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Assessment of embankment construction QC/QA procedures in Iowa

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Assessment of embankment construction QC/QA procedures in Iowa

by

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Dedication

This thesis would not be complete without a mention of the support given to me by my Heavenly Father, my parents and the church committee (little family in Stonebrook) for whom this thesis is dedicated.

这篇研究生论文之所以可以完成全依仗于我的上帝，我的父母与 Stone brook 教会兄弟姐妹的帮助与支持，在此感谢！

Special for

My mother, Chunge Xue

And

My father, Xiaoying Chen

特别献给

我的母亲：薛春阁女士

和

我的父亲：陈晓莹先生

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LIST OF SYMBOLS

Symbol	Description	Unit
a	Radius of loading plate	mm
d_0	Measured peak deflection	mm
σ_0	applied stress	Mpa
E_{LWD}	Elastic modulus	MPa
S_m	Maximum saturation	—
n	Shape factor	—
p	the parameter which influences the width of the upper portion of the curve	—
CBR	California bearing ratio	%
Cc	Coefficient of curvature	—
Cu	Coefficient of uniformity	—
D_{10}	Diameter corresponding to 10% finer	mm
D_{30}	Diameter corresponding to 30% finer	mm
D_{60}	Diameter corresponding to 60% finer	mm
DCP	Dynamic cone penetrometer	—
D_i	Radial distance from the center of the plate to the sensor	mm
F_{200}	Fines content	%
f	Shape factor for LWD	—
G_s	Specific gravity	—
k	Stiffness estimated from a static plate load test	kPa/m
LWD	Light weight deflectometer	—
P	Applied load at surface	N
Pa	Atmospheric pressure	MPa
v	Poisson's ratio	—
LL	Liquid limit	—
PL	Plastic limit	—
PI	Plastic index	—
QC	Quality control	—
QA	Quality assurance	—
R^2	Coefficient of determination	—
RMSE	Root mean square standard error	—
γ_d	Dry unit weight	lb/ft^3
γ_{dmax}	Maximum dry unit weight	lb/ft^3
w_{opt}	Optimum moisture content	%
ϕ	Mold Size	in.

ABSTRACT

Proper quality control during embankment construction is critical to ensure long-term performance. Currently, the Iowa Department of Transportation (DOT) uses a specification that involves moisture control or moisture and density control as part of the quality control (QC) and quality assurance (QA) process. A review of other state DOT specifications revealed that a majority of them also were similar to the Iowa DOT specifications, although some differences existed in terms of what the limits are. Recent testing by Iowa State University researchers has revealed that embankments are frequently constructed outside the moisture and density control limits, even though the QC/QA testing data showed otherwise. To further evaluate this issue, this study was undertaken to evaluate QC/QA operations on 27 active earthwork construction projects in Iowa. As one aspect of the study, dynamic cone penetrometer (DCP) was used to measure the strength/stiffness properties of compacted fill materials, and the compacted layer thicknesses.

The overall goal of this project is assess the current state-of-the-practice of earthwork construction QC/QA practices in the State of Iowa, in reference to the state-of-the-practice by other state agencies, and develop recommendations for better practices.

Field testing indicated that 17 out of the 22 field projects with 20 to 100% of the moisture and density test results outside the QC/QA acceptance limits. This is a problem that should be addressed by improved process control procedures. DCP is one test method that can help address the problem. An approach to set target values for DCP testing in reference to moisture and density is provided in this thesis. The target values can be implemented in-situ, in lieu for moisture and density testing, to rapidly assess problem areas.

CHAPTER 1. INTRODUCTION

This chapter discusses the industry and technical problems addressed in this project. The research goal, specific objectives, and a discussion of the significance of this research are presented in the following discussion. The final section of this chapter forecasts the organization of the thesis.

PROBLEM STATEMENT

Poor quality control during embankment construction has been attributed to premature failures in embankments with slope instability problems and uneven road surfaces, which all lead to traffic safety and road maintenance issues (Jung et al. 2012; White et al. 2004). Currently, the Iowa Department of Transportation (DOT) uses a specification that involves moisture control or moisture and density control as part of the quality control (QC) and quality assurance (QA) process. The current QC/QA specifications are a result of previous Iowa Highway Research Board embankment quality research projects (Bergeson and Jahren 1999; Bergeson et al. 1998; White et al. 2002; White et al. 2007). However, recent field testing by Iowa State University on earthwork construction projects revealed that embankments are frequently constructed outside the moisture and density control limits, even though the QC/QA testing data showed otherwise. To further evaluate this issue, this study was undertaken to evaluate QC/QA operations on multiple earthwork construction projects across Iowa. As one aspect of the study, dynamic cone penetrometer (DCP) was used to measure the strength/stiffness properties of compacted fill materials, and the compacted layer thicknesses. The DCPs measure the penetration resistance of compacted fill layers up to a depth up to 1 m (ASTM 2009). DCP presents a relatively low-cost and rapid in-situ test method that can be used for QC/QA during embankment construction. However, proper guidance on how to develop and implement target values for DCP measurements is not well documented.

GOAL OF THE RESEARCH

The overall goal of this project is to assess the current state-of-the-practice of earthwork construction QC/QA practices in the State of Iowa, in reference to the state-of-the-practice by other state agencies, and develop recommendations for better practices.

OBJECTIVES

The objectives of my research are to:

- Compile the current QC/QA specifications and practices from all 50 state transportation agencies;
- Evaluate Iowa DOT QC/QA practices on embankment construction projects by conducting field studies on multiple project sites across the state;
- Evaluate DCP test procedure to develop recommendations for an alternative QC/QA procedure.

SIGNIFICANCE OF THE RESEARCH

Departments of transportation, contractors, tax payers, and researchers will benefit from my research that evaluates the current state-of-the-practice for embankment construction with detailed field testing over multiple field project sites with wide range of materials.

ORGANIZATION OF THE THESIS

Following this introductory chapter, this thesis is organized into five additional chapters. Chapter 2 reviews previous literature and provides background information for the study. Chapter 3 describes the laboratory and field test methods, and chapter 4 summarizes the laboratory and in situ properties that characterize the tested materials. Chapter 5 presents the results and analyses for the tests performed and discusses these findings. Chapter 6 summarizes the conclusions and outcomes derived from this research, discusses how these conclusions can be applied in construction practice and offers suggestions for future research. Supporting materials are included as appendices that follow the list of works cited.

KEY TERMS

Predicted CBR value, function of CBR, DCP, LWD, quality control, quality assurance, Proctor compaction, and moisture-density compaction energy relationship.

CHAPTER 2. BACKGROUND/LITERATURE REVEIW

This chapter presents a literature review of the embankment compaction procedure and requirements, and several in situ tools can be used for QC and QA during embankment constructions. The material contained in this chapter will describe how this research is related to and builds off of past research.

The literature review covers five main topics: context of project, field implications of compaction, Iowa compaction studies, recent advancements in QC and QA, and state specifications for embankment constructions.

CONTEXT OF THE PROJECT

This research is about an embankment project that was sponsored by Iowa DOT and jointly carried out by the Center for Earthworks Engineering at Iowa State University in 2014. This project aimed at reviewing grading projects statewide and assess the implementation of compaction with moisture control and contractor quality control operations during embankment construction, and provide recommendations. Figure 1 is the geologic ages of the soils map with project locations. The highlight area are the county project located, and the stars represent the specific project sites. Harrison County have not been visited because of the time conflict.

