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Compaction Quality Assessment and Soil-cement Stabilization for Iowa Embankment Construction

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Compaction quality assessment and soil-cement stabilization for Iowa embankment construction

by

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ABSTRACT

This study set out to independently evaluate the quality of compaction using the current specifications. The independent testing results showed that higher rates of data fell outside of the target limits, while in many cases the contractor QC and the agency quality assurance (QA) data fell within the target limits. Statistical analysis results from this study showed some improvements over results from previous projects in terms of the percentage of data that fell within the specification limits. However, QC/QA results are not consistently meeting the target limits/values. Intelligent compaction (IC) technology offers a new and alternative way to control compaction quality. In this study, comparative IC results and in situ point test results involving traditional moisture-density test measurements and performance-based measurements such as light weight deflectometer elastic modulus and dynamic penetration index values were evaluated. Results show that this alternative method can contribute to improved process control, but careful calibration is required.

Based on the field observation of often wet materials at various sites, a laboratory and numerical study was performed to evaluate an approach to assess compaction quality in terms of controlling post-construction settlement of the fill. Results indicated that this approach can be helpful, but empirical relationships between moisture-density-soil index property and consolidation parameters are required to be able to effectively implement such an approach. Some correlations were developed in this study, but must be further validated.

Embankment subgrade soils in Iowa are generally rated as fair to poor as construction materials with low bearing strength, high volumetric instability, and durability problems. Cement stabilization offers opportunities to improve these soils conditions. A laboratory investigation was designed and executed in this study with the main objective of developing correlations between soil index properties, unconfined compressive strength (UCS) and cement content. A total of 28 granular and non-granular materials obtained from 9 active construction sites in Iowa were tested using 4 to 12% type I/II portland cement contents. Specimens were prepared using Iowa State University 2 in. by 2 in. compaction apparatus and tested for 28 day UCS with and without vacuum saturation. Results indicated that statistically significant relationships exist between soil index properties, UCS and cement content.

CHAPTER 1. INTRODUCTION

Problem Statement

Embankments are critical components of infrastructure that support pavement systems and bridge approaches. Embankments are designed to provide the specified elevation for the performance life of the structure. The quality of embankment construction directly influences the performance of the supported infrastructure and the cost of future maintenance and reconstruction. A quality embankment requires proper selection of fill materials, adequate moisture and density control, and adequate compaction. Desirable engineering properties for a quality embankment include adequate strength, stability, and density; low permeability; low shrink swell behavior; and low collapsibility depending on the design requirement.

Embankment subgrade soils in Iowa are generally rated as fair to poor as construction materials, with a majority of the soils classifying as A-4 to A-7-6 according to the AASHTO Soil Classification System (AASHTO 2012). These soils can exhibit low bearing strength, high volumetric instability, and freeze/thaw or wet/dry durability problems. Therefore, proper field construction controls and the accompanying quality control (QC) and quality assurance (QA) processes are important to achieve the desired embankment quality. In addition, the Iowa Department of Transportation (DOT) is considering the use of portland cement as an additive for stabilizing embankment materials in situ.

Past research in Iowa shows that significant variability exists in the final compaction moisture content for embankment fills and that this is largely influenced by the generally wet ground conditions of borrow materials and rainfall events during the Iowa construction season (Larsen 2007, White and Bergeson 1999). The variability of dynamic cone penetrometer (DCP) index values in surficial lifts has been observed to be high. Bergeson et al. (1998) found that a significant contributor to slope instability issues and pavement roughness problems was that embankment fill materials were being placed outside the specified moisture and density control limits. In addition, wet soils compacted near the zero air voids curve can result in high pore pressure as subsequent lifts are placed and compacted, which can lead to reduced shear strength. This action can create shear stresses on potential failure surfaces, which can lead to subgrade instability and/or slope failures (Lambe and Whitman 1969).

A specification for contractor moisture QC in roadway embankment construction has been in use for approximately 10 years in Iowa on about 190 projects. The use of this QC specification originated from Iowa Highway Research Board (IHRB) embankment quality research projects from the late 1990s. Since then, the Iowa Department of Transportation (DOT) has specified compaction with moisture control on most embankment work under pavements. The motivation for the research described in this dissertation was based on work performed by Iowa State University (ISU) researchers at a few recent grading projects that demonstrated that embankments were being constructed outside moisture control limits, even though the contractor QC and QA testing showed that all work was being performed within the control limits. This finding initiated the need for a more detailed study and testing at several active grading projects across Iowa.

Research Objectives

This research was initiated to evaluate the quality of embankments constructed per current Iowa DOT embankment construction specifications, especially moisture-density QC/QA. An ISU research team conducted in situ moisture-density and stiffness measurements of compacted fill at eight active embankment construction sites in six Iowa counties. A total of 28 granular and non-granular materials were collected from these sites for laboratory soil classification and soil index property testing.

Embankment subgrade soils in Iowa are generally rated as fair to poor as construction materials with low bearing strength, high volumetric instability, and durability problems. Cement stabilization offers opportunities to improve these soils conditions. A laboratory investigation was designed and executed in this study with the main objective of developing relationships between soil index properties, unconfined compressive strength (UCS) and cement content.

The research team set out to coordinate with the Iowa DOT Office of Construction and Materials and the Iowa DOT Office of Design Soils Design Section to select 8 to 12 projects for field testing. Projects were selected to be representative of the soil and project conditions statewide. Figure 1 shows the selected project locations in reference to surficial soil types in Iowa.

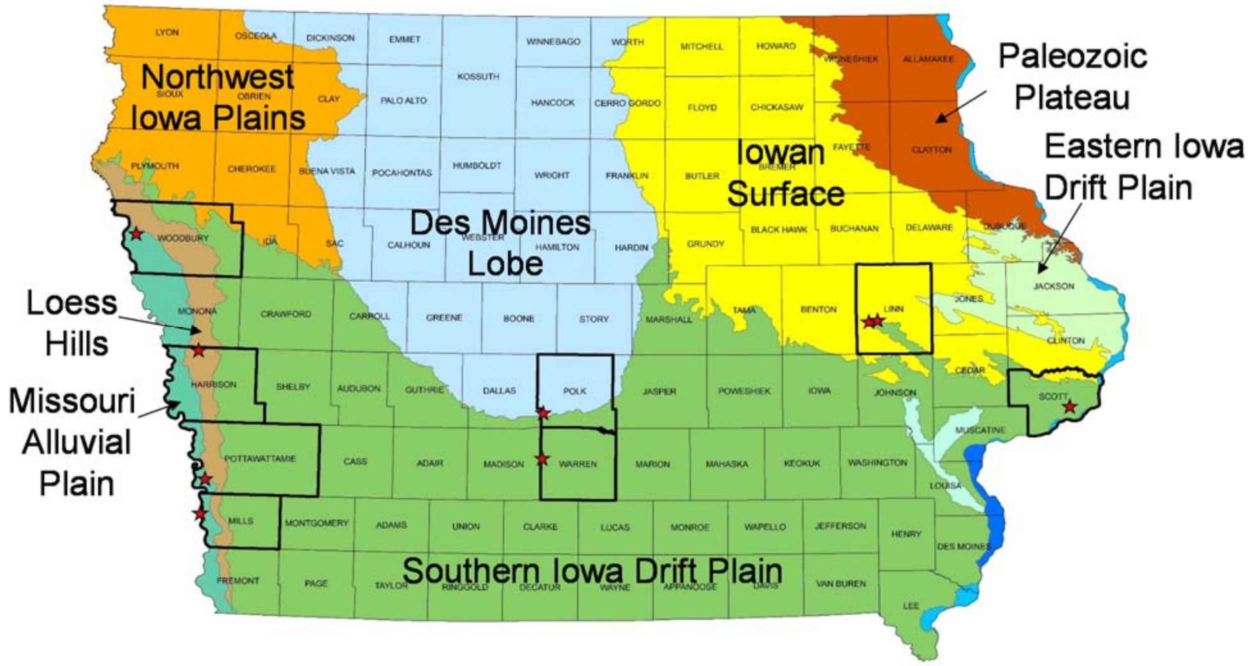


Figure 1. Eleven project sites identified for field evaluation

Once the projects were identified, the research team traveled to the selected sites for in situ testing. The in situ testing areas were typically sections of about 1,000 ft in length. At each site, 10 to 30 moisture and dry density measurements were collected to provide a statistically significant dataset for analysis. Representative bulk samples were collected from each site for laboratory characterization. Using the field test results, comparisons were made to the project target requirements for moisture content and density. DCP tests were also performed to study the lift thickness and stability uniformity. For project sites where data were available, the data generated by the Iowa DOT and contractor were included with the ISU data to provide additional analysis of the QC/QA results.

In terms of the cost of the implemented moisture and density specifications, Table 1 summarizes the unit bid prices for the awarded contracts for the 11 projects identified in Figure 1.

Table 1. Summary of bid costs for implementation of Iowa DOT moisture and moisture-density specification

County	Specification	Unit Price per Cubic Yard	Total Quantity (Cubic Yards)	Total Cost (USD)
Linn	Moisture	$0.40	602,243	$240,897.20
Woodbury	Moisture	$0.80	360,776	$288,620.80
Mills	Moisture	$0.20	224,025	$44,805.00
Warren	Moisture	$0.21	170,752	$35,857.92
Polk	Moisture	$0.80	166,710	$133,368.00
Scott	Moisture	$0.10	119,267	$11,926.70
Pottawattamie	Moisture	$1.02	107,753	$109,908.06
Linn	Moisture	$0.35	64,331	$22,515.85
Harrison	Moisture	$0.40	60,327	$24,130.80
Linn	Moisture-Density	$0.80	79,583	$63,666.40
Linn	Moisture-Density	$0.75	55,507	$41,630.25
			TOTAL	**$1,017,327.00**

Of these projects, nine included a moisture control specification while two included a moisture-density control specification. On average, the cost of implementing a moisture control specification was about $0.49/cubic yard (cy), and the cost of implementing a moisture-density control specification was about $0.78/cy.

In addition, a demonstration project located on US highway 65 near Altoona, Iowa, was initiated as a pilot project to provide hands-on experience to the contractor with intelligent compaction technology for embankment fill construction. The project was established through a partnership between Iowa State University, Iowa Department of Transportation, and Caterpillar, Inc. The ISU research team was present on site to conduct in situ testing beyond what was required in the project specification for demonstration purposes. In situ point testing was conducted at selected locations to develop correlations with the IC measurements. Point testing included drive core testing for dry density (γ_d) and moisture content (w), dynamic cone penetrometer (DCP) testing for dynamic penetration index (DPI), and light weight deflectometer (LWD) testing for elastic modulus (E_{LWD}). Zorn LWD testing was conducted with 200 mm diameter and 300 mm diameter plate setups. The machine was set up with real time kinematic (RTK) global positioning system (GPS), onboard display, and data documentation/software systems. The RTK-GPS measurements were used to determine pass coverage and analyze empirical correlations between spatial IC-measurement values (MVs) and in situ point measurements.

The following are the key research objectives of this study:

- Assess the current state-of-practice in terms of how compaction specifications are implemented in state of Iowa
- Compare the independent ISU in situ test results to the in situ data conducted by contractor QC and DOT QA
- Evaluate cement stabilization as a method for shallow ground improvement
- Develop a relatively simple and easy-implemented standard procedure for DOT to design cement stabilization for a given project
- Develop and understand the relationships between soil index properties, UCS and cement content
- Conduct laboratory and numerical studies to assess quality in terms of post-construction consolidation of fill
- Analyze intelligent compaction data and develop future specification options to improve quality

Organization of the Dissertation

Following this Introduction chapter, this dissertation consists of another seven chapters: Background and literature review, Testing and Analysis Methods, Materials, Field Test Results, Lab Test Results, Data Analysis and Discussion, and Conclusions and Recommendations.

This study consists of four aspects. The first aspect is to assess the current state-of-practice in terms of how compaction specifications are implemented in state of Iowa. And this involves a quality control testing as a part of the contractor, and the specification language about what type of testing, how the compaction needs to be performed, and how the meeting that quality control specification, also same for quality assurance specification where DOT needs to do certain types of monitoring testing. The second aspect is to evaluate cement stabilization as a method for shallow ground improvement. This evaluation was limited to laboratory testing in my study and was more looked at as a procedural development. And the DOT can use this in their practice to design a certain type of cement stabilization method. The third aspect is to use an alternative way to control compaction quality. The laboratory and numerical studies to assess quality in terms of post-construction consolidation of fill were conducted. The fourth aspect is to analyze intelligent compaction data and develop future specification options to improve quality.

CHAPTER 2. BACKGROUND/LITERATURE REVIEW

In this chapter, a brief summary of previous embankment quality evaluation projects in Iowa is provided along with the ISU testing results from those projects, an overview of intelligent compaction research and implementation projects undertaken in Iowa for embankment construction is provided, and a summary of the earthwork QC/QA specifications followed by different state departments of transportation is provided along with alternative specification options introduced by some state DOTs for moisture-density control.

IHRB TR-401 Phase I Summary

Phase I research was initiated as a result of internal Iowa DOT studies that raised concerns about the quality of embankments currently being constructed. Some large embankments had recently developed slope stability problems resulting in slides that encroached on private property and damaged drainage structures. In addition, pavement roughness was observed shortly after roads were opened to traffic, especially for flexible pavements at transitions from cut to fill and on grade and pave projects. These problems raised questions regarding the adequacy of the Iowa DOT embankment construction specifications. The primary objective of Phase I was to evaluate the quality of embankments being constructed under the current specifications.

The in situ moisture contents relative to optimum moisture content (Δw) and the relative compaction (RC) test results obtained from the Phase I study are summarized as histograms in Figure 2.

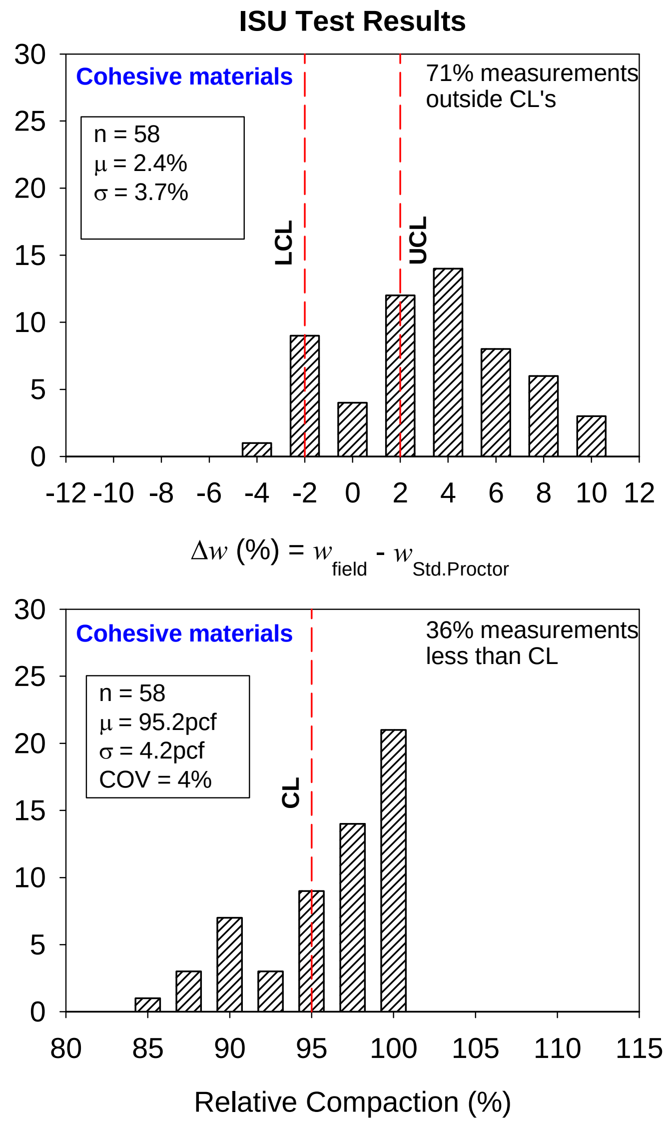


Figure 2. IHRB TR-401 Phase I: Histograms of moisture and relative compaction test results from ISU testing

The results indicate that about 37% of the RC test measurements and 71% of the moisture content test measurements were outside of the control limits. Based on the overall test results and field observations from Phase I, Bergeson et al. (1998) indicated that consistent embankment quality was not being attained under the existing Iowa DOT specifications at that time.

IHRB TR-401 Phase II Summary

Phase II research was initiated to investigate different methods and techniques that could be used to improve the Iowa DOT soil classification and compaction control specifications based on observations and data collected at small-scale pilot compaction studies. Histogram plots of in situ test results are summarized in Figure 3.

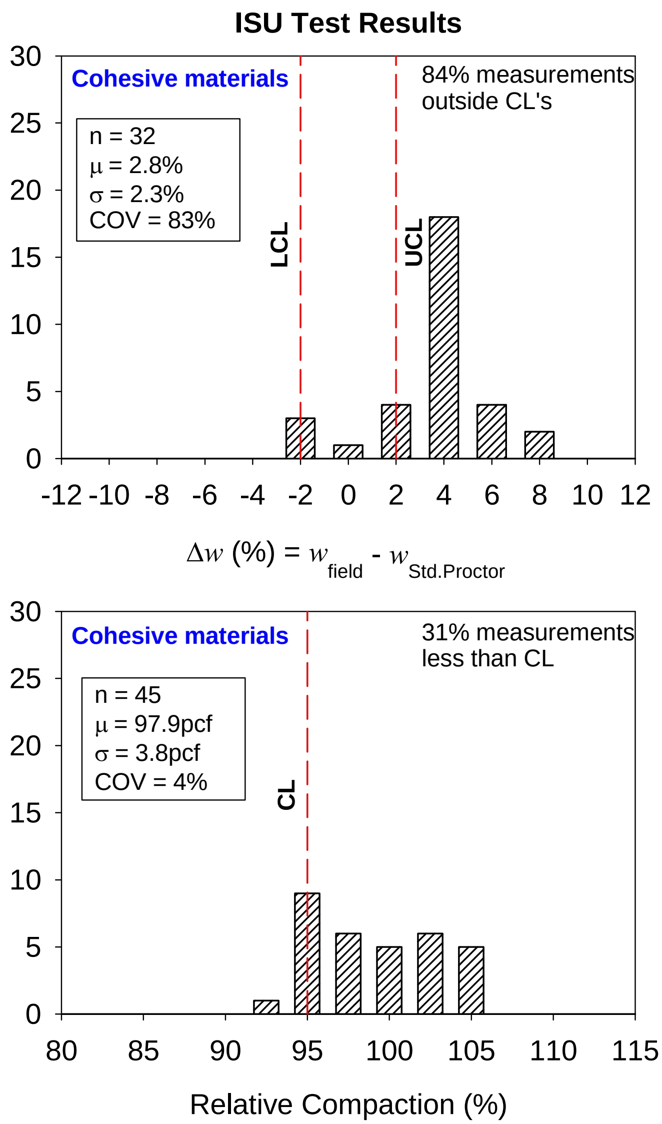


Figure 3. IHRB TR-401 Phase II: Histograms of moisture and relative compaction test results from ISU testing

Similar to the Phase I test results, about 31% of the RC test measurements and 84% of the moisture content test measurements were outside of the control limits.

The results from the pilot studies indicated that new specifications were required that better account for the differences between the behavior of cohesive and cohesionless soils. The Iowa Empirical Performance Classification (IEPC) system was developed. Compared with former specifications, the IEPC considered many more of the factors that affect the engineering properties of soil. The use of DCP testing was also proposed as a supplement to field moisture-density quality control testing in both cohesive and cohesionless soils because DCP results provide in situ measurements of fill strength and can be used to assess the variability of fill strength with depth (White and Bergeson 1999).

IHRB TR-401 Phase III Summary

Field testing on active project sites similar that of previous phases was continued during Phase III. The results are summarized in Figure 4, which shows that about 24% of the RC test measurements and 42% of the moisture measurements were outside of the control limits.

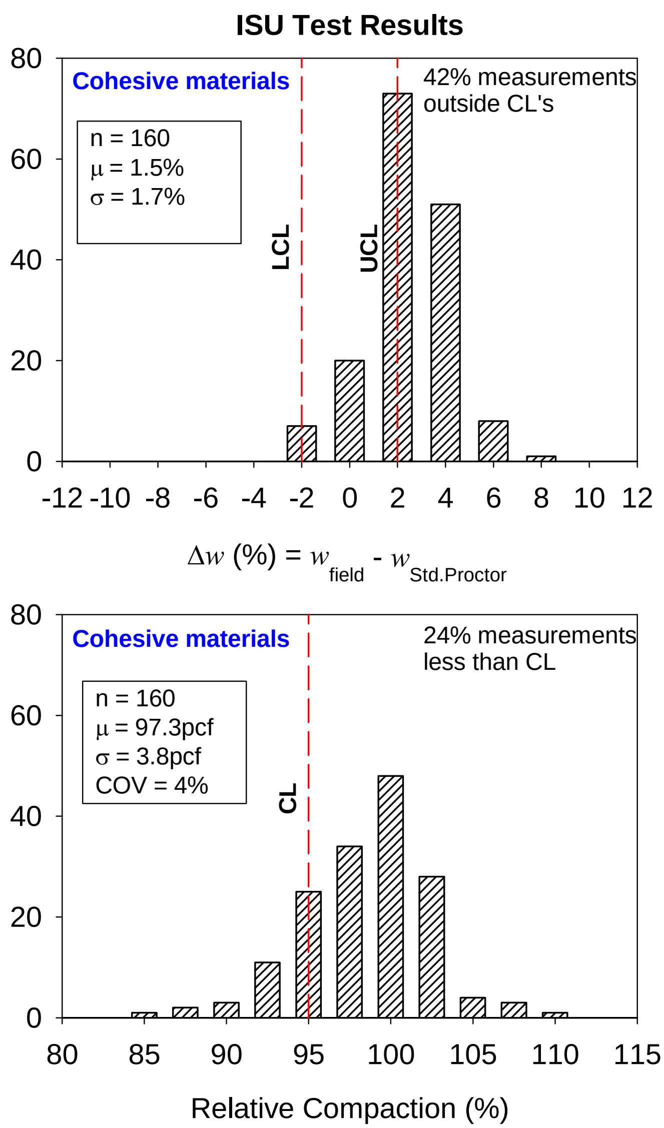


Figure 4. IHRB TR-401 Phase III: Histograms of moisture and density test results

Phase III research focused on creating a comprehensive earthwork construction specification, the Quality Management Earthwork (QM-E) program, which incorporated the findings and recommendations of the previous two research phases into a practical field construction specification. The QM-E was then implemented on a full-scale pilot project to field test and refine elements of the proposed program for cohesionless soils. The results of this pilot project were promising. The soil classification system worked well in both the design and construction phases of the project, having required only minor modifications. The special provisions of the QM-E program, developed jointly with the Iowa DOT, also worked well and required minimal alteration. Ultimately, the overall quality of the embankment fill showed improvement, as indicated by DCP testing and the additional discing that was required. The cost of this improvement was nominal, 3.3% for the additional discing and the application of the QM-E program, in comparison to the perceived improvement in quality (White et al. 2002).

IHRB TR-492 Phase IV Summary

In situ moisture and density field test results from active project sites during Phase IV are summarized in Figure 5, which shows that about 26% of the RC test measurements and 75% of the moisture measurements were outside of the control limits.

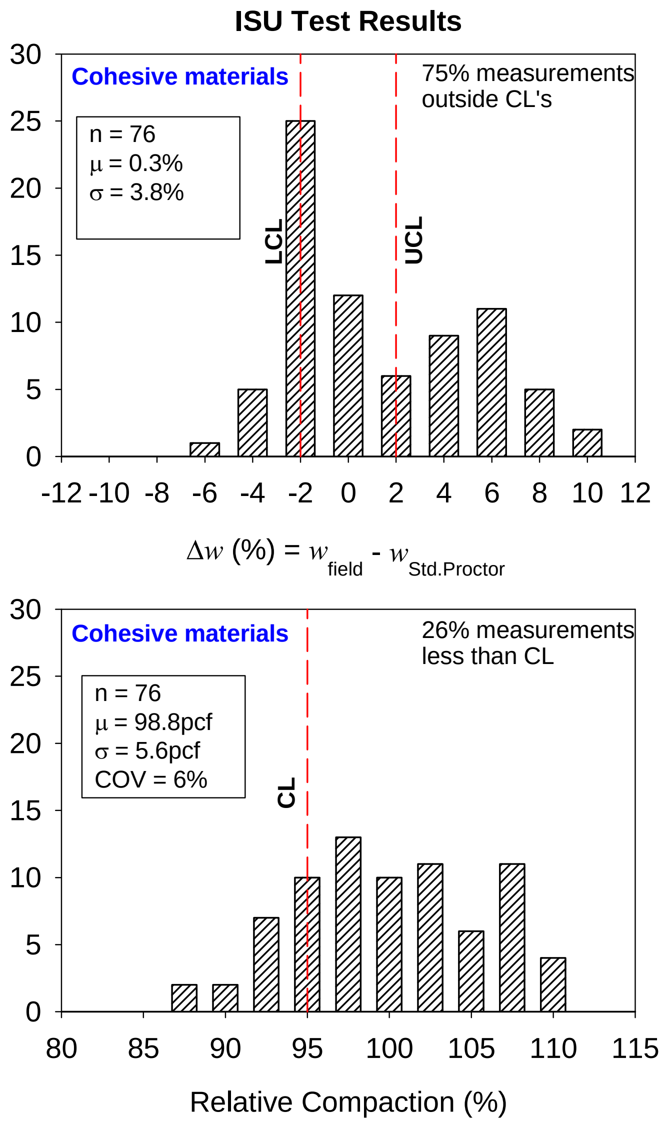


Figure 5. IHRB TR-401 Phase IV: Histograms of moisture and density test results

The costs of implementing the QM-E program in the previous project had been relatively small, but it was believed that if the fill material were considerably more difficult to moisture condition, as is the case with cohesive soils, the special provisions might prove unreasonable and expensive. Therefore, a second full-scale pilot project was conducted on cohesive soils. The goals of this pilot project were to (1) field test and refine elements of the QM-E program for cohesive soils, (2) train additional contractor and Iowa DOT personnel in the Certified Grading Technician Level I program, and (3) review other state DOT earthwork specifications for potential modifications to the QM-E special provision. Smaller field studies were also conducted prior to the pilot project to establish the state of the practice in Iowa for construction of earthen embankments in unsuitable soils (White et al. 2007).

Compaction Theories

The compaction of foundation layer soils is an important task on highway construction projects. Shear strength, permeability, and compressibility of soil are important properties in highway embankment construction. Hilf (1991) indicated that shear strength of soil could be increased, and permeability and compressibility of soil could be decreased during the compaction process. Proctor (1933) developed a laboratory test method to determine moisture-density relationship of soils. And he theorized that the moisture within dry of optimum moisture content caused capillary effects that resisted compaction. As the moisture was increased, lubrication of particles allowed greater rearrangement of particles to occur and greater densities to be achieved. The optimum moisture content condition occurred when the soil voids entirely filled with water and a minimum amount of voids that cannot removed via compaction. Increasing the moisture even further will result in increasing amount of voids, decreasing density.

Hilf (1956) was one of the first researchers to apply the concepts of effective stress to explain compaction process. He found the relationship between the effects of capillary pressure, pore air pressure and the shape of the Proctor curve. On the dry side of optimum, there are pore spaces between soil particles and this allows air to be expelled. As moisture is increased, the curvature of menisci start to be flatter and the resistance to compaction is also reduced, which allows higher density to be achieved. As moisture keeps increasing and exceeds the optimum moisture, the air inside of soil particles were trapped and increase the inner pressure of soil particles. And this pressure resisted the compaction and resulted in density decrease.

Barden and Sides (1970) and Seed and Chan (1959) described the effects of compaction on the microscopic structure of clay. As moisture was at dry side of optimum moisture, large macropores existed between macropeds within the clay and these were very resistant to distortion, so the effectiveness of compaction was reduced. As the moisture was increased, these peds became weaker and their ability to reduce compaction was diminished. Eventually, at moisture close to optimum contents, these peds became wet enough that compaction easier results in ped deformation and the macropores were filled with deformed soil. And as moisture was at wet side of optimum moisture, layers of water between soil particles increase in size and result in reduced densities.

Some researchers reported that most QC/QA specifications were developed according to the Proctor test in highway construction practice (Handy and Spangler 2007, Walsh et al. 1997).

Statistical Quality Control

Statistical quality control methods are primarily used to control and assure that a process is working properly and effectively. The objective of the QC/QA process is to monitor and alert the contractor and project owner that some aspect of the process has changed, such as moisture content, density, lift thickness, etc. The control chart is a plot of process performance versus time. A control chart consists of two major parts, observed values and control limits, upper control limit and lower control limit. It is clear to observe the observed values which fall outside of the control limits (ASTM 1951). Vardeman (1998) indicated that the control chart does not provide much information of what is causing the problem and is a simple tool to present the measurements.

Carpenter and Oglio (1964) indicated that statistical quality control plays an important role in implementing specification limits. The specification limits set the level of quality desired, and can be used to motivate the contractor or inspector to provide the quality control desired.

Beaton (1968) suggested that control chart should be accepted as formal contract documents and the moving average and chain sampling should be utilized. The chain sampling is a method to find and combine different useful and relevant information to the initial sampling, especially for small population sampling (Morgan 2008). Davis (1953) recommended to use a cumulative frequency control chart and concluded that averaging values were not reliable for process monitoring. Sherman et al. (1966) also concluded that it was limiting to use statistical methods of quality control for embankment construction.

Intelligent Compaction

Traditional drive core cylinder and nuclear moisture-density testing have played an important role in earthwork quality assessment specifications in the US for decades. This form of QC/QA can be effective but has shortcomings due to regulations, test reproducibility, limited test frequency, small sample size, and that density serves only as a surrogate to strength and stiffness design requirements (White et al. 2013).

Good pavement performance depends more on the uniformity of the subgrade and embankment materials than on the ultimate strength or stiffness of the placement (White et al. 2004). Once a minimum stiffness is achieved pavement performance depends greatly on the spatial variability of the subgrade and embankment. Soils and aggregates are not homogenous and variability of these materials is inherent in their use for construction. Pavement performance can be optimized by controlling the variability of the subgrade and embankment stiffness, limiting the differential stresses within the pavement.

Intelligent compaction technology, roller-integrated compaction monitoring (RICM) is the recording and real-time display of roller parameters and roller-ground interaction values. This includes roller operation parameters, position, roller-ground interaction parameter values, and temperature. RICM for vibratory roller compactor was introduced at the first International Conference on Compaction in 1980 (Thurner and Sandstrom 1980, Forssblad 1980). A major component of RICM, and the component that lends itself most readily to the development of a statistically-based risk management approach to embankment construction, is the recording of an index parameter relating to the compactness or stiffness of the material. Combined with near 100% coverage for data collection, this index value provides the basis for statistical analysis of the embankment quality. While every roller manufacturer provides a slightly different index value for stiffness, each can be correlated to a common stiffness measurement from one or more of several QA tests that can be performed. A relatively new measurement technology, machine drive power (MDP), was developed based on the principal of rolling resistance due to drum sinkage, and can be applied in cohesionless and cohesive materials. The advantage is that MDP is compatible with vibratory and static modes. A significant amount of research has been conducted to evaluate the MDP measurements technology at Iowa State University since 2004 (White et al. 2005, White et al. 2007a, White et al. 2007b, White and Thompson 2008, Thompson and White 2008).

The advantages of RICM measurements are that they are reported electronically on a near-continuous basis and are available to the contractor in real time, so the construction process can be controlled around identifying “soft spots” that need remediation and achieving design target values. The primary weakness with soil stiffness assessment is that moisture control remains the critical factor in the construction process; however moisture control is the critical factor in density assessment under current specifications as well.

Many research studies were conducted over the past 4 decades to develop relationships between different RICM technologies and soil physical and mechanical properties (Thurner and Sandstrom 1980, Forssblad 1980, Floss et al. 1983, Samaras et al. 1991, Brandl and Adam 1997, Krobe 2001, Preisig et al. 2003, Thompson and White 2008, White and Thompson 2008, White et al. 2005, 2007a, 2007b, 2008, 2008a, 2014, Vennapusa and White 2014).

The Iowa DOT has been experimenting with RICM for several years, but has had limited success due to the delay in the post-processing of the data and due to the manufacturer's limited availability of equipment to contractors. Recent advancements in the processing and real-time display, along with improvements in equipment availability make this technology viable for development of new specifications for earthwork.

Preliminary Study

The Iowa DOT cosponsored the IHRB TR-495 study for preliminary evaluation of intelligent compaction (IC) technologies in collaboration with Caterpillar, Inc. (CAT). This study was initiated in 2003 to begin evaluating a compaction monitoring technology developed by Caterpillar, Inc. The technology comprised an instrumented prototype padfoot roller to monitor changes in machine power output resulting from soil compaction and the corresponding changes in machine-soil interaction. The roller was additionally outfitted with a global positioning system (GPS), such that coverage and machine power could be mapped and viewed in real-time during compaction operations. White et al. (2004a) summarized the findings from the field pilot studies conducted at CAT facilities in Peoria, Illinois, and on an earthwork grading project in West Des Moines, Iowa. The significant research findings from the Phase I study are summarized as follows:

- Multiple linear regression analyses were performed using machine power and various field measurements (nuclear moisture and density, DCP index, and Clegg impact value [CIV]). The coefficient of determination (R^2) values of the models indicated that compaction energy accounts for more variation in dry unit weight than the DCP index or CIV.
- Incorporating moisture content in the regression analyses improved model R^2 values for DCP index and CIV and indicated the influence of moisture content on strength and stiffness.

- The compaction monitoring technology showed a high level of promise for use as a QC/QA tool but was demonstrated for a relatively narrow range of field conditions.

The results of this proof-of-concept study provided evidence that machine power may reliably indicate soil compaction with the advantages of 100% coverage and real-time results. Additional field trials were recommended, however, to expand the range of correlations to other soil types, roller configurations, lift thicknesses, and moisture contents. The observed promise of using such compaction monitoring technology in earthwork QC/QA practices also required the development of guidelines for its use, including a statistical framework for analyzing the near-continuous data.

Implementation Program

The Iowa DOT Intelligent Compaction Research and Implementation program was initiated in summer 2009. Three field demonstration projects were conducted in Iowa as part of Phase I of this research program to evaluate three different IC measurement technologies (White et al. 2010): (1) machine drive power (MDP) measurement technology on a Caterpillar CP56 padfoot roller on a US 30 embankment construction project, (2) continuous compaction value (CCV) technology on a Sakai SW880 dual vibratory smooth drum asphalt roller on an asphalt overlay project, and (3) compaction meter value (CMV) technology on a Volvo SD116DX smooth drum vibratory roller on a granular base/subbase layer construction project on I-29. Phase II focused on hot-mix asphalt (HMA) paving projects and is therefore not discussed in this dissertation.

Data obtained from the embankment construction project on US 30 with Caterpillar's MDP technology indicated that the subgrade materials were relatively wet (on average about 5% wet of optimum) during construction. MDP measurements obtained over multiple lifts of embankment fill materials indicated that a "soft" zone with relatively low values on the bottom lift reflected through four successive lifts with similarly low values in that zone. Geostatistical analysis was conducted on the georeferenced IC data, which indicated that variability decreased and spatial continuity improved as additional lifts were placed. Results also indicated that multiple non-linear regression analysis incorporating moisture content improved correlations between light weight deflectometer elastic modulus (E_{LWD}) values and MDP measurements, while there was no statistically significant correlation between dry density and MDP measurements.

Data obtained from the granular base/subbase layer construction project on I-29 using the CMV system included calibration test strips and production area test beds (TBs) with correlations between CMV measurements and in situ nuclear gauge dry density, DCP-California bearing ratio (CBR), and E_{LWD} values. Data from multiple passes indicated that the CMV data were repeatable. CMV maps were able to effectively delineate "soft" and "stiff" zones effectively. Correlations were statistically significant between CMV IC measurements and E_{LWD} and DCP-CBR point measurements, while there was no statistically significant relationship between dry density and CMV measurements.

Soil Stabilization with Cement

Soil stabilization with cement applied on a wide range of soils was studied over the past 6 decades (Balmer 1958, Abboud 1973, Mitchell 1976, Uddin et al. 1997, Lo and Wardani 2002, Lorenzo and Bergado 2004, Sariosseiri 2008, Sariosseiri and Muhunthan 2009, Sariosseiri et al. 2011, Sarkar et al. 2012, Rashid et al. 2014, Riaz et al. 2014).

Spangler and Patel (1950) reported the results of a laboratory study of the effect of various percentages of Portland cement upon the engineering properties of soils frequently used in highway construction in southwest Iowa. They showed that the plastic limit was increased as cement admixture content increased, and plasticity index was decreased as cement admixture content increased because the liquid limit was decreased.

Horpibulsuk (2012) reported the effect of various percentages cement mixture on the specimen's strength development. Three strength development zones were presented: active, inert, and deterioration zone. In the active zone, the pores smaller than 0.1 micron significantly decreased due to cement hydration process, so the strength increased significantly. However, as content of cement additives increased, the desired water was not adequate for hydration, so the strength and quantity of cementitious materials decreased.

Various studies have previously developed the similar relationship between cement dosage and modified soil strength and other engineering properties, such as liquid limit, plasticity index, etc. (Qubain et al. 2006, Sariosseiri et al. 2011, Du et al. 2013, Rashid et al. 2014).

Summary of Earthwork QC/QA Specifications in the US

The standard and supplemental specifications of 50 state departments of transportation were reviewed and are summarized in this section. These standards and specifications are organized separately for granular and non-granular materials in Appendices A and B, respectively. The critical components of the specifications included in the summary are equipment, gradation, placement of materials and compaction method, disc and compaction passes, lift thickness, and moisture content and density/relative compaction requirements.

The QC/QA requirements varied between states and the material types as follows: (1) moisture control only, (2) density control only, (3) moisture and density control, (4) moisture and density control depending on the compaction method, and (5) only moisture or moisture-density control depending on the project. Figure 6 and Figure 7 graphically depict which states have different QC/QA requirements for granular and non-granular materials.

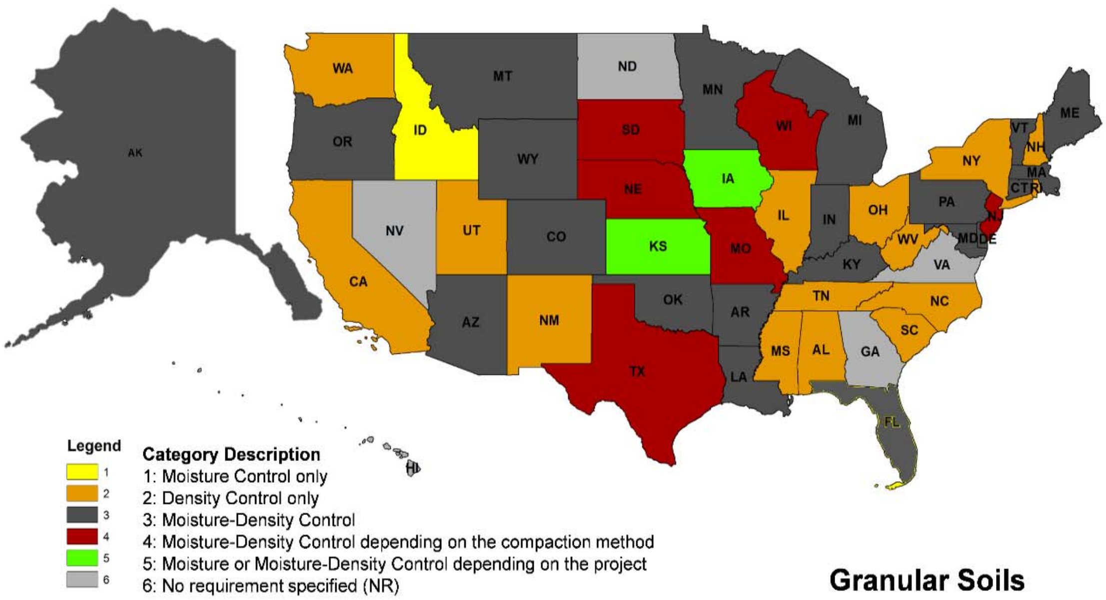


Figure 6. QC/QA requirements for granular materials in the US

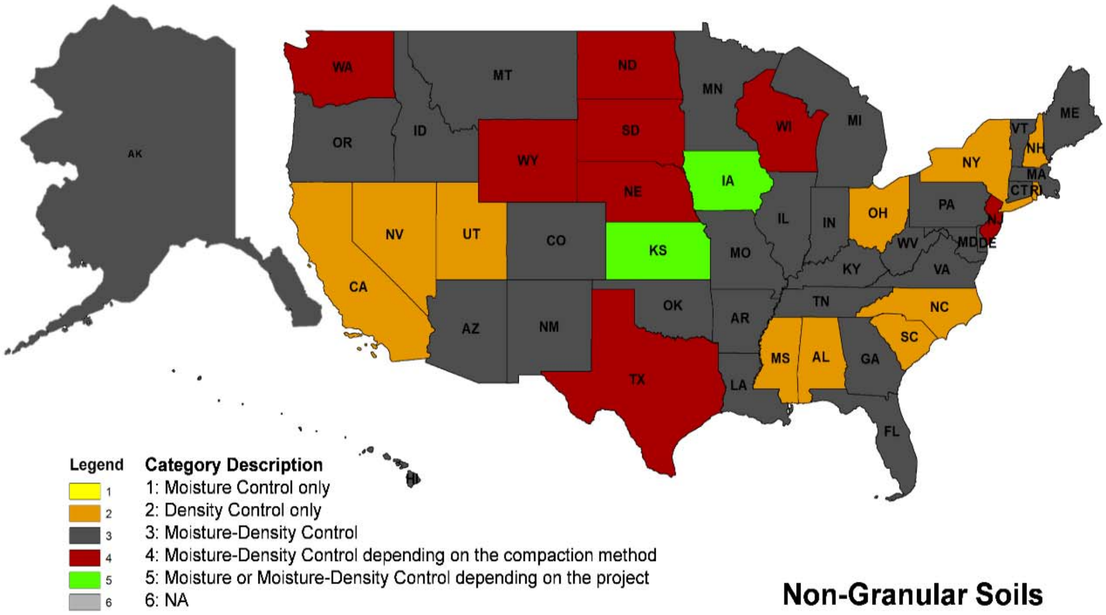


Figure 7. QC/QA requirements for non-granular materials in the US

For granular materials, the most common requirement is moisture and density control, which 21 states require. The second most frequently used requirement is density control only, which 15 states require. One state requires only moisture control; six states require different moisture and density controls depending on the compaction method; two states require moisture or moisture and density control depending on the project. The remaining four states do not specify any requirements in their standard specifications.

For non-granular materials, the most common requirement is moisture and density control, which 29 states require. The second most frequently used requirement is density control only, which 11 states require. Eight states require different moisture and density controls depending on the compaction method; the remaining two states require either moisture or moisture and density control depending on the project.

Alternative Specification Options

Two state DOTs (Minnesota and Indiana) provide alternative specification options to moisture and density control for QA. Both states are currently using these as special provisions in their project specifications.

The Minnesota DOT (MnDOT) provides specification target values for granular materials using DCP and light weight deflectometer (LWD) values (Siekmeier et al. 2009). The target values are based on the grading number (GN) and field moisture content (determined by a field oven-dry test) of the material (Table 2).

Table 2. DCP index target values for granular materials

Grading Number	Moisture Content (percent of dry weight)	Maximum Allowable DPI, mm/blow	Target LWD Modulus Using Dynatest, MPa*§	Target LWD Modulus Using Zorn, MPa*§	Target LWD Deflection Using Zorn, mm*
3.1 – 3.5	< 5.0	10	120	80	0.38
	5.0 – 8.0	12	100	67	0.45
	> 8.0	16	75	50	0.63
3.6 – 4.0	< 5.0	10	120	80	0.38
	5.0 – 8.0	15	80	53	0.56
	> 8.0	19	63	42	0.71
4.1 – 4.5	< 5.0	13	92	62	0.49
	5.0 – 8.0	17	71	47	0.64
	> 8.0	21	57	38	0.79
4.6 – 5.0	< 5.0	15	80	53	0.56
	5.0 – 8.0	19	63	42	0.71
	> 8.0	23	52	35	0.86
5.1 – 5.5	< 5.0	17	71	47	0.64
	5.0 – 8.0	21	57	38	0.79
	> 8.0	25	48	32	0.94
5.6 – 6.0	< 5.0	19	63	42	0.71
	5.0 – 8.0	24	50	33	0.90
	> 8.0	28	43	29	1.05

* LWDs should have a falling mass of 10 kg, plate diameter of 20 cm, and drop height of 50 cm.
§ Modulus calculation assumes a Poisson's ratio of 0.35, and the loading plate is assumed to be rigid. Modulus calculation for Zorn assumes a constant stress of 0.2 MPa, while applied stress is measured for Dynatest.
Source: Siekmeier et al. (2009)

The GN is determined based on sieve analysis test results. The LWD target values are provided in terms of elastic modulus determined from two different manufacturers (Zorn and Dynatest) and deflection values using a Zorn LWD.

MnDOT also provides specification target values for non-granular materials using DCP and LWD based on the plastic limit and field moisture content of the material (Table 3).

Table 3. DCP index and LWD deflection target values for non-granular materials

Plastic Limit (%)	Estimated Optimum Moisture (%)	Field Moisture as a Percent of Optimum Moisture (%)	DPI at Field Moisture (mm/blow)	LWD Deflection Targets Using Zorn	
				Minimum (mm)	Maximum (mm)
non-plastic	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	1.0	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	1.0	1.6
20-24	15-19	70-74	18	0.8	1.4
		75-79	21	0.9	1.6
		80-84	24	1.0	1.7
		85-89	28	1.2	1.9
		90-94	32	1.4	2.1
25-29	20-24	70-74	24	1.0	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	2.0
		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	3.0

Source: Siekmeier et al. 2009

The optimum moisture content of the material is estimated using the plastic limit of the material, based on empirical relationships MnDOT developed for Minnesota soils. LWD target values are provided in terms of minimum and maximum deflection values using a Zorn LWD.

The Indiana DOT provides specifications with target limits for using DCP to determine the in situ strength of granular soils, non-granular soils, and chemically modified soils (Indiana DOT 2015a, Indiana DOT 2015b). Table 4 summarizes the criteria the Indiana DOT uses based on the maximum dry density and optimum moisture content for non-granular materials (sandy soils listed in Table 4 are presumed to be sandy clay soils because they are referenced as non-granular material) and granular soils with different maximum particle sizes.

Table 4. QA requirements using DCP test measurements for different non-granular materials

Textural Classification	Maximum Dry Density (lb/ft^3)	Optimum Moisture Content Range (%)	Acceptable Minimum DCP Blows for 6 in. Penetration	Acceptable Minimum DCP Blows for 12 in. Penetration
Non-Granular Soils				
Clay Soils	< 105	19 - 24	6	—
	105 - 110	16 - 18	7	
	111 - 114	14 - 15	8	
Silty soils	115 - 116	13 - 14	—	9
	117 - 120			11
Sandy soils	121 - 125	8 - 12	—	12
	> 125			15
Granular Soils A-1, A-2, and A-3 Soils (with 100% Passing)				
No. 30 sieve	N/A			6
No. 4 sieve				7
½ in. sieve				11
1 in. sieve				16

Source: Indiana DOT 2015b

The DCP criteria are provided based on the allowable number of DCP blows to 6 in. penetration for clay soils and to 12 in. penetration for sandy and silty clay soils and granular soils. The maximum dry density and optimum moisture content are determined following a graphical procedure based on the one-point Proctor test for non-granular soils (Indiana DOT 2015b). Indiana DOT specifications also allow using LWD testing for QA, but target limits are not provided in the specifications.

CHAPTER 3. METHODS

The research team performed field tests at embankment construction sites and conducted laboratory tests of embankment fill materials obtained from those sites.

Field Testing Methods

DCP and in situ drive cylinder tests were conducted to assess newly constructed embankment compaction properties. A GPS was used to record the location of test points in each test section.

Drive Cylinder

Drive cylinder tests were conducted in accordance with ASTM D2937-10 (2010). A thin-wall, 4.0 in. diameter cylinder was driven into a compacted lift with a driving head to obtain relatively undisturbed samples. The cylinders then were carefully excavated (Figure 8), placed in a zip-sealed bag, and transported to the laboratory in a humid cooler for laboratory testing.

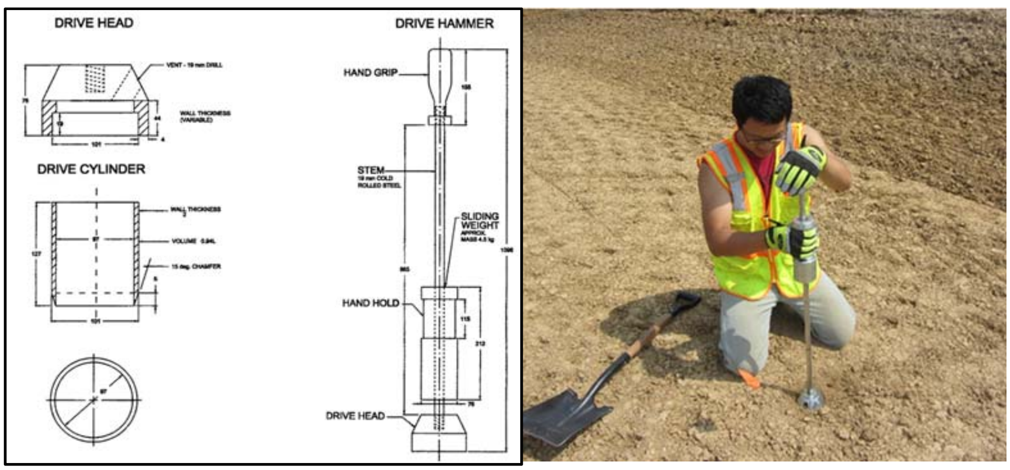


Figure 8. Schematic of drive cylinder (left) and ISU researcher performing in situ testing (right)

The samples then were processed in the laboratory to measure the wet unit weight, and a sample was obtained to determine moisture content in accordance with ASTM D2216-10 (2010).

Dynamic Cone Penetrometer (DCP)

DCP testing was conducted in accordance with ASTM D6951-09 (2015). The DCP tip was driven into soil by lifting the 17.6 lb sliding hammer up to the handle and then releasing it (Figure 9).

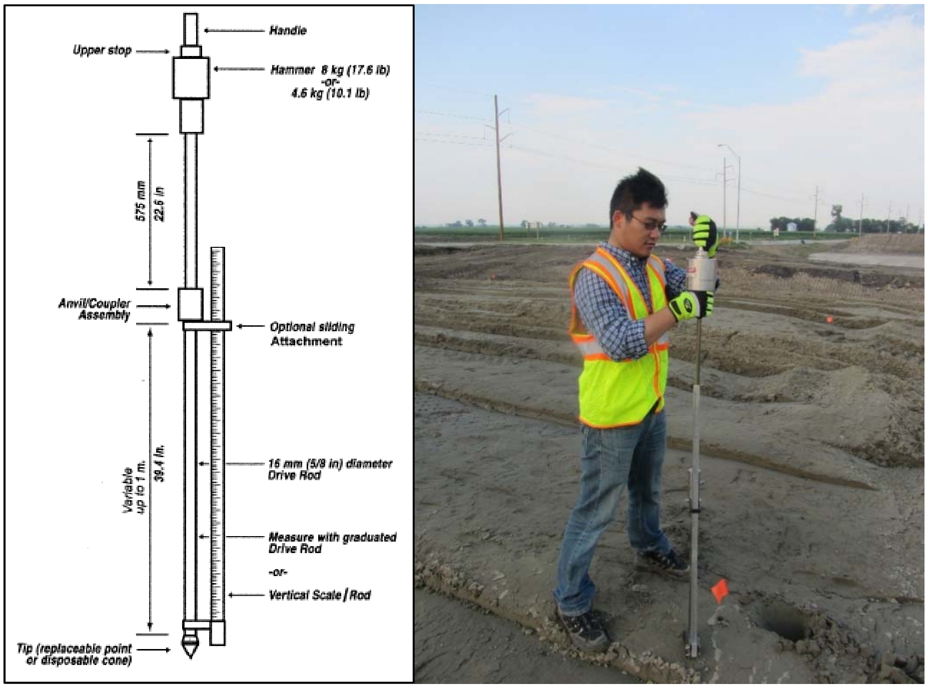


Figure 9. Schematic of DCP device (left) and ISU research team performing in situ testing (right)

The total penetration for a given number of blows was measured and recorded in mm/blow, which is referred to as DCP penetration index (DPI) and is used to estimate in situ CBR from the following equations:

For CH soils $$CBR = \frac{1}{0.002871\ (\mathrm{DPI})} \quad (1)$$

For CL soils and CBR<10 $$CBR = \frac{1}{(0.017019\ \ \mathrm{DPI})^2} \quad (2)$$

For all other soils $$CBR = \frac{292}{(\mathrm{DPI})^{1.12}} \quad (3)$$

A chart of CBR versus depth and cumulative blows versus depth was plotted for each test bed. The plots presented the change in CBR with increasing depth and the change in cumulative blows with increasing depth. The charts were visually designed to indicate the stiffness of the compacted fills, with higher CBR values indicating higher stiffness. Depths of 8 in. and 12 in. were selected to present the performance of compaction. The cumulative blows at 8 in. and 12 in.

were obtained from this chart, and then corresponding DPI and CBR values were calculated according to Equations 1 through 3, whichever is appropriate (Figure 10).

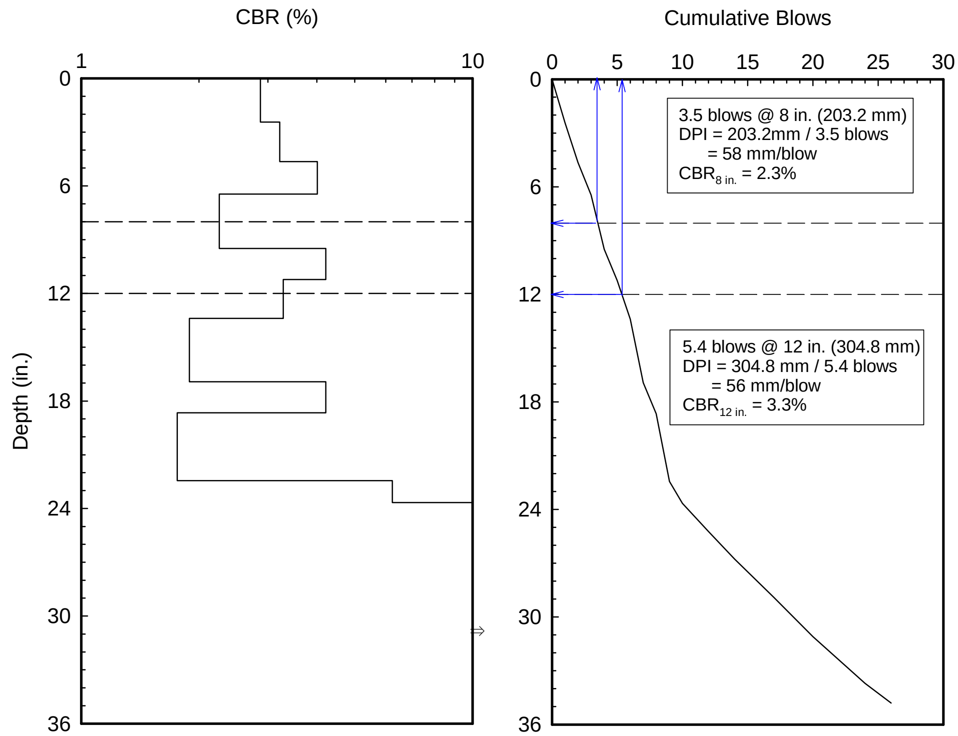


Figure 10. Example DCP-CBR values and cumulative blows with depth plots and interpretation of average values for 8 in. and 12 in. depths

A flow chart of DCP data collection and analysis is shown in Figure 11.

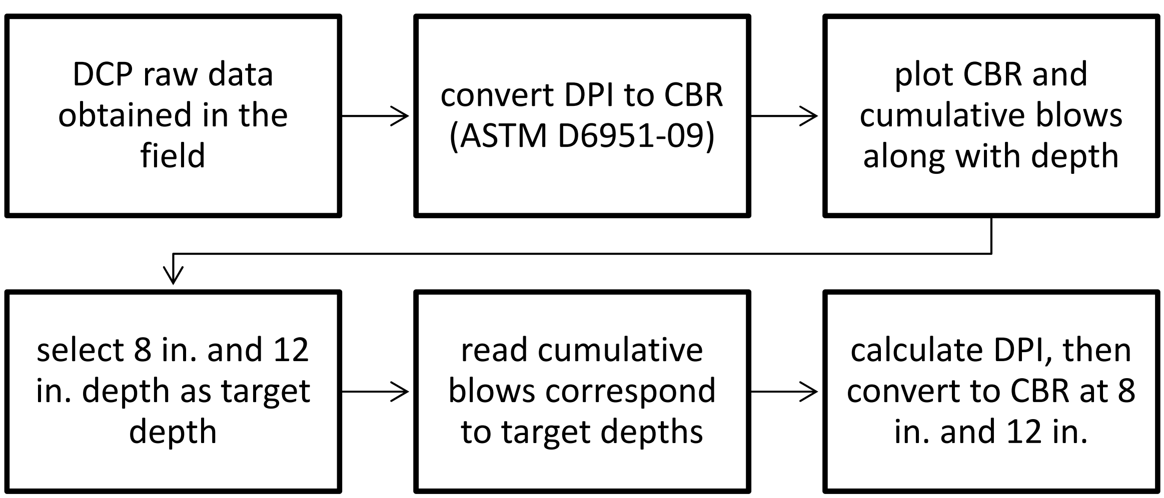


Figure 11. Flow chart used for collecting and analyzing DCP data

To evaluate the uniformity of the compacted fill, the weighted average and variation of the DCP index values were determined in accordance with the following equations (White et al. 2007):

$$\textit{DCP index (for a test layer of thickness H)} = \frac{1}{H}\sum_{i=1}^{n} d_i^2 \qquad \textbf{(4)}$$

$$\textit{Average variation in DCP index} = \frac{1}{H}\sum_{i=2}^{n} |d_i - d_{i-1}| d_{i-1} \qquad \textbf{(5)}$$

where, n = total number of blows, d_i = penetration distance for the ith blow, and H = depth of the test layer.

The average DCP index value and the variation in the DCP index values were compared with the maximum values recommended by White et al. (2007), as summarized in Table 5.

Table 5. DCP index target values

Soil Classification		Average DCP Index (mm/blow)	Variation in DCP Index (mm/blow)
Cohesive	Select	65	35
	Suitable	70	40
	Unsuitable	70	40
Granular	Select	35	35
	Suitable	45	45

Source: White et al. 2007

The CBR values calculated from these data were also compared with the relative ratings presented in Chapter 6 of the Iowa Statewide Urban Design and Specifications (SUDAS) Design Manual (Table 6).

Table 6. CBR values for subgrade soils

CBR (%)	Material	Rating
20 to 30	Subgrade	Very good
10 to 20	Subgrade	Fair-good
5 to 10	Subgrade	Poor-fair
< 5	Subgrade	Very poor

Source: SUDAS 2013

Global Positioning System (GPS)

To locate the in situ testing points at each construction project, a Trimble R8 Model 3 GPS device was used to obtain real-time kinematic (RTK) GPS measurements by connecting to Iowa real-time network stations (Figure 12).

Figure 12. Location information measured by GPS device

Sampling

The ISU research team met with the project's resident construction engineer (RCE) or the Iowa DOT field engineer and/or the contractor foreman to discuss which areas had passed QA with approximate starting and end stations. Depending on the size of the area that was passed, up to 15 locations that were uniformly spaced in a systematic pattern through the middle of the test area were selected for moisture and density testing. Two examples of sampling patterns are shown in Figure 13.

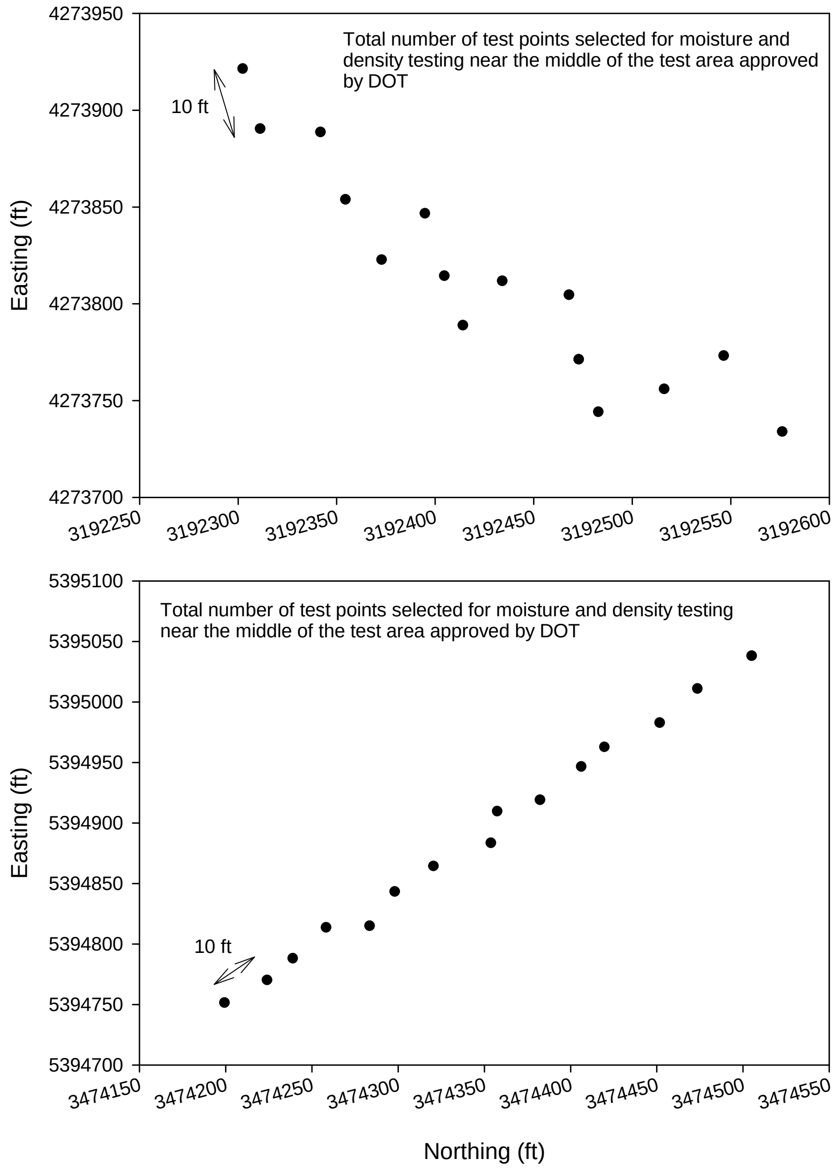


Figure 13. Two patterns of in situ testing point selection: Pottawattamie County project (top) and Linn County 77 project (bottom)

DCP tests were typically only performed at every third test point (i.e., DCP tests were performed only at 5 locations if there were 15 total test locations).

Intelligent Compaction RICM

The use of machine drive power (MDP) technology as a measure of soil compaction is a concept originated from the study of vehicle-terrain interaction (Bekker 1969). The advantage of this technology is that measurements are output to a computer screen in the cab of the roller in real time to allow the operator to identify areas of poor compaction and make necessary rolling pattern changes (White et al .2005).

MDP uses the concepts of rolling sinkage and resistance to determine the required energy consumption to overcome the resistance to motion (White and Thompson 2008). A sensor is installed on the roller to monitor hydraulic pressure and flow at torque converters of the roller. MDP is calculated as

$$\text{MDP} = \text{Pg} - \text{Wv}(\sin\alpha + \frac{A'}{g}) - (\text{mv} + \text{b}) \qquad \textbf{(6)}$$

where

MDP = machine drive power (kJ/s),

Pg = gross power needed to move the machine (kJ/s),

W = roller weight (kN),

A' = machine acceleration (m/s^2),

g = acceleration of gravity (m/s^2),

α = slope angle (roller pitch from a sensor),

v = roller velocity (m/s), and

m (kJ/m) and b (kJ/s) = machine internal loss coefficients specific to a particular machine (White et al. 2005).

In this study, MDP is a relative value relating to the material properties of the calibration surface, which is a hard compacted surface and MDP is equal to 0 kJ/s. Thus compacted materials having positive MDP values indicate that they are less compacted than the calibration surface, and the compacted materials having negative MDP values indicate that they are more compacted than the calibration surface. The MDP values obtained from the machine were recalculated to range from 1 to 150 using Eq. 7 (referred as MDP_{40}). The calibration surface with MDP = 0 kJ/s was scaled to MDP_{40} = 150 and a soft surface with MDP = 54.23 kJ/s was scaled to MDP_{40} = 1.

$$\text{MDP}_{40} = 150 - 2.75\,(\text{MDP}) \qquad \textbf{(7)}$$

Laboratory Testing

Representative soil materials were collected from each construction site and used for conducting the following laboratory tests:

Soil Index Properties

Particle size analysis was conducted in accordance with ASTM D422-63 (2010). The distribution of particle sizes larger than 75 μm (opening size of the No. 200 sieve) was determined by sieving, and the distribution of particle sizes smaller than 75 μm was determined by the hydrometer method. Atterberg limit testing was conducted in accordance with ASTM D4318-10 (2010) using the wet preparation method. Liquid limit tests were performed using the multipoint method (Figure 14).

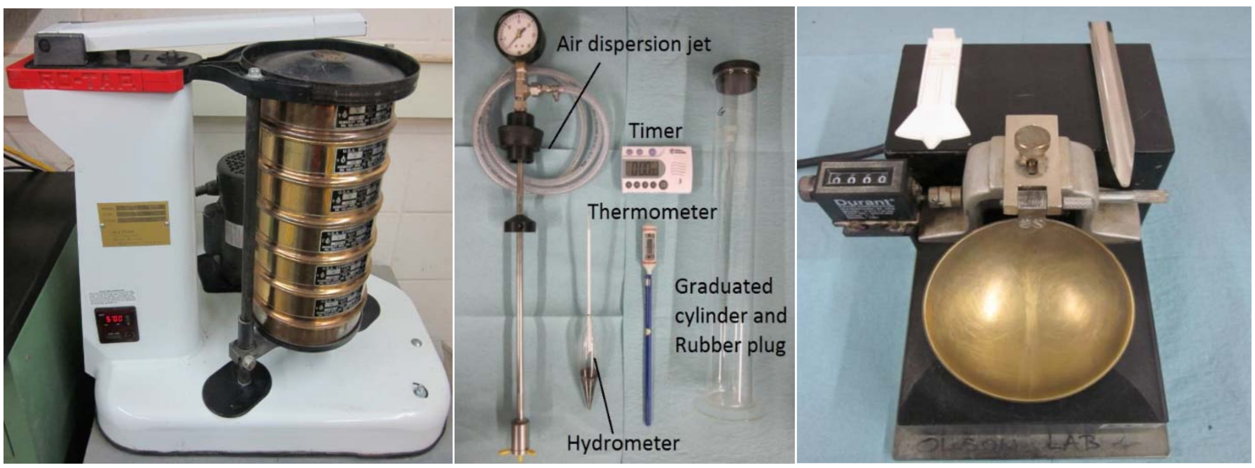


Figure 14. Soil classification equipment (left to right: sieve analysis, hydrometer test, and Atterberg limit test)

Based on these results, each sample was classified according to the Unified Soil Classification System (USCS) and AASHTO M 145 (AASHTO 2012) Soil Classification System. The specific gravity of each sample was determined in accordance with ASTM D854-14 (2014) Method A.

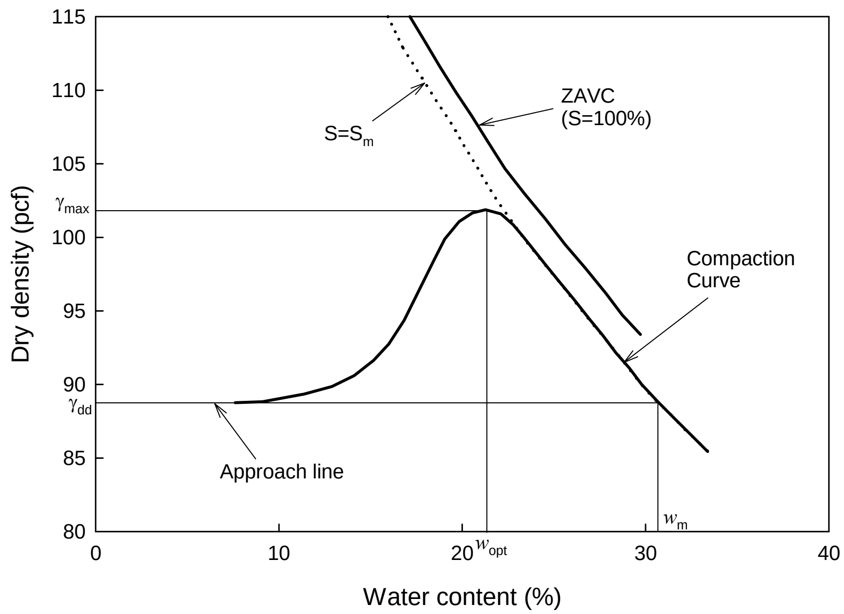
Compaction Characteristics

The relationship between the moisture and dry unit weight of embankment materials was determined in accordance with ASTM D698-12e2 (2012) and ASTM D1557-12e1 (2012). The appropriate method was chosen based on the grain size distributions for each sample. Method A was applicable for all soil materials. The tests were performed at five moisture contents, and the optimum moisture-density characteristics were obtained by fitting the data to the Li and Sego Fit model (Equation 5):

$$\gamma_d\ (w) = \frac{G_s\, \gamma_w}{\left(1 + \frac{w\, G_s}{S_m - S_m \left(\frac{w_m - w}{w_m}\right)^{n+1} \left(\frac{w_m^n + p^n}{(w_m - w) + p^n}\right)}\right)} \tag{8}$$

where, γ_d = dry density of the soil, G_s = specific gravity of the soil, γ_w = density of water, w = moisture content of the soil, S_m = maximum degree of saturation, w_m = moisture content at S_m, and n and p are shape factors.

Figure 15 shows the fit model, the relationship, and the relevant parameters.



Reproduced from Li and Sego 2000

Figure 15. Density curve

The boundary condition on the wet side of optimum, S_m, can be determined from the wet side of the compaction curve running parallel to the zero air void curve. The boundary condition on the dry side of w_{opt} is the dry density (γ_{dd}). The shape factor n affects the dome portion of the compaction curve. When n is increased, the dome portion becomes sharper; when n is decreased, the dome portion tends to flatten. Shape factor p influences the width of the upper portion of the curve without affecting shape factor n or boundary conditions S_m and γ_{dd}. To make a correct fit, S_m and w_m were first determined based on the data to establish the boundary of the curve, and shape factors n and p were adjusted until a maximum correlation coefficient (R^2) between the measured and the predicted values was achieved.

ISU 2 in. by 2 in. Compaction

ISU 2 in. by 2 in. compaction apparatus is described in O'Flaherty et al. (1963). The test procedure was used to prepare 2 in. diameter by 2 in. height (2 x 2) samples for UCS testing (Figure 16).

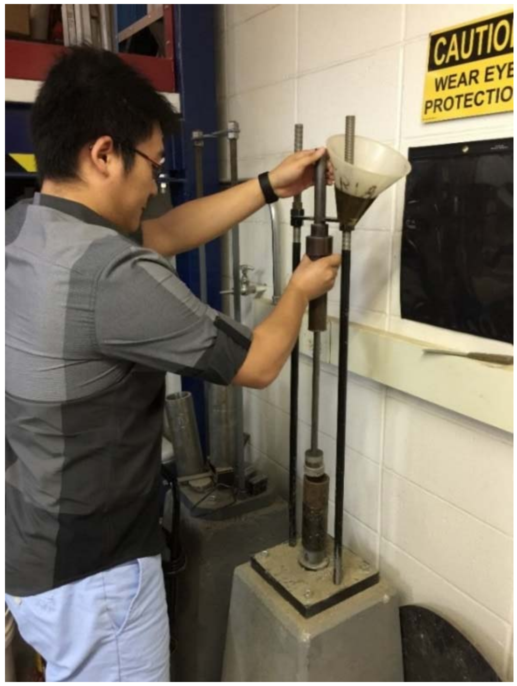


Figure 16. ISU 2 in. by 2 in. specimen compaction

Samples were compacted at their respective standard Proctor optimum moisture content. For cement treated materials, the optimum moisture content was determined using Eq. 3 with a water to cement (w/c) ratio of 0.25:

$$w_{\text{opt soil + cement}} = [(\%\text{ cement added by weight})\text{ x }(\text{w/c ratio})] + w_{\text{opt soil}} \quad \textbf{(9)}$$

The test procedure involved placing loose material in the compaction apparatus and dropping a 5 lb. hammer from a drop height of about 12 in. in a 2 in. diameter steel mold. O'Flaherty et al. (1963) provided guidance on the number of blows required to obtain standard Proctor densities for different soil types, as summarized in Table 2. The number of blows were selected based on the soil type and equal number of blows were applied on both sides of the sample, to compact the sample uniformly.

Table 7. Number of drop-hammer blows (O'Flaherty et al. 1963)

AASHTO Soil Type	Total number of drop-hammer blows
A-7 and A-6	6
A-4	7
A-3, A-2, and A-1	14

After compaction, the 2 x 2 specimens were sealed using plastic wrap and aluminum foil, and were placed in sealed plastic bag. Cement stabilized specimens were cured for 7 days at 110^{o}F, to simulate 28 day curing strength (Winterkkorn and Pamukcu 1990). Unstabilized specimens were tested shortly after compaction (no curing). Three samples were prepared at each cement content.

Unconfined Compressive Strength (UCS)

The cured specimens were tested for UCS (Figure 17) in general accordance with ASTM D 1633-00 (ASTM 2007). The standard requires use of either 4 in. diameter by 4.584 in. height Proctor samples with a height to diameter (h/d) ratio of 1.15 or or 2.8 in. diameter by 5.6 in. height samples with a h/d ratio of 2.0. Instead, 2 x 2 specimens were used in this study which have a h/d ratio of 1.0. Based on laboratory evaluations, White et al. (2005a) concluded that the UCS determined from 2 x 2 specimens can be multiplied by 0.86 to correlate with UCS of Proctor sized samples (h/d = 1.15) or 0.90 to correlate with samples that have h/d = 2. ASTM D1633-00 also provides a similar guidance in relating UCS on samples with h/d=1.15 to samples with h/d=2 as follows: "*If desired, make allowance for the ratio of height to diameter (h/d) by multiplying the compressive strength of Method B specimens* [with h/d = 2.0] *by factor 1.10. This converts the strength for an h/d ratio of 2.00 to that for the h/d ratio of 1.15 commonly used in routine testing of soil-cement.*"

Figure 17. Specimen failure after measurement of UCS

The cured specimens were tested in unsaturated and saturated condition. The specimens were saturated using the vacuum saturated method as described in ASTM C593-06 (ASTM 2011a). The specimens were placed on a perforated Plexiglas plate in a vacuum vessel (Figure 18), and the chamber was evacuated using 24 in. of mercury for 30 minutes. Then the vacuum vessel was flooded to a depth sufficient to cover the soil specimens. After one hour of soaking, the specimens were removed from the vessel to conduct UCS testing. For samples that become fragile and cannot be removed from water for UCS testing, the UCS is reported as 0 psi.

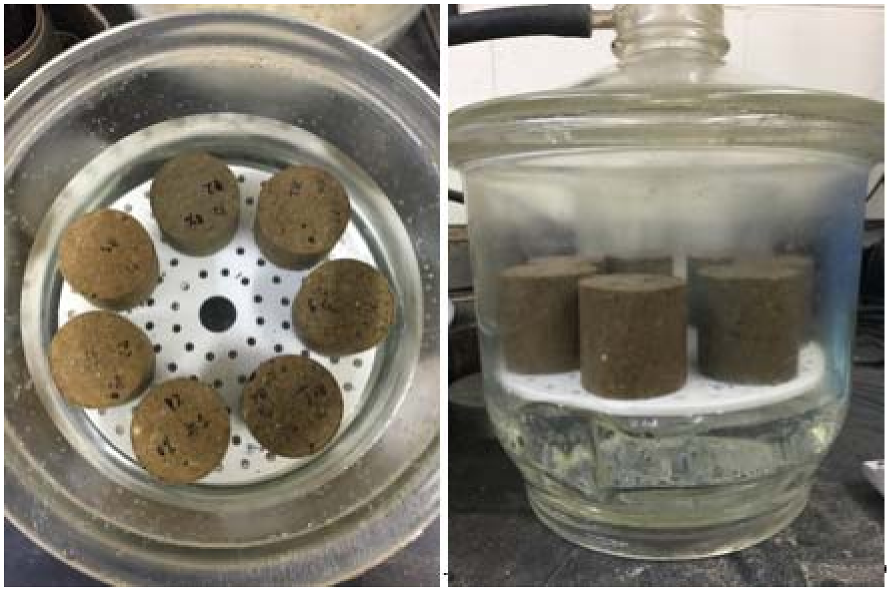


Figure 18. Vacuum saturation of cement stabilized specimens

One-dimensional Consolidation Properties

One-dimensional consolidation tests were conducted on samples trimmed from drive core cylinders (Figure 19). When specimens were obtained from the field, they were subjected to overburden pressure, which can be calculated by filling materials wet density multiply material filling thickness. Overburden pressures during the process of specimen trimming, the pressure was released. To eliminate the effect of released overburden pressure, loading, unloading, and reloading stages were applied to each specimen (Figure 20). When the applied loading pressure reached the overburden pressure, unloading stage started, and then reloading stage started. The time-deformation readings were collected in accordance with ASTM D2435-11 (ASTM 2011a). Successive load increments were applied after 100% primary consolidation was reached. The void ratio versus applied pressure curve was plotted, and the coefficient of consolidation (c_v), compression index (c_c) and swelling index (c_s) were calculated. Double sided drainage was applied during the consolidation testing process. An example of consolidation test results showing applied stress versus void ratio values for load, unload, re-load, and unload steps are shown in Figure 20.

Figure 19. One-dimensional consolidation testing equipment and specimen

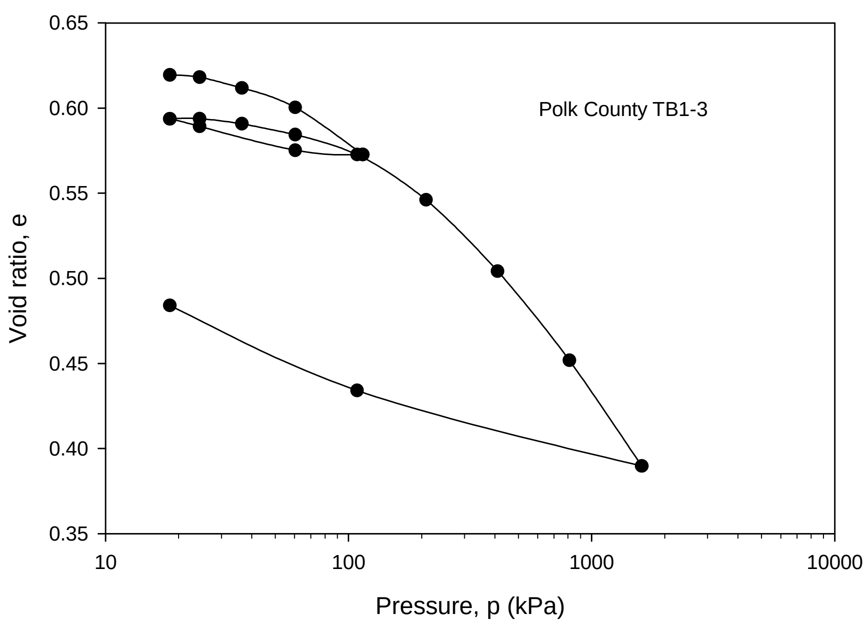


Figure 20. Example of consolidation test results

The c_v, c_c, and c_s were calculated as follows:

$$c_v = \frac{T H_{D_{50}}^2}{t} \qquad \textbf{(10)}$$

where,

c_v = coefficient of consolidation;

T_{50} = a dimensionless time factor; and

$H_{D_{50}}$ = half of the specimen height.

