerock Base	152.428	115.206	344.853	N/A	35.175	259.361	180.575	186.861		176.845	170.517	169.628
7/8/2014	SR-23NB	Below Optimum	Limerock Base	152.428	134.530	298.652	N/A	37.000	338.055	183.352	181.299		177.312	183.850	170.961
5/29/2014	CR-210	Below Optimum	Stabilized Subgrade	122.283	80.213	212.133	N/A	45.367	177.083	93.675	170.882		179.454		178.530
5/29/2014	CR-210	Below Optimum	Stabilized Subgrade	122.283	74.619	N/A	N/A	49.617	158.583	92.871			180.542	183.727	178.434
									MAD	**27.581**	**14.739**	**14.912**	**23.345**	**14.840**	**22.808**
									RMSE	**35.116**	**19.785**	**19.290**	**29.751**	**21.205**	**29.352**
									MAPE	**20.33%**	**11.52%**	**12.18%**	**18.09%**	**11.77%**	**18.88%**

Appendix G: Florida Department of Transportation Acceptance Sampling and Testing

G.1 Sampling and Testing for Acceptance

The Florida Department of Transportation (FDOT) uses verified quality control (QC) data generated by the contractor for the acceptance of the compacted soil layers. In this pass/fail system, typically four consecutive passing QC density tests, one per lot, are obtained. A density verification test (VT) test (one per four lots) is then performed. The VT can be performed by FDOT personnel or Consultant Engineering and Inspection (CEI) personnel who are under contract with the FDOT to perform such testing.

This is per the Code of Federal Regulations, Part 637, Subpart B, Quality Assurance Procedures for Construction, "Quality control sampling and testing results may be used as part of the acceptance decision provided that the quality of the material has been validated by the verification testing and sampling."

This is the FDOT approach. Other state DOTs may do otherwise; e.g., results for acceptance are from testing performed by either DOT employees or CEI personnel and not by the contractor. Background pertaining to the structure of the acceptance decision process as conducted by state DOTs is provided for informational purposes only.

G.2 Laboratory and Field Test Procedures

Laboratory and field test procedures developed by the American Association of State Highway and Transportation Officials (AASHTO) and/or the American Society for Testing and Materials (ASTM) as well as the applicable Florida Sampling and Testing Methods (FM) are as follows:

AASHTO T 2 / ASTM D75 - Sampling Aggregates

AASHTO T 248 / ASTM C702 - Reducing Samples of Aggregate to Testing Size

AASHTO T 11 / ASTM C117 - Materials Finer Than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing

AASHTO T 27 / ASTM C136 - Sieve Analysis of Fine and Coarse Aggregates

AASHTO T 255 / ASTM C566 - Total Moisture Content of Aggregate by Drying

AASHTO T 87 / ASTM D421 - Dry Preparation of Disturbed Soil and Soil Aggregate Samples for Test

AASHTO T 89 / ASTM D4318 - Determining the Liquid Limits of Soils

AASHTO T 90 / ASTM D4318 - Determining the Plastic Limit and Plasticity Index of Soils

AASHTO T 88 / ASTM D422 - Particle Size Analysis of Soils

AASHTO T 265 / ASTM D2216 - Laboratory Determination of Moisture Content of Soils

AASHTO T 180 / ASTM D1557- Moisture-Density Relations of Soils Using a 10-lb Rammer and an 18-in. Drop

AASHTO T 99 / ASTM D698 - Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in. Drop

AASHTO M 145 The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes

ASTM D 4643 Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Heating

FM 1-T002 Sampling Coarse and Fine Aggregate

FM 1-T011 Total Amount of Material Finer Than 0.075 mm (No. 200) Sieve in Aggregate

FM 1-T248 Reducing Field Samples of Aggregate to Testing Size

FM 1-T058 (AASHTO R058) Dry Preparation of Disturbed Soil and Soil Aggregate Samples for Tests

FM 1-T088 Particle Size Analysis of Soils

FM 1-T089 Determining the Liquid Limit of Soils

FM 1-T090 Determining the Plastic Limit and Plasticity Index of Soils

FM 1-T180 Moisture Density Relations of Soils Using a 10-lb. Rammer and an 18-in. [FM 5-521]

FM 1-T238 Density of Soils and Bituminous Concrete Mixtures in Place by the Nuclear Method

FM 1-T265 Laboratory Determination of Moisture Content of Soils

FM 5-507 Determination of Moisture Content by Means of a Calcium Carbide Gas Pressure Moisture Tester

FM 5-515 Limerock Bearing Ratio

FM 5-521 Moisture Density Relations of Soils Using a 10-lb. Rammer and an 18-in. Drop

FM 5-525 The Moisture-Density Relations of Soils Using a 5.5lb Rammer and a 12 in. Drop

FM 5-527 Non-repetitive Static Plate Load Test of Soils and Flexible Pavement Components

Additional language may have to be added as supplemental FMs as requirements applicable to Florida conditions become necessary.
http://www.dot.state.fl.us/statematerialsoffice/administration/resources/library/publications/fstm/fstm.shtm

The latest versions of the referenced documents can be obtained by contacting:

American Association of State Highway and Transportation Officials (AASHTO)
444 N. Capitol Street N.W., Suite 249
Washington, D.C. 20001
Telephone: (202) 624-5800
http://www.transportation.org/

American Society for Testing and Materials (ASTM)
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959
Telephone: (610) 832-9585
Fax: (610)832-9555
http://www.astm.org/

Florida Department of Transportation
State Materials Office
5007 Northeast 39th Avenue
Gainesville, Florida 32609-2604
Telephone: (352) 955-6600; Toll-Free: (866) 374-3368 Ext. 2813
Fax: (352) 955-6644
materials@dot.state.fl.us

G.3 American Society of Testing and Materials Procedures

The American Society for Testing and Materials (ASTM) has approved procedures for the use of several of the devices described throughout this report, particularly the Dynamic Cone Penetrometer (DCP) and Lightweight Deflectometers (LWD). Summaries of these methods, the first pertaining to the DCP and the second and third pertaining to LWDs, are provided as follows.

ASTM D6951 – 09 Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications [41]

ASTM E2835 - 11 Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load Test Device [42]

ASTM E2583 – 07 (2011) Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD) [43]

ASTM D6758-08 Standard Test Method for Measuring Stiffness and Apparent Modulus of A Soil and Soil-Aggregate In-Place by Electro-Mechanical Method [44]

ASTM D5874-02 (2007) Standard Test Method for Determination of the Impact Value of a Soil [45]

Appendix H: Other State DOTs' Dynamic Cone Penetrometer and Lightweight Deflectometer Procedures and Specifications

Section numbering in this appendix is as it appears in the original documents.

H.1 Minnesota Department of Transportation Modified Dynamic Cone Deflectometer

5-692.255 mod

MODIFIED DYNAMIC CONE PENETROMETER (DCP)

A. History and Development

The Dynamic Cone Penetrometer was first introduced to the Minnesota Department of Transportation (Mn/DOT) at the Minnesota Road Research Project (Mn/ROAD). Since 1993 the DCP has been used by Mn/DOT as an acceptance tool for the compaction of pavement edge drain trenches. In 1999 the Penetration Index Method for compaction acceptance of base aggregate Classes 5, 6, and 7 was adapted by Mn/DOT, which requires the use of the DCP as the testing device.

B. Description of Device

The Dynamic Cone Penetrometer consists of two 16 mm (5/8-in.) diameter shafts coupled near the midpoint. The lower shaft contains an anvil and a pointed tip, which is driven into unbound materials by dropping a sliding hammer contained on the upper shaft onto the lower anvil. The strength is determined by measuring the penetration of the lower shaft into the unbound materials. This value is recorded in mm (in.) per blow and is known as the Penetration Index (PI).

C. Equipment

The DCP is composed of the following elements (see Fig. 1, 5-692.255 mod).

1. Handle: The handle is located at the top of the device. It is used to hold the DCP shafts plumb and to limit the upward movement of the hammer.
2. Hammer: The 8 kg (17.61 lb.) Hammer is manually raised to the bottom of the handle and then dropped (allowed to free fall) to transfer energy through the lower shaft to the cone tip. The upper shaft guides the hammer.
3. Upper Shaft: The upper shaft is a 16 mm (5/8-in.) diameter steel shaft on which the hammer moves. The length of the upper shaft allows the hammer to drop a distance 575 mm (22.6 in.).

4. Anvil: The anvil serves as the lower stopping mechanism for the hammer. It also serves as a connector between the upper and lower shaft. This allows for disassembly, which reduces the size of the instrument for transport.
5. Lower Shaft: The lower shaft is a 16 mm (5/8-in.) diameter steel shaft, of variable length up to 1 m (39.4 in.) in length, marked in 5-mm (0.2-in.) increments for recording the penetration after each hammer drop.
6. Cone: The cone measures 20 mm (0.787 in.) in diameter. The cone tip has a 60-degree angle (see Fig. 2 5-692.255 mod).