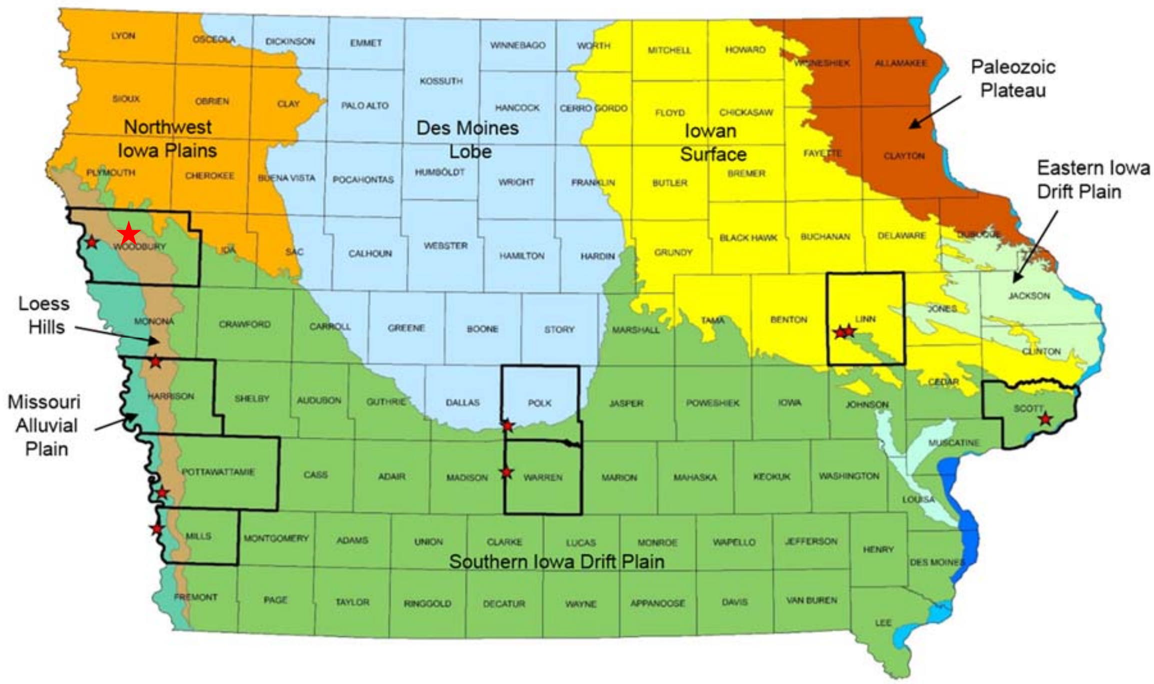


Figure 1. Location of the project sites

FIELD IMPLICATIONS OF COMPACTION

In 1933, R.R. Proctor invented a laboratory test method to determine moisture-density relationship of soils (Proctor 1933), now known as the Proctor Compaction Test. Proctor found that as the compaction energy increased, the dry density of the soil would increase but the optimum moisture content would decrease. During soil compaction, the particles were pushed together and the air voids between each particles were reduced which can create greater density. At a certain point, the minimum void space occurs that the voids within the soil are entirely filled with the water and a small amount of air that cannot be removed by compaction. After the critical point, increasing the moisture even further will result in increasing amount of voids, and thus decreased density.

After Proctor compaction method was developed, most embankment constructions are recommended to use the data from laboratory tests on compacted specimens (Walsh et al.

1997) Walsh et al. pointed out that for economic reasons, although the profession has developed an understanding of relationships between properties and compaction density/water content, it has become routine practice to focus on relative compaction or some combination of precedence rather than desired material properties to establish compaction specifications. So a corresponding spatial variability of relative compaction should be anticipated.

The primary objective of the initial research was provide a comprehensive evaluation of potential problems in compaction control, and address the sources of field variability in relative density. This report (Walsh et al. 1997) indicate that it is not wise to translate experience gained from a specification in a particular region to another region because each region has different soil conditions and compaction processes, he suggest engineers to evaluate the “average” condition of the fill based on projected field compaction results, the field results can necessarily base on appropriate lab data when dealing with unfamiliar soils or specifications.

Based on this and other research, the Iowa DOT conducted research to evaluate their embankment construction specifications.

IOWA COMPACTION STUDIES

Four previous investigations of compaction specifications have been conducted in Iowa. The following sections summarize the findings of these investigations.

A specification for contractor moisture QC testing in roadway embankment construction has been in use for approximately 10 years in Iowa on 190 projects. During these years, Iowa specification have been improved and modified hundreds of times.

ISU researchers have been involved in embankment projects since Phase I in 1997 (Bergeson et al. 1998). Table 1 summarizes the categories and specifications identified in Phase I.

Table 1. QC/QA in embankment specifications

Category	Current Specification
Moisture Control	Current Iowa DOT specification does not require moisture control on the embankment except for subgrade treatments. The specification is similar for 31 other states. The other 19 states required specific moisture on the embankment.
Density Control	Current Iowa DOT specification for density control requires sheepsfoot walkout for achieving adequate compaction of an embankment, a minimum 95 percent of Std. Proctor maximum density is the specification used by the majority of the states.
Lift Thickness	Current Iowa DOT specification for lift thickness is 200 mm (8 in.). This is comparable to the majority of the other DOTs.
Strips	8 DOTs require control strips, Iowa does not.
Foundation Preparation	Current Iowa DOT specification does not require discing or embankment foundations. However, 19 other states do require that the foundation of the embankment be disced or scarified before any embankment is placed, regardless of the embankment height.

Phase I report find out the current methods for evaluating compaction during construction are not adequate. The one-point Proctor test does not adequately identify soil properties or verify field compact, and the "sheepsfoot walkout" method is not a reliable indicator of the degree of compaction for all kinds of soil. Compacting cohesive soils that are placed wet of optimum and near 100% saturation can potentially result in embankments with low shear strength and stability, high pore pressures, and possible slope failures and rough pavements. For cohesionless soil, it is necessary to use vibratory compaction, spot-check with DCP required minimum 90% relative density.

Phase II report (Bergeson and Jahren 1999) mainly aimed at evaluating and developing alternative soil design and embankment construction specification, assess various QC and acceptance procedures with a variety of in situ test methods including DCP, and develop and design rapid field soil identification methods.

Table 2 summarizes the categories and specifications identified in Phase II. Phase II report recommend a flow chart for future QC/QA program in Iowa (Figure 2).

Table 2. Recommended test for different soil type

Soil Type	Lab Test	Field Test
Cohesionless Soils	Grain size distribution; Hydrometer analysis; Standard Proctor; Relative density; Percent finer than the No. 200 sieve; Iowa modified relative density test	DCP index Test; Speedy Moisture; Nuclear density/moisture; Army corps density sampler; Drill rig mounted dynamic cone test
Cohesive Soils	Grain size distribution; Hydrometer analysis; Standard Proctor; moisture content; Unconfined Compressive Strength	observations of fill placement, in-place moisture and density testing, and dynamic cone penetrometer (DCP) index testing

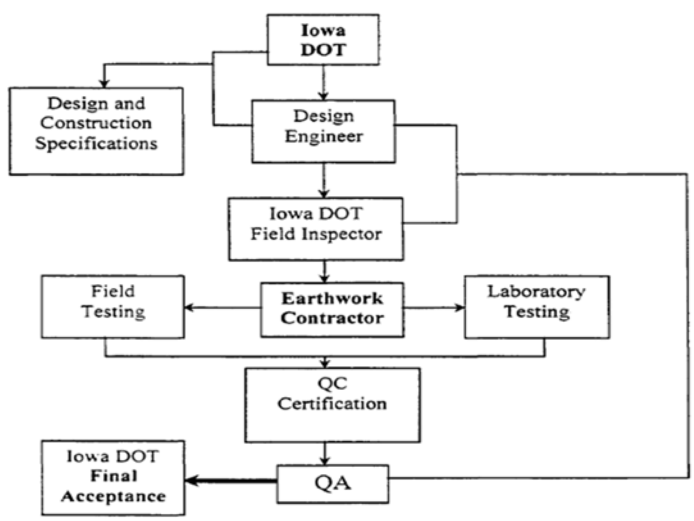


Figure 2. Possible Iowa DOT flow chart for future QC/QA program (Bergeson and Jahren 1999)

The primary objective of (White et al. 2002) report was find out whether the new soil classification system and construction specification improved the embankment quality or not, and develop a quality management-earthwork (QM-E) program which can improve overall embankment quality while balancing the additional cost and time.