An example for c_v calculation follows,

The 14th loading stage (410.4 kPa) was selected to calculate the c_v. According to the time-deformation curve, the deformation dial reading at 0% consolidation and deformation dial reading at 100% consolidation were recorded as 1.4 mm and 1.75 mm, respectively. Then the half-thickness of specimen at 50% consolidation was calculated as 9.3 mm. The time for 50% consolidation was recorded as 7 minutes in accordance with the time-deformation curve. The c_v can be calculated by Eq. 10 as 3.74 x 10^{-3} in^2/min.

$$c_c = \frac{\Delta e}{\Delta log\sigma}, \; c_s = \frac{\Delta e}{\Delta log\sigma} \quad \textbf{(11)}$$

where,

c_c = compression index;

c_s = swelling index;

Δe = variation of void ratio; and

$\Delta log\sigma$ = variation of pressure.

The overconsolidation ratio (OCR) for a soil can be defined as:

$$OCR = \frac{\sigma_c'}{\sigma'} \quad \textbf{(12)}$$

where,

σ_c' = preconsolidation pressure of a specimen; and

σ' = present effective vertical pressure.

For normally consolidated (OCR = 1) soil, the primary consolidation settlement is calculated as:

$$S_c = \frac{C_c H}{1+e_0}\log\left(\frac{\sigma_0'+\Delta\sigma'}{\sigma_0'}\right) \quad \textbf{(13)}$$

For overconsolidated (OCR>1) soil, the primary consolidation settlement is calculated as:

If $\sigma_0'+\Delta\sigma'\leq\sigma_c'$, $S_c = \frac{C_s H}{1+e_0}\log\left(\frac{\sigma_0'+\Delta\sigma'}{\sigma_0'}\right)$ **(14)**

If σ_0'+$\Delta\sigma$'≥σ_c', $S_c = \frac{C_s\,H}{1+e_0}\log\left(\frac{\sigma_c'}{\sigma_0'}\right) + \frac{C_c\,H}{1+e_0}\log(\frac{\sigma_0'+\Delta\sigma'}{\sigma_c'})$ **(15)**

where,

S_c = settlement of primary consolidation;

c_s = swelling index;

c_c = compression index;

H = soil layer thickness;

e_0 = initial void ratio;

σ_0' = initial effective overburden pressure;

$\Delta\sigma$' = change of effective overburden pressure; and

σ_c' = preconsolidation pressure of a specimen (Das 2010).

Statistical Analysis Methods

t-test

To compare the differences between the field results obtained from the previous project phases and the field results obtained from the current project, a *t*-test analysis was performed. The main objective of this analysis was to assess whether there is a statistically significant difference in the number or percentage of test locations that did not meet the moisture and density control limits. A *t*-test analysis was performed for unequal sample size and unequal variances between the different project phase results. The test was set up with a research hypothesis that the mean values of the measurements obtained in one project (μ_0) were higher than those obtained in another project (μ_1).

The approximate *t*-value (represented as t') was calculated using the following equation (Ott and Longnecker 2008):

$$t' = \frac{\mu_0 - \mu_1}{\sqrt{\frac{s_0^2}{n_0} + \frac{s_1^2}{n_1}}} \tag{16}$$

where, n_0 and n_1 = number of measurements from two different projects, μ_0 and μ_1 = mean values of measurements from two different projects, and s_0 and s_1 = standard deviation of measurements from two different projects. The observed t'-values were then compared with the minimum t'-values for a one-tailed test, with the degrees of freedom (DOF) calculated using Equations (10) and (11), at a 95% confidence level (i.e., α = 0.05):

$$DOF = \frac{(n_0-1)(n_1-1)}{(1-c)^2(n_0-1)+c^2(n_1-1)} \tag{17}$$

where,

$$c = \frac{s_0^2/n_0}{\frac{s_0^2}{n_0}+\frac{s_1^2}{n_1}} \tag{18}$$

If the observed *t*-values were higher than the minimum *t'*-values, then it was concluded that there is sufficient evidence that the mean values of each project were different.

Logistic Regression

In this project, a logistic regression model (Ott and Longnecker 2008, Hosmor and Lemeshow 2005) is used to present the difference between two given categories, or two treatments. This objective of the logistic regression is to fit the data with the logistic curve, which is also known as the sigmoid curve,

$$p = \frac{1}{1+e^{-(\beta_0+\beta_1 x)}} \tag{19}$$

Or the linearized form,

$$ln\left(\frac{p}{1-p}\right) = \beta_0 + \beta_1 x \tag{20}$$

In order to judge how likely an event is to happen, an effective way is to calculate its probability. The reason to use the Logistic regression is that for each independent variable *x*, it calculates a probability *p*.

In this project, we need to define some reference variables to digitalize the data in order to use the Logistic regression model. For example, if we want to compare the RC (%) between embankment phase I and TR677, we can use x=1 to represent phase I and x=0 to represent TR677. The measurement for RC (%) can be either RC≥95% or not. We can use variable y=1 to represent the occurrence of RC≥95%, and use y=0 when RC<95%. Thus, we finish the digitalization of the data set, and the logistic model calculates the probability p, when y=1 for given x. For instance, given the embankment phase I, i.e., x=1, and the probability of y=1 means the probability that the data from embankment phase I is within the specification. We write the probability as p=[y=1|x=1], and the logistic regression's result is

$$p = p[y = 1|x = 1] = \frac{1}{1+e^{-(\beta_0+\beta_1 x)}} \quad \textbf{(21)}$$

In the example above, the two embankment phases are compared with the Logistic regression model. However, the Logistic regression model can be generalized for multiple embankment treatments with different digitalization for phases. For instance, if we want to compare all the five phases, we can digitalize the phases as

Table 8. Digitalization of All the Projects

Project	Digitalization
Phase I	x=(1 0 0 0)
Phase II	x=(0 1 0 0)
Phase III	x=(0 0 1 0)
Phase IV	x=(0 0 0 1)
TR677	x=(0 0 0 0)

The 0's and 1's do not have physical meanings, but they are used to identify different projects in the model. Then the logistic model can be expressed as

$$p = p[y = 1|x] = \frac{1}{1+e^{-(\beta_0+\beta_1^T x)}} \quad \textbf{(22)}$$

Where x can be taken from the table above.

In order to tell the difference between each project, β_1 should be different from 0 statistically, otherwise the model will return the same probability value, p, for all the projects. A chi-square test is used to test if β_1 is different from 0 significantly. The mechanism of the chi-square test is to compare the likelihoods of two competing models. In this study the two competing models are (a): a model where both have the same percentage, i.e., $\beta_1 = 0$ and (b): a model where each group is allowed to have its own percentage, i.e., $\beta_1 \neq 0$. The null hypothesis is that all the projects has the same probability value, p, for y=1; and the alternative hypothesis is that at least two projects have different probability values.

The test statistic then is calculated as:

$$D = -2\, ln \left[\frac{likelihood\ of\ model\ a}{likelihood\ of\ model\ b}\right] \quad (23)$$

The *D* values are then compared to chi-square distribution with the number of degree of freedom equal to the number of parameters in the model b minus the number of parameters in model a. In this study model a is estimating a single overall mean, so there is one parameter, while model b is estimating a mean for each group so there are 5 parameters. Thus the above would get compared to a chi-square distribution with 4 degrees of freedom. A small *p*-value indicates the null hypothesis was rejected and conclude that the probability, p, for y=1 are different between at least two of the projects.

The difference between two projects, I and II, is

$$ln\left(\frac{p_I}{1-p_I}\right) - ln\left(\frac{p_{II}}{1-p_{II}}\right) = ln\left(\frac{\frac{p_I}{1-p_I}}{\frac{p_{II}}{1-p_{II}}}\right) \qquad \textbf{(24)}$$

Which is referred to as the odds ratio. The table of odds ratio estimations presented in results section are the exponential values from Eq. (19), and the exponential function changes the scale of the probability, p, from log-scale to normal scale.

CHAPTER 4. MATERIALS

The embankment materials consisted of cohesive soils at eight project sites and cohesionless granular soils at one project site. Cohesive materials were collected from 25 test beds, and 6 were classified as select, 18 were classified as suitable, and 1 was classified as unsuitable per Iowa DOT Standard Specifications Section 2102: Soil Classification (Iowa DOT 2015). Granular soils collected from three test beds were classified as suitable per the same specification.

The parent materials of the cohesive soils were glacial till and loess. The parent material for the granular soils was alluvium material from the Missouri River floodplain. Manufactured materials were used at one project site. Table 9 through Table 14 summarize the parent materials, particle size analyses, Atterberg limits, specific gravities, soil classifications, and Proctor compaction test results for each project location. The grain size distribution curves of the embankment fill materials obtained from each project location are shown in Appendix C.

For cement stabilization, type I/II Portland cement was used in this study.

Table 9. Soil index properties of embankment materials obtained from Polk County

Parameter	Polk County TB1	Polk County TB2	Polk County TB3	Polk County TB4
	5/29/2014	6/7/2014	8/5/2014	8/19/2014
Parent Material	Glacial till	Glacial till	Glacial till	Glacial till
Gravel content (%) (> 4.75 mm)	0.4	3.9	2.6	1.8
Sand content (%) (4.75 mm – 75 µm)	11.6	25.8	28.7	24.6
Silt content (%) (75 µm – 2 µm)	66.4	34.7	45.8	50.9
Clay content (%) (< 2 µm)	21.6	35.6	22.9	22.7
Liquid limit, LL (%)	49	45	36	34
Plastic limit, PL (%)	28	34	20	17
Plastic Index, PI (%)	21	11	16	17
AASHTO classification	A-7-6(21)	A-7-5(8)	A-6(9)	A-6(11)
USCS classification	CL	CL	CL	CL
USCS Description	Lean Clay	Lean clay with sand	Sandy lean clay	Lean clay with sand
Iowa DOT Material Classification	Suitable	Suitable	Suitable	Suitable
Soil Color	Olive Brown	Olive Brown	Very dark greyish brown	Olive Brown
Specific Gravity, G_s	2.673	2.679	2.670	2.672
Std. Proctor, w_{opt} (%)	19.6	20.0	16.0	16.0
Std. Proctor, γ_{dmax} (lb/ft^3)	103.9	104.0	110.6	110.6
Mod. Proctor, w_{opt} (%)	16.0	13.6	11.5	11.5
Mod. Proctor, γ_{dmax} (lb/ft^3)	112.3	120.0	122.0	123.0

Table 10. Soil index properties of embankment materials obtained from Warren County and Linn County 79

	Warren County TB1	**Warren County TB2**	**Warren County TB3 (Grey)**	**Warren TB3 County (Brown)**	**Linn County-79**
Parameter	6/3/2014	7/22/2014	8/4/2014	8/4/2014	6/6/2014
Parent Material	Glacial till	Glacial till	Glacial till	Glacial till	weathered loess
Gravel content (%) (> 4.75 mm)	2.0	5.0	0.7	0.6	0.7
Sand content (%) (4.75 mm – 75 µm)	27.5	31.6	18.7	29.2	46.0
Silt content (%) (75 µm – 2 µm)	37.3	31.9	39.1	33.7	26.4
Clay content (%) (< 2 µm)	33.2	31.5	41.5	36.5	26.9
Liquid limit, LL (%)	44	40	54	40	31
Plastic limit, PL (%)	31	19	20	20	25
Plastic Index, PI (%)	13	21	34	20	6
AASHTO classification	A-7-5(9)	A-6(11)	A-7-6(28)	A-6(13)	A-4(1)
USCS classification	CL	CL	CH	CL	CL-ML
USCS Description	Lean clay with sand	Sandy lean clay	Fat clay with sand	Sandy lean clay	Sandy silty clay
Iowa DOT Material Classification	Suitable	Select	Unsuitable	Suitable	Suitable
Soil Color	Olive Brown	Light olive Brown	Very dark grey	Olive Brown	Olive Brown
Specific Gravity, G_s	2.676	2.673	2.715	2.674	2.684
Std. Proctor, w_{opt} (%)	16.5	15.8	21.0	17.0	13.5
Std. Proctor, γ_{dmax} (lb/ft^3)	111.1	113.8	102.0	109.5	117.4
Mod. Proctor, w_{opt} (%)	11.0	9.8	13.6	10.5	9.0
Mod. Proctor, γ_{dmax} (lb/ft^3)	123.9	128.5	115.5	125.0	130.8

Table 11. Soil index properties of embankment materials obtained from Linn County 77

Parameter	Linn County-77 TB1	Linn County-77 TB2	Linn County-77 TB3	Linn County-77 TB4	Linn County-77 TB5
	6/6/2014	7/8/2014	7/15/2014	8/1/2014	9/8/2014
Parent Material	Glacial till	Glacial till	Glacial till	Glacial till	Glacial till
Gravel content (%) (> 4.75 mm)	1.8	1.3	11.3	1.1	2.0
Sand content (%) (4.75 mm – 75 μm)	37.6	42.6	36.1	39.9	40.3
Silt content (%) (75 μm – 2 μm)	32.9	30.9	31.2	35.6	34.8
Clay content (%) (< 2 μm)	27.7	25.2	21.4	23.4	22.9
Liquid limit, LL (%)	31	34	33	32	30
Plastic limit, PL (%)	12	16	11	16	16
Plastic Index, PI (%)	19	18	22	16	14
AASHTO classification	A-6(8)	A-6(7)	A-6(7)	A-6(6)	A-6(5)
USCS classification	CL	CL	CL	CL	CL
USCS Description	Sandy lean clay	Sandy lean clay	Sandy lean clay	Sandy lean clay	Sandy lean clay
Iowa DOT Material Classification	Select	Select	Select	Select	Select
Soil Color	Very dark grey	Olive Brown	Very dark grey	Very dark grey	Very dark grey
Specific Gravity, G_s	2.683	2.670	2.673	2.672	2.674
Std. Proctor, w_{opt} (%)	12.9	13.0	12.0	11.7	12.6
Std. Proctor, γ_{dmax} (lb/ft^3)	118.4	116.0	119.5	119.5	119.0
Mod. Proctor, w_{opt} (%)	8.8	9.0	8.0	8.1	8.6
Mod. Proctor, γ_{dmax} (lb/ft^3)	130.8	129.5	131.0	132.1	130.0

Table 12. Soil index properties of embankment materials obtained from Pottawattamie County and Woodbury County I-29

Parameter	Pottawattamie County TB1	Pottawattamie County TB2	Woodbury County I-29 TB1	Woodbury County I-29 TB2	Woodbury County I-29 TB3
	7/2/2014	7/10/2014	7/9/2014	7/10/2014	8/7/2014
Parent Material	Manufactured materials	Manufactured materials	Alluvium	Alluvium	Alluvium
Gravel content (%) (> 4.75 mm)	7.3	5.3	0.2	0.0	1.7
Sand content (%) (4.75 mm – 75 μm)	10.1	25.5	78.4	83.2	81.1
Silt content (%) (75 μm – 2 μm)	56.2	48.0	15.5	12.6	11.6
Clay content (%) (< 2 μm)	26.4	21.2	5.9	4.2	5.6
Liquid limit, LL (%)	43	42	NP	NP	NP
Plastic limit, PL (%)	18	19	NP	NP	NP
Plastic Index, PI (%)	25	23	NP	NP	NP
AASHTO classification	A-7-6(20)	A-7-6(14)	A-2-4	A-2-4	A-2-4
USCS classification	CL	CL	SM	SM	SM
USCS Description	Lean clay with sand	Sandy lean clay	Silty sand	Silty sand	Silty sand
Iowa DOT Material Classification	Suitable	Suitable	Suitable	Suitable	Suitable
Soil Color	Dark brown	Very dark greyish brown	Olive Brown	Very dark greyish brown	Very dark greyish brown
Specific Gravity, G_s	2.697	2.709	2.657	2.654	2.654
Std. Proctor, w_{opt} (%)	17.5	17.5	17.5	15.5	15.0
Std. Proctor, γ_{dmax} (lb/ft^3)	106.0	106.3	102.5	102.8	104.5
Mod. Proctor, w_{opt} (%)	13.5	12.8	15.5	14.5	13.0
Mod. Proctor, γ_{dmax} (lb/ft^3)	117.5	117.5	109.2	105.0	110.0

Table 13. Soil index properties of embankment materials obtained from Scott County and Mills County

Parameter	Scott County TB1	Scott County TB2	Scott County TB3	Mills County TB1	Mills County TB2
	7/16/2014	7/31/2014	9/19/2014	6/26/2014	6/26/2014
Parent Material	Loess	Loess	Loess	Loess	Loess
Gravel content (%) (> 4.75 mm)	0.1	1.0	2.0	0.1	3.9
Sand content (%) (4.75 mm – 75 µm)	1.0	24.3	29.2	3.1	6.4
Silt content (%) (75 µm – 2 µm)	72.9	45.5	45.9	70.6	34.9
Clay content (%) (< 2 µm)	26.0	29.2	22.9	26.2	54.8
Liquid limit, LL (%)	39	35	28	38	36
Plastic limit, PL (%)	32	24	17	34	31
Plastic Index, PI (%)	7	11	11	4	5
AASHTO classification	A-4(10)	A-6(8)	A-6(5)	A-4(7)	A-4(6)
USCS classification	CL-ML	CL	CL	CL-ML	CL-ML
USCS Description	Silty Clay	Lean clay with sand	Sandy lean clay	Silty clay	Silty clay
Iowa DOT Material Classification	Suitable	Suitable	Suitable	Suitable	Suitable
Soil Color	Dark olive brown	Dark yellowish brown	Olive Brown	Dark yellow brown	Brown
Specific Gravity, G_s	2.680	2.672	2.673	2.725	2.726
Std. Proctor, w_{opt} (%)	16.5	15.5	13.0	17.0	16.0
Std. Proctor, γ_{dmax} (lb/ft^3)	108.0	111.1	119.5	108.5	110.8
Mod. Proctor, w_{opt} (%)	13.0	11.2	9.2	13.0	12.0
Mod. Proctor, γ_{dmax} (lb/ft^3)	118.0	122.5	131.0	117.2	119.5

Table 14. Soil index properties of embankment materials obtained from Woodbury County US 20

Parameter	Woodbury County (US20) TB1 9/26/2014	Woodbury County (US20) TB2 9/26/2014	Woodbury County (US20) TB3 10/18/2014	Woodbury County (US20) TB4 10/18/2014
Parent Material	very deep loess	very deep loess	very deep loess	very deep loess
Gravel content (%) (> 4.75 mm)	0.0	0.0	0.1	0.0
Sand content (%) (4.75 mm – 75 μm)	8.8	1.3	4.2	6.4
Silt content (%) (75 μm – 2 μm)	68.8	73.3	69.6	72.0
Clay content (%) (< 2 μm)	22.4	25.4	26.1	21.6
Liquid limit, LL (%)	32	35	35	31
Plastic limit, PL (%)	25	27	23	24
Plastic Index, PI (%)	7	8	12	7
AASHTO classification	A-4(7)	A-4(9)	A-6(12)	A-4(7)
USCS classification	CL-ML	CL	CL	CL-ML
USCS Description	Silty clay	Lean clay	Lean clay	Silty clay
Iowa DOT Material Classification	Suitable	Suitable	Suitable	Suitable
Soil Color	Olive Brown	Olive Brown	Olive Brown	Olive Brown
Specific Gravity, G_s	2.717	2.679	2.673	2.720
Std. Proctor, w_{opt} (%)	16.0	18.4	18.0	16.0
Std. Proctor, γ_{dmax} (lb/ft^3)	110.0	106.0	106.7	110.5
Mod. Proctor, w_{opt} (%)	12.4	14.0	14.0	13.0
Mod. Proctor, γ_{dmax} (lb/ft^3)	120.0	117.0	117.5	119.6

CHAPTER 5. FIELD TEST RESULTS

To evaluate compliance with embankment compaction QC/QA requirements, field testing was conducted on nine active Iowa DOT embankment projects. Field activities included in-place moisture and density testing using drive core testing, and DCP testing. Bulk samples collected from the project sites were tested in the laboratory to determine the soil index properties, as summarized in Chapter 3. Table 15 summarizes the project location information, ISU field testing activities, and the availability of QC/QA testing.

Table 15. Summary of project information

Project Number	Project ID	Location	County	ISU Field Testing		QC Data during ISU Testing	QA Data during ISU Testing
1	IM-035-2(365)67--13-77	Northeast side of Intersection between I-35 and Grand Ave, Polk, IA	Polk	TB1: 5/29/14	15 DC, 5 DCP	NA	NA
		Northeast side of Intersection between I-35 and Grand Ave, Polk, IA	Polk	TB2: 6/7/14	N/A	NA	NA
		Southeast side of Intersection between I-35 and E.P. True Parkway, Polk, IA	Polk	TB3: 8/5/14	15 DC, 5 DCP	NA	NA
		Southeast side of Intersection between I-35 and E.P. True Parkway, Polk, IA	Polk	TB4: 8/19/14	15 DC, 5 DCP	w and γ_d	NA
2	IM-035-2(353)54--13-91	Beside I-35, Hoover St, and NW 97th St, Warren, IA	Warren	TB1: 6/3/14	15 DC, 5 DCP	w	NA
		Beside I-35, Hoover St, and NW 97th St, Warren, IA	Warren	TB2: 7/22/14	15 DC, 5 DCP	w	NA

Table 15 continued

Project Number	Project ID	Location	County	ISU Field Testing		QC Data during ISU Testing	QA Data during ISU Testing
		Intersection between I-35 and Hwy 92, Warren, IA	Warren	TB3: 8/4/14	15 DC, 5 DCP	*w*	NA
3	NHSX-100-1(77)--3H-57	New constructed Collins Rd near Old Ferry Rd, Linn, IA	Linn	TB1: 6/6/14	15 DC, 5 DCP	*w*	NA
		New constructed Collins Rd near Old Ferry Rd, Linn, IA	Linn	TB2: 7/8/14	N/A	*w*	NA
		New constructed Collins Rd near Covington Rd, Linn, IA	Linn	TB3: 7/15/14	20 DC, 8 DCP	*w*	NA
		New constructed Collins Rd near Covington Rd, Linn, IA	Linn	TB4: 8/1/14	15 DC, 5 DCP	*w*	NA
		New constructed Collins Rd near Old Ferry Rd, Linn, IA	Linn	TB5: 9/8/14	15 DC, 5 DCP	*w*	NA
4	NHSX-100-1(79)--3H-57	New constructed Collins Rd near Edgewood Rd NE, Linn, IA	Linn	6/6/14	15 DC, 5 DCP	w and γ_d	w and γ_d
5	NHSX-534-1(85)--3H-65	West side of Intersection between I-29 and Platteview, Mills, IA	Mills	TB1: 6/26/14	15 DC, 6 DCP	NA	NA
		East side of Intersection between I-29 and Platteview, Mills, IA	Mills	TB2: 6/26/14	15 DC, 6 DCP	NA	NA

Table 15 continued

Project Number	Project ID	Location	County	ISU Field Testing		QC Data during ISU Testing	QA Data during ISU Testing
6	IM-NHS-080-1(364)3--03-78	Ramp at Intersection between I-80 and S Expressway St, Pottawattamie, IA	Pottawattamie	TB1: 7/2/14	15 DC, 5 DCP	w and γ_d	w and γ_d
		Ramp at Intersection between I-80 and S Expressway St, Pottawattamie, IA	Pottawattamie	TB2: 7/10/14	15 DC, 5 DCP	w and γ_d	w and γ_d
7	IM-029-6(186)136--13-97	Southeast side of Intersection between I-29 and 260th St, Woodbury, IA	Woodbury I-29	TB1: 7/9/14	15 DC, 7 DCP	w	w
		Southeast side of Intersection between I-29 and 260th St, Woodbury, IA	Woodbury I-29	TB2: 7/10/14	15 DC, 6 DCP	w	w
		Southeast side of Intersection between I-29 and 260th St, Woodbury, IA	Woodbury I-29	TB3: 8/7/14	15 DC, 5 DCP	w	w
8	IM-074-1(234)0--13-82	Northeast side of Intersection between I-74 and E 67th St, Scott, IA	Scott	TB1: 7/16/14	15 DC, 5 DCP	NA	NA
		Northwest side of Intersection between I-74 and E 67th St, Scott, IA	Scott	TB2: 7/31/14	15 DC, 5 DCP	NA	NA
		Northeast side of Intersection between I-74 and E 67th St, Scott, IA	Scott	TB3: 9/19/14	15 DC, 5 DCP	NA	NA

Table 15 continued

Project Number	Project ID	Location	County	ISU Field Testing		QC Data during ISU Testing	QA Data during ISU Testing
9	NHSX-020-1(116)--3H-97	Northwest side of Intersection between US 20 and Jasper Ave, Woodbury, IA	Woodbury (US20)	TB1: 9/26/14	15 DC, 5 DCP	NA	NA
		Northeast side of Intersection between US 20 and Minnesota Ave, Woodbury, IA	Woodbury (US20)	TB2: 9/26/14	15 DC, 5 DCP	NA	NA
		Northwest side of Intersection between US 20 and Jasper Ave, Woodbury, IA	Woodbury (US20)	TB3: 10/18/14	15 DC, 5 DCP	NA	NA
		Northeast side of Intersection between US 20 and Minnesota Ave, Woodbury, IA	Woodbury (US20)	TB4: 10/18/14	15 DC, 5 DCP	NA	NA

DC – Drive core cylinder
DCP – Dynamic cone penetrometer
GPS measurements were obtained at each test location.
NA – Not available

The results of testing and evaluation are described in the following sections.

Project 1. Polk County

Overview

The ISU research team conducted field testing at this grading project site on 05/29/14, 06/07/14, 08/05/14, and 08/19/14. No field testing was performed on 06/07/14 (TB2) due to rain, but material was obtained to conduct Proctor testing. The fill materials obtained at the time of testing consisted of glacial till materials and were classified as A-7-6(21), A-7-5(8), A-6(9), and A-6(11) by the AASHTO Soil Classification System and as CL by the USCS.

At this site, the project specification required achievement of 95% relative compaction and moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test. The equipment used during construction is shown in Figure 21 through Figure 27.

Figure 21. Polk County Project 1: Caterpillar MT-35 scraper used to collect and place loose fill materials

Figure 22. Polk County Project 1: Caterpillar 740B dump truck used to place loose fill materials

Figure 23. Polk County Project 1: Caterpillar 143H motor grader used to level the embankment surface

Figure 24. Polk County Project 1: Disc used to dry embankment materials

Figure 25. Polk County Project 1: Caterpillar D6T dozer used for grading and lift thickness adjustment

A disc was used to break down and aerate the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the pull-behind sheepsfoot roller (Figure 26).

Figure 26. Polk County Project 1: Pull-behind sheepsfoot roller used for soil compaction

Polymer geogrid was used for reinforcement near the embankment toe (Figure 27).

Figure 27. Polk County Project 1: Geogrid placed near embankment toe

Field observations indicated that the material obtained from the borrow area at the time of ISU testing was relatively wet, and pumping was observed under haul truck tires.

ISU Field Test Results

In situ moisture content and dry density test results are compared with laboratory Proctor test results in Figure 28, Figure 29, and Figure 30.

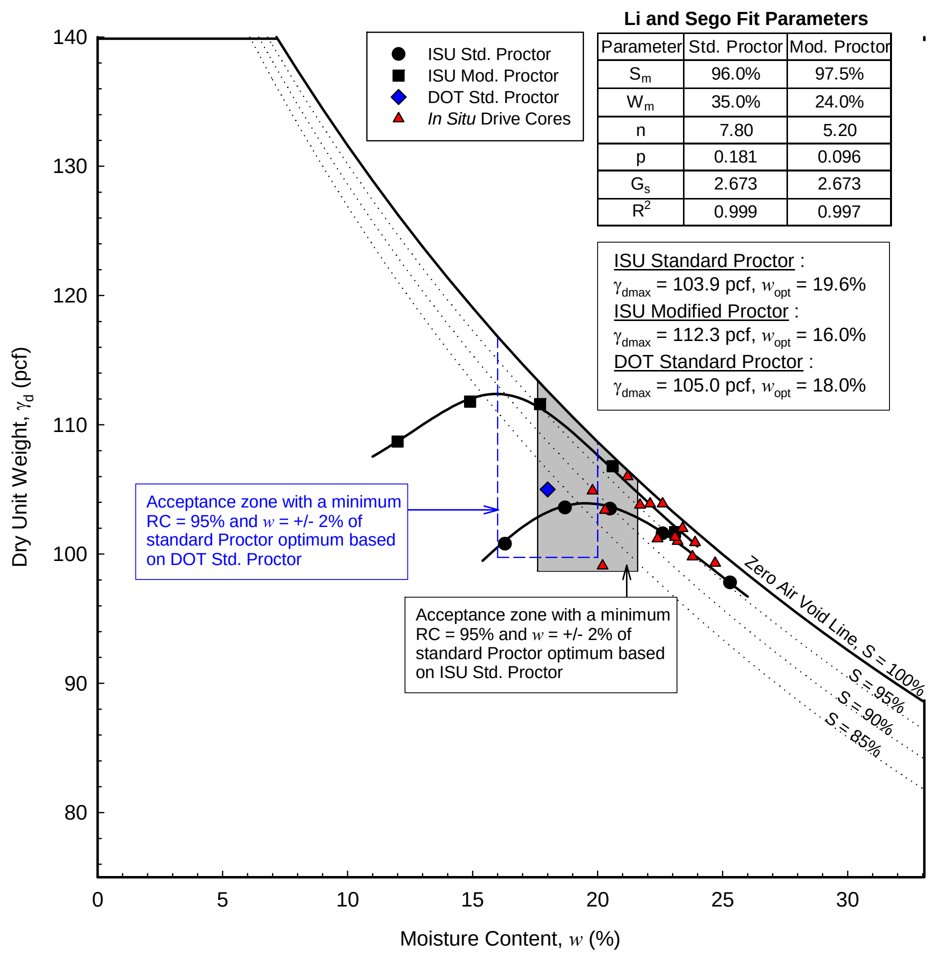


Figure 28. Polk County Project 1 TB1: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

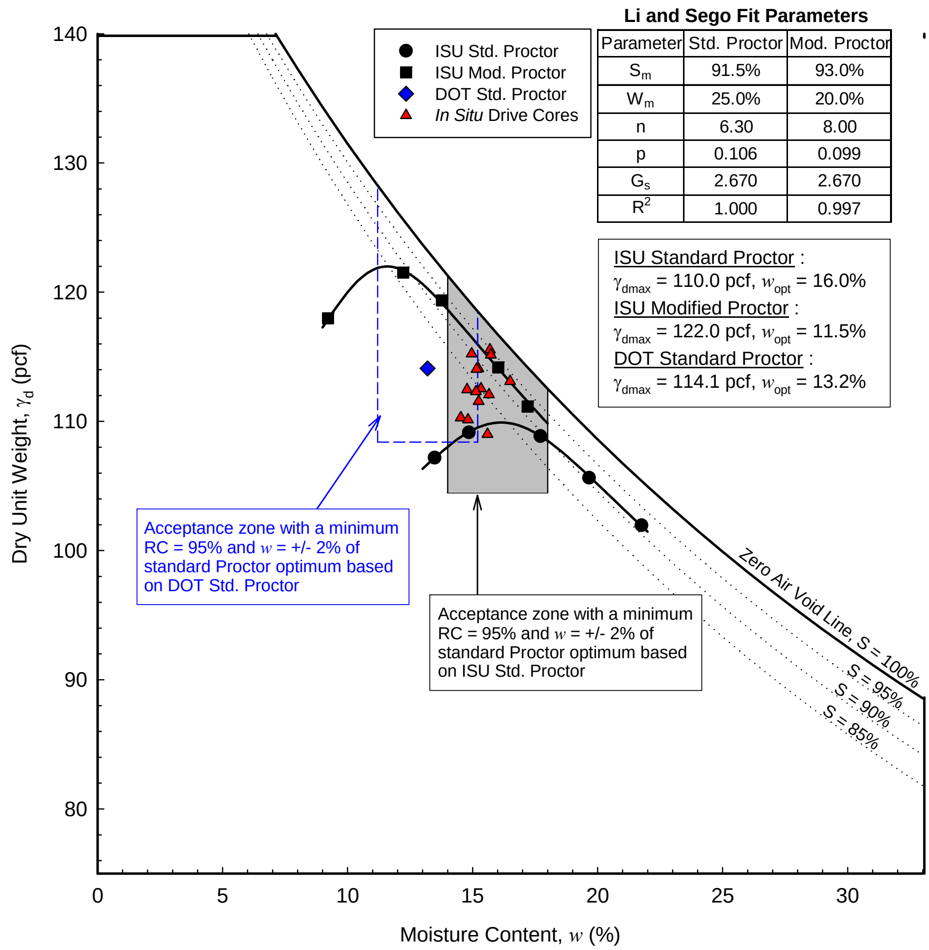


Figure 29. Polk County Project 1 TB3: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

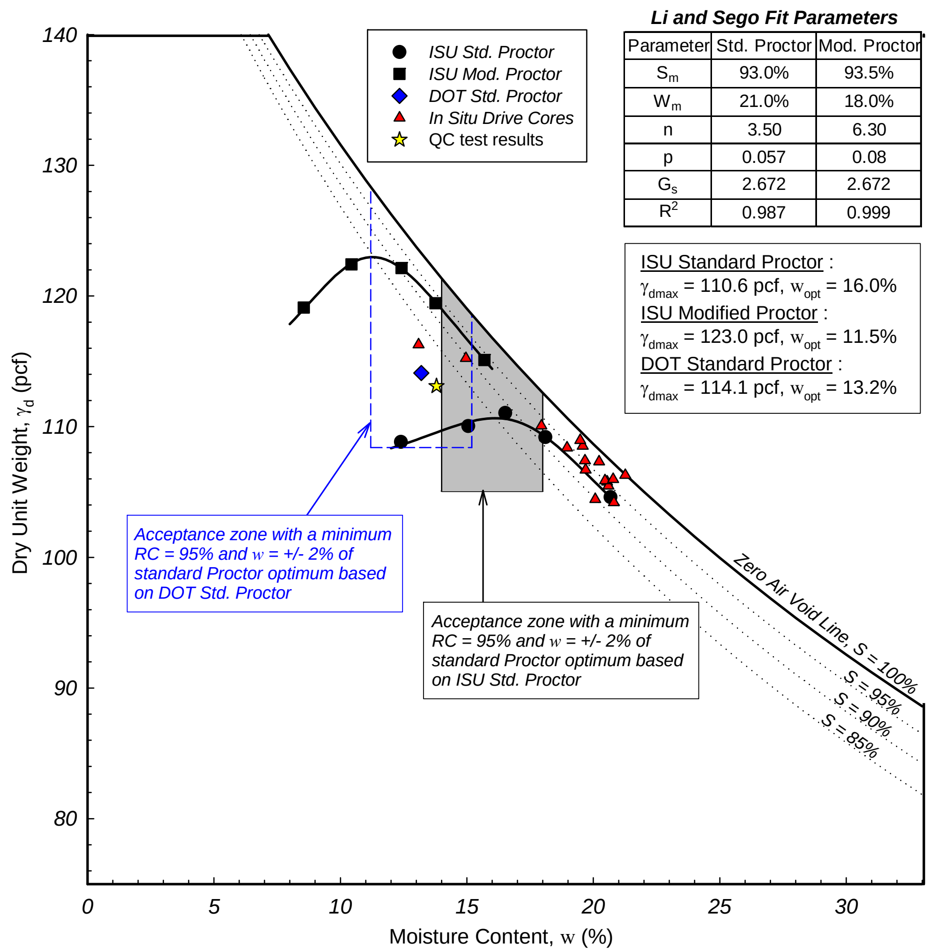


Figure 30. Polk County Project 1 TB4: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

The Proctor test results used by the Iowa DOT showed optimum moisture contents about 1.6% to 2.8% lower than those determined from ISU testing. Similarly, the Proctor test results used by the Iowa DOT showed maximum dry densities about 1.1 to 4.1 lb/ft^3 higher than those determined from ISU testing.

To determine whether the field measurements met the specification requirements, Figures 23 through 25 also show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density. Maximum dry density, optimum moisture content, and the acceptance zone used by the Iowa DOT at the time of ISU testing are also shown in the figures for reference and comparison.

The field test results indicated that the relative compaction of the material ranged from approximately 95% to over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -1.5% and +7.2% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 28 through Figure 30 indicate that a majority of the ISU tests on TB1 and TB4 fell outside the specification limit, with material generally > 2% wet of optimum moisture content and close to the 95% to 100% saturation line.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 31 through Figure 33 for the three TBs.

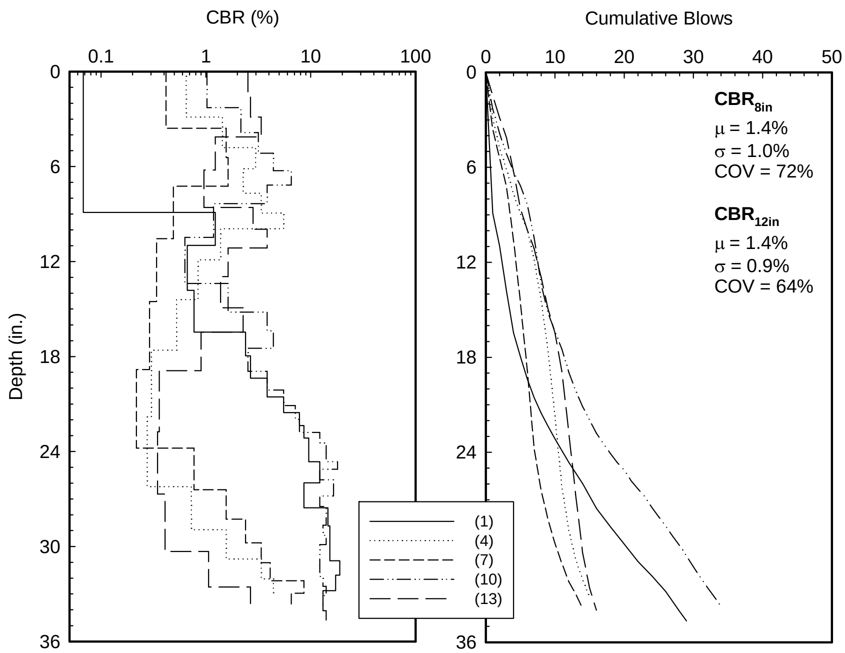


Figure 31. Polk County Project 1 TB1: DCP-CBR values and cumulative blows with depth profiles

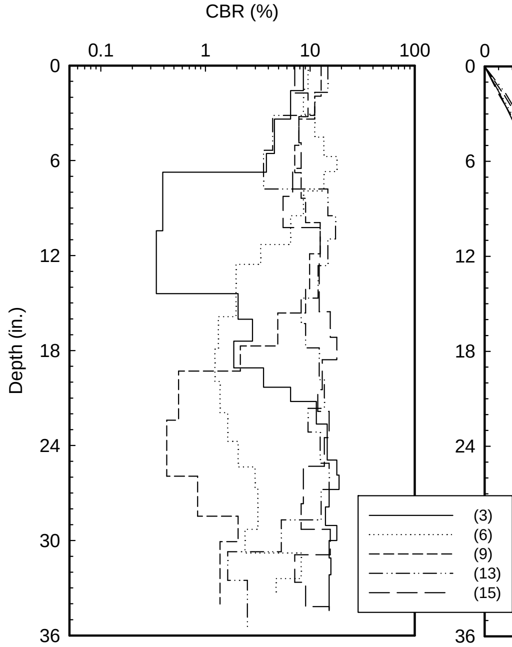


Figure 32. Polk County Project 1 TB3: DCP-CBR values and cumulative blows with depth profiles

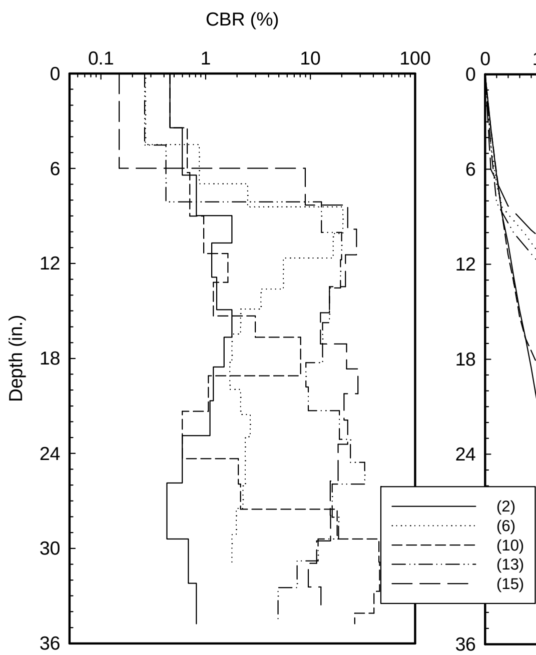


Figure 33. Polk County Project 1 TB4: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. varied between 0.6% and 8.2% and the average CBR value in the top 12 in. varied between 1.4% and 8.6% among the three test beds.

The results indicate that the CBR values are generally higher when the material is within the moisture control limit, as in the case of TB2, and vice versa, as in the cases of TB1 and TB3.

Summary statistics of the field measurements with average, range, standard deviation, and coefficient of variation (COV) are summarized in Table 14.

Table 16. Polk County Project 1: Summary of field testing

Parameter	Polk County TB1 5/29/2014	Polk County TB2 6/7/2014	Polk County TB3 8/5/2014	Polk County TB4 8/19/2014
Relative Compaction				
Average (%)	97.8	N/A	103.0	96.8
Range (%)	95 to 101.6	N/A	99.6 to 105.5	93.9 to 104.8
Standard Deviation (%)	0.02	N/A	0.02	0.03
COV (%)	2	N/A	2	3
$\Delta w\% = w_{field}\% - w_{opt}\%$				
Average (%)	2.6	N/A	-0.7	3.0
Range (%)	-0.2 to +7.2	N/A	-1.5 to +0.5	-3.4 to +4.8
Standard Deviation (%)	1.92	N/A	0.49	1.97
COV (%)	73	N/A	-73	65
$CBR_{8\ in.}$				
Average (%)	1.4	N/A	8.2	0.6
Range (%)	0.1 to 2.7	N/A	4.5 to 12.3	0.4 to 1.1
Standard Deviation (%)	1.0	N/A	2.8	0.3
COV (%)	72	N/A	35	47
$CBR_{12\ in.}$				
Average (%)	1.4	N/A	8.6	3.4
Range (%)	0.2 to 2.1	N/A	2.6 to 11.4	0.7 to 8.0
Standard Deviation (%)	0.9	N/A	3.6	3.0
COV (%)	64	N/A	42	89

Control Charts

The contractor QC data and ISU data are reported in Figure 34 in the form of control charts monitoring the dry unit weight and moisture content of the compacted fills.

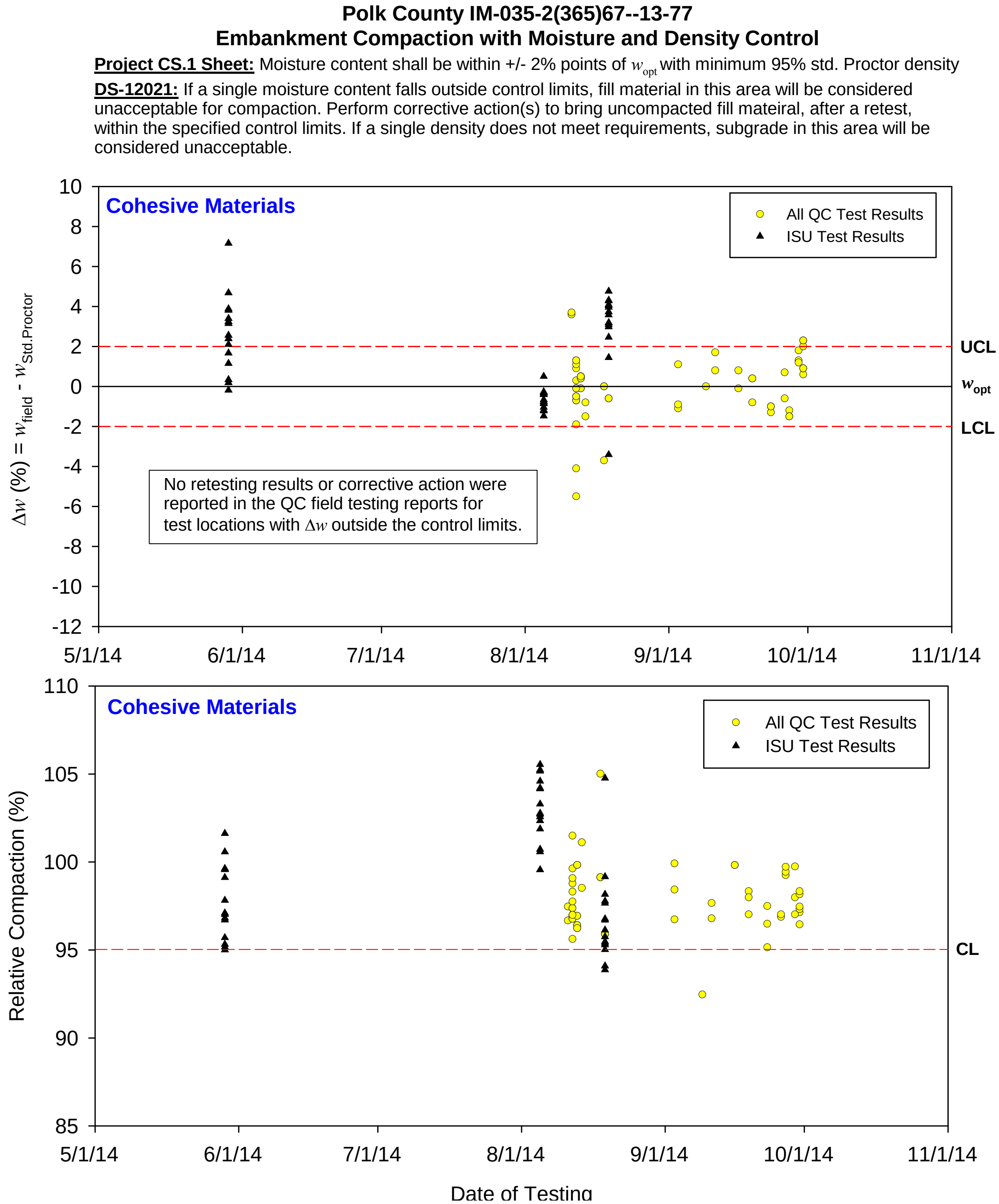


Figure 34. Polk County Project 1: Moisture and density control chart

The control chart data are presented as histograms in Figure 35.

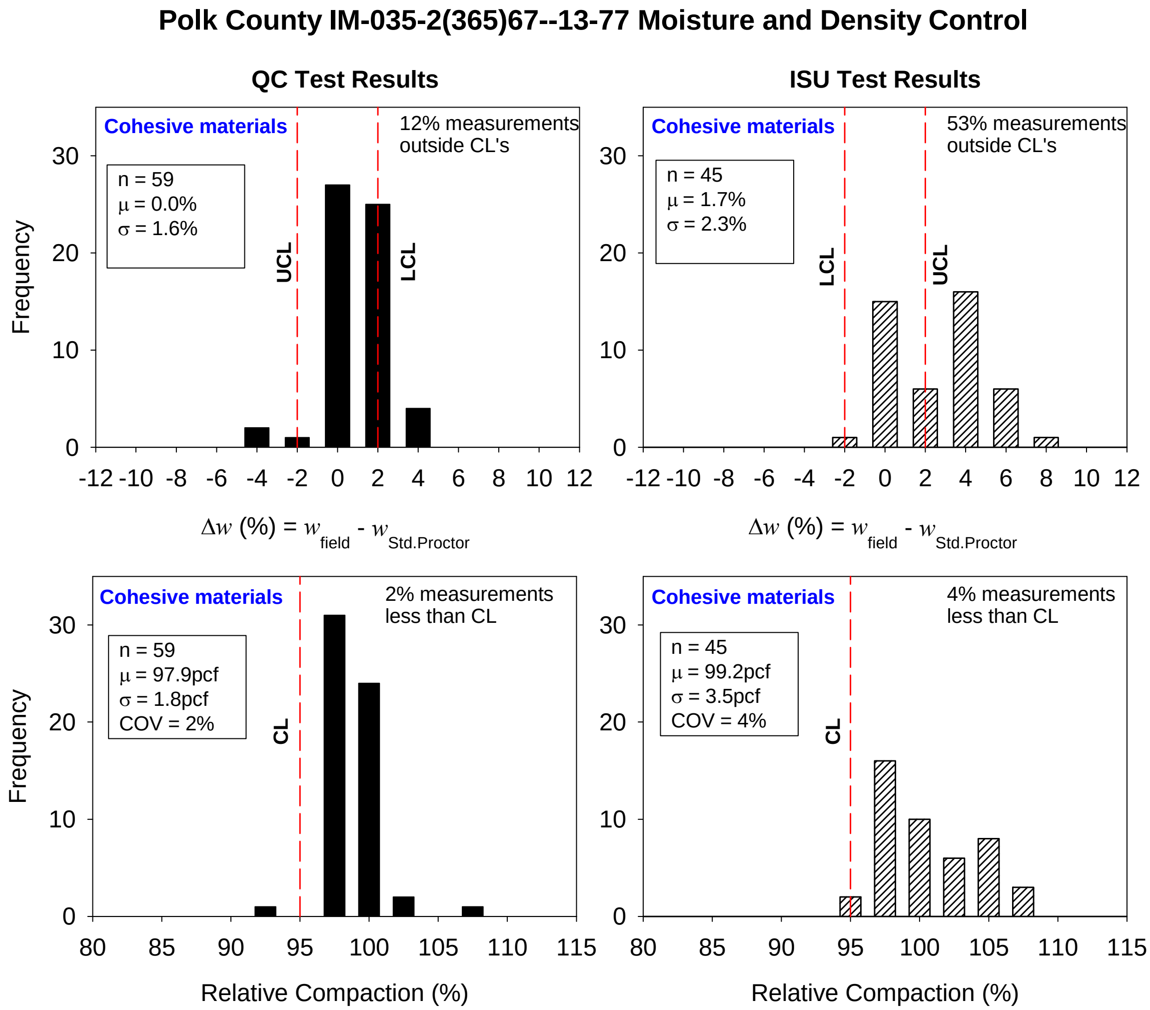
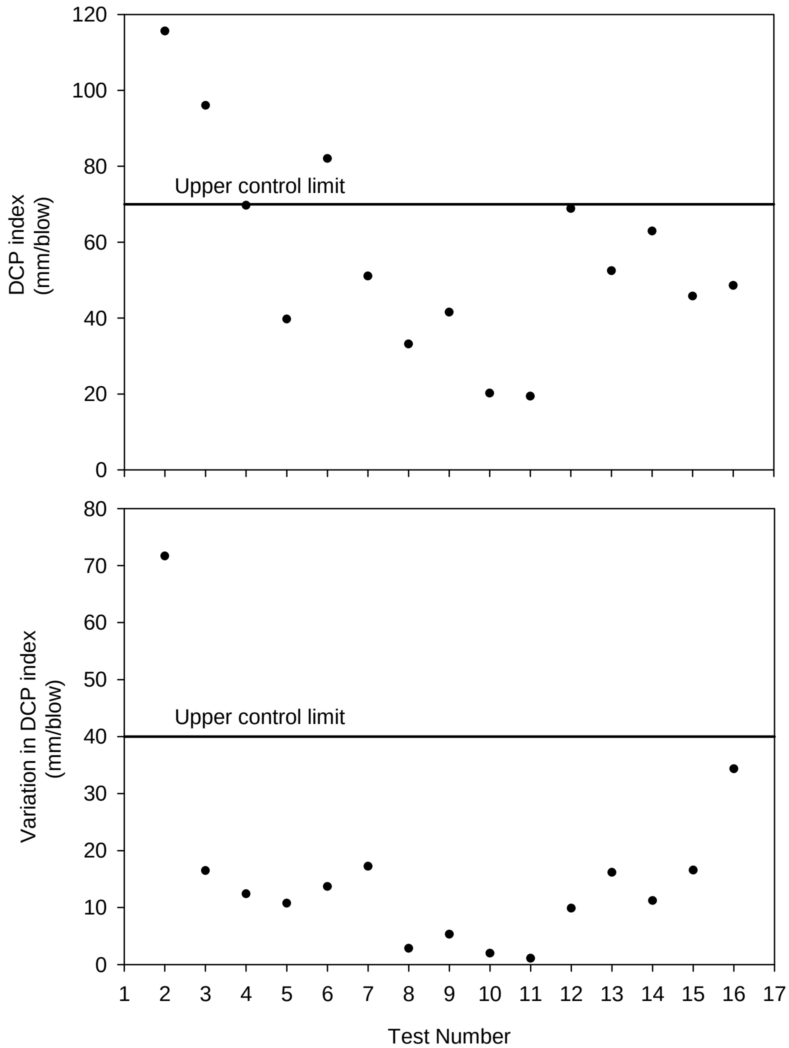


Figure 35. Polk County Project 1: Histograms of moisture and density control results

The data presented in the control charts and histograms indicate that a majority (98%) of the QC data showed relative compaction > 95%, and a majority (87%) of the data fell within the moisture control limits. The ISU testing results show that 96% of the data showed relative compaction > 95%, and only 47% of the data were within the moisture control limits.

Figure 36 shows control charts for DCP index values at a depth of 600 mm.

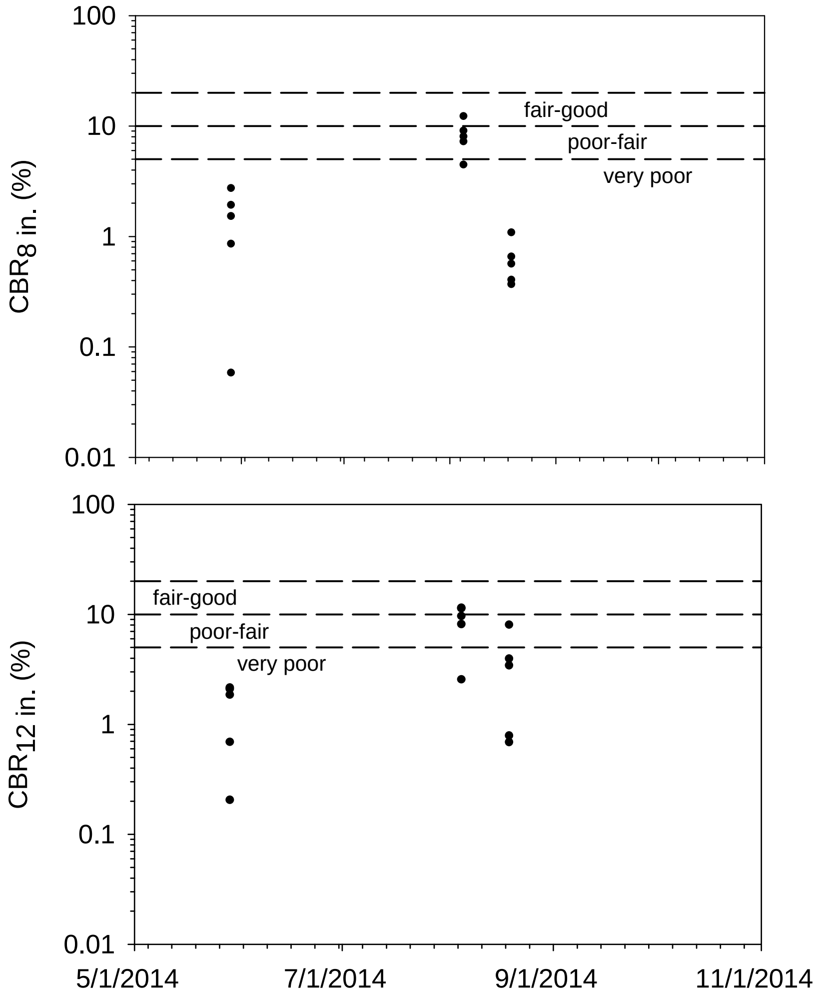


White et al. 2007

Figure 36. Polk County Project 1: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged between 19 and 116 mm/blow, and three points of all of the data exceeded the upper control limit. The variation in the DCP index control chart shows that DCP index variation fell between 10.8 and 16.6 mm/blow at 13 of the 15 points, with one point showing about 72 mm/blow.

Figure 37 shows control charts for CBR values for the top 8 and 12 in. of the compacted lift.



SUDAS 2013

Figure 37. Polk County Project 1: CBR control charts with CBR quality ratings

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicate that 67% of the $CBR_{8in.}$ and 67% of the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 2. Warren County

Overview

The ISU research team conducted field testing at this grading project site on 06/03/14, 07/22/14, and 08/04/14. The fill materials obtained at the time of testing consisted of glacial till materials and were classified as A-7-5(9), A-6(11), A-7-6(28), and A-6(13) by the AASHTO Soil Classification System and CL and CH by the USCS.

At this site, the project specification required achievement of moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test. The equipment used during construction is shown in Figure 38 through Figure 40.

Figure 38. Warren County Project 2: Caterpillar D6T dozer used to control lift thickness

Figure 39. Warren County Project 2: Caterpillar MT-35 scraper used to collect and place loose fill materials

Figure 40. Warren County Project 2: Sheepsfoot roller used for soil compaction

During onsite observation, no disc was used to break down and aerate the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the pull-behind sheepsfoot roller (Figure 40).

ISU Field Test Results

In situ moisture content and dry unit density test results are compared with laboratory Proctor test results in Figure 41 through Figure 44.

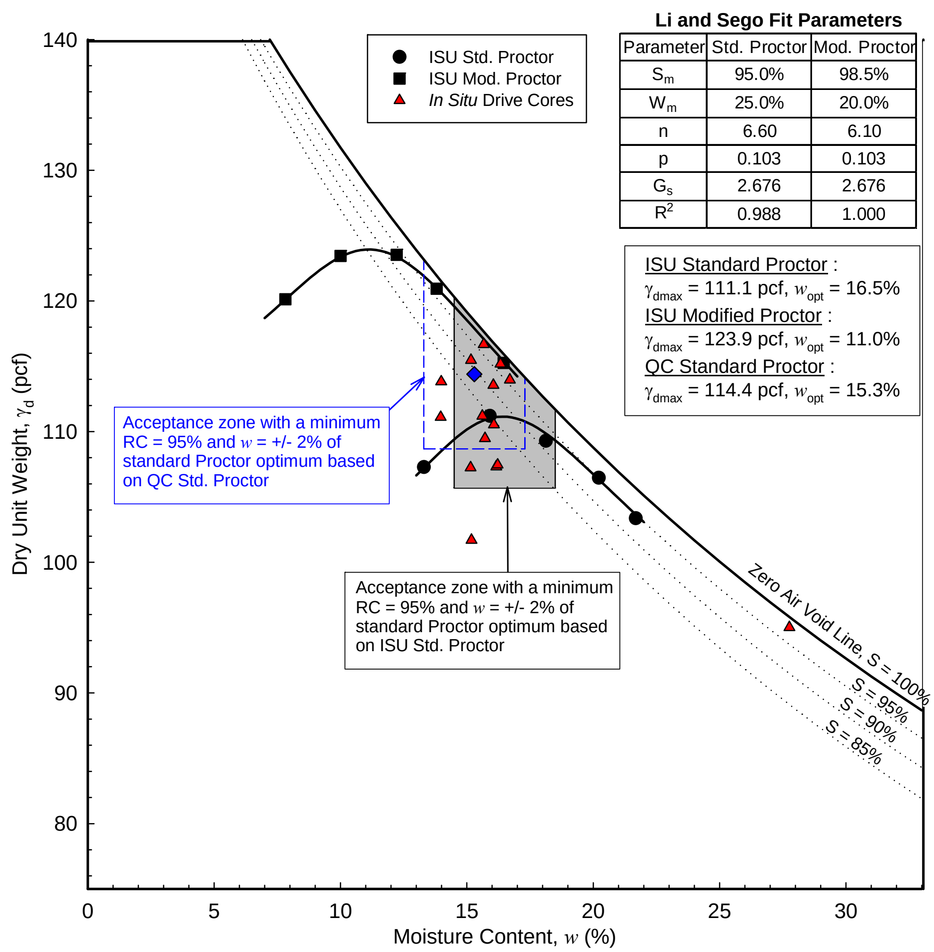


Figure 41. Warren County Project 2 TB1: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

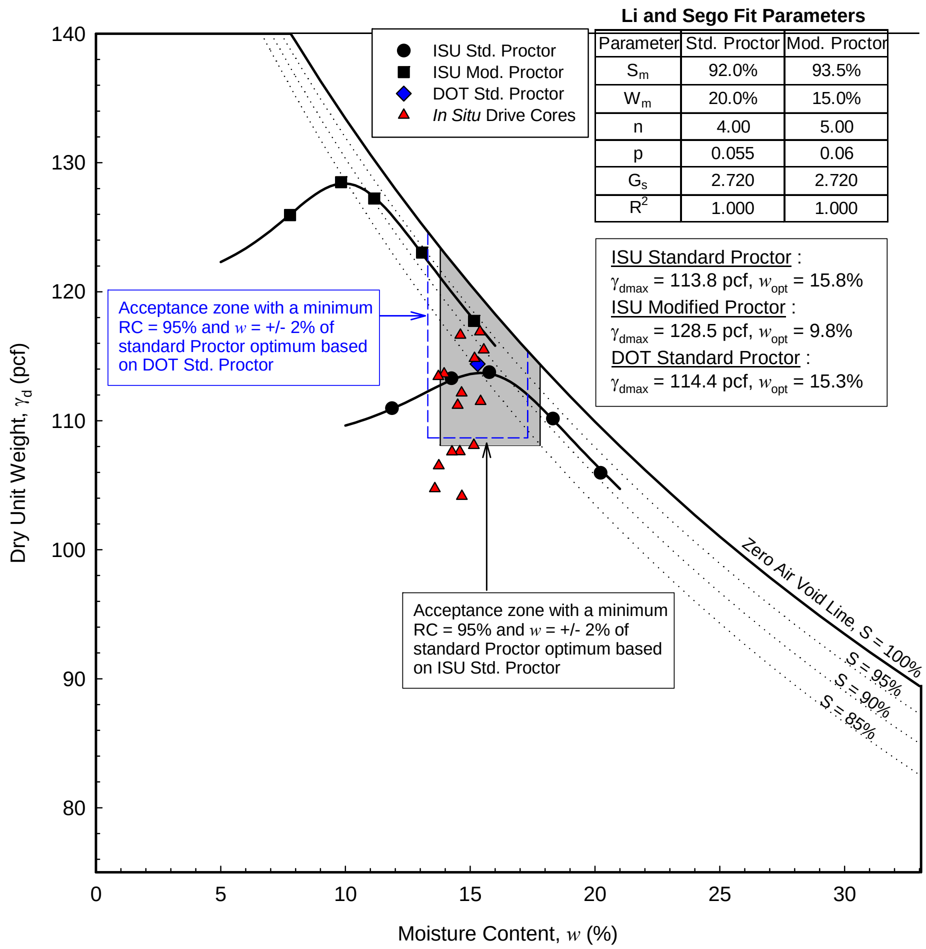


Figure 42. Warren County Project 2 TB2: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

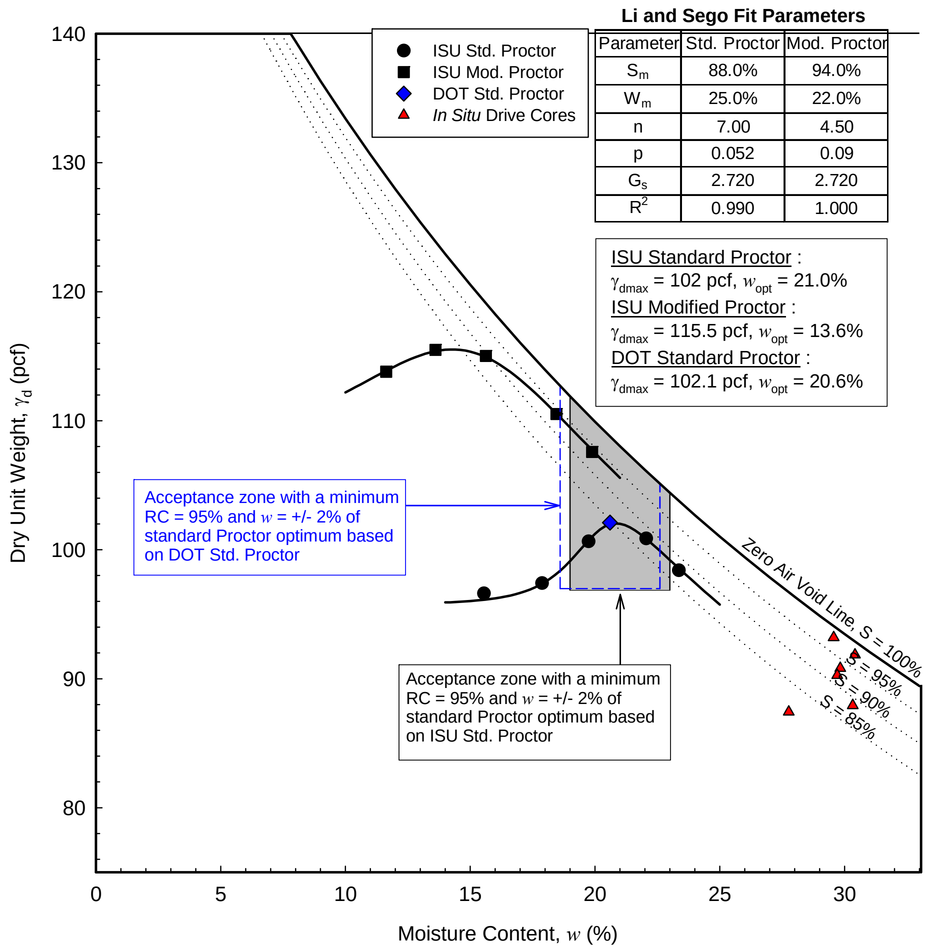


Figure 43. Warren County Project 2 TB3 (gray soil): Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

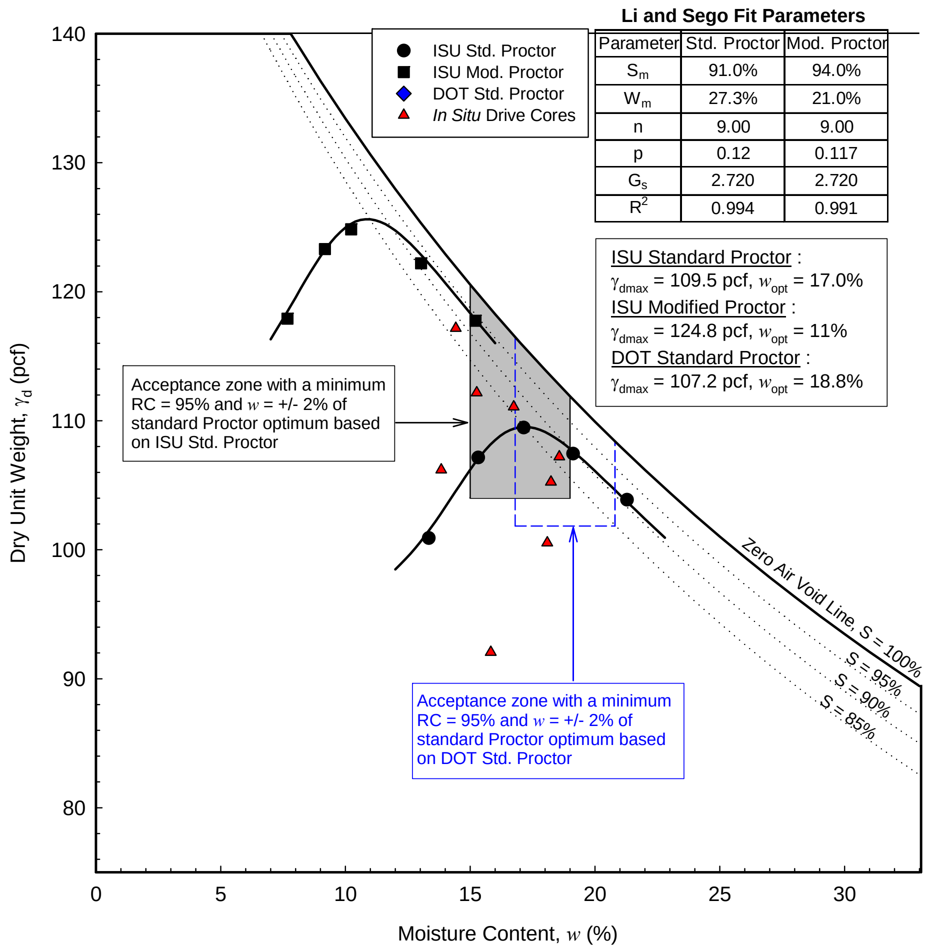


Figure 44. Warren County Project 2 TB3 (brown soil): Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

The Proctor test results used by the Iowa DOT showed optimum moisture contents about 1.2% lower than those determined from ISU testing (Figure 41). Similarly, the Proctor test results used by the Iowa DOT showed maximum dry densities about 3.3 lb/ft^3 higher than those determined from ISU testing.

To determine whether the field measurements met the specification requirements, Figures 36 through 39 also show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density. Maximum dry density, optimum moisture content, and the acceptance zone used by the Iowa DOT at the time of ISU testing are also shown in the figures for reference and comparison.

Field test results indicate that the relative compaction of the material ranged from approximately 84.1% to over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -3.2% to +11.8% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 43 indicate that the results of the ISU tests on TB3 (gray soil) fell outside the specification limit, with material generally > 2% wet of optimum moisture content and close to the 90% to 100% saturation line.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 45, Figure 46, and Figure 47 for the three TBs.

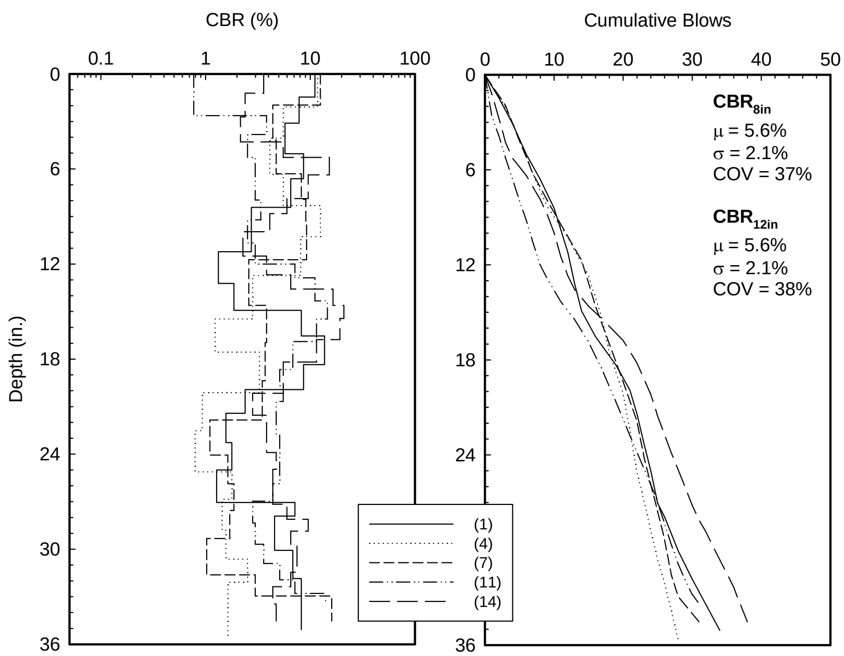


Figure 45. Warren County Project 2 TB1: DCP-CBR values and cumulative blows with depth profiles

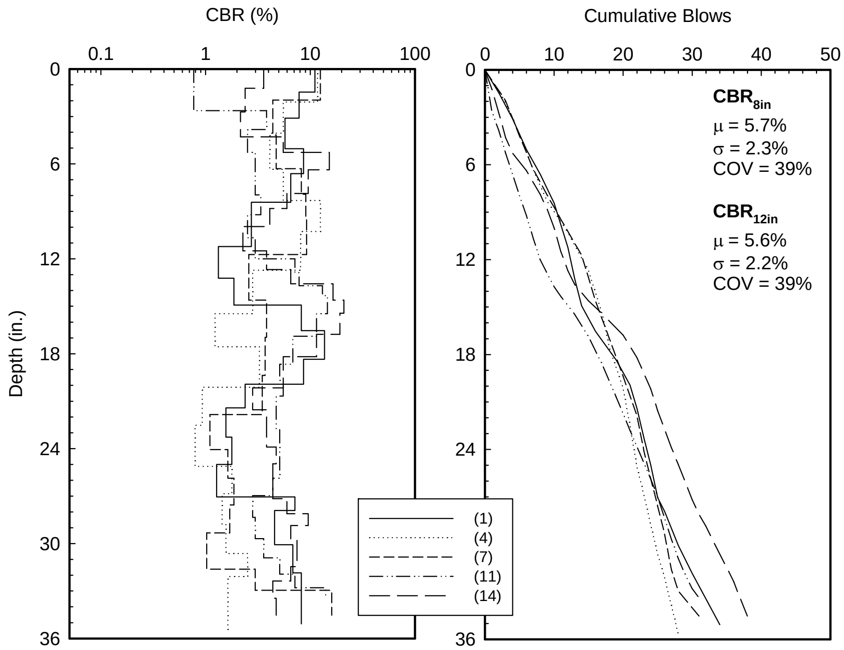


Figure 46. Warren County Project 2 TB2: DCP-CBR values and cumulative blows with depth profiles

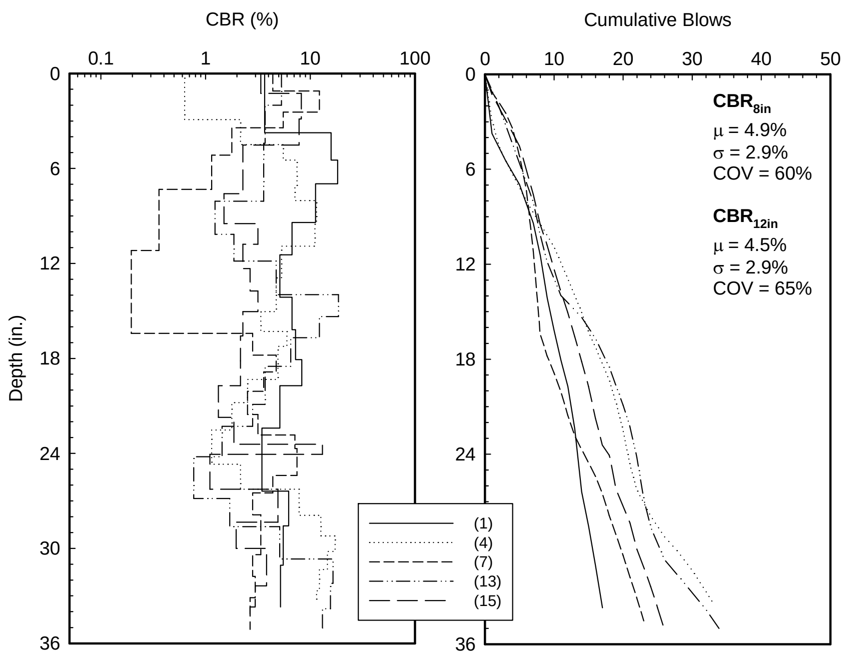


Figure 47. Warren County Project 2 TB3: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. varied between 4.9% to 5.7% and the average CBR value in the top 12 in. varied between 4.5% to 5.6% among the three TBs. The results indicate that the CBR values are generally higher when the material is within the moisture control limit, as in the cases of TB1 and TB2, and vice versa, as in the case of TB3.

Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized in Table 17.

Table 17. Warren County Project 2: Summary of field testing

Parameter	Warren County TB1 6/3/2014	Warren County TB2 7/22/2014	Warren County TB3 8/4/2014
Relative Compaction			
Average (%)	98.8	97.5	93.6
Range (%)	85.4 to 104.8	91.5 to 102.7	84.1 to 107.0
Standard Deviation (%)	0.05	0.04	0.07
COV (%)	5	4	7
$\Delta w\% = w_{field}\% - w_{opt}\%$			
Average (%)	0.4	-1.2	3.3
Range (%)	-2.0 to +11.8	-2.2 to +0.3	-3.2 to +9.4
Standard Deviation (%)	3.25	0.65	4.78
COV (%)	842	-54	145
$CBR_{8\ in.}$			
Average (%)	5.6	5.7	4.9
Range (%)	2.1 to 7.4	2.0 to 7.7	2.8 to 9.9
Standard Deviation (%)	2.1	2.3	2.9
COV (%)	37	39	60
$CBR_{12\ in.}$			
Average (%)	5.6	5.6	4.5
Range (%)	2.4 to 7.6	2.3 to 7.7	1.9 to 9.4
Standard Deviation (%)	2.1	2.2	2.9
COV (%)	38	39	65

Control Charts

The contractor QC data and ISU data are reported in Figure 48 in the form of control charts monitoring the dry density and moisture content of the compacted fills.

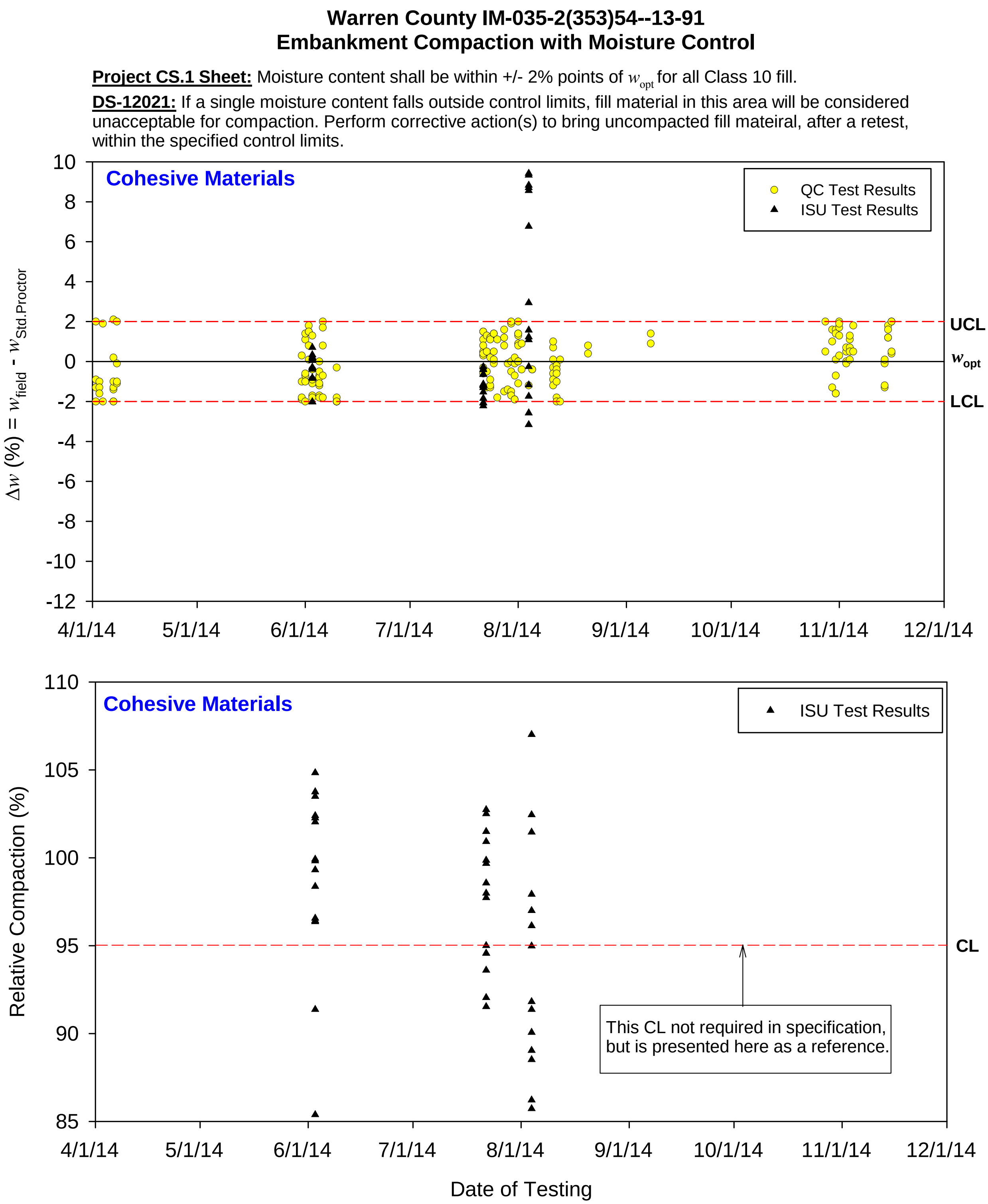


Figure 48. Warren County Project 2: Moisture control chart

The control chart data are presented as histograms in Figure 49.