D. Operation Points of Caution

1. Always use caution to avoid pinching fingers between the hammer and the anvil during testing, use the handle to hold shafts plumb. **Do not hold the DCP near the anvil area.**
2. It is important to lift the hammer slowly and drop it cleanly, allowing it to rest on the anvil for at least one second before raising it for another drop. Lifting and dropping too rapidly may affect results because the hammer's full energy may not be allowed to transfer to the lower shaft. This will cause incorrect test results.

E. Test Procedure - Base Aggregate (2211.3C3)

1. Record the gradation % passing values that represent the area to be tested by the DCP, on the attached Modified DCP Procedure 2005-06 form or spreadsheet. If using the form, calculate the Grading Number (GN) by using the formula on the form. If using the spreadsheet, the computer calculates this information (see Fig. 3 5-692.255 mod).
2. Locate a level and undisturbed area (test site) that is representative of the material to be tested.
3. Record the Test #, Date, Station, Offset, and Test Layer Depth on the Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table (see Fig. 3 5-692.255 mod).
4. Place the DCP device on the base aggregate test site. Record the initial reading using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.). (Place this information on the attached Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **Initial Reading** column.)

5. To properly seat the DCP (cone tip), two hammer blows are required. Therefore, carefully raise the sliding weighted hammer until it meets the handle, and then release the hammer under its own weight. Repeat this process one more time for a total of two complete blows.
6. Record the penetration measurement after seating using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.). (Place this information on the attached Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **Reading after seating (2 blows)** column.) (See Fig. 3 5-692.255 mod.)
7. Carefully raise the hammer until it meets the handle, and then release the hammer under its own weight. Repeat this process two more times for a total of three times.
8. Record the final penetration measurement using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.). (Place this information on the attached Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **Reading after test (3 blows)** column.) (See Fig. 3 5-692.255 mod.)
9. After using the DCP, obtain a sample of material and determine the moisture content of the aggregate base by using the pan drying method or a Super Speedy. Record the moisture content on the Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **MC (%)** column. (See Fig. 3 5-692.255 mod.)
10. If using the Modified DCP Procedure 2005-06 form, fill in the **Maximum Allowable SEAT & Maximum Allowable DPI** columns; this information is in the Penetration Requirements table by using the recorded **GN** & **MC**. Next calculate the **SEAT** by using the following formula:

    SEAT = **Reading after seating (2 blows) - Initial Reading**

    Compare the calculated **SEAT** and compare it the **Maximum Allowable SEAT column**, if **SEAT** is larger than the **Maximum Allowable SEAT**, the **SEAT** fails. If the **SEAT** is smaller than the **Maximum Allowable SEAT**, the **SEAT** passes.

11. Next calculate the **DPI** by using the following formula:

$$\textbf{DPI} = \frac{\{\textbf{Reading after test (3 blows) - Reading after seating (2 blows)}\}}{3}$$

Compare the calculated **DPI** and compare it the **Maximum Allowable DPI** column; if the **DPI** is larger than the **Maximum Allowable DPI**, the **Ave. DPI** fails. If the **DPI** is smaller than the **Maximum Allowable DPI**, the **DPI** passes.

Next determine the **Adequate Layer?** by using the following formula:

Adequate Layer? = {Reading after test (3 blows) – Initial Reading} < Test Layer Depth

If the **{Reading after test (3 blows) – Initial Reading}** is larger than the **Test Layer Depth,** the answer is No. If the **{Reading after test (3 blows) – Initial Reading}** is less than the **Test Layer Depth,** the answer is Yes.

To determine whether the **Test Pass or Fail**, check the **Seat Pass or Fail**, **DPI Pass or Fail**, and **Adequate Layer?** columns. If any of the three columns has Fail or No, the **Test** Fails. If all three columns have Pass or Yes, the **Test** Passes.

If using the Modified DCP Procedure 2005-06 spreadsheet, all the above information is calculated by the computer and to determine if the test passes or fails look in the Test Pass or Fail column for the answer.

12. For test purposes, the approximate test layer in compacted thickness is located in the Penetration Index chart on Fig. 3, 5-692.255 mod.

F. Test Procedure - Granular Subgrade Material (2105.3F3)

1. Record the gradation % passing values that represent the area to be tested by the DCP, on the attached Modified DCP Procedure 2005-06 form or spreadsheet. If using the form, calculate the Grading Number (GN) by using the formula on the form. If using the spreadsheet, the computer calculates this information (see Fig. 3 5-692.255 mod).

2. Locate a level and undisturbed area (test site) that is representative of the material to be tested.
3. Record the Test #, Date, Station, Offset, and Test Layer Depth on the Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table (see Fig. 3 5-692.255 mod).
4. Place the DCP device on the granular material test site. Record the initial reading using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.). (Place this information on the attached Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **Initial Reading** column.) (See Fig. 3 5-692.255 mod).
5. To properly seat the DCP (cone tip), two hammer blows are required. Therefore, carefully raise the sliding weighted hammer until it meets the handle, and then release the hammer under its own weight. Repeat this process one more time for a total of two complete blows.
6. Record the penetration measurement after seating using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.). (Place this information on the attached Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **Reading after seating (2 blows)** column.) (See Fig. 3 5-692.255 mod.)
7. Carefully raise the hammer until it meets the handle, and then release the hammer under its own weight. Repeat this process two more times for a total of three times.
8. Record the final penetration measurement using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.). (Place this information on the attached Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **Reading after test (3 blows)** column.) (See Fig. 3 5-692.255 mod.)
9. After using the DCP, obtain a sample of material and determine the moisture content of the granular material by using the pan drying method or a Super Speedy. Record the moisture content on the Modified DCP Procedure 2005-06 form or spreadsheet, in the DCP Data table, under **MC (%)** column. (See Fig. 3 5-692.255 mod.)
10. If using the Modified DCP Procedure 2005-06 form, fill in the **Maximum Allowable SEAT** & **Maximum Allowable DPI** columns; this information is in the Penetration

Requirements table by using the recorded **GN** & **MC**. Next calculate the **SEAT** by using the following formula:

SEAT = **Reading after seating (2 blows) - Initial Reading**

Compare the calculated **SEAT** and compare it the **Maximum Allowable SEAT column**, if **SEAT** is larger than the **Maximum Allowable SEAT**, the **SEAT** fails. If the **SEAT** is smaller than the **Maximum Allowable SEAT**, the **SEAT** passes.

Next calculate the **DPI** by using the following formula:

$$\mathbf{DPI} = \frac{\{\textbf{Reading after test (3 blows) - Reading after seating (2 blows)}\}}{3}$$

Compare the calculated **DPI** and compare it the **Maximum Allowable DPI** column; if the **DPI** is larger than the **Maximum Allowable DPI**, the **Ave. DPI** fails. If the **DPI** is smaller than the **Maximum Allowable DPI**, the **DPI** passes.

11. Next determine the **Adequate Layer?** by using the following formula:

Adequate Layer? = {Reading after test (3 blows) – Initial Reading} < Test Layer Depth

If the **{Reading after test (3 blows) – Initial Reading}** is larger than the **Test Layer Depth,** the answer is No. If the **{Reading after test (3 blows) – Initial Reading}** is less than the **Test Layer Depth,** the answer is Yes.

To determine whether the **Test Pass or Fail**, check the **Seat Pass or Fail**, **DPI Pass or Fail**, and **Adequate Layer?** columns. If any of the three columns has Fail or No, the **Test** Fails. If all three columns have Pass or Yes, the **Test** Passes.

If using the Modified DCP Procedure 2005-06 spreadsheet, all the above information is calculated by the computer; and to determine if the test passes or fails, look in the **Test Pass or Fail** column for the answer (see Fig. 3, 5-692.255 mod).

12. For test purposes, a layer will be considered 300 mm (1 ft.) in compacted thickness.

G. Test Procedure - Edge Drain Trench Filter Aggregate (2502)

1. After the compaction of the first 15 m (50 ft.) of filter aggregate within the edge drain trench has been completed, determine the location of three test sites that are 3 m (10 ft.) apart within that first 15 m (50 ft.).

2. Calculate the number of hammer drops (blows) necessary to ‘properly test the trench filter aggregate but not damage the edge drain pipe by subtracting 150 mm (6 in.) from the depth of the trench to be tested and dividing that total by 75 for metric measurements or 3 for English measurements. If necessary, round this number <u>down</u> to the next whole number (see Fig. 4, 5-692.225 mod).

   Example: If the trench depth equals 650 mm (26 in.).

   Then 650 mm (26 in.) minus 150 mm (6 inches) equals 500 mm (20 in.).
   Then 500 mm (20 in.) divided by 75 (for Metric) or 3 (for English) equals 6.7 or 6.