Phase III report recommend a minimum QC/QA test frequency as shown in Table 3, and the required stability/strength and uniformity acceptance criteria that are measured by the DCP (Table 4).

Table 3. Minimum QC/QA test frequency (White et al. 2002)

Test	Minimum Test Frequency
Compacted lift thickness Moisture content of compacted fill Density Dynamic cone penetrometer (DCP)	Concurrently every 1,000 m^3 (compacted)
Determination of soil performance classification and moisture control limits	One every 25,000 m^3 or if there is a change in material as determined by the engineer

Table 4. Requirement for mean DCP index indicating stability

Soil Performance Classification		Maximum Mean DCP Index (mm/blow)	Maximum Mean Change in DCP Index (mm/blow)
Cohesive	Select	75	35
	Suitable	85	40
	Unsuitable	95	40
Intergrade	Suitable	45	45
Cohesionless	Select	35	35

The Phase III report recommended the following accepted differences between the contractor QC and Iowa DOT engineer QA tests:

- differences between the contractor's and engineer's moisture content test results will be considered acceptable if moisture content is within 1.0% based on dry weight of soil.

- Differences between the contractor's and engineer's in-place density tests will be considered acceptable if the dry density is within ± 80 kg/m^3.
- Differences between the contractor's and engineer's proctor test results will be considered acceptable if the optimum dry density is within ±80 kg/m^3 and the optimum moisture is within ±1.5% based on dry weight.

The primary objective for (White et al. 2007) report are review of the QC/QA practices of other state departments of transportation (DOTs) and agencies for potential applications in the proposed QM-E program, demonstrate the QM-E program on a full-scale pilot projects in unsuitable soils, and Improve data collection, management, and report generation for QC/QA operations.

The results from Phase IV report shows the data collected from the field indicate DCP index control limits could be set more tightly. ISU developed an acceptable zone of DCP index values (Figure 3) which later was determined using the CBR-DCP index correlation for "all other soils," equation 3, in ASTM D 6951-03 for the moisture control limits specified by the QM-E. Use proposed CBR technique for the creation of soil specific DCP target values can addresses some of the shortcomings of using blanket control limits for broad ranges of soils classifications.

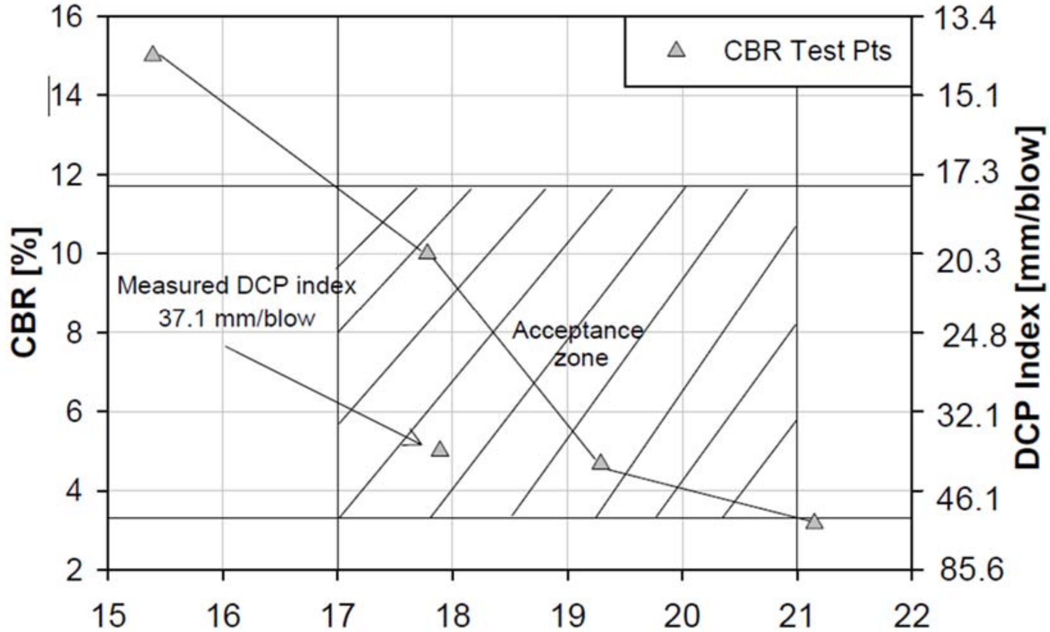


Figure 3. CBR testing results for unsuitable soil sample

RECENT ADVANCEMENTS IN QC AND QA

Non-uniform support conditions can contribute to distresses in pavement layers and cause fatigue cracks at the surface (Jung et al. 2012; White et al. 2004). Traditional QC/QA method are time consuming and sometimes cannot provide very direct and reliable data for the embankment quality. Burnham and Johnson (1993) report recommend DCP can be an effective tool that could provide some information to characterize field subgrade conditions. The objective of this report was to explore ways that DCPs could effectively be used by Minnesota pavement and materials engineers and to perform the testing, analysis, and learning necessary for establishing relationships between DCP test results and other commonly used foundation parameters such as cohesion, friction angle, California Bearing Ratio (CBR), and modulus of subgrade reaction (k).

DCP test can be used at preliminary soil surveys, construction control, structural evaluation of existing pavements, and measuring the frost/thaw depth in cold climate pavements during the spring months.

Larsen focuses on the application of that construction specification to a pilot project in high plasticity clay soils. This report (Larsen et al. 2007)is one of the few documented cases of applying a strength based earthwork quality control procedure for cohesive fill.

This report introduced a method that was developed to create a new generation of DCP control limits. This method utilizes CBR testing, conducted across a range of moisture contents to develop a DCP index acceptance zone. This method has considerable potential because if it were successfully implemented it could eliminate the need to include density testing in the QM-E pilot specification.

DCP tests are recommended in the pavement design guide AASHTO (2008) to estimate the parameters like CBR, elastic modulus from empirical relationship. Minnesota DOT have DCP for density control during embankment construction in supplemental specification (Mn/DOT 2014), Table 5 provide the maximum allowable penetration for DCP, grading number determined by Form G&B-203.

Table 5. Maximum allowable penetration for DCP

Grading Number	Moisture Content (percent of dry weight)	Maximum Allowable DPI, mm/blow	Maximum Allowable Seat, mm
3.1 – 3.5	< 5.0	10	No Requirement
	5.0 – 8.0	12	
	> 8.0	16	
3.6 – 4.0	< 5.0	10	
	5.0 – 8.0	15	
	> 8.0	19	
4.1 – 4.5	< 5.0	13	
	5.0 – 8.0	17	
	> 8.0	21	
4.6 – 5.0	< 5.0	15	
	5.0 – 8.0	19	
	> 8.0	23	
5.1 – 5.5	< 5.0	17	
	5.0 – 8.0	21	
	> 8.0	25	
5.6 – 6.0	< 5.0	19	
	5.0 – 8.0	24	
	> 8.0	28	

In field, the soil strength will be determined by the DCP in accordance with ITM 509 and the moisture content will be determined in accordance with ITM 506. In lab, the DCP criteria will be established on representative soils by performing ASTM D 1140, AASHTO T 88, AASHTO T 89, AASHTO T 90, and AASHTO T 99 using Method A for soils and Method C for granular materials. Table 6 shows that Indiana DOT have DCP in their standard specification for compaction control (Indiana/DOT 2016)

Table 6. Compaction control using DCP blow counts

Textural Classification	**Maximum Dry Density (pcf)**	**Optimum Moisture Content Range (%)**	**Acceptable Minimum DCP value for 6 in.**	**Acceptable Minimum DCP value for 12 in.**
CLAY SOILS				
Clay	< 105	19 - 24	6	-
Clay	105 - 110	16 - 18	7	
Clay	111 - 114	14 - 15	8	
SILTY SOILS				
Silty	115 - 116	13 - 14	-	9
Silty	117 - 120			11
SANDY SOILS				
Sandy	121 - 125	8 - 12	-	12
Sandy	> 125			15
GRANULAR SOILS - STRUCTURE BACKFILL AND A-1, A-2, A-3 SOILS				
No. 30	-	-	-	6
No. 4				7
1/2 in.				11
1 in.				16

Minnesota DOT have DCP and LWD in provisional specification (Siekmeier et al. 2009), from this report, standard Proctor test is not the only parameter to determine optimum moisture content. Table 7 demonstrates that from DCP and LWD target values, the plastic limit and optimum moisture content can be estimated.