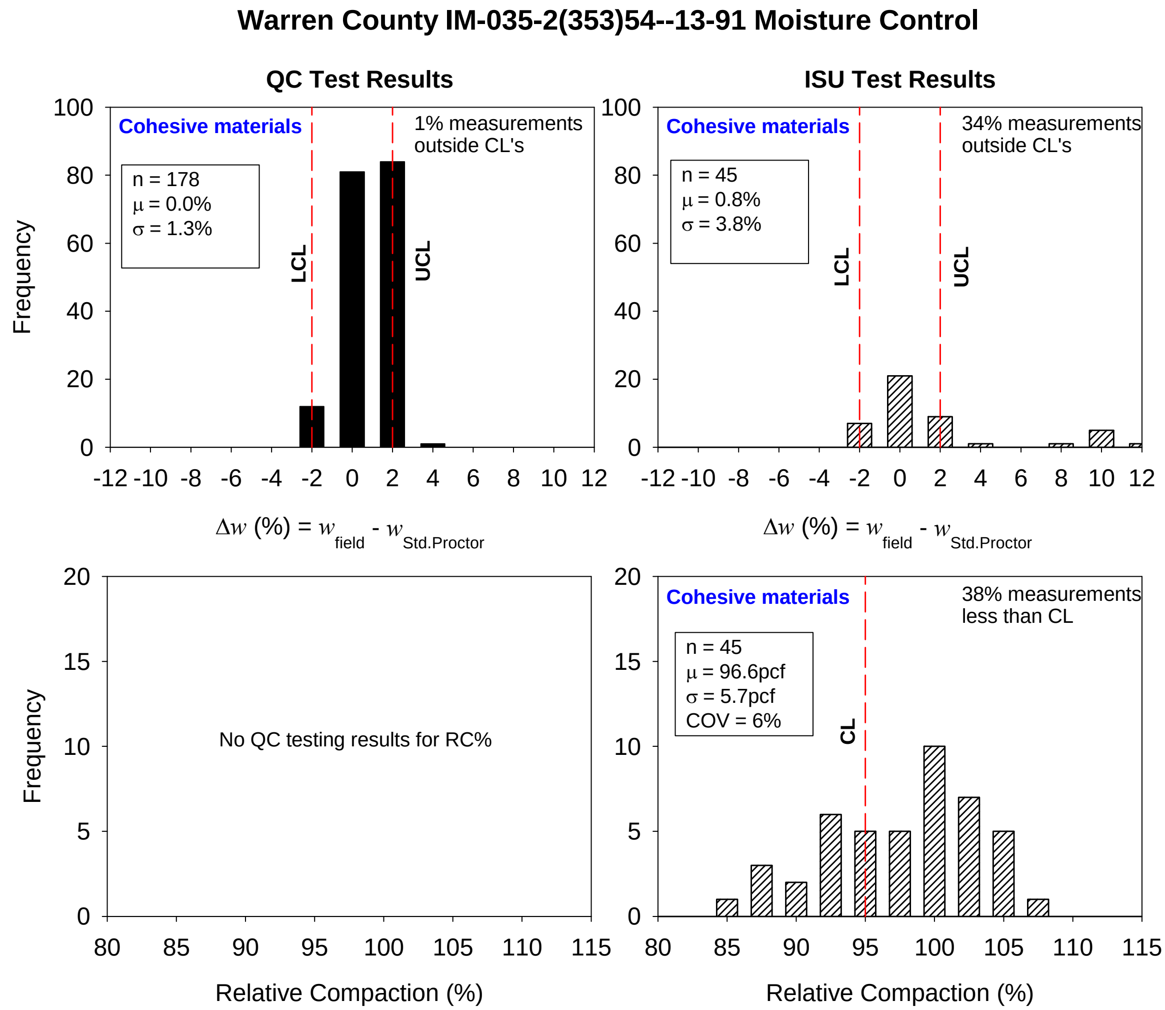
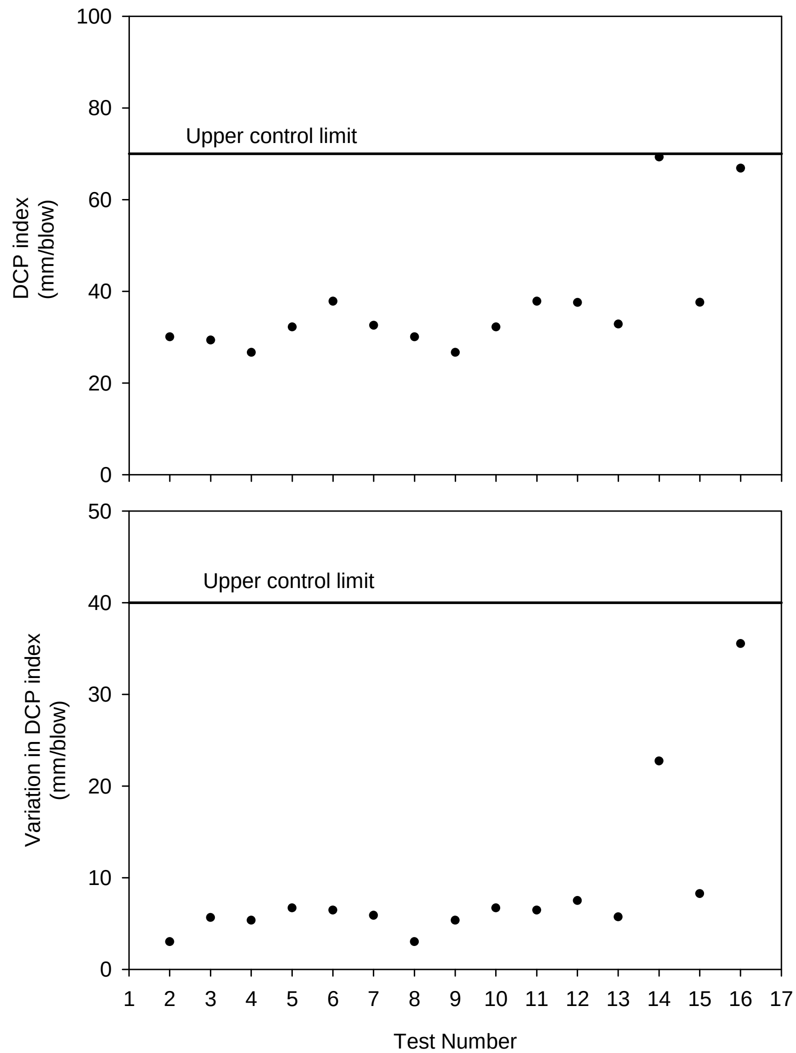


Figure 49. Warren County Project 2: Histograms of moisture and density control results

The data presented in the control charts and histograms indicate that 99% of QC data fell within the moisture control limits. The ISU testing results show that 62% of the data showed relative compaction > 95%, and 67% of the data were within the moisture control limits. Figure 50 shows control charts for DCP index values at a depth of 600 mm.

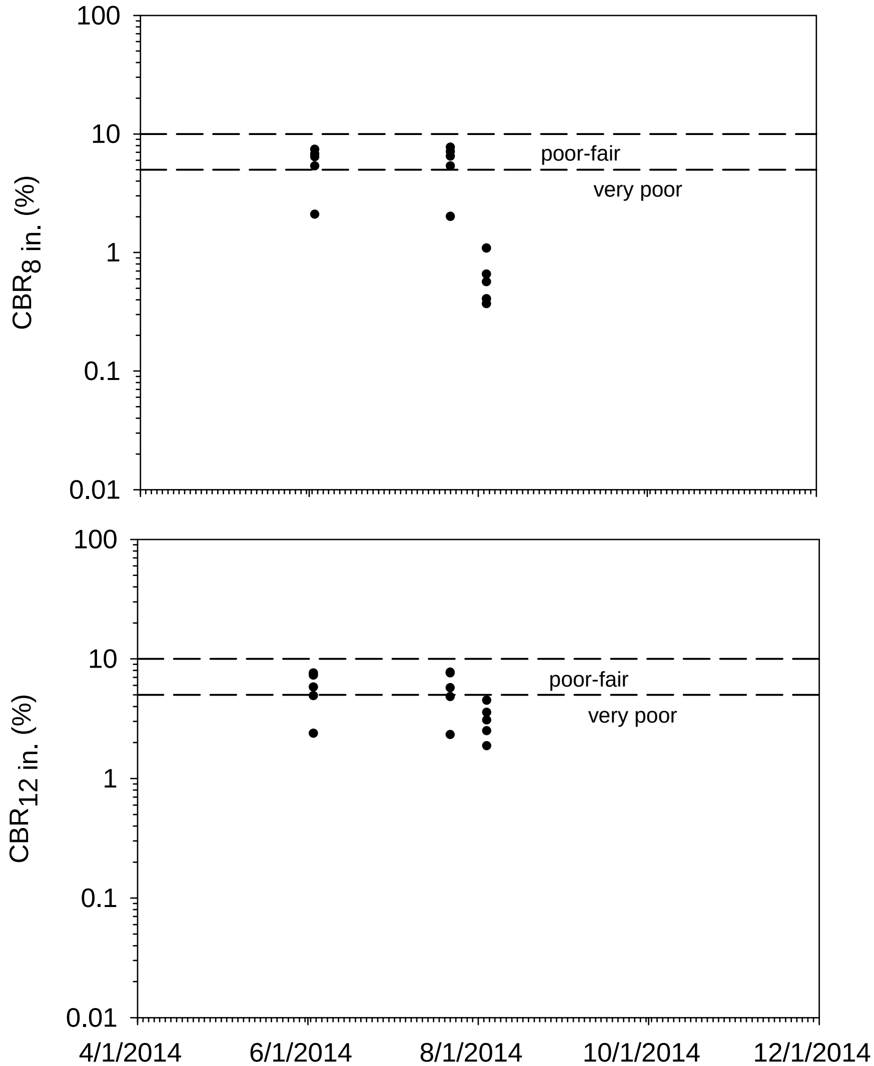


White et al. 2007

Figure 50. Warren County Project 2: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged between 26.6 and 69.3 mm/blow, and all of the data are within the control limit. The variation in the DCP index control chart shows that DCP index variation fell between 3.0 and 8.25 mm/blow, except for two points with 22.7 and 35.5 mm/blow, respectively.

Figure 51 shows control charts for CBR values for the top 8 and 12 in. of the compacted fills.



SUDAS 2013

Figure 51. Warren County Project 2: CBR chart with CBR quality rating

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicate that 47% of the $CBR_{8in.}$ and 60% of the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 3. Linn County 77

Overview

The ISU research team conducted field testing at this grading project site on 06/06/14, 07/08/14, 07/15/14, 08/01/14, and 09/08/14. No field testing for TB2 was performed on 07/08/14 (TB2) due to rain, but material was obtained to conduct Proctor testing. The fill materials obtained at the time of testing consisted of glacial till materials and were classified as A-6(8), A-6(7), A-6(6), and A-6(5) by the AASHTO Soil Classification System and as CL by the USCS.

At this site, the project specification required achievement of 95% relative compaction and moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test for cohesionless materials, and the specification only required achievement of moisture content within ±2.0% of the optimum moisture content for cohesive materials. The equipment used during construction is shown in Figure 52 through Figure 56.

Figure 52. Linn County Project 3: Caterpillar 390D excavating material from borrow source

Figure 53. Linn County Project 3: Caterpillar D6R dozer used to control lift thickness

Figure 54. Linn County Project 3: Disc cultivator used to dry embankment materials

Figure 55. Linn County Project 3: Sheepsfoot roller used for soil compaction

Figure 56. Linn County Project 3: Caterpillar 14M motor grader used to level the embankment surface

A disc was used to break down and aerate the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the pull-behind sheepsfoot roller (Figure 55).

Field observations indicated that the material obtained from the borrow area at the time of ISU testing was relatively wet, and seepage was observed (Figure 57).

Figure 57. Linn County Project 3: Seepage at the construction site

ISU Field Test Results

In situ moisture content and dry density test results are compared with laboratory Proctor test results in Figure 58 through Figure 61.

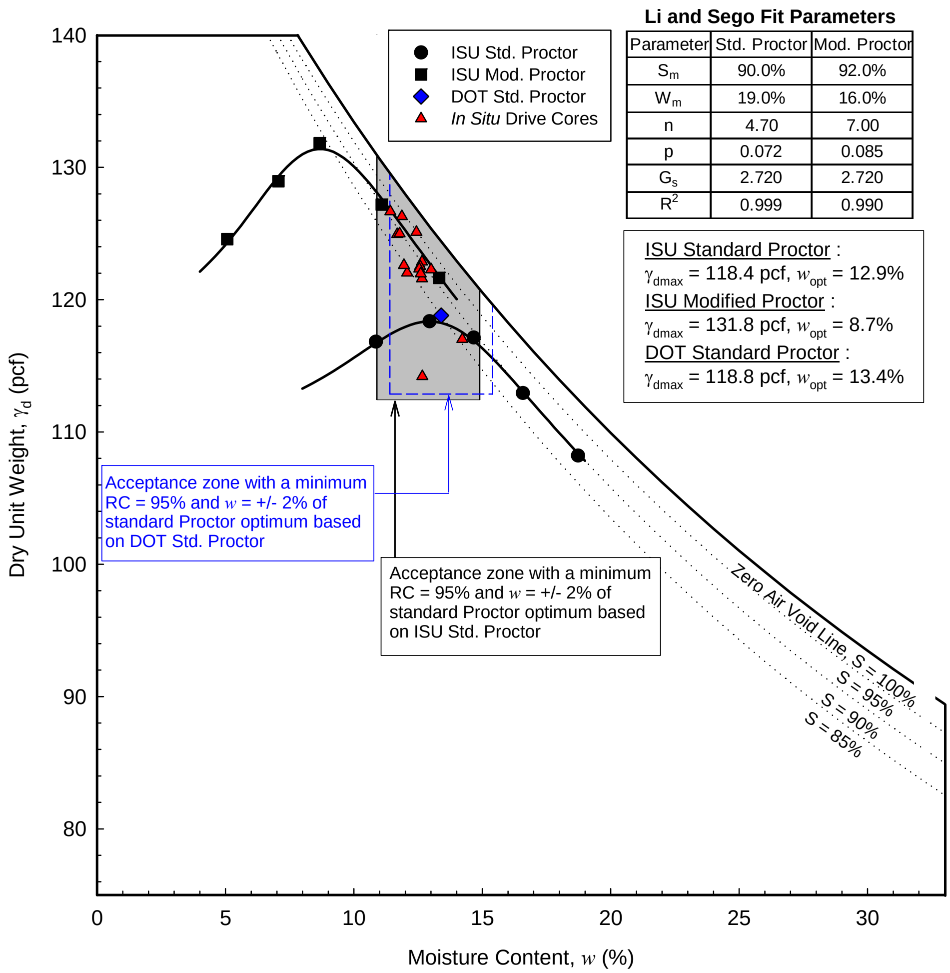


Figure 58. Linn County Project 3 TB1: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

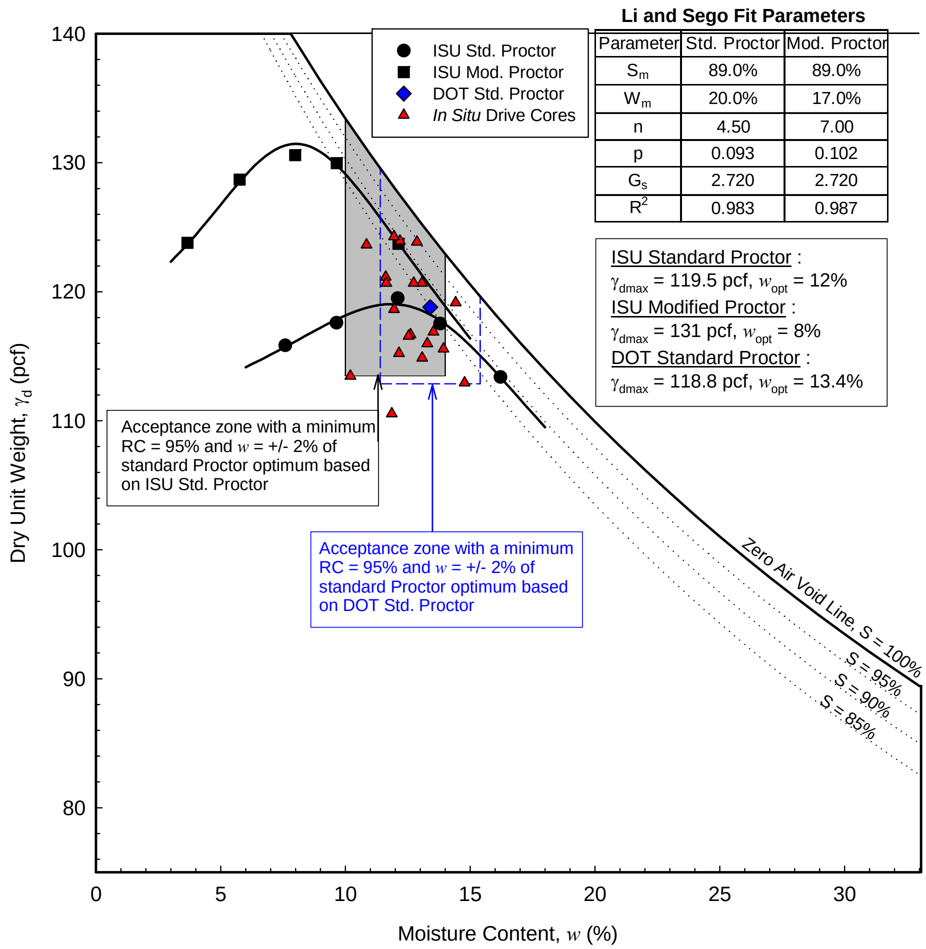


Figure 59. Linn County Project 3 TB3: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

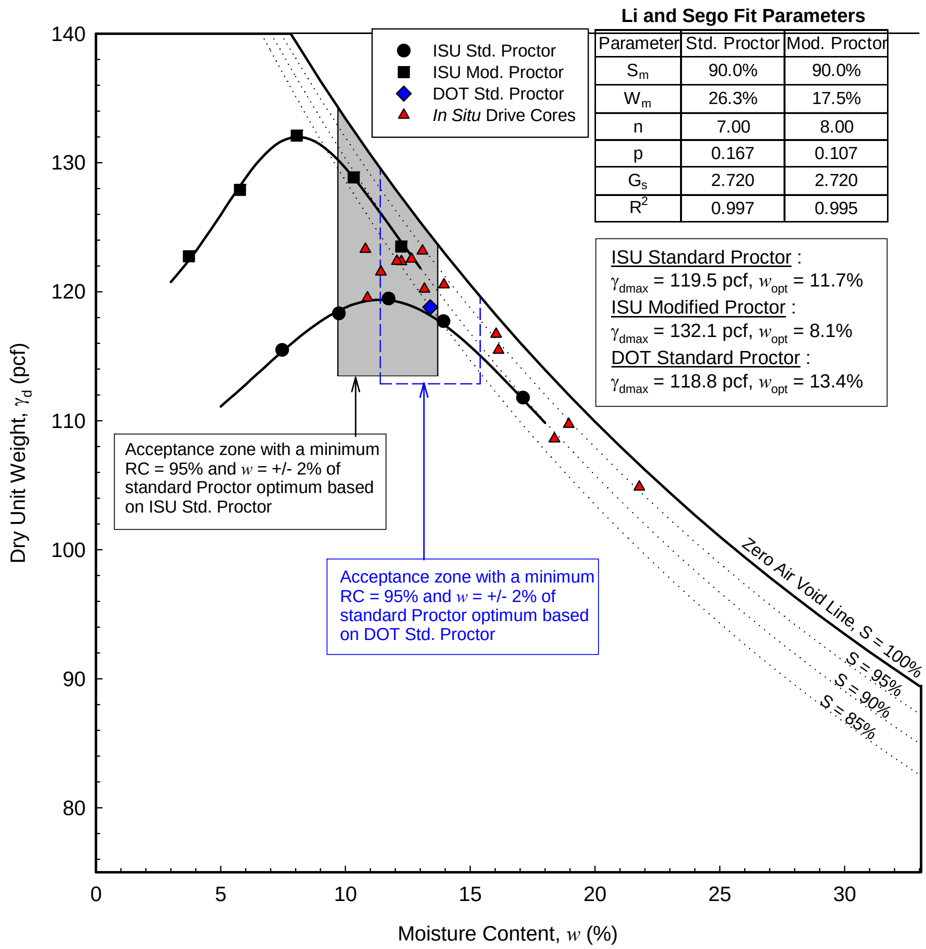


Figure 60. Linn County Project 3 TB4: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

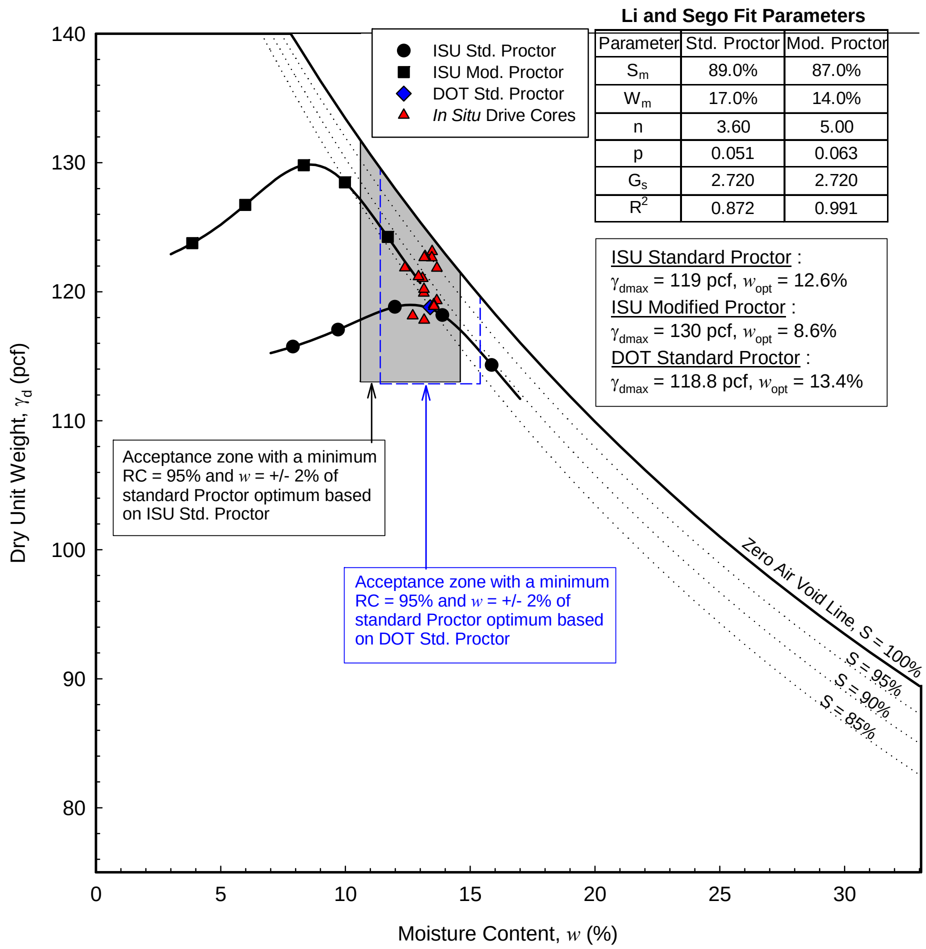


Figure 61. Linn County Project 3 TB5: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

To determine whether the field measurements met the specification requirements, Figures 53 through 56 also show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density.

Field test results indicate that the relative compaction of the material ranged from approximately 87.8% to over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -3.0% and +10.1% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 58 to Figure 61 indicate that a few of the ISU tests on TB4 fell outside of the specification limit, with material generally > 2% wet of optimum moisture content and close to the 95% to 100% saturation line.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 62 through Figure 65 for the four TBs.

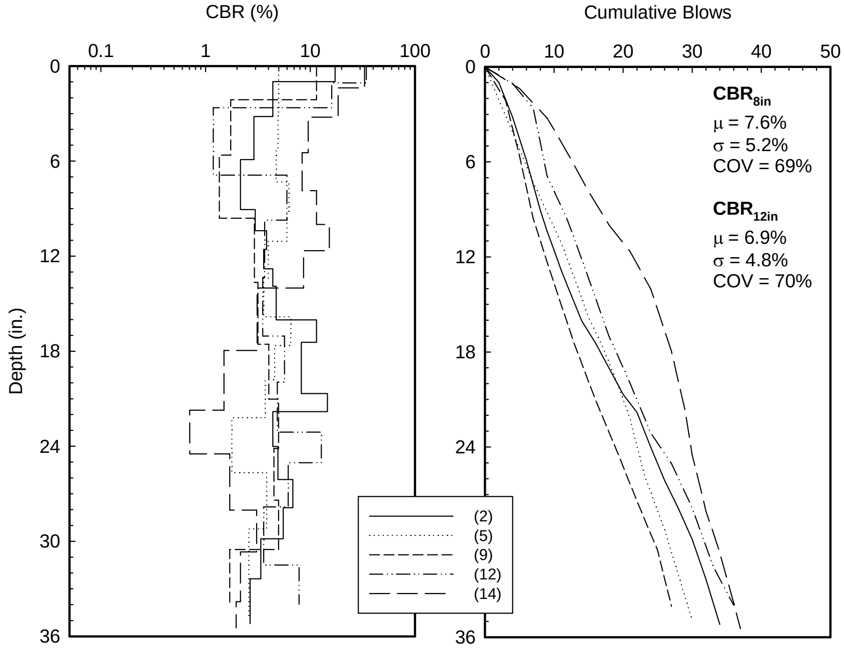


Figure 62. Linn County Project TB1: DCP-CBR values and cumulative blows with depth profiles

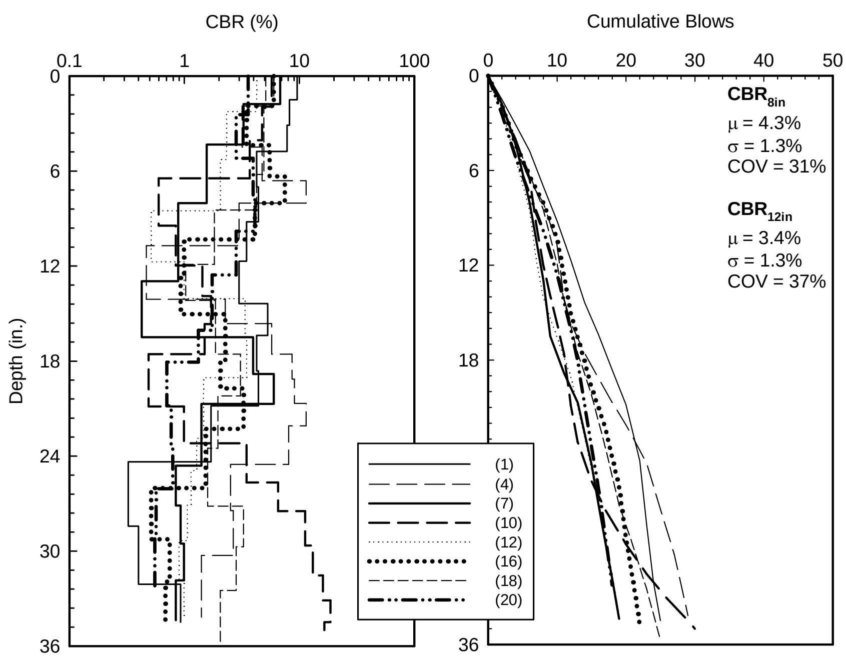


Figure 63. Linn County Project 3 TB3: DCP-CBR values and cumulative blows with depth profiles

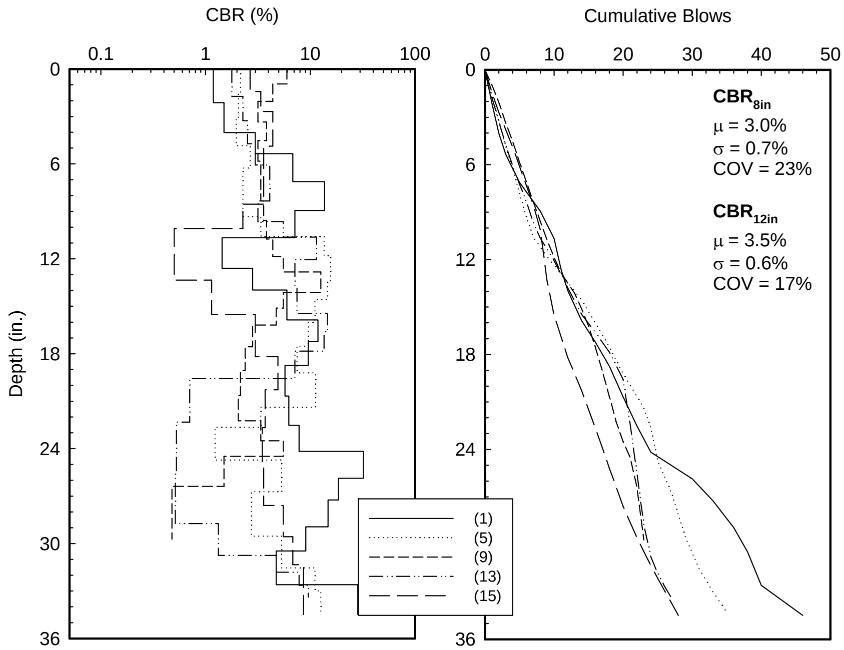


Figure 64. Linn County Project 3 TB4: DCP-CBR values and cumulative blows with depth profiles

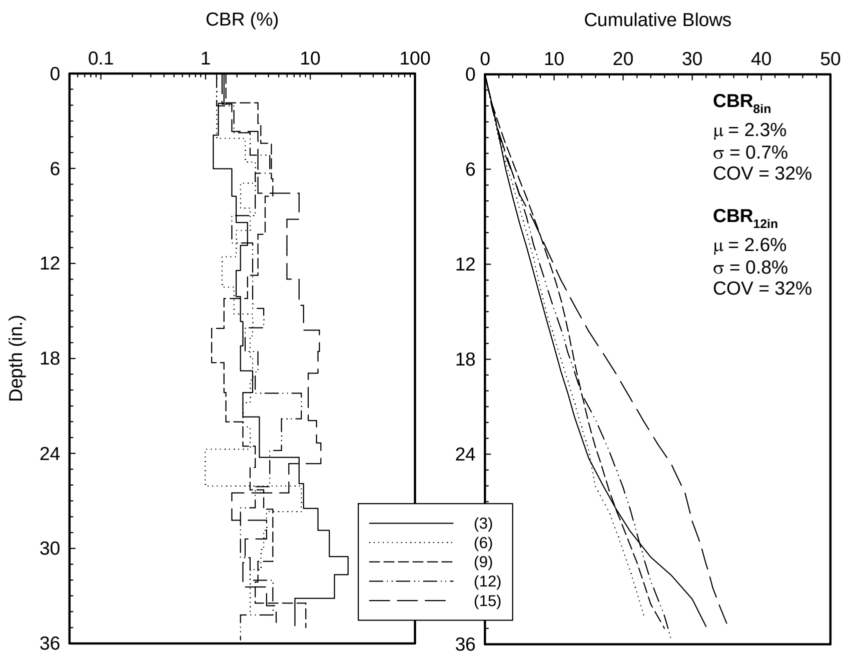


Figure 65. Linn County Project 3 TB5: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. varied between 2.3% and 7.6% and the average CBR value in the top 12 in. varied between 2.6% and 6.9% among the four test beds.

The results do not indicate the trend that the CBR values are generally higher when the material is within the moisture control limit.

Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized Table 18.

Table 18. Linn County Project 3: Summary of field testing results

Parameter	Linn County-77 TB1 6/6/2014	Linn County-77 TB2 7/8/2014	Linn County-77 TB3 7/15/2014	Linn County-77 TB4 8/1/2014	Linn County-77 TB5 9/8/2014
Relative Compaction					
Average (%)	103.5	N/A	100.1	98.8	101.4
Range (%)	96.5 to 107.0	N/A	93.4 to 105.0	87.8 to 103.2	99.0 to 103.5
Standard Deviation (%)	0.03	N/A	0.03	0.05	0.01
COV (%)	3	N/A	3	5	1
$\Delta w\% = w_{field}\% - w_{opt}\%$					
Average (%)	-0.8	N/A	-0.6	2.5	0.9
Range (%)	-1.8 to +1.0	N/A	-3.0 to +1.6	-0.9 to +10.1	0.1 to +1.4
Standard Deviation (%)	0.68	N/A	1.13	3.31	0.36
COV (%)	-86	N/A	-175	131	39
$CBR_{8\ in.}$					
Average (%)	7.6	N/A	4.3	3.0	2.3
Range (%)	3.3 to 16.1	N/A	2.7 to 6.6	2.1 to 3.6	1.4 to 3.2
Standard Deviation (%)	5.2	N/A	1.3	0.7	0.7
COV (%)	69	N/A	31	23	3
$CBR_{12\ in.}$					
Average (%)	6.9	N/A	3.4	3.5	2.6
Range (%)	2.9 to 15.1	N/A	1.8 to 5.6	2.7 to 4.3	1.7 to 3.6
Standard Deviation (%)	4.8	N/A	1.3	0.6	0.8
COV (%)	70	N/A	37	17	32

Control Charts

The contractor QC data and ISU data are reported in Figure 66, Figure 67, and Figure 68 in the form of control charts monitoring the dry unit weight and moisture content of the compacted fills.

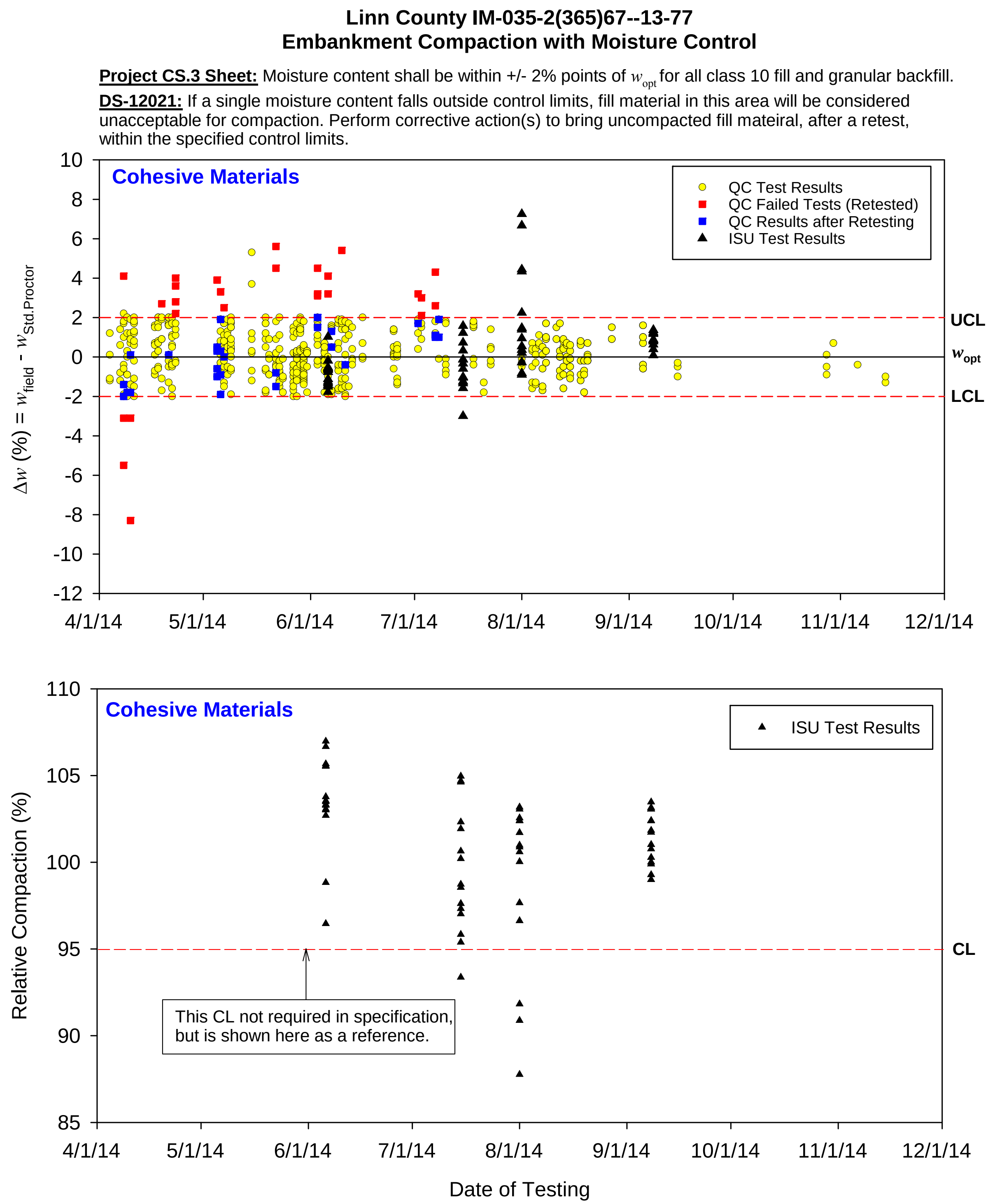


Figure 66. Linn County Project 3: Moisture control chart (cohesive materials)

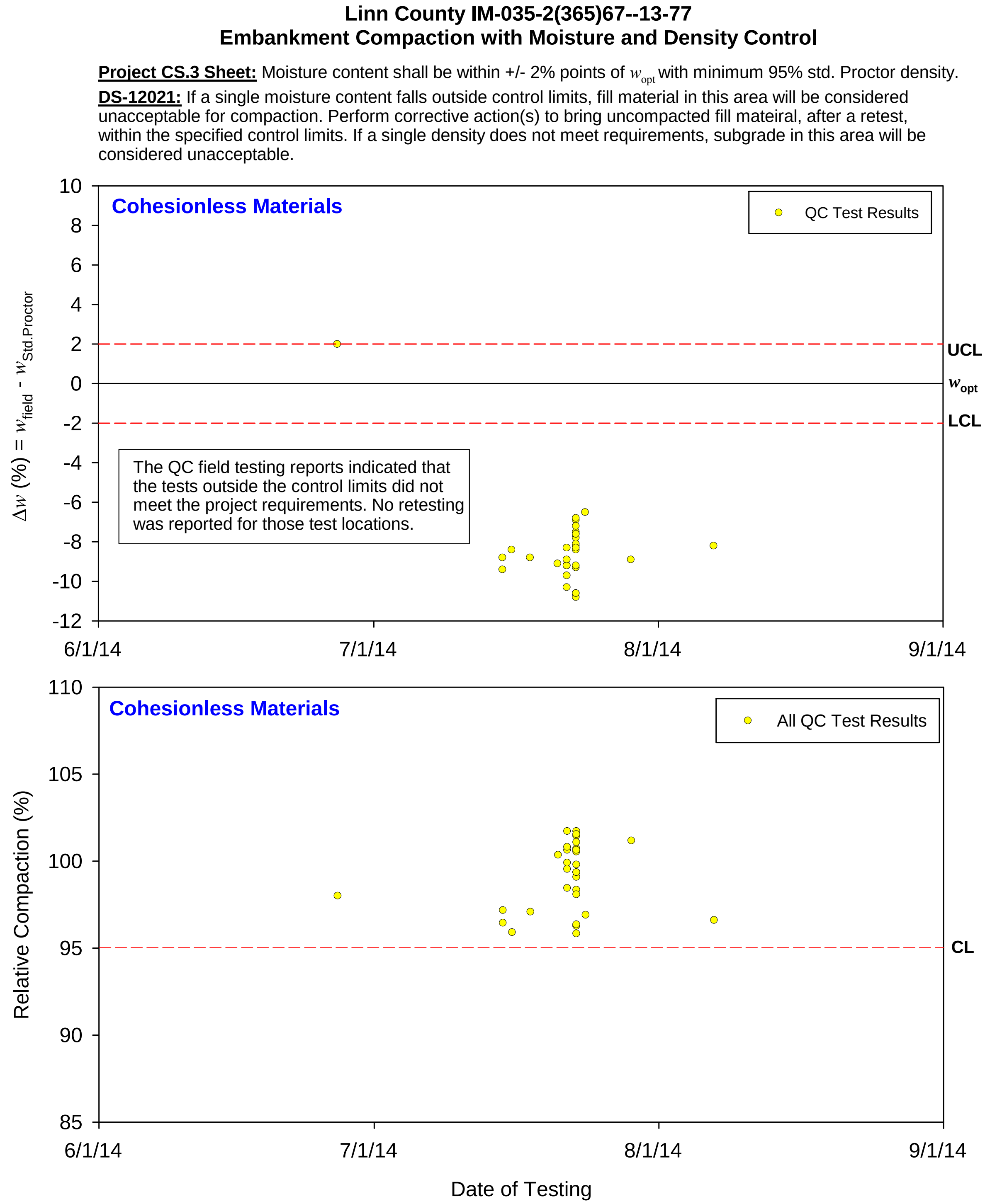


Figure 67. Linn County Project 3: Moisture and density control charts (cohesionless materials)

Linn County IM-035-2(365)67--13-77
Embankment Compaction with Moisture Control

Project CS.3 Sheet: Moisture content shall be within +/- 2% points of w_{opt} for class 10 fill and granular backfill.
DS-12021: If a single moisture content falls outside control limits, fill material in this area will be considered unacceptable for compaction. Perform corrective action(s) to bring uncompacted fill mateiral, after a retest, within the specified control limits.

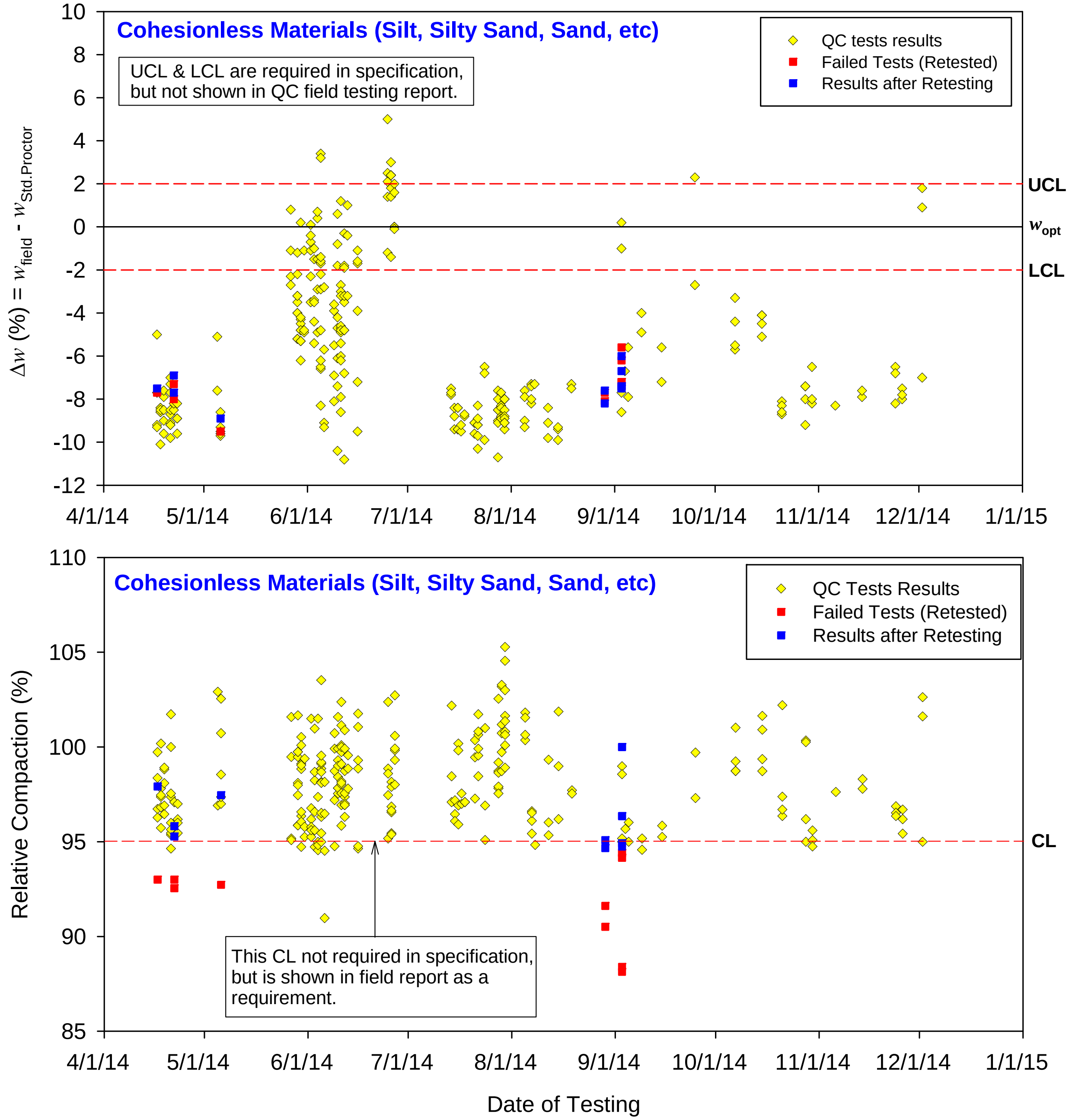


Figure 68. Linn County Project 3: Moisture control chart (cohesionless materials)

The control chart data are presented as histograms in Figure 69, Figure 70, and Figure 71.

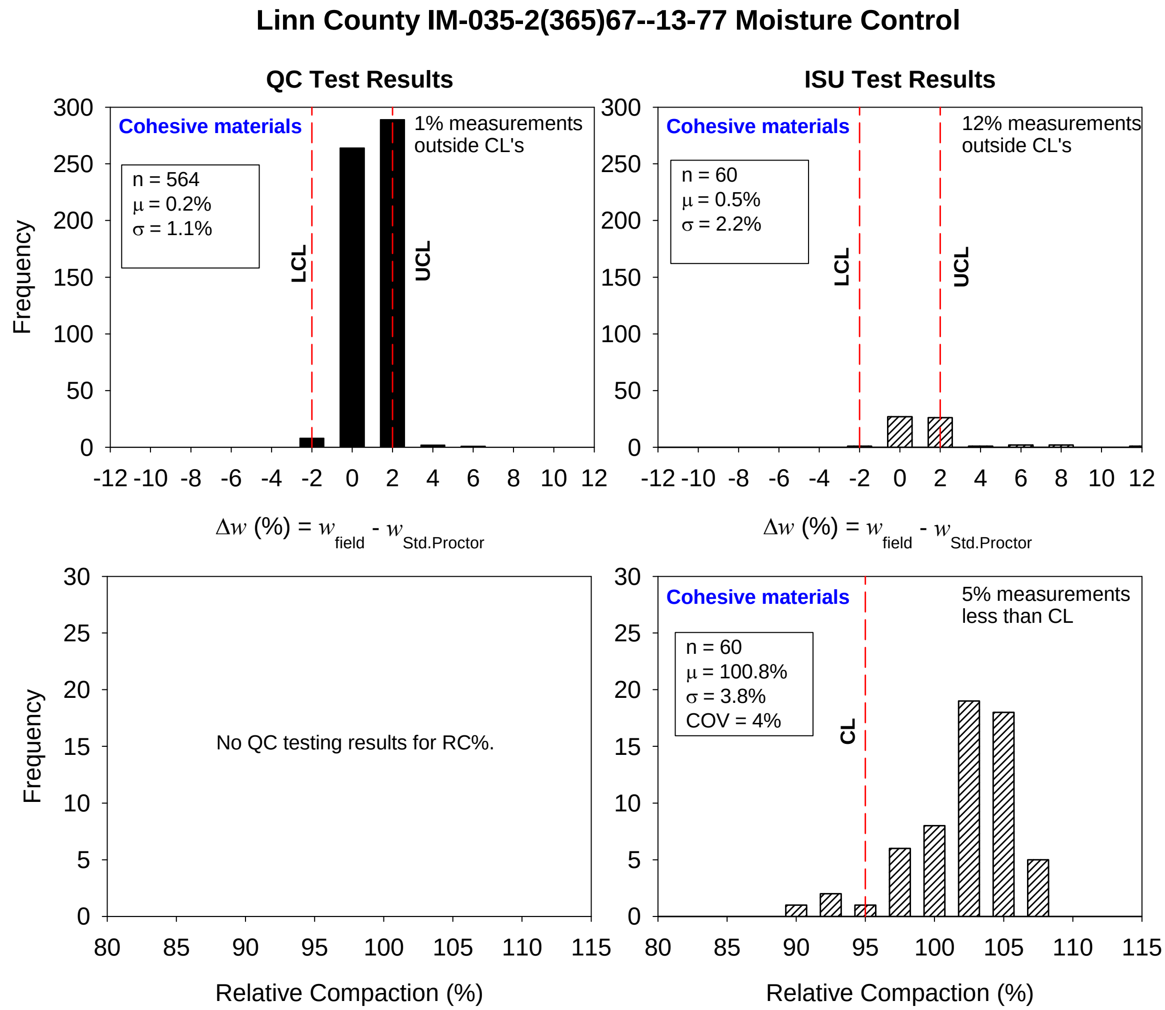


Figure 69. Linn County Project 3: Histograms of moisture and density control results (cohesive materials)

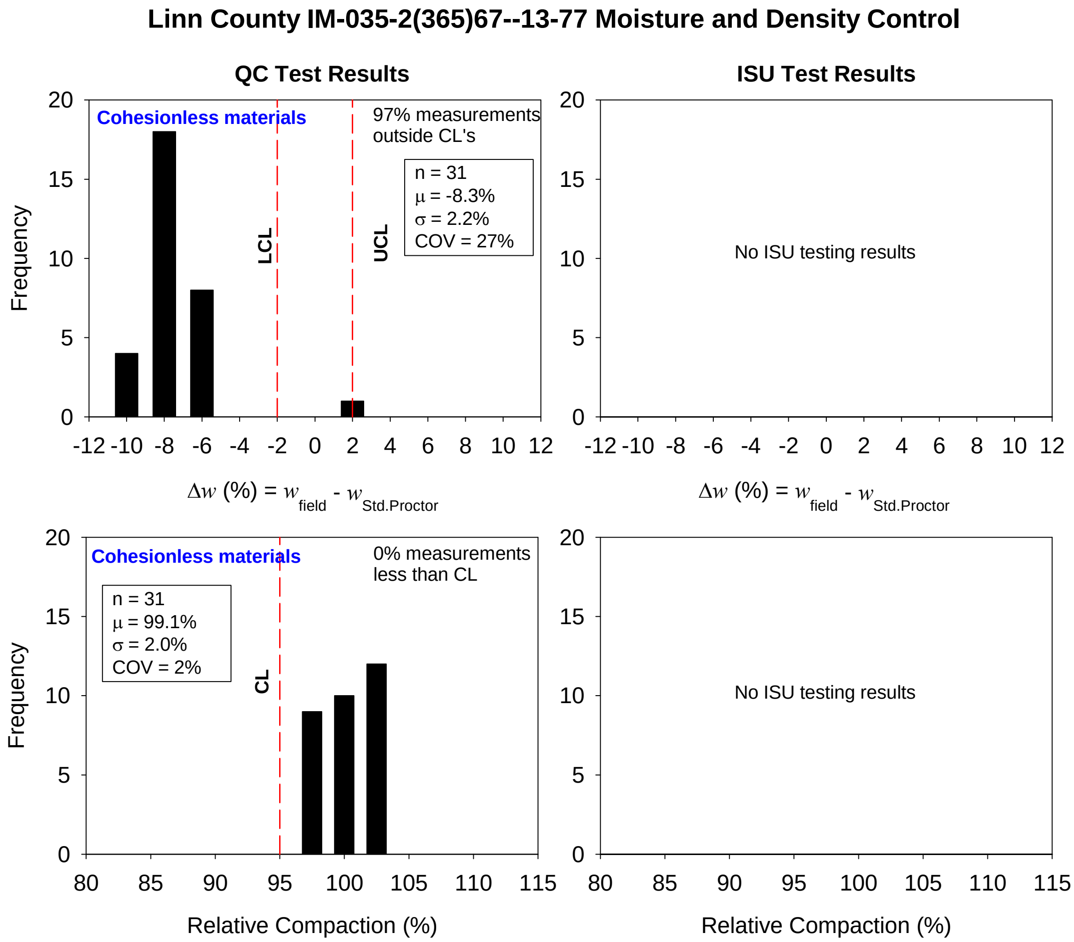


Figure 70. Linn County Project 3: Histograms of moisture and density control results (cohesionless materials)

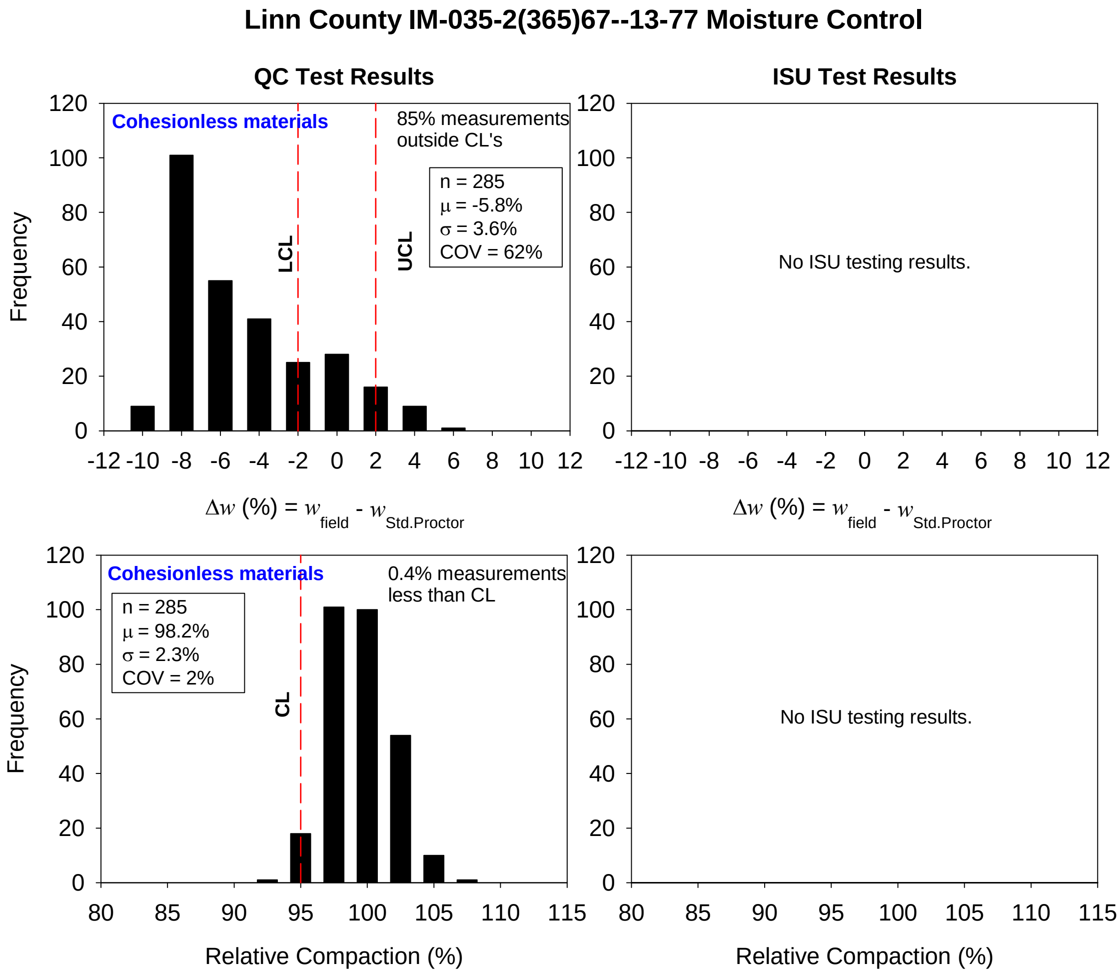
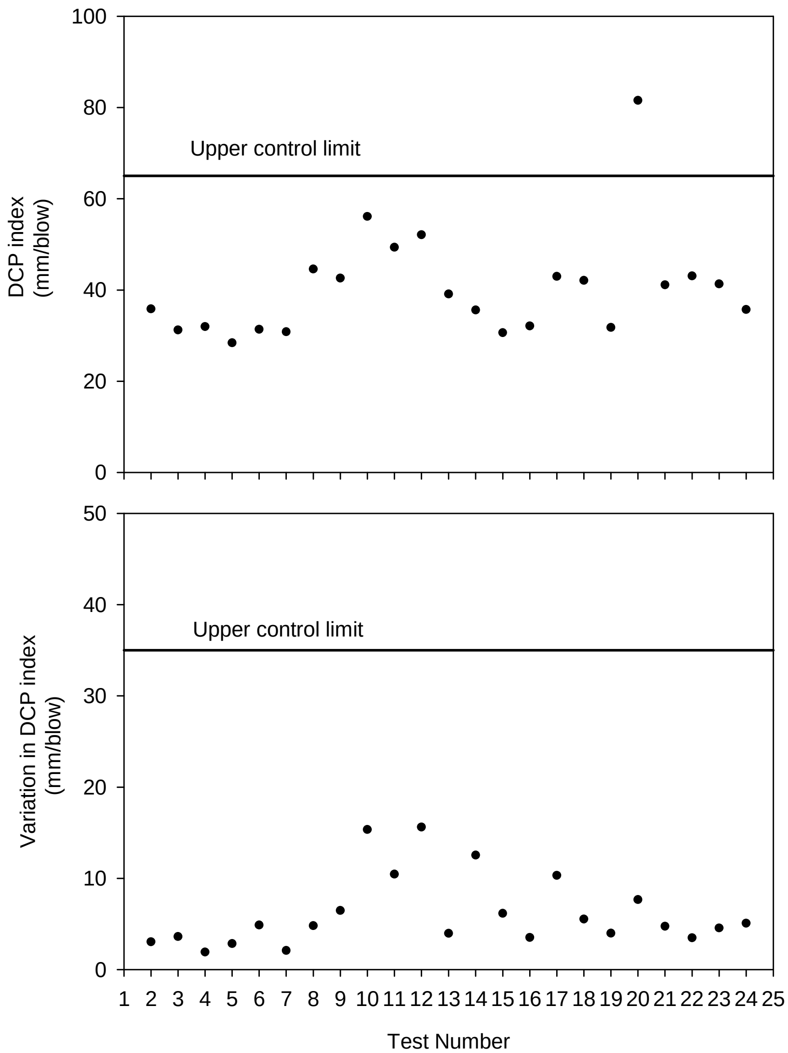


Figure 71. Linn County Project 3: Histograms of moisture control results (cohesionless materials)

The data presented in the control charts and histograms indicate that 99% of the QC data for cohesive materials fell within the moisture control limits, and all QC data for cohesionless materials showed relative compaction > 95%, with only 3% of the data falling within the moisture control limits. For the moisture control–only project, 15% of the data fell within the moisture control limits. The ISU testing results show that 95% of the data showed relative compaction > 95%, and only 88% of the data were within the moisture control limits for cohesive materials.

Figure 72 shows control charts for DCP index values at a depth of 600 mm.

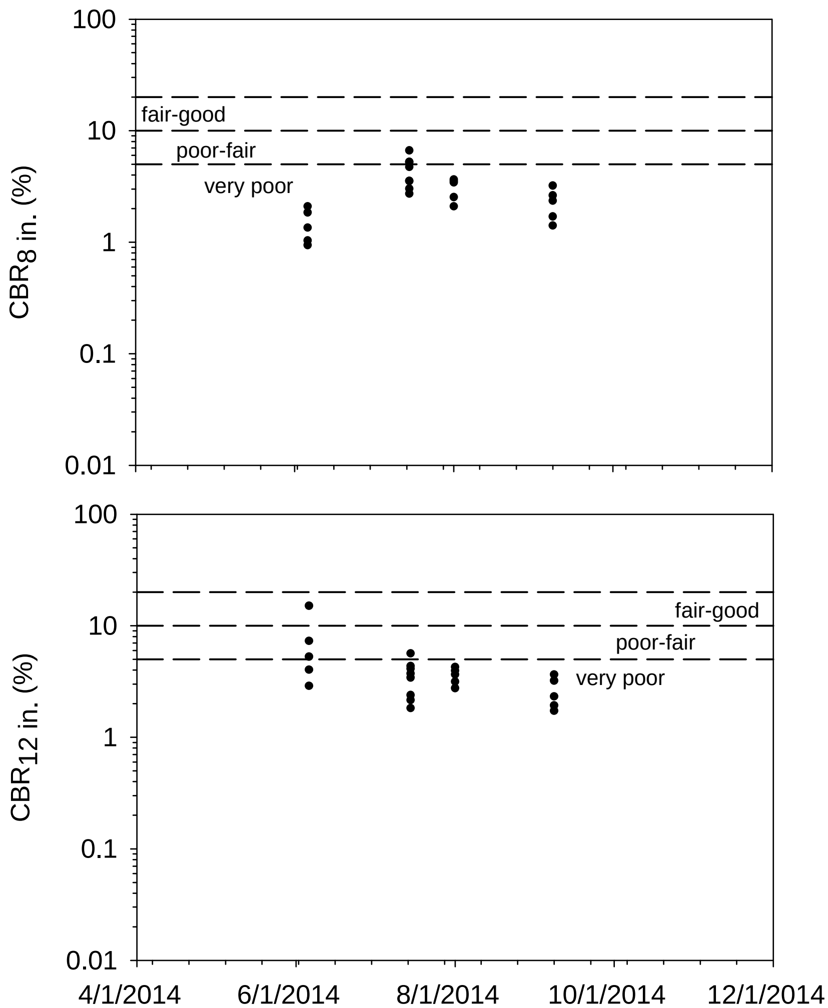


White et al. 2007

Figure 72. Linn County Project 3: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged from 28.4 to 81.5 mm/blow, and one point of all of the data exceeded the upper control limit. The variation in the DCP index control chart shows that DCP index variation fell between 1.9 and 15.6 mm/blow.

Figure 73 shows control charts for CBR values for the top 8 and 12 in. of the compacted fills.



SUDAS 2013

Figure 73. Linn County Project 3: CBR chart with CBR quality rating

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicate that 87% of the $CBR_{8in.}$ and 83% of the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 4. Linn County-79

Overview

The ISU research team conducted field testing at this grading project site on 06/06/14. The fill materials obtained at the time of testing consisted of weathered loess materials and were classified as A-4(1) by the AASHTO Soil Classification System and CL-ML by the USCS.

At this site, the project specification required achievement of 95% relative compaction and moisture content within ±2.0% of the optimum moisture content determined from the standard

Proctor test for cohesionless materials, and the specification only required achievement of moisture content within ±2.0% of the optimum moisture content for cohesive materials. The equipment used during construction is shown in Figure 74 through Figure 79.

Figure 74. Lynn County Project 4: Caterpillar 740 dump truck used to place loose fill materials

Figure 75. Linn County Project 4: Sheepsfoot roller used for soil compaction

Figure 76. Linn County Project 4: Contractor conducting QC tests

Figure 77. Linn County Project 4: Iowa DOT engineer conducting QA tests

Figure 78. Linn County Project 4: ISU in situ drive cylinder test

Figure 79. Linn County Project 4: Disc cultivator used to dry embankment materials

A disc was used to break down and aerate the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the pull-behind sheepsfoot roller (Figure 75). The contractor QC, Iowa DOT QA, and ISU testing processes are shown in Figure 76, Figure 77, and Figure 78, respectively.

ISU Field Test Results

In situ moisture content and dry density test results are compared with laboratory Proctor test results in Figure 80.

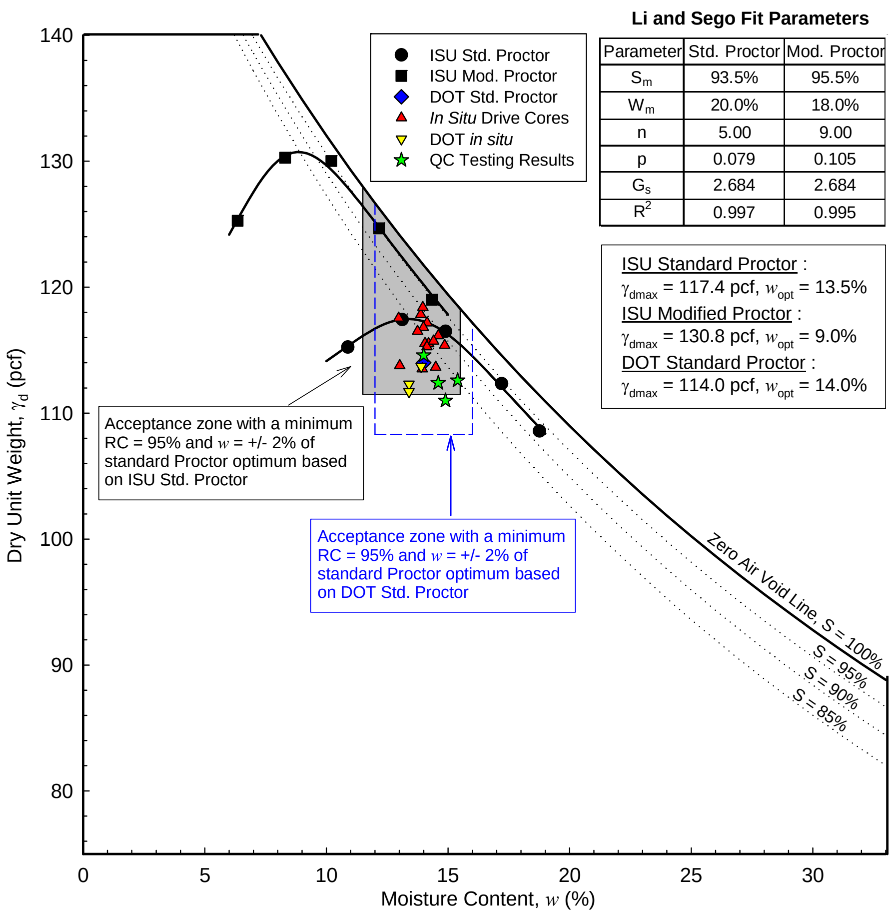


Figure 80. Linn County Project 4: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

The Proctor test results used by the Iowa DOT showed optimum moisture contents about 0.5% lower than those determined from ISU testing. Similarly, the Proctor test results used by the Iowa DOT showed maximum dry densities about 3.4 lb/ft^3 higher than those determined from ISU testing.

To determine whether the field measurements met the specification requirements, Figure 75 also shows an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density. Maximum dry density, optimum moisture content, and the acceptance zone used by the Iowa DOT at the time of ISU testing is also shown in the figure for reference and comparison.

Field test results indicate that the relative compaction of the material was over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -0.5% and +1.4% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 80 indicate that all contractor QC, Iowa DOT QA, and ISU test results fell within the specification limit. DCP-CBR values and cumulative blows with depth profiles are shown in Figure 81.

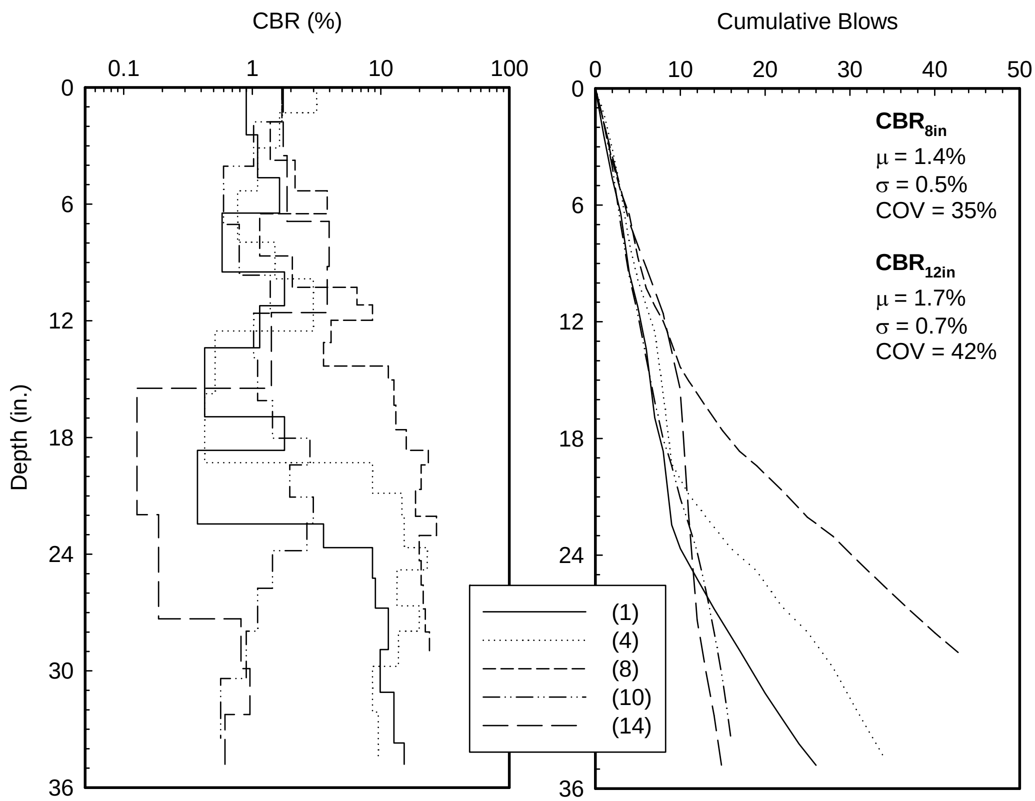


Figure 81. Linn County Project 4: DCP-CBR values and cumulative blows with depth profiles

The average CBR value in the top 8 in. was 3.7%, and the average CBR value in the top 12 in. was 4.1%.

Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized in Table 19.

Table 19. Linn County Project 4: Summary of field testing results

Parameter	Linn 79 County 8/4/2014
Relative Compaction	
Average Relative compaction (%)	103.8
Range of Relative compaction (%)	101.6 to 106.0
Standard Deviation (%)	0.01
COV (%)	1
$\Delta w\% = w_{field}\% - w_{opt}\%$	
Average Δw (%)	0.5
Range of Δw (%)	-0.5 to +1.4
Standard Deviation (%)	0.01
COV (%)	97
$CBR_{8\ in.}$	
Average CBR at 8 in. (%)	3.7
Range of CBR at 8 in. (%)	2.9 to 4.6
Standard Deviation (%)	0.7
COV (%)	20
$CBR_{12\ in.}$	
Average CBR at 12 in. (%)	4.1
Range of CBR at 12 in. (%)	3.0 to 5.1
Standard Deviation (%)	1.0
COV (%)	24

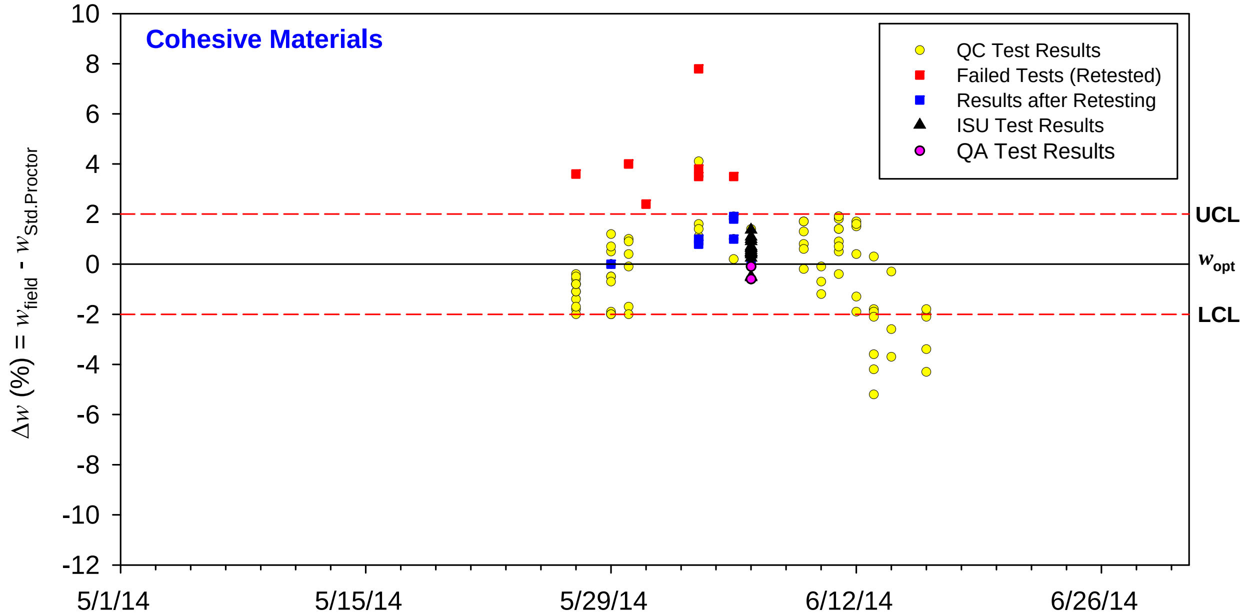
Control Charts

The contractor QC data and ISU data are reported in Figure 82 and Figure 83 in the form of control charts monitoring the dry unit weight and moisture content of the compacted fills.

Linn County NHSX-100-1(79)--3H-57
Embankment Compaction with Moisture Control

Project CS.3 Sheet: Moisture content shall be within +/- 2% points of w_{opt} for all Class 10 fill.

DS-12021: If a single moisture content falls outside control limits, fill material in this area will be considered unacceptable for compaction. Perform corrective action(s) to bring uncompacted fill mateiral, after a retest, within the specified control limits.



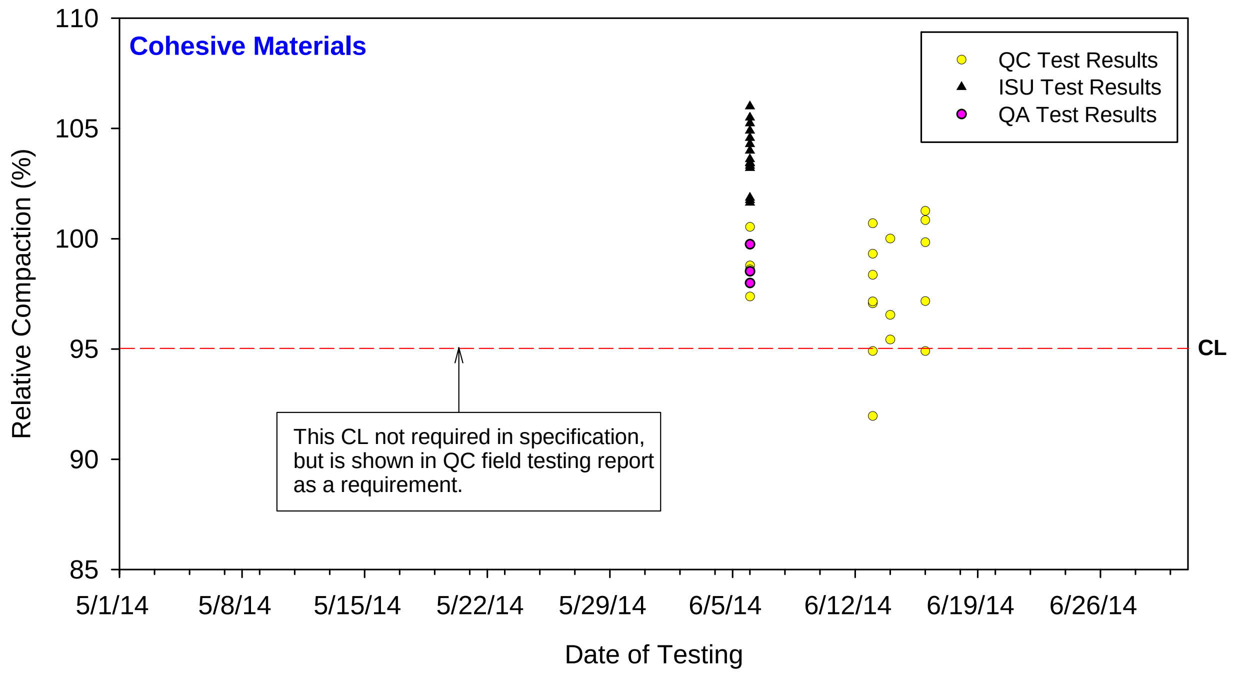


Figure 82. Linn County Project 4: Moisture control chart (cohesive materials)

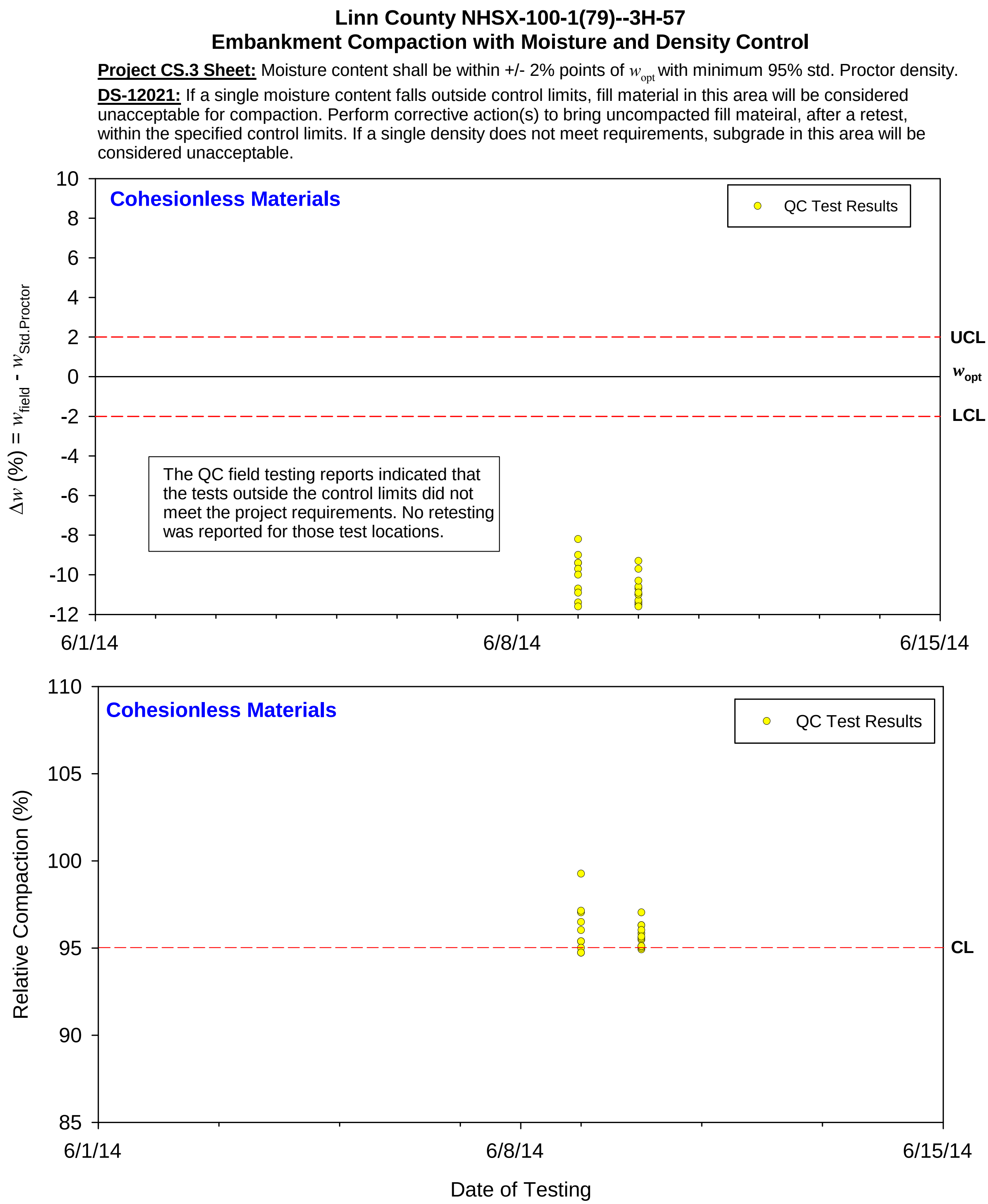


Figure 83. Linn County Project 4: Moisture and density control chart (cohesionless materials)

The control chart data are presented as histograms in Figure 84 and Figure 85.