3. Place the DCP on test site #1 and seat the coned tip of the device by slightly tapping the lower anvil with the hammer until the coned tip is just out of sight.
4. After seating, record the penetration measurement using the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.). [Use form TP-2170 –02(rev 11/05)] (See Fig. 5 5-692.255 mod.)
5. Carefully raise the hammer until it meets the handle, and then release the hammer under its own weight. Repeat this process until the total number of hammer drops equals the required number of blows as calculated in step 2. Also, beware and avoid the chance of penetrating the edge drain pipe at the bottom of the trench when the compaction of the trench is less than passing.
6. Record the final penetration measurement from the graduated rule on the DCP. The measurement is taken to the nearest 2.5 mm (0.1 in.).
7. Subtract the measurement in step 4 from the measurement in step 6 and then divide the difference of the measurements by the number of blows required for testing.

The result is the penetration index. If necessary, follow the formula on the test form to convert from mm to in.

8. Use the same procedures as outlined above for testing sites #2 and #3.
9. Add the three penetration index results from test site #1, #2, and #3 and divide that total by 3 in order to calculate the average of all three tests. Round off the average of the tests to the nearest 1 mm (0.1 in.) (see Grading and Base Manual 5-692.805).

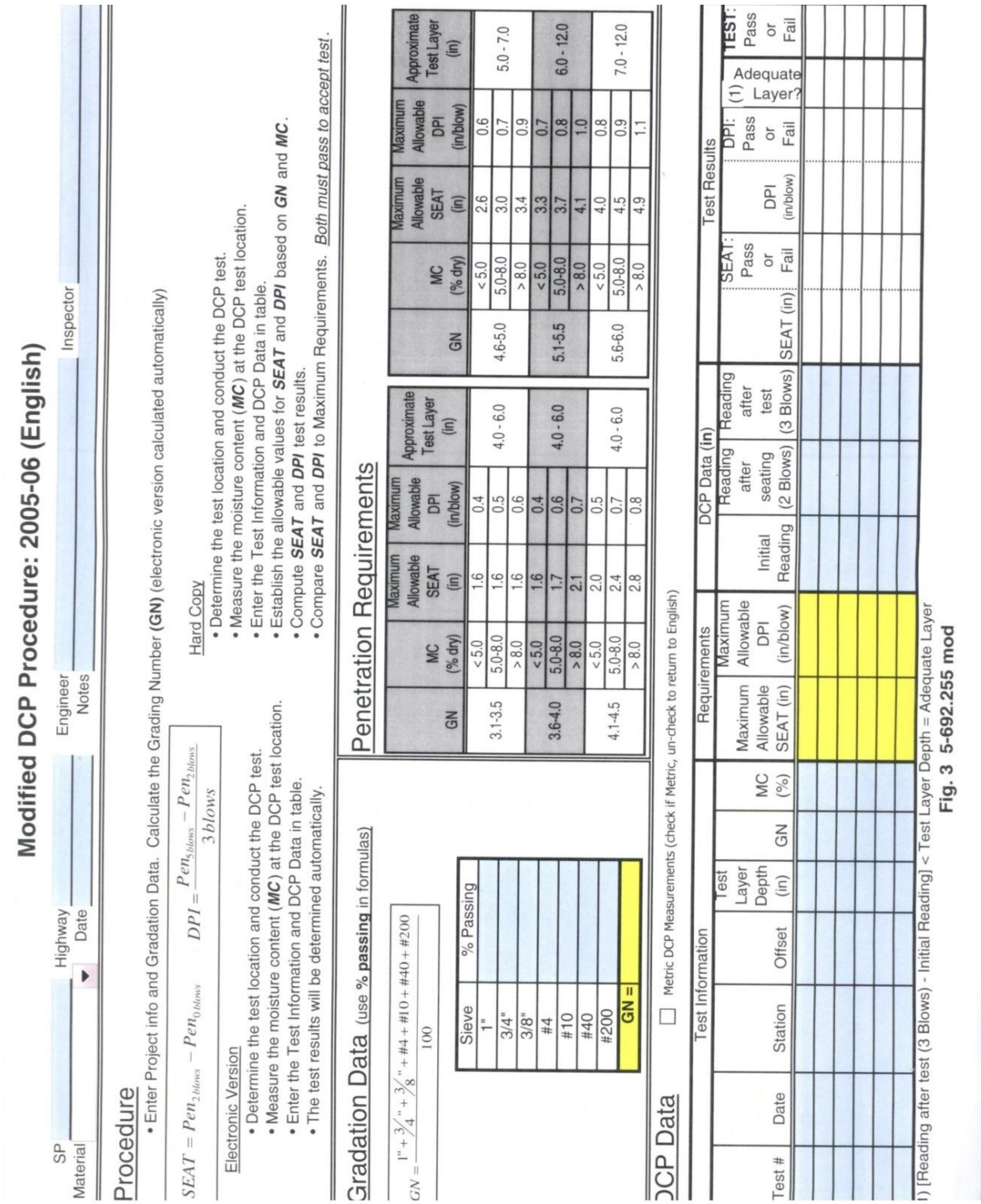
H. Maintenance and Handling

Because the Dynamic Cone Penetrometer is driven into the ground, sometimes into very hard soil layers, regular maintenance and care are required. To ensure that the DCP operates properly, the following guidelines must be followed.

1. Monitor the condition of the connections to the anvil and handle. When the connections use bolts, pins, or set screws, extra bolts, pins, or set screws should be kept in the DCP carrying cases because they frequently become stripped or broken and may need to be replaced during testing.
2. Keep the upper shaft clean. Lubricate very lightly with oil if binding develops. Frequently wipe both shafts clean with a soft cloth during use.
3. Monitor the DCP for excessive wear on any of the components and make repairs as needed. Because the DCP is a standardized testing device, its overall weight and dimensions must not change from specifications.
4. The cone tip should be replaced when the diameter of its widest section is reduced by more than 10 percent (2 mm [0.08 in.]) or rocks gouge the cone's surface. Inspect the cone tip before and after each test. Nevertheless, the cone tip should be replaced at least once a year.
5. Never extract the DCP from the test hole by forcefully striking the hammer against the handle. Striking the handle causes accelerated wear and may lead to broken welds and connections. At least once a year, all welds on the DCP should be critically inspected for hairline or larger cracks.
6. Do not lay the device on the ground when not in use. The DCP should be kept in its carrying case to avoid bending the shafts. Straightness of the shafts is extremely important. The hammer cannot free fall if the shafts are bent. The straightness of the

shafts should be critically measured and reviewed each year prior to the start of construction season.

Figure H.1 is not included in the Mn/DOT procedure but is provided here to illustrate the test procedure example given previously.



Modified DCP Procedure: 2005-06 (English)

SP | Material | Highway | Date | Engineer | Notes | Inspector

Procedure

- Enter Project info and Gradation Data. Calculate the Grading Number **(GN)** (electronic version calculated automatically)

$$SEAT = Pen_{2\,blows} - Pen_{0\,blows} \qquad DPI = \frac{Pen_{5\,blows} - Pen_{2\,blows}}{3\,blows}$$

Electronic Version

- Determine the test location and conduct the DCP test.
- Measure the moisture content (***MC***) at the DCP test location.
- Enter the Test Information and DCP Data in table.
- The test results will be determined automatically.

Hard Copy

- Determine the test location and conduct the DCP test.
- Measure the moisture content (***MC***) at the DCP test location.
- Enter the Test Information and DCP Data in table.
- Establish the allowable values for ***SEAT*** and ***DPI*** based on ***GN*** and ***MC***.
- Compute ***SEAT*** and ***DPI*** test results.
- Compare ***SEAT*** and ***DPI*** to Maximum Requirements. Both must pass to accept test.