Table 7. Target DPI and LWD deflection values for fine grained soils (Siekmeier et al. 2009)

Plastic Limit	**Estimated Optimum Moisture**	**Field Moisture as a Percent of Optimum Moisture**	**DCP Target DPI at Field Moisture**	**Zorn Deflection Target at Field Moisture minimum**	**Zorn Deflection Target at Field Moisture maximum**
[%]	[%]	[%]	[mm/drop]	[mm]	[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

Table 7. Continued

Plastic Limit	Estimated Optimum Moisture	Field Moisture as a Percent of Optimum Moisture	DCP Target DPI at Field Moisture	Zorn Deflection Target at Field Moisture minimum	Zorn Deflection Target at Field Moisture maximum
[%]	[%]	[%]	[mm/drop]	[mm]	[mm]
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

These target values can be used for quality assurance of unbound materials during pavement foundation construction with minimal verification at specific project locations. The target DPI value is changeable and involving with different numbers of seating drops and measurement drops. This report recommend to use three seating drops and five to ten measurement drops. Table 8 provides DPI and LWD target values according to a material's grading number and moisture content.

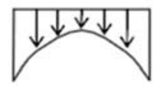
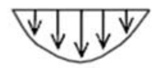
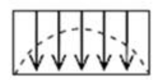
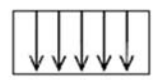
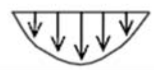
Table 8. DCP and LWD target values for granular materials (Siekmeier et al. 2009)

Grading Number	Moisture Content	Target DPI	Target DPI Modulus CSIR	Target LWD Modulus Dynatest	Target LWD Modulus Zorn	Target LWD Deflection Zorn
GN	%	mm/drop	MPa	MPa	MPa	mm
3.1-3.5	5 - 7	10	97	120	80	0.38
	7 - 9	12	80	100	67	0.45
	9 - 11	16	59	75	50	0.63
3.6-4.0	5 - 7	10	97	120	80	0.38
	7 - 9	15	63	80	53	0.56
	9 - 11	19	49	63	42	0.71
4.1-4.5	5 - 7	13	73	92	62	0.49
	7 - 9	17	55	71	47	0.64
	9 - 11	21	44	57	38	0.79
4.6-5.0	5 - 7	15	63	80	53	0.56
	7 - 9	19	49	63	42	0.71
	9 - 11	23	40	52	35	0.86
5.1-5.5	5 - 7	17	55	71	47	0.64
	7 - 9	21	44	57	38	0.79
	9 - 11	25	37	48	32	0.94
5.6-6.0	5 - 7	19	49	63	42	0.71
	7 - 9	24	38	50	33	0.90
	9 - 11	28	32	43	29	1.05

All the LWD devices calculate elastic modulus from a measured contact stress and peak deflection of the loading plate or soil directly under the plate based on elastic half-space theory and the assumption of stress (Vennapusa and White 2009). For the granular materials tested, the Zorn E_{LWD} increases with increasing plate contact stresses with stiffer material presenting a greater increase in E_{LWD}.

The stress distribution under a plate depends on both plate type and soil type (Terzaghi and Peck 1967), the summary of shape factor are present in Table 9.

Table 9. Summary of shape factors in E_{LWD} estimation (Vennapusa and White 2009)

Plate type	Soil type	Stress distribution (shape)		Shape factor (f)
Rigid	Clay (elastic material)	Inverse Parabolic		$\pi/2$
Rigid	Cohesionless sand	Parabolic		8/3
Rigid	Material with intermediate characteristics	Inverse Parabolic to Uniform		$\pi/2$ to 2
Flexible	Clay (elastic material)	Uniform		2
Flexible	Cohesionless Sand	Parabolic		8/3

Effect of Parameters on CBR

The most important parameter to evaluate subgrade/subbase strength for the pavement design is the CBR value. Miscellaneous laboratory tests report U.S. Army Corps of Engineers (1950) presents several special factors like moisture content, dry density, mold size and soil type for CBR penetration test. The major finding are

- The diameter of the mold generally affects the CBR, and the effect is more pronounced on cohesionless soils and soils with low plasticity.
- The 6 in. diameter mold are closer to the value attained from field in place tests than for lager diameter molds.
- Materials which contains gravel larger than ¾” should be removed and replacing with equal percentages by weight of sizes ¾” to 3/8” and 3/8” to No.4 sieve.

- Cohesionless sand should be compacted by modified AASHTO effort in saturated status to obtain a satisfactory density.

CBR value of uniform soils having similar characteristics can be determined by DCP results (Gill et al. 2010). The variation in CBR value under different conditions has been expressed by a dimensionless term California bearing ratio index (CBRI) (Choudhary et al. 2010)

$$CBRI_1 = {}^{CBR_{LS}}/_{CBR_{DCPS}} \tag{1}$$

$$CBRI_2 = {}^{CBR_{DCP}}/_{CBR_{DCPS}} \tag{2}$$

where,

CBR_{LS} = laboratory soaked CBR value at in situ density

CBR_{DCP} = DCP based in situ CBR value at field moisture and in situ density

CBR_{DCPS} = DCP based in situ CBR value under soaked condition

Figure 4 describes the variation of $CBRI_1$ and $CBRI_2$ with compaction level. And the linear equation are given as follows:

$$CBRI_1 = 0.0007(compaction\ level) + 1.4646 \tag{3}$$

$$CBRI_2 = -0.0015(compaction\ level) + 2.1465 \tag{4}$$

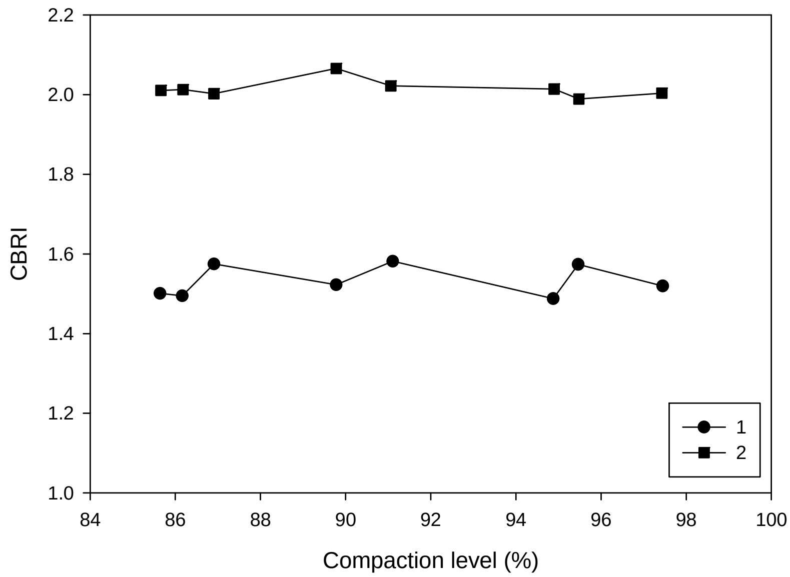


Figure 4. Variation of CBRI vs. compaction level (Bandyopadhyay and Bhattacharjee 2010)

Field CBR test is costly and not always cost effective for pavement evaluation before or after construction. The field CBR values at in situ conditions lies in between unsoaked and four days soaked values from DCP results (Bandyopadhyay and Bhattacharjee 2010).

Iowa SUDAS manual indicates that a subgrade generally requiring a CBR of 10 or greater is considered good and can support heavy loading without excessive deformation. Table 10 provide the range of CBR value for different soils. Relative ratings of support conditions based for CBR values for subbase and subgrade layers per (SUDAS 2015) is provided in Table 11.