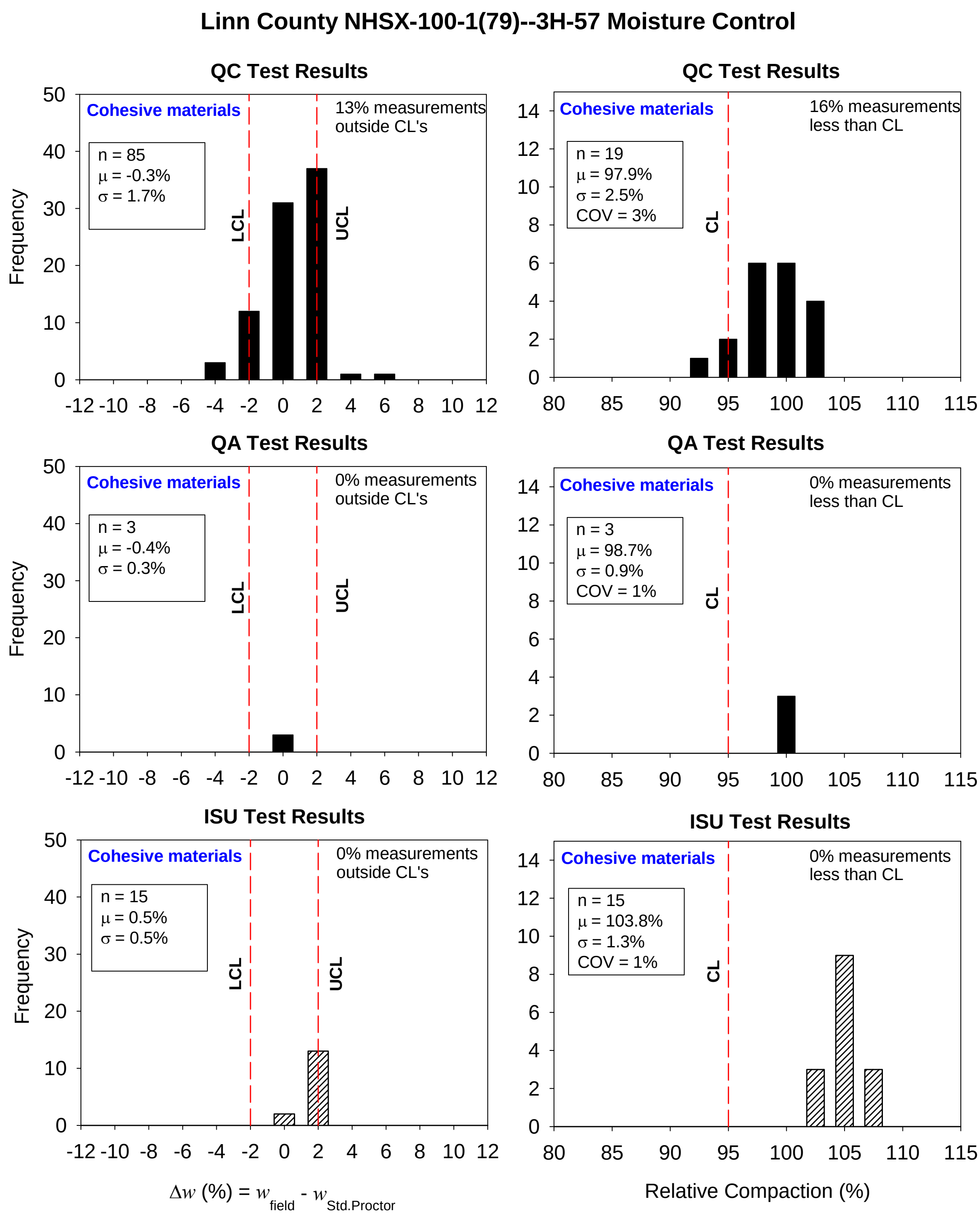


Figure 84. Linn County Project 4: Histograms of moisture and density control results (cohesive materials)

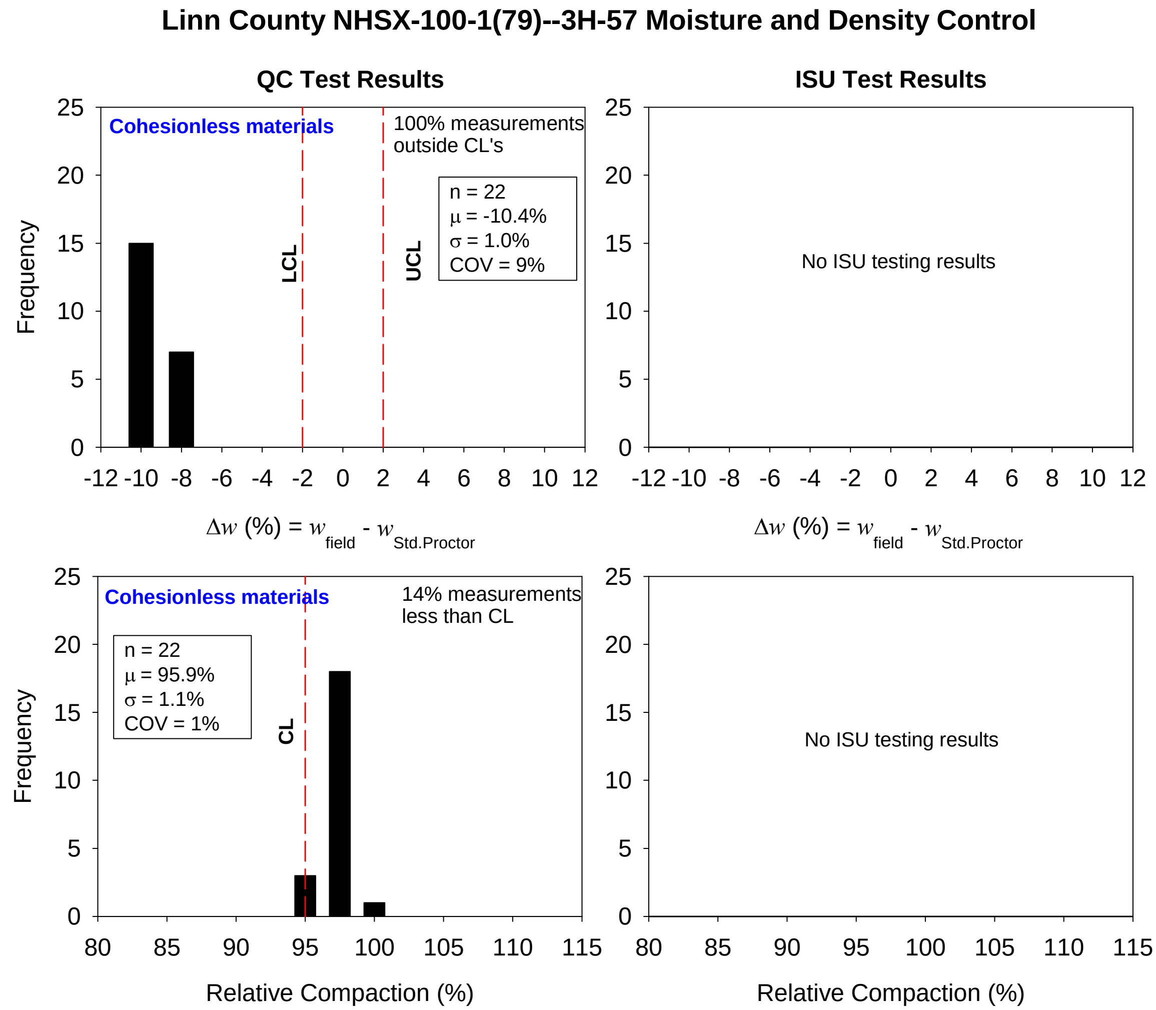
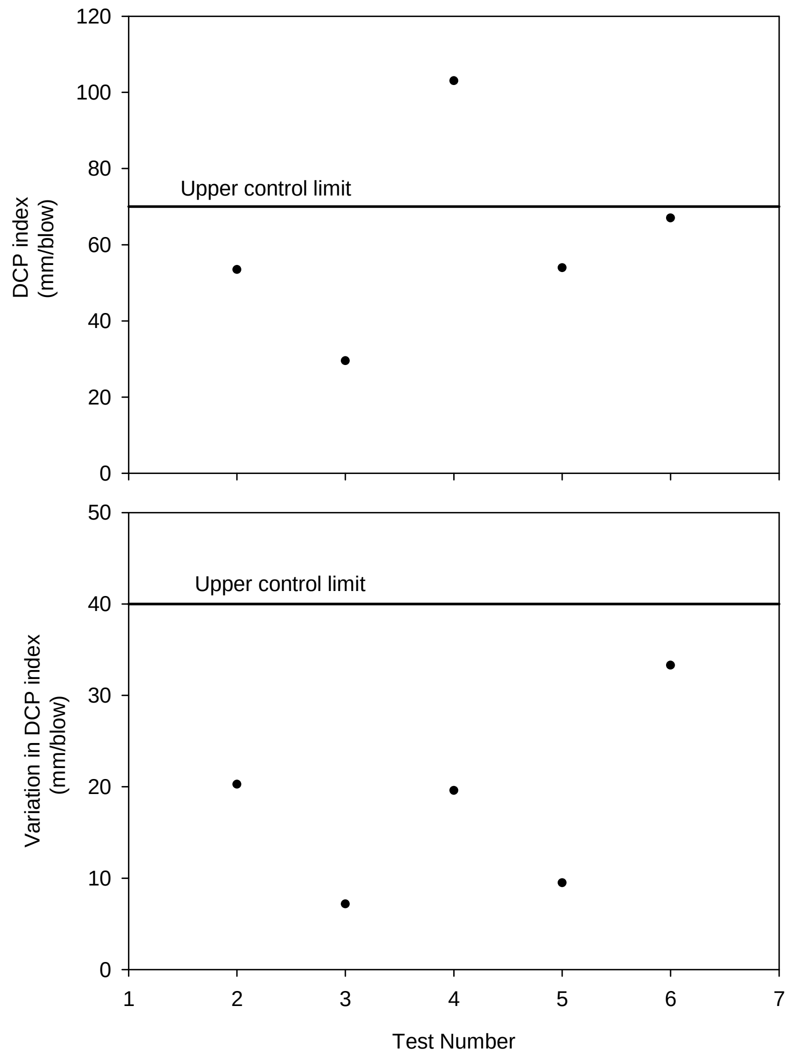


Figure 85. Linn County Project 4: Histograms of moisture and density control results (cohesionless materials)

The data presented in the control charts and histograms indicate that 84% of the QC data showed relative compaction > 95%, and a majority (87%) of the data fell within the moisture control limits for cohesive materials. For cohesionless materials, 86% of the QC data showed relative compaction > 95%, but all of the moisture measurements were dry of the moisture control limits. All of the DOT QA data met the moisture and density specifications for cohesive materials. The ISU testing results show that all data showed relative compaction > 95%, and all data were within the moisture control limits for cohesive materials.

Figure 86 shows control charts for DCP index values at a depth of 600 mm.

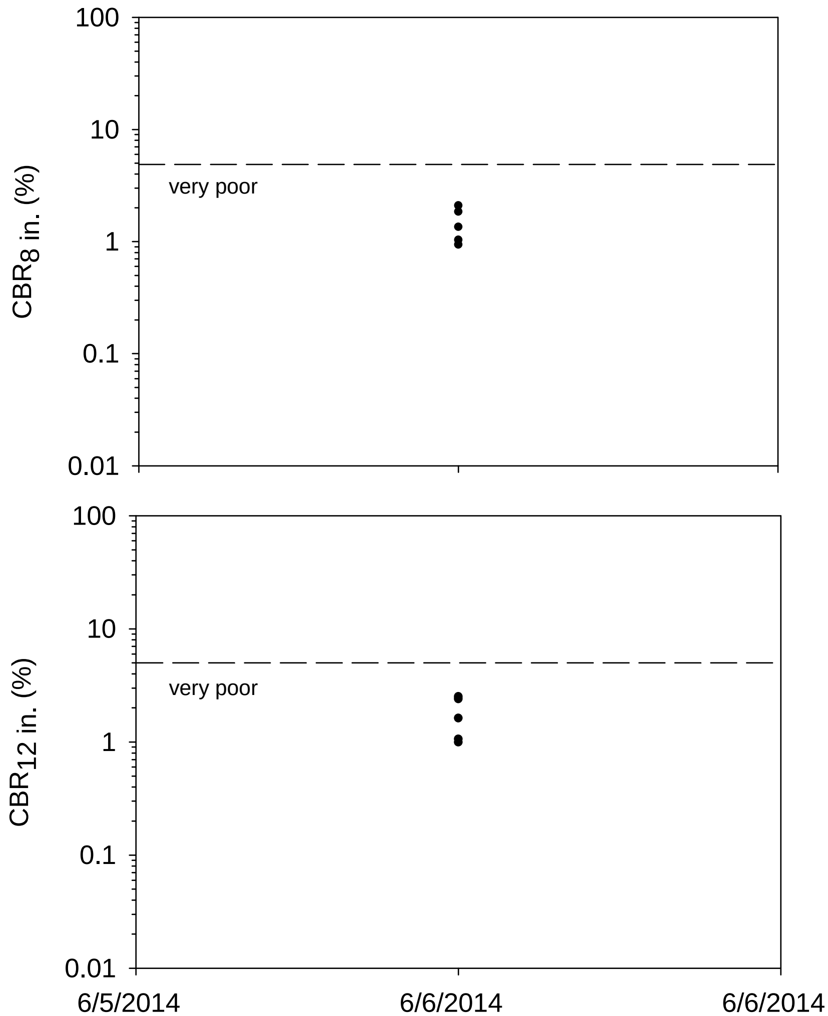


White et al. 2007

Figure 86. Linn County Project 4: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged from 29.5 to 103.0 mm/blow, and one point of all data exceeded the control limit. The variation in the DCP index control chart shows that DCP index variation fell between 7.2 and 33.3 mm/blow.

Figure 87 shows control charts for CBR values for the top 8 and 12 in. of the compacted lift.



SUDAS 2013

Figure 87. Linn County Project 4: CBR chart with CBR quality rating

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicate that all of the $CBR_{8in.}$ and $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 5. Mills County

Overview

The ISU research team conducted field testing at this grading project site on 06/26/14. The fill materials obtained at the time of testing consisted of loess and were classified as A-4(6) and A-4(7) by the AASHTO Soil Classification System and CL-ML by the USCS.

At this project site, the project specification required achievement of moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test. The equipment used during construction is shown in Figure 88 through Figure 90.

Figure 88. Mills County Project 5: Caterpillar 621E scraper used to collect and place loose fill materials

Figure 89. Mills County Project 5: Caterpillar D6R dozer used to control lift thickness

Figure 90. Mills County Project 5: Sheepsfoot roller used for soil compaction

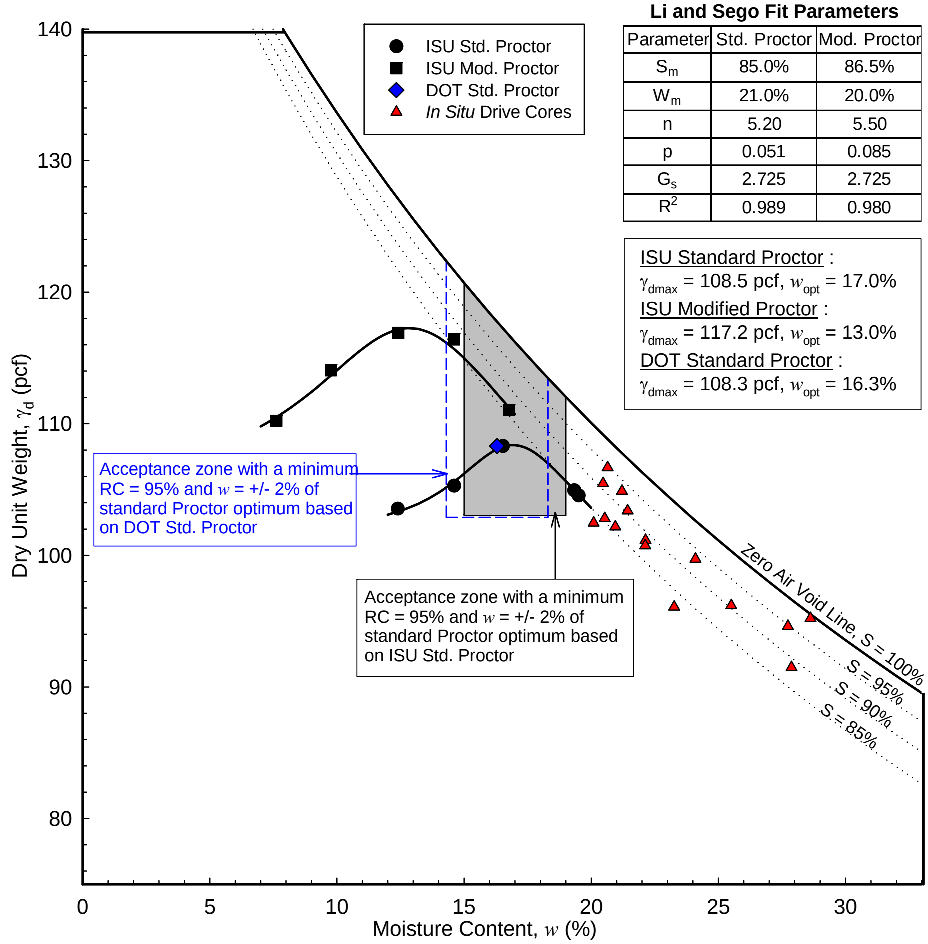
Disc was not used to break down and aerated the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the pull-behind sheepsfoot roller (Figure 90).

A wet area in the center of the construction site was observed (Figure 91).

Figure 91. Mills County Project 5: Very wet materials in the center of the construction site

ISU Field Test Results

In situ moisture content and dry density test results are compared with laboratory Proctor test results in Figure 92 and Figure 93.



Li and Sego Fit Parameters

Parameter	Std. Proctor	Mod. Proctor
S_m	85.0%	86.5%
W_m	21.0%	20.0%
n	5.20	5.50
p	0.051	0.085
G_s	2.725	2.725
R^2	0.989	0.980

Figure 92. Mills County Project 5 TB1: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

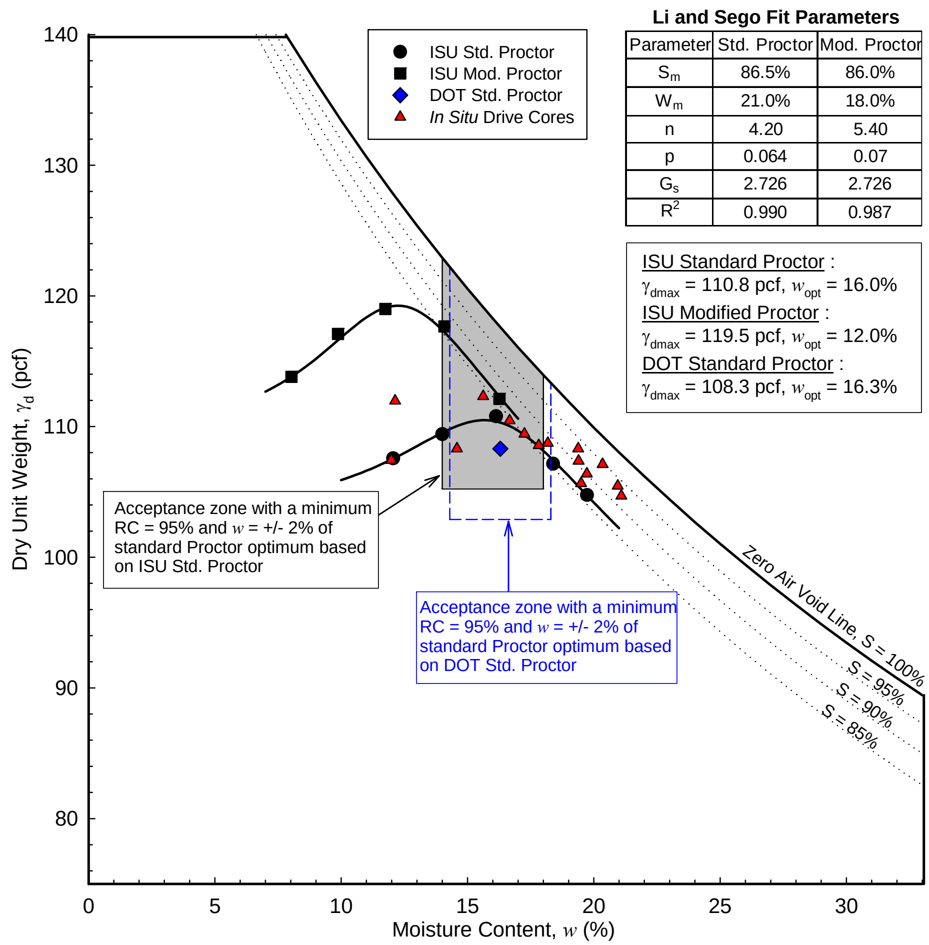


Figure 93. Mills County Project 5 TB2: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

The Proctor test results of TB1 used by the Iowa DOT showed optimum moisture contents about 0.7% lower than those determined from ISU testing, and the Proctor test results of TB2 used by the Iowa DOT showed optimum moisture contents about 0.3% higher than those determined from ISU testing. Similarly, the Proctor test results used by the Iowa DOT showed maximum dry densities about 0.2 to 1.5 lb/ft^3 lower than those determined from ISU testing.

To determine whether the field measurements met the specification requirements, Figures 87 and 88 also show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density. Maximum dry density, optimum moisture content, and the acceptance zone used by the Iowa DOT at the time of ISU testing are also shown in the figures for reference and comparison.

Field test results indicate that the relative compaction of the material ranged from approximately 84.3% to over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -4.0% and +11.6% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 92 and Figure 93 indicate that a majority of the ISU tests on TB1 and TB2 fell outside the specification limit, with material generally > 2% wet of optimum moisture content and close to the 95% to 100% saturation line.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 94 and Figure 95 for the two TBs.

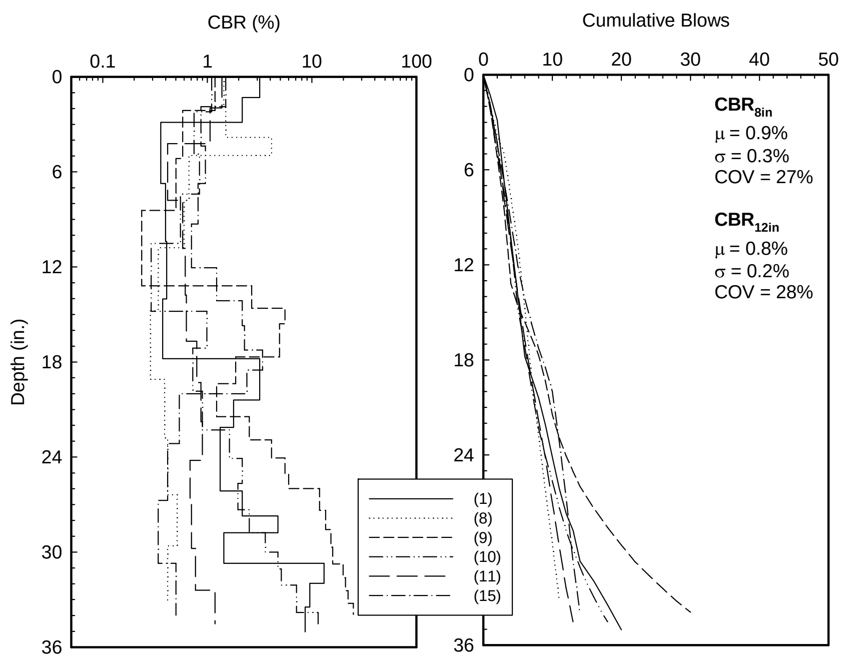


Figure 94. Mills County Project 5 TB1: DCP-CBR values and cumulative blows with depth profiles

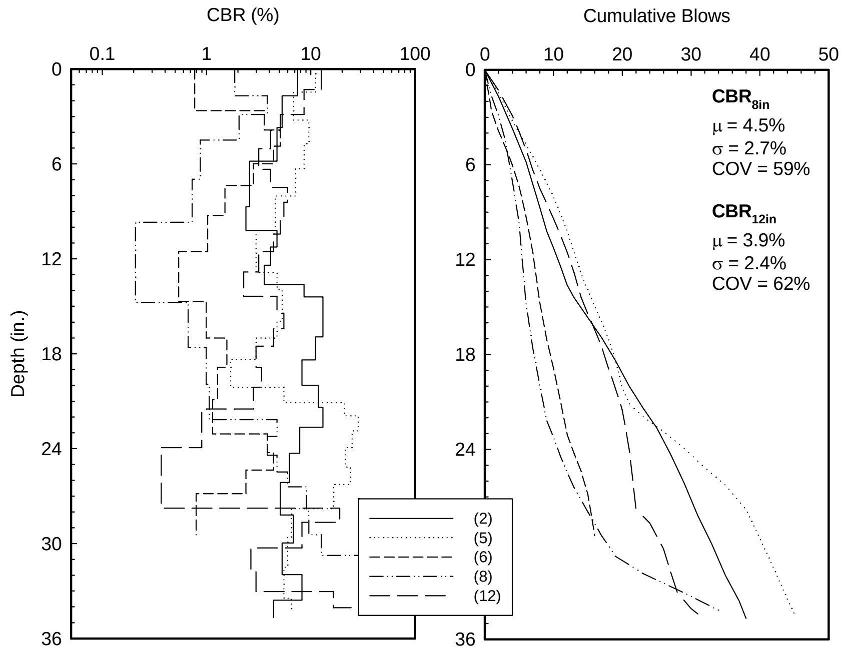


Figure 95. Mills County Project 5 TB2: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. varied between 2.9% and 6.8% and the average CBR value in the top 12 in. varied between 2.6% and 6.2% between the two test beds. The results indicate that the CBR values are generally higher when the material is within the within the moisture control limit, as in the case of TB2, and vice versa, as in the case of TB 1.

Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized in Table 20.

Table 20. Mills County Project 5: Summary of field testing results

Parameter	Mills County TB1 6/26/2014	Mills County TB2 6/26/2014
Relative Compaction		
Average Relative compaction (%)	92.4	97.6
Range of Relative compaction (%)	84.3 to 98.3	94.5 to 101.4
Standard Deviation (%)	0.04	0.02
COV (%)	4	2
$\Delta w\% = w_{field}\% - w_{opt}\%$		
Average Δw (%)	6.1	1.6
Range of Δw (%)	3.1 to +11.6	-4.0 to +5.1
Standard Deviation (%)	2.96	0.03
COV (%)	48	179
$CBR_{8\ in.}$		
Average CBR at 8 in. (%)	2.9	6.8
Range of CBR at 8 in. (%)	2.5 to 3.7	3.9 to 9.8
Standard Deviation (%)	0.4	2.4
COV (%)	14	35
$CBR_{12\ in.}$		
Average CBR at 12 in. (%)	2.6	6.2
Range of CBR at 12 in. (%)	2.0 to 3.1	3.2 to 8.8
Standard Deviation (%)	0.4	2.4
COV (%)	16	39

Control Charts

The contractor QC data and ISU data are reported in Figure 96 in the form of control charts monitoring the dry unit weight and moisture content of the compacted fills.

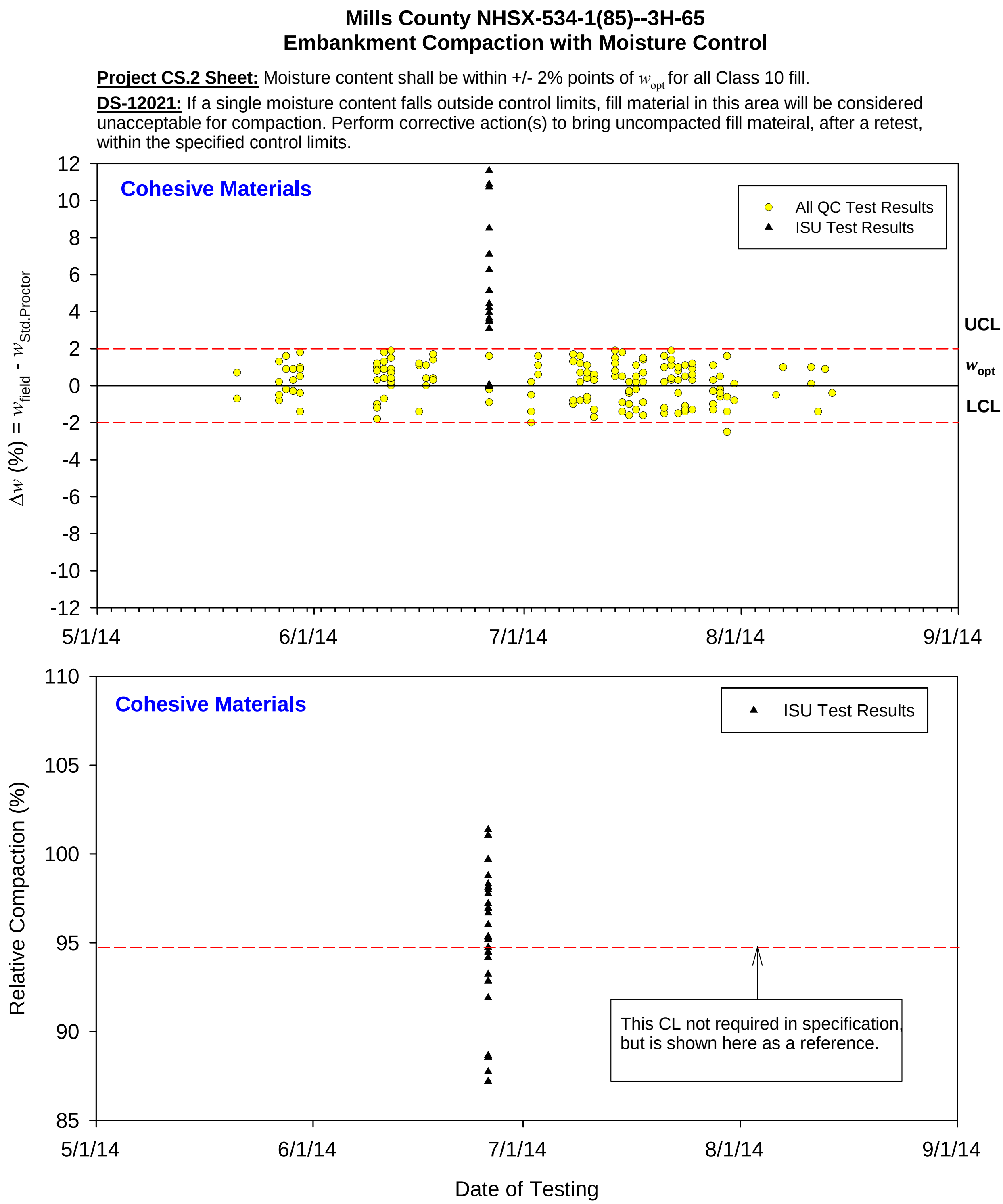


Figure 96. Mills County Project 5: Moisture control chart

The control chart data are presented as histograms in Figure 97.

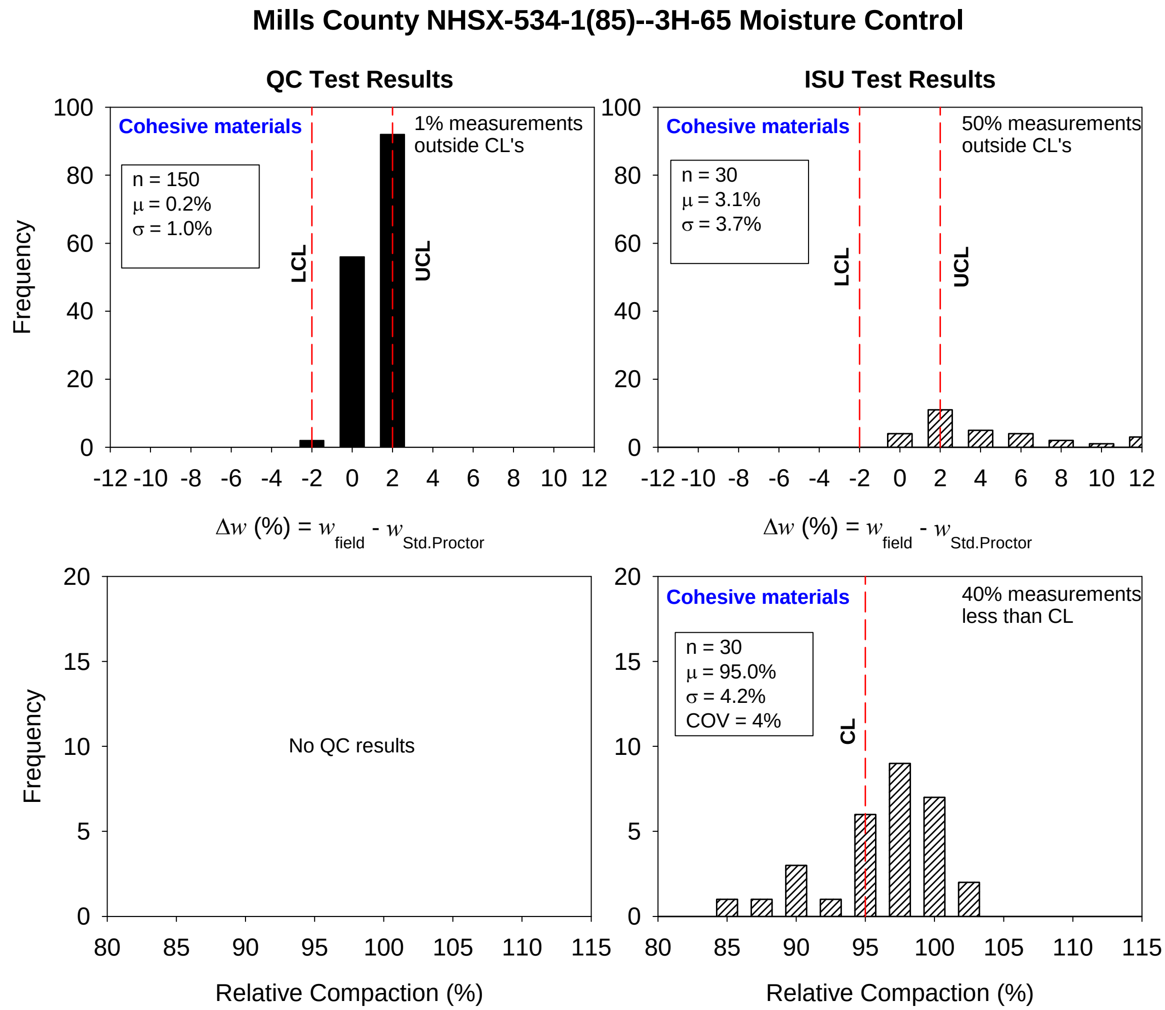
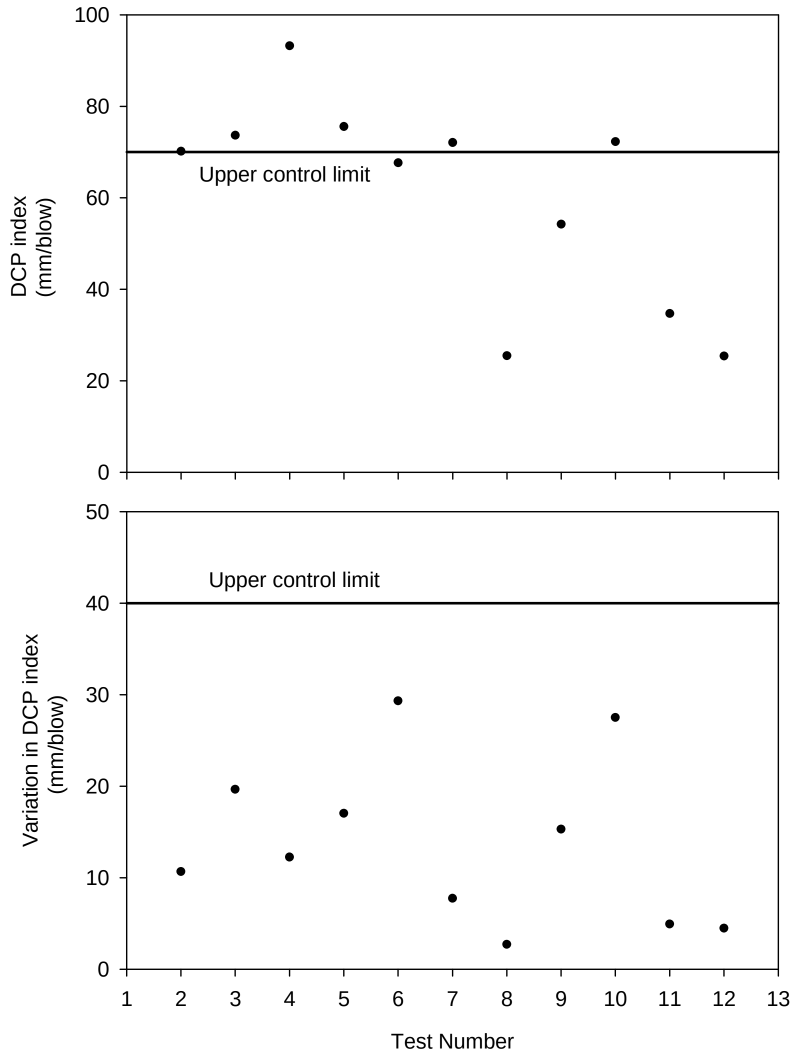


Figure 97. Mills County Project 5: Histograms of moisture and density control results

The data presented in the control charts and histograms indicate that a majority (99%) of the data fell within the moisture control limits. The ISU testing results show that 60% of the data showed relative compaction > 95%, and 50% of the data were within the moisture control limits.

Figure 98 shows control charts for DCP index values at a depth of 600 mm.

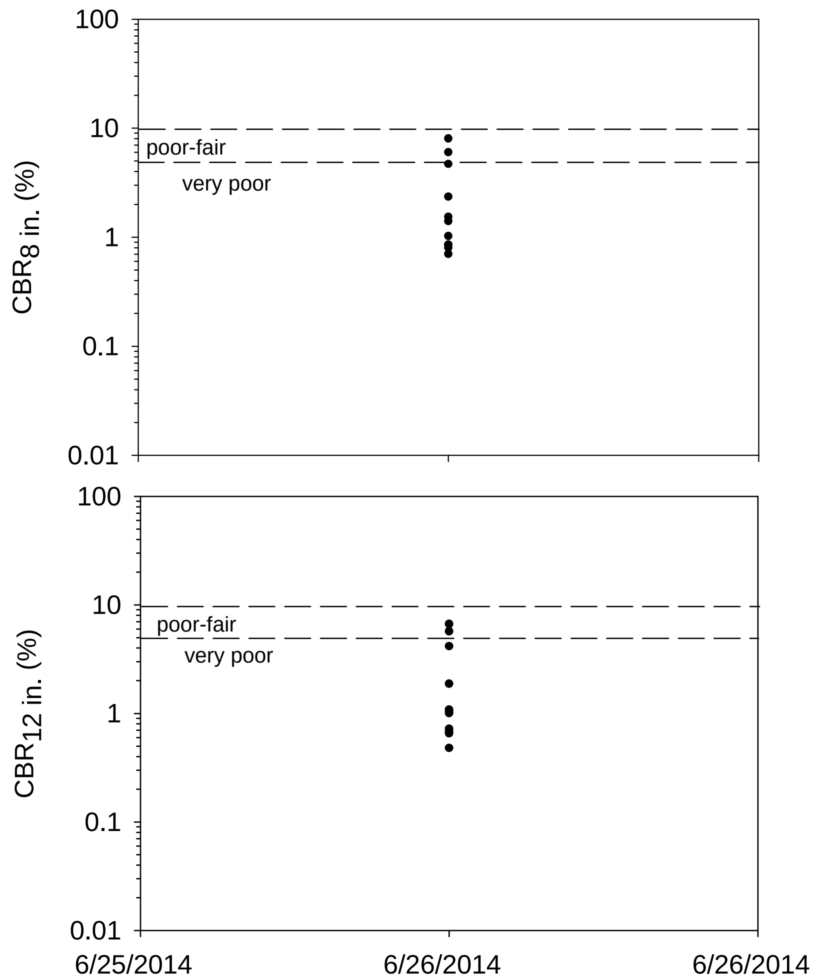


White et al. 2007

Figure 98. Mills County Project 5: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged from 25.4 to 93.2 mm/blow, and five points of all the data exceeded the upper control limit. The variation in the DCP index control chart shows that DCP index variation fell between 2.7 and 29.3 mm/blow.

Figure 99 shows control charts for CBR values for the top 8 and 12 in. of the compacted lift.



SUDAS 2013

Figure 99. Mills County Project 5: CBR chart with CBR quality rating

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicate that 82% of the $CBR_{8in.}$ and 82% of the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 6. Pottawattamie County

Overview

The ISU research team conducted field testing at this grading project site on 07/02/14 and 07/10/14. The fill materials obtained at the time of testing consisted of manufactured materials classified as A-7-6(20) and A-7-6(14) by the AASHTO Soil Classification System and CL by the USCS.

At this project site, the project specification required achievement of moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test. The equipment used during construction is shown in Figure 100 through Figure 103.

Figure 100. Pottawattamie County Project 6: Caterpillar dozer used to control lift thickness

Figure 101. Pottawattamie County Project 6: Caterpillar 851B dozer with sheepsfoot roller wheel used for soil compaction

Figure 102. Pottawattamie County Project 6: Dynapac CA250-II vibratory smooth drum roller used for soil compaction

Figure 103. Pottawattamie County Project 6: Disc cultivator used to dry embankment materials

A disc was used to break down and aerate the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the sheepsfoot roller (Figure 101).

Sheepsfoot walkout was observed during the site visits. A vibratory smooth drum roller was used to level the testing strip (Figure 102).

ISU Field Test Results

In situ moisture content and dry density test results are compared with laboratory Proctor test results in Figure 104 and Figure 105.

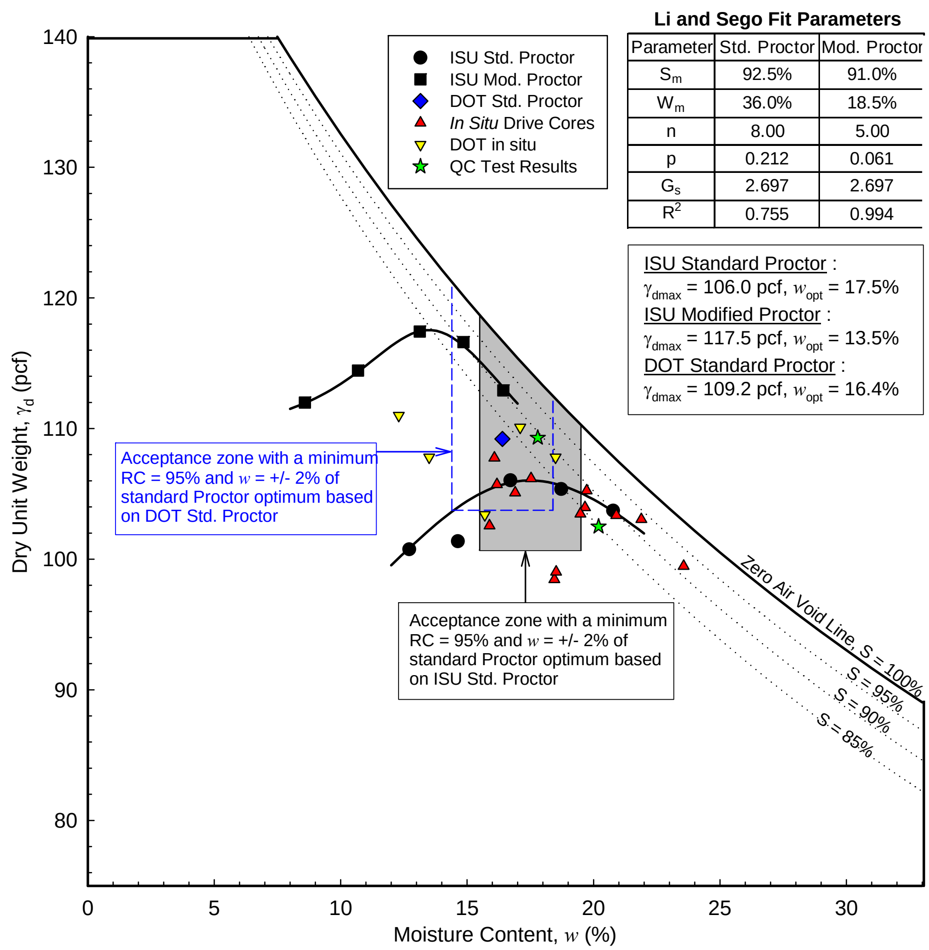


Figure 104. Pottawattamie County Project 6 TB1: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

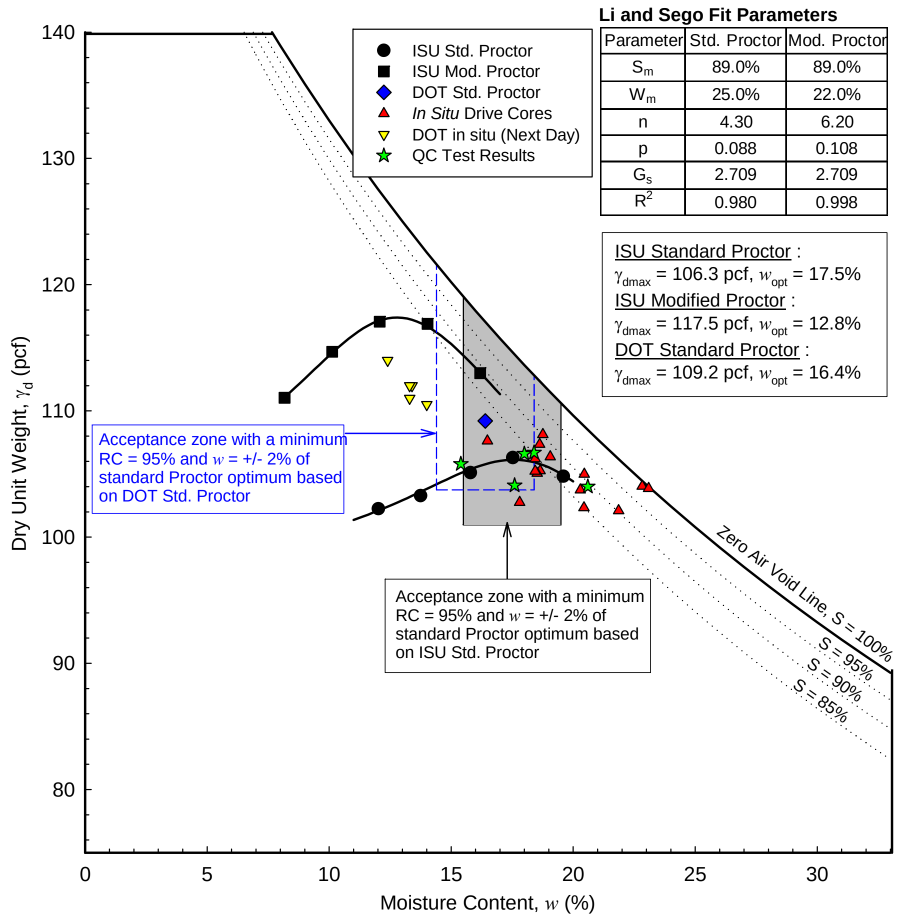


Figure 105. Pottawattamie County Project 6 TB2: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

The Proctor test results used by the Iowa DOT showed optimum moisture contents about 1.1% lower than those determined from ISU testing. Similarly, the Proctor test results used by the Iowa DOT showed maximum dry densities about 2.9 to 3.2 lb/ft^3 higher than those determined from ISU testing.

To determine whether the field measurements met the specification requirements, Figures 99 and 100 also show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density. Maximum dry density, optimum moisture content, and the acceptance zone used by the Iowa DOT at the time of ISU testing are also shown in the figures for reference and comparison.

Field test results indicate that the relative compaction of the material ranged from approximately 90.3% to over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -1.6% and +6.1% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 104 and Figure 105 indicate that 43% of the ISU test results on TB1 and TB2 fell outside the specification limit, with material generally > 2% wet of optimum moisture content. The QC test results were obtained from the contractor during the ISU testing visit. One test point did not meet the moisture specification, but there was no information available on the datasheet provided if that was retested.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 106 and Figure 107 for the two TBs.

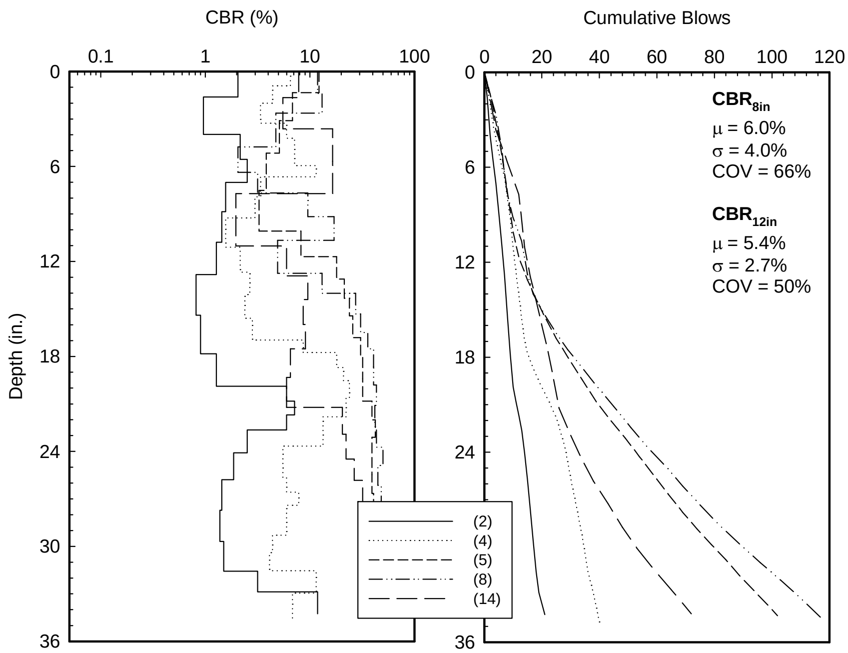


Figure 106. Pottawattamie County Project 6 TB1: DCP-CBR values and cumulative blows with depth profiles

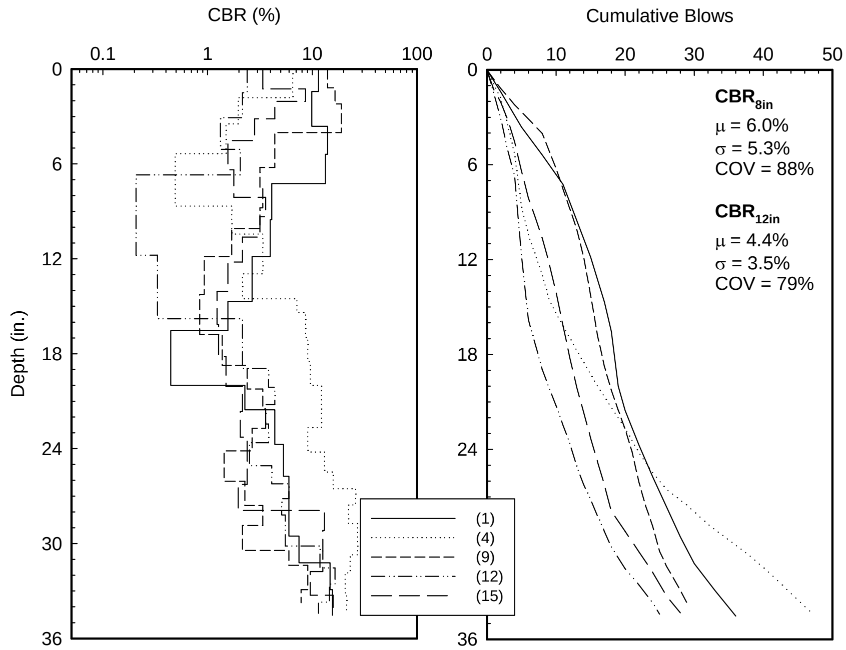


Figure 107. Pottawattamie County Project 6 TB2: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. was 6.0% and the average CBR value in the top 12 in. varied between 4.4% and 5.4% between the two test beds. The results indicate that the CBR values are generally higher when the material is within the within the moisture control limit, as in the case of TB1, and vice versa, as in the case of TB2.

Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized in Table 21.

Table 21. Pottawattamie County Project 6: Summary of field testing results

Parameter	Pottawattamie County TB1 7/2/2014	Pottawattamie County TB2 7/10/2014
Relative Compaction		
Average Relative compaction (%)	96.9	98.6
Range of Relative compaction (%)	90.3 to 101.7	95.9 to 101.5
Standard Deviation (%)	0.03	0.02
COV (%)	3	2
$\Delta w\% = w_{field}\% - w_{opt}\%$		
Average Δw (%)	1.4	1.8
Range of Δ*w* (%)	-1.6 to +6.1	-1.3 to +5.3
Standard Deviation (%)	2.23	0.02
COV (%)	162	105
$CBR_{8\ in.}$		
Average CBR at 8 in. (%)	6.0	6.0
Range of CBR at 8 in. (%)	1.7 to 12.6	1.5 to 11.8
Standard Deviation (%)	4.0	5.3
COV (%)	66	88
$CBR_{12\ in.}$		
Average CBR at 12 in. (%)	5.4	4.4
Range of CBR at 12 in. (%)	1.6 to 8.5	0.9 to 8.7
Standard Deviation (%)	2.7	3.5
COV (%)	50	79

Control Charts

The contractor QC data and ISU data are reported in Figure 108 in the form of control charts monitoring the dry unit weight and moisture content of the compacted fills.

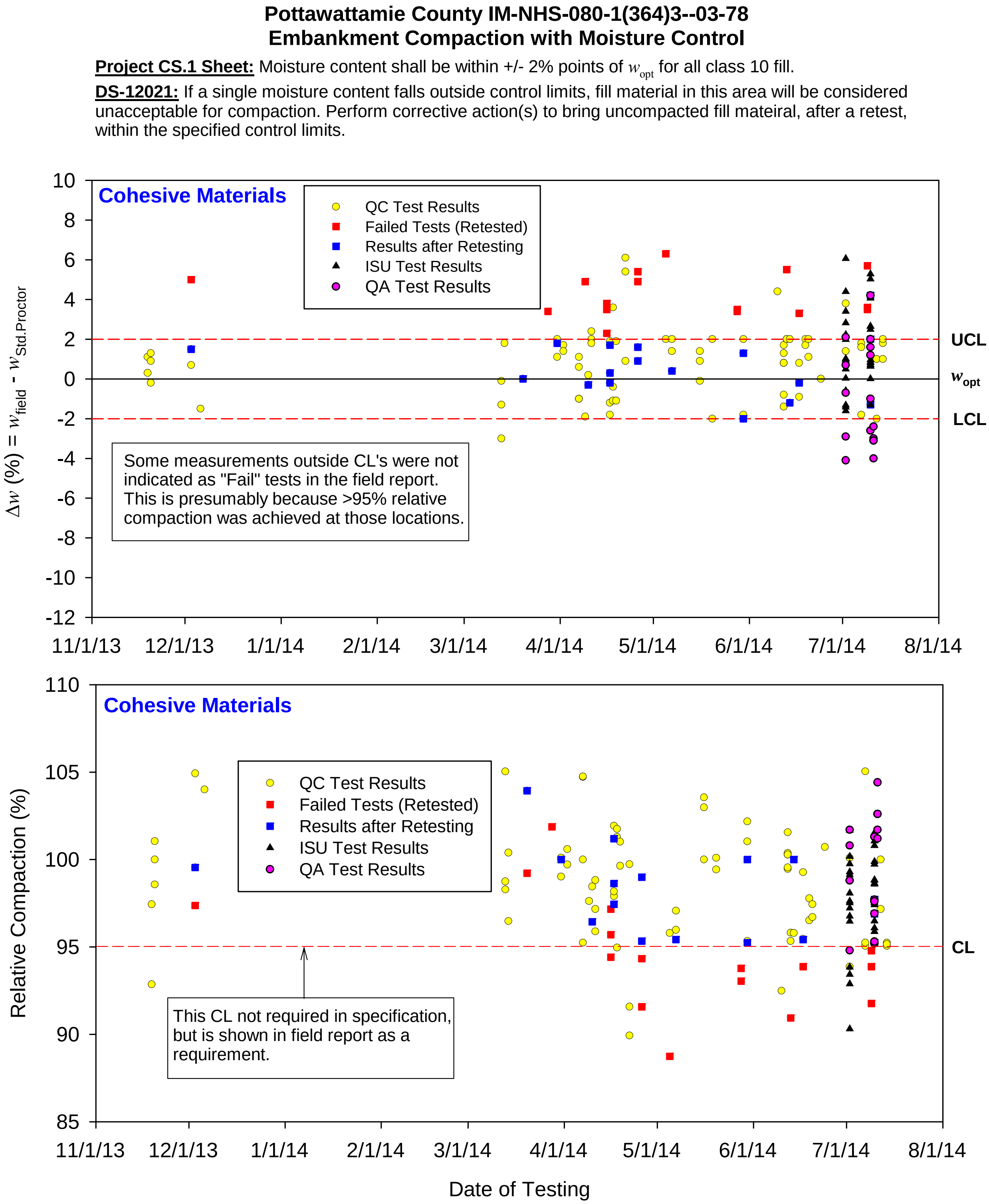


Figure 108. Pottawattamie County Project 6: Moisture control chart

The control chart data are presented as histograms in Figure 109.

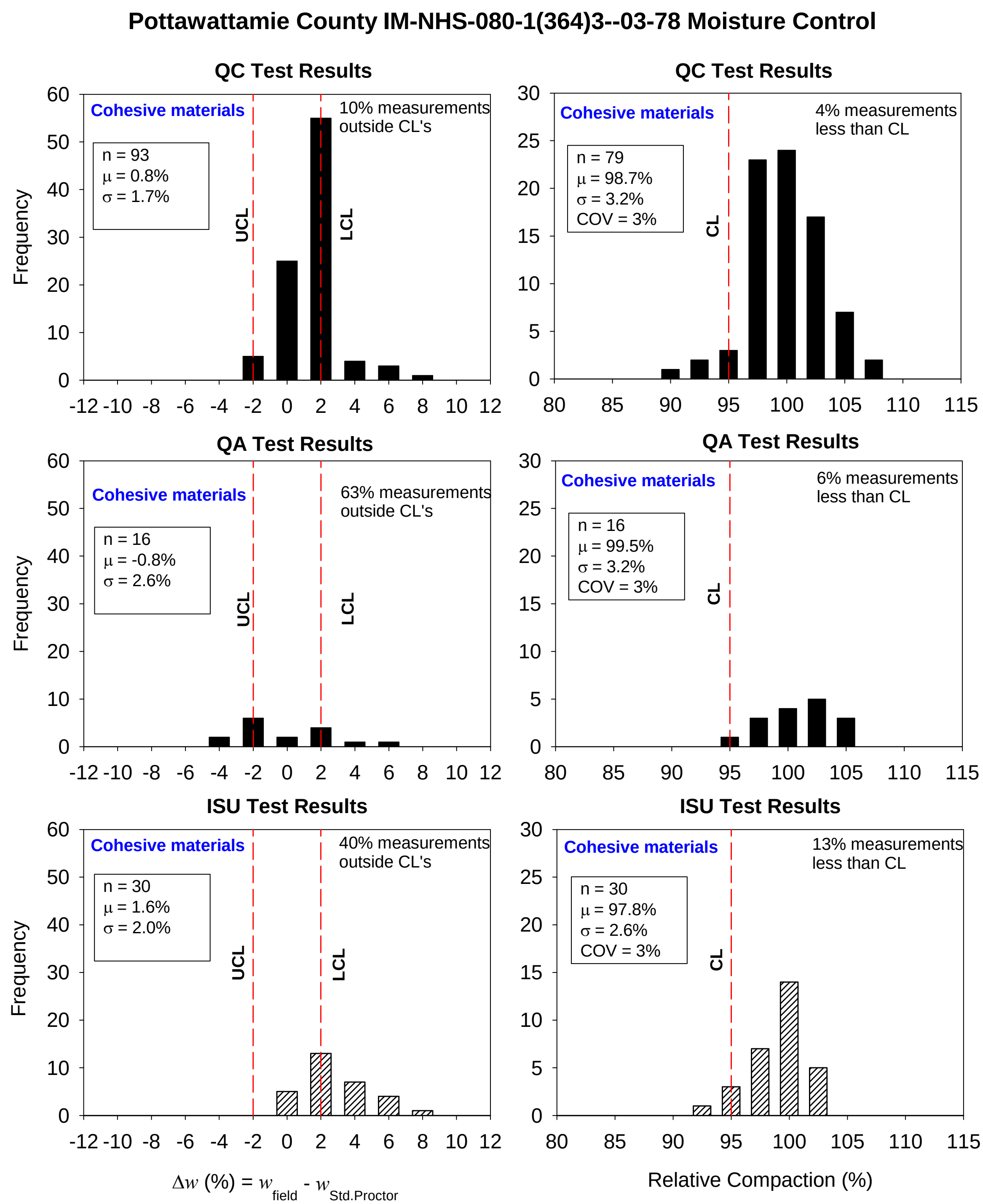
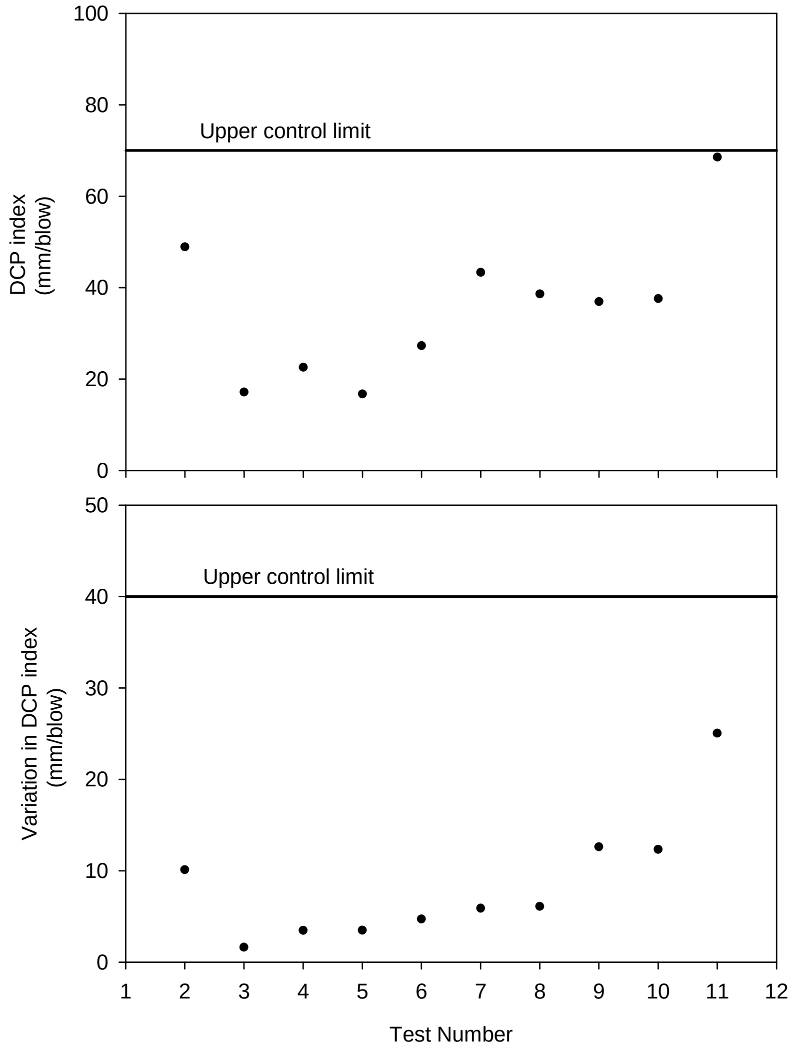


Figure 109. Pottawattamie County Project 6: Histograms of moisture and density control results

The data presented in the control charts and histograms indicate that 96% of the QC data showed relative compaction > 95%, and a majority (91%) of the data fell within the moisture control limits. QA testing results showed 37% of the data with relative compaction > 95%; and, 94% of the data fell within the moisture control limits. The ISU testing results showed 87% of the data with relative compaction > 95%; and, 60% of the data were within the moisture control limits.

Figure 110 shows control charts for DCP index values at a depth of 600 mm.

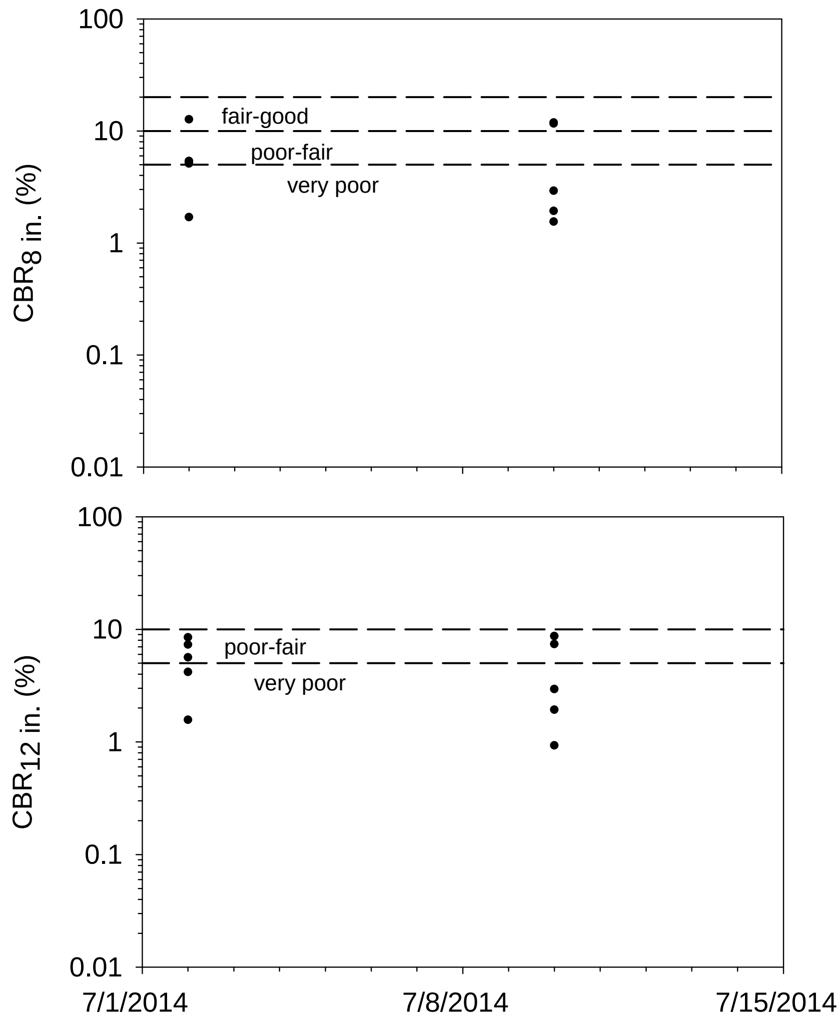


White et al. 2007

Figure 110. Pottawattamie County Project 6: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged from 16.7 to 68.5 mm/blow, and all of the data were within the control limit. The variation in the DCP index control chart shows that DCP index variation fell between 1.6 and 12.3 mm/blow, except for one point that showed about 25.0 mm/blow.

Figure 111 shows control charts for CBR values for the top 8 and 12 in. of the compacted lift.



SUDAS 2013

Figure 111. Pottawattamie County Project 6: CBR control charts with CBR quality ratings

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). Results indicated that 40% of the $CBR_{8in.}$ and 50% of the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 7. Woodbury County I-29

Overview

The ISU research team conducted field testing at this grading project site on 07/09/14, 07/10/14, and 08/07/14. The fill materials obtained at the time of testing consisted of alluvium materials and were classified as A-2-4 by the AASHTO Soil Classification System and SM by the USCS.

At this project site, the project specification required achievement of moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test. The equipment used during construction is shown in Figure 112 through Figure 114.

Figure 112. Woodbury County Project 7: Dump truck used to place loose fill materials

Figure 113. Woodbury County Project 7: Caterpillar D6T dozer used to control lift thickness

Figure 114. Woodbury County Project 7: Caterpillar CS56B vibratory smooth drum roller used for soil compaction

A vibratory smooth drum roller was used to compact the fills, which consisted of cohesionless materials (Figure 114). The lifted fill materials were very wet, and seepage was observed (Figure 115).

Figure 115. Woodbury County Project 7: Seepage at the construction site

ISU Field Test Results

To determine whether the field measurements met the specification requirements, Figure 116 through Figure 118 show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density.

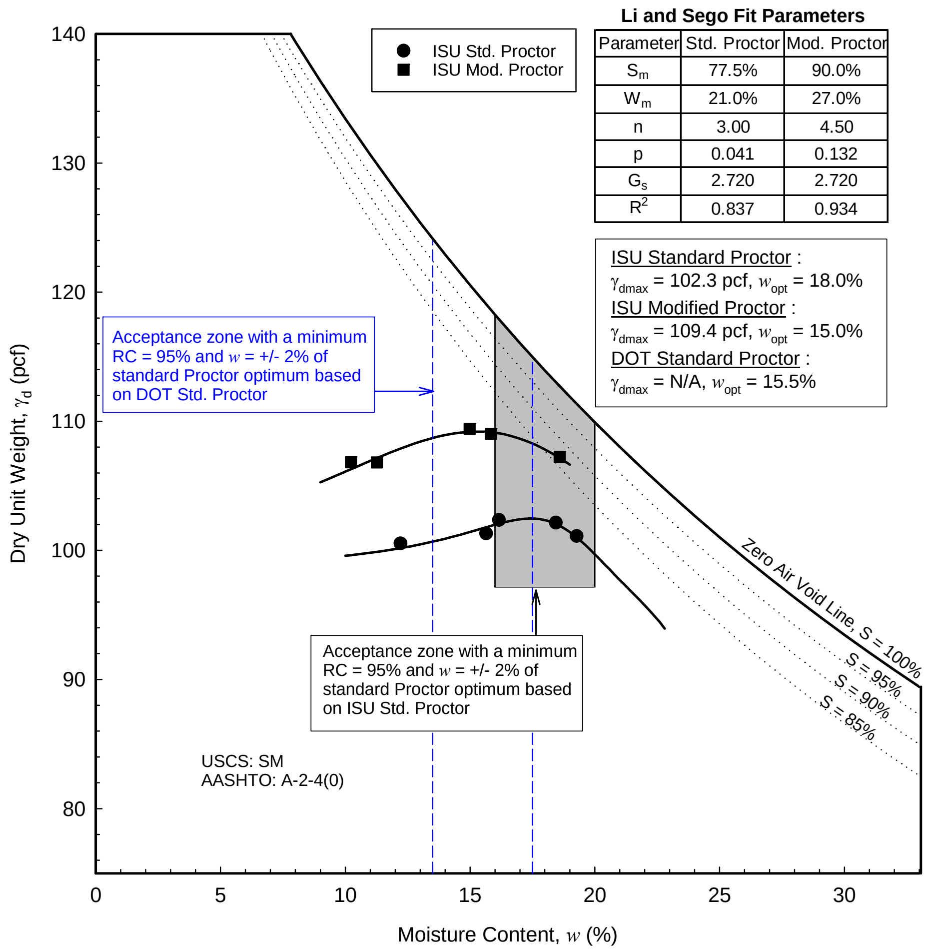


Figure 116. Woodbury County Project 7 TB1: Laboratory Proctor compaction test results with acceptance zone

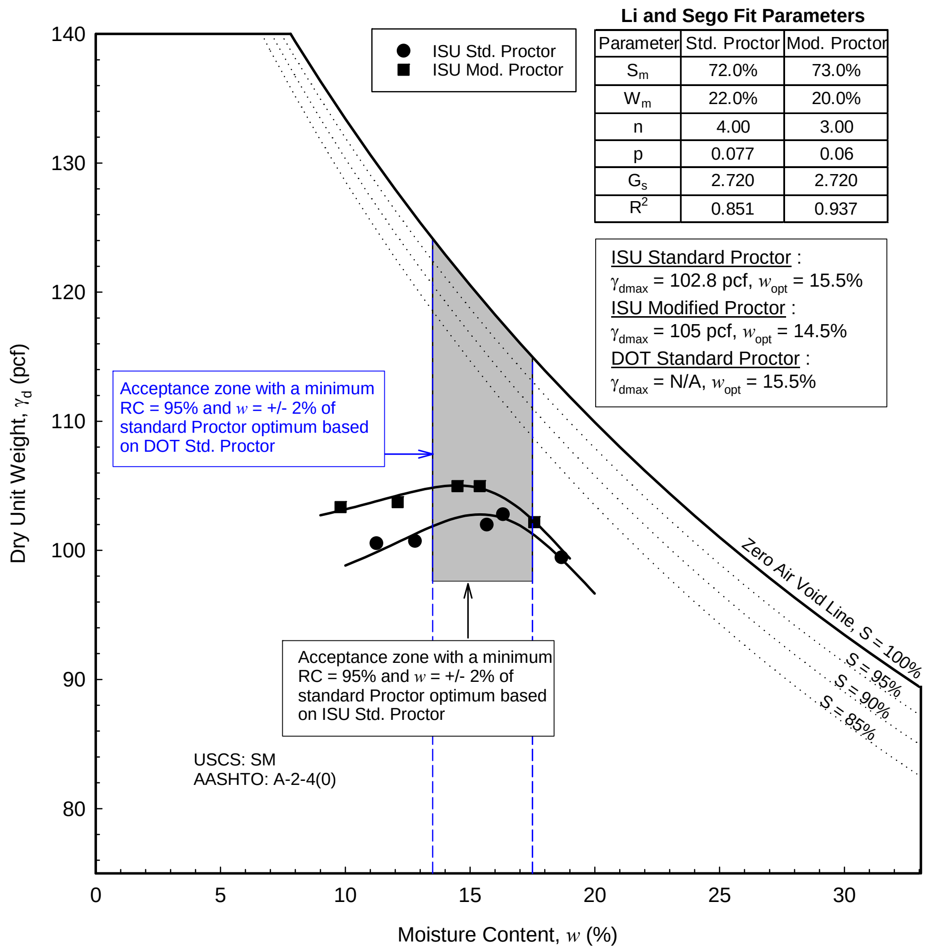


Figure 117. Woodbury County Project 7 TB2: Laboratory Proctor compaction test results with acceptance zone

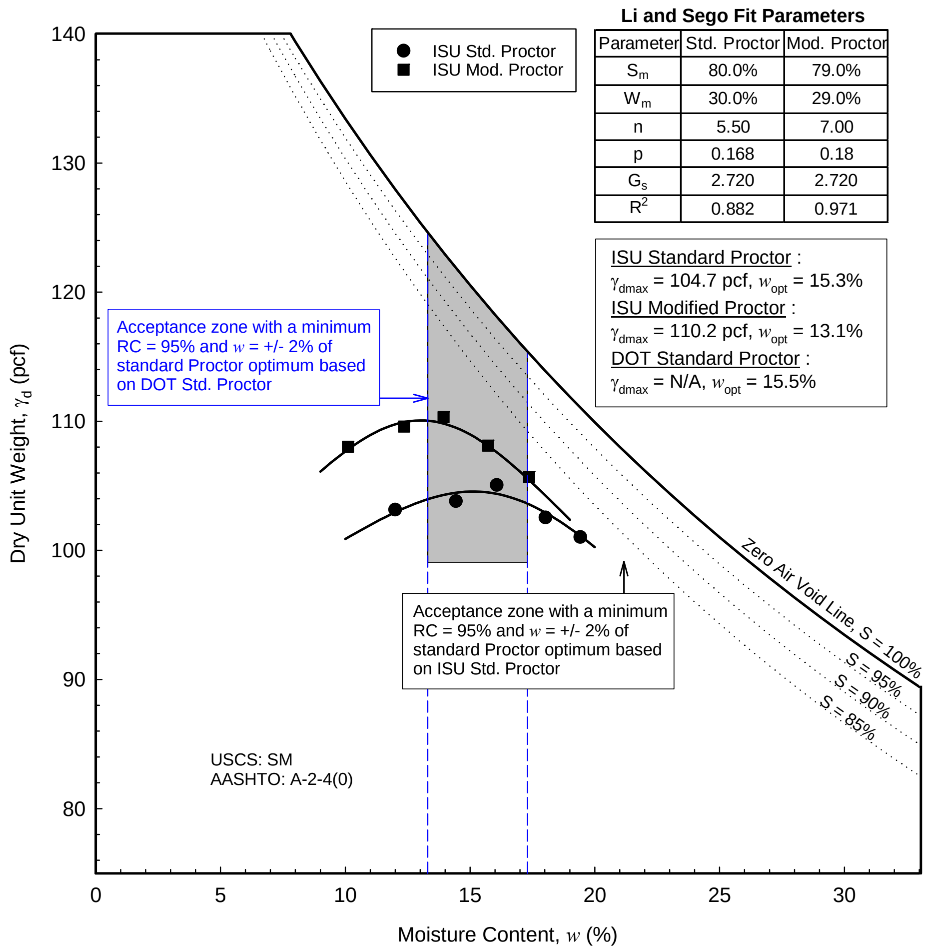


Figure 118. Woodbury County Project 7 TB3: Laboratory Proctor compaction test results with acceptance zone

Field density measurements were not performed at this site, but moisture content samples were obtained from the TBs and are presented in the control charts.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 119 through Figure 121 for the three TBs.

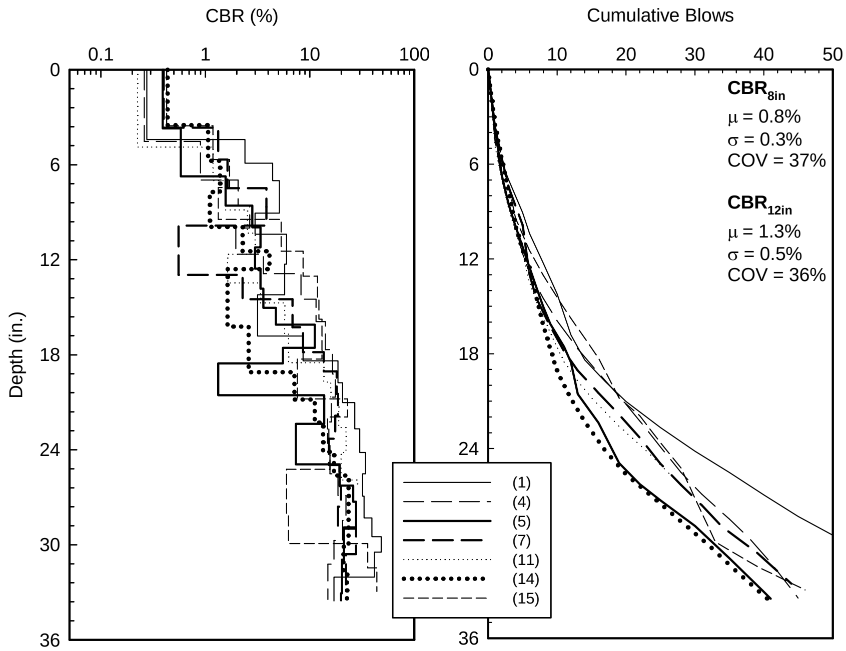


Figure 119. Woodbury County Project 7 TB1: DCP-CBR values and cumulative blows with depth profiles

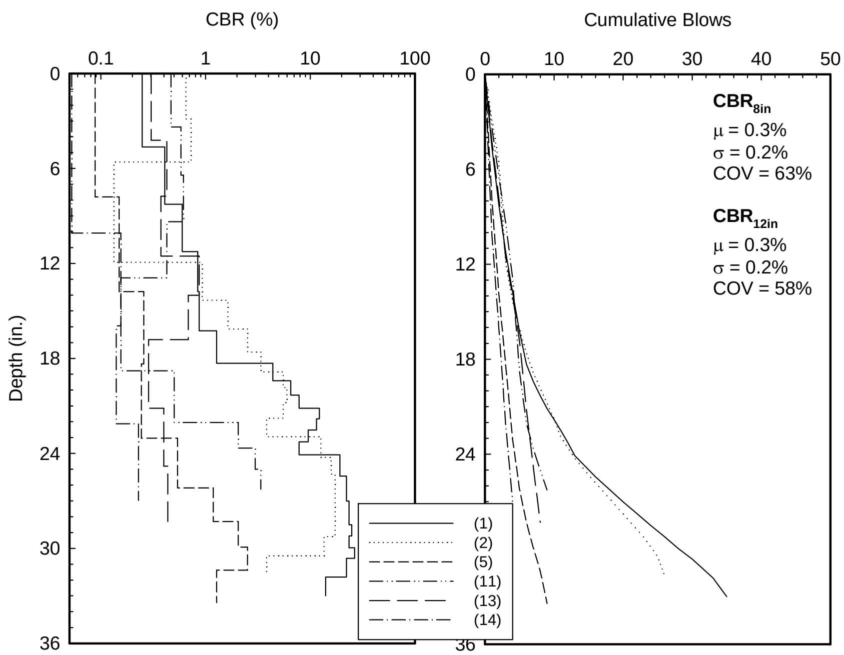


Figure 120. Woodbury County Project 7 TB2: DCP-CBR values and cumulative blows with depth profiles

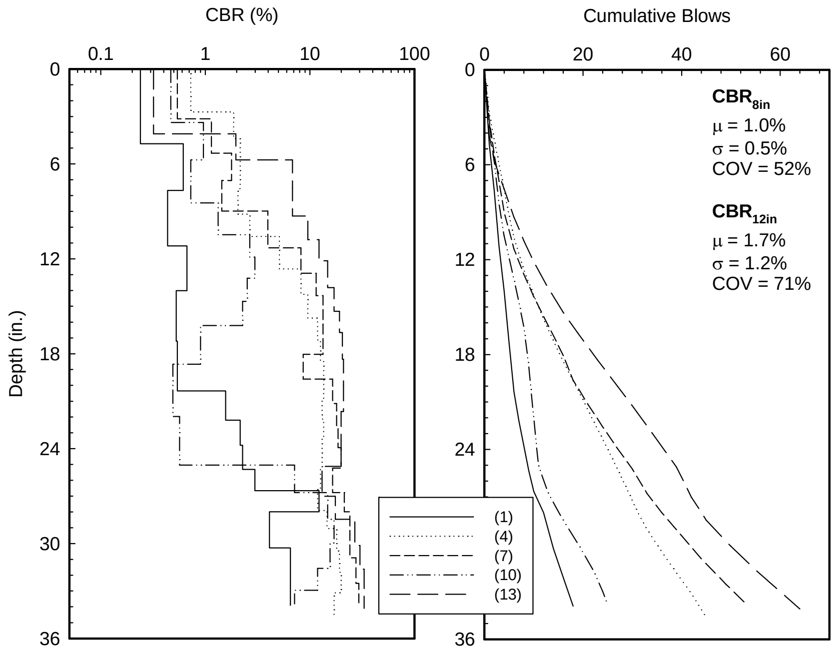


Figure 121. Woodbury County Project 7 TB3: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. varied between 1.5% and 3.0% and the average CBR value in the top 12 in. varied between 1.5% and 3.9% among the three test beds.

Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized in Table 22.

Table 22. Woodbury County Project 7: Summary of field testing results

Parameter	Woodbury County I-29 TB1 7/9/2014	Woodbury County I-29 TB2 7/10/2014	Woodbury County I-29 TB3 8/7/2014
Relative Compaction			
Average (%)	N/A	N/A	N/A
Range (%)	N/A	N/A	N/A
Standard Deviation (%)	N/A	N/A	N/A
COV (%)	N/A	N/A	N/A
$\Delta w\% = w_{field}\% - w_{opt}\%$			
Average (%)	5.5	6.9	-0.2
Range (%)	-2.1 to +13.8	+3.9 to +8.9	-1.6 to +1.6
Standard Deviation (%)	4.2	1.4	0.9
COV (%)	76	21	-381
$CBR_{8\ in.}$			
Average (%)	2.6	1.5	3.0
Range (%)	2.1 to 3.6	0.8 to 2.2	1.7 to 4.1
Standard Deviation (%)	0.5	0.6	1.0
COV (%)	20	41	32
$CBR_{12\ in.}$			
Average (%)	3.5	1.5	3.9
Range (%)	2.9 to 4.7	0.6 to 2.2	1.8 to 6.2
Standard Deviation (%)	0.7	0.6	1.7
COV (%)	19	39	44

Control Charts

The contractor QC data, Iowa DOT QA data and ISU data are reported in Figure 122 in the form of control charts monitoring the moisture content of the compacted fills.

The control chart data are presented as histograms in Figure 123.

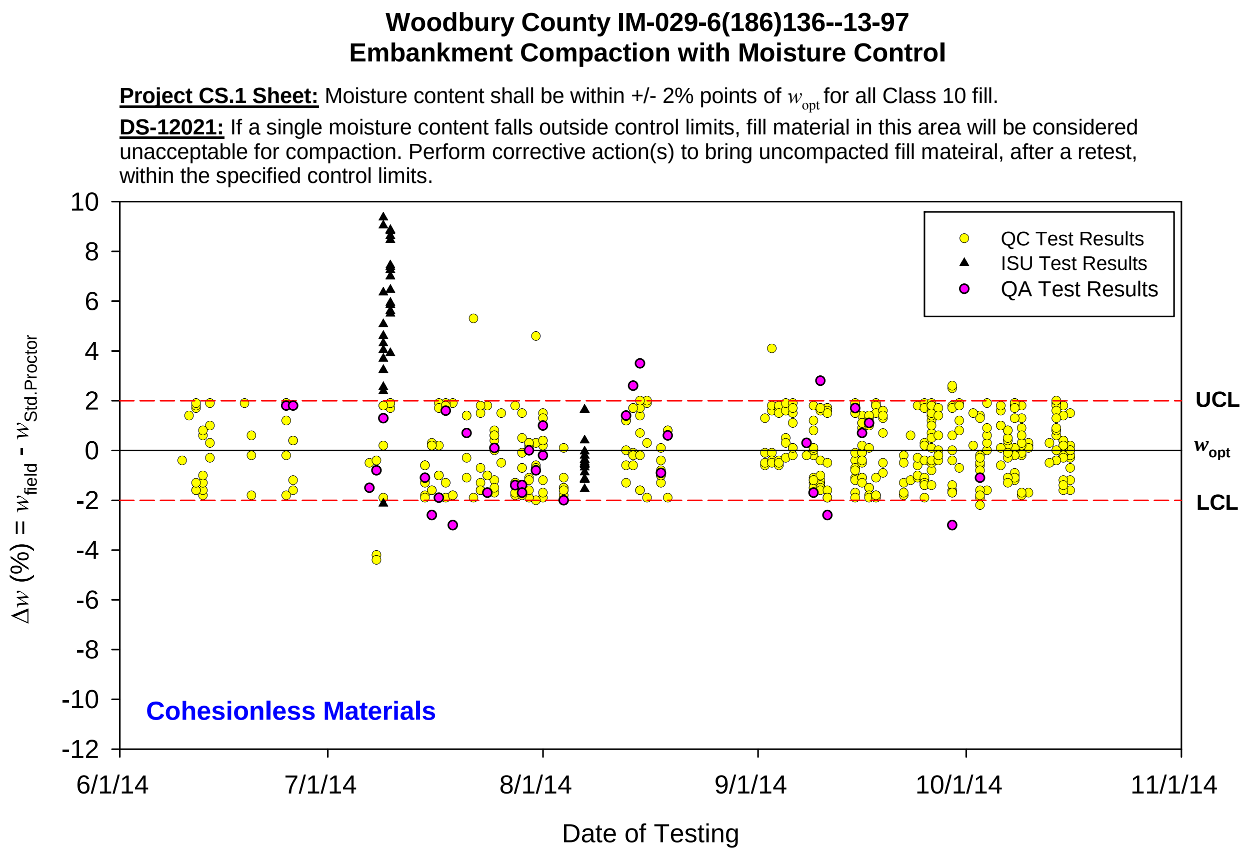


Figure 122. Woodbury County Project 7: Moisture control chart

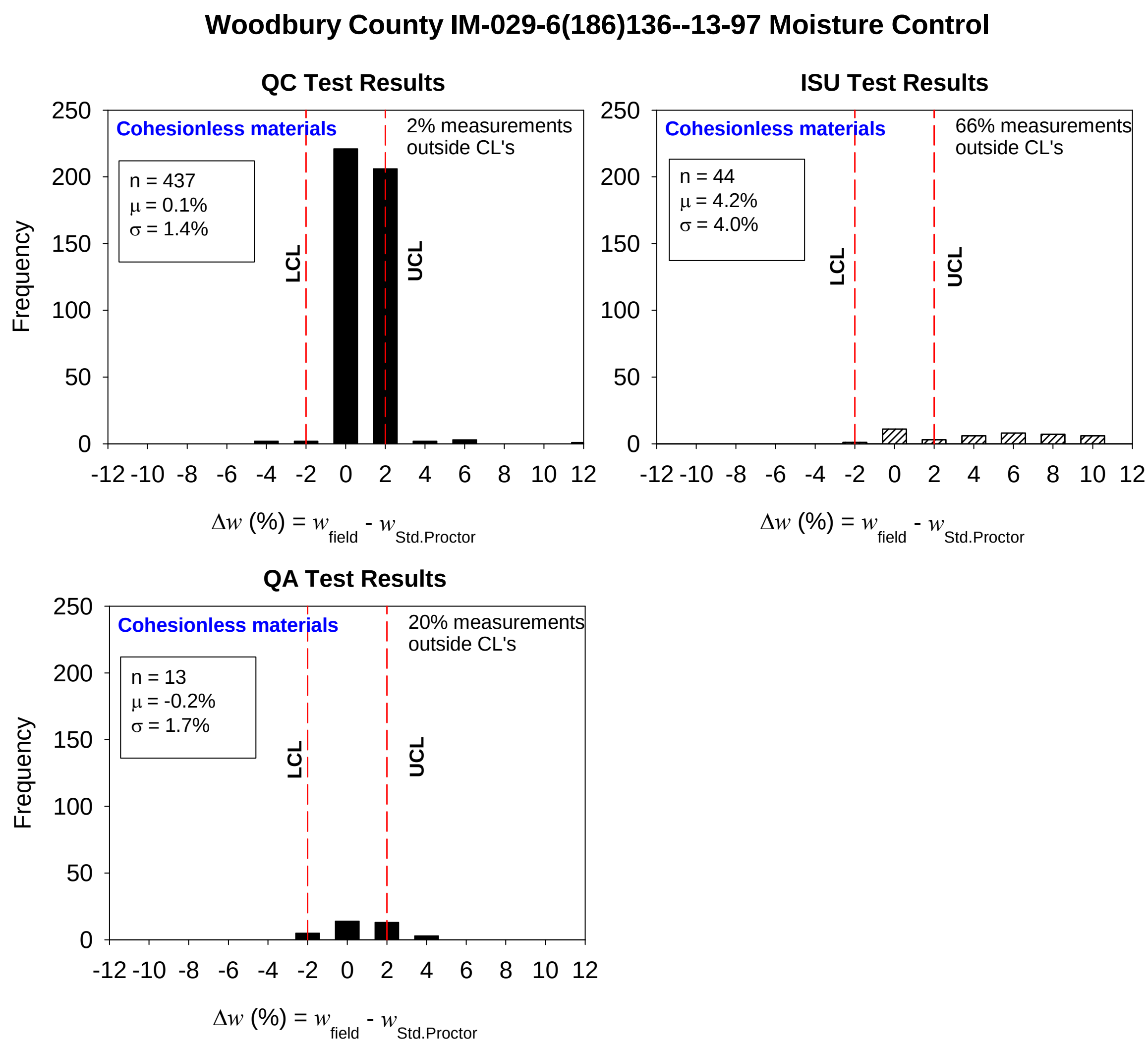
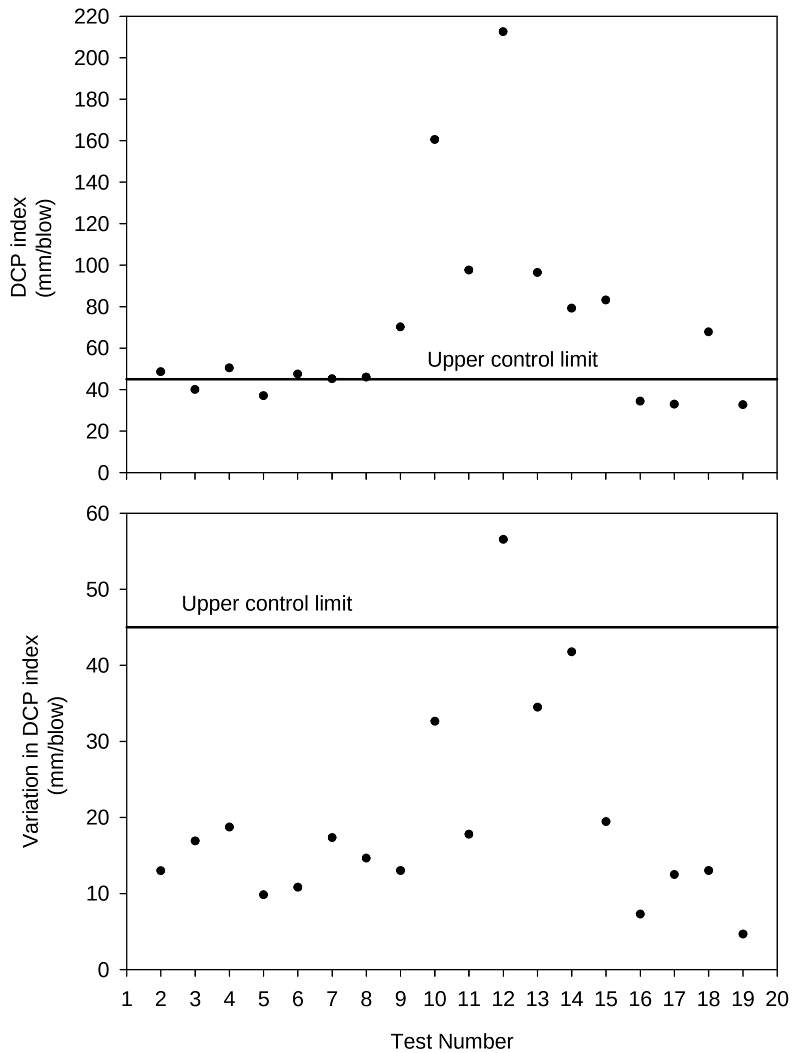


Figure 123. Woodbury County Project 7: Histograms of moisture control results

The data presented in the control charts and histograms indicate that most (98%) of the data fell within the moisture control limits. The QA testing results showed that 80% of the data were within the moisture control limits. The ISU testing results showed that only 34% of the data were within the moisture control limits.

Figure 124 shows control charts for DCP index values at a depth of 600 mm.

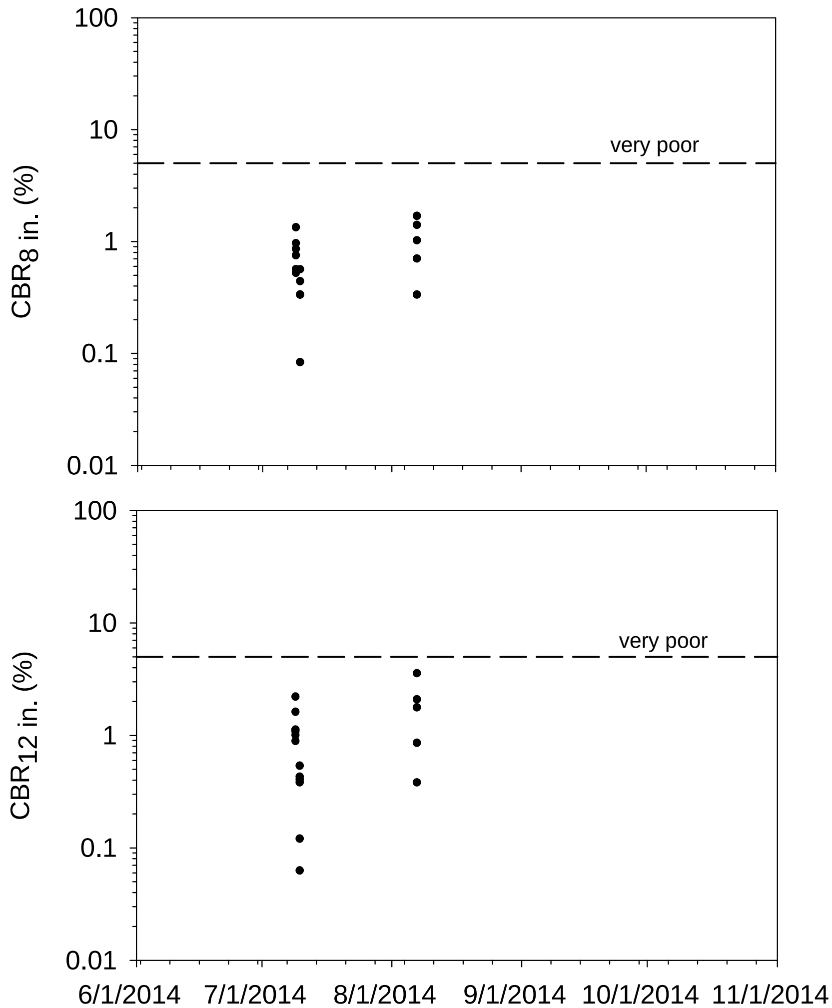


White et al. 2007

Figure 124. Woodbury County Project 7: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged between 33 and 213 mm/blow, and 13 points of all of the data exceeded the upper control limit. The variation in the DCP index control chart shows that DCP index variation fell between 4.6 and 41.8 mm/blow at 17 of the 18 points, with 1 point showing about 56.5 mm/blow.

Figure 125 shows control charts for CBR values for the top 8 and 12 in. of the compacted lift.



SUDAS 2013

Figure 125. Woodbury County Project 7: CBR control charts with CBR quality ratings

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicate that all of the $CBR_{8in.}$ and the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 8. Scott County

Overview

The ISU research team conducted field testing at this grading project site on 07/16/14, 07/31/14, and 09/19/14. The fill materials obtained at the time of testing consisted of loess materials and were classified as A-4(10), A-6(8), and A-6(5) by the AASHTO Soil Classification System and CL and CL-ML by the USCS.

At this project site, the project specification required achievement of moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test. The equipment used during construction is shown in Figure 126 through Figure 130.

Figure 126. Scott County Project 8: Caterpillar 349E used to excavate materials from borrow source

Figure 127. Scott County Project 8: Caterpillar dozer used to control lift thickness

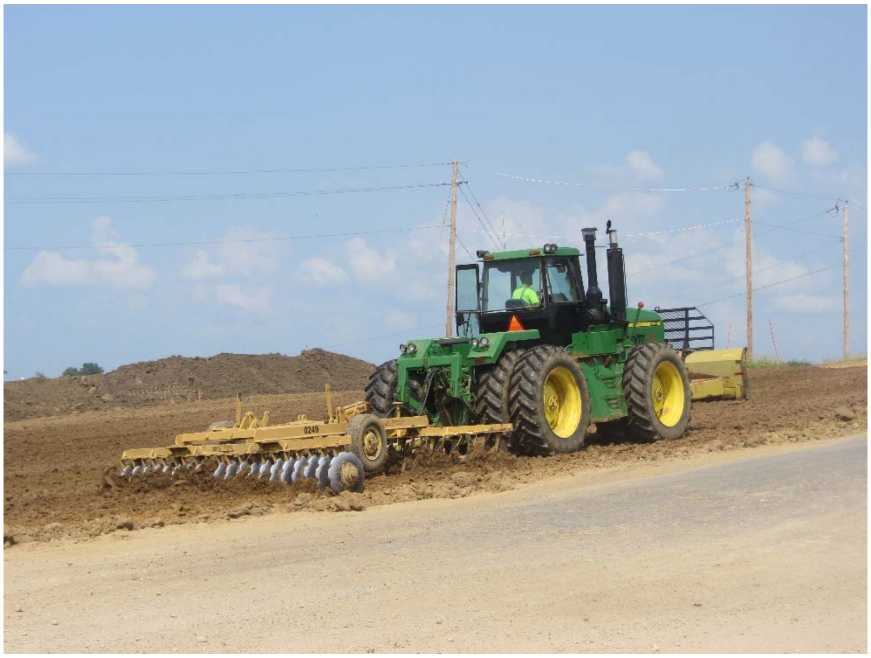


Figure 128. Scott County Project 8: Disc cultivator used to dry embankment materials

Figure 129. Scott County Project 8: Sheepsfoot roller used for soil compaction

Figure 130. Scott County Project 8: Dynapac padfoot roller used for soil compaction

A disc was used to break down and aerate the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the pull-behind sheepsfoot roller (Figure 129). Sheepsfoot walkout was observed during the site visits. Field observations indicated that the material obtained from the borrow area at the time of ISU testing was relatively wet.

ISU Field Test Results

In situ moisture content and dry density test results are compared with laboratory Proctor test results in Figure 131, Figure 132, and Figure 133.

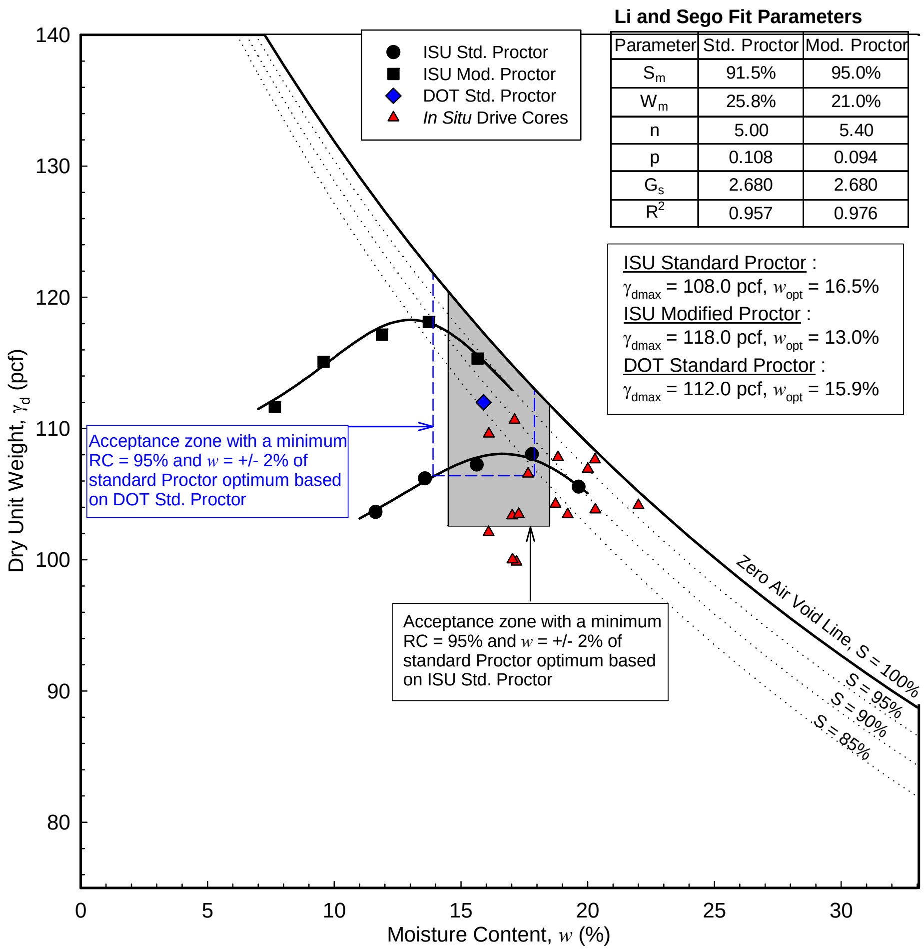


Figure 131. Scott County Project 8 TB1: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

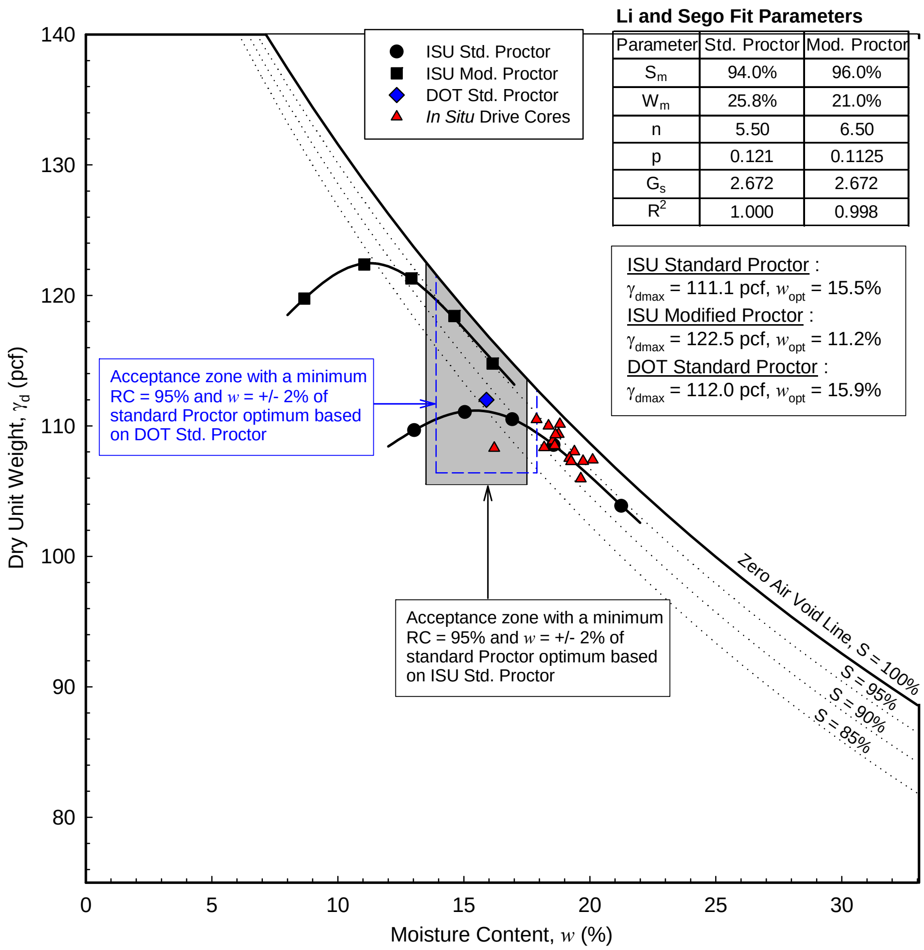


Figure 132. Scott County Project 8 TB2: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

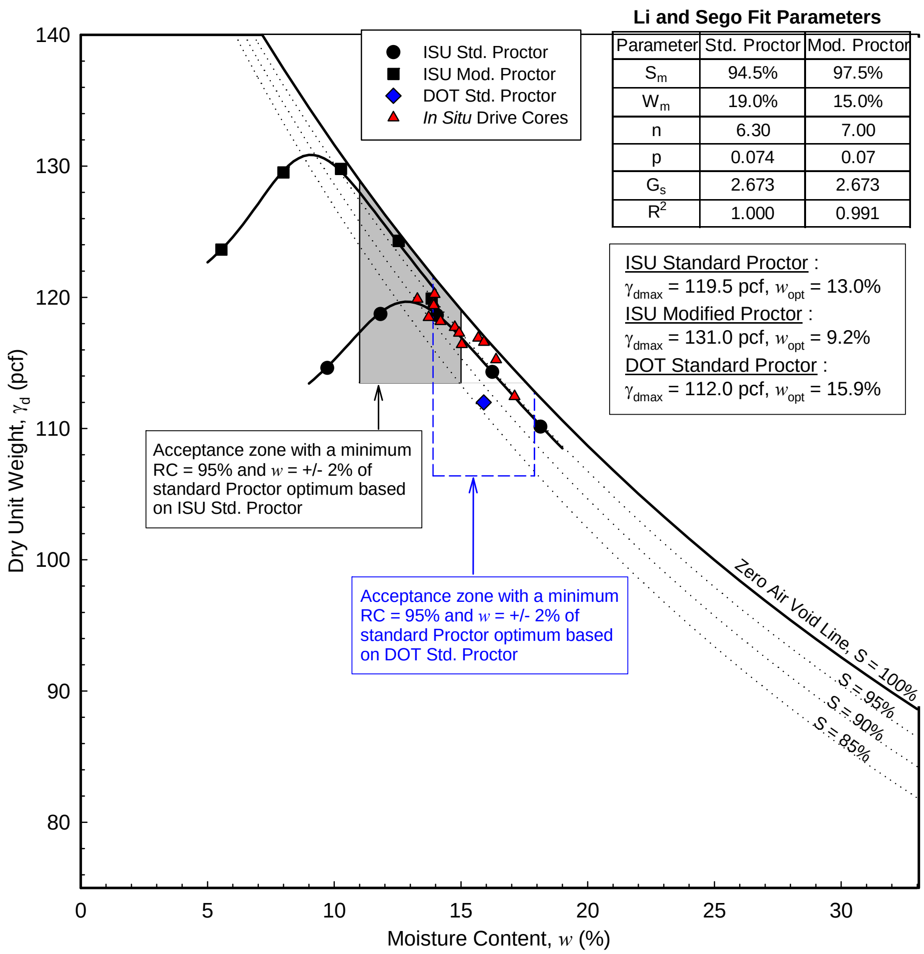


Figure 133. Scott County Project 8 TB3: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

The Proctor test results used by the Iowa DOT showed optimum moisture contents about 0.6% lower than those determined from ISU testing in the case of TB1 and 0.4% to 2.9% higher than those determined from ISU testing in the cases of TB2 and TB3. Similarly, the Proctor test results used by the Iowa DOT showed maximum dry densities about 0.9 to 4.0 lb/ft^3 higher than those determined from ISU testing in the case of TB1 and TB2 and 7.5 lb/ft^3 lower than those determined from ISU testing in the case of TB3.

To determine whether the field measurements met the specification requirements, Figures 126 through 128 also show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density. Maximum dry density, optimum moisture content, and the acceptance zone used by the Iowa DOT at the time of ISU testing are also shown in the figures for reference and comparison.

Field test results indicate that the relative compaction of the material ranged from approximately 92.4% to over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -0.4% and +7.1% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 131, Figure 132, and Figure 133 indicate that a majority of the ISU tests on TB2 fell outside the specification limit, with material generally > 2% wet of optimum moisture content and close to the 95% to 100% saturation line.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 134 through Figure 136 for the three TBs.

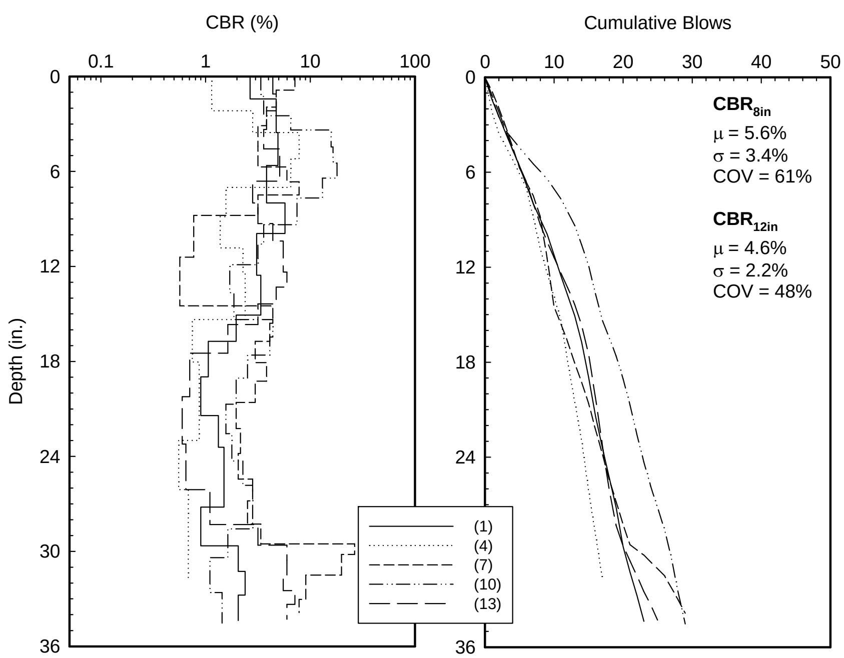


Figure 134. Scott County Project 8 TB1: DCP-CBR values and cumulative blows with depth profiles

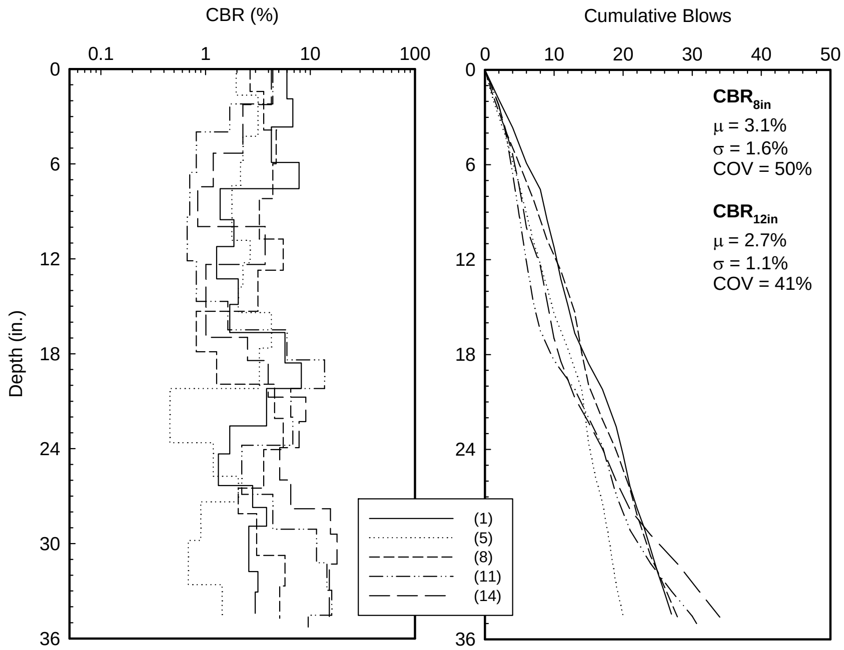


Figure 135. Scott County Project 8 TB2: DCP-CBR values and cumulative blows with depth profiles

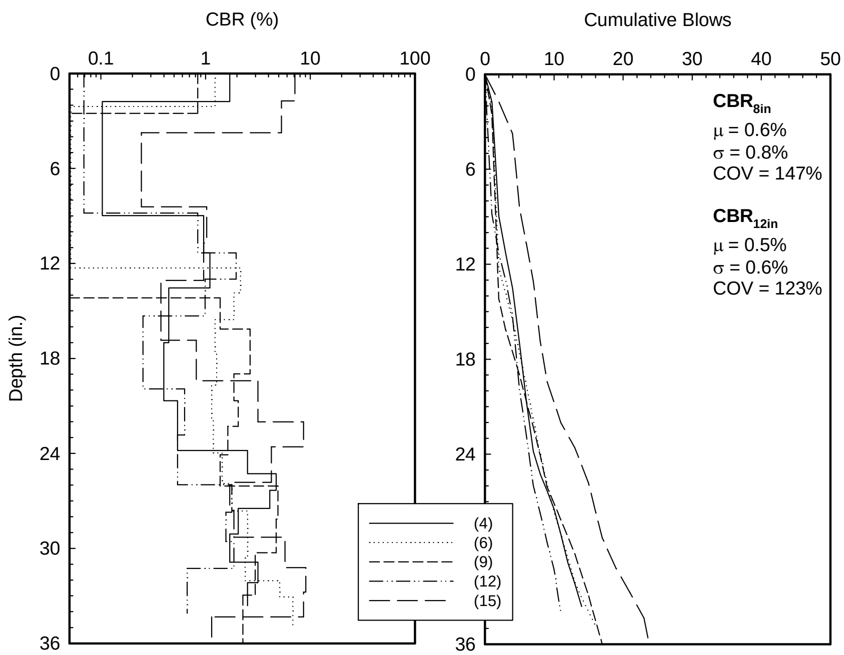


Figure 136. Scott County Project 8 TB3: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. varied between 0.6% and 7.6% and the average CBR value in the top 12 in. varied between 0.5% and 7.0% among the three test beds.

Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized in Table 23.

Table 23. Scott County: Summary of field testing

Parameter	Scott County TB1 7/16/2014	Scott County TB2 7/31/2014	Scott County TB3 9/19/2014
Relative Compaction			
Average (%)	97.1	97.5	98.0
Range (%)	92.4 to 102.4	95.3 to 99.4	92.5 to 100.6
Standard Deviation (%)	0.03	0.01	0.02
COV (%)	3	1	2
$\Delta w\% = w_{field}\% - w_{opt}\%$			
Average (%)	1.8	3.3	2.3
Range (%)	-0.4 to +5.5	0.7 to +4.6	0.3 to +7.1
Standard Deviation (%)	0.02	0.93	1.77
COV (%)	96	29	77
$CBR_{8\ in.}$			
Average (%)	7.6	3.1	0.6
Range (%)	6.2 to 11.6	1.8 to 5.5	0.1 to 2.0
Standard Deviation (%)	2.2	1.6	0.8
COV (%)	29	50	147
$CBR_{12\ in.}$			
Average (%)	7.0	2.7	0.5
Range (%)	5.5 to 10.0	1.3 to 3.9	0.1 to 1.6
Standard Deviation (%)	1.8	1.1	0.6
COV (%)	25	41	123

Control Charts

The contractor QC data, Iowa DOT QA data and ISU data are reported in Figure 137 in the form of control charts monitoring the dry unit weight and moisture content of the compacted fills.

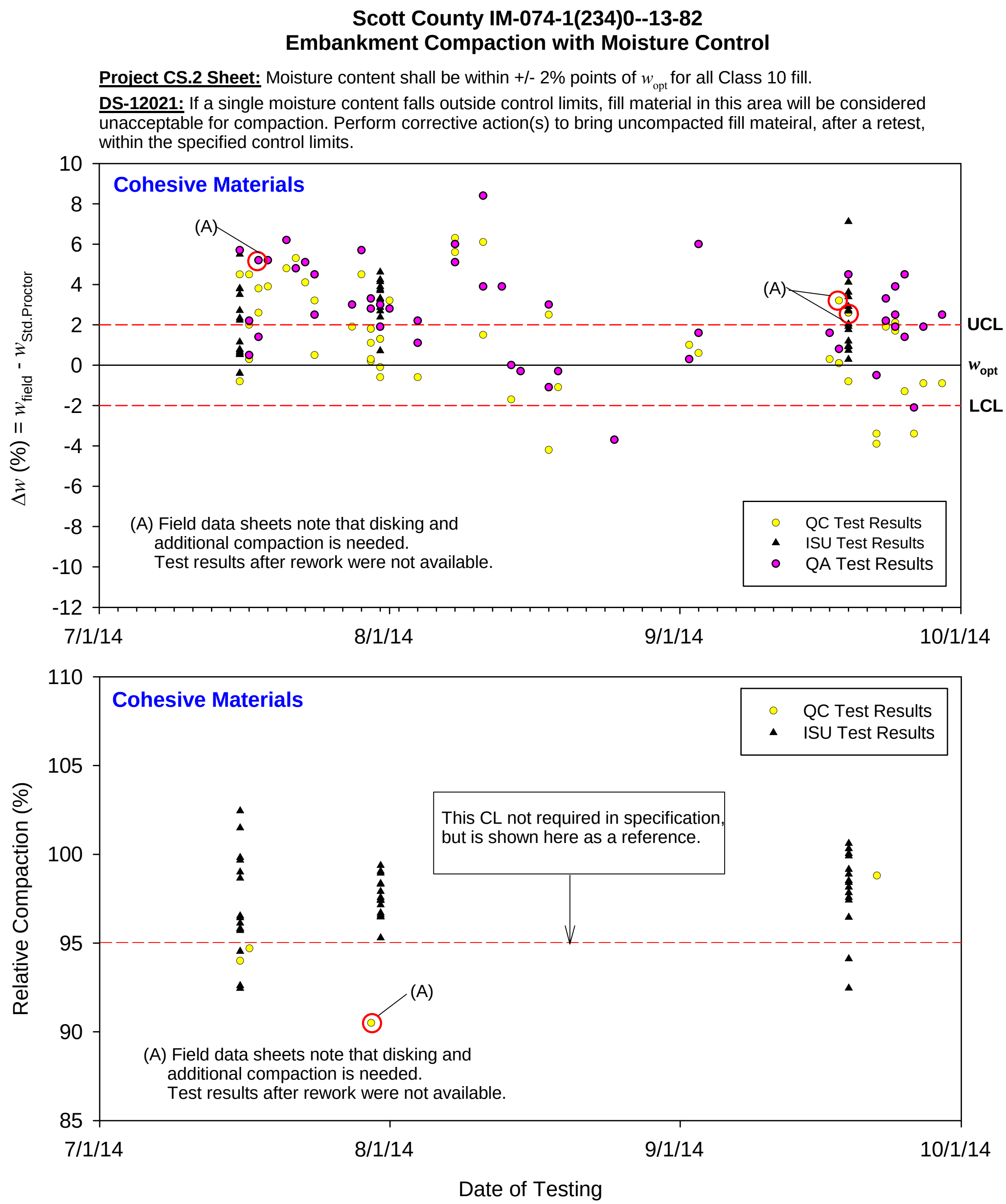


Figure 137. Scott County Project 8: Moisture control chart

The control chart data are presented as histograms in Figure 138.

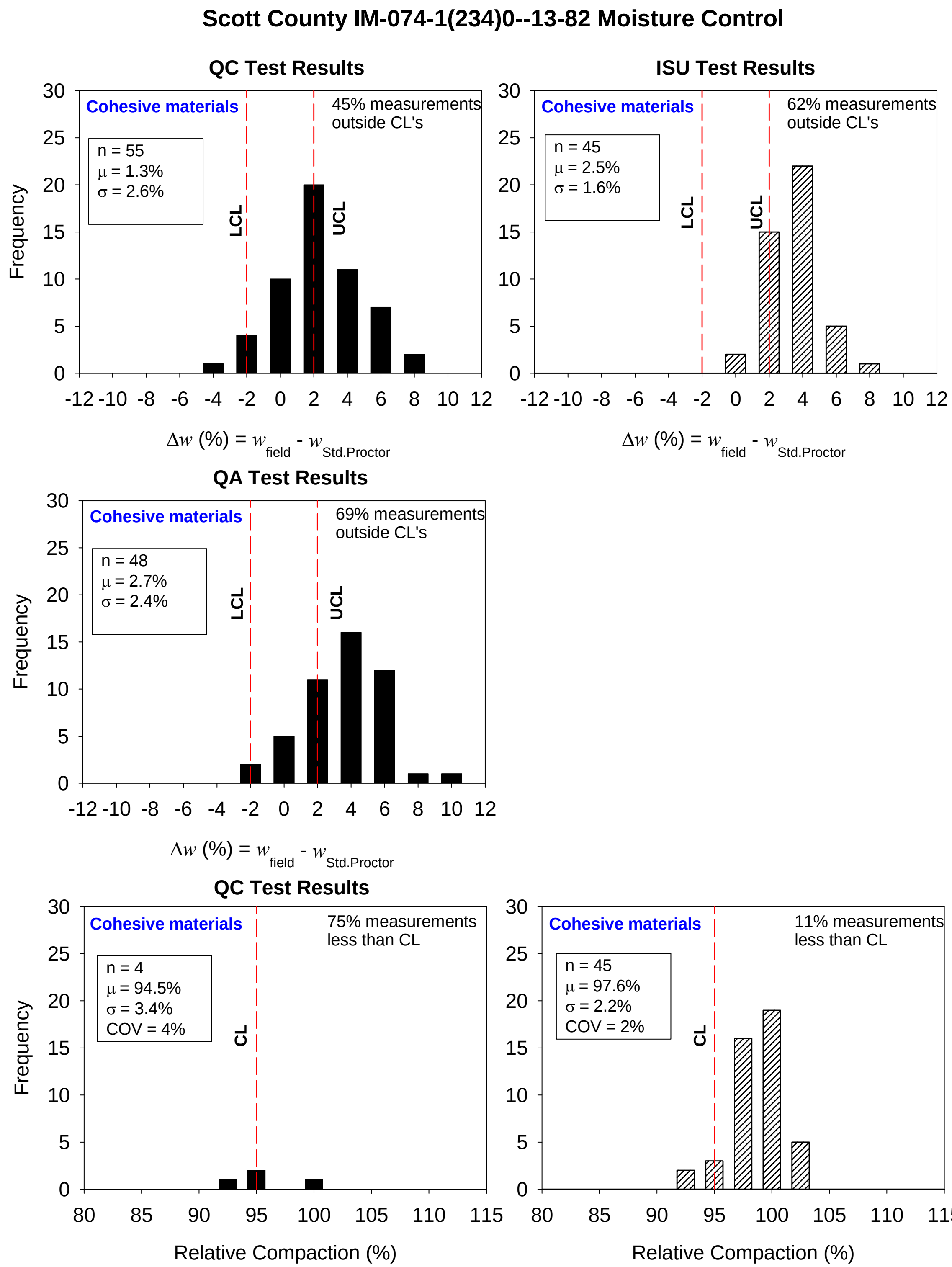
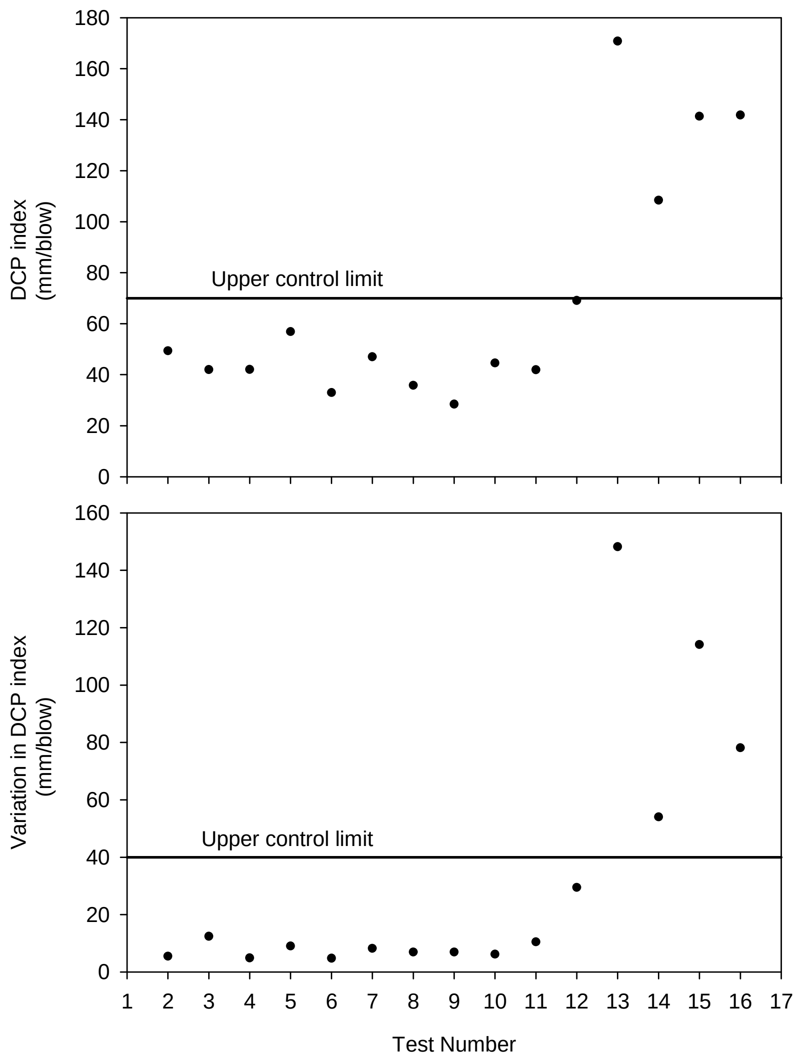


Figure 138. Scott County Project 8: Histograms of moisture control results

The data presented in the control charts and histograms indicate that 25% of the contractor QC data showed relative compaction > 95%, and 55% of the data fell within the moisture control limits. The QA testing results show that 31% of the data fell within the moisture control limits. The ISU testing results showed that 89% of the data showed relative compaction > 95%, and 38% of the data were within the moisture control limits.

Figure 139 shows control charts for DCP index values at a depth of 600 mm.

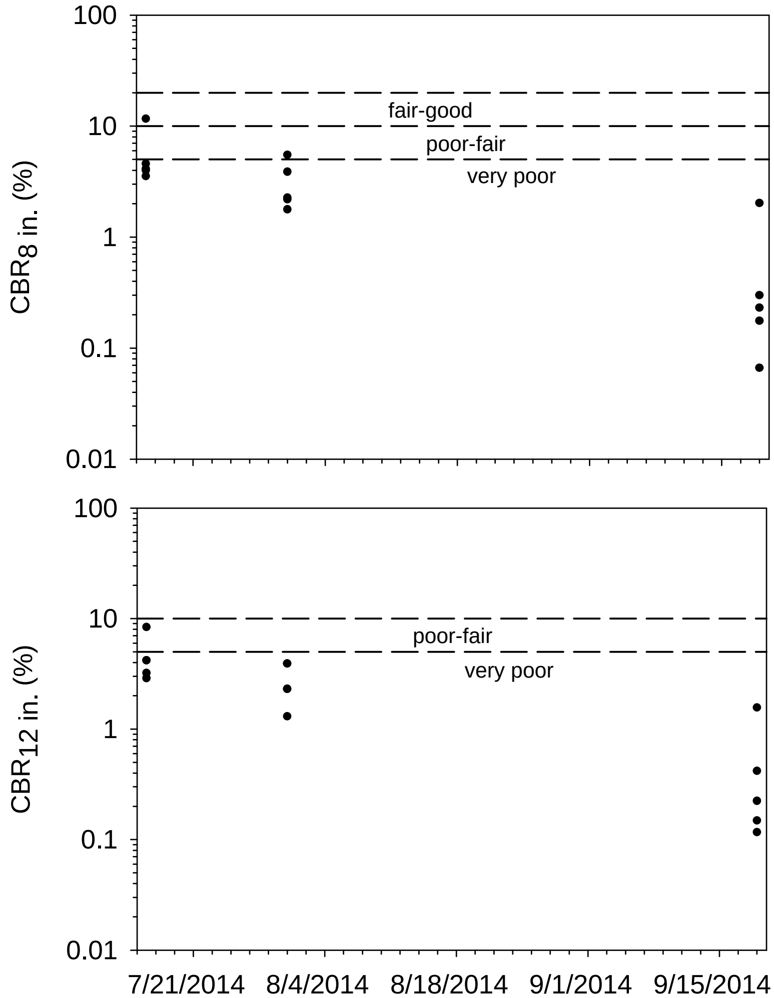


White et al. 2007

Figure 139. Scott County Project 8: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged from 28.4 to 170.8 mm/blow, and four points of all data exceeded the control limit. The variation in the DCP index control chart shows that DCP index variation between 5.5 and 29.4 mm/blow. Four points exceeded the control limit, with values of 148.17, 54.0, 114.1, and 78.1 mm/blow, respectively.

Figure 140 shows control charts for CBR values for the top 8 and 12 in. of the compacted lift.



SUDAS 2013

Figure 140. Scott County Project 8: CBR control charts with CBR quality ratings

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicate that 87% of the $CBR_{8in.}$ and 93% of the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

Project 9. Woodbury County US 20

Overview

The ISU research team conducted field testing at this grading project site on 09/26/14 and 10/18/14. The fill materials obtained at the time of testing consisted of very deep loess materials and were classified as A-4(7), A-4(9), and A-6(12) by the AASHTO Soil Classification System and CL and CL-ML by the USCS.

At this project site, the project specification required achievement of moisture content within ±2.0% of the optimum moisture content determined from the standard Proctor test. The equipment used during construction is shown in Figure 141 through Figure 145.

Figure 141. Woodbury County Project 9: Caterpillar 631D motor scraper used to collect and place loose fill materials

Figure 142. Woodbury County Project 9: Caterpillar D6N dozer used to control lift thickness

Figure 143. Woodbury County Project 9: Caterpillar 140H motor grader used to level the embankment surface

Figure 144. Woodbury County Project 9: Caterpillar CS56 series vibratory smooth drum roller used for soil compaction

Figure 145. Woodbury County Project 9: Sheepsfoot roller used for soil compaction

A disc was used to break down and aerate the wet soil. Compaction was achieved in part from the haul equipment and five to eight passes of the pull-behind sheepsfoot roller (Figure 145). Sheepsfoot walkout was observed during the site visits.

ISU Field Test Results

In situ moisture content and dry density test results are compared with laboratory Proctor test results in Figure 146 through Figure 149.

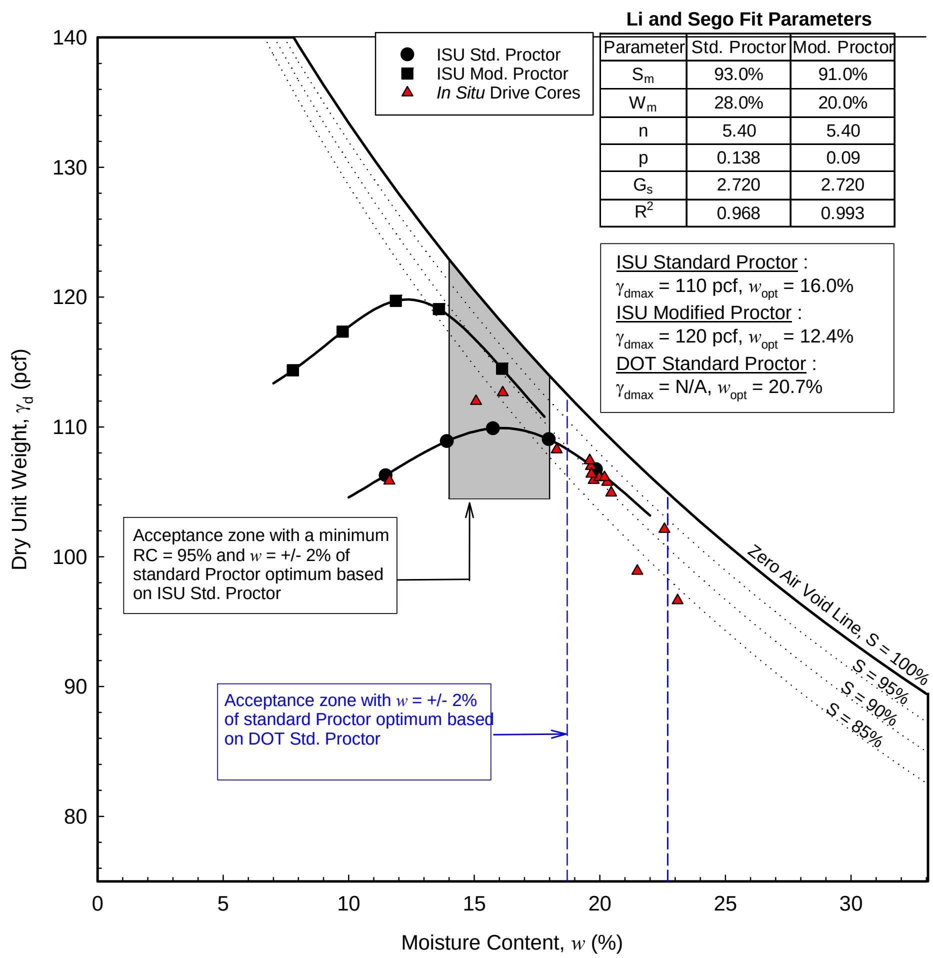


Figure 146. Woodbury County Project 9 TB1: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

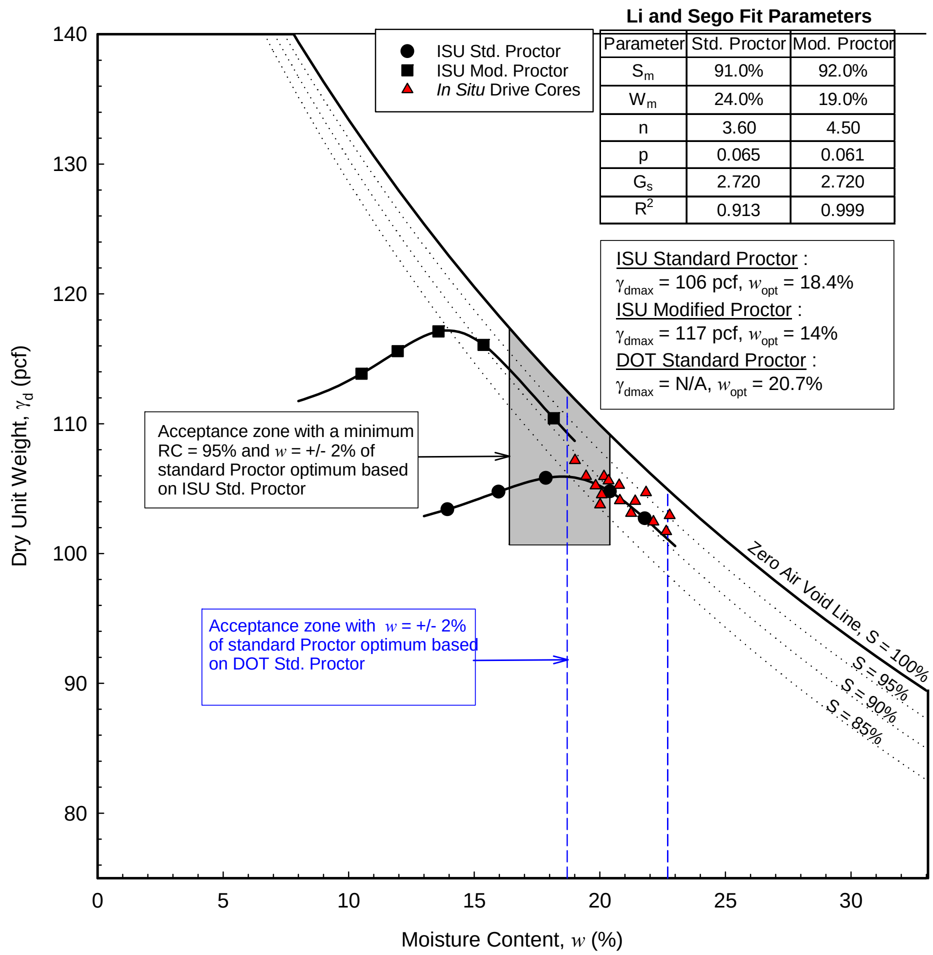


Figure 147. Woodbury County Project 9 TB2: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

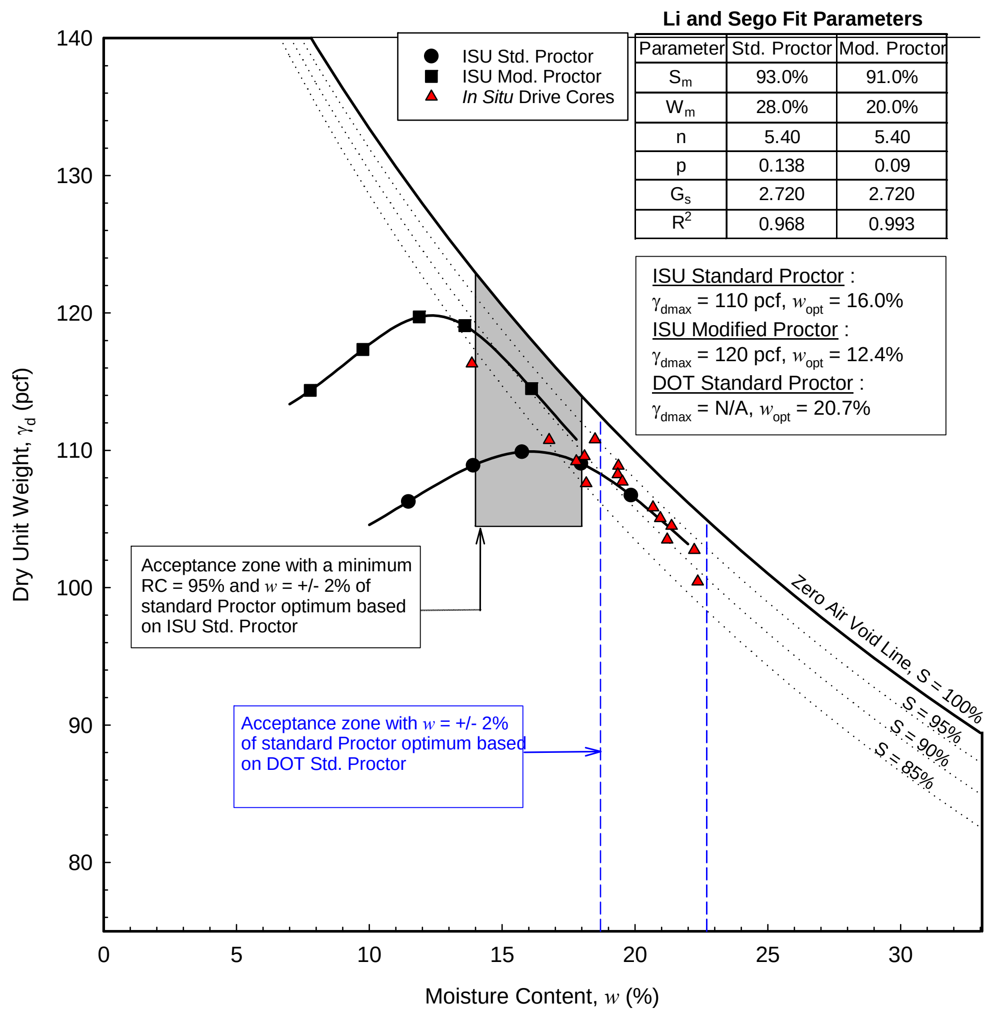


Figure 148. Woodbury County Project 9 TB3: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

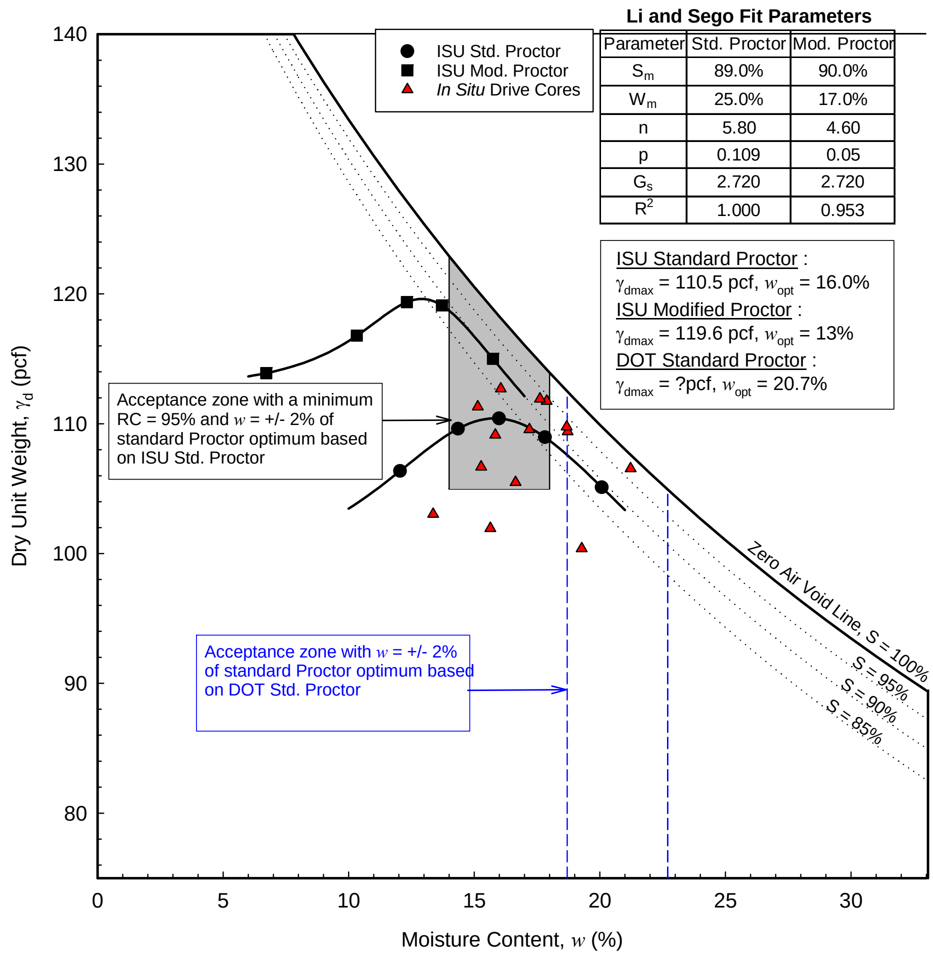


Figure 149. Woodbury County Project 9 TB4: Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits

The Proctor test results used by the Iowa DOT showed optimum moisture contents about 2.3% to 4.7% lower than those determined from ISU testing. The maximum dry density data from the Iowa DOT standard Proctor test are not available.

To determine whether the field measurements met the specification requirements, Figures 141 through 144 also show an acceptance range of ±2.0% of the standard Proctor optimum moisture content and 95% of standard Proctor density. Optimum moisture content and the acceptance zone used by the Iowa DOT at the time of ISU testing are also shown in the figures for reference and comparison.

Field test results indicate that the relative compaction of the material ranged from approximately 87.4% to over 100% of the standard Proctor maximum dry density, with in situ moisture content ranging between -4.4% and +7.1% of the optimum moisture content, as determined from the ISU testing.

The in situ moisture and dry density test results presented in Figure 146 to Figure 149 indicate that a majority of the ISU tests on TB1, TB2, and TB3 fell outside the specification limit, with material generally > 2% wet of optimum moisture content and close to the 90% to 95% saturation line.

DCP-CBR values and cumulative blows with depth profiles are shown in Figure 150 through Figure 153 for the four TBs.

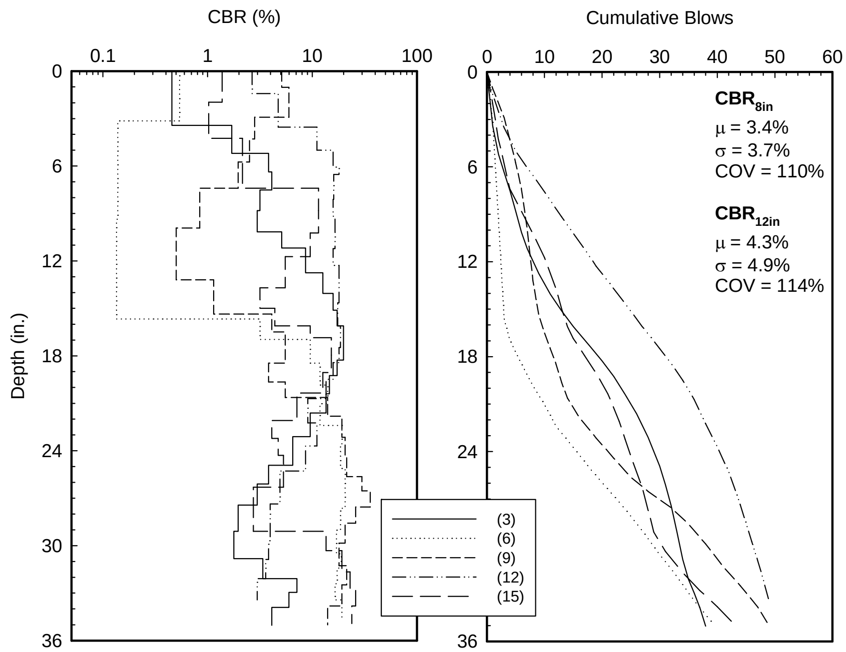


Figure 150. Woodbury County Project 9 TB1: DCP-CBR values and cumulative blows with depth profiles

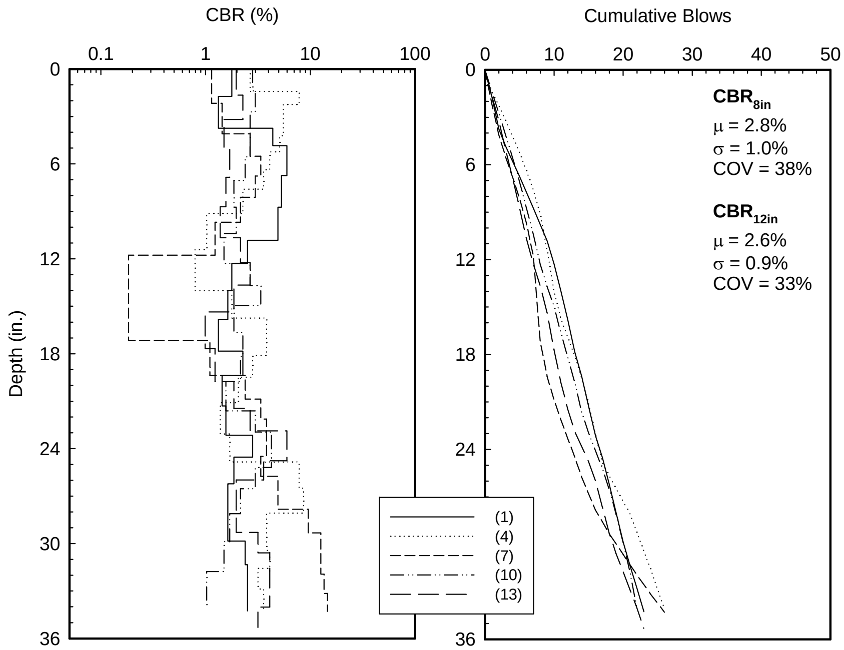


Figure 151. Woodbury County Project 9 TB2: DCP-CBR values and cumulative blows with depth profiles

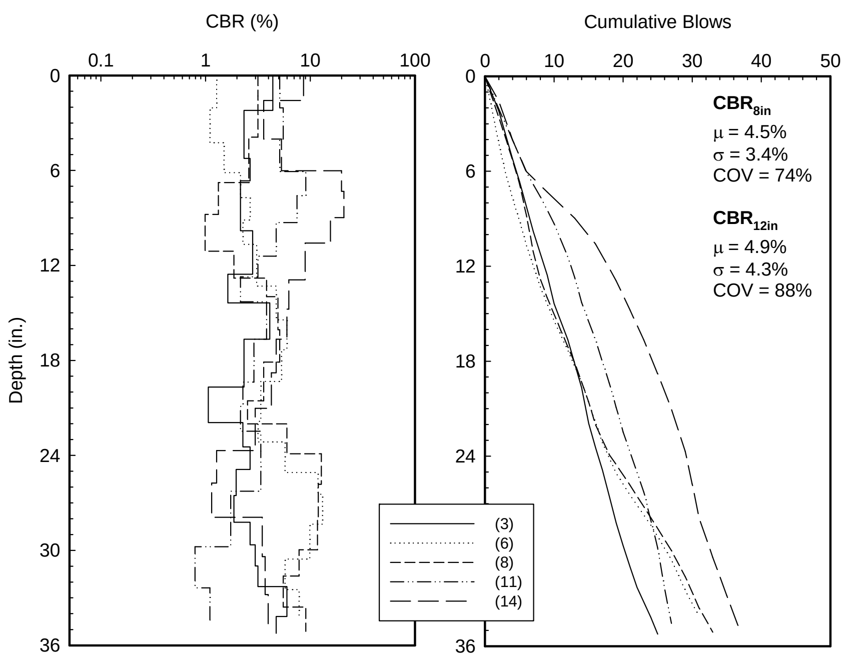


Figure 152. Woodbury County Project 9 TB3: DCP-CBR values and cumulative blows with depth profiles

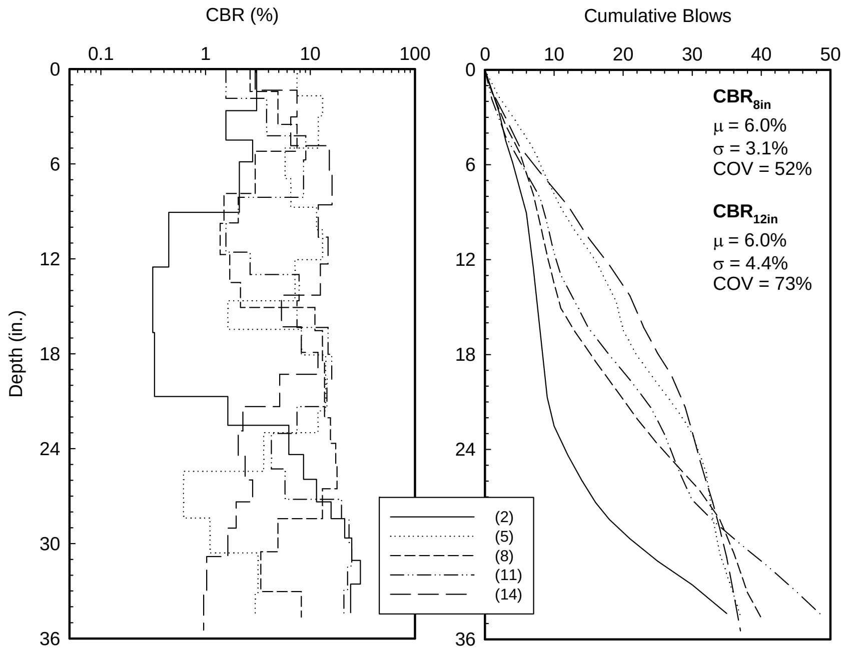


Figure 153. Woodbury County Project 9 TB4: DCP-CBR values and cumulative blows with depth profiles

The average CBR value (per TB) in the top 8 in. varied between 2.8% and 8.1% and the average CBR value in the top 12 in. varied between 2.6% and 7.8% among the four test beds. The results indicate that the CBR values are generally higher when the material is within the moisture control limit, as in the cases of TB2 and TB3, and vice versa, as in the cases of TB1 and TB4.

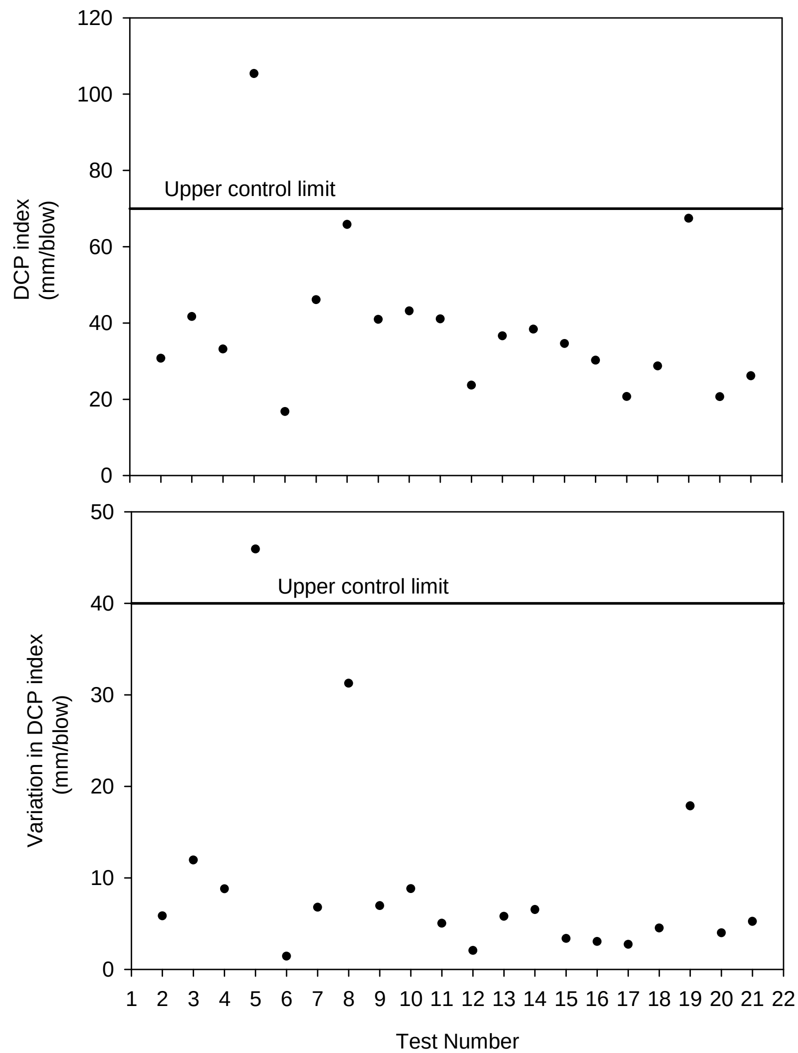
Summary statistics of the field measurements with average, range, standard deviation, and COV are summarized in Table 24.

Table 24. Woodbury County Project 9: Summary of field testing

Parameter	Woodbury County (US20) TB1 9/26/2014	Woodbury County (US20) TB2 9/26/2014	Woodbury County (US20) TB3 10/18/2014	Woodbury County (US20) TB4 10/18/2014
Relative Compaction				
Average (%)	95.7	99.9	100.7	97.6
Range (%)	87.4 to 101.9	97.3 to 102.6	94.1 to 109.0	90.8 to 102.0
Standard Deviation (%)	0.04	0.01	0.04	0.04
COV (%)	4	1	4	4
$\Delta w\% = w_{field}\% - w_{opt}\%$				
Average (%)	3.2	2.3	1.4	1.0
Range (%)	-4.4 to +7.1	0.5 to +4.3	-4.1 to +4.4	-2.6 to +5.2
Standard Deviation (%)	2.95	1.15	2.27	2.04
COV (%)	93	49	168	196
$CBR_{8\ in.}$				
Average (%)	5.3	2.8	4.5	8.1
Range (%)	1.4 to 10.8	1.7 to 4.3	1.4 to 9.8	5.0 to 11.0
Standard Deviation (%)	3.5	1.0	3.4	2.5
COV (%)	65	38	74	31
$CBR_{12\ in.}$				
Average (%)	6.1	2.6	4.8	7.8
Range (%)	1.3 to 12.7	1.8 to 3.7	1.8 to 11.7	4.2 to 11.8
Standard Deviation (%)	4.2	0.9	4.2	3.3
COV (%)	69	33	87	42

Control Charts

Figure 154 shows control charts for DCP index values at a depth of 600 mm.

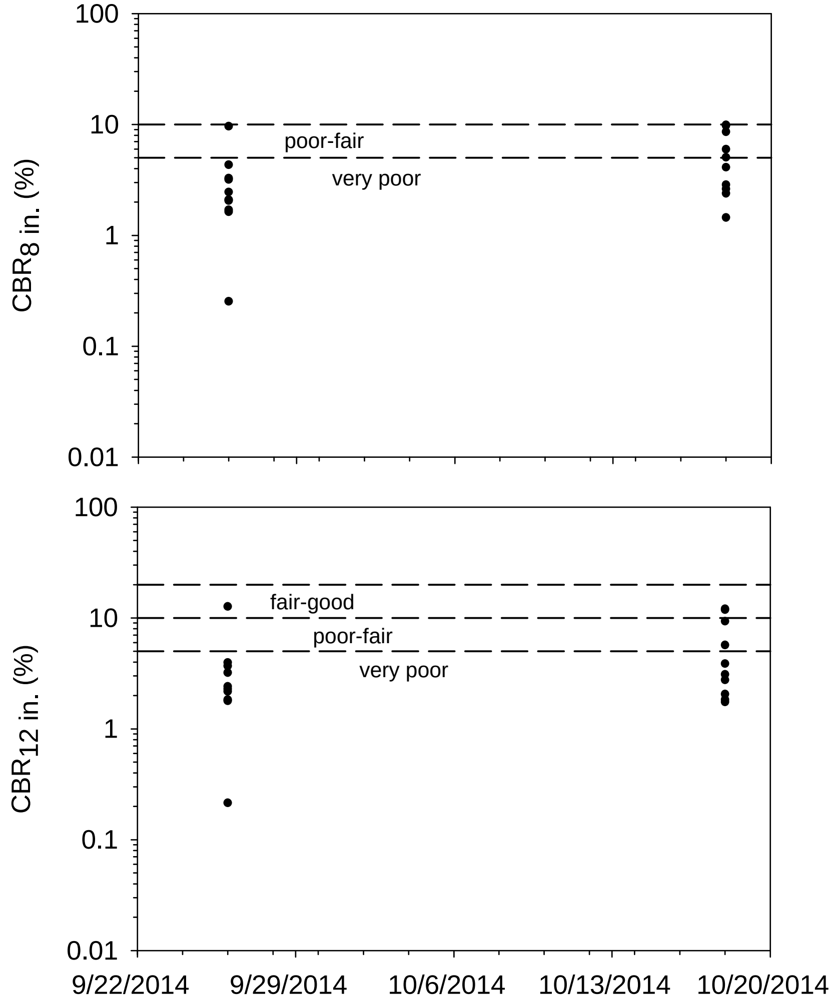


White et al. 2007

Figure 154. Woodbury County Project 9: Control charts with control limits for DCP index and variation in DCP index

The weighted average DCP index values ranged between 16.7 and 105.4 mm/blow, and one point exceeded the control limit. The variation in the DCP index control chart shows that DCP index variation fell between 1.4 and 31.2 mm/blow, except for one point that showed 45.9 mm/blow.