Gradation Data (use **% passing** in formulas)

$$GN = \frac{1'' + 3/4'' + 3/8'' + \#4 + \#10 + \#40 + \#200}{100}$$

Sieve	% Passing
1"	
3/4"	
3/8"	
#4	
#10	
#40	
#200	
GN =	

Penetration Requirements

GN	MC (% dry)	Maximum Allowable SEAT (in)	Maximum Allowable DPI (in/blow)	Approximate Test Layer (in)
3.1-3.5	< 5.0	1.6	0.4	4.0 - 6.0
	5.0-8.0	1.6	0.5	
	> 8.0	1.6	0.6	
3.6-4.0	< 5.0	1.6	0.4	4.0 - 6.0
	5.0-8.0	1.7	0.6	
	> 8.0	2.1	0.7	
4.1-4.5	< 5.0	2.0	0.5	4.0 - 6.0
	5.0-8.0	2.4	0.7	
	> 8.0	2.8	0.8	
4.6-5.0	< 5.0	2.6	0.6	5.0 - 7.0
	5.0-8.0	3.0	0.7	
	> 8.0	3.4	0.9	
5.1-5.5	< 5.0	3.3	0.7	6.0 - 12.0
	5.0-8.0	3.7	0.8	
	> 8.0	4.1	1.0	
5.6-6.0	< 5.0	4.0	0.8	7.0 - 12.0
	5.0-8.0	4.5	0.9	
	> 8.0	4.9	1.1	

☐ Metric DCP Measurements (check if Metric, un-check to return to English)

DCP Data

Test Information							Requirements		DCP Data (in)			Test Results					
Test #	Date	Station	Offset	Test Layer Depth (in)	GN	MC (%)	Maximum Allowable SEAT (in)	Maximum Allowable DPI (in/blow)	Initial Reading	Reading after seating (2 Blows)	Reading after test (3 Blows)	SEAT (in)	SEAT: Pass or Fail	DPI (in/blow)	DPI: Pass or Fail	(1) Adequate Layer?	TEST: Pass or Fail

(1) [Reading after test (3 Blows) - Initial Reading] < Test Layer Depth = Adequate Layer

Fig. 3 5-692.255 mod

Figure H.1: Mn/DOT Modified DCP Procedure data sheet

Minnesota Department of Transportation DCP Pilot Project

Numbering contained in the following section is as it appears in the original document.

The following information is for deleting the Penetration Index Method and replacing it with the Modified Penetration Index Method in the Special Provisions.

2105 Excavation and Embankment

1.) Add the following section in the Standard Specifications: <u>2105.3F3 Modified Penetration Index Method</u>

F3 Modified Penetration Index Method

The full thickness of each layer of Select and Granular Borrow subgrade materials shall be compacted to achieve a penetration index value as described in the modified dynamic cone penetrometer (DCP) test procedure, as determined by an Mn/DOT standard dynamic cone penetrometer (DCP) device. For test purposes, a layer will be considered to be 1 ft. (300 mm) in compacted thickness. Two DCP tests shall be conducted at selected sites within each 4,000 cu. yd. (3,000 m^3) (CV) of constructed subgrade. If either of the tests fails to meet the specified requirements, the material represented by the test shall be recompacted and retested for penetration index compliance.

2211 Aggregate Base

1.) Delete the following section in the Standard Specifications: <u>2211.3C3 Penetration Index Method</u>

2.) Add the following section in the Standard Specifications: <u>2211.3C3 Modified Penetration Index Method</u>

C3 Modified Penetration Index Method

The full thickness of each layer of Class 3, 5, 6, or 7 shall be compacted to achieve a penetration index value as described in the modified dynamic cone penetrometer (DCP) test procedure, as determined by an Mn/DOT standard dynamic cone penetrometer (DCP) device. For test purposes, a test layer will be described in the modified dynamic cone penetrometer (DCP) test procedure. Two DCP tests shall be conducted at selected sites within each 800 m^3 (1000 cu. yd.) (CV) of constructed base course. If either of the tests fails to meet the specified requirements, the material represented by the test shall be recompacted and retested for penetration index compliance.

Water shall be applied to the base material during the mixing and spreading operations so that at the time of compaction the moisture content is no less than five percent of dry weight.

H.2 Minnesota Department of Transportation Dynamic Cone Penetrometer Specification

2502.3 CONSTRUCTION REQUIREMENTS

G Pavement Edge Drain Type

Construct 3 in [75 mm] diameter edge drains. Use pavement edge drains to collect and discharge water infiltrating into the pavement system from rain or snow melt, and spring-thaw seepage.

Compact the filter aggregate with equipment capable of achieving a minimum of 95 percent of maximum density for the full depth of the trench. Before beginning routine trenching and backfilling, construct a test trench at least 50 ft. [15.2 m] long that has the same requirements as the production work. The Engineer will measure adequacy of compaction in the test trench with a Department-supplied Dynamic Cone Penetrometer (DCP).

The Department defines successful compaction as penetration resistances no greater than 3 in [75 mm] per DCP hammer blow. The Engineer will base successful compaction on the average of three DCP readings for similar depths in three tests taken 10 ft. [3 m] apart. Unless otherwise directed by the Engineer, begin penetration readings from the point where the DCP equipment stabilizes after setup in the trench.

Do not compact greater than 24 in [600 mm] of filter aggregate in any one layer. The Contractor may use smaller lifts or make more than one pass of the compactor to achieve a minimum of 95 percent of maximum density throughout the compacted depth, unless otherwise directed by the Engineer. Do not run the compactor at a rate greater than 60 ft. per min [18.3 m per min] unless otherwise approved by the Engineer based on DCP test results. Stop the trenching operation if the compaction method or source of trench backfill changes, or compaction effort yields insufficient density, until the Engineer performs additional DCP testing and approves corrections. After compaction and leveling, extend the filter aggregate up onto the adjacent pavement as shown on the plans.

H.3 Indiana Department of Transportation Dynamic Cone Penetrometer Procedure

Indiana Department of Transportation
Field Determination of Strength Using Dynamic Cone Penetrometer
ITM No. 509-14P

1.0 SCOPE.

1.1 This method covers the procedure for determination of the strength of materials using a Dynamic Cone Penetrometer (DCP).

1.2 The DCP may be used for clay, silty, or sandy soils; granular soils; chemically modified soils; or as directed by the Department. Granular soils with aggregate that is 100% passing the 3/4 in. sieve and structural backfill sizes 1 in., 1/2 in., No. 4 and No. 30 shall be tested with the DCP.

1.3 This ITM may involve hazardous materials, operations, and equipment and may not address all of the safety problems associated with the use of the test method. The user of the ITM is responsible for establishing appropriate safety and health practices and determining the applicability of regulatory limitations prior to use.

2.0 REFERENCES.

2.1 AASHTO Standards.

T 99 Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop

2.2 ASTM Standards.

D 6951 Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

2.3 ITM Standards.

ITM 512 Field Determination of Maximum Dry Density and Optimum Moisture Content of Soil

3.0 TERMINOLOGY. Definitions for terms and abbreviations shall be in accordance with the Department’s Standard Specifications, Section 101 and the following:

3.1 Soil. Cohesive material with more than 35% passing the No. 200 sieve. Clay, silty, and sandy soils are determined by the maximum dry density in accordance with AASHTO T 99 or ITM 512 using the Department Family of Curves and are defined as follows:

3.1.1 Clay Soil. Soil with a maximum dry density of 112 lb/ft3 or less

3.1.2 Silty Soil. Soil with a maximum dry density greater than 112 lb/ft3 and less than or equal to120 lb/ft3

3.1.3 Sandy Soil. Soil with a maximum dry density greater than 120 lb/ft3

3.2 Granular Soil. Soil that is non-cohesive with 35% or less material passing the No. 200 sieve.

3.3 Chemically Modified Soil. Soil that has been modified with Portland cement, fly ash, lime, cement by-product, or a combination of these materials.

4.0 SIGNIFICANCE AND USE.

4.1 This test method is used to assess in situ strength of undisturbed soil, compacted soil, and granular material. The penetration rate of the DCP may be used to estimate and identify strata thickness, shear strength of strata, and other material characteristics.

5.0 APPARATUS.

5.1 Dynamic Cone Penetrometer (Appendix A), with a 17.6 lbm steel drop hammer located between the handle and coupler assembly on a 0.625 in. diameter steel rod. The steel rod is required to be a minimum of 24 in. in length and be threaded on both ends to allow the attachment of a cone on one end and an anvil on the other end. The distance from the bottom of the hammer to the coupler assembly is 22.6 in. On the bottom of the rod is a replaceable hard sharp conical tip with an included angle of 60° and a diameter at the base of 0.79 in. (Note 1). The rod shall have 0.5 in. graduations. A ruler may be used to indicate the required penetration of the DCP on the steel rod.

Note 1 – A disposable cone may be used in chemically modified or other hard soils to avoid damage to the equipment, which may be caused by driving the hammer upward to extract the cone from the soil.

6.0 PROCEDURE

6.1 Check the DCP components for deficiencies, replace any damaged parts, and assemble the equipment as shown in Appendix A. All joints are required to be securely tightened.

6.2 Verify that the graduated rod (Note 2) and cone, or cone adaptor when using a disposable cone, are free of materials from the previous use.

Note 2 – The length of the graduated rod ranges from 24 in. to 40 in. The Department may select the length used based on a safe operating condition.

6.3 Scrape any loose material away from the site to be tested

6.4 Hold the DCP in a vertical or plumb position by the handle, and seat the cone such that the cone base is flush with the surface of the material to be tested. The DCP is held by the anvil. The initial reading is taken from the cone base. Do not record the number of blows required to seat the cone to the cone base.