Table 10. Typical CBR value for various soils (SUDAS 2015)

Material Description	CBR
SC: clayey sand	10-20
CL: lean clays, sandy clays, gravelly clays	5-15
ML: silts, sandy silts	5-15
OL: organic silts, lean organic clays	4-8
CH: fat clays	3-5
MH: plastic silts	4-8
OH: fat organic clays	3-5

Table 11. Relative ratings of subbase and subgrade layers based on CBR values (SUDAS 2015)

CBR (%)	Layer	Rating
> 80	Subbase	Excellent (E)
50 to 80	Subbase	Very Good (VG)
30 to 50	Subbase	Good (G)
20 to 30	Subgrade	Very Good (VG)
10 to 20	Subgrade	Fair to Good (F to G)
5 to 10	Subgrade	Poor to Fair (P to F)
< 5	Subgrade	Very Poor (VP)

STATE SPECIFICATIONS FOR EMBANKMENT CONSTRUCTION

I located the most current standard and supplemental specifications from the websites of all 50 state departments of transportation. I downloaded the documents in pdf format and created two Excel spreadsheets, one for granular and another for non-granular materials to track information about the specifications I consulted and specifications for embankment construction including equipment; gradation; placement of embankment materials and compaction method; disk and compaction passes; lift thickness; moisture content (w); and dry density (γ_d).

For granular materials, the most common requirements is moisture and density control, which 21 states require. The second most frequently used is density control only, which 15 states require. Only one states requires moisture control only; six states require multiple moisture and density control depends on compaction method; two states require moisture or moisture and density control depending on the project. The other five states do not have requirements. 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 multiple moisture and density control depending on the compaction method; the other two states require moisture or moisture and density control depending on project. Figure 5 and Figure 6 show the geographic location of those states have different QC/QA requirements for granular and non-granular materials. Figure 7 shows the number of states which have different QC/QA requirements for granular and non-granular materials. The specific summaries for state specification are located in the appendix.