Figure 155 shows control charts for CBR values for the top 8 and 12 in. of the compacted lift.



SUDAS 2013

Figure 155. Woodbury County Project 9: CBR control charts with CBR quality ratings

The control charts show CBR ratings per the SUDAS Design Manual guidance regarding subgrade design and construction (SUDAS 2013). The results indicated that 70% of the $CBR_{8in.}$ and 75% of the $CBR_{12in.}$ data showed CBR < 5, which is rated as very poor.

CHANPTER 6. LAB TEST RESULTS

This chapter presents the results obtained from laboratory tests. This chapter contains two parts, one is cement stabilization results, and another is one-dimensional consolidation test results.

Cement Stabilization

A summary of the F_{200}, Atterberg limits, GI, and Iowa DOT material suitability classification results for materials stabilized with different cement contents are presented in Table 25. Detailed results are provided in Appendix A. In the following sections of this chapter, the results and analysis are separately for F_{200}, Atterberg limits, GI, and UCS, to present the influence of cement stabilization on these properties.

Table 25. Summary of soil index properties and Iowa DOT suitability classifications at different cement contents

County and Test Bed	Cement content (%)	F_{200} (%)	LL (%)	PI (%)	GI	Iowa DOT Suitability
Polk TB1	0	88	49	21	21	suitable
	4	74.1	41	13	10	suitable
	8	64.5	40	8	5	suitable
	12	53.1	40	0	0	suitable
Polk TB2	0	70.3	45	11	8	suitable
	4	59.3	43	13	7	suitable
	8	47.9	41	10	3	suitable
	12	45.7	38	0	0	suitable
Polk TB3	0	68.7	36	16	9	suitable
	4	58.5	34	6	2	suitable
	8	41.1	35	0	0	suitable
	12	32.3	36	0	0	suitable
Polk TB4	0	73.6	34	17	11	suitable
	4	61.9	36	0	0	suitable
	8	40.6	38	0	0	suitable
	12	40.4	34	0	0	suitable
Warren TB1	0	70.5	44	13	9	suitable
	4	60.4	38	14	7	suitable
	8	36.8	41	0	0	suitable
	12	27.4	38	0	0	suitable
Warren TB2	0	63.4	40	21	11	select

Table 25 continued

County and Test Bed	Cement content (%)	F_{200} (%)	LL (%)	PI (%)	GI	Iowa DOT Suitability
	4	55.7	39	15	6	select
	8	34.4	38	0	0	suitable
	12	25.7	34	0	0	suitable
Warren TB3	0	80.6	54	34	28	unsuitable
	4	70.7	42	17	11	suitable
	8	51.8	44	12	4	suitable
	12	31	40	0	0	suitable
Linn 79 TB1	0	53.3	31	6	1	suitable
	4	40.8	29	12	1	suitable
	8	28.6	28	0	0	suitable
	12	21.2	29	0	0	suitable
Linn 77 TB1	0	60.6	31	19	8	select
	4	49.9	34	16	5	select
	8	38.8	33	10	1	suitable
	12	29.4	33	0	0	suitable
Linn 77 TB2	0	56.1	34	18	7	select
	4	51.3	34	12	3	select
	8	41	32	0	0	suitable
	12	22.4	31	0	0	suitable
Linn 77 TB3	0	52.6	33	22	7	select
	4	43.1	32	11	2	select
	8	20.4	32	0	0	suitable
	12	15.8	35	0	0	suitable
Linn 77 TB4	0	59	32	16	6	select
	4	48	43	16	5	select
	8	37	43	14	1	select
	12	33.6	39	0	0	suitable
Linn 77 TB5	0	57.7	30	14	5	select
	4	52.9	34	15	5	select
	8	31.2	33	9	0	suitable
	12	23.4	33	0	0	suitable
Pottawattamie TB1	0	82.6	43	25	20	suitable
	4	78.6	39	9	8	suitable
	8	52.3	40	7	2	suitable
	12	37.5	36	0	0	suitable
Pottawattamie TB2	0	69.2	42	23	14	suitable
	4	60.5	36	5	2	suitable
	8	42.5	36	4	0	suitable

Table 25 continued

County and Test Bed	Cement content (%)	F_{200} (%)	LL (%)	PI (%)	GI	Iowa DOT Suitability
	12	35.3	37	0	0	suitable
Mills TB1	0	96.8	38	4	7	suitable
	4	88	35	8	8	suitable
	8	49.8	34	2	0	suitable
	12	34.5	36	0	0	suitable
Mills TB2	0	89.7	36	5	6	suitable
	4	72.6	34	5	4	suitable
	8	48.3	34	2	0	suitable
	12	29.4	35	0	0	suitable
Scott TB1	0	98.9	39	7	10	suitable
	4	85.2	34	8	7	suitable
	8	52.1	34	3	0	suitable
	12	34.9	35	0	0	suitable
Scott TB2	0	74.7	35	11	8	suitable
	4	61	33	6	2	suitable
	8	46.9	32	0	0	suitable
	12	40	34	0	0	suitable
Scott TB3	0	68.8	28	11	5	suitable
	4	56.4	31	9	3	suitable
	8	37.9	31	1	0	suitable
	12	25.1	33	0	0	suitable
Woodbury (US20) TB1	0	91.2	32	7	7	suitable
	4	65.4	33	7	4	suitable
	8	53.9	33	2	0	suitable
	12	39	34	0	0	suitable
Woodbury (US20) TB2	0	98.7	35	8	9	suitable
	4	76.3	41	10	8	suitable
	8	50.5	40	5	1	suitable
	12	33.8	43	0	0	suitable
Woodbury (US20) TB3	0	95.7	35	12	12	suitable
	4	69.8	40	9	6	suitable
	8	43.2	40	6	1	suitable
	12	32.4	41	0	0	suitable
Woodbury (US20) TB4	0	93.6	31	7	7	suitable
	4	79.1	32	6	4	suitable
	8	51.6	32	1	0	suitable
	12	32.9	33	0	0	suitable
	0	21.4	NV	0	0	suitable

Table 25 continued

County and Test Bed	Cement content (%)	F_{200} (%)	LL (%)	PI (%)	GI	Iowa DOT Suitability
Woodbury (I29) TB1	4	9.3	NV	0	0	suitable
	8	9	NV	0	0	suitable
	12	8.6	NV	0	0	select
Woodbury (I29) TB2	0	16.8	NV	0	0	suitable
	4	7.7	NV	0	0	suitable
	8	7.1	NV	0	0	suitable
	12	7.4	NV	0	0	suitable
Woodbury (I29) TB3	0	17.2	NV	0	0	suitable
	4	8.2	NV	0	0	suitable
	8	9.5	NV	0	0	suitable
	12	8.3	NV	0	0	select

Fines Content (F_{200})

Results of F_{200} versus cement content are presented in Figure 156 and Figure 157. The results indicated that F_{200} decreased with increasing cement content. Statistical analysis was conducted to predict F_{200} after treatment as a function of cement content, F_{200} before treatment, and Atterberg limits. Results are summarized in Table 10. Cement content, F200 before treatment, and LL were found to be statistically significant. PI and PL parameters were not statistically significant. Measured versus predicted F_{200} (after treatment) results from the multi-variate model are presented in Figure 158. The model showed an R^2 of about 0.9 and RMSE of about 7%.

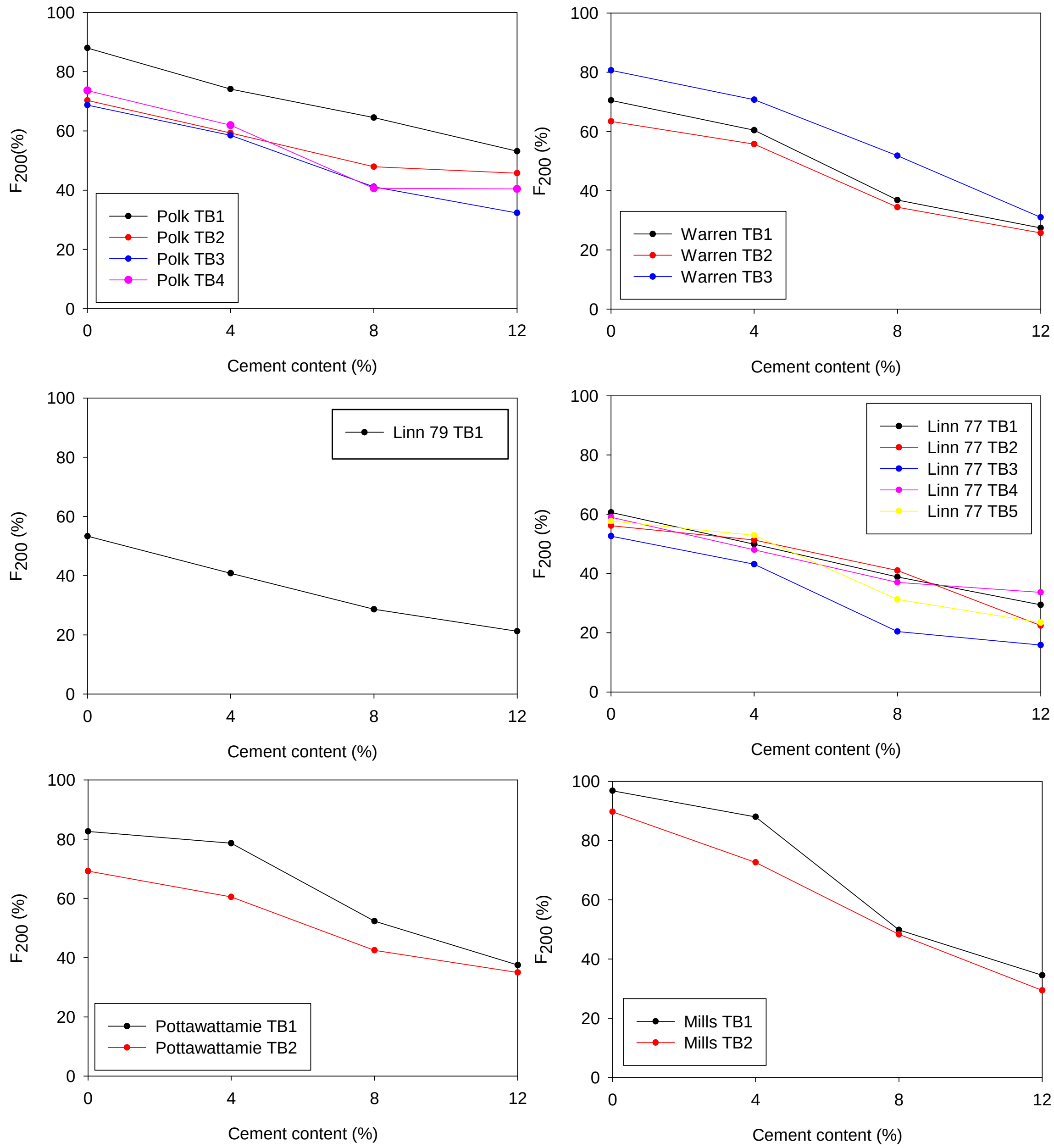


Figure 156. F_{200} versus cement content

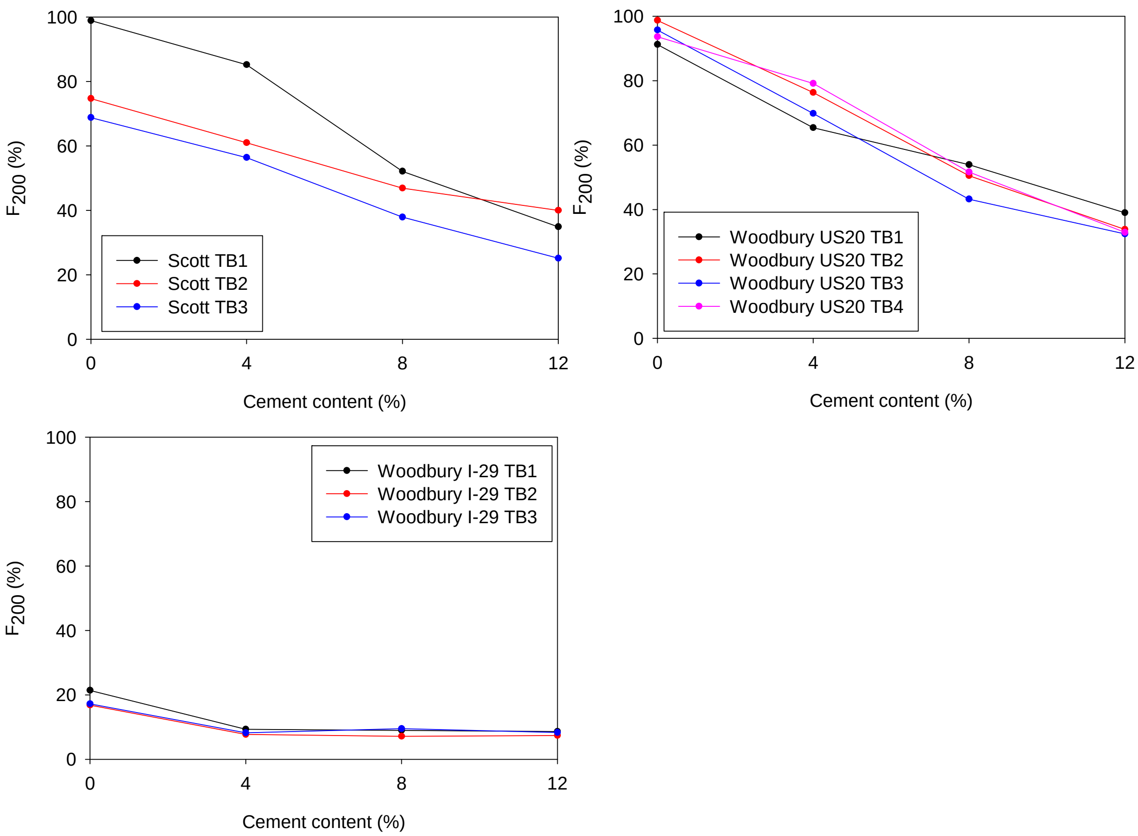


Figure 157. F_{200} versus cement content (continued)

Table 26. Multi-variate analysis results to predict F_{200} after cement stabilization

Parameter	Value	t Ratio	Prob> |t|	R^2	RMSE
Intercept	18.92	3.96	< 0.0001	0.898	6.588
Cement Content (%)	-3.74	-24.88	< 0.0001		
F_{200} before treatment (%)	0.607	13.23	< 0.0001		
LL (%)	0.306	2.79	0.0064		
Prediction expression	F_{200} after treatment (%) = 18.92 - 3.74 x cement content (%) + 0.607 x F_{200} (%) + 0.306 x LL (%)				

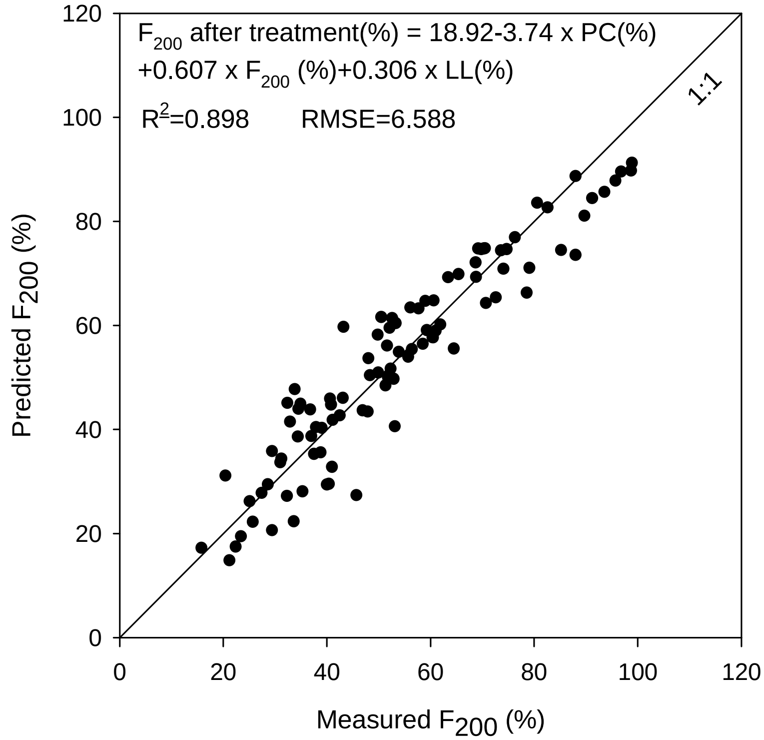


Figure 158. Comparison of measured F_{200} and predicted F_{200}

Atterberg Limits

Plasticity charts showing relationship between LL and PI for unstabilized and stabilized soils with 4%, 8%, and 12% cement content are shown in Figure 158 to Figure 162, respectively. F_{200} versus of PI results are shown in Figure 163. LL and PI versus cement content are presented in Figure 164 to Figure 166.

With the exception of a few materials (Polk TB4, Linn 79, Linn 77 TB4), the LL and PI of all materials decreased with increasing cement content. The one untreated soil classified as “unsuitable”, classified as “suitable” after stabilized with cement. Some of the “select” untreated soils classified as “suitable” after stabilized with cement, because of reduction in PI. All of the soils classified as “suitable” at 12% cement content because of no plasticity.

Statistical analysis was conducted to predict PI after treatment as a function of cement content, cement content, clay content, silt content, and LL. Results are summarized in Table 11. Cement content and clay content were found to be statistically significant, while the remaining parameters were not statistically significant. Measured versus predicted PI (after treatment) results from the multi-variate model are presented in Figure 167. The model showed an R^2 of about 0.5 and RMSE of about 5%.

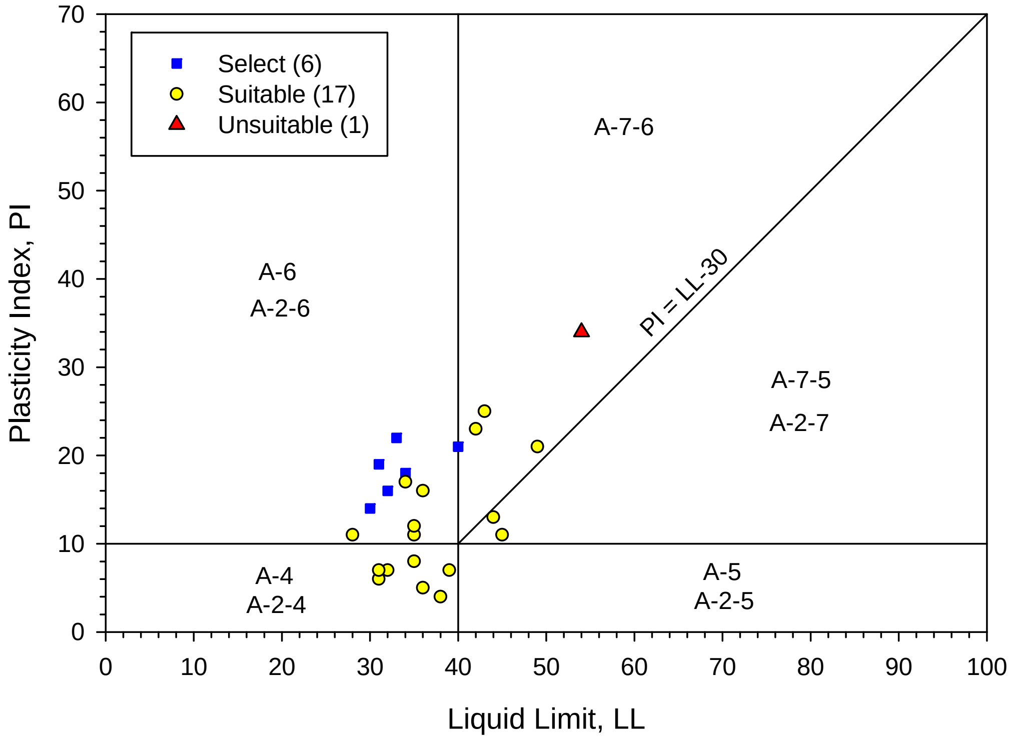


Figure 159. Plasticity chart with results of unstabilized soils

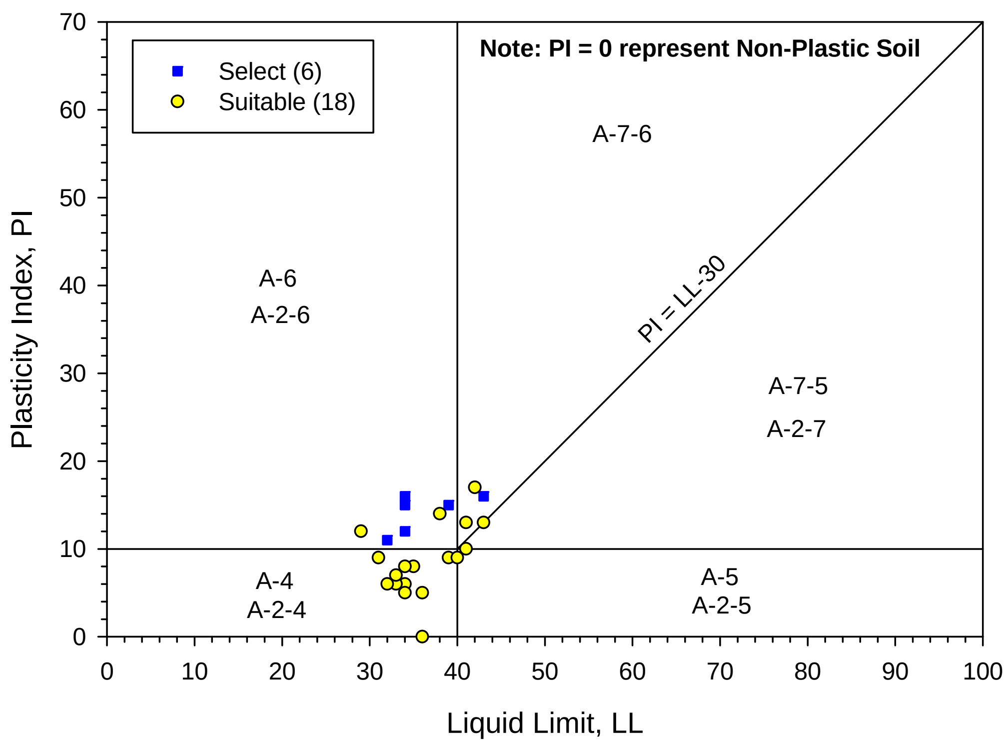


Figure 160. Plasticity chart with results of 4% cement stabilized soils

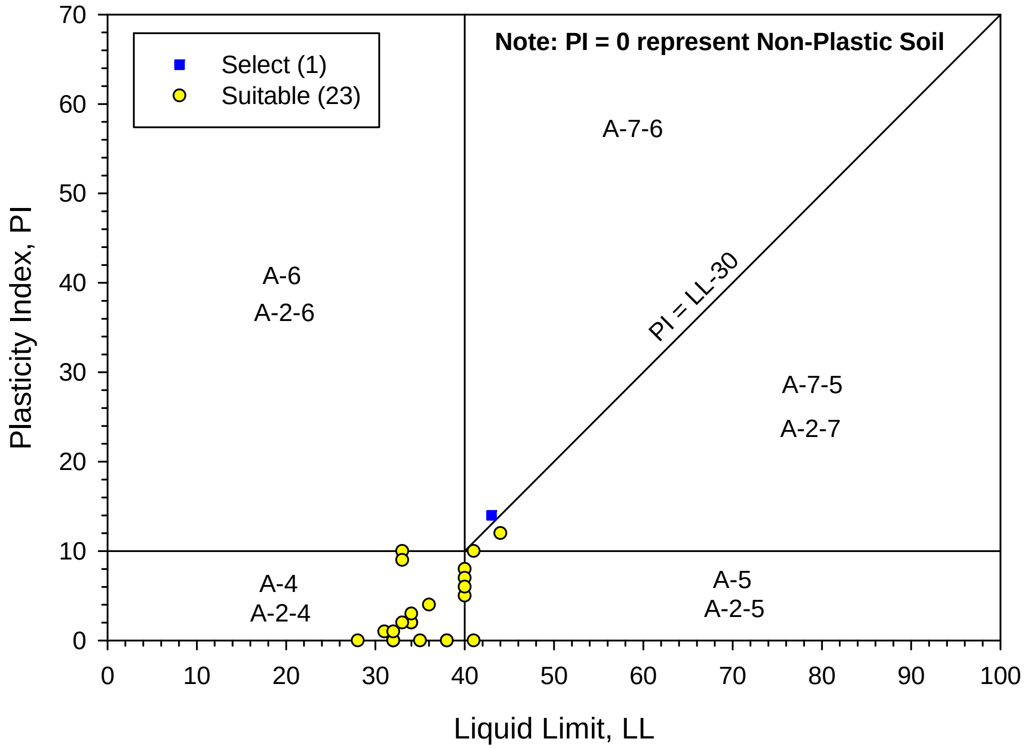


Figure 161. Plasticity chart with results of 8% cement stabilized soils

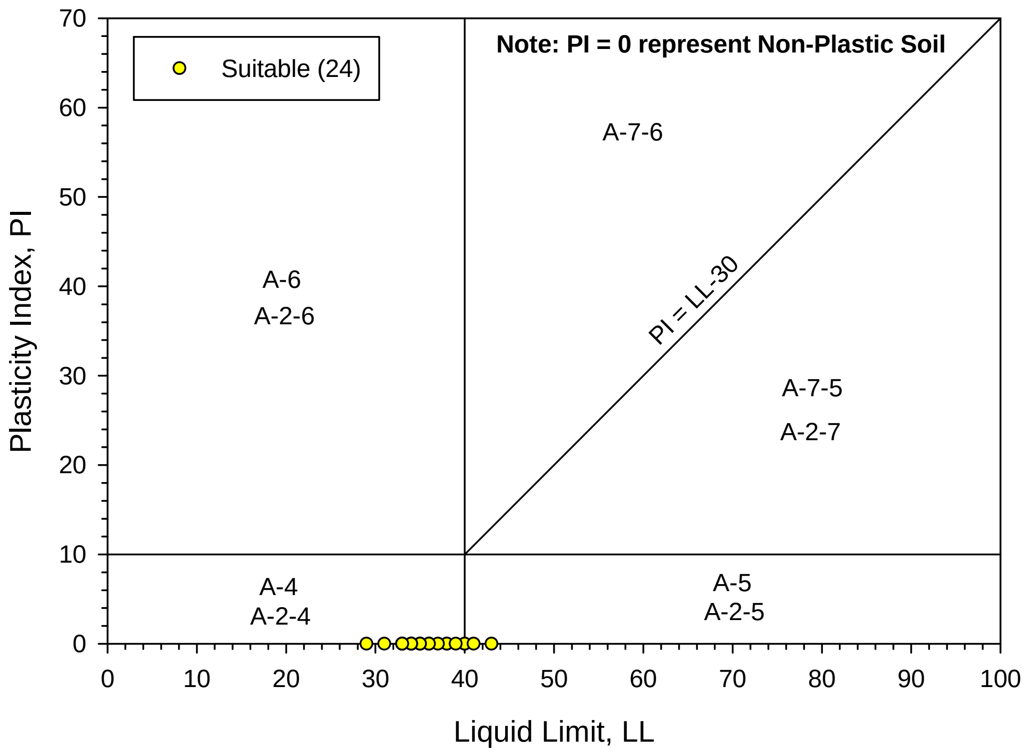


Figure 162. Plasticity chart with results of 12% cement stabilized soils

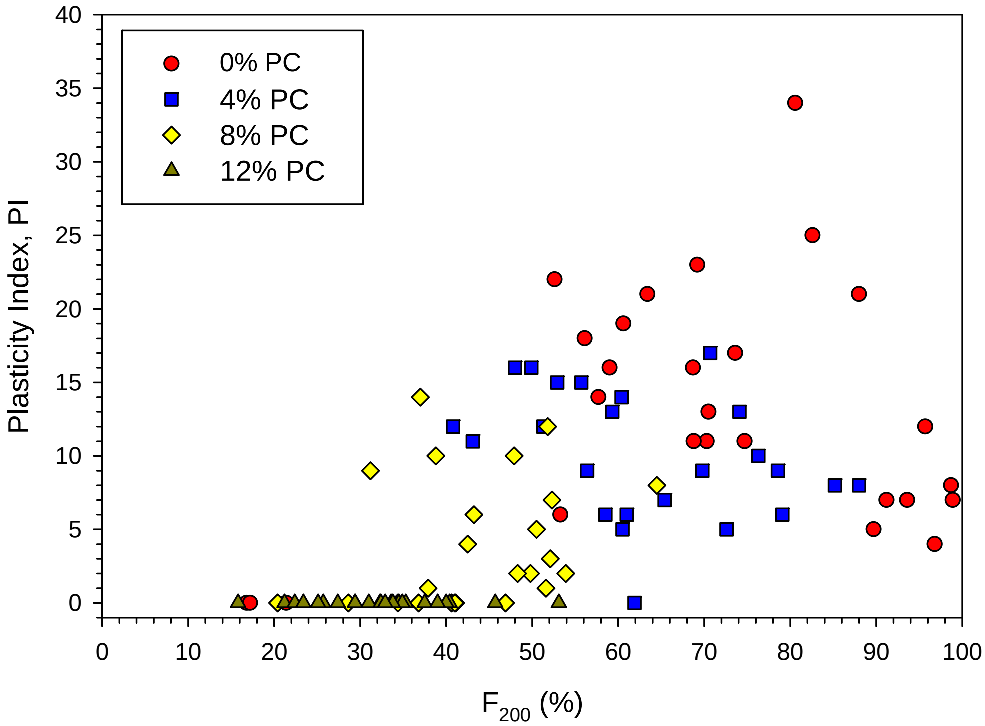


Figure 163. PI versus F200 for unstabilized and stabilized soils

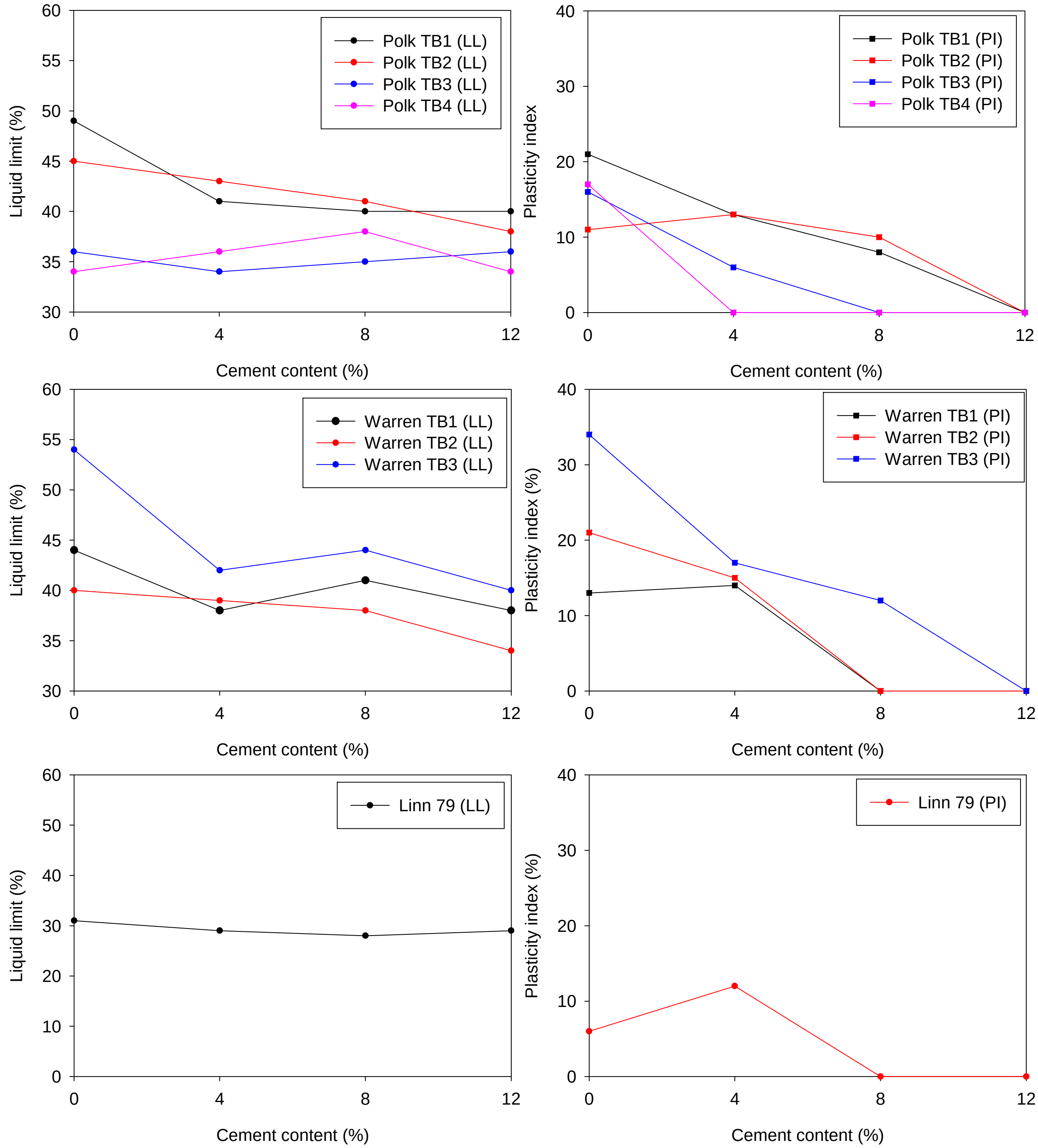


Figure 164. LL and PI versus cement content

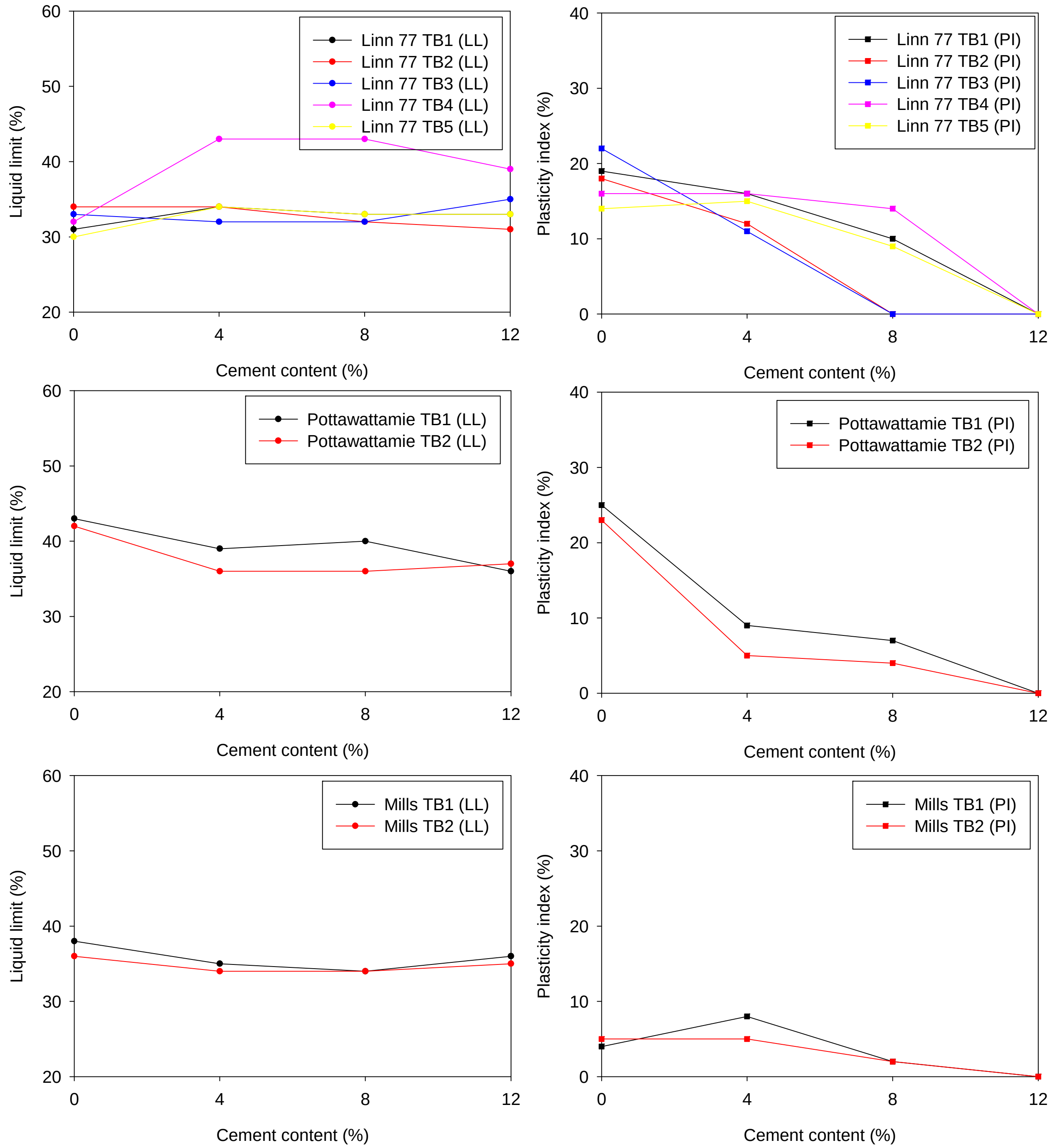


Figure 165. LL and PI versus cement content (continued)

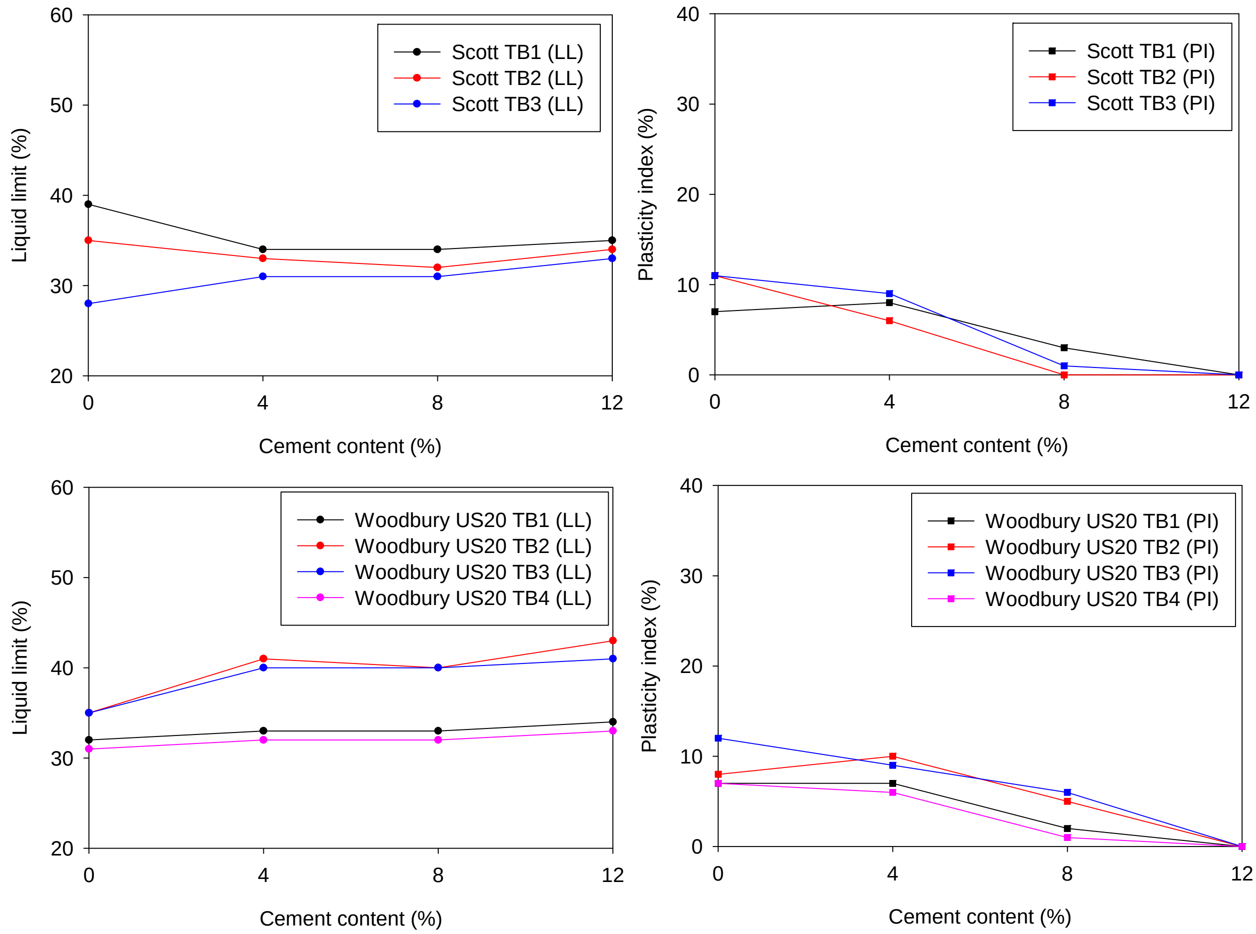


Figure 166. LL and PI versus cement content (continued-2)

Table 27. Multi-variate analysis results to predict PI after cement stabilization

Parameter	**Value**	**t Ratio**	**Prob> |t|**	**R^2**	**RMSE**
Intercept	8.664	5.85	< 0.0001	0.509	5.101
Cement Content (%)	-1.102	-10.04	< 0.0001		
Clay content (%)	0.172	3.49	0.0007		
Prediction expression	F_{200} after treatment (%) = 8.664 – 1.102 x cement content (%) + 0.172 x Clay content (%)				

Note: Silt content, sand content, and LL were not statistically significant

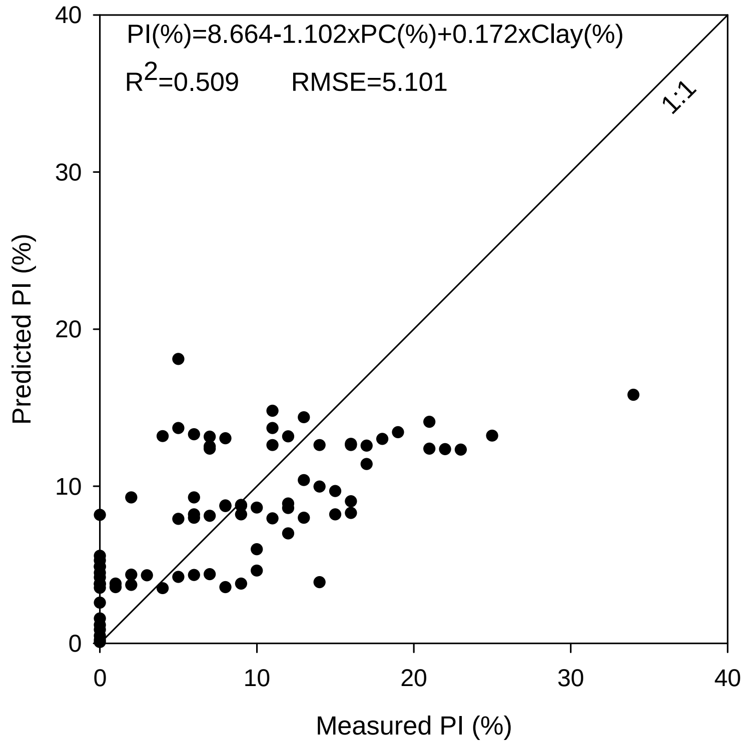


Figure 167. Comparison of measured PI and predicted PI

AASHTO Group Index (GI)

GI versus cement content results are presented Figure 168 to Figure 169. For a majority of the soils, the GI values decreased with increasing cement content. Statistical analysis was conducted to predict GI after treatment as a function of cement content, clay content, silt content, F_{200}, LL, and PI. Results are summarized in Table 12. Cement content, F_{200}, LL, and PI were found to be statistically significant, while the remaining parameters were not statistically significant. Measured versus predicted GI (after treatment) results from the multi-variate model are presented in Figure 170. The model showed an R^2 of about 0.7 and RMSE of about 3.

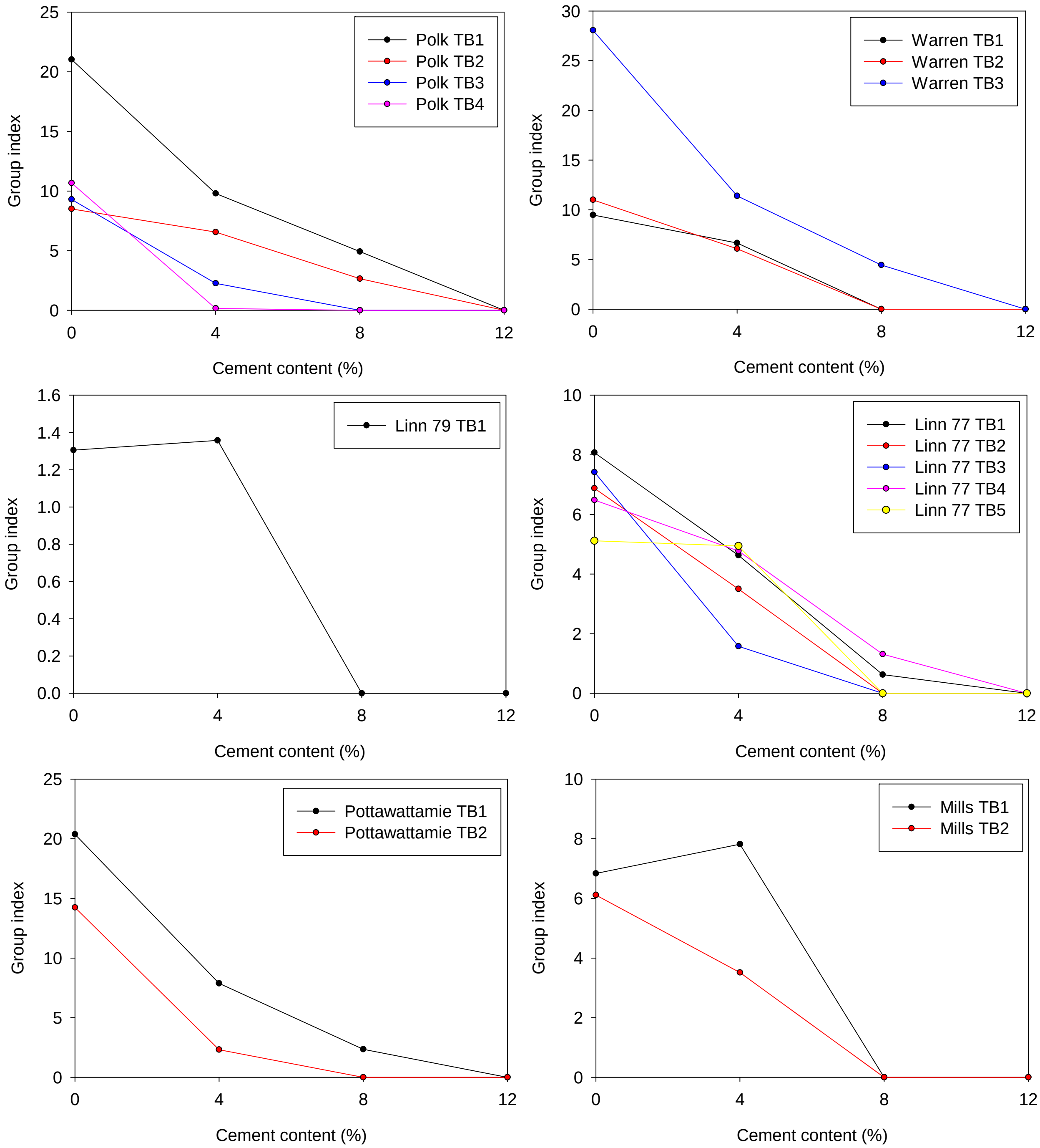


Figure 168. AASHTO group index versus cement content

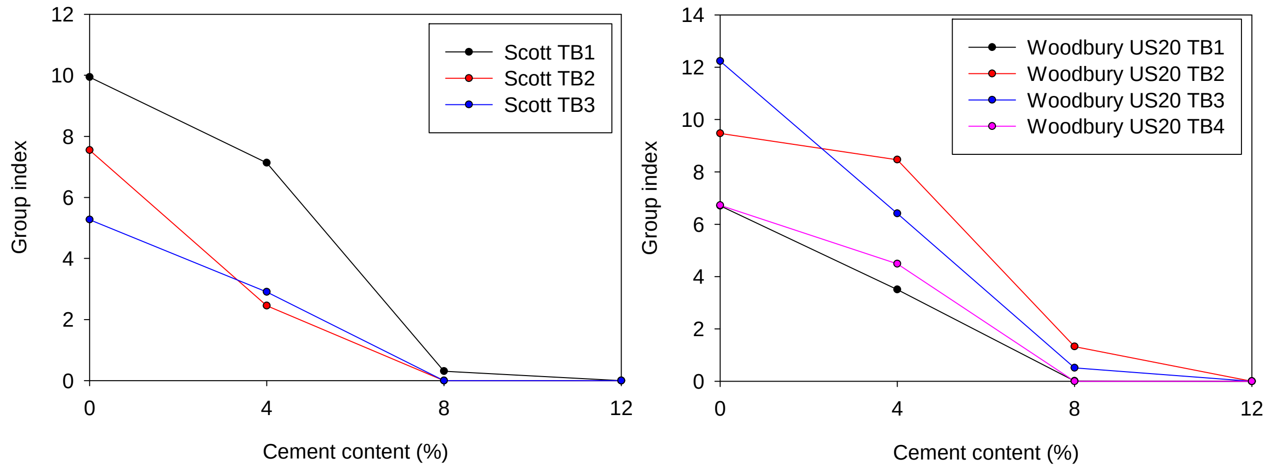


Figure 169. AASHTO group index versus cement content (continued)

Table 28. Multi-variate analysis results to predict GI after cement stabilization

Parameter	Value	t Ratio	Prob> |t|	R^2	RMSE
Intercept	-4.540	-2.23	0.0281	0.708	2.774
Cement Content (%)	-0.844	-13.33	<0.0001		
F_{200} (%)	0.069	2.85	0.0055		
LL (%)	0.157	2.98	0.0164		
PI (%)	0.172	2.45	0.0037		
Prediction expression	GI = - 4.540 – 0.844 x cement content (%) + 0.069 x F_{200} (%) + 0.157 x LL (%) + 0.172 x PI (%)				

Note: Silt content and clay content were not statistically significant.

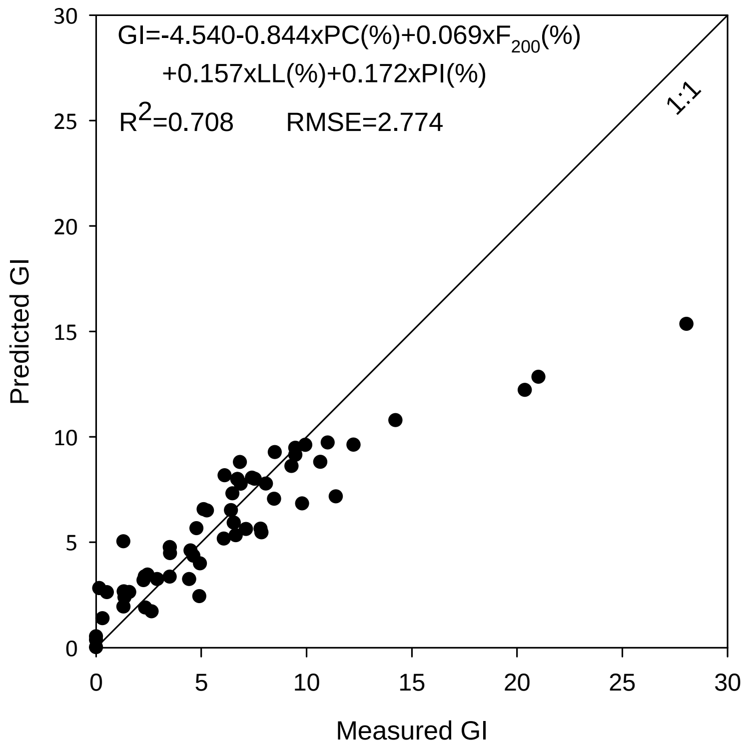


Figure 170. Comparison of measured group index and predicted group index

Unconfined Compressive Strength

Figure 171 to Figure 173 present the results of unsaturated and vacuum saturated UCS of the materials at different cement contents. A linear regression line is fit to the data to define the relationship between UCS and cement content. Results indicated increasing UCS with increasing cement content, as expected. For a majority of the unstabilized materials, the soil specimens became fragile after vacuum saturation and could not be retrieved from the vessel. For those soils, UCS of 0 psi is reported herein. Vacuum saturated stabilized specimens resulted in UCS measurements that were on average about 1.5 times lower than the unsaturated specimens. The ratio of unsaturated and vacuum saturated UCS of stabilized specimens ranged from about 1.1 to 2.5.

Statistical analysis was conducted to predict unsaturated and vacuum saturated UCS as a function of cement content, sand content, clay content, silt content, F_{200}, LL, and PI. Results are summarized in Table 13 and Table 14. Cement content, sand content, F_{200}, and LL were found to be statistically significant, while the remaining parameters were not statistically significant. Measured versus predicted UCS results from the multi-variate model are presented in Figure 174 and Figure 175. The models showed an R^2 of about 0.85 and RMSE of about 75 psi for vacuum saturated UCS and 97 psi for unsaturated UCS.

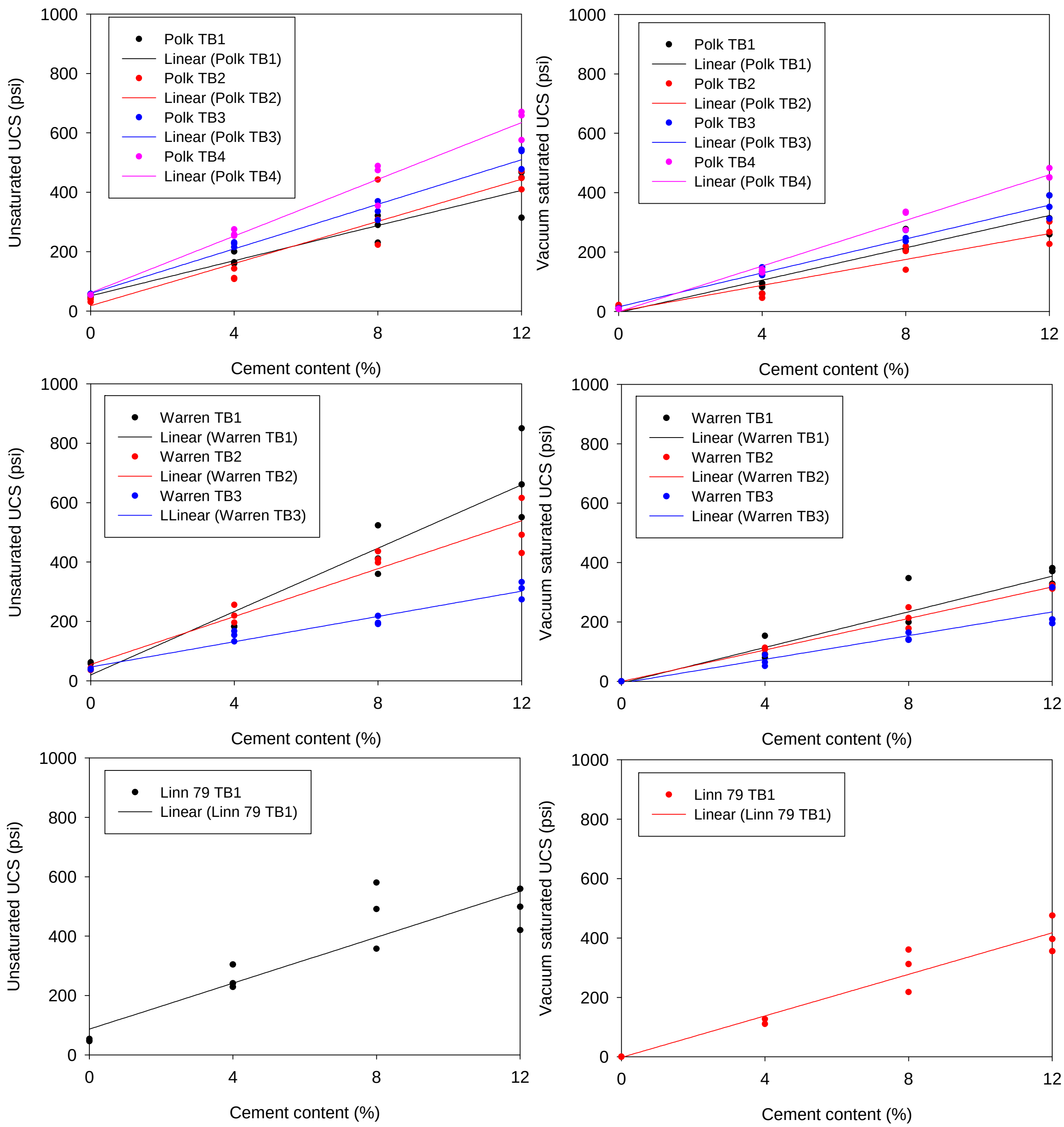


Figure 171. Unsaturated and vacuum saturated UCS versus cement content

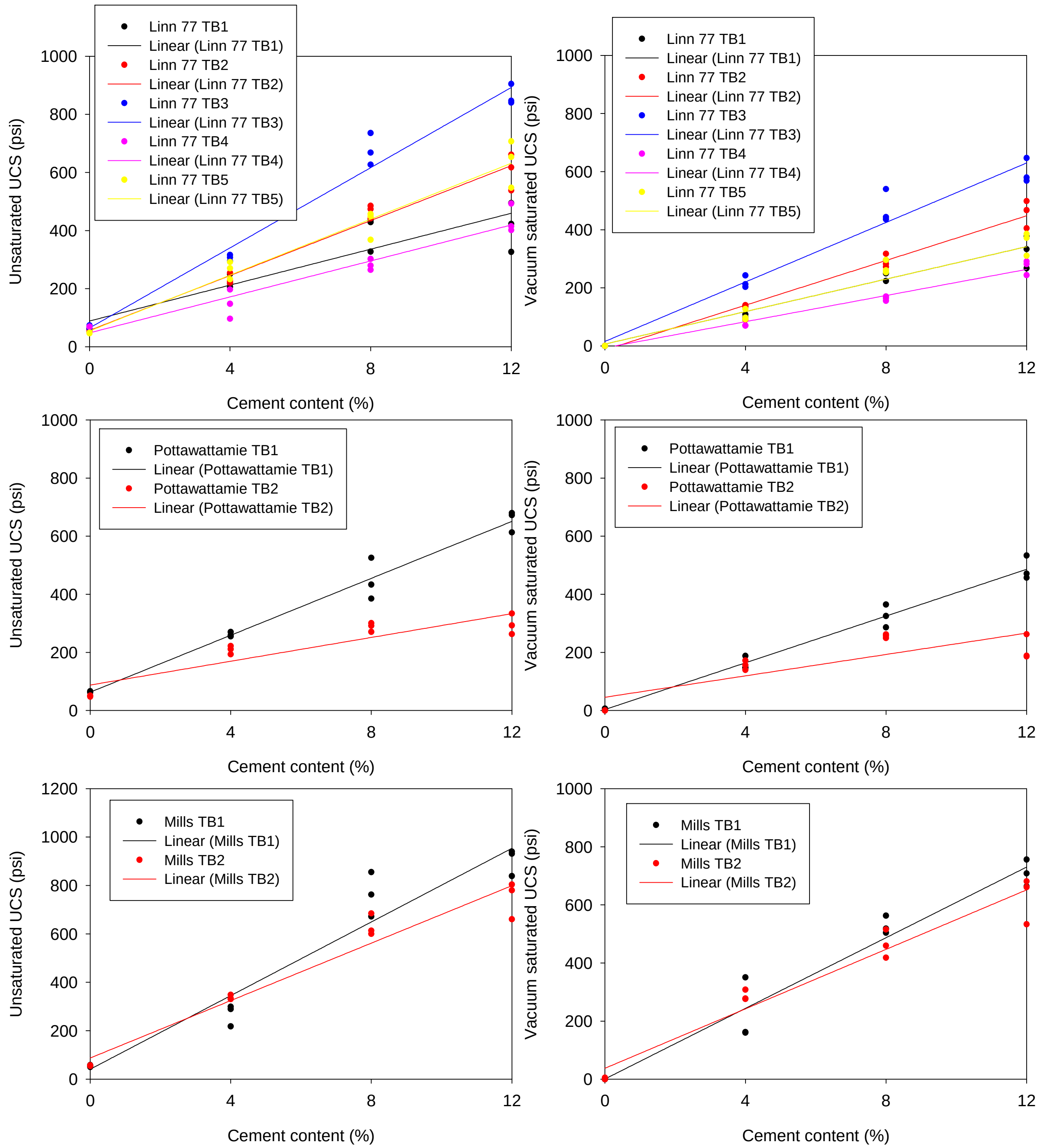


Figure 172. Unsaturated and vacuum saturated UCS versus cement content (continued)

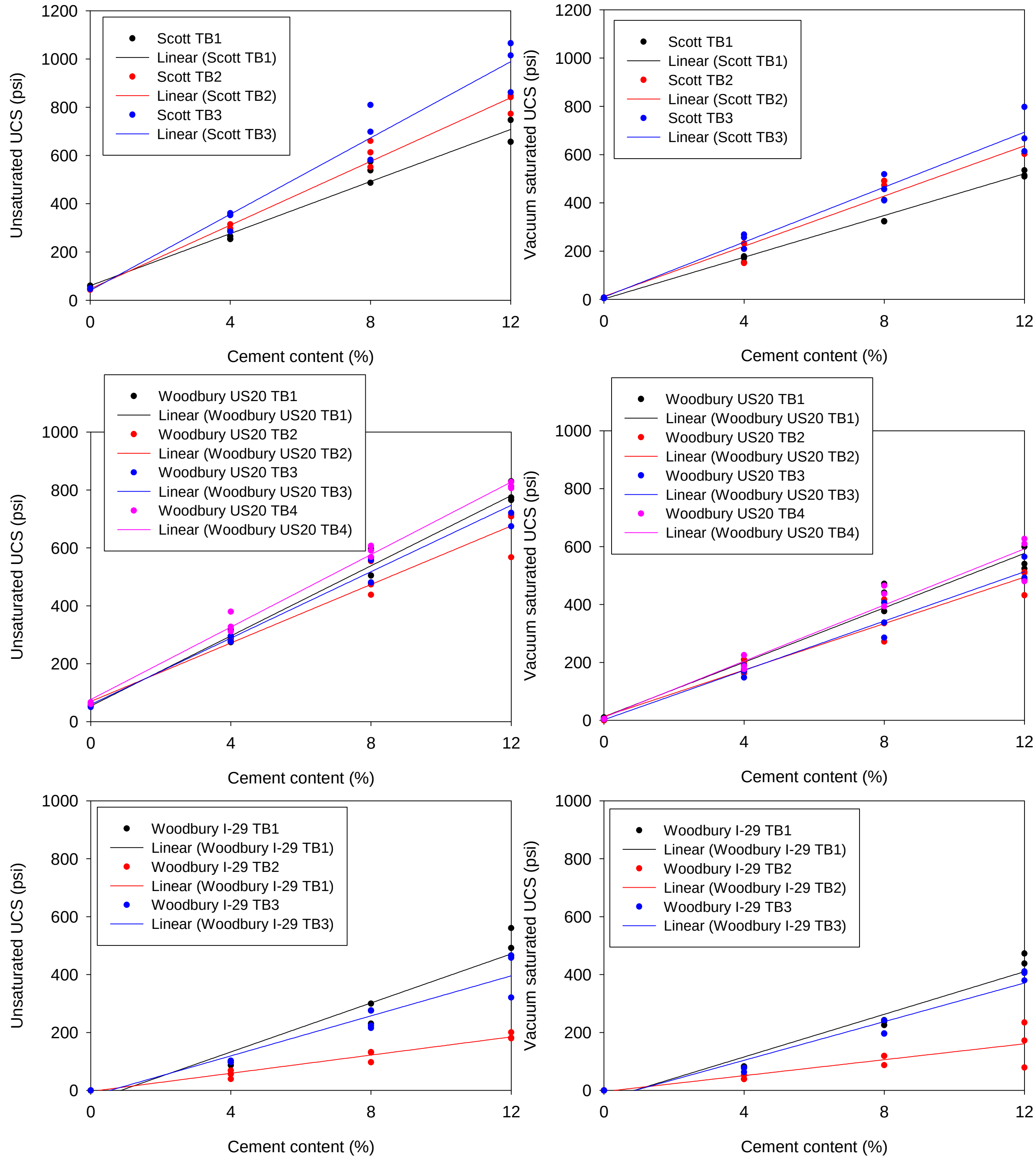


Figure 173. Unsaturated and vacuum saturated UCS versus cement content (continued-2)

Table 29. Multi-variate analysis results to predict unsaturated UCS

Parameter	Value	t Ratio	Prob> |t|	R^2	RMSE
Intercept	1465.38	3.61	0.0005	0.848	97.418
Cement content (%)	48.69	21.90	<0.0001		
Sand (%)	-13.26	-3.13	0.0023		

Table 29 continued

Parameter	Value	t Ratio	Prob> |t|	R^2	RMSE
F_{200} (%)	-9.24	-2.35	0.0209		
LL (%)	-11.28	-6.77	<0.0001		
Prediction expression	UCS (psi) = 1465.38 + 48.69 x cement content (%)- 13.26 x Sand (%) - 11.28 x LL (%) - 9.24 x F_{200} (%)				

Note: Silt content and clay content were not statistically significant.

Table 30. Multi-variate analysis results to predict vacuum saturated UCS

Parameter	Value	t Ratio	Prob> |t|	R^2	RMSE
Intercept	1151.32	3.7	0.0004	0.850	74.704
Cement content (%)	37.33	21.89	<0.0001		
Sand (%)	-11.40	-3.51	0.0007		
F_{200} (%)	-7.70	-2.56	0.0123		
LL (%)	-8.37	-6.55	<0.0001		
Prediction expression	UCS (psi) = 1151.323 + 37.329 x cement content (%) - 11.401 x Sand (%) - 8.372 x LL (%) - 7.703 x F_{200} (%)				

Note: Silt content and clay content were not statistically significant.

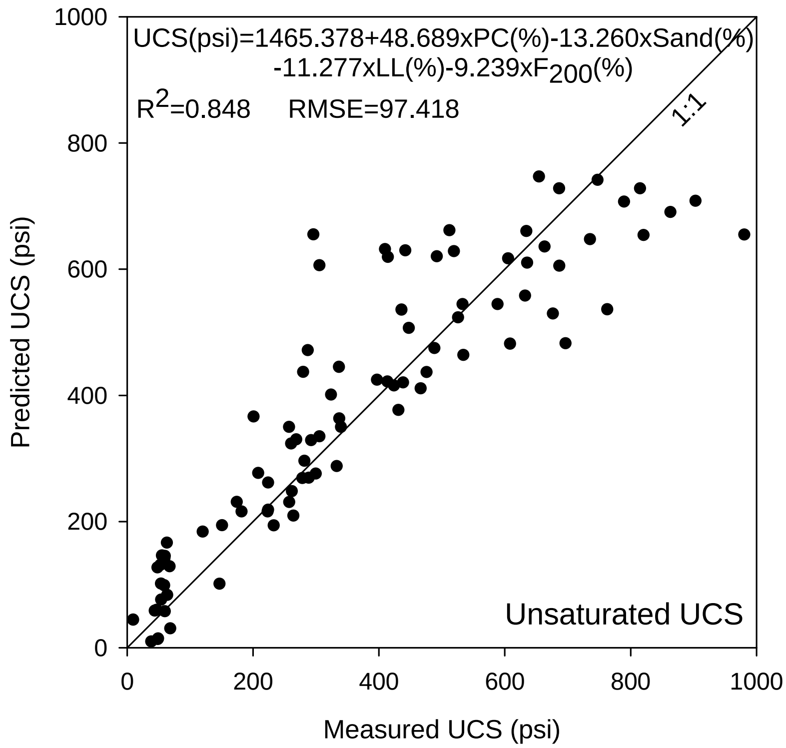


Figure 174. Comparison of measured unsaturated UCS and predicted unsaturated UCS

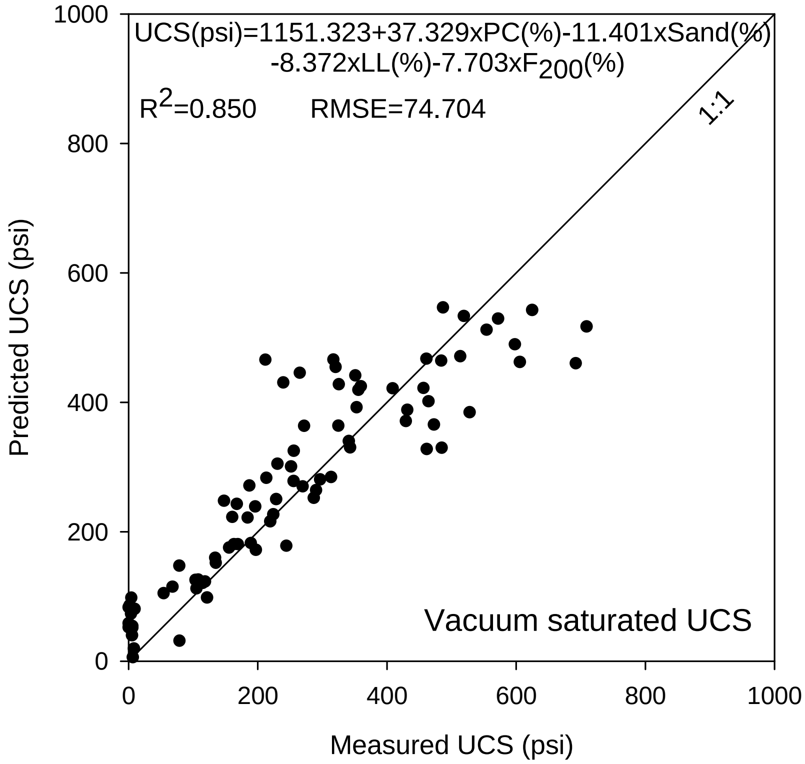


Figure 175. Comparison of measured vacuum saturated UCS and predicted UCS

One-dimensional Consolidation

Specimens from 21 embankment construction test beds were tested to determine the compression index, swelling index, and coefficient of consolidation. These parameters are used in settlement estimation. The detailed data was summarized in the Appendix.

To study the effect of compaction energy on consolidation properties, three groups of consolidation tests were performed. The specimens for consolidation test were obtained from Proctor test samples directly. Three different compaction energies were perform in Proctor tests. Table 31 summarized the input parameters of Proctor tests. The compaction energy was increased as an order from standard-minus to standard-plus.

Table 31. Summary of the Proctor-consolidation input parameters

Energy level	Layers	Blows per layer	Hammer weight (lb)	Drop height (ft)	Energy (ft-lbf/ft^3)	Optimum moisture (%)
Standard-minus	3	15	5.5	1.0	7425	19.0
Standard	3	25	5.5	1.0	12375	18.6
Standard-plus	5	25	5.5	1.0	20790	17.5

Figure 176 is the Proctor curve for Iowa loess at three different compaction energies. As compaction energy was increased, the dry unit weight of specimen was increased either, and the optimum moisture content was decreased. The optimum moistures were 16.2% to 19.2%, and the dry unit weights were 102.3 pcf to 108.8 pcf. The compression indices and swelling indices were also different at different moisture content and different compaction energy. The effect of compaction energy on the consolidation parameters was discussed in the following chapter.

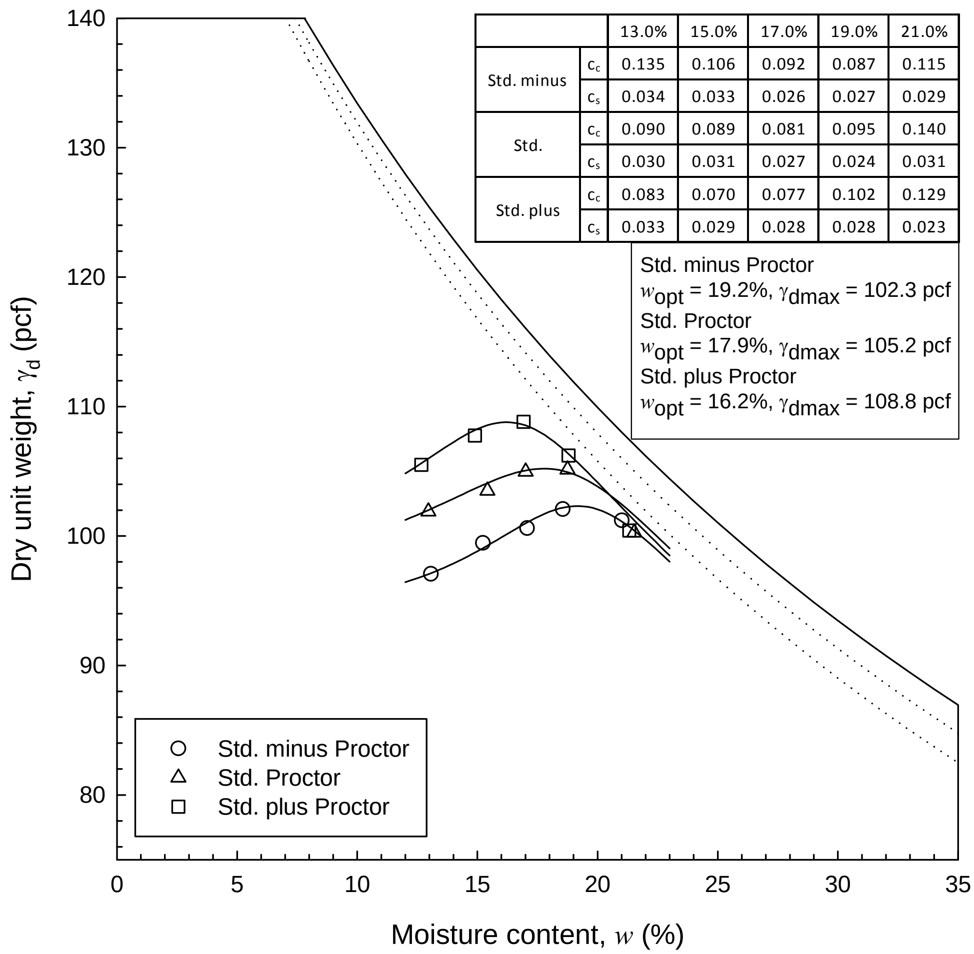


Figure 176. Proctor curve for Iowa loess at three compaction energies

Statistical analysis was conducted to predict c_c as a function of void ratio (e), D60, D85, and liquid limit (LL). Results are summarized in Table 32. Void ratio, D60, D85, and LL were found to be statistically significant, while the remaining parameters were not statistically significant. Measured versus predicted compression index from the multi-variate model are presented in Figure 177. The model showed an R^2 of about 0.674 and RMSE of about 0.018.

Table 32. Multi-variate regression results to predict compression index c_c

Parameter	Value	t ratio	Prob>|t|	R^2	RMSE
Intercept	-0.079	-3.55	0.0008	0.674	0.018
Void ratio, e	0.066	2.06	0.0443		
D60 (mm)	0.4	3.47	0.0011		
D85 (mm)	-0.043	-2.85	0.0062		
LL (%)	0.004	7.68	<0.0001		

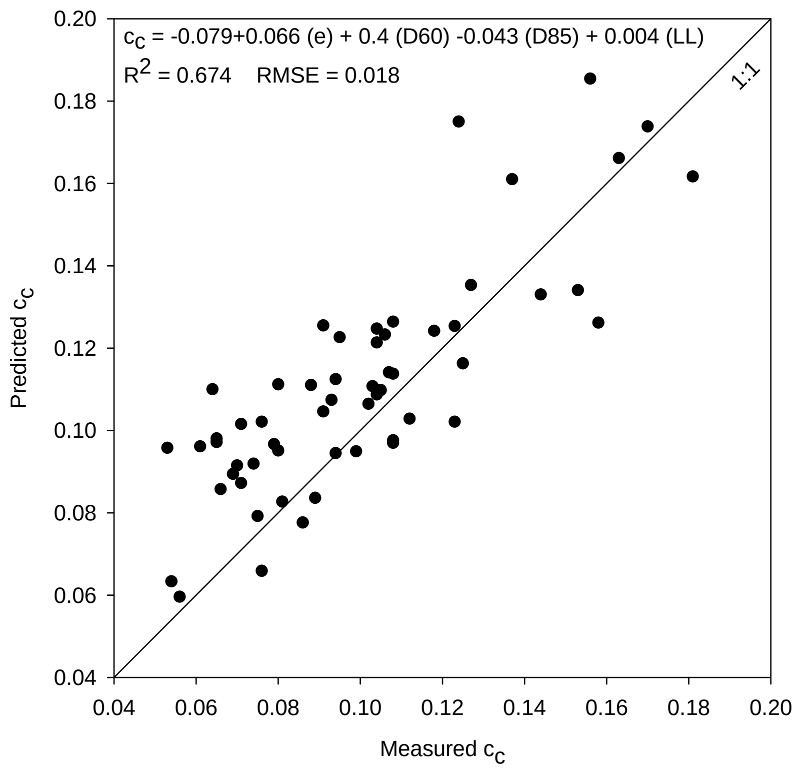


Figure 177. Correlations between compression index (c_c) and engineering properties of soil

Statistical analysis was conducted to predict swelling index as a function of void ratio (e), clay content, liquid limit (LL), plastic limit (PL). Results are summarized in Table 33. Void ratio, clay content, LL, and PL were found to be statistically significant, while the remaining parameters were not statistically significant. Measured and predicted swelling index from the multi-variate model are presented in Figure 178. The model showed an R^2 of about 0.489 and RMSE of about 0.008.

Table 33. Multi-variate regression results to predict swelling index c_s

Parameter	Value	t ratio	Prob>|t|	R^2	RMSE
Intercept	0.030	4.33	<0.0001	0.49	0.0084
Void ratio, e	-0.064	-4.81	<0.0001		
Clay content (%)	-0.0006	-3.94	0.0002		
LL (%)	0.001	6.14	<0.0001		
PL (%)	0.0006	2.89	0.0056		

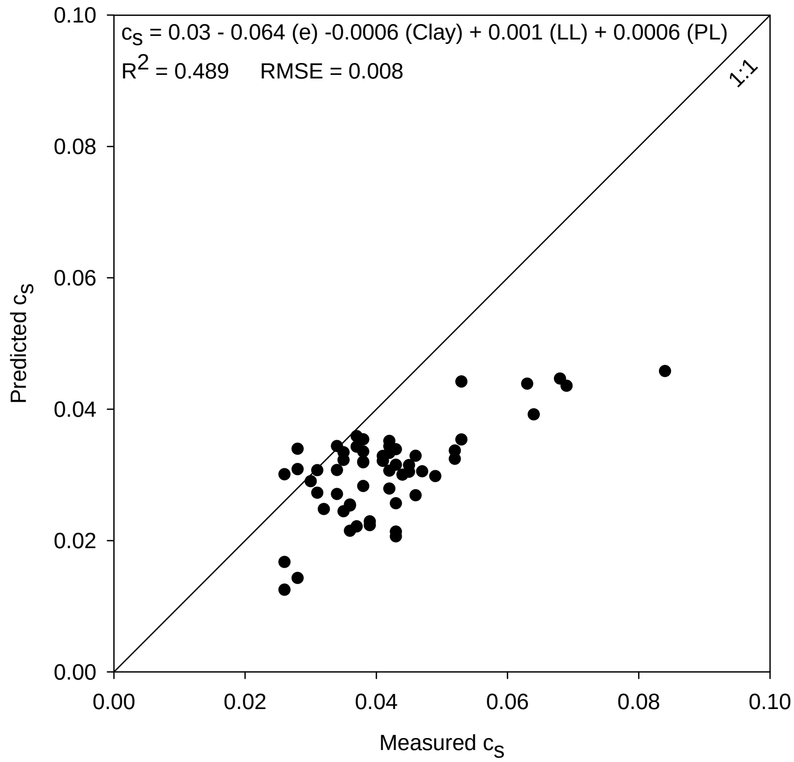


Figure 178. Correlations between swelling index and engineering properties of soil

CHAPTER 7. DATA ANALYSIS AND DISCUSSION

Field Test Results

Figure 179 compares the standard Proctor optimum moisture content and maximum dry unit weight selected by the Iowa DOT for QA testing and the corresponding values measured by the ISU research team for all project sites. The dotted line (1:1 line) represents an ideal condition in which the DOT Proctor and ISU Proctor data are in exact agreement, while the black solid line represents the best regression fit. The dash lines represent the acceptable limits of variation between two values obtained from two different laboratories for CL soils, per ASTM D698. A few soils were classified as CH and SM, and these soils are identified as different colored symbols on the figure along with the allowable limits of variation per ASTM D698. The dash-dot lines represent the allowable limits of variation between two values obtained from different laboratories, per AASHTO T 99-01 (2009). Note that AASHTO T 99 does not provide different allowable variation limits for different soil types, as ASTM D698.