6.5 Raise the hammer to the handle at the top of the upper rod without impacting the handle. Let the hammer drop freely on the anvil to drive the cone into the material (Note 3).

Note 3 – Large aggregates or rock strata may stop the penetration of the DCP. If after several blows relatively little penetration is achieved, the DCP is required to be moved 1 ft. away from the initial site and a new test conducted.

6.6 Count the number of blows for the required penetration of the DCP into the material. The penetration of the DCP depends on the material variability and resistance of the material. If the DCP does not penetrate the required depth of material after 25 blows, the test is discontinued and the material is considered to be in compliance with the strength requirements.

6.7 Remove the DCP rod from the material by forcefully raising the hammer to strike the bottom until the DCP is free from the material (Note 4).

Note 4 – Care is required to be taken to keep the DCP vertical or plumb during the extraction process to prevent bending of the rod. Do not rock the rod back and forth or in a circle to free the device from the material. The extraction of the rod in chemically modified soil is easier with a disposable cone.

Note 5 – When transporting the DCP, do not allow a horizontally laying rod to support the weight of the hammer. The DCP should be disassembled, placed in the case, or placed horizontally with the hammer resting next to the anvil. Both the upper and lower rod will bend if they are required to support the weight of the hammer.

7.0 ACCEPTANCE CRITERIA.

7.1 Clay Soil. For clay soil, the strength of the soil is measured after completion of the compaction of each 6 in. of the soil. The number of blows of the DCP is measured for a penetration of 6 in. into the soil.

7.2 Silty Soil. For silty soil, the strength of the soil is measured after completion of compaction for each 12 in. of the soil. The number of blows of the DCP is measured for a penetration of 12 in. into the soil.

7.3 Sandy Soil. For sandy soil, the strength of the soil is measured after completion of compaction for each 12 in. of the soil. The number of blows of the DCP is measured for a penetration of 12 in. into the soil.

7.4 Chemically Modified Soils. For chemically modified soils, the strength of the soil is measured after completion of the compaction of each type of chemical modification.

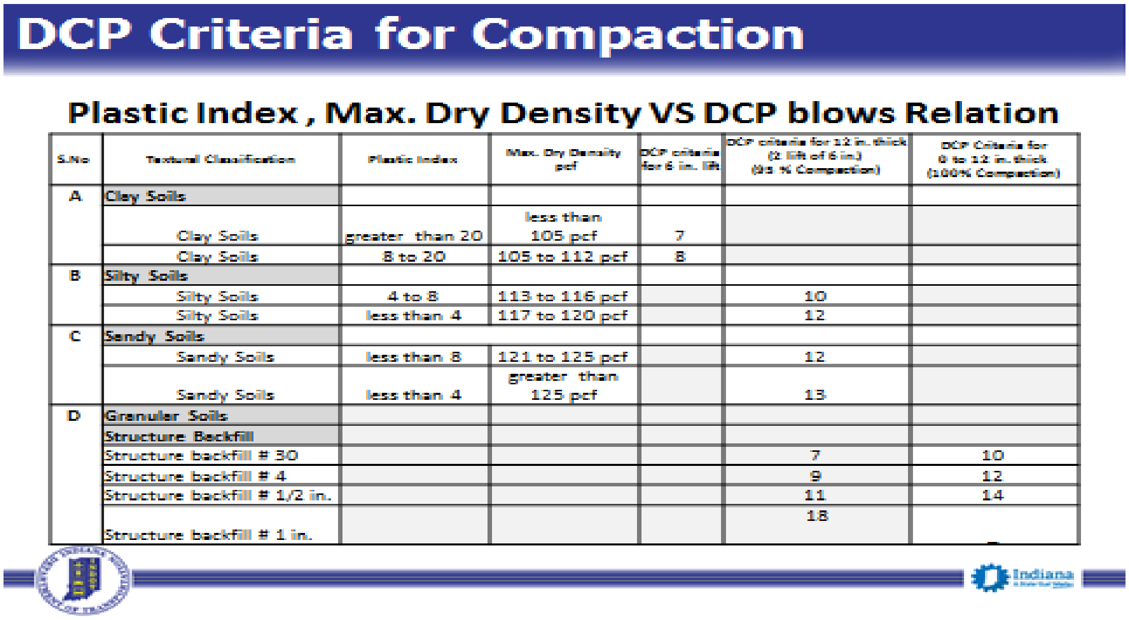
7.4.1 For an 8 in. lift, the number of blows of the DCP is measured for a penetration of 8 in. into the soil.

7.4.2 For a 14 in. lift, the number of blows of the DCP is measured initially for a penetration of the top 6 in. of the lift. A separate number of blows of the DCP is measured for a penetration into the bottom 8 in. of the lift.

7.5 Granular Materials. For granular materials, the strength of the material is measured after completion of compaction for each 12 in. of the material. The number of blows of the DCP is measured for a penetration of 12 in. into the granular material.

8.0 REPORT. Report the number of blows to obtain the required penetration of the DCP.

Figures H.2 and H.3 are not included in INDOT procedure ITM No. 509-14P but are provided here to illustrate the test procedure example given.



DCP Criteria for Compaction

Plastic Index , Max. Dry Density VS DCP blows Relation

S.No	Textural Classification	Plastic Index	Max. Dry Density pcf	DCP criteria for 6 in. lift	DCP criteria for 12 in. thick (2 lift of 6 in.) (95 % Compaction)	DCP Criteria for 0 to 12 in. thick (100% Compaction)
A	Clay Soils					
	Clay Soils	greater than 20	less than 105 pcf	7		
	Clay Soils	8 to 20	105 to 112 pcf	8		
B	Silty Soils					
	Silty Soils	4 to 8	113 to 116 pcf		10	
	Silty Soils	less than 4	117 to 120 pcf		12	
C	Sandy Soils					
	Sandy Soils	less than 8	121 to 125 pcf		12	
	Sandy Soils	less than 4	greater than 125 pcf		13	
D	Granular Soils					
	Structure Backfill					
	Structure backfill # 30				7	10
	Structure backfill # 4				9	12
	Structure backfill # 1/2 in.				11	14
	Structure backfill # 1 in.				18	–

Figure H.2: INDOT DCP Criteria for Compaction

The number of DCP blows should be compared with the minimum required DCP blow count based on the optimum moisture content of the tested material.

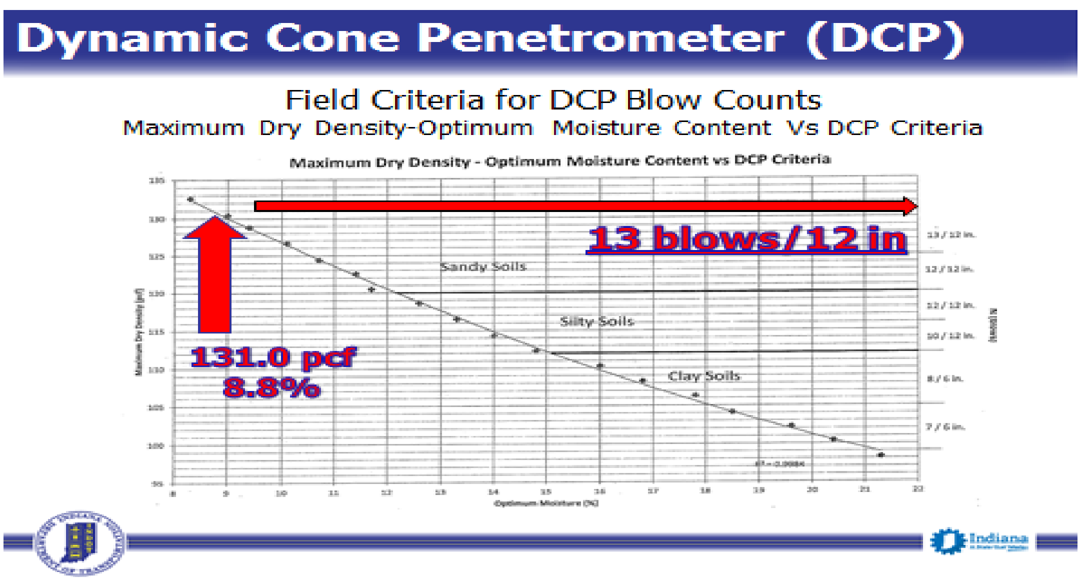


Figure H.3: INDOT Field Criteria for DCP Blow Counts

H.4 Indiana Department of Transportation Dynamic Cone Penetrometer Specification

203.24.1 Compaction Acceptance with DCPT

The compaction will be determined by dynamic cone penetrometer testing (DCPT) in accordance with ASTM D 6951 using a 17.6 lb (8 kg) hammer. The moisture content shall be controlled within -3 and +2 percentage points of the optimum moisture content determined in accordance with AASHTO T 99.