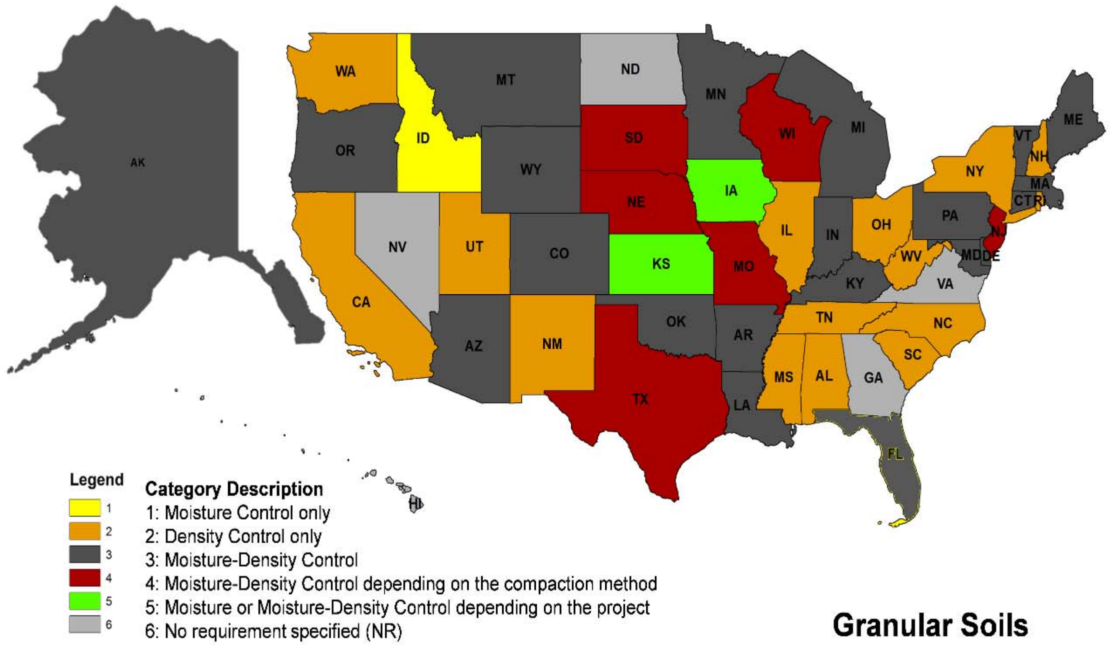


Figure 5. QC/QA requirements for granular materials in U.S

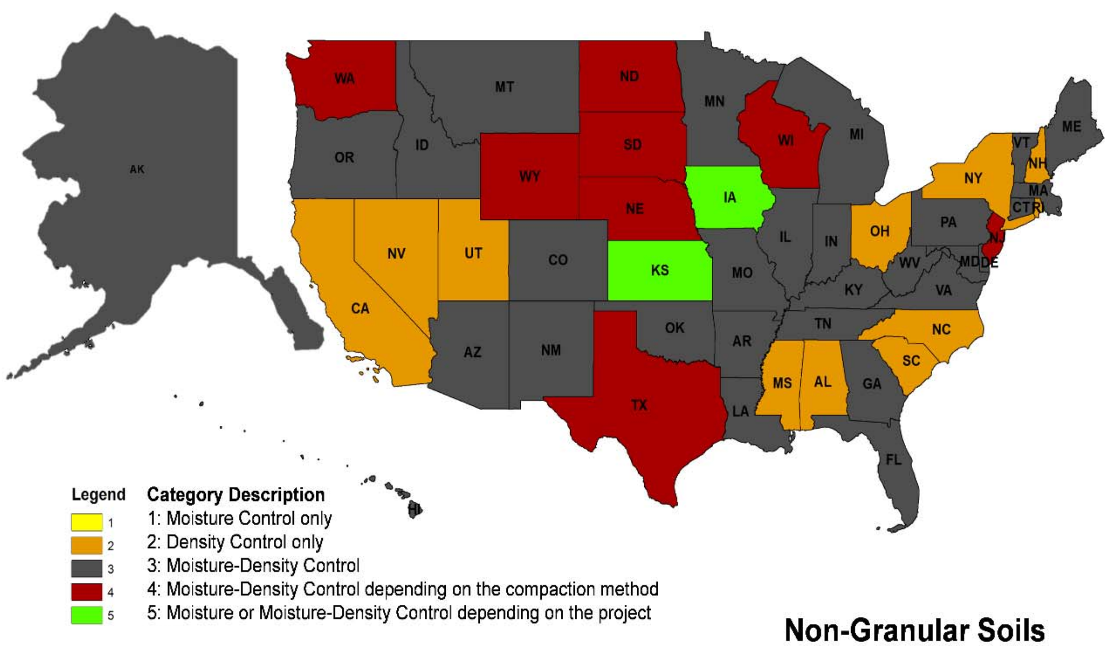


Figure 6. QC/QA requirements for non-granular materials in U.S

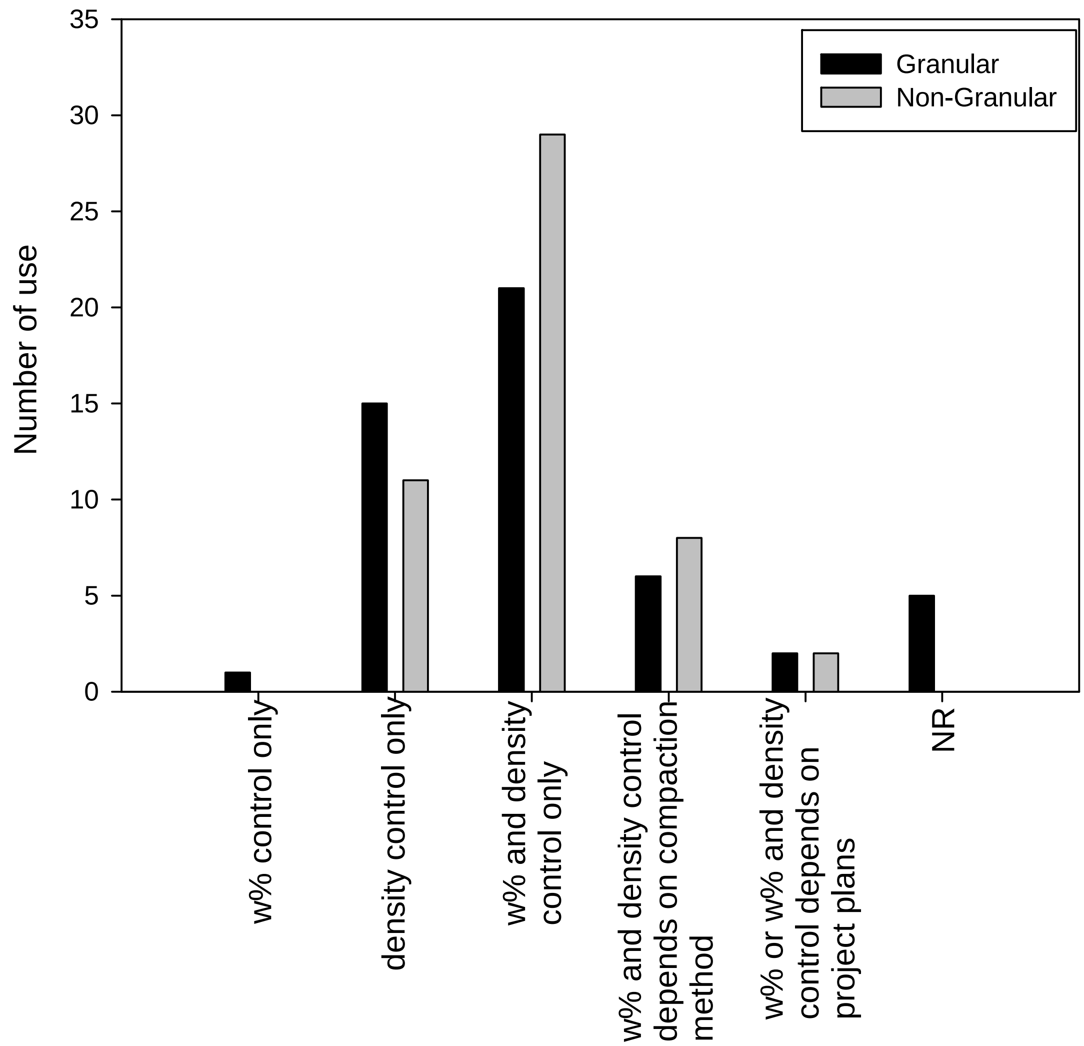


Figure 7. Number of different use of QC/QA requirements for granular and non-granular materials in U.S

Moisture Control Requirements

The current Iowa DOT specifications require ≤ +/-2% of w_{opt} for both granular and non-granular materials. No requirement specified is the most common moisture control method for granular materials, and ≤ +/-2% of w_{opt} and NR are the most common moisture control method in U.S for non-granular materials. The specifications include a wide range of required moisture contents. Obviously, there is not a consistent philosophy as to what moisture content provides the best compaction and stability. Some states like Minnesota use relative moisture content inside of using relationship with w_{opt}. Some states like Iowa and Indiana have very specify moisture range of $w_{opt.}$ Some states like Missouri have no moisture content requirements. Table 12 and Table 13 show the different moisture content requirements for granular and non-granular materials of all 50 state DOTs

Table 12. Moisture control for granular materials

Moisture Control (w)	Number of states
NR	18
≤ +/-2% of w_{opt}	7
Base on plans or approve by engineer	5
At or near w_{opt}	4
Adjust to meet specify density	3
-4% to +2% of w_{opt}	3
≤ +/-2% of w_{opt} or 0% to +3% of w_{opt} depends on soil gradation	1
-4% to w_{opt}	1
Suitable	1
≤ +/-5% of w_{opt}	1
-2% to +1% of w_{opt} or -3% to w_{opt} depends on soil type	1
≤ +/-4% of w_{opt} or -4% to +6% of w_{opt} or adjust to meet specify density depends on compaction method	1
≤ +2% of w_{opt} and more than the moisture content will cause unstable	1
Relative moisture content with density requirements	1
Adjust to meet specify density, show in plans or NR depends on compaction method	1
-3% to w_{opt}	1

Note: NR = no requirements specified

Table 13. Moisture control for non-granular materials

Moisture Control (w)	Number of states
≤ +/-2% of w_{opt}	8
NR	8
At or near w_{opt}	5
Suitable	3
Adjust to meet specify density	2
-4% to +2% of w_{opt}	2
Approved by contractor or engineer	2

Table 13. Continued

Moisture Control (w)	Number of states
≤ +3% of w_{opt} or ≤ w_{opt}	2
≤ +3% of w_{opt}	1
≤ +/-5% of w_{opt}	1
-2% to +1% of w_{opt}, silt or loess material from -3% to w_{opt}	1
120% of w_{opt} for top 2 ft	1
≤ +/-2% of wopt, AASHTO T180	1
≤ +/-2% of w_{opt} or 0% to +3% of w_{opt} depends on soil gradation	1
Relative moisture content with density requirements	1
Adjust to meet specify density or NR	1
-5% to w_{opt} or w_{opt} to +4% depends on compaction method	1
≤+/-2% of w_{opt} or -4 to 0% of w_{opt} depends on compaction method	1
w_{opt} to +5% or -4% to +5% of w_{opt}, or NR	1
-3% to w_{opt}	1
≤ +/-4% of w_{opt} or -4% to +6% of w_{opt} or adjust to meet specify density depends on compaction method	1
-4% to +3% of w_{opt}	1
≤ +2% of w_{opt} and more than the moisture content will cause unstable	1
≤ +3% of w_{opt} or NR depends on compaction method	1
-4% to +2% of w_{opt} or NR	1
depends on PI or compaction equipment	1

Note: NR = no requirements specified

Density Control

The current Iowa DOT specification requires to compact the first layer no less than 90% of maximum density, then compact each succeeding layer to no less than 95% of maximum density based on AASHTO T99 for both granular and non-granular materials. As can been see from Table 14 and Table 15, a minimum of 95% of standard Proctor maximum density is the specification used by the majority of the states for both granular and non-granular materials.

Many states utilize different density specifications for different embankment layers. For example, Illinois requires if embankment ≤ 1.5 ft (450 mm), all lifts ≥ 95 % of the standard laboratory density. If the embankment height is between 1.5 ft and 3 ft (450 mm and 900 mm) inclusive, the first lift ≥ 90 %, and the balance to a minimum of 95 % of the standard laboratory density. If ≥ 3 ft (900 mm), the lower 1/3 of the embankment, but not to exceed the lower 2 ft (600 mm), shall be compacted in a manner that will yield a minimum of 90 % of standard laboratory density to the uppermost lift of that portion of the embankment. The next 1 ft (300 mm) ≥ 93 %, and the balance ≥ 95 % of the standard laboratory density.