Figure 179 shows that there were variations between ISU Proctor data and Proctor data selected for QA by the Iowa DOT. It is possible that these differences resulted from variations in the test methods and procedures that were used to obtain these measurements. For instance, at most sites the field DOT engineers conducted Proctor tests using hand-operated equipment, while ISU Proctor tests were conducted using automatic machine-operated equipment. Also, the materials selected by ISU directly from the test area could have been slightly different from the Proctor database that the DOT used for comparing their field measurements. A comparison between the measured and selected values showed a standard error of 2.9 lb/ft^3 for maximum dry density and 2.1% for optimum moisture content. The difference in optimum moisture content was as high as 4% and the difference in maximum dry density was as high as 6.5 lb/ft^3.

For maximum dry density, AASHTO T 99 allows 4.5 lb/ft^3 variation between two test results from different laboratories, while ASTM D698 allows 2.3 lb/ft^3 to 3.9 lb/ft^3, depending on the soil type. Results indicated that only 1 of 19 test results fell outside the allowable limits per AASHTO T 99, while 7 of 19 fell outside the allowable limits per ASTM D698. For optimum moisture content, AASHTO T 99 allows variation of 15% from the mean of the two test results, while ASTM D698 allows a variation of 1.5% to 1.8%, depending on the soil type. Only 3 of 26 test results fell outside the allowable limits per AASHTO T 99, while 7 of 26 fell outside the allowable limits per ASTM D698.

For maximum dry density, AASHTO T 99 allows 4.5 lb/ft^3 variation between two test results from different laboratories, while ASTM D698 allows 2.3 lb/ft^3 to 3.9 lb/ft^3, depending on soil type. Only 1 of 19 test results fell outside the allowable limits per AASHTO T 99, while 7 of 19 fell outside the allowable limits per ASTM D698. For optimum moisture content, AASHTO T 99 suggests an acceptable variation of 15% from the mean of the two test results, while ASTM D698 suggests an acceptable variation of 1.5% to 1.8%, depending on soil type. Only 3 of 26 test results fell outside the allowable limits per AASHTO T 99, while 7 of 26 fell outside the allowable limits per ASTM D698.

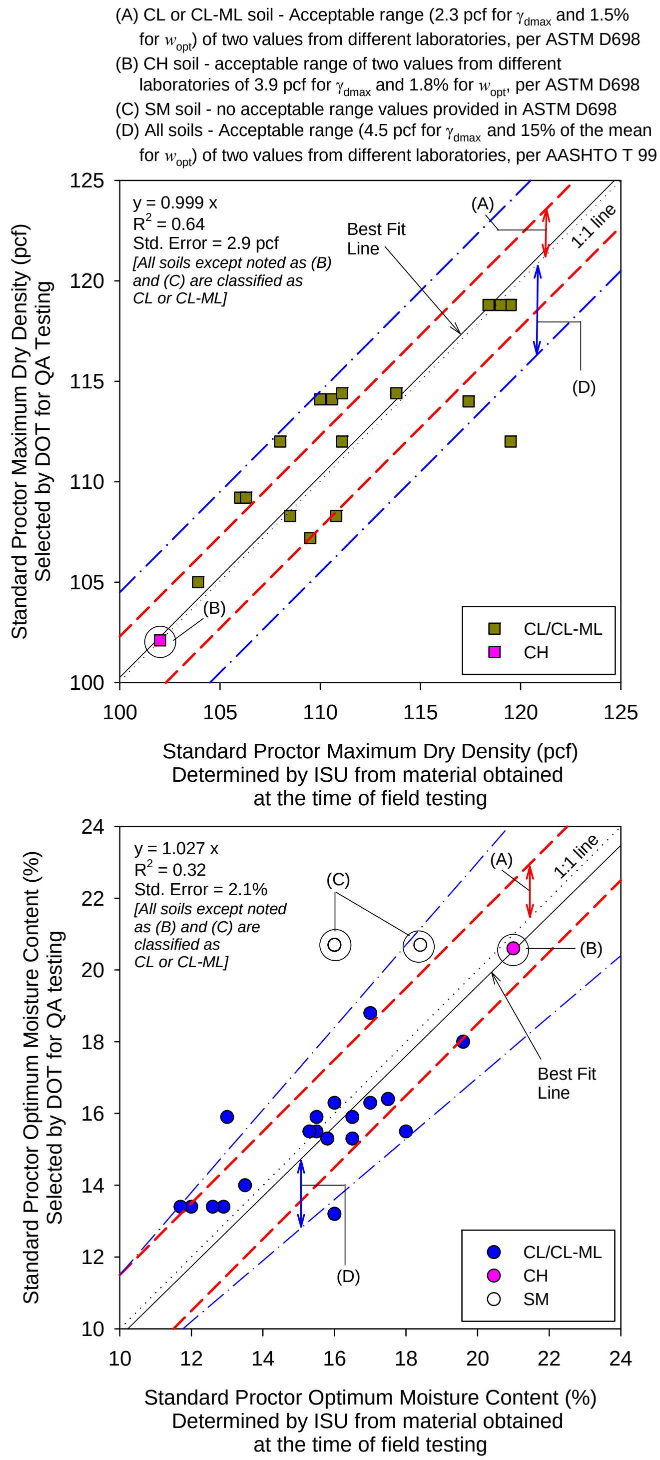


Figure 179. Comparison between Proctor test results (optimum moisture content and maximum dry density) selected by the Iowa DOT for QA testing and measured Proctor test results from the ISU research team for all project sites

Table 34 shows a summary of the percentage of test points outside of the specification control limits in the contractor QC data, the Iowa DOT QA data, and the ISU testing data.

Table 34. Summary of the percentage of test points outside of the specification control limits in contractor QC data, Iowa DOT QA data, and ISU data

Project [Dates of Testing]	**Materials**	**Specification**	**No. of Tests**	**% of Data outside Specification Control Limits for Final Test Results**		
				Contractor QC Testing	**Iowa DOT QA**	**ISU Testing**
Polk [QC: 8/11/14-9/30/14] [ISU: 5/29/14, 8/5/14, 8/19/14]	Cohesive	Moisture	59 (QC) 45 (ISU)	5 (dry) 7 (wet)	—	2 (dry) 51 (wet)
		Density	56 (QC) 45 (ISU)	2	—	4
Warren [QC: 4/2/14-11/6/14] [ISU: 6/3/14, 7/22/14, 8/4/14]	Cohesive	Moisture	178 (QC) 45 (ISU)	1 (wet)	—	16 (dry) 18 (wet)
		Density	45 (ISU)	*	*	38
Linn-77 [QC: 4/4/14-12/2/14] [ISU: 6/6/14, 7/15/14, 8/1/14, 9/8/14]	Cohesive	Moisture	564 (QC) 60 (ISU)	1 (wet)	—	2 (dry) 10 (wet)
		Density	60 (ISU)	*	*	5
	Cohesionless	Moisture	31 (QC)	97 (dry)	—	—
		Density	31 (QC)	0	—	—
	Cohesionless	Moisture	285 (QC)	81 (dry) 4 (wet)	—	—
Linn-79 [QC: 5/27/14-6/16/14] [ISU: 6/6/14]	Cohesive	Moisture	85 (QC) 3 (QA) 15 (ISU)	11 (dry) 2 (wet)	0	0
		Density	15 (ISU)	*	*	0
	Cohesionless	Moisture	22 (QC)	100 (dry)	—	—
		Density	22 (QC)	14	—	—
Mills [QC: 5/21/14-8/14/14] [ISU: 6/26/14]	Cohesive	Moisture	150 (QC) 30 (ISU)	1 (dry)	—	50 (wet)
		Density	30 (ISU)	*	*	40

Table 34 continued

Project [Dates of Testing]	Materials	Specification	No. of Tests	% of Data outside Specification Control Limits for Final Test Results		
				Contractor QC Testing	Iowa DOT QA	ISU Testing
Pottawattamie [QC: 11/19/13-7/14/14] [QA: 7/2/14-7/11/14] [ISU: 7/2/14, 7/10/14]	Cohesive	Moisture	93 (QC) 16 (QA) 30 (ISU)	1 (dry) 9 (wet)	50 (dry) 13 (wet)	40 (wet)
		Density	30 (ISU)	*	*	13
Woodbury-I29 [QC: 6/10/14-10/16/14] [QA: 6/25/14-10/3/14] [ISU: 7/9/14, 7/10/14, 8/7/14]	Cohesionless	Moisture	437 (QC) 35 (QA) 45 (ISU)	1 (dry) 1 (wet)	11 (dry) 9 (wet)	2 (dry) 64 (wet)
Scott [QC: 7/16/14-9/22/14] [QA: 7/11/14-9/29/14] [ISU: 7/16/14, 7/31/14, 9/19/14]	Cohesive	Moisture	55 (QC) 48 (QA) 45 (ISU)	9 (dry) 36 (wet)	4 (dry) 65 (wet)	62 (wet)
		Density	5 (QC) 45 (ISU)	75	*	11
Woodbury-US20 [ISU: 9/26/14, 10/18/14]	Cohesive	Moisture	59 (ISU)	—	—	5 (dry) 51 (wet)
		Density	59 (ISU)	*	*	20

— Data not available; * not required; dry = dry of optimum moisture content; wet = wet of optimum
Note: The percentage of QC data outside of the specification control limits was calculated according to contractor Proctor results, and the percentage of ISU data outside of the specification control limits was calculated according to ISU Proctor results.

For cohesive materials, 1% to 45% of the QC moisture measurements were outside of the specification control limits (1% to 11% dry of the lower control limit, 1% to 36% wet of the upper control limit), while 2% to 75% of the QC density measurements were less than the 95% RC limit. Iowa DOT QA data for the Scott County and Pottawattamie County projects were available (for limited testing dates) and are summarized in Table 34.

The data show that 63% of the moisture measurements (50% dry of the lower control limit and 13% wet of the upper control limit) were outside of the specification control limits in the Pottawattamie County project. In the Scott County project, 69% of the moisture measurements

(4% dry of the lower control limit and 65% wet of the upper control limit) were outside of the specification control limits. The ISU testing results at one project site showed all test measurements met the moisture and density specification limits. At the remaining project sites, 12% to 62% of the ISU moisture measurements were outside of the specification control limits (2% to 16% dry of the lower control limit and 10% to 62% wet of the upper control limit), and 4% to 40% of the ISU density measurements were less than the 95% RC limit.

For cohesionless materials, the contractor QC results on one site (Woodbury I-29) show that 2% of the moisture measurements were outside of the control limits. Iowa DOT QA data at the same site show that 20% of the moisture measurements (11% dry of the lower control limit and 9% wet of the upper control limit) were outside of the specification control limits. ISU testing at the same site show that 66% of the moisture content measurements were outside of the specification control limits (2% dry, 64% wet).

Two other project sites with cohesionless materials (Linn-77 and Linn-79) show 85 to 100% of the moisture measurements outside of the control limits, of which a majority of the measurements (81% to 100%) were dry of the lower control limit. The Linn-77 project showed that all density measurements were > 95% RC, while Linn-79 project showed 14% of density measurements were < 95% RC.

One-dimensional Consolidation

According to the lab test results, it was observe that the compression indices and swelling indices are influenced by compaction energy, moisture content, and dry unit weight. The compression and swelling indices were changed due to the change of moisture content and compaction energy (Figure 180). At the optimum moisture content, the compression index is lowest. At dry side of optimum moisture and wet side of optimum moisture, the compression index is increased. And it is also observed that compression index is decreased as the compaction energy is increased. The compression index is the slope of compression part of the e-logσ curve as higher compaction energy was applied, higher dry unit weight was achieved. It is concluded that the specimen with higher dry unit weight is more difficult to consolidate than the specimen with low dry unit weight. So the slope of the compression part of the e-logσ curve is lower when the specimen has higher dry unit weight. For the swelling indices, a relatively similar trend was observed.

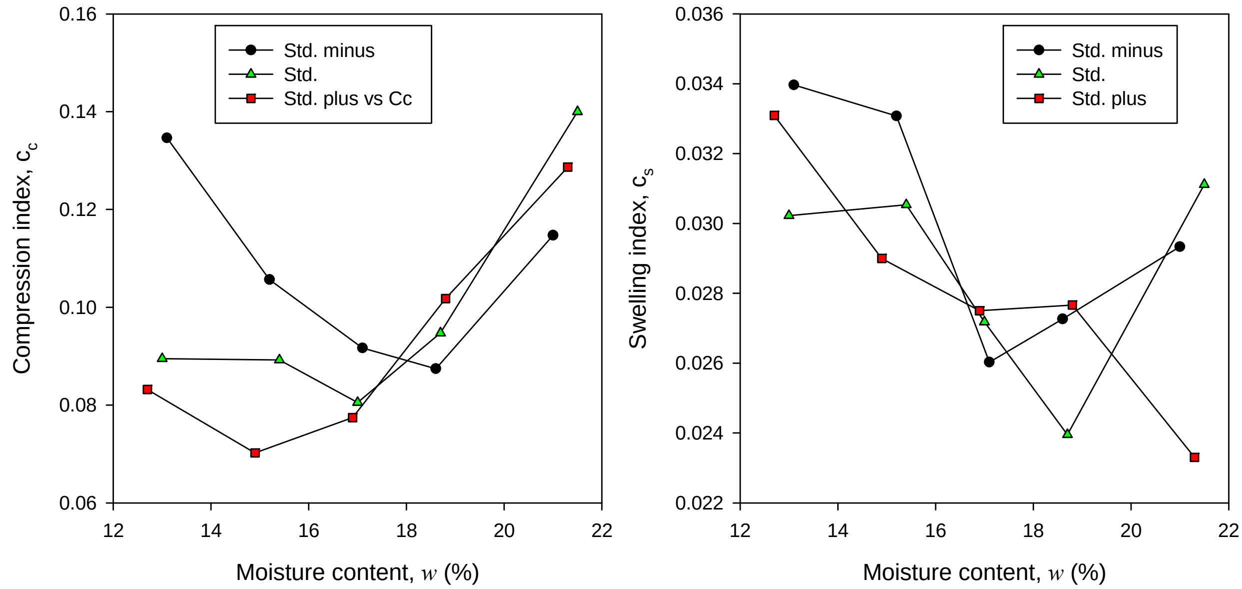


Figure 180. Compression and swelling indices were influenced by moisture content and compaction energy

To quantify the effect of soil index properties and in situ measurements on consolidation properties, the regressions were conducted (Figure 181). The compression index increased as the moisture content increased, especially after the moisture content reached about 20%. This finding is contradictory to the previous finding indicated in Figure 180. The data was mixed without distinguishing the measurements with different compaction energy. This is a possible reason why the shape of moisture-c_c curve is not a reversed Proctor curve. It is obvious to find c_c decreased as the dry unit weight increased. And c_s decreased as moisture content increased. The relation expressions were presented in the following figure. Only the relation expressions with relatively high coefficient of determination (R^2>0.4) were presented in the figure.

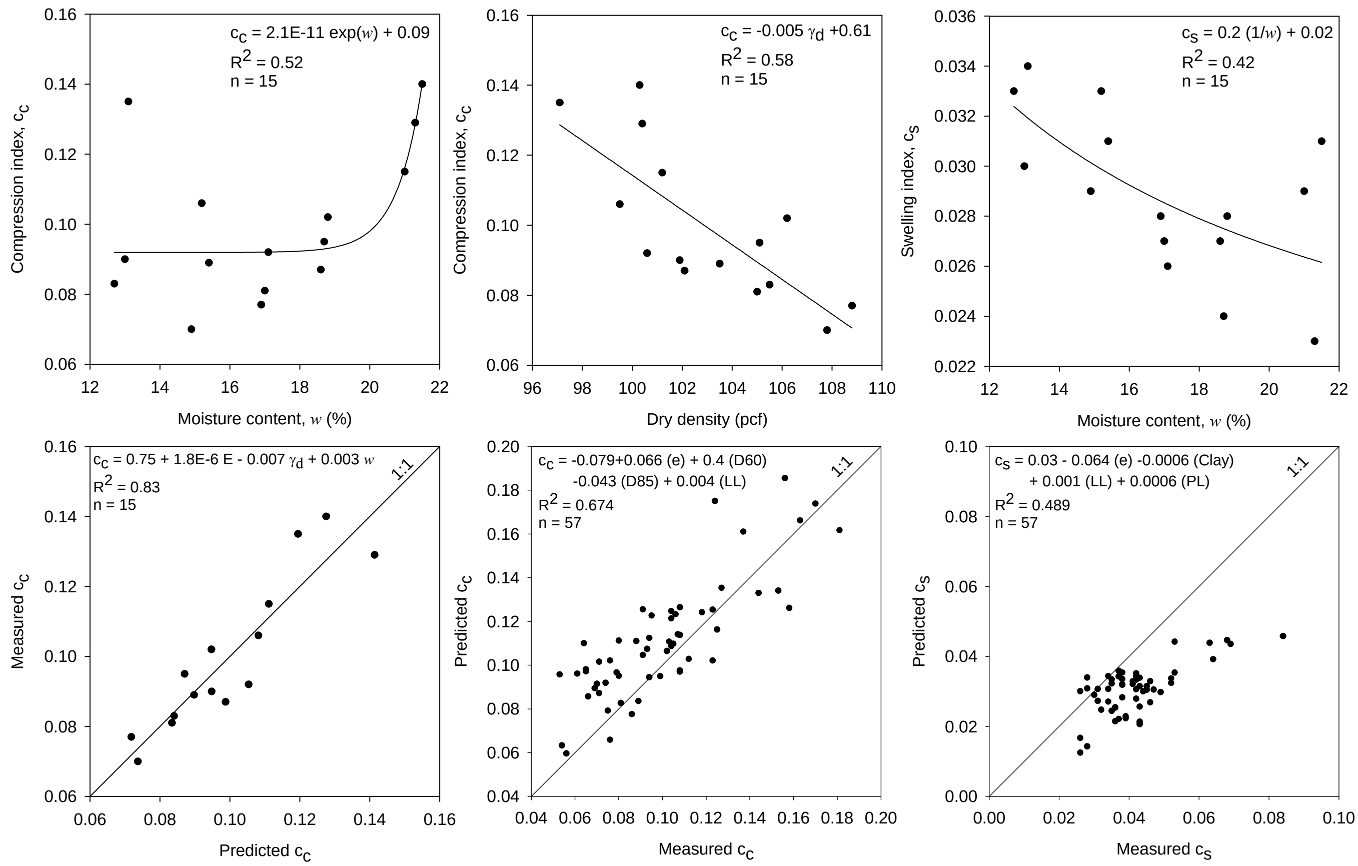


Figure 181. Correlations between Cc, Cs and soil index properties and in situ measurements

Figure 182 presents the linear relationship between moisture content, dry unit weight and compression index with R^2=0.72 for Iowa loess. The compression index was changed as the moisture content and dry unit weight were changed. The effect of dry unit weight on compression index is higher than the effect of moisture content.

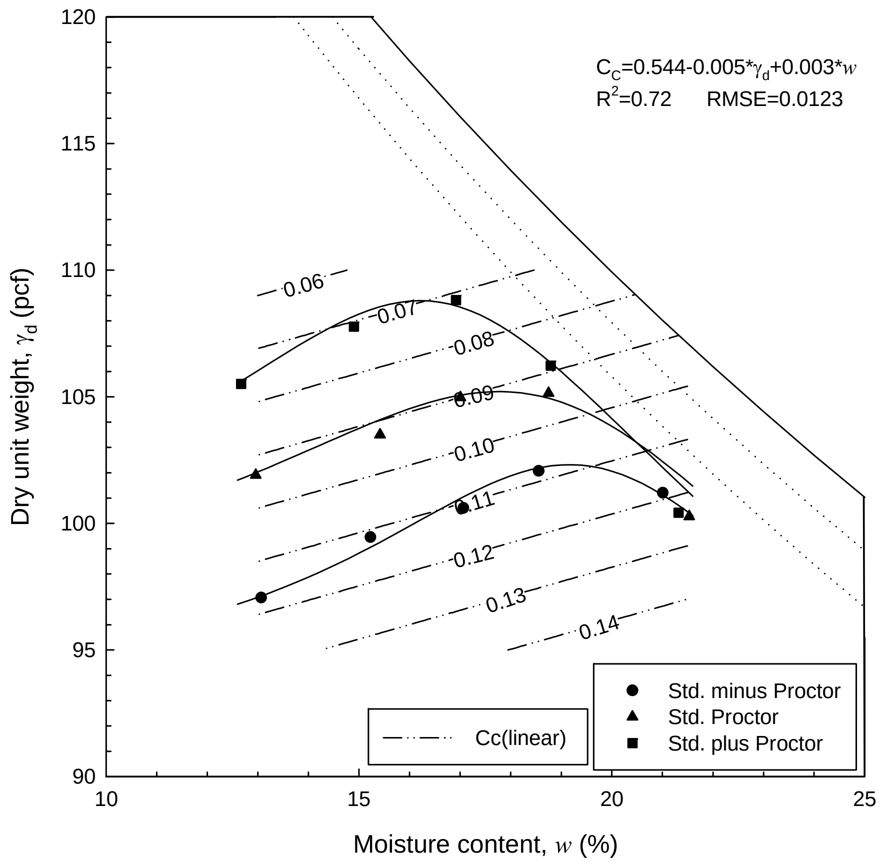


Figure 182. Statistical relationship between moisture content, dry unit weight and compression index for Iowa loess

Figure 183 shows the linear relationship between moisture content, dry unit weight and swelling index with R^2=0.43 for Iowa loess. The swelling index was changed as the moisture content and dry unit weight were changed. Compare to the compression index, the effect of moisture content o swelling index is higher than the effect of dry unit weight.

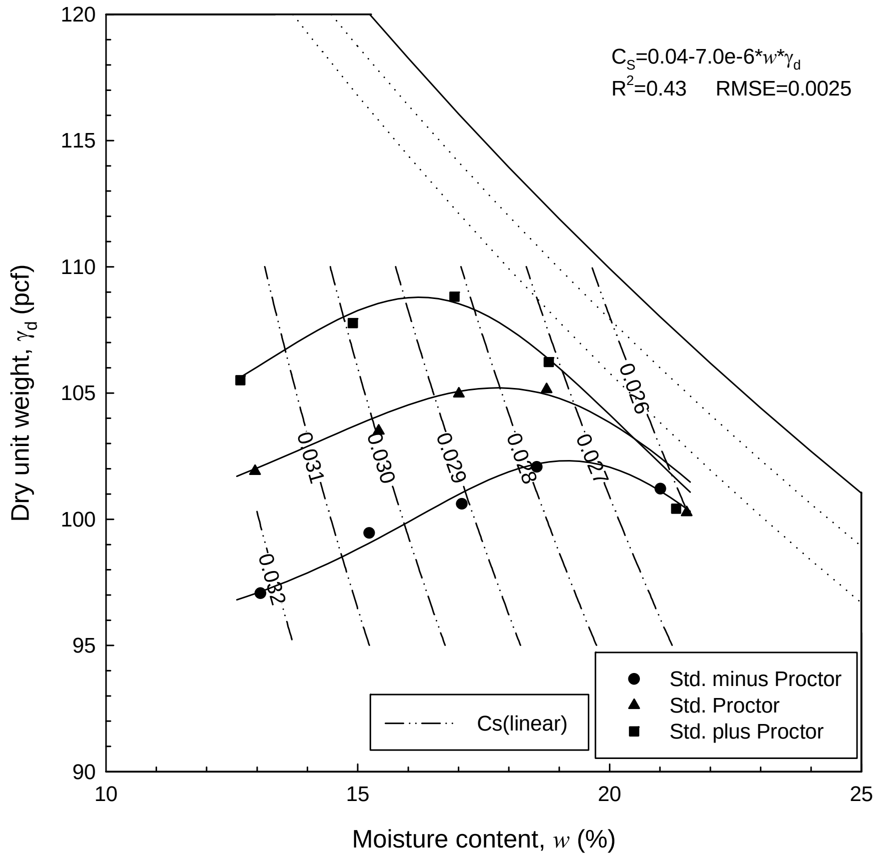


Figure 183. Statistical relationship between moisture content, dry unit weight and swelling index for Iowa loess

For clay, the linear relationship between moisture content, dry unit weight and compression index with R^2=0.47 was presented in Figure 184. Due to the low coefficient of determination, the relationship between moisture content, dry unit weight and swelling index was not presented here. The effect of moisture content on compression index was higher than the effect of dry unit weight.

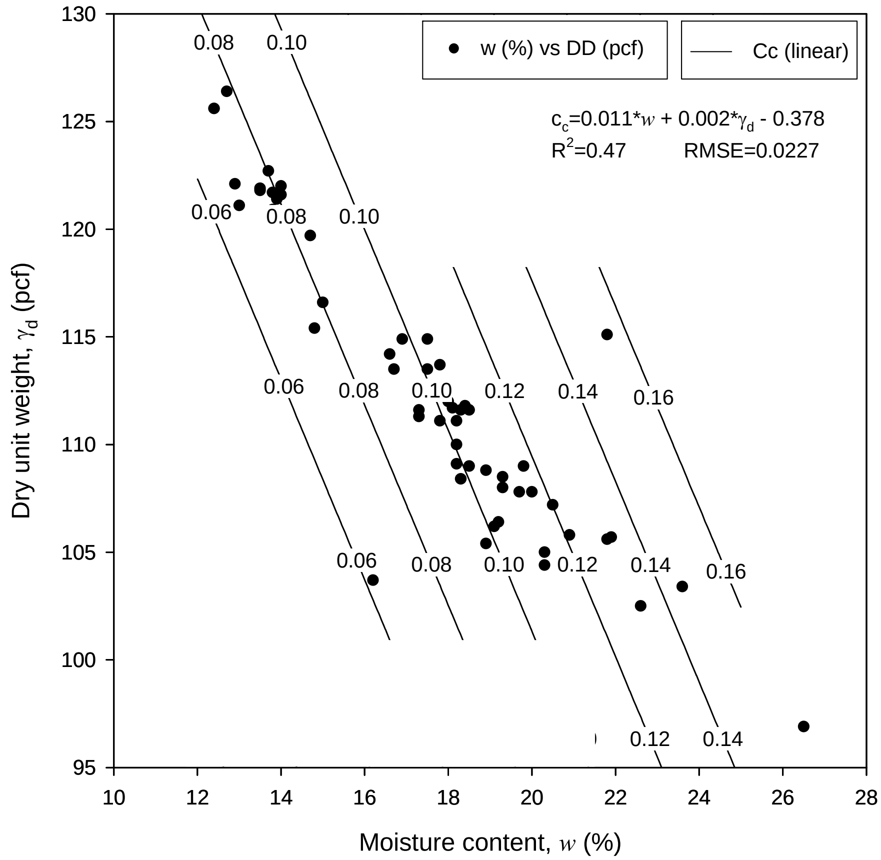


Figure 184. Statistical relationship between moisture content, dry unit weight and compression index for clay

Currently, the embankment construction specification in Iowa requires a desired moisture content range and dry unit weight range. However, according to the results above, it is obvious that compression and swelling index were influenced by moisture content and dry unit weight easily. The change of compression and swelling index will be resulted in differential settlement, which is harmful for the pavement long-term performance. So it is not adequate that specification only requires moisture content and dry unit weight in terms of performance.

Consolidation Finite Element Analysis

SIGMA/W of Geo-Slope was used to conduct the numerical analysis in this study. To determine the displacement of each lift during construction, a function called staged construction in SIGMA/W was applied. The staged construction function allows researchers to define the construction process, material properties, etc. In this study, an embankment model with 20 lifts was simulated. The mesh properties were generated automatically by SIGMA/W (Figure 185). It is assumed that the foundation layer beneath embankment is infinite wide and the bedrock layer was below the foundation layer. Thus, the boundary condition at bottom of the foundation layer is fixed-x and fixed-y. The boundary condition of left and right side of the foundation layer is fixed-y only.

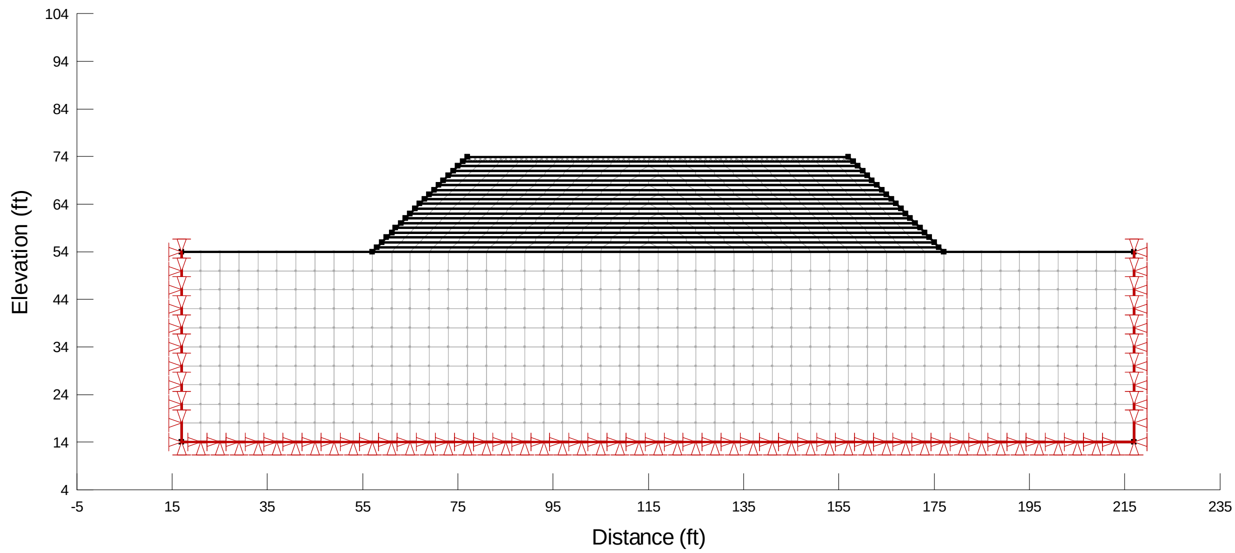


Figure 185. Mesh properties of embankment model at the original state

The hyperbolic constitutive model described by Duncan et al. (1980) was applied to the embankment fill materials. The bulk modulus is assumed to be constant during loading while the elastic modulus varies according to a hyperbolic relationship (Duncan and Chang 1970).

SIGMA/W has an initial modulus which is implemented as an estimation algorithm. The earth pressure coefficient needs to be input to calculate the initial confining stress. The major principle stress is assumed equal to the vertical stress.

To reduce the input requirements of the hyperbolic model while retaining the non-linearity of the volume response, the initial modulus is calculated as bulk modulus (B) multiply 3(1-2(Poisson's ratio, μ). The purpose of this assumption is to retain the confining stress-dependency of the bulk modulus.

Table 35 summaries the soil material properties of foundation layer and embankment layer.

Table 35. Soil properties of foundation and embankment layers

	Unit weight (pcf)	Poisson's ratio	Material model	Cc	Cs
Embankment fill	126.3	0.4	hyperbolic	0.137	0.053
Foundation fill	111.2	0.4	hyperbolic	0.170	0.035

Figure 186 and Figure 187 show the deformation properties of the embankment. The middle part of the embankment was consolidated heavier than the two sides of the embankment. And the direction of consolidation was vertical in the middle part, and gradually changed to be horizontal at the side of the embankment.

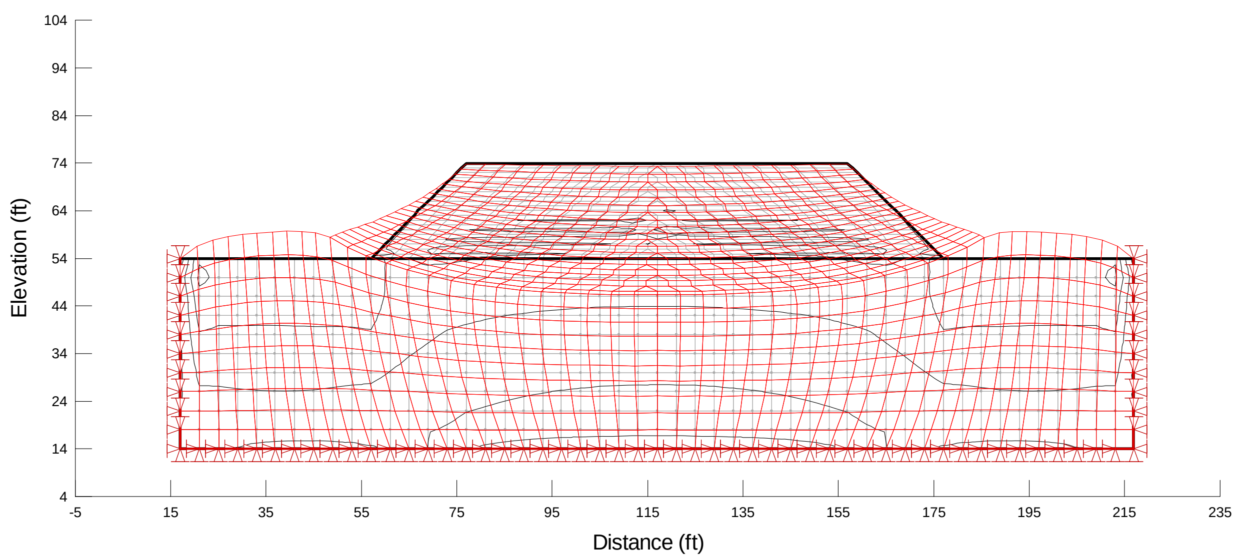


Figure 186. Mesh properties after the final lift of embankment constructed

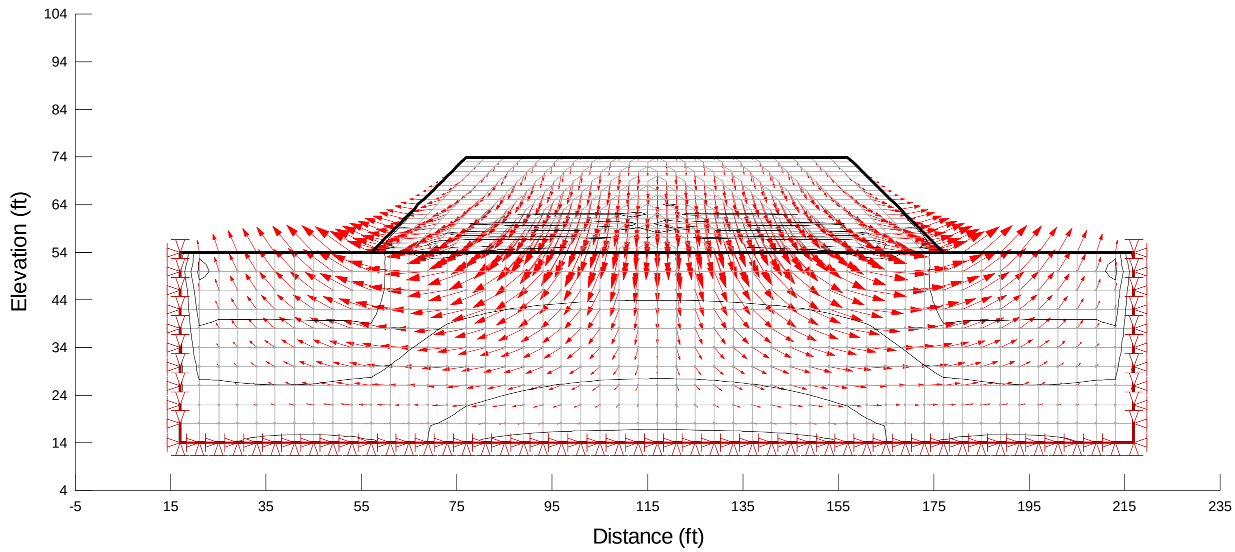


Figure 187. Displacement vectors for the final lift of embankment constructed

Figure 188 shows the vertical settlement profile of the centerline of the embankment. It is easy to observe that the settlement was increased as the depth increased, and the first lift had the highest settlement of 0.24 ft, and the settlement rate was also increased. Because the overburden pressure above the first lift was higher and higher along with the embankment construction, and then was achieved to a highest value than the other 19 lifts.

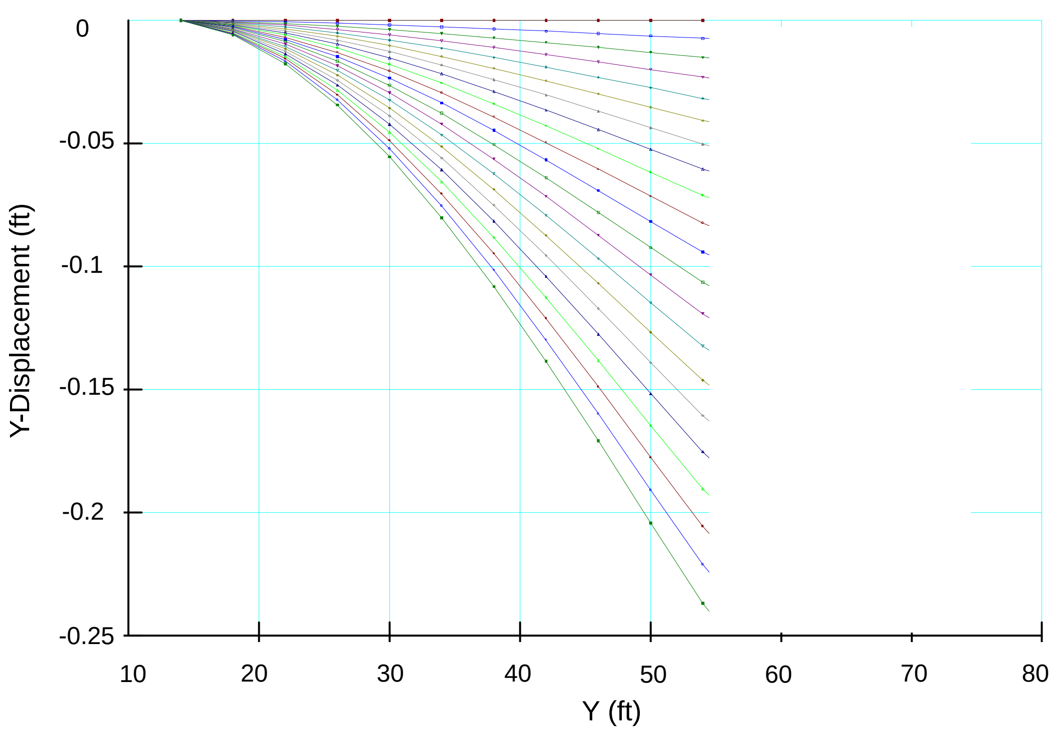


Figure 188. Vertical settlement profile of the centerline of the embankment

Figure 189 presents the cross sectional view of embankment settlement. Similar to the previous discussion, the middle part of the embankment had higher consolidation. And the settlement profile is relatively parabolic. At the two sides of the embankment, the settlement was increased upward. Because the sides of the embankment were at the boundary location and had no constraint.

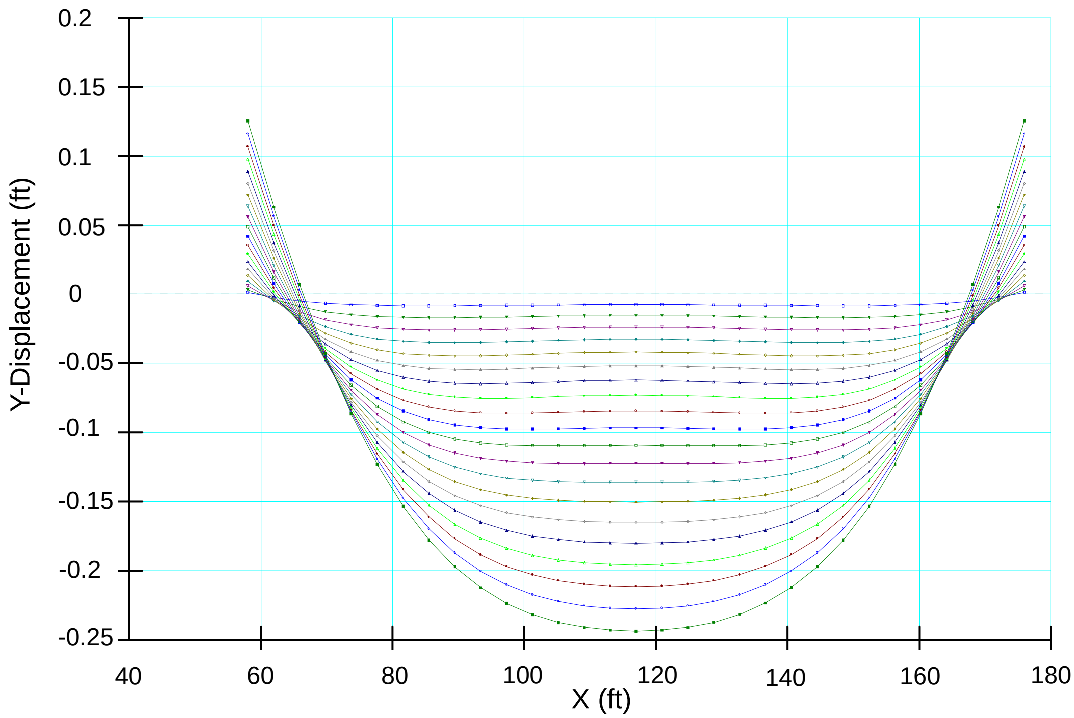


Figure 189. Settlement of cross sectional view of embankment

According to the results above, due to the middle part of the embankment consolidated faster and greater than the sides of the embankment, it is important and worthy to control the construction process to eliminate the differential settlement.

Statistical Analysis of Field Data

In this section, the results obtained from this project are compared with the results obtained from the previous projects to assess whether there was any statistically significant improvement in the implementation of the current earthwork QC/QA specifications.

Table 36 provides a summary of the percentage of ISU test points outside of the specification control limits for the Δw and RC measurements from each of the previous project phases in comparison with the measurements from the current project (IHRB TR-677).

Table 36. Summary of the percentage of test points outside of the specification control limits

Project	Moisture difference, Δ*w* (%)	Relative compaction, RC (%)
Phase I	71	36
Phase II	84	31
Phase III	42	24
Phase IV	75	26
TR-677 (This project)	42	16

To visualize the data spread from each of the previous project phases and the current project, box plots are presented in Figure 190 and Figure 191 for Δ*w* and RC, respectively.

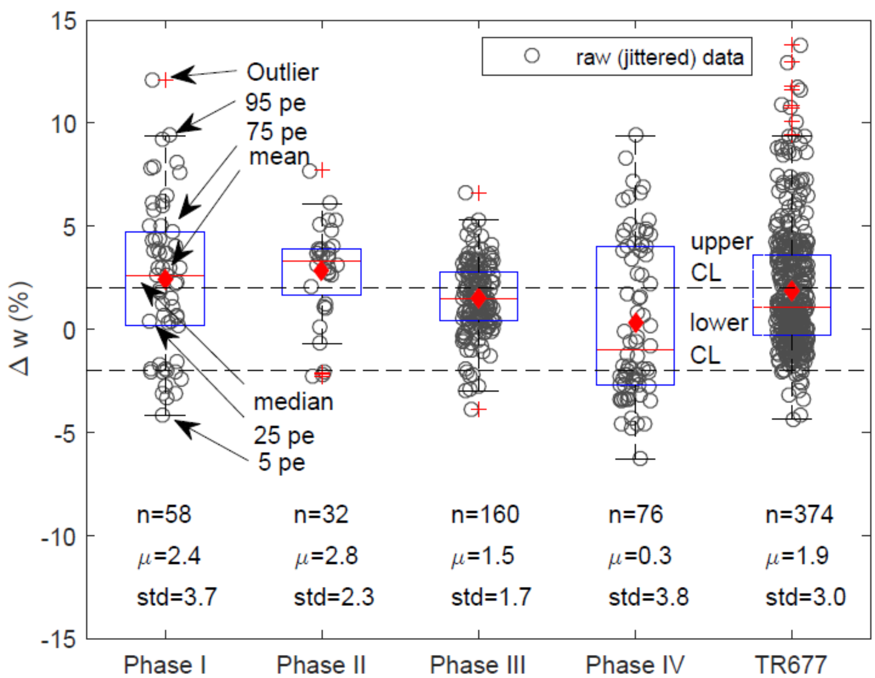


Figure 190. Boxplot of moisture difference for previous and current projects

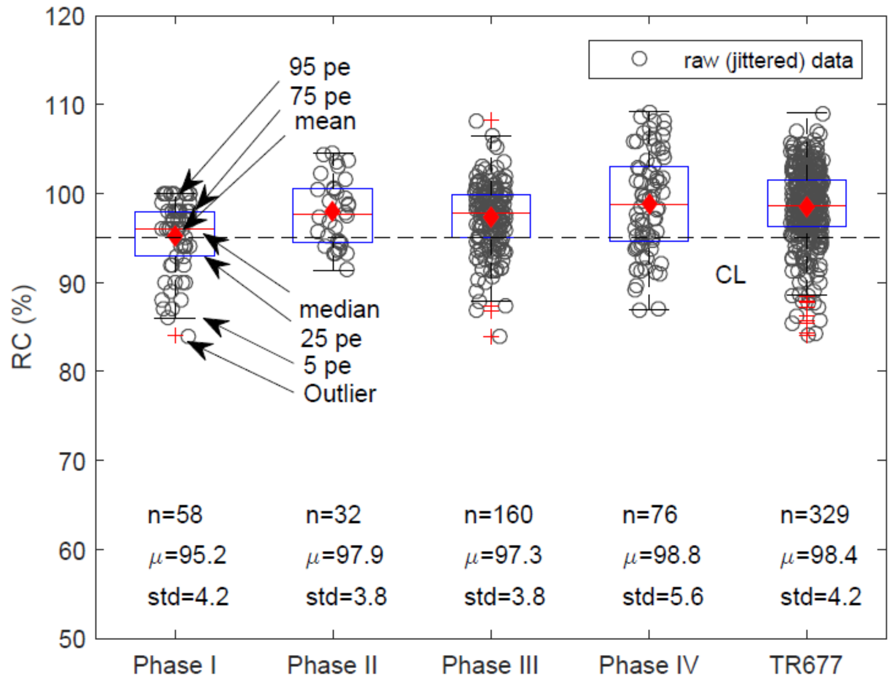


Figure 191. Boxplot of relative compaction for previous and current projects

The box plots show the raw data; the mean and median values; and the 5th, 25th, 75th, and 95th percentiles. The mean (μ) and standard deviation (σ) values for the two measurements are summarized in Table 37.

Table 37. Summary of the mean and standard deviation values for each project

Statistic	Phase I	Phase II	Phase III	Phase IV	IHRB TR-677
n	58	32	160	76	374 (Δw), 329 (RC)
μ0,1 (Δw)	2.4	2.8	1.5	0.3	1.9
μ0,1 (RC)	95.2	97.9	97.3	98.8	98.4
σ (Δw)	3.7	2.3	1.7	3.8	3.0
σ (RC)	4.2	3.8	3.8	5.6	4.2

Table 38 provides the results of *t*-test analyses, showing *t*- and *p*-values in a matrix comparing the Δ*w* measurements for each of the previous projects and the current project.

Table 38. Summary of *t*- and *p*-values from *t*-test results comparing Δ*w* measurements obtained from Phases I through IV and IHRB TR-677

Project	Phase I	Phase II	Phase III	Phase IV	TR-677
Phase I	—	0.587 (0.279)	**-1.873 (0.033)**	**-3.195 (0.001)**	-1.127 (0.132)
Phase II	-0.587 (0.279)	—	**-3.042 (0.002)**	**-4.105 (<0.001)**	**-2.140 (0.019)**
Phase III	**1.873 (0.033)**	**3.042 (0.002)**	—	**-2.494 (0.007)**	**1.654 (0.049)**
Phase IV	**3.195 (0.001)**	**4.105 (<0.001)**	**2.494 (0.007)**	—	**3.212 (0.001)**
TR677	1.127 (0.132)	**2.140 (0.019)**	**-1.654 (0.049)**	**-3.212 (0.001)**	—

The values below the black shaded boxes compare the Δw of the column - the Δw of the row, and the values above the gray shaded boxes compare the Δw of the row - the Δw of the column.
Values in bold are statistically significant at the 95% confidence level (≤ 0.05).

Table 39 provides the results of logistic regressions, showing the odds ratios and *p*-values in a matrix comparing the percentage of data within the moisture control limits for Δ*w* for each of the previous projects and the current project.

Table 39. Summary of odds ratio and *p*-values from logistic regressions comparing the percentage of data within the moisture control limits from Phases I through IV and IHRB TR-677

Project	Phase I	Phase II	Phase III	Phase IV	TR-677
Phase I	—	0.447 (0.155)	**3.344 (<0.001)**	0.804 (0.577)	**3.086 (<0.001)**
Phase II	2.238 (0.155)	—	**7.519 (<0.001)**	1.799 (0.289)	**6.897 (<0.001)**
Phase III	**0.299 (<0.001)**	**0.133 (<0.001)**	—	**0.240 (<0.001)**	0.923 (0.673)
Phase IV	1.244 (0.577)	0.556 (0.289)	**4.164 (<0.001)**	—	**3.846 (<0.001)**
TR677	**0.324 (<0.001)**	**0.145 (<0.001)**	1.084 (0.673)	**0.260 (<0.001)**	—

The values below the black shaded boxes compare the % of data within the limits for the column ÷ the % of data within the limits for the row, and the values above the gray shaded boxes compare the % of data within the limits for the row ÷ the % of data within the limits for the column.
Values in bold are statistically significant at the 95% confidence level (≤ 0.05).

The results indicate that there are statistically significant differences between the results obtained from previous phases and the current project. The odds ratios indicate that the data obtained from the IHRB TR-677 project had a comparatively higher percentage of data within the control limits compared to all previous project phases, which suggests improvement.

Similarly to the results of the *t*-test and logistic regression analyses for Δw, Table 40 provides the results of *t*-test analyses showing the *t*- and *p*-values for RC, and Table 41 provides the results of logistic regressions showing the odds ratios and *p*-values to compare the percentage of data within the limits for RC.

Table 40. Summary of *t*- and *p*-values from *t*-test results comparing RC measurements obtained from Phases I through IV and IHRB TR-677

Project	Phase I	Phase II	Phase III	Phase IV	TR-677
Phase I	—	**3.155 (0.001)**	**3.322 (0.001)**	**4.276 (<0.001)**	**5.398 (<0.001)**
Phase II	**-3.155 (0.001)**	—	-0.901 (0.186)	0.947 (0.173)	0.761 (0.226)
Phase III	**-3.322 (0.001)**	0.901 (0.186)	—	**2.173 (0.016)**	**3.034 (0.001)**
Phase IV	**-4.276 (<0.001)**	-0.947 (0.173)	**-2.173 (0.016)**	—	-0.476 (0.318)
TR677	**-5.398 (<0.001)**	-0.761 (0.226)	**-3.034 (0.001)**	0.476 (0.318)	—

The values below the black shaded boxes compare the RC of the column - the RC of the row, and the values above the gray shaded boxes compare the RC of the row - the RC of the column.
Values in bold are statistically significant at the 95% confidence level (≤ 0.05).

Table 41. Summary of odds ratio and *p*-values from logistic regression results comparing the percentage of data above the density control limit (95% RC) from Phases I through IV and IHRB TR-677

Project	Phase I	Phase II	Phase III	Phase IV	TR677
Phase I	—	1.248 (0.636)	1.821 (0.069)	1.590 (0.220)	**3.096 (<0.001)**
Phase II	0.801 (0.636)	—	1.460 (0.373)	1.272 (0.602)	**2.475 (0.027)**
Phase III	0.549 (0.069)	0.685 (0.373)	—	0.872 (0.669)	**1.698 (0.028)**
Phase IV	0.629 (0.220)	0.786 (0.602)	1.147 (0.669)	—	**1.946 (0.027)**
TR677	**0.323 (<0.001)**	**0.404 (0.027)**	**0.589 (0.028)**	**0.514 (0.027)**	—

The values below the black shaded boxes compare the % of data above the limit for the column ÷ the % of data above the limit for the row, and the values above the gray shaded boxes compare the % of data above the limit for the row ÷ the % of data above the limit for the column.
Values in bold are statistically significant at the 95% confidence level (≤ 0.05).

The results indicate that there are statistically significant differences between the results obtained from previous phases and the current project. The odds ratios indicate that the data obtained from the IHRB TR-677 project had a comparatively higher percentage of data within the control limits compared to all previous project phases, which suggests improvement.

Intelligent Compaction

The intelligent compaction field tests were conducted in July and August of 2013. MDP and pass count were obtained by the IC roller. In situ point-MVs ($E_{LWD\text{-}Z3}$, γ_d, w, CBR) were obtained after roller passes at four test locations. The compaction was performed by operating the roller in forward gears in vibrate mode.

A summary of MDP_{40} and in situ point-MV statistics are presented in Table 42. The summarized data shows that the dry unit weight had a great effect on MDP_{40} as the LWD modulus and CBR were similar. The dry unit weight of material obtained in July is higher than the dry unit weight obtained in August. However, the MDP_{40} obtained in July is lower than the data obtained in August. It is further confirmed that strength, dry unit weight, sometimes is not adequate to reflect the compaction performance.

Table 42. Summary statistics of in situ test results

Measurement value	n	μ	σ	COV (%)
Data collected in July				
Measurement value	**n**	**μ**	**σ**	**COV (%)**
MDP_{40} (at in situ test point location)	28	81.9	11.7	14.3
Dry unit weight, γ_d (pcf)	28	112.1	5	4.4
Relative compaction, RC (%)	28	100.4	4.5	4.4
Moisture content, *w* (%)	28	16.5	2.7	16.6
Modulus, E_{LWD-Z3} (MPa)	28	11.6	6.2	53.3
CBR_{300} (%)	28	5.3	5.2	97.8
Data collected in August				
Measurement value	**n**	**μ**	**σ**	**COV (%)**
MDP_{40} (at in situ test point location)	21	89.8	13.3	14.8
Dry unit weight, γ_d (pcf)	20	99	4.7	4.7
Moisture content, *w* (%)	21	17.7	3.5	20
Modulus, E_{LWD-Z3} (MPa)	21	11.5	5.6	48.2
CBR_{300} (%)	21	3.7	3.3	89.8

Regression analysis between MDP_{40} and in situ point-MVs was conducted in this study (Figure 192, Figure 193, and Figure 194). For data obtained in July, the correlations between LWD modulus and MDP_{40} yielded a relatively strong linear relationship with R^2 =0.63-0.69. However, the correlations between MDP_{40} and other in situ point measurements yielded relatively weak relationship with R^2<0.35 (Figure 192). Multivariate regression analysis was also performed, but it is difficult to find a correlations between MDP_{40} and in situ point measurements. The tested location in August consisted of three test beds. There is no correlation between combined MDP_{40} and in situ point measurements. Thus, the data was analyzed test bed by test bed separately. Similarly, the correlations between MDP_{40} and LWD modulus yielded relatively strong non-linear relationships with R^2 = 0.41-0.65. It is also noticeable that parabolic relationships between MDP_{40} and moisture content were observed in TB1 and TB3 with R^2 = 0.37 – 0.57. However, the two correlations were reversed. In TB1, the MDP_{40} was lowest at the optimum moisture content. In TB3, the MDP_{40} was highest at the optimum moisture content. The dry unit weight is achieved to be highest at the optimum moisture content. It is reinforced that unit weight is not adequate to reflect real compaction performance.

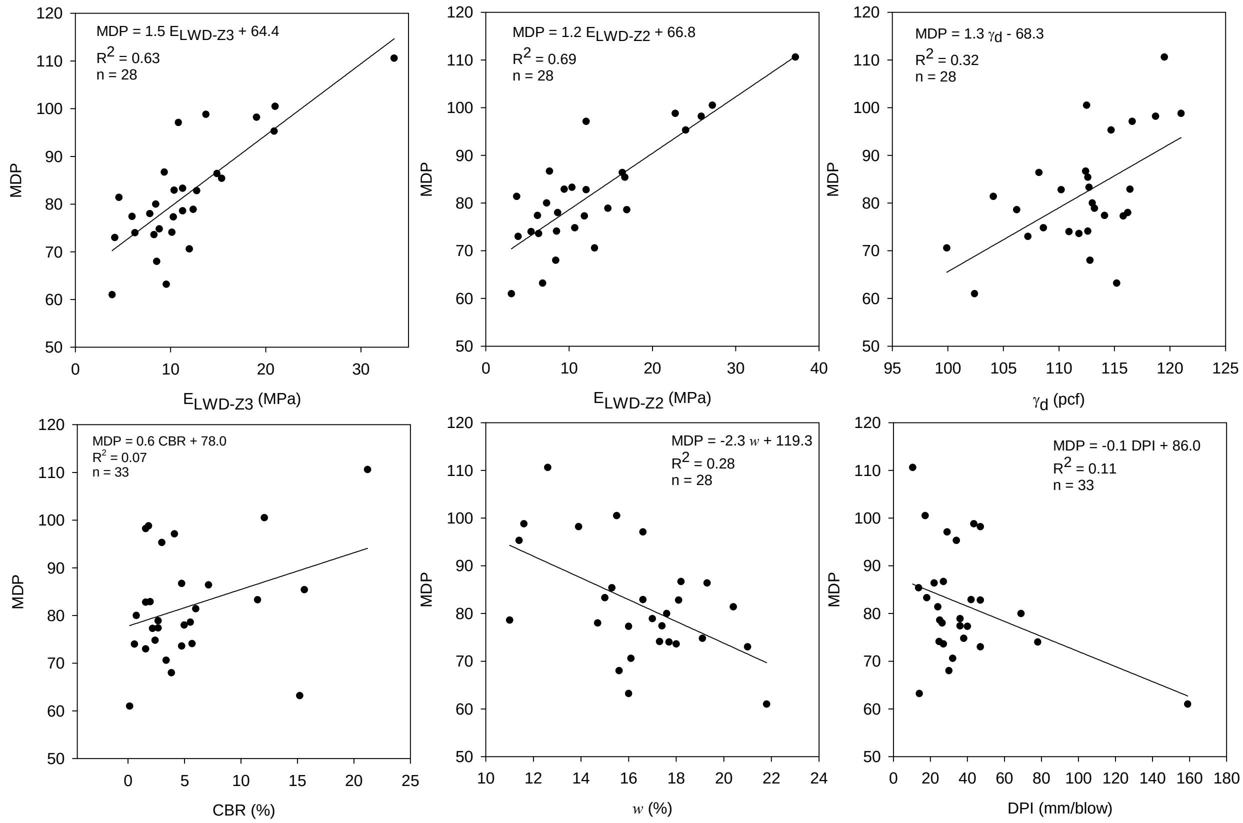


Figure 192. Correlations between MDP_{40} and in situ point measurements – July

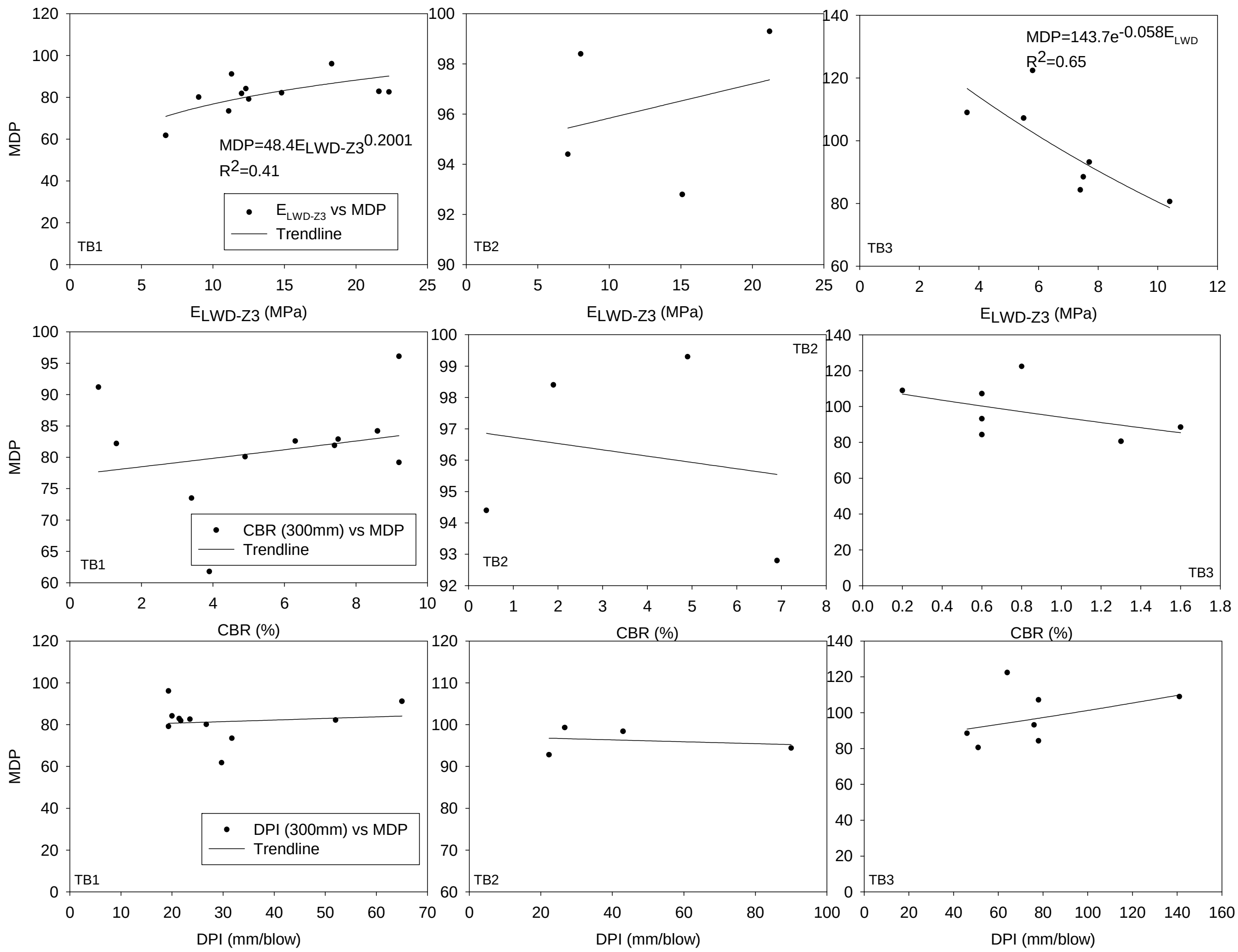


Figure 193. Correlations between MDP_{40} and in situ point measurements – August

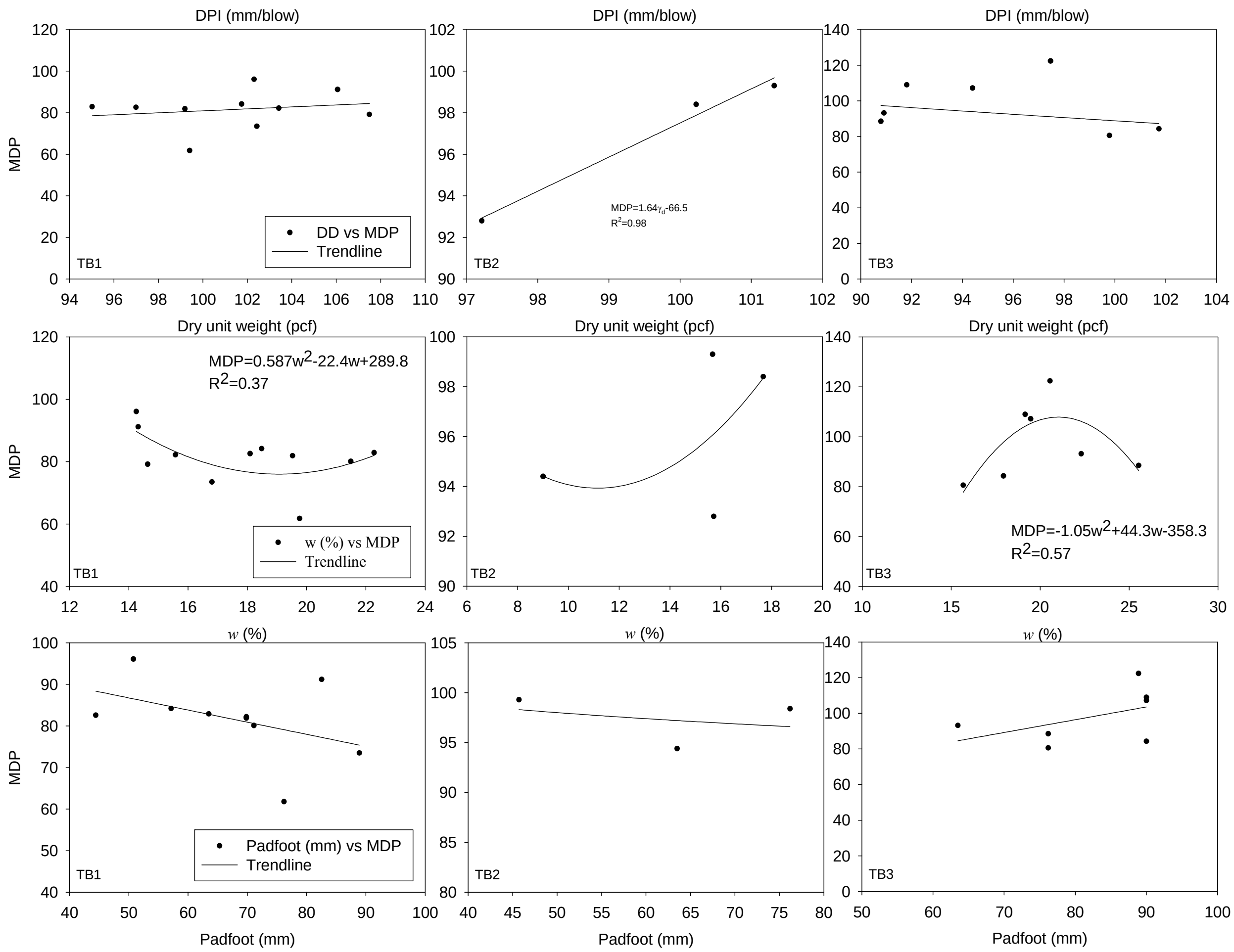


Figure 194. Correlations between MDP$_{40}$ and in situ point measurements - August (continued)

Figure 195 and Figure 196 present the GIS color mapping figure with MDP$_{40}$ and pass count for July and August data, respectively. The GIS color map with MDP$_{40}$ presents MDP measurement from the last roller pass. These figures clearly indicate the soft and stiff part of the testing location and the number of passes performed on the testing location. The west part of July test bed was passed once, and the MDP$_{40}$ was 90 to 110.

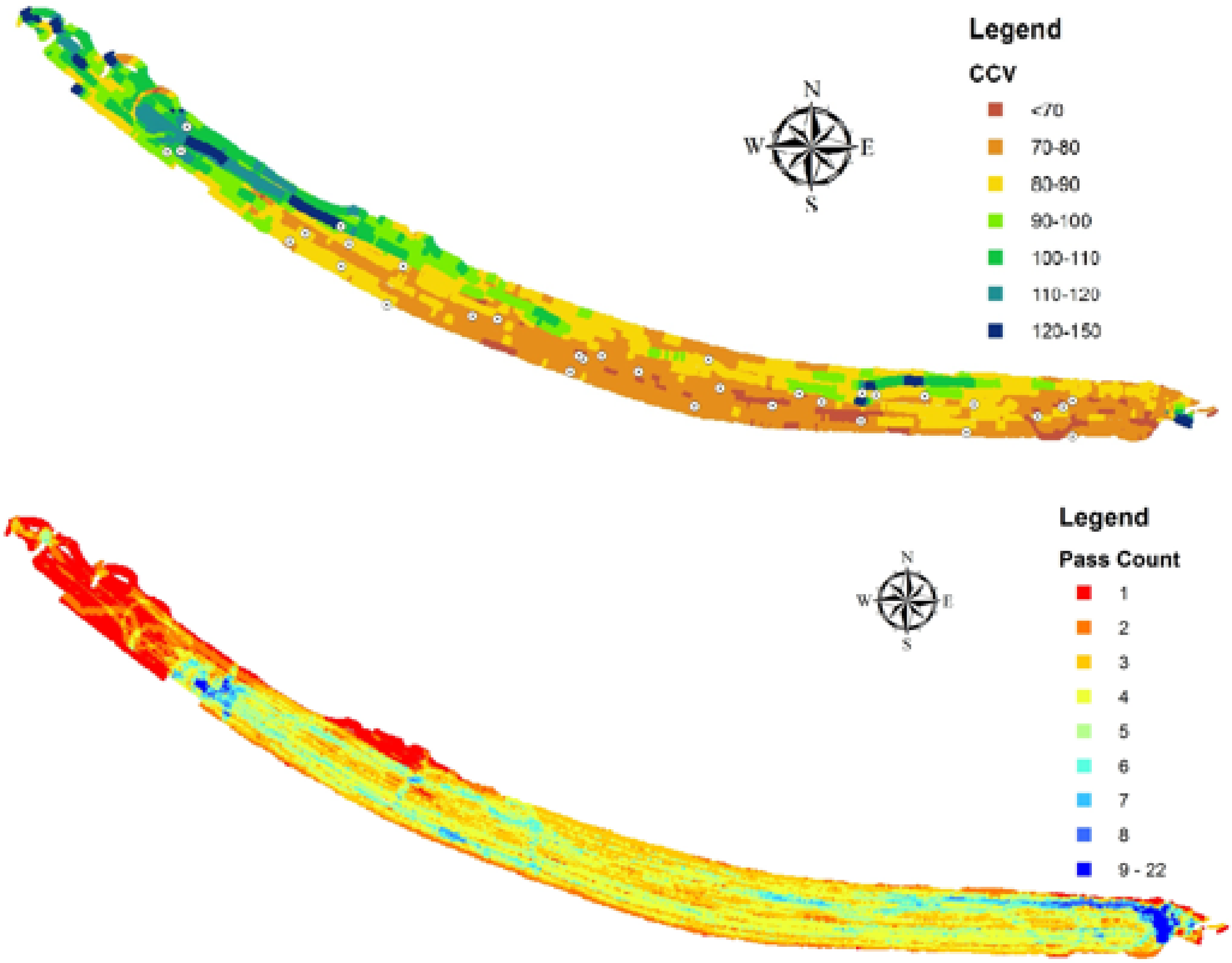


Figure 195. Intelligent compaction MDP measurements and pass count values for July data

For the data obtained in August, it is obvious that the MDP_{40} was increased as more numbers of roller passes. TB1 was only passed once, and the MDP_{40} was below 70. In TB2, the roller pass count was increased to 3, the MDP_{40} was also increased to 90-110. In TB3, the increased MDP_{40} with more passes was also observed.

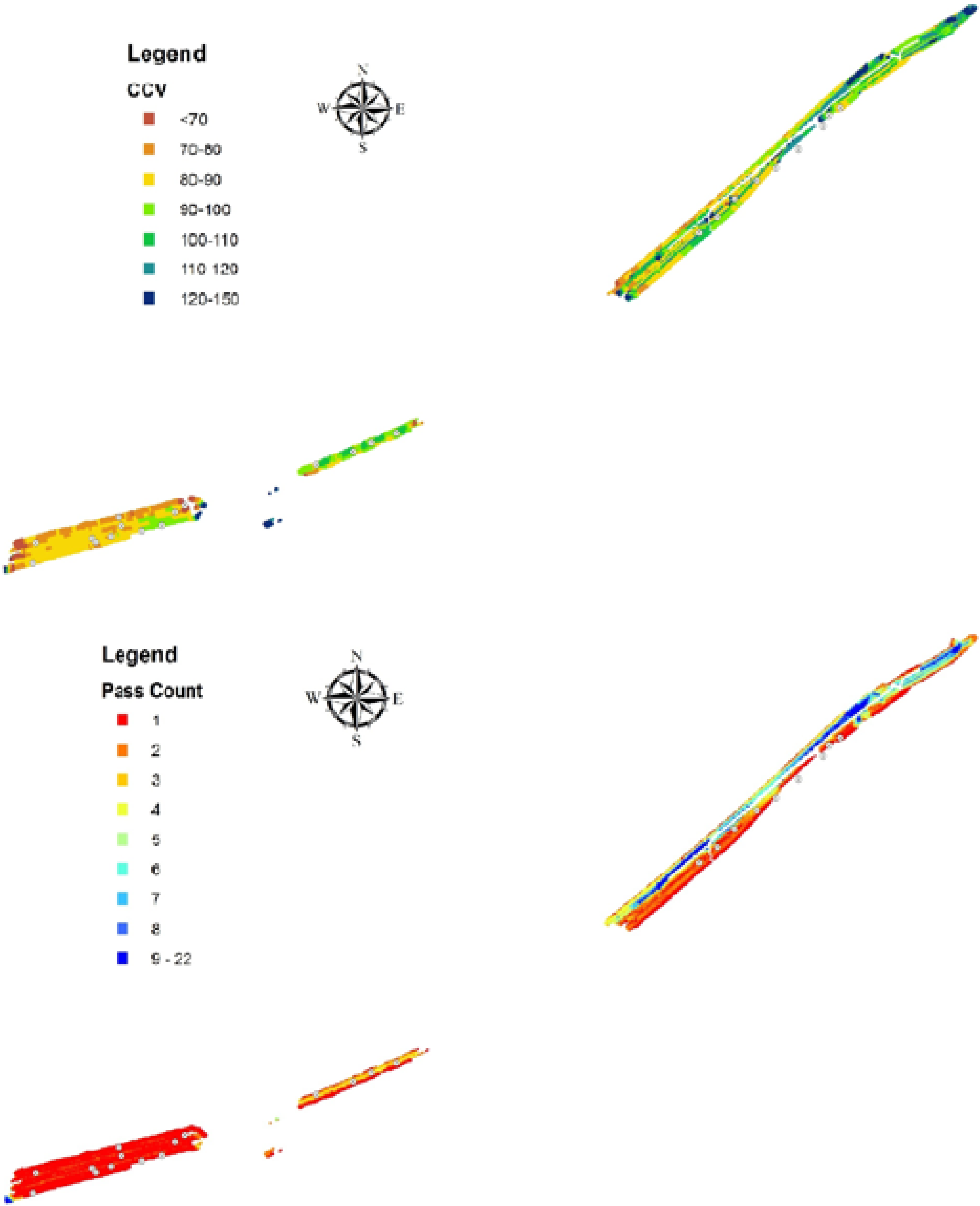


Figure 196. Intelligent compaction MDP measurements and pass count values for August data

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

Summary and Conclusions

The current study set out to study the impact of the current specifications in terms of quality compaction and to identify further areas for improvement given recent advancements in compaction measurement systems and in situ testing technologies. Field testing was conducted on nine active construction sites in Iowa with materials consisting of glacial till, western Iowa loess, and alluvium sand. Drive cylinder tests were performed to determine in situ moisture content and dry density; DCP tests were performed to determine CBR profiles with depth. Laboratory tests consisted of Proctor and soil classification testing. Field test results from ISU testing were assessed to determine whether the data were within the moisture control limits (±2% of optimum moisture content) and above the minimum relative compaction control limit (95% of standard Proctor test). The data that were available from contractor QC testing and Iowa DOT QA testing were also assessed in comparison with ISU test results.

Key findings from this study are as follows:

- For cohesive materials, the contractor QC data showed that 1% to 45% of moisture measurements were outside of the specification and 2% to 75% of density measurements were outside of the specification. Iowa DOT QA data at two project sites showed that 63% to 69% of moisture measurements were outside of the specification. ISU testing results showed all test measurements within the moisture and density specification limits at one project site. At the remaining project sites, 12% to 62% of ISU moisture measurements were outside of the specification; and, 4% to 40% of ISU density measurements were outside of the specification.
- For cohesionless materials, the contractor QC results at one site showed that 2% of the moisture measurements were outside of the control limits. Iowa DOT QA data at the same site showed that 20% of the moisture measurements (11% dry of the lower control limit and 9% wet of the upper control limit) were outside of the specification control limits. ISU testing at the same site showed that 66% of the moisture content measurements were outside of the specification control limits (2% dry, 64% wet).
- Two other project sites with cohesionless materials showed 85% to 100% of the moisture measurements outside of the control limits, of which a majority of the measurements

(81% to 100%) were dry of the lower control limit. One of the sites showed that all density measurements were > 95% RC, while the other showed 14% of density measurements were < 95% RC.

- DCP results showed that the compacted fills have relatively low and variable CBR values, about 0.6% to 8.2% for 8 in. depth and 0.5% to 8.6% for 12 in. depth.
- During in situ construction observations at cohesive fill materials projects, discing did not effectively aerate wet fill material.
- During in situ observations, cohesionless fill materials were very wet and seepage even occurred. The CBR values (0.3% to 1.0% at 8 in. depth and 0.3% to 1.7% at 12 in. depth) also indicated weak support conditions.
- Proctor tests conducted by ISU using representative material obtained from each test section where field testing was conducted showed optimum moisture contents and maximum dry densities that are different from what was selected by the Iowa DOT for QC/QA testing. Comparison between the measured and selected values showed a standard error of 2.9 lb/ft^3 for maximum dry density and 2.1% for optimum moisture content. The difference in optimum moisture content was as high as 4% and the difference in maximum dry density was as high as 6.5 lb/ft^3.
- For maximum dry density, AASHTO T 99 allows 4.5 lb/ft^3 variation between two test results from different laboratories, while ASTM D698 allows 2.3 lb/ft^3 to 3.9 lb/ft^3, depending on the soil type. Results indicated that only 1 of 19 test results fell outside the allowable limits per AASHTO T 99, while 7 of 19 fell outside the allowable limits per ASTM D698.
- For optimum moisture content, AASHTO T 99 allows variation of 15% from the mean of the two test results, while ASTM D698 allows a variation of 1.5% to 1.8%, depending on the soil type. Only 3 of 26 test results fell outside the allowable limits per AASHTO T 99, while 7 of 26 fell outside the allowable limits per ASTM D698.
- Statistical analysis indicated statistically significant differences between the Δw and RC results obtained from this project and the previous embankment research projects. The results indicated that data obtained from the current IHRB TR-677 project had a higher percentage of data that were within the control limits for Δw and above the control limit

for RC compared to all previous project phases. This suggests improvement over the previous project results.

Results of a laboratory study focused on cement stabilization of 28 soils obtained from 9 active construction sites in Iowa are presented in this dissertation. The materials consisted of glacial till, western Iowa loess, and alluvium sand. Type I/II portland cement was used for stabilization of these materials. 2 x 2 specimens of stabilized and unstabilized materials were prepared, cured, and tested for UCS with and without vacuum saturation. F_{200}, AASHTO group index (GI), and Atterberg limits were tested before and after stabilization. The results were analyzed using multi-variate statistical analysis to assess influence of the various soil index properties on post-stabilization material properties. Key findings from the test results and analysis are as follows:

- F_{200} of the material decreased with increasing cement content for a majority of the soils. The percent cement content, F_{200} before treatment, and liquid limit were found to be statistically significant in predicting the F_{200} after treatment. The multi-variate model showed an R^2 of about 0.9 and RMSE of about 7% in predicting the F_{200} after treatment.
- With the exception of a few materials, the liquid limit and plasticity index of all materials decreased with increasing cement content. The one untreated soil classified as "unsuitable", classified as "suitable" after stabilized with cement. Some of the untreated soils that were classified as "select", classified as "suitable" after stabilized with cement. The classifications changed because of reduction in plasticity index. All soils classified as "suitable" at 12% cement content because they had no plasticity. The percent cement content and clay content parameters were found to be statistically significant in predicting the plasticity index of materials after stabilization. The multi-variate model showed an R^2 of about 0.5 and RMSE of about 5%.
- The GI values decreased with increasing cement content for a majority of the soils. The percent cement content, F_{200}, liquid limit, and plasticity index parameters were found to be statistically significant in predicting the group index values after treatment. The multi-variate model showed an R^2 of about 0.7 and RMSE of about 3.
- The UCS of specimens increased with increasing cement content, as expected. The average saturated UCS of the unstabilized materials varied between 0 and 57 psi. The average saturated UCS of stabilized materials varied between 44 and 287 psi at 4%

cement content, 108 and 528 psi at t 8% cement content, and 162 and 709 psi at 12% cement content. The draft laboratory testing and evaluation procedure for cement stabilization mix design provided in Appendix E targets a 100 psi saturated unconfined compressive strength. The UCS of the saturated specimens was on average 1.5 times lower than of the unsaturated specimens.