The Department will establish the criteria for DCPT acceptance of compaction by performing the sieve analysis, liquid limit, plastic limit, and optimum moisture and maximum density testing in accordance with AASHTO T 88, T 89, T 90, and T 99, respectively, on representative samples of the soils to be used. The required blow counts will be determined based on the laboratory tests for each soil type.

Test sections shall be constructed in the presence of a Geotechnical representative with the available equipment of the Contractor to determine the roller type, pattern, and the number of passes for verification of the blow counts for a 6 in. (150 mm) lift. The Office of Geotechnical Engineering will be contacted prior to construction of the test sections to determine the number of test sections required for the evaluation of the DCPT process. The embankment shall be constructed in two 6 in. (150 mm) successive lifts placed in accordance with 203.23. The Engineer will select an area approximately 100 ft. (30 m) long and 20 ft. (6 m) wide within each lift for a test section. The test section in the second lift will be approximately in the same location as the test section in the first lift. The soil immediately below the test section in the first lift shall be proof rolled in accordance with 203.26 prior to construction of the lift.

Moisture tests will be performed in accordance with ITM 506 at 2 random locations and DCPT will be performed at 4 random locations in each lift. The locations will be determined in accordance with ITM 802. The moisture content shall be controlled within -3 and +2 percentage points of the optimum moisture content. Blow counts greater than 10 or less than 4 will be discarded and a new random test location will be selected in the test section in that lift. If all of the test section blow counts are outside of the range of 10 to 4, the Office of Geotechnical Engineering will be contacted for determination of the target blow counts.

H.5 Minnesota Department of Transportation Lightweight Deflectometer Procedure

S-xx **(2105, 2106, 2211, 2215) LIGHT WEIGHT DEFLECTOMETER (LWD) DEFLECTION METHOD**
03/28/12

All forms and the Grading and Base Manual are available on the Grading and Base Website. Unless otherwise designated all test procedures are in the Grading and Base Manual.

The following is added MnDOT 2105.1, 2106.1, 2211.1 and 2215.1 "DESCRIPTION":

S-xx.1 Definitions

(A) "*Road Core*" is the area below the grading grade to the bottom of excavation and between the following:

(1) Embankment height ≤ 30 ft [10 m], from the grading grade point of intersections (P.I.s) with a 1:1 (V:H) slope, and

(2) Embankment height > 30 ft [10 m]: from the grading grade point of intersections (P.I.s) with a 1:1½ (V:H) slope.

(B) "*Lift*" is a unit of material within a *Layer* that is placed for compaction.

(C) "*Layer*" is the total embankment thickness for each material type and may be comprised of a single or multiple *Lifts.*

(D) "Deflection Test Measurement" is the average deflection measured from the fourth, fifth and sixth drops in a testing sequence.

(E) "Seating Drops" are the first, second and third drops in the testing sequence and are not used for acceptance.

(F) "Control Strip" is an area, with uniform material properties, that is constructed for use in determining the maximum LWD deflection at optimum compaction.

(G) "Compaction Curve" is defined as the relationship between the average of the Deflection Test Measurements and the roller passes.

(H) "LWD Target Value (LWD-TV)" is the maximum deflection allowed by either **Option 1: Control Strip Method** or **Option 2: Comparison Testing Method** for a given soil type, source and lift.

The following is added MnDOT 2105.3, 2106.3, 2211.3 and 2215.3 "CONSTRUCTION REQUIREMENTS":

S-xx.2 Construction Requirements

(A) Construction of Control Strip for Option 1: Control Strip Method

(1) Construct a Control Strip, within the Road Core, meeting the requirements of Table 1 and in accordance with 2105, 2106, 2211 or 2215.

Table 1					
Required Control Strip Dimensions (Note 1)					
Specification	**Material / Location**	**Length**	**Width (ft)**	**Thickness**	**Number of Lifts**
2211, 2215	Base	≥ 300 ft (100 m)	Layer	Layer	1
2105, 2106	Roadbed Embankment Soil (Excavation & Borrow)	≥ 300 ft (100 m)	Excavated Embankment Width	Planned layer thickness, but not exceeding a maximum thickness of 4 ft (1.2 m)	1 Every 1 ft
2105, 2106	Misc., Trench, Culvert or other Tapered Constr. Embankment Soil Granular Bridge Approach Treatments & Other Embankment Adj. to Structures (Excavation & Borrow)	≥ 10 ft (3 m)	Excavated Embankment Width	Planned layer thickness, but not exceeding a maximum thickness of 4 ft (1.2 m)	1 Every 2 ft

NOTE 1: Or as determined by the Engineer.

(2) Compact each lift until the decrease in the average of the Deflection Test Measurements is less than 5 percent of the measurement for the previous pass.

(3) A new Control Strip is constructed when:

(a) There is an observable variation in material properties and/or

(b) The moisture content of the material changes by ± 2 percent of the moisture content of the associated Control Strip.

(4) Collect and save a material sample from each Control Strip for comparison to the material being compacted during production.

(5) Control Strip construction is incidental to the compaction requirements.

(B) Compacting

During production, compact the entire lift to achieve a Deflection Test Measurement that is:

(1) ≤ 1.10*LWD-TV (Option 1: Control Strip Method)

(2) ≤ LWD-TV (Option 2: Comparison Test Method)

The following is added MnDOT 2105.3, 2106.3, 2211.3 and 2215.3 “CONSTRUCTION REQUIREMENTS”:

S-xx.2 Agency Verification Testing (VT)

(A) Option 1: Control Strip Method

(1) The Engineer will perform LWD Deflection Testing at the rates per Table 2 during construction of the control strip.

Table 2 LWD Testing Rates during Control Strip Construction			
Specification	**Material**	**Form Number**	**Minimum Required Agency (Field Test Rate)**
2211, 2215	Base	G&B-601	4 Tests / Roller Pass / Lift
2105, 2106	Roadbed Embankment Soil (Excavation & Borrow)		
2105, 2106	Misc., Trench, Culvert or other Tapered Constr. Granular Bridge Approach Treatments & Other Embankment Adj. to Structures (Excavation & Borrow)	G&B-602	2 Tests / Roller Pass / Lift

(2) Determine the LWD-TV for each lift of the control strip as follows:

(a) The Engineer will create a compaction curve by plotting the Deflection Test Measurements versus the pass count using form G&B-603.

(b) The LWD-TV for each lift is reflective of the breaking point of the curve (the Deflection Test Measurement value at the minimum deflection of the curve). Use the average of the Deflection Test Measurement values for this pass count as the LWD-TV for the given lift.

(c) For fill thicknesses:
$\leq$ 4 ft (1.2 m) use the LWD-TV determined for the respective lift and
$>$ 4 ft (1.2 m) use the LWD-TV determined on the 4 ft (1.2 m) lift for all subsequent lifts.

(3) The Engineer will perform VT at the rates as specified in Table 3.

Table 3
LWD Testing Rates
(when Target Values Determined with Option 1: Control Strip Method)

Specification	Material	Form Number	Minimum Required Agency (Field Test Rate)
2211	Base		1 LWD test / 500 yd^3 (CV) 1 LWD test / 400 m^3 (CV)
2215	Base		1 LWD test / 3,000 yd^2 1 LWD test / 2,000 m^2
	Roadbed Embankment Soil (Excavation & Borrow)		1 LWD test / 2,000yd^3 (CV) 1 LWD test / 1,500 m^3
2105, 2106	Misc., Trench, Culvert or other Tapered Constr. Embankment Soil (Excavation & Borrow)	G&B-604	1 per 2-ft Thickness / 250 ft 1 per 600-mm Thickness/ 75 m
	Granular Bridge Approach Treatments & Other Embankment Adj. to Structures **(See Note 1)**		1 per 2-ft Thickness 1 per 600-mm Thickness

Note 1: This testing rate applies to when the embankment adjacent to structures is constructed separately from the roadbed embankment.

(B) Option 2: Comparison Test Method

The Engineer will perform VT using the rates as specified in Table 4, the appropriate forms specified in Table 5 and the following steps:

(1) **Step 1**: Perform six (**6**) sets of comparison tests, using the LWD Deflection Method with either the DPI (for granular and base) or the Specified Density Method (for non-granular). The tests for each set should be spaced 1 ft (300-mm) longitudinally from each other. Space the six comparison tests, throughout the first 4,000 yd^3, per step 1 of Table 4.

Determine the LWD-TV by using the maximum Deflection Test Measurement where DPI or density measurement values are passing. Continue step 1, until there are 6 passing comparison tests in locations that are close to failure.

(2) **Step 2**: Perform a maximum of ten (**10**) LWD tests before re-validating the LWD-TV.

(3) **Step 3**: Complete three (**3**) additional comparison tests as per step 1.

Determine the LWD-TV by using the maximum Deflection Test Measurement at the comparison locations, from both step 1 and all step 3 values, where the DPI or density measurement values are passing.