Table 14. Density control for granular materials

Density Control (γ_d)	Number of States
≥ 95% of maximum γ_d, AASHTO T99	19
NR	6
Specify by equipment or pass numbers	4
95% of maximum γ_d or specify by compaction equipment depends on compaction method	2
Specify by plans or approved by engineer	2
≥ 90% of maximum γ_d, AASHTO T99	1
≥ 100% of maximum γ_d, AASHTO T99	1
≥ 90% of maximum γ_d, AASHTO T180	1
≥ 95% of maximum γ_d, AASHTO T180	1
Top 1 ft ≥ 97% of maximum γ_d, rest ≥ 92% of maximum γ_d	1
Top 1 ft ≥ 100% of maximum γ_d, rest ≥ 97% of maximum γ_d	1
Top 2 ft ≥ 95% of maximum γ_d, rest ≥ 90% of maximum γ_d	1
Top 3 ft ≥ 95% of maximum γ_d, rest ≥ 90% of maximum γ_d	1
Top 6 ft ≥ 95% of maximum γ_d, rest ≥ 90% of maximum γ_d, or no further consolidation	1
≥ 95% of maximum γ_d, rest ≥ 90% of maximum γ_d	1
First embankment layer ≥ 90% of maximum γ_d, succeed layer ≥ 95% of maximum γ_d	1
95% or 100% of maximum γ_d based on relative moisture content	1
basement soil 95% of maximum γ_d, design soil 98% of maximum γ_d	1
96% of maximum γ_d and no single point shall less than 92% of maximum γ_d	1
90% of maximum γ_d or approved by engineer depends on compaction method	1
98% of maximum γ_d or no further consolidation depends on compaction method	1
Follow plan or NR depends on compaction method	1

Note: NR = no requirements specified

Table 15. Density control standards used by states for non-granular materials

Density Control (γ_d)	Number of States
$\geq$ 95% of maximum γ_d, AASHTO T99	22
$\geq$ 90% of maximum γ_d, AASHTO T99	2
$\geq$ 95% of maximum γ_d, AASHTO T180	2
95% of maximum γ_d or specified by compaction equipment depends on compaction method	2
90% or 95% of maximum γ_d or specify by compaction equipment depends on compaction method	2
$\geq$ 100% of maximum γ_d, AASHTO T99	1
$\geq$ 90% of maximum γ_d, AASHTO T180	1
$\geq$ 95% of maximum γ_d, rest $\geq$ 90% of maximum γ_d	1
Top 6 in. $\geq$ 95% of maximum γ_d, rest $\geq$ 90% of maximum γ_d,	1
Top 6 in. $\geq$ 100% of maximum γ_d, rest $\geq$ 95% of maximum γ_d,	1
Top 6 in. $\geq$ 95% of maximum γ_d, rest $\geq$ 90% of maximum γ_d, or no further consolidation	1
Top 12 in. $\geq$ 100% of maximum γ_d, rest $\geq$ 95% of maximum γ_d	1
Top 12 in. $\geq$ 97% of maximum γ_d, rest $\geq$ 92% of maximum γ_d, AASHTO T180	1
Top 12 in. $\geq$ 100% of maximum γ_d, rest $\geq$ 97% of maximum γ_d,	1
Top 3 ft $\geq$ 95% of maximum γ_d, rest $\geq$ 90% of γ_d,	1
First embankment layer $\geq$ 90% of maximum γ_d, succeed layer $\geq$ 95% of maximum γ_d	1
95% or 100% of maximum γ_d based on relative moisture content	1
basement soil 95% of maximum γ_d, design soil 98% of maximum γ_d	1
98% or 100% or 102% of maximum γ_d	1
96% of maximum γ_d and no single point shall less than 92% of maximum γ_d	1
Specify by plans or approved by engineer	1
Specify by plans or NR	1
$\geq$ 95% of maximum γ_d for embankment $\leq$ 1.5ft, first lift $\geq$ 90%, rest $\geq$ 95% of maximum γ_d for embankment between 1.5 ft and 3 ft. lower 1/3 or 2ft $\geq$ 90%, next 1 ft $\geq$ 93%, rest $\geq$ 95% of maximum γ_d for embankment $\geq$ 3 ft	1
$\geq$ 102% of maximum γ_d for material have maximum γ_d from 90 to 104.9 pcf, $\geq$ 100% of maximum γ_d for material have maximum γ_d from 105 to 119.9 pcf, $\geq$ 98% of maximum γ_d for material have maximum γ_d over 120 pcf	1
$\geq$ 98% of maximum γ_d for PI $\leq$ 15, $\geq$ 98% and $\leq$ 102% of maximum γ_d for15 < PI $\leq$ 35, $\geq$ 95% and $\leq$ 100% of maximum γ_d for PI > 35	1

Lift Thickness

The current Iowa DOT specification for lift thickness is 8 in. (200 mm) for non-granular materials which is comparable to the majority of the other DOT's. Iowa DOT specification have maximum 4 ft. lift thickness requirements for granular/rock materials. The most common used lift thickness requirements is less than 2 ft. for granular materials (especially for rock embankments). Table 16 and Table 17 show the different maximum lift thickness for the DOT's throughout the United States.

Table 16. Maximum lift thickness for granular materials

Maximum lift thickness	Number of states
2 ft	13
3 ft	10
8 in.	4
12 in.	4
6 in.	3
15 in.	3
18 in.	3
4 ft	2
NR	1
8 to 12 in.	1
8 or 12 in.	1
8 in. or 3 ft	1
6 in., except engineer approve 8 in.	1
8 in., up to 2 ft for rock embankment exceed 5 ft	1
Smaller than largest rock	1
Approved by engineer	1

Note: NR = no requirements specified
Lift thickness are loess measurement unless specified

Table 17. Maximum lift thickness for non-granular materials

Maximum lift thickness	Number of states
8 in.	27
12 in.	7
10 in.	3
8 in. or 12 in. depends on compaction method	3
9 in.	2

Table 17. Continued

Maximum lift thickness	Number of states
4 in. compacted thickness	1
12 in. compacted thickness	1
6 in. or 12 in. depends on soil type	1
8 in. or 16 in. depends on compaction method	1
4 in., 6 in. or 8 in. depends on compaction method	1
8 in., 12 in. or NR depends on compaction method	1
4 in., 18 in. or 2ft depends on compaction method	1
Not exceed equipment allowance	1

Note: lift thickness are loess measurement unless specified

Disk/Compaction Passes

For granular materials, 14 out of 50 states give specific procedure and rules to help control the embankment quality. For example, Indiana separate the compaction passes by two different soil type. It requires minimum of three passes with the static roller and minimum of 2 passes with the vibratory roller for shale and/or soft rock embankment and minimum of 6 passes with static roller and minimum of two passes with vibratory tamping-foot roller for shale and thinly layered limestone.

For non-granular materials, 13 out of 50 states give specific procedure and rules to help control the embankment quality. For example, Minnesota have specific requirements of soil size while disk and the roller speed during compaction passes. It requires to disk soils with greater than 20 percent passing the No. 200 [75 µm] sieve, and 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.

CHAPTER 3. METHODS

To evaluate the in situ soil compaction properties during earthwork construction, several field and laboratory tests were conducted in this study. Field tests were conducted on 27 test beds at 9 different project sites in Iowa. Samples obtained from field were transported to laboratory for additional tests.

The purpose of this chapter is to describe the methods used to conduct the various tests. Applicable American Society for Testing and Materials (ASTM) standards were followed in conducting the tests and calculating the required parameters. A brief description of the test methods and any deviations from the standard are provided below for each test method.

FIELD TESTS

Table 18 summarizes the field tests and the applicable ASTM standard test methods followed in this study.

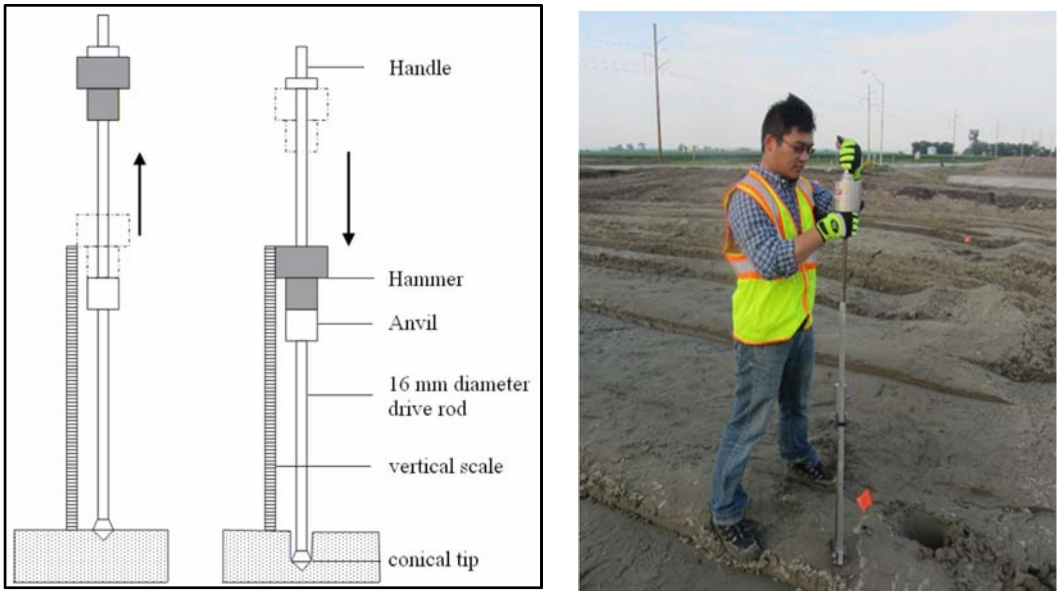
Table 18. Field test standards

Field Test	Test Standard
Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method	ASTM D2937-10
Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications	ASTM D6951-03
Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD)	ASTM E2583-11