- The percent cement content, sand content, fines content, and liquid limit were found to be statistically significant in predicting unsaturated and vacuum saturated UCS. The models showed an R^2 of about 0.85 and RMSE of about 75 psi for vacuum saturated specimens and 97 psi for unsaturated specimens.

Results of a laboratory study focused on one-dimensional consolidation of 25 soils obtained from 8 active construction sites in Iowa are presented in this dissertation. All specimens were performed loading, unloading, and reloading cycles. Key findings from the test results and analysis are as follows:

- The compression index was influenced by moisture content and compaction energy.
- As the compaction energy was increased, the compression index was decreased.
- The compression index was lowest as the moisture content was optimum. As the moisture content of soil was drier or wetter of the optimum moisture content, the compression index was increased.
- The plot of moisture content versus compression index was relatively reversed to Proctor curve.
- The correlations between moisture content, dry unit weight and compression index was developed with an R^2 of about 0.52 and 0.58, respectively.
- The correlations between moisture content and swelling index was developed with an R^2 of about 0.42.
- Multi-variate regression analysis showed that correlations existed between moisture content, dry unit weight and compression and swelling index of Iowa loess. And dry unit weight had greater effect on compression index than moisture content, moisture content had greater effect on swelling index than dry unit weight.
- For clay, multi-variate regression analysis showed that a correlation existed between moisture content, dry unit weight and compression index. And dry unit weight had greater effect on compression index than moisture content.

The finite element analysis for staged embankment construction was conducted by SIGMA/W. The key findings from the simulation results are as follows:

- The middle part of the embankment had higher settlement than the sides of the embankment.
- The displacement direction of the middle part of the embankment was vertical, and the displacement direction of the sides of the embankment was relatively horizontal.
- The consolidation of lower lift was keep increased as the embankment was constructed upward. The higher stress was applied on the lift, the faster consolidation occurred.
- The settlement profile of the embankment in cross sectional view was similar to a parabolic shape. And the differential settlement was observed.

Intelligent compaction results of a case study obtained from Highway 65 in Altoona of Iowa are presented in the dissertation. Two construction sites were tested in July and August 2013. The intelligent compaction measurement MDP_{40} and the in situ point measurements (moisture content, dry unit weight, CBR, $E_{LWD\text{-}Z3}$) were collected for analysis. Key findings from the test results and analysis are as follows:

- The correlations between MDP_{40} and in situ point stiffness measurements were developed with R^2 of about 0.41 to 0.69.
- There is no significant correlations observed between MDP_{40} and moisture content, dry unit weight, CBR, and padfoot penetration.
- Even though the IC MDP measurements were located as close as the in situ point measurements, there were still some error existed during GIS matching. So it is a possible reason why the correlations between IC MDP measurements and in situ point measurements were not significant.

Recommendations

Based on the field testing and observations documented in this dissertation, although the results show a statistically significant improvement over previous projects, QC/QA results are not consistently meeting the specification. Recommendations are provided herein for improvements to the current specifications in terms of three options, as described below. A one-page summary of the proposed recommendations is provided in Figure 197.

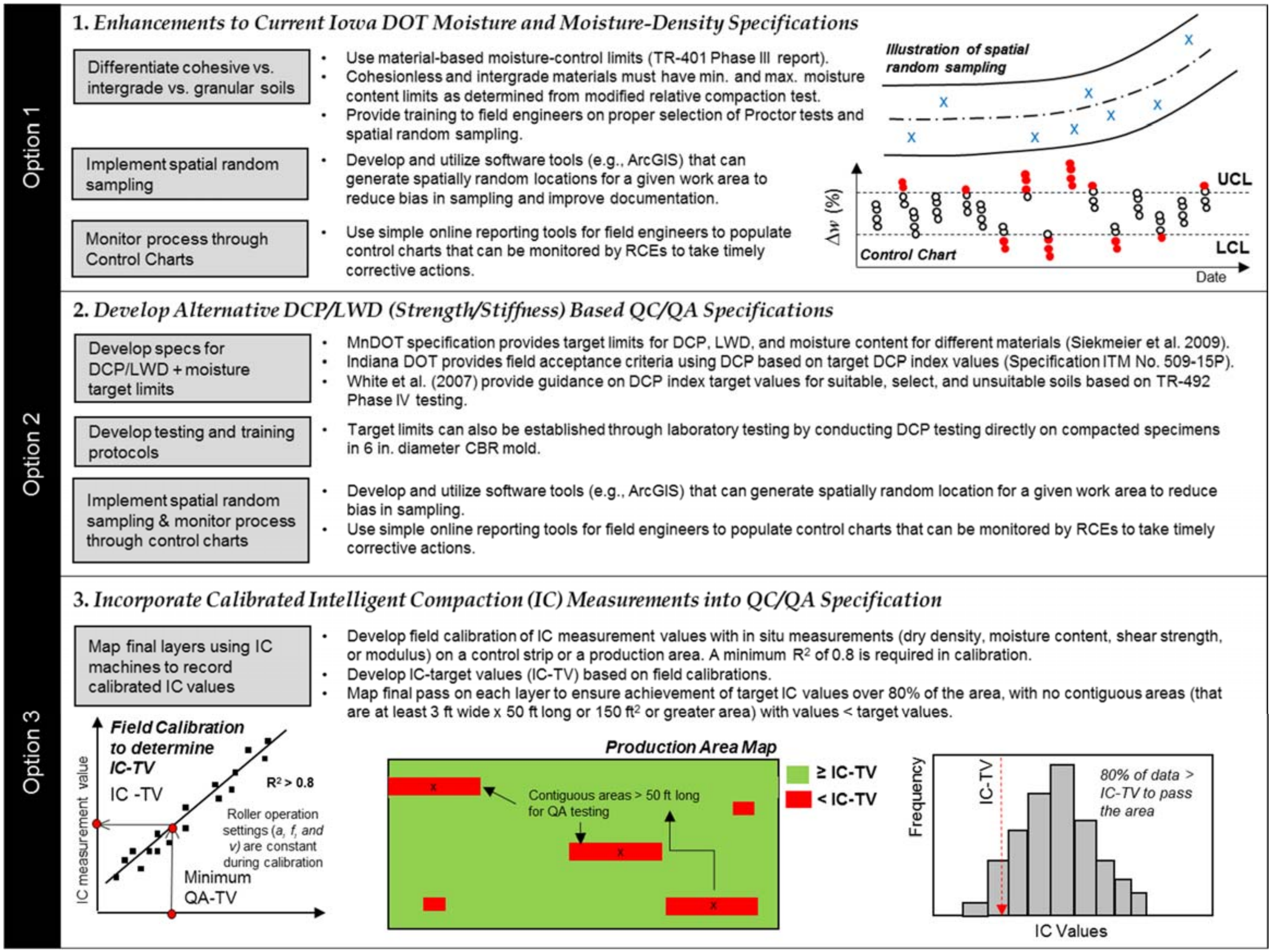


Figure 197. Recommended specification options for future QC/QA

Option 1: Enhance the Current Iowa DOT Moisture and Moisture-Density Specifications

This option has three key aspects that will provide enhancements to the current specifications:

1. The moisture and density control limits should differentiate between cohesive versus intergrade versus cohesionless materials. Material-based moisture control limits should be selected, and guidance regarding this topic is provided in the IHRB TR-640 Phase III project report (White et al. 2002).
2. Although the current specifications call for spatial random sampling, it was not conclusive whether or not a truly random sampling pattern was followed during QC/QA field testing. It is recommended that a simple software tool be developed that can generate spatially random locations for a given work area (starting and ending stations) to reduce bias in sampling and improve documentation.
3. The current process requires field engineers (for both QC and QA) to manually write data hard copy on field data sheets and share data via DocExpress. In many cases, data were not available on DocExpress for at least several months after the testing had been completed. It is recommended that simple digital online reporting tools be developed for field engineers where the data can be efficiently entered and RCEs can monitor the process through control charts. This reporting system will allow the RCEs to take immediate corrective actions when data are falling outside the control limits.

Option 2: Develop Alternative DCP/LWD-based (Strength/Stiffness-based) QC/QA Specifications

DCP and LWD test procedures provide a measure of strength and stiffness, which is a performance-related measurement. Two state DOTs (Minnesota and Indiana) have developed DCP and LWD specifications with target limits for QA. A summary of these specifications is provided in Chapter 2 under the section titled Alternative Specification Options. These specifications provide guidance on the DCP index or blow count target values based on different material types. Based on Phase IV testing, White et al. (2007) also provided DCP index target values for suitable, select, and unsuitable soils that can be utilized.

Using an existing database for target limits can be challenging and sometimes not appropriate for certain materials. Therefore, pilot projects are recommended to evaluate the feasibility of using those values. As an alternative to using existing target values, material- and project-specific target values can be determined via DCP testing on compacted specimens in 6 in. diameter Proctor or CBR molds at different moisture and density conditions. This testing will require additional training for field engineers to properly implement the procedures and develop target values.

Option 3: Incorporate Calibrated Intelligent Compaction (IC) Measurements into QC/QA Specifications

As noted in previous Iowa DOT projects, the use of IC technology represents a paradigm shift in terms of process control and acceptance procedures for embankment construction when compared to the current moisture or moisture-density specifications. Example specifications for implementing IC technologies for embankment and pavement foundation layer construction have been published in the technical literature (e.g., ISSMGE 2005, Mooney et al. 2010, White et al. 2009, FHWA 2014, Scott et al. 2014). These specifications vary in the way IC data are used in the process control (QC) and acceptance (QA) processes. These alternative specifications should be reviewed for possible implementation in Iowa.

A rather straight forward way of using IC measurements is to generate color-coded maps to identify "weak" areas and conduct a stratified random sampling in the "weak" areas for testing. This form of specification is rather straight forward to implement, but it can be expensive in terms of the number of locations to be tested because the IC measurements are not calibrated to soil engineering properties. Examples of such a specification are described in Mooney et al. (2010) and White et al. (2009).

Proper implementation of IC technology requires a specification that has a statistically framed QC/QA approach, wherein the IC measurement values are properly calibrated to the soil engineering properties that are assumed in the design process. When embankment materials are compacted, there is a need to ensure that the resulting soil engineering properties are satisfactory for the intended purposes (e.g., limit the effects of post-construction volume changes on saturation, provide adequate bearing capacity under embankment loads, and/or provide adequate support capacity to the pavement surface layer under traffic loads).

One way to implement this approach is to require the contractor to develop and produce a statistically valid calibration between in situ QA tests (density, moisture, modulus, or strength) and IC measurement values and develop an IC target value based on the calibration. A statistically valid calibration should provide an R^2 value of ≥ 0.80. Production areas can then be mapped to produce straight forward maps that show pass/fail areas (green/red or black/white), which can then be used to identify areas for QA testing using a stratified sampling approach. The final pass on each layer should be mapped to ensure achievement of target IC values over 80% of the area, with no contiguous areas (that are at least 3 ft wide x 50 ft long or 150 ft^2 or greater in area) that have values lower than the IC target values.

Other Considerations

The new process control procedures and specifications should be developed with the objective of achieving the desirable design engineering properties, including adequate strength and stability, low permeability, low shrink-swell behavior, and low collapsibility. In lieu of relying on compaction density and moisture content control, typical embankment material treatment/stabilization options to improve performance are summarized in Table 43.

Table 43. Typical embankment material treatment/stabilization options to improve performance

Treatment/Stabilization Method	Issues that Can Be Mitigated
Engineered Subgrade Compaction with Moisture, Density, and Lift Thickness Control	• Excessive and differential settlement • Post-construction volume change (shrink-swell or collapse) due to moisture variations
Portland Cement Stabilization	• Frost heave and thaw softening • Post-construction volume change (shrink-swell or collapse) due to moisture variations • Wet/soft subgrade conditions during construction (to serve as construction platform)
Fly Ash Stabilization of Subgrade (Self-Cementing)	• Wet/soft subgrade conditions during construction (to serve as construction platform) • Post-construction volume change (shrink-swell or collapse) due to moisture variations
Lime Stabilization	• Shrink-swell potential (applicable for high plasticity clays)

Table 43 continued

Treatment/Stabilization Method	Issues that Can Be Mitigated
Geosynthetic Reinforcement	• Poor support (low CBR/shear strength) during construction (to serve as construction platform)

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APPENDIX A. STATE SPECIFICATION FOR EMBANKMENT CONSTRUCTION OF GRANULAR MATERIALS

Table 44. Specifications of embankment construction for granular materials

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
AL	2012	specify density	NR	maximum 8 in. loess thickness	NR	≥ 95% of maximum γd	
AK	2015	specify density	NR	maximum 8 in. loess thickness	≤ +/-2% of w_{opt}	≥ 95% of maximum γd	
AZ	2011	specify density	NR	less than maximum rock size or 2 ft	at or near w_{opt}	≥ 95% of maximum γd	If asphaltic concrete is to be placed directly on the subgrade, the top six in. of the embankment must be compacted to 100 percent of its maximum density. Material to be placed in dikes must be compacted to at least 95 percent of its maximum density.
AR	2014	specify density	The cleared surface shall then be completely broken up by plowing, scarifying, or disking to a minimum depth of 6 in. (150 mm).	8 to 12 in.	near w_{opt}	≥ 95% of maximum γd	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
CA	2010	specify density	NR	Over 50% by volume use max. rock size; From 25% to 50% by volume use Max. rock size up to 3 feet; Less than 25% by volume, 8 in. in areas between rocks larger than 8 in..	NR	0.5 foot below the grading plane for the width between the outer edges of shoulders and 2.5 ft below the finished grade for the width of the traveled way plus 3 ft on each side require ≥ 95% of maximum γd. Others ≥ 90% of maximum γd.	
CO	2011	specify density	NR	less than maximum rock size or 3 ft	≤ +/-2% of wopt; Soils having greater than 35 percent passing the 75 µm (No. 200) sieve shall be compacted to 0 to +3% of wopt	≥ 95% of maximum γd	
CT	2008	specify density	NR	maximum 3 ft loess thickness	at wopt	≥ 95% of maximum γd in accordance with AASHTO T 180, Method D.	
DE	2001	NR	NR	maximum 2 ft loess thickness	≤ +/-2% of wopt	≥ 95% of maximum γd by AASHTO T 99 Method C, Modified.	
FL	2015	NR	NR	NR	NR	Compact top 6 in ≥ 100% of maximum γd	
GA	2013	NR	Ensure that thickness of the lifts and the compaction are approved by the Engineer.				

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
HI	2005	NR	NR	maximum 1 ft loess thickness	(a) Two passes of a 50-ton compression-type roller. (b) Two passes of a vibratory roller having minimum dynamic force of 40,000 pounds impact per vibration and minimum frequency of 1,000 vibrations per minute. (c) Eight passes of a 10-ton compression-type roller. (d) Eight passes of a vibratory roller having minimum dynamic force of 30,000 pounds impact per vibration and minimum frequency of 1,000 vibrations per minute.		
ID	2012	Class A Compaction	NR	maximum 18 in. loess thickness	From -4% to +2% of w_{opt} determined by AASHTO T 99 or AASHTO T 180.	NR	
IL	2012	specify density	NR	maximum 6 in. loess thickness or maximum 8 in. approved by engineer	decided by engineer	≥ 100% of maximum γd of the standard laboratory density.	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
IN	2016	The compaction shall be accomplished with an approved vibratory tamping-foot roller in conjunction with a static tamping-foot roller.	Shale and/or Soft Rock Embankment: minimum of 3 passes with the static roller and a minimum of 2 passes with the vibratory roller. The rollers shall not exceed 3 mph (5 km/h) during these passes. Shale and Thinly Layered Limestone: The minimum number of passes with static roller and the vibratory tamping-foot roller shall be 6 static and 2 vibratory.	Rock Embankment: maximum 8 in. loess thickness top 2 ft of embankment. Embankment exceeds 5 feet, less than maximum rock size or 4 ft loess thickness. Embankment is 5 ft or less, less than maximum rock size or 2 ft loess thickness. Shale and/or Soft Rock Embankment: 8 in. (200 mm) maximum loose lifts; Shale and Thinly Layered Limestone: 8 in. (200 mm) maximum loose lifts	from -2% to +1% of wopt, silt or loess material from -3% to wopt	≥ 95% of maximum γd in accordance with AASHTO T 99	Maximum density and optimum moisture content shall be determined in accordance with AASHTO T 99 using method C for granular materials
IA	2012	Do not use compaction equipment	NR	NR	≤ +/-2% of w_{opt} based on standard Proctor optimum moisture content	First layer ≥ 90% of maximum γd. succeeding layer ≥ 95% of maximum γd	For compaction of sand or other granular material, use either a self-propelled pneumatic roller meeting the requirements or self-propelled vibratory roller meeting the requirements
KS	2015	Type B: Roller Walk out/ roller can support on its feet/ 90% of standard density	NR	less than maximum rock size or 2 ft	Specified on construction plans unless approved by Engineer	specified in the Contract Documents	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
KY	2012	specify density	minimum disk diameter of 2 feet	maximum 2 ft loess thickness	≤ +/-2% of w_{opt} determined according to KM 64-511.	≥ 95% of maximum γd as determined according to KM 64- 511. AASHTO Y 99	
LA	2006	specify density	NR	maximum 15 in. loess thickness or specify on plans	≤ +/-2% of w_{opt} established in accordance with DOTD TR 415 or TR 418	≥ 95% of maximum γd determined in accordance with DOTD TR 415 or TR 418	
ME	2014	specify density	NR	maximum 3 ft loess thickness	Adjust to meet specify density	≥ 90% of maximum γd in accordance with AASHTO T 180, Method C or D,	
MD	2008	specify density	NR	less than maximum rock size or 2 ft	≤ +/-2% of wopt	1 ft below the top of subgrade≥ 92% of maximum γd per T 180. Top 1 ft ≥ 97% of maximum γd.	
MA	1995	specify density	NR	maximum 3 ft loess thickness	at wopt	≥ 95% of maximum γd by AASHTO T 99	
MI	2012	specify density	NR	maximum 3 ft loess thickness	Soil moisture content must be between 5 percent and optimum moisture.	≥ 95% of maximum γd	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
MN	2014	NR	One pass over each strip covered by the tire for granular soils at an operating speed from 2.5 mph to 5 mph. Disc soils with greater than 20 percent passing the No. 200 [75 µm] sieve.	maximum 1 ft loess thickness	Excavation Depth Below Grading Grade < 30 in., Relative Moisture Content 65% to 102% - Compact to 100% of maximum density; / Excavation Depth Below Grading Grade ≥ 30 in., Relative Moisture Content 65% to 115% - Compact to 95% of maximum density or compact with 4 passes of a roller		
MS	2007	specify density	NR	less than maximum rock size or 3 ft	maintained by contractor and approved by engineer	For basement and design soils, the required density shall be ≥ 95% of maximum γd and ≥ 98% of maximum γd, respectively.	
MO	2014	Compaction of Embankment and Treatment of Cut Areas with Moisture and Density Control	The compactive effort on rocky material shall making four complete passes on each layer with a tamping-type roller or two complete passes on each layer with a vibratory roller.	maximum 1 ft loess thickness or maximum 2 ft rock size too big	NR	≥ 90% of maximum γd	Tampers or feet of tamping-type roller ≥ 6 in. from the surface of the drum with a minimum load on each tamper of 250 psi. The vibratory roller shall have 16 to 20 tons compacting power.
		Not Constructed with Density or Moisture and Density Control.	All equipment movements over the entire embankment area and of at least 3 complete passes with a tamping-type roller over the entire area to be compacted.			Each layer of compacted by three complete passes of the tamping-type roller. A vibratory roller may be used if approved by the engineer.	Compactive efforts shall be continued, if necessary, until the tamping ft penetrate no more than 2 in. (50 mm) into the layer of material being compacted

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
MT	2014	NR	NR	When the excavated material contains more than 25% rock by volume, 6 in. or larger in its greatest dimension, place the embankment in layers 2 in. thicker than the maximum size rock in the material not to exceed 24 in. loose thickness. Individual rocks and boulders larger than 24 in. in diameter may be placed in the embankment if the rocks do not exceed 48 in. vertical height after placement,	≥ 95% of maximum γ_d with ≤ +/-2% of w_{opt}		
NE	2007	Class I	NR	maximum 1 ft loess thickness	Class I: NR	Class I: NR	
		Class II	NR	maximum 8 in. loess thickness	Class II: Adjust to meet require density.	Class II: NR	
		Class III	NR		Class III: shown in the plans.	Class III: shown in the plans.	
NV	2014	NR	Minimum of 3 complete passes each layer at speed not exceeding 8 km/hr (5 mph)	minimum 2 ft loess thickness	NR	NR	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
NH	2010	specify density	NR	minimum 4 ft loess thickness	NR	≥ 95% of maximum γd	For earth materials under approach slabs and for earth materials within 10 ft (3 m) of the back of structures not having approach slabs, at least 98 percent of maximum density shall be obtained
NJ	2015	Control Fill Method	Pneumatic-Tired Roller 5 minimum pass; Dynamic Compactor Number of passes to optimize density; 3-Wheel 10-Ton Roller 4 minimum pass; Dynamic Compactor (Vibratory roller with 6-ton min. static weight at drum) 2 to 5	less than 1.5 times maximum rock size or 3 ft	NR	≥ 95% of maximum γd determined according to AASHTO T 99, Method C,	
		Directed Method				passes per lift specify by equipment	
NM	2014	specify density	NR	maximum8 in. loess thickness	NR	≥ 95% of maximum γd	
NY	2015	specify density	The compactive effort (number of passes and travel speed) is uniformly applied and not less than that specified for the given equipment class and lift thickness.	maximum 6 in. loess thickness	determined by contractor	≥ 95% of maximum γd of Standard Proctor Maximum Density will be required	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
NC	2012	specify density	NR	maximum 3 ft loess thickness	NR	≥ 95% of maximum γd in accordance AASHTO T 99	
ND	2014	NR	NR	less than maximum rock size or 2 ft	NR	NR	
OH	2013	specify density	For soil or granular material, when a test section is used, use a minimum compactive effort of 8 passes with a steel wheel roller having a minimum effective weight of 10 tons (9 metric tons). Compact Type D and Type E granular material using at least ten passes of a smooth drum vibratory roller having a minimum effective weight of 10 tons (9 metric tons).	maximum 6 in. loess thickness, or less than 6 in. more than maximum rock size or 3 ft	NR	specify by pass numbers	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
OK	2014	specify density	for rock fill layers 12 in thick or less, 4 pass using 50 ton compression type roller; 4 pass using vibratory roller with dynamic force of at least 40500 lbf per cycle and frequency of at least 16 Hz; 8 pass using 22 ton compression type roller; 8 pass using vibratory roller with dynamic force of at least 29250 lbf per cycle and frequency of at least 16 Hz for rock layer thicker than 12 in., increase the number of roller-passes for each additional 6 in. increment by the number required for first 12 in.	maximum 2 ft loess thickness	for A-4 or A-5 soil groups, from -4% to 0% of wopt	specify by pass numbers	
OR	2015	specify density	NR	maximum 15 in. loess thickness or less than maximum rock size or 3 ft	from -4% to +2% of wopt	≥ 95% of maximum γd	

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
PA	2015	specify density	NR	less than maximum rock size or 3 ft	from -3% to 0% of wopt	≥ 97% of maximum γd determined according to PTM No. 106, Method B. Top 3 ft of embankment ≥100% of maximum γd.	
RI	2013	specify density	NR	maximum 3 ft loess thickness	NR	Embankment of 3 ft below subgrade shall be compacted ≥ 90% of maximum γd. The remainder of the roadway section up to subgrade shall be compacted ≥ 95% of maximum γd.	
SC	2015	specify density	NR	Maximum 8 in. loess thickness top 2 ft of embankment. Embankment exceeds 5 feet, less than maximum rock size or 4 ft loess thickness. Embankment is 5 ft or less, less than maximum rock size or 2 ft loess thickness.	Suitable moisture	≥ 95% of maximum γd	
SD	2004	Specified Density Method	The disk shall be a tandem disk approximately 12 ft wide with eight disk blades, approximately 36 in. in diameter, per row,	less than maximum rock size or 3 ft loess thickness	if w_{opt} of embankment soil is 0% to 15%, require 95% or Greater maximum γd, and -4% to +4% of w_{opt} control; if w_{opt} of embankment soil is 15% or Greater, require 95% or Greater maximum γd, and -4% to +6% of w_{opt} control		

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
		Ordinary Compaction Method	and shall weigh approximately 11,800 pounds (5350 kg). This requirement will be waived for A-3 and A-2-4(0) soils.		Adjust to meet require density	Compaction may be accomplished with any type of equipment, which with adequate moisture content will give uniform satisfactory results.	
TN	2015	specify density	Provide a minimum of 3 passes with the static roller and 2 passes with the vibratory roller. The Engineer may direct additional passes with either or both rollers until satisfactory breakdown and compaction is accomplished.	maximum 3 ft loess thickness	NR	Non-Degradable Rock: Rolling is not required if the rock embankment consists of sound, non-degradable material placed in greater than 10 in. layers; Degradable Rock: provide a minimum of 3 passes with the static roller and 2 passes with the vibratory roller.	
TX	2014	Ordinary Compaction.	NR	maximum 18 in. loess thickness	NR	Compact each layer until there is no evidence of further consolidation	
		Density Control			For PI ≤ 15, no moisture content required, density ≥ 98% γd		

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
UT	2015	specify density	NR	maximum 6 in. compacted thickness	Maintain appropriate moisture for compaction during processing.	Acceptance is on a lot-by-lot basis when average density is ≥ 96% of maximum γd and no single determination is lower than 92 percent.	
VT	2011	specify density	The water shall be uniformly and thoroughly incorporated into the soil by disking, harrowing, blading, or other approved methods.	maximum 24 in. loess thickness	≤ +2% of w_{opt} or less than the quantity will cause unstable	≥ 90% of maximum γd determined by AASHTO T 99, Method C. Top 24 in. of any embankment ≥ 95% of maximum γd.	
VA	2014	specify density	disking or punching the mulch partially into the soil;	less than maximum rock size	NR	Density requirements may be waived.	
WA	2015	NR	NR	maximum 18 in. loess thickness unless rock size over 18 in.	NR	Use compression roller or vibratory roller. The roller shall make one full coverage for each 6 in., or any fraction of 6 in. of lift depth. When lift depth is 18 in. or less, the Contractor may use a compression roller or a vibratory roller make four full coverages for each 6 in., or any fraction of 6 in., lift depth.	Use 50-ton compression roller or vibratory roller have at least 40,000 lbs impact per vibration and at least 1,000 vibrations per min. Use a 10-ton compression roller or vibratory roller having a dynamic force of at least 30,000-pounds impact per vibration and at least 1,000 vibrations per min.

Table 44 continued

State	Spec Date	Placement/ compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
WV	2011	NR	NR	maximum 6 in. compacted thickness	NR	≥ 95% of maximum γd when less than 40% particles by weight retained on 3/4 in. sieve	
WI	2014	Standard Compaction	NR	maximum 12 in. loess thickness	NR	Compact each layer of the embankment until the compaction equipment achieves no further significant consolidation.	
		Special Compaction				Embankments ≤ 6 ft, ≥ 95% of maximum γd. Embankments ≥ 6 ft, 6 ft below subgrade ≥ 90% of maximum γd, rest 6 ft to finish subgrade ≥ 95% of maximum γd	
WY	2015	Special Compaction	NR	maximum 12 in. loess thickness when rock size over 8 in.	from -4% to +2% of wopt	place and compact material above the 6 in scarified layer ≥ 95% of maximum γd. AASHTO T 99	

APPENDIX B. STATE SPECIFICATION FOR EMBANKMENT CONSTRUCTION OF NON-GRANULAR MATERIALS

Table 45. Specifications of embankment construction for non-granular materials

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
AL	2012	specify density	NR	maximum 8 in. loess thickness	NR	≥ 95% of maximum γd	
AK	2015	specify density	During the winter, compact 3 passes per layer with sheep's foot compactor/roller or vibratory grid roller and until frozen chunks are reduced in size to less than 2 in. in any dimension.	maximum 8 in. loess thickness	≤ +/-2% of w_{opt}	≥ 95% of maximum γd	
AZ	2011	specify density	NR	maximum 8 in. loess thickness	at or near w_{opt}	≥ 95% of maximum γd	If asphaltic concrete placed directly on the subgrade, the top 6 in. of the embankment must be compacted to 100% of maximum γd. Material to be placed in dikes must be compacted ≥ 95% of maximum γd
AR	2014	specify density	The cleared surface shall then be completely broken up by plowing, scarifying, or disking to a minimum depth of 6 in.	maximum 10 in. loess thickness	at or near w_{opt}	≥ 95% of maximum γd	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
CA	2010	specify density	NR	maximum 8 in. loess thickness	NR	0.5 foot below the grading plane for the width between the outer edges of shoulders and 2.5 ft below the finished grade for the width of the traveled way plus 3 ft on each side require ≥ 95% of maximum γd. Others ≥ 90% of maximum γd.	
CO	2011	specify density	NR	maximum 8 in. loess thickness	≤ +/-2% of wopt; Soils having greater than 35 percent passing the 75 µm (No. 200) sieve shall be compacted to 0 to +3% of wopt	≥ 95% of maximum γd determined in accordance with AASHTO T 180	
CT	2008	specify density	NR	maximum 12 in. loess thickness	at wopt	≥ 95% of maximum γd in accordance with AASHTO T 180, Method D.	
DE	2001	specify density	NR	maximum 8 in. loess thickness	≤ +/-2% of wopt	≥ 95% of maximum γd as determined by AASHTO T 99 Method C, Modified.	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
FL	2015	specify density	NR	For A-3 and A-2-4 Materials with up to 15% fines: max 12 in. compacted thickness; For A-1, Plastic materials and A-2-4 Materials with greater than 15% fines: max 6 in. compacted thickness	Adjust to meet specify density	≥ 100% of maximum γ_d as determined by AASHTO T-99, Method C,	
GA	2013	specify density	NR	maximum 8 in. loess thickness	the range of wopt	≥ 95% of maximum γd within 1 ft of the top of the embankment. Top 1 ft of the embankment, ≥ 100% of maximum γd.	
HI	2005	specify density	NR	maximum 9 in. loess thickness	≤ +/-2% of w_{opt} in accordance with AASHTO T 180.	≥ 95% of maximum γd. Top 6 in. of in-situ material and embankment material below top 2 ft of subgrade, requires ≥ 90% of maximum γd	
ID	2012	**Class A Compaction.** Default compaction method. less than 10% retained on the 3 in. sieve; and more than or equal to 30 percent retained on the ¾" sieve, minimum of 95 percent of maximum dry density by AASHTO T 99 Method C	NR	maximum 8 in. loess thickness	from -4% to +2% of w_{opt} determined by AASHTO T 99 or AASHTO T 180.E13	≥ 95% of maximum γd	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
		Class B Compaction. Top 12 in still using class A compaction. by routing construction equipment uniformly over the entire surface of each layer.					
		Class C Compaction. Shown on the plans or as directed by the Engineer. Use class A compaction to a depth of 8 in.					
		Class D Compaction. approved by engineer		maximum 12 in. loess thickness			
IL	2012	specify density	NR	maximum 8 in. loess thickness	120% of w_{opt} for top 2 ft	If embankment ≤ 1.5 ft, all lifts ≥ 95% of maximum γd. If the embankment height is between 1.5 ft and 3 ft inclusive, the first lift ≥ 90% of maximum γd, and the balance ≥ 95% of maximum γd. If embankment ≥ 3 ft, the lower 1/3 of the embankment, but not to exceed the lower 2 ft, ≥ 90% of maximum γd. The next 1 ft ≥ 93% of maximum γd, and the balance≥ 95% of maximum γd.	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
IN	2016	Embankment With Density Control: Compacting equipment shall include at least one 3 wheel roller or other approved equipment provide a smooth and even surface. Embankment Without Density Control: compacted with crawler-tread equipment or with approved vibratory equipment, or both.	NR	Embankment With Density Control: maximum 8 in. loess thickness; Embankment Without Density Control: maximum 6 in. loess thickness; location inaccessible to the compacting equipment, maximum 4 in. loess thickness	from -2% to +1% of wopt, silt or loess material from -3% to w_{opt}	≥ 95% of maximum γd in accordance with AASHTO T 99	DCP were used in compaction of chemically modified soils: Acceptance testing for compaction of chemically modified soils will be performed on the finished grade with a DCP in accordance with ASTM D6951
IA	2012	Type A: compaction requiring a minimum of 1 rolling per in. depth of each lift. A further requirement is that the roller continues operation until it is supported on its feet, or the equivalent.	Disk the area with a least one pass of a tandem axle disk or 2 passes with a single axle disk prior to compaction.	maximum 8 in. loess thickness	≤ +/-2% of w_{opt}	Compact the first layer ≥ 90% of maximum γd. Compact each succeeding layer ≥ 95% of maximum γd.	1. If the type of compaction is not specified, Type A compaction will be required. 2. When compaction with moisture and density control is specified, any type of equipment which will produce the desired results may be used for compaction.
		Type B: refers to compaction requiring a specified number of diskings and roller coverages, or the equivalent.	One disking per 2 in. of loose thickness.				

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
		Other Method: Reasonably uniform throughout the compacted lift; At least 95% of maximum density, determined according to Materials Laboratory Test Method No. Iowa 103.	NR				
KS	2015	Type AAA: 100% of Standard Density	NR	maximum 8 in. loess thickness	≤ +/-5% of wopt	specified in the Contract Documents	
		Type AA 95% of Standard Density					
		Type A 90% of Standard Density					
KY	2012	specify density	minimum disk diameter of 2 ft	maximum 12 in. loess thickness	≤ +/-2% of w_{opt} determined according to KM 64-511.	≥ 95% of maximum γd as determined according to KM 64-511	
LA	2006	specify density	NR	maximum 12 in. loess thickness	≤ +/-2% of w_{opt} established in accordance with DOTD TR 415 or TR 418	≥ 95% of maximum γd in accordance with DOTD TR 415 or TR 418	
ME	2014	specify density	NR	maximum 8 in. loess thickness	Adjust to meet specify density	≥ 90% of maximum γd in accordance with AASHTO T 180, Method C or D	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
MD	2008	specify density	the entire surface of each lift shall be traversed by not less than one tread track of heavy equipment or compaction shall be achieved by a minimum of 4 complete passes of a sheepsfoot, rubber tired or vibratory roller.	maximum 8 in. loess thickness	≤ +/-2% of wopt	1 ft below the top of subgrade ≥ 92% of maximum γd per T 180. Top 1 ft ≥ 97% of maximum γd.	
MA	1995	specify density	NR	maximum 12 in. loess thickness	at wopt	≥ 95% of maximum γd by AASHTO T 99	
MI	2012	specify density	NR	maximum 9 in. loess thickness	≤ +3% of wopt	≥ 95% of maximum γd	
MN	2014	100% Relative Density for ≤ 3ft Below Grading Grade of Road Core	Make two passes over each strip covered by the tire width for non-granular soils at an operating speed from 2.5 mph to 5 mph. Disc soils with greater than 20	maximum 12 in. loess thickness	Excavation Depth Below Grading Grade < 30 in., Relative Moisture Content 65% to 102% - Compact to 100% of maximum γd; / Excavation Depth Below Grading Grade ≥ 30 in., Relative Moisture Content 65% to 115% - Compact to 95% of maximum γd or compact with 4 passes of a roller		
		100% Relative Density Within the Minimum of Either the Horizontal Distance Equal to the Full Height of a Structure or within 3 ft of a Structure					Compact the entire lift to achieve a dynamic cone penetration index (DPI) value during embankment compaction

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
		95% Relative Density Remaining embankment in the road core	percent passing the No. 200 [75 µm] sieve.				Use the Specified Density method for acceptance for materials not meeting the requirements, and use the granular penetration index method for materials meeting the requirements of 2105.1A7,
MS	2007	specify density	NR	maximum 8 in. loess thickness	maintained by contractor and approved by engineer	For basement and design soils, the required density shall be ≥ 95% of maximum γd and ≥ 98% of maximum γd, respectively.	
MO	2014	Compaction of Embankment and Treatment of Cut Areas with Moisture and Density Control	At least 3 complete passes with a tamping-type roller over the entire area to be compacted. Compactive efforts shall be continued, if necessary, until the tamping ft penetrate no more than 2 in. (50 mm) into the layer of material being compacted.	maximum 8 in. loess thickness	when embankments less than 30 ft, ≤ +3% of wopt; Embankment more than 30 ft, ≤ w_{opt} for loess soil	≥ 90% of maximum γd	When eliminate rubbery condition of embankment, it may be required soils have a moisture content below the optimum during compacting work, except LL ≥ 40, where placed in embankments within 5 ft (1.5 m) of the top of the finished subgrade or where encountered in areas of cut compaction.

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
MT	2014	NR	Using a tandem type construction disk with a maximum disk spacing of 14 in. (355 mm) and a minimum worn disk diameter of 25 in. (635 mm).	maximum 8 in. loess thickness	≥ 95% of maximum γd with ≤ +/-2% of wopt		
NE	2007	Class I	NR	maximum 12 in. loess thickness	NR	NR	
		Class II	NR	maximum 8 in. loess thickness	Adjust to meet specify density	NR	
		Class III	NR	maximum 8 in. loess thickness	Adjust to meet specify density	Shown in the plans.	
NV	2014	specify density	NR	maximum 8 in. loess thickness	moisture content within the prescribed limits	≥ 95% of maximum γd by Test method No. Nev. T108	Compact base of cuts, Natural ground less than 1.5m (5ft) not less than 90% of maximum density determined by Test method No. Nev. T108;
NH	2010	specify density	NR	maximum 12 in. loess thickness	NR	≥ 95% of maximum γd	For earth materials under approach slabs, at least 98 percent of maximum density shall be obtained.
NJ	2015	End-Dumping Method	Pneumatic-Tired Roller 5 minimum pass; Padfoot Roller 8 minimum pass	NR	NR	NR	
		Control Fill Method		maximum 12 in. loess thickness		≥ 95% of maximum γd according to AASHTO T 99, Method C,	
		Directed Method		maximum 8 in. loess thickness		passes per lift specify by equipment	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
		Density Control Method		maximum 12 in. compacted thickness		≥ 95% of maximum γd	
NM	2014	specify density	NR	maximum 8 in. loess thickness	General -5% to 0 of wopt. For soils PI ≥ 15, 0% to +4% of wopt	≥ 95% of maximum γd	
NY	2015	specify density	The compactive effort (number of passes and travel speed) is uniformly applied and not less than that specified for the given equipment class and lift thickness.	Not exceed equipment allowance	determined by contractor	≥ 95% of maximum γd of Standard Proctor Maximum Density will be required.	
NC	2012	specify density	NR	maximum 10 in. loess thickness	NR	≥ 95% of maximum γd in accordance AASHTO T 99	
ND	2014	Compaction Control, Type A.	NR	maximum 12 in. loess thickness	for ND T180, 0% to +5% of w_{opt}; for ND T99, -4% to +5% of wopt	ND T180 requires ≥ 90% of maximum γd; ND T99 requires ≥ 95% of maximum γd	
		Compaction Control, Type B.		maximum 12 in. loess thickness	NR	Use a sheepsfoot roller until the roller pads penetrate the surface a maximum of 0.5 in.	
		Compaction Control, Type C.		maximum 8 in. loess thickness	NR	NR	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
OH	2013	specify density	NR	maximum 8 in. loess thickness	NR	If maximum γd from 90 to 104.9 lb/ft^3, requires at least 102% maximum dry density compaction energy; if maximum γd from 105 to 119.9 lb/ft^3, requires at least 100% maximum dry density; if maximum γd more than 120 lb/ft^3, requires at least 98% maximum dry density	
OK	2014	specify density	NR	maximum 8 in. loess thickness	≤ +/-2% of wopt, for A-4 or A-5 soil groups, from -4% to 0% of wopt	≥ 95% of maximum γd	
OR	2015	specify density	NR	maximum 8 in. loess thickness	from -4% to +2% of wopt	≥ 95% of maximum γd	
PA	2015	specify density	NR	maximum 8 in. loess thickness	from -3% to 0% of wopt	Compact embankment for its full width ≥ 97% of maximum γd according to PTM No. 106, Method B. Compact top 3 ft of embankment for full width to ≥ 100% of maximum γd.	
RI	2013	specify density	NR	maximum 12 in. compacted thickness	NR	Embankment of 3 ft below subgrade shall be compacted ≥ 90% of maximum γd. The remainder of the roadway section compacted ≥ 95% of maximum γd.	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
SC	2015	specify density	NR	maximum 8 in. loess thickness	Suitable moisture	≥ 95% of maximum γd	
SD	2004	Specified Density Method	The disk shall be a tandem disk approximately 12 ft wide with 8 disk blades, approximately 36 in. in diameter, per row, weigh approximately 11,800 pounds. This requirement waived for A-3 and A-2-4(0) soils.	maximum 8 in. loess thickness	if w_{opt} of embankment soil is 0% to 15%, require 95% or Greater maximum γd, and -4% to +4% of w_{opt} control; if w_{opt} of embankment soil is 15% or greater, require 95% or greater maximum γd, and -4% to +6% of w_{opt} control		
		Ordinary Compaction Method			Adjust to meet specify density	Compaction may be accomplished with any type of equipment, which with adequate moisture content will give uniform satisfactory results.	
TN	2015	specify density	NR	maximum 10 in. loess thickness	when 95% of maximum density is required, ≤ wopt. When 100% of maximum density is required, ≤ ±3% of wopt.	Compact each layer ≥ 95% of maximum γd. Unless otherwise specified, compact the top 6 in. of the roadbed in both cut and fill sections ≥ 100% of maximum γd	
TX	2014	Ordinary Compaction.	NR	maximum 8 in. loess thickness	Compact each layer until there is no evidence of further consolidation		
		Density Control		maximum 16 in. loess thickness or 12 in. compacted thickness	For PI ≤ 15, no moisture content required, density requires ≥ 98% of γd; For 15 < PI ≤ 35, moisture content should not less than Wopt, density requires 98% of γd ≤ γd ≤ 102% of γd; For PI > 35, moisture content should not less than Wopt, density requires 95% of γd ≤ γd ≤ 100% of γd		

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
Utah	2015	specify density	NR	maximum 12 in. loess thickness	Maintain appropriate moisture for compaction during processing.	≥ 96% of maximum γd and no single determination is lower than 92 percent.	
VT	2011	specify density	The water shall be uniformly and thoroughly incorporated into the soil by disking, harrowing, blading, or other approved methods.	maximum 8 in. loess thickness	≤ +2% of w_{opt} or less than the quantity will cause unstable	≥ 90% of maximum γd as determined by AASHTO T 99, Method C. the top 24 in. ≥ 95% of maximum γd.	
VA	2014	specify density	disking or punching the mulch partially into the soil;	maximum 8 in. loess thickness	≤ ±2% of wopt.	≥ 95% of maximum γd	
WA	2015	Method A	NR	maximum 2 ft loess thickness	NR	The Contractor shall compact each layer by routing loaded haul equipment over its entire width.	
		Method B		Top 2 ft, maximum 4 in. loess thickness. Below top 2 ft, maximum 8 in.	≤ +3% of wopt.	2 ft below finish subgrade ≥ 90% of maximum γd, rest 2 ft to finish subgrade ≥ 95% of maximum γd	

Table 45 continued

State	Spec Date	Placement/compaction Method	Disk/Passes	Lift Thickness	w	DD	Other Requirements
		Method C		loess thickness. Up to maximum 18 in. loess thickness after engineer permit		≥ 95% of maximum γd	
WV	2011	specify density	NR	maximum 4 in. compacted thickness	from - 4% to +3% of w_{opt} while material having less than 40% by weight retained on 3/4 in. sieve	≥ 95% of maximum γd when less than 40% particles by weight retained on 3/4 in. sieve	
WI	2014	Standard Compaction	NR	maximum 8 in. loess thickness	NR	Compact each layer of the embankment until the compaction equipment achieves no further significant consolidation.	
		Special Compaction				Embankments ≤ 6 ft, ≥ 95% of maximum γd. Embankments ≥ 6 ft, 6 ft below subgrade ≥ 90% of maximum γd, rest 6 ft to finish subgrade ≥ 95% of maximum γd	
WV	2015	with moisture and density control	NR	maximum 8 in. loess thickness	from -4% to +2% of wopt	≥ 90% of maximum γd	
		without moisture and density control			NR		

APPENDIX C. GRAIN SIZE DISTRIBUTION OF EMBANKMENT MATERIALS

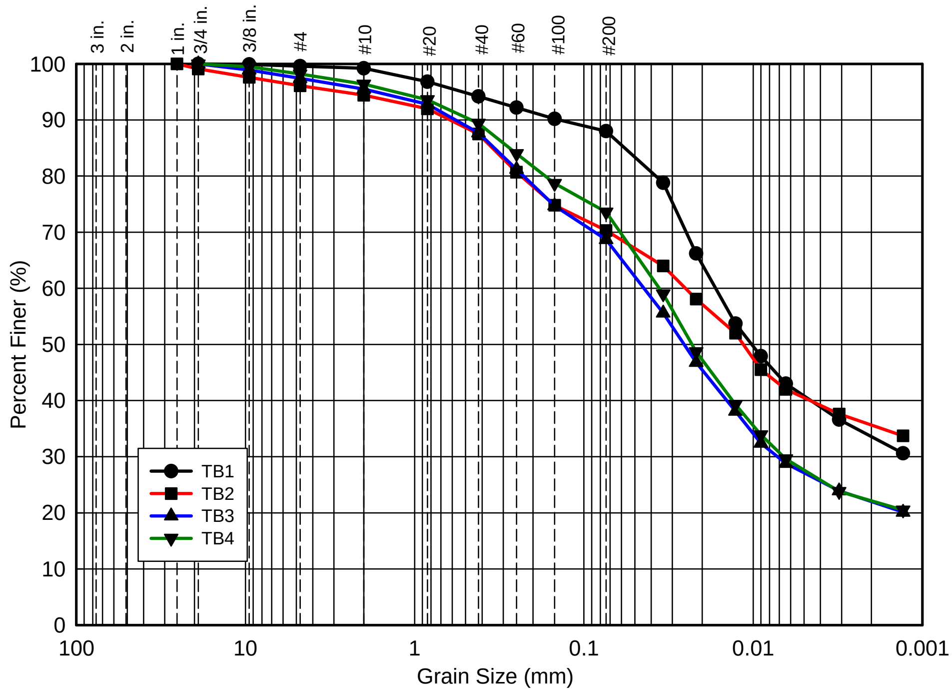

Figure 198. Polk County Project 1: Grain size distribution of embankment materials

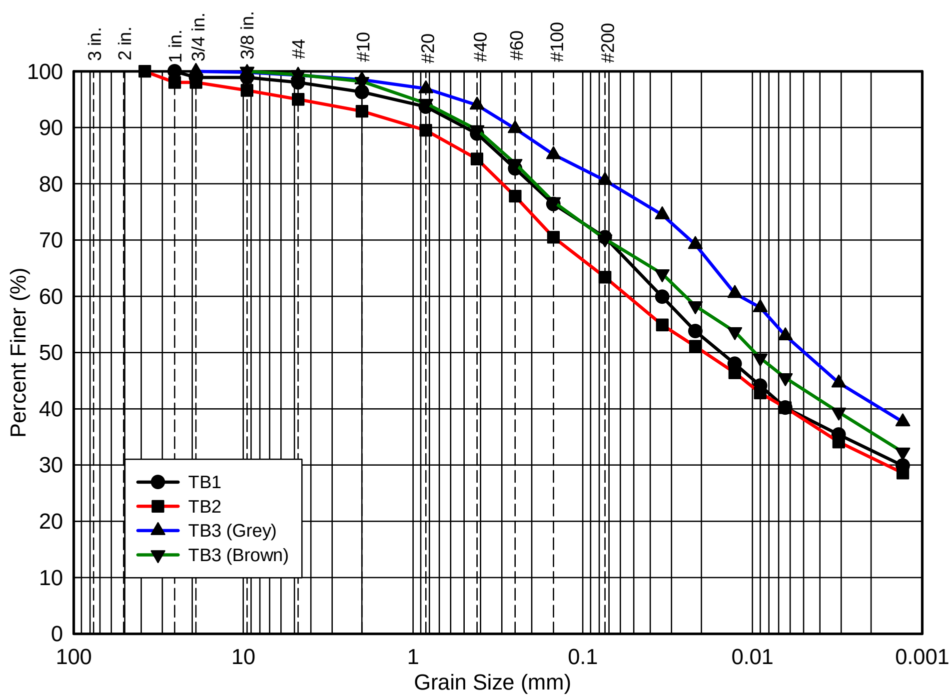

Figure 199. Warren County Project 2: Grain size distribution of embankment materials

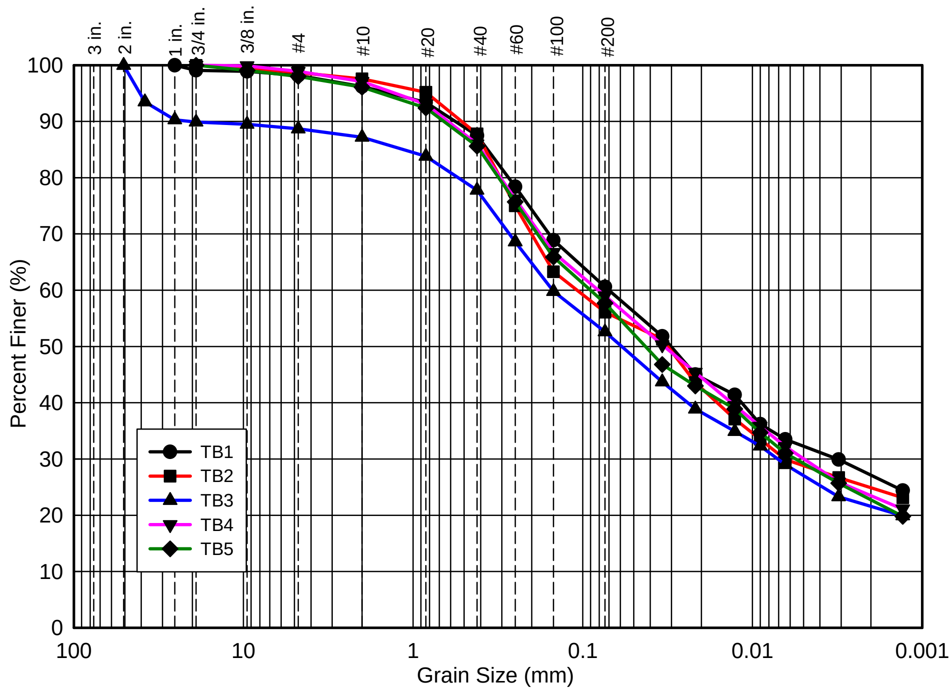

Figure 200. Linn County Project 3: Grain size distribution of embankment materials

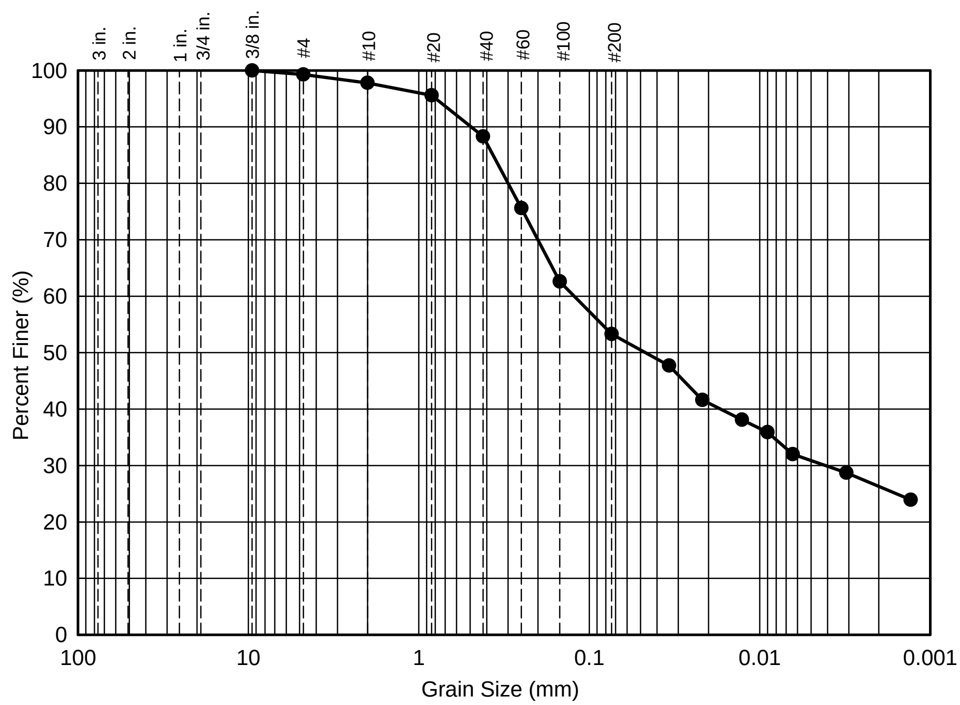

Figure 201. Linn County Project 4: Grain size distribution of embankment materials

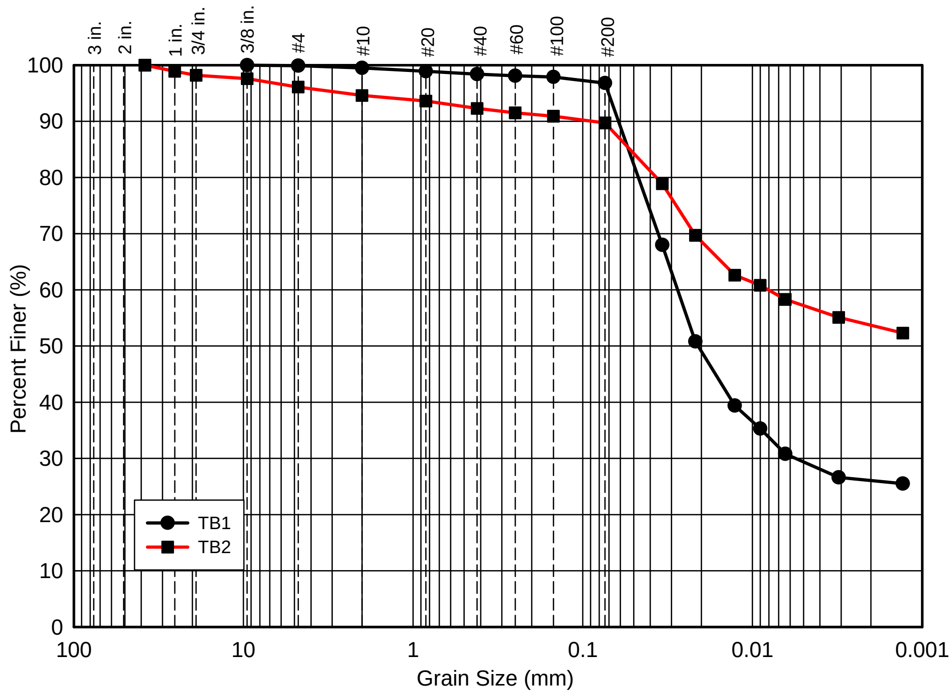

Figure 202. Mills County Project 5: Grain size distribution of embankment materials

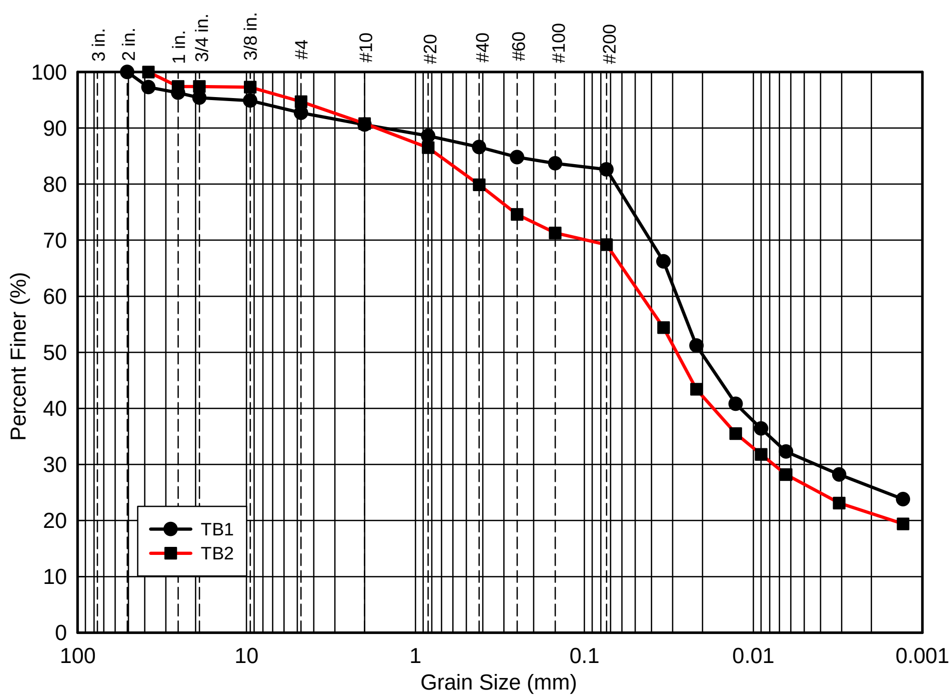

Figure 203. Pottawattamie County Project 6: Grain size distribution of embankment materials

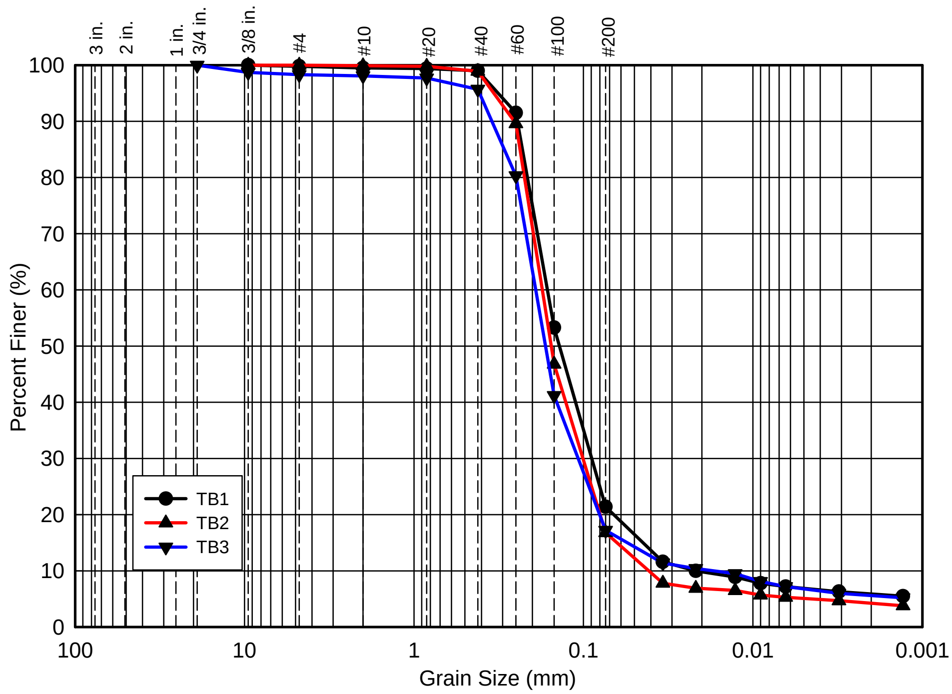

Figure 204. Woodbury County Project 7: Grain size distribution of embankment materials

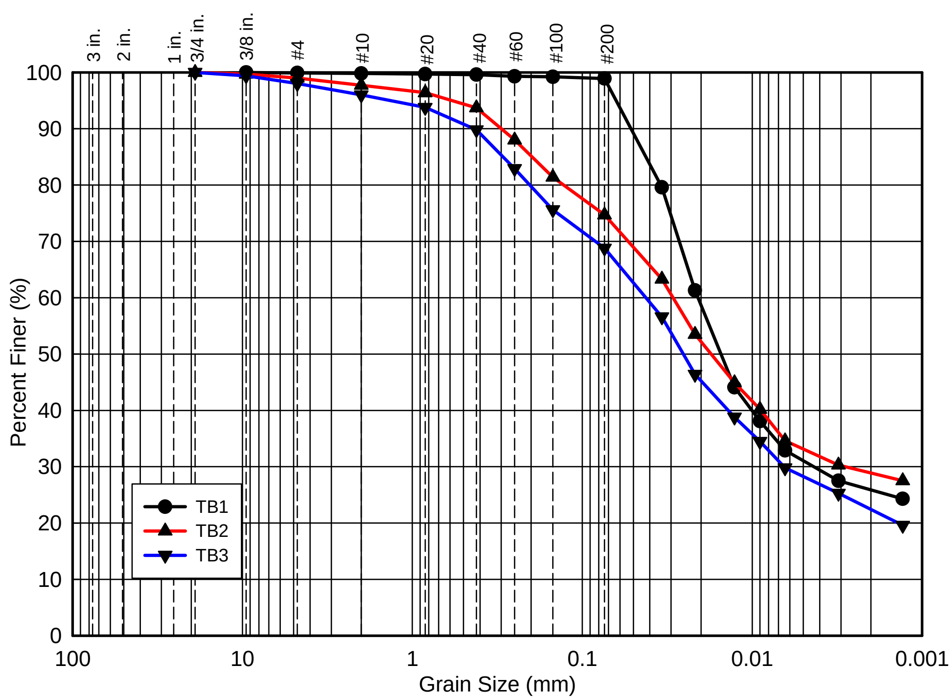

Figure 205. Scott County Project 8: Grain size distribution of embankment materials

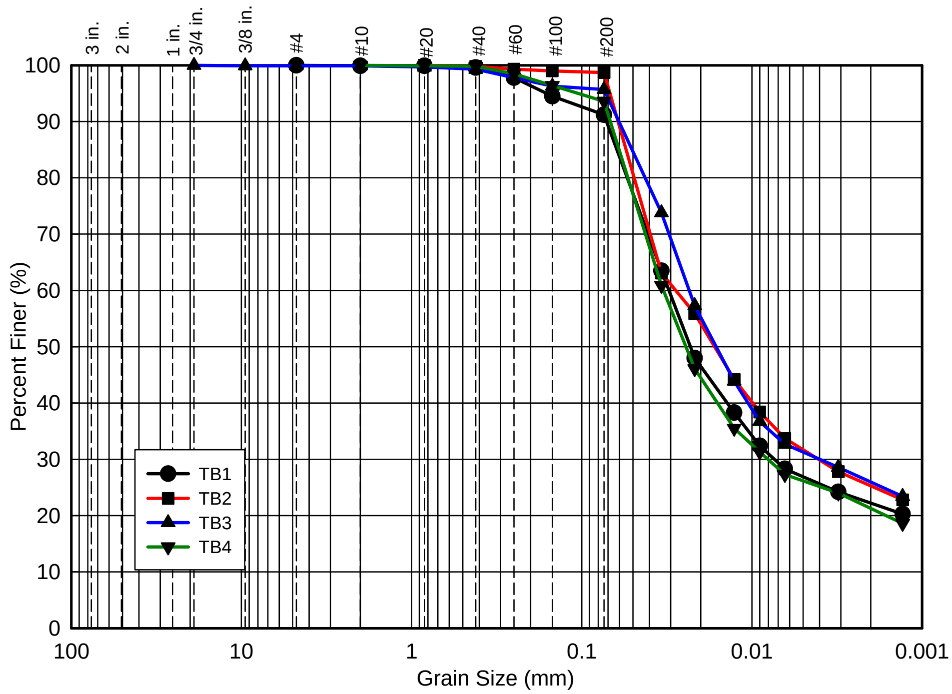

Figure 206. Woodbury County Project 9: Grain size distribution of embankment materials

APPENDIX D: LABORATORY TEST RESULTS

Test Bed	Treated soil properties									Untreated soil properties					
	Cement content (%)	UCS (psi)		Atterberg limits			F_{200}	Group index	Iowa DOT Material Suitability	Gravel content (%)	Sand content (%)	Silt content (%)	Clay content (%)	USCS Classification	AASHTO Classification
		Unsaturated	Saturated	LL	PL	PI									
Polk TB1	0	50.4	8.5	49	28	21	88	21	suitable	0.4	11.6	66.4	21.6	CL	A-7-6(21)
	4	174.3	78.6	41	28	13	74.1	10	suitable						
	8	279.9	230.6	40	32	8	64.5	5	suitable						
	12	409.8	320.7	40	NP	0	53.1	0	suitable						
Polk TB2	0	36.8	18.7	45	34	11	70.3	8	suitable	3.9	25.8	34.7	35.6	CL	A-7-5(8)
	4	120.2	54.3	43	30	13	59.3	7	suitable						
	8	324	187.1	41	31	10	47.9	3	suitable						
	12	442	265.2	38	NP	0	45.7	0	suitable						
Polk TB3	0	9.6	56.9	36	20	16	68.7	9	suitable	2.6	28.7	45.8	22.9	CL	A-6(9)
	4	224.1	134.2	34	28	6	58.5	2	suitable						
	8	336.7	251.7	35	NP	0	41.1	0	suitable						
	12	519.4	351.2	36	NP	0	32.3	0	suitable						
Polk TB4	0	54.2	8.3	34	17	17	73.6	11	suitable	1.8	24.6	50.9	22.7	CL	A-6(11)
	4	261.8	135.1	36	NP	0	61.9	0	suitable						
	8	438.5	313.6	38	NP	0	40.6	0	suitable						
	12	634.4	461.2	34	NP	0	40.4	0	suitable						
Warren TB1	0	59.3	0	44	31	13	70.5	9	suitable	2	27.5	37.3	33.2	CL	A-7-5(9)
	4	181.9	107.9	38	24	14	60.4	7	suitable						
	8	431.1	228.6	41	NP	0	36.8	0	suitable						
	12	686.9	359.7	38	NP	0	27.4	0	suitable						
Warren TB2	0	38.3	0	40	19	21	63.4	11	select	5	31.6	31.9	31.5	CL	A-6(11)
	4	223.3	103.7	39	24	15	55.7	6	select						
	8	413.7	213.3	38	NP	0	34.4	0	suitable						
	12	512	317.2	34	NP	0	25.7	0	suitable						
Warren TB3	0	38.7	0	54	20	34	80.6	28	unsuitable	0.7	18.7	39.1	41.5	CH	A-7-6(28)
	4	150.8	68	42	25	17	70.7	11	suitable						
	8	201	147.8	44	32	12	51.8	4	suitable						
	12	305.6	239.7	40	NP	0	31	0	suitable						
Linn 79 TB1	0	48.9	0	31	25	6	53.3	1	suitable	0.7	46	26.4	26.9	CL-ML	A-4(1)
	4	257.7	118.4	29	17	12	40.8	1	suitable						
	8	475.8	296.5	28	NP	0	28.6	0	suitable						
	12	492.2	408.8	29	NP	0	21.2	0	suitable						
Linn 77 TB1	0	60	0	31	12	19	60.6	8	select	1.8	37.6	32.9	27.7	CL	A-6(8)
	4	224.2	114.7	34	18	16	49.9	5	select						
	8	397.1	255.5	33	23	10	38.8	1	suitable						
	12	414.3	325.6	33	NP	0	29.4	0	suitable						
Linn 77 TB2	0	53.1	0	34	16	18	56.1	7	select	1.3	42.6	30.9	25.2	CL	A-6(7)
	4	233.2	121.5	34	22	12	51.3	3	select						
	8	466.6	290.4	32	NP	0	41	0	suitable						
	12	605.3	456.7	31	NP	0	22.4	0	suitable						
Linn 77 TB3	0	67.5	0	33	11	22	52.6	7	select	11.3	36.1	31.2	21.4	CL	A-6(7)

	4	305.6	219.3	32	21	11	43.1	2	select						
	8	676.6	472.9	32	NP	0	20.4	0	suitable						
	12	863.4	598	35	NP	0	15.8	0	suitable						
Linn 77 TB4	0	68.6	0	32	16	16	59	6	select	1.1	39.9	35.6	23.4	CL	A-6(6)
	4	146.8	78.8	43	27	16	48	5	select						
	8	281.9	163.1	43	29	14	37	1	select						
	12	436	271.9	39	NP	0	33.6	0	suitable						
Linn 77 TB5	0	47.1	0	30	16	14	57.7	5	select	2	40.3	34.8	22.9	CL	A-6(5)
	4	264.4	105.2	34	19	15	52.9	5	select						
	8	424.2	269.6	33	24	9	31.2	0	suitable						
	12	635.5	355.8	33	NP	0	23.4	0	suitable						
Pottawattamie TB1	0	63.9	5.3	43	18	25	82.6	20	suitable	7.3	10.1	56.2	26.4	CL	A-7-6(20)
	4	260.7	160.6	39	30	9	78.6	8	suitable						
	8	447.6	324.9	40	33	7	52.3	2	suitable						
	12	654.6	486.8	36	NP	0	37.5	0	suitable						
Pottawattamie TB2	0	49.3	0	42	19	23	69.2	14	suitable	5.3	25.5	48	21.2	CL	A-7-6(14)
	4	208.4	155.5	36	31	5	60.5	2	suitable						
	8	287.2	255.8	36	32	4	42.5	0	suitable						
	12	296	211.9	37	NP	0	35.3	0	suitable						
Mills TB1	0	53.9	0	38	34	4	96.8	7	suitable	0.1	3.1	70.6	26.2	CL-ML	A-4(7)
	4	268.8	224	35	27	8	88	8	suitable						
	8	762.9	528.1	34	32	2	49.8	0	suitable						
	12	903.1	709.1	36	NP	0	34.5	0	suitable						
Mills TB2	0	55.4	1.7	36	31	5	89.7	6	suitable	3.9	6.4	34.9	54.8	CL-ML	A-4(6)
	4	337.1	286.9	34	29	5	72.6	4	suitable						
	8	632.4	464.3	34	32	2	48.3	0	suitable						
	12	747.7	624.8	35	NP	0	29.4	0	suitable						
Scott TB1	0	59.2	5.8	39	32	7	98.9	10	suitable	0.1	1	72.9	26	CL-ML	A-4(10)
	4	257.3	167.7	34	26	8	85.2	7	suitable						
	8	533.2	353	34	31	3	52.1	0	suitable						
	12	686.7	519	35	NP	0	34.9	0	suitable						
Scott TB2	0	44	6.6	35	24	11	74.7	8	suitable	1	24.3	45.5	29.2	CL	A-6(8)
	4	299.8	197.2	33	27	6	61	2	suitable						
	8	608.6	484.9	32	NP	0	46.9	0	suitable						
	12	820.7	605.9	34	NP	0	40	0	suitable						
Scott TB3	0	48.4	5.8	28	17	11	68.8	5	suitable	2	29.2	45.9	22.9	CL	A-6(5)
	4	333	244.3	31	22	9	56.4	3	suitable						
	8	696.6	461.5	31	30	1	37.9	0	suitable						
	12	980.6	692.4	33	NP	0	25.1	0	suitable						
Woodbury (US20) TB1	0	60.2	9.3	32	25	7	91.2	7	suitable	0	8.8	68.8	22.4	CL-ML	A-4(7)
	4	292.3	184.4	33	26	7	65.4	4	suitable						
	8	525.8	429.3	33	31	2	53.9	0	suitable						
	12	789.7	554.3	34	NP	0	39	0	suitable						
Woodbury (US20) TB2	0	59.7	0	35	27	8	98.7	9	suitable	0	1.3	73.3	25.4	CL	A-4(9)
	4	278.6	189.4	41	31	10	76.3	8	suitable						

	8	488.4	341	40	35	5	50.5	1	suitable						
	12	663.3	484	43	NP	0	33.8	0	suitable						
Woodbury (US20) TB3	0	52.9	3.8	35	23	12	95.7	12	suitable	0.1	4.2	69.6	26.1	CL	A-6(12)
	4	288.7	169	40	31	9	69.8	6	suitable						
	8	534.4	343	40	34	6	43.2	1	suitable						
	12	735.7	513.7	41	NP	0	32.4	0	suitable						
Woodbury (US20) TB4	0	63.3	4.4	31	24	7	93.6	7	suitable	0	6.4	72	21.6	CL-ML	A-4(7)
	4	339.8	196	32	26	6	79.1	4	suitable						
	8	588.6	431.6	32	31	1	51.6	0	suitable						
	12	815	572.2	33	NP	0	32.9	0	suitable						
Woodbury (I29) TB1	0	0	0	NV	NP	NP	21.4	0	suitable	0.2	78.4	15.5	5.9	SM	A-2-4
	4	94.7	81.7	NV	NP	NP	9.3	0	suitable						
	8	268.6	234.9	NV	NP	NP	9	0	suitable						
	12	506.2	439.8	NV	NP	NP	8.6	0	select						
Woodbury (I29) TB2	0	0	0	NV	NP	NP	16.8	0	suitable	0	83.2	12.6	4.2	SM	A-2-4
	4	54.6	43.8	NV	NP	NP	7.7	0	suitable						
	8	120.2	108.2	NV	NP	NP	7.1	0	suitable						
	12	187.4	161.7	NV	NP	NP	7.4	0	suitable						
Woodbury (I29) TB3	0	0	0	NV	NP	NP	17.2	0	suitable	1.7	81.1	11.6	5.6	SM	A-2-4
	4	100	72.5	NV	NP	NP	8.2	0	suitable						
	8	238.3	211.4	NV	NP	NP	9.5	0	suitable						
	12	414.6	398.6	NV	NP	NP	8.3	0	select						

APPENDIX E. IOWA DOT PROPOSED INTERNAL MEMORANDUM FOR CEMENT STABILIZATION OF SOILS

CEMENT STABILIZATION OF SOILS

GENERAL

This procedure describes procedures for sampling and testing, and requirements for submittal and approval of mix design for cement stabilized soils.

SAMPLING AND MATERIALS

Each soil sample to be used in chemical stabilization shall be 75 pounds (35 kg). This sample size will also provide for tests to be performed according to Materials IM 545.

The cement used for stabilization shall meet the requirements of Type I or I/II from Section 4101.

SAMPLE PREPARATION AND TESTING

Laboratory tests on untreated soil shall be performed according to Materials IM 545. The material suitability should be classified in accordance with Section 2102. Additionally, sulfate content of the soil shall be determined per AASHTO T290. If the soil consists of soluble sulfate content > 3,000 ppm or the material classifies as unsuitable, chemical stabilization shall not be performed unless consulted with the engineer.

For each soil type, prepare three samples each for the following four mixes:

- Mix 1: Untreated soil
- Mix 2: 2% cement
- Mix 3: 4% cement
- Mix 4: 6% cement.

To determine the quantity of cement to add to the soil, multiply the cement percentage by the dry weight of the soil. Use cement that is from the same source(s) that will be used during construction.

First, the moisture-density relationship of the different mixtures shall be determined. Then, unconfined compressive strength testing shall be performed at target moisture contents, as described below.

Moisture-Density Relationship

The moisture versus dry density relationship of untreated and cement-treated samples shall be determined using one of the following alternatives:

Alternative 1:

- *Untreated Samples:* The maximum dry density and optimum moisture content of the untreated samples shall be determined using standard Proctor test in accordance with ASTM D698-12 [Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lb/ft^3 (600 kN-m/m^3)). A minimum 3-point Proctor is recommended.
- *Treated Samples:* The maximum dry density and optimum moisture content shall be determined in accordance with ASTM D558-11 [Standard Test Methods for Moisture-Density (Unit Weight) Relations of Soil-Cement Mixtures]. All treated samples must be compacted within 1 hour of mixing. A minimum 3-point Proctor is recommended.

Alternative 2:

The maximum dry density and optimum moisture content of untreated and treated samples shall be determined using the Iowa State University 2” by 2” Moisture-Density Test Method, per Chu and Davidson (1955). In preparing samples using the 2” by 2” method, use the following table for guidance on the total number of drop-hammer blows depending on the soil type to obtain results similar to the standard Proctor test.

Total number of drop-hammer blows	Soil type (based on AASHTO system)
6	A7 and A6
7	A4
14	A3, A2, and A1

Alternative 3:

First, determine the optimum moisture content of the untreated soil using standard Proctor test in accordance with ASTM D698-12 [Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12400 ft-lbf/ft^3 (600 kN-m/m^3))]. Then use the following equation to determine the optimum moisture content of treated samples, by using a water to cement (w/c) ratio of 0.25:

$$w_{\text{opt soil + cement}} = [(\%\ \text{cement added by weight}) \times (\text{w/c ratio})] + w_{\text{opt soil}}$$

Unconfined Compressive Strength

The unconfined compressive strength (UCS) tests shall be conducted on compacted samples at respective optimum moisture contents for untreated and treated soils, in accordance with ASTM D1633-00 (2007) [Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders]. As an alternative, tests can be conducted on 2” by 2” samples prepared per Alternative 2 above.

For each mix, prepare three samples for UCS testing for a total of twelve samples. Wrap each sample immediately after compaction with a plastic wrap and aluminum foil and store in a moisture-proof and airtight bag. All treated samples shall be cured at 100ºF (38ºC) for 7 days. Untreated samples shall be cured for no more than 24 hours.

After curing, all samples shall be vacuum saturated in accordance with ASTM C593-06 (2011) Section 11 [Standard Specification for Fly Ash and Other Pozzolans for Use with Lime for Soil Stabilization]. For samples that become fragile and cannot be retrieved from water for UCS testing, report the UCS as 0 psi.

Target cement content determination

The data obtained from UCS testing shall be plotted on a graph with cement content on x-axis and saturated UCS on y-axis. The average UCS of three samples shall be reported on the y-axis. The cement content corresponding to a saturated UCS of 100 psi shall be determined. 0.5% cement shall be added to determine the target cement content for the field application, as illustrated in Figure 1.

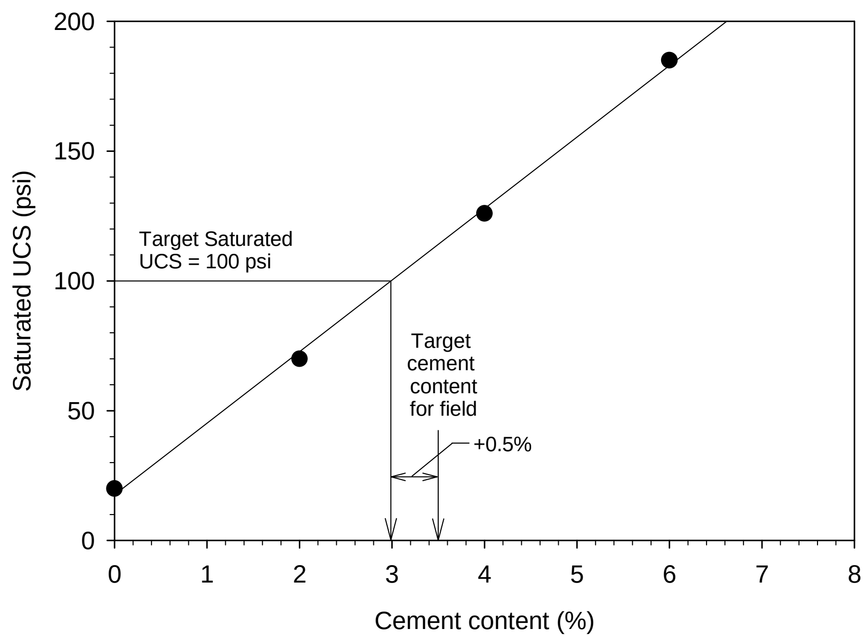

Figure 1. Determination of target cement content for field application

REPORTS

Each report shall contain the following for untreated soil:

- Sample ID number and location
- Atterberg Limits
- Percent Gravel, Sand, Silt, and Clay
- Textural classification
- AASHTO classification
- Proctor density and optimum moisture
- Percent Carbon Content
- Sieve analysis (Percent Passing)

- Sulfate content

Additionally, each report shall contain the following for untreated and treated soils (for each soil type, there will be a total of twelve samples):

- Percent cement added in each mixture
- Maximum dry density and optimum moisture content, and the alternative procedure followed as described in this IM.
- Unconfined compressive strength – for each sample

Submit a graph similar to Figure 1 with average saturated UCS versus % of cement in the mixture with the recommended rate of chemical stabilization for review and approval by the Engineer.