(4) **Step 4:** Repeat steps (2) and (3).

(5) **Step 5:** Repeat steps (1) through (4) for new soil types or sources.

Table 4		
Minimum Testing Rates for Comparison Testing		
Step	**Description**	**Minimum Testing Rate**
1	6 Comparison Tests	First 4,000 yd^3 (3,000 m^3)
2	10 LWD Tests	1 / 4,000 yd^3 (1 / 3,000 m^3)
3	3 Comparison Tests	Next 4,000 yd^3 (3,000 m^3)

Table 5	
Form Numbers	
Form Number	**Description**
G&B-605	Option 2: Comparison Test Method – LWD Production Form
G&B-606	Option 2: Comparison Test Method – LWD-Target Value Summary
G&B-203	(Table 2105-6, 2106-6) DCP Penetration Index Method
G&B-204	(Table 2211-3) DCP Penetration Index Method
G&B-205	Full Depth Reclamation – DCP Penetration Index Method

(C) Perform additional measurements for acceptance of re-compacted areas.

(D) Re-evaluate the selected LWD-TV when failing results consistently occur and adequate compaction is observed through quality compaction.

H.6 Indiana Department of Transportation Lightweight Deflectometer Procedure

Indiana Department of Transportation
Field Determination of Strength Using Lightweight Deflectometer
ITM No. 508-12T

1.0 SCOPE

1.1 This test method covers the determination of deflections with a Light Weight Deflectometer (LWD).

1.2 The LWD may be used for granular soils, coarse aggregates, chemically modified soils, or as directed by the Department. Granular soils with aggregates retained on the 3/4 in. sieve, coarse aggregate sizes No. 43, 53, and 73, and structural backfill sizes 2 in. and 1.5 in. shall be tested with the LWD.

1.3 The LWD test relates deflection with the Dynamic Elastic Modulus and is defined as the maximum axial stress of a material in sinusoidal loading divided by the maximum axial strain during that loading.

1.4 The values stated in SI metric units are to be regarded as standard, as appropriate for a specification with which this ITM is used.

1.5 This ITM may involve hazardous materials, operations, and equipment and may not address all of the safety problems associated with the use of the test method. The user of the ITM is responsible for establishing appropriate safety and health practices and determining the applicability of regulatory limitations prior to use.

2.0 REFERENCES.

2.1 ASTM Standards.

E 2583 Measuring Deflections with a Light Weight Deflectometer (LWD)

3.0 TERMINOLOGY. Definitions for terms and abbreviations shall be in accordance with the Department's Standard Specifications, Section 101 and the following:

3.1 Granular Soil. Soil that is non-cohesive with 35% or less material passing the No. 200 sieve.

3.2 Coarse Aggregate. Aggregate that has a minimum of 20% retained material on the No. 4 sieve.

3.3 Chemically Modified Soil. Soil that has been modified with Portland cement, fly ash, lime, cement by-product, or a combination of these materials

4.0 SIGNIFICANCE AND USE. This ITM shall be used to determine the surface deflection resulting from an application of an impulse load using the LWD. The resulting deflections are used to determine the stiffness of granular materials in embankments and other applications.

5.0 APPARATUS.

5.1 Force-Generating Device, 10 kg ±0.1kg falling weight with a guide system, lock pin and spring assembly. The mass of the guide rod is 5 kg ±0.25 kg and the maximum impact force is 7.07 kN. The fixed drop height shall be in accordance with the manufacturer recommendation.

5.2 Loading Plate, made of steel, having dimensions of 300 mm in diameter and 20 mm in thickness. The plate shall have two handles and weigh 15 kg ±0.25 kg.

5.3 Deflection Sensor, capable of measuring the maximum vertical movement with an accelerometer. The accelerometer is required to be attached to the center of the plate.

5.4 Data Processing and Storage System, capable of displaying and recording the loading data, deflection data, and the test location for each test.

5.5 Miscellaneous equipment such as a spade, broom, trowel, and cotton gloves.

6.0 TEST AREA PREPARATION. The test area shall be leveled so that the entire undersurface of the load plate is in contact with the material being tested. Loose and protruding material shall be removed. If required, any unevenness shall be filled with fine sand. The test shall not be conducted if the temperature is below freezing. The test area shall be at least 1.5 times larger than the loading plate.

7.0 PROCEDURE.

7.1 Rotate the loading plate approximately 45° back and forth to seat the plate. The plate should not move laterally with successive drops of the falling weight.

7.2 Place the force generating device onto the loading plate. Hold the guide rod perpendicular to the loading plate.

7.3 Conduct three seating drops by raising the falling weight to the release mechanism, allowing the hammer to fall freely, and catching the falling weight after the weight rebounds from striking the plate.

7.4 Following the three seating drops, conduct three drops of the falling weight and record the data for each drop. A test is considered invalid if the operator does not catch the falling weight after the weight rebounds from the load plate or the load plate moves laterally. A new test area is required at least 2 ft. away from the original area of testing when the test is invalid. If the change in deflection is 10% or greater for any two consecutive drops, the material shall require additional compaction or aeration and steps 7.1, 7.2, and 7.3 shall be repeated.

7.5 Record the smartcard number and the test drop deflection measurements on the data collection form.

8.0 CALCULATIONS. Calculate the average deflection of the three drops after the seating drops.

9.0 REPORT. Report the average deflection in mm.

H.7 Minnesota Department of Transportation Lightweight Deflectometer Specification

Strikethroughs are as they appear in the original document.

From Mn/DOT Specifications (Modified). Proposed revisions (October 2014) are shown as strikethrough.

(2105) LWD QUALITY COMPACTION METHOD (TARGET VALUE)

Modified Spec.

The LWD is used in lieu of testing compaction compliance by use of a sand cone, nuclear gauge or DCP. The target values are only for a Zorn 2000 or 3000 LWD. The Target Values would need to be adjusted, if a different device is used. Unless otherwise designated, all test procedures are in the Grading and Base Manual. Use form G&B-604 or similar, as found on the Grading and Base Website.

The following is added Mn/DOT 2105.1, 2106.1, and 2211.1 "DESCRIPTION":

a. DEFINITIONS

"Deflection Test Measurement" is the average deflection measured from the fourth, fifth, and sixth drop in the testing sequence. The first, second, and third drop in the testing sequence are seating drops.

"Seating Drops" are the first, second, and third drops in the testing sequence and are not used for acceptance.

"Compaction Curve" is defined as the relationship between the average of the Deflection Test Measurements and the roller passes.

"LWD-TV" is the LWD Target Value (LWD-TV), the Maximum Allowable Deflection ~~(or Minimum Allowable Elastic Modulus)~~ values allowed for a given soil or material type.

"Maximum Allowable Deflection" is the maximum settlement allowed beneath the loading plate.

~~"Minimum Allowable Elastic Modulus" reflects the material's tendency to be deformed elastically (non-permanently) when a force is applied to it.~~

b. LWD TESTING CONSTRAINTS

(A) Do not test within 600 mm (**2 ft.**) of the water table.

(B) The LWD Deflection Method cannot be used for embankment thicknesses less than:

(1) 25 mm (**6 in.**) when no site preparation is needed, or

(2) 460 mm (**12 in.**) when site preparation is needed to meet the required LWD test depths.

The required LWD test depths are presented in the document "LWD Deflection Method Test Procedures," which is available on the Grading and Base Website.

c. Construction Requirements

Compact the entire lift to achieve the LWD-TV per Table S-xx.1. ~~Either~~ Use LWD-TV parameter Maximum Allowable Deflection~~. or Minimum Allowable Elastic Modulus) may be used, unless specifically designated in the contract.~~ Ensure the same LWD-TV parameter is used throughout the entire project.

Re-evaluate the selected LWD-TV, and contact the Grading and Base Engineer when failing results consistently occur and adequate compaction is observed through quality compaction.

Table S-xx.1

LWD Target Values

Specification	**Material Type**	**Maximum Allowable Deflection (mm)**	**Minimum Allowable Elastic Modulus (MPa)**
2105 or 2106	Granular	0.78	~~40~~
	Clay and Clay Loam	1.47	~~20~~
2211	Base	0.55	50

d. Testing Rates

Test Rates shall follow the Dynamic Cone Penetration (DCP) Index Method schedule of materials as provided in these Special Provisions.

e. Moisture Requirements

The moistures requirements shall follow Mn/DOT specifications 2211, 2221, and as provided in these Special Provisions.

End of Mn/DOT specification.