Dynamic Cone Penetrometer (DCP)

DCP tests were conducted by driving a 20 mm diameter, 60° disposable cone into the ground, using a 17.6 lb (8 kg) hammer raised and dropped from a height of 22.6 in. (575 mm). Penetration depth for given numbers of blows were measured to determine the

DCP penetration resistance (PR) values in units of mm/blow. A schematic and a picture of the DCP device are shown in Figure 8.



(a) (b)

Figure 8. (a) Schematic of dynamic cone penetrometer (Larsen et al. 2007) and (b) in situ DCP testing

The PR is empirically correlated with California Bearing Ratio (CBR) using the following relationships in accordance with ASTM D6951-03 (ASTM 2003).

for CH soils $$CBR = \frac{1}{0.002871\,(\mathrm{PR})} \tag{5}$$

for CL soils for CBR<10 $$CBR = \frac{1}{(0.017019\,(\mathrm{PR}))^2} \tag{6}$$

for all other soils $$\mathrm{CBR} = \frac{292}{(PR)^{1.12}} \tag{7}$$

Averages of CBR and PR of the top 8 in. and the top 12 in. were selected to represent the properties of the compacted fill layers as shown in Figure 9.

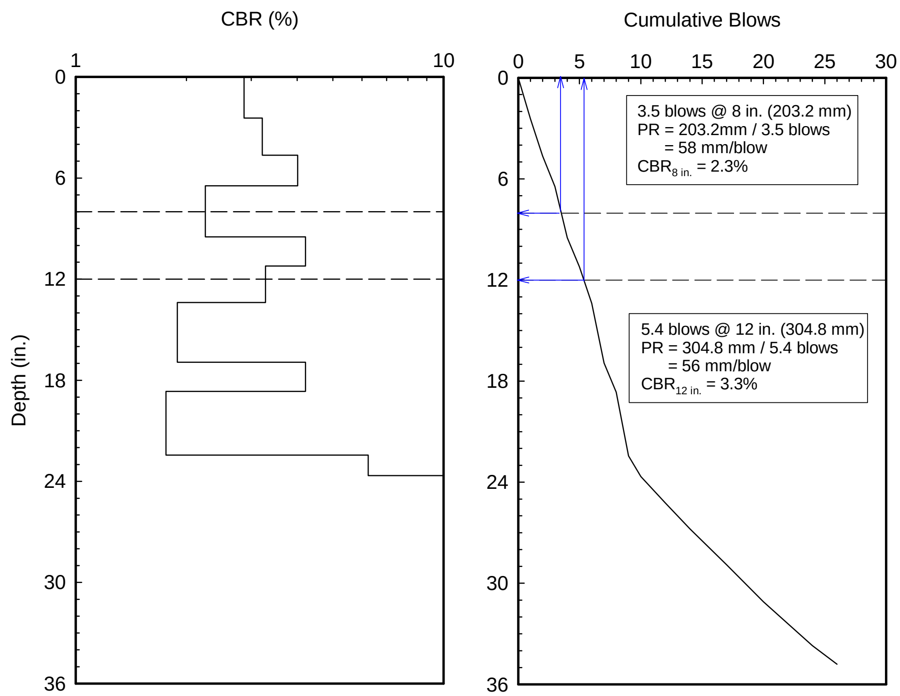


Figure 9. DCP-CBR profile at Linn County #2 project TB1 point 1

Light Weight Deflectometer (LWD)

LWD tests were conducted in accordance with the ASTM E2583-11 (ASTM 2011) test method. A Zorn LWD setup with a 12 in. (304.8 mm) diameter plate, a 22 lb (10 kg) drop weight, and a drop height of 22.3 in. (720 mm) was used in this study (Figure 10). Test was conducted by performing three seating drops followed by three measurement drops. The average deflection of the three measurement drops was used for calculating elastic modulus using Equation 4.

$$E_{LWD} = \frac{(1-v^2)\sigma_0 a}{d_0} \times f \tag{8}$$

where:

E_{LWD} = elastic modulus (MPa);

v = Poisson's ratio (assumed as 0.4);

σ_0 = applied stress (MPa);

a = radius of the plate (mm);

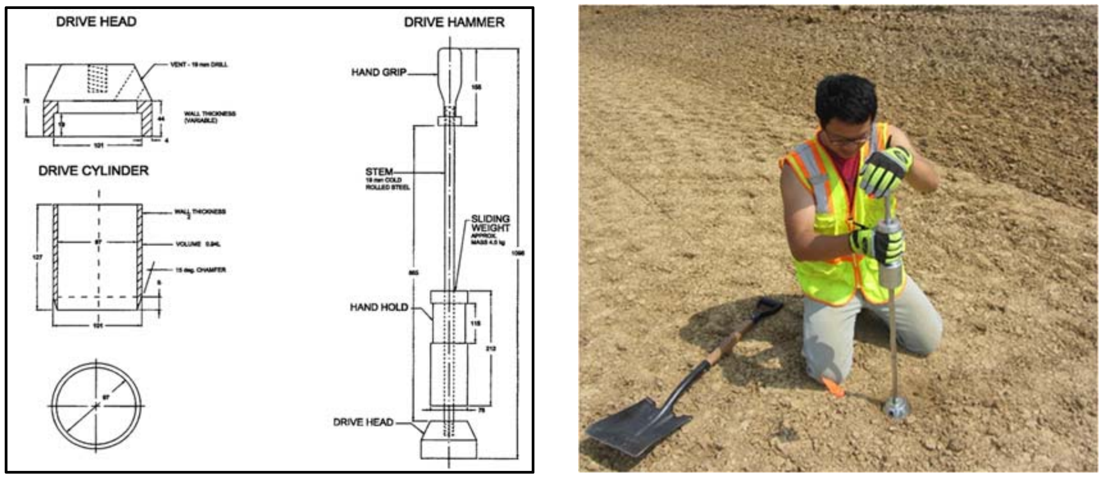
d_0 = measured peak deflection (mm); and

f = shape factor assumed as $\pi/2$, because of inverse parabolic stress distribution expected on cohesive materials with a rigid plate (Vennapusa and White 2009).

Figure 10. Light weight deflectometer test

Drive Cylinder

ASTM D2937-10 (ASTM 2010) drive cylinder test method was used to determine in situ moisture and dry density. Thin wall 4 in. diameter cylinder with a driving head were driven into the soil to obtain relatively undisturbed samples. The cylinders were then carefully excavated from the soil, sealed in a plastic bag, and placed in a humid cooler and transported back to the laboratory. The samples were processed in the laboratory to determine unit weight and moisture.



(a) (b)

Figure 11. (a) Schematic of drive cylinder test (ASTM 2010) and (b) picture of drive cylinder test in situ

Global Position 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 measurement by connecting to Iowa real-time network (RTN) stations (Figure 12).

Figure 12. GPS measured test point locations

LABORATORY TESTS

Table 19 summarizes the laboratory tests conducted in this study in accordance with applicable ASTM standards.

Table 19. Laboratory test standards