The Minnesota Department of Transportation conducted numerous studies correlating soil properties, densities, and portable equipment output. The target values obtained from this research are being re-evaluated (October 2014) for incorporation in the Mn/DOT Standard Specifications. For DCP and LWD Target Values, see *"Using the Dynamic Cone Penetrometer and Light Weight Deflectometer for Construction Quality Assurance"* (Siekmeier et al., 2009). Similar tables from the previously referenced project are also included in NCHRP Synthesis 456, *"Non-Nuclear Methods for Compaction Control of Unbound Materials"* (Nazzal, 2014).

H.8 Indiana Department of Transportation Lightweight Deflectometer Specification

Revisions are italicized. Proposed changes are shown as strike through.

LIGHT WEIGHT DEFLECTOMETER TESTING

(Revised 3/4/2014 3/~~3/4/13~~)

DESCRIPTION

This work shall consist of testing aggregates or chemically modified soils with a Light Weight Deflectometer, LWD.

MAXIMUM ALLOWABLE DEFLECTION

The maximum allowable deflection will be determined based on *either following criteria or* a test section for each material type.

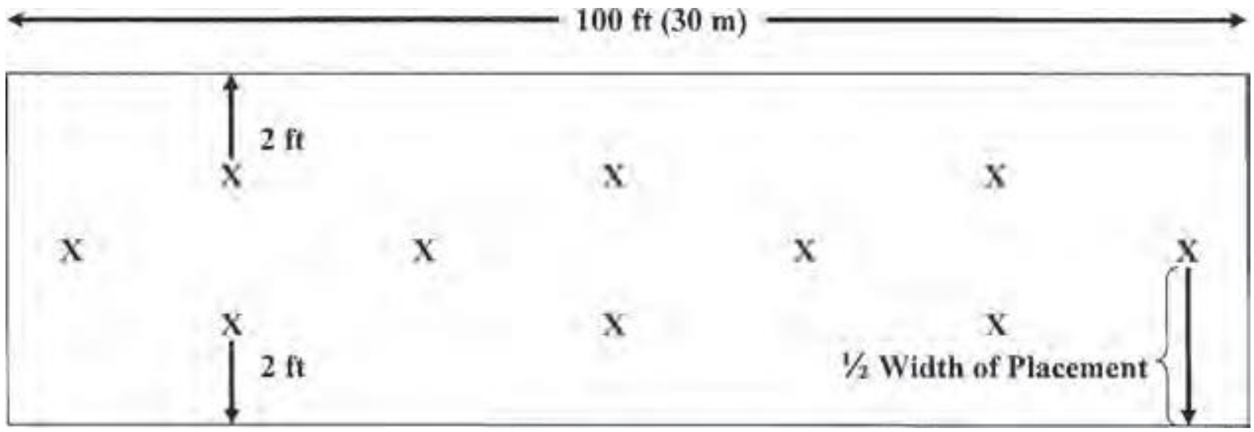
Material type	Maximum allowable deflection
Lime Modified soils	0.30 mm
Cement modified soils	0.27 mm
Aggregates over lime modified soils	0.30 mm
Aggregates over cement modified soils	0.27 mm

TEST SECTIONS

Test sections shall be constructed in the presence of a representative of the Office of Geotechnical Services with the available equipment of the Contractor to determine the roller type, pattern, and number of passes for the maximum allowable deflection.

The Engineer will select an area approximately 100 ft. (30 m) by the width of the material placed for the test section. Areas not meeting these minimum criteria will be considered. The subgrade shall be proof-rolled in accordance with 203.26 prior to construction of the test section for aggregates. Chemically modified soils shall be cured at least ~~48~~ *24* hours prior to testing of the test section.

One ~~two~~ moisture test will be performed in accordance with AASHTO T 255 for aggregates prior to compaction of the test section. The ~~average of the two~~ moisture content values shall be controlled within -*3* ~~-6~~ percentage points of optimum moisture content and the optimum moisture content. Ten LWD tests will be performed on the test section at the following approximate locations:



Aggregate Test Section with LWD only

A test section shall be constructed and LWD testing will be performed to determine the maximum allowable deflection if only the LWD is used. The roller shall be operated in the *medium setting* vibratory mode and initially 4 passes shall be placed on the aggregate in the test section. The average deflection of the 10 random tests will be determined after completion of the 4 passes. One additional pass of the roller in the vibratory mode shall be made and 10 LWD tests will be taken at the same locations. If the difference between the average LWD test values obtained from passes 4 and 5 is equal to or less than *0 .02* ~~0.01~~ mm, the compaction will be considered to have peaked and the average of the 10 LWD values at 5 passes will be used as the maximum allowable deflection. If the difference between the average LWD test values is greater than 0.01 mm, an additional roller pass in the vibratory mode shall be placed and 10 LWD tests will be taken at the same locations. This procedure will continue until the difference of the average of the 10 LWD tests between consecutive roller passes is equal to or less than *0.02* ~~0.01~~ mm. *A minimum of six roller passes shall be performed to determine the allowable deflection and t* ~~T~~he maximum allowable deflection will be the lowest average of the 10 LWD test values.

Aggregate Test Section with Density Control

In the aggregate test section, LWD testing will be performed concurrently with density testing performed in accordance with AASHTO T 310*/ or AASHTO T 191*. The density shall meet the requirements of 301.06. *A minimum of six roller passes shall be performed to determine the allowable deflection*. The maximum allowable deflection will be the average of the 10 LWD test values.

Chemically Modified Soil Test Section with LWD and DCP

The LWD testing of the chemically modified test section will be conducted concurrently with the requirements of 215.09 or *maximum allowable deflection as specified.* Chemically modified soils shall be cured at least *24* ~~48~~ hours prior to testing of the test section. The maximum allowable deflection will be the average of the 10 LWD test values.

COMPACTION ACCEPTANCE WITH LIGHT WEIGHT DEFLECTOMETER

The maximum allowable deflection will be determined from the test section or specified. Acceptance testing with a LWD shall be in accordance with ITM 508. The optimum moisture content and gradation will be determined by performing AASHTO T 99 Method C, AASHTO T 11, and AASHTO T 27 on representative samples of the aggregates.

The moisture content of the aggregate shall be controlled within-*3* ~~-6~~ percentage points of the optimum moisture content and the optimum moisture content *at the time placement. Water shall be added in stockpiles only.* ~~If additional moisture is required, the water shall be applied uniformly over the full area of the material with a mechanical device approved by the Engineer.~~ The frequency of the moisture tests for aggregates will be one moisture test per day. ~~The moisture content of the chemically modified soil shall be in accordance with 215.09.~~

Acceptance of the compaction of aggregates or chemically modified soils will be determined by averaging three LWD tests obtained at a random station determined in accordance with ITM 802. *The acceptance LWD and moisture testing will be performed after 24 hours of compaction.* The location of the three tests will be at 2 ft. from each edge of the construction area and at 1/2 of the width of the construction area. The average deflection shall be equal to or less than the maximum allowable deflection determined by the test section. The frequency of testing will be one test for each 800 tons for compacted aggregate and one test for each 1400 yd^3 of chemically modified soil.

H.9 Missouri Department of Transportation Dynamic Cone Penetrometer Specifications

Missouri Department of Transportation Specification Section 304, Aggregate Base Course, allows the DCP to be used for testing the compaction of Type 7 base aggregates as an alternative to standard density testing.

304.3.4.3 Type 7 aggregate base under both roadway and shoulders shall be compacted to achieve an average dynamic cone penetration index value through the base lift thickness less than or equal to 0.4 in. per blow, as determined by a standard dynamic cone penetrometer (DCP) device with a 17.6 lb. hammer meeting the requirements of ASTM D6951.

304.3.4.3.1 Water shall be applied to the Type 7 base material during the mixing and spreading operations so that at the time of compaction the moisture content is not less than 5 percent of the dry weight.

304.3.4.3.2 Type 7 base shall be tested with the DCP within 24 hours of placement and final compaction.

304.4 Quality Control/Quality Assurance (QC/QA).

304.4.1 Quality Control. The contractor shall control operations to ensure the aggregate base, in place, meets the specified requirements for density, thickness, gradation, deleterious, and plasticity index. Tests shall be taken at random locations designated by the engineer at the following frequencies:

Contractor Frequency: 1 per 1,000 tons, minimum of 1 per day.

Engineer Frequency 1 per 4,000 tons, minimum of 1 per project

1007.4 Type 7 Aggregate.

1007.4.1 Type 7 aggregate for base shall consist of crushed stone, sand and gravel, or reclaimed asphalt or concrete. The aggregate shall not contain more than 15 percent deleterious rock and shale. The fraction passing the No. 40 sieve shall have a plasticity index not to exceed six. Any sand, silt and clay, and deleterious rock and shale shall be uniformly distributed throughout the material.

1007.4.2 Type 7 aggregate shall be in accordance with the following gradation requirements:

Sieve	Percent by Weight
Passing 1.5-in.	100
Passing 1-in.	70-100
Passing No. 8	15-50
Passing No. 200	0-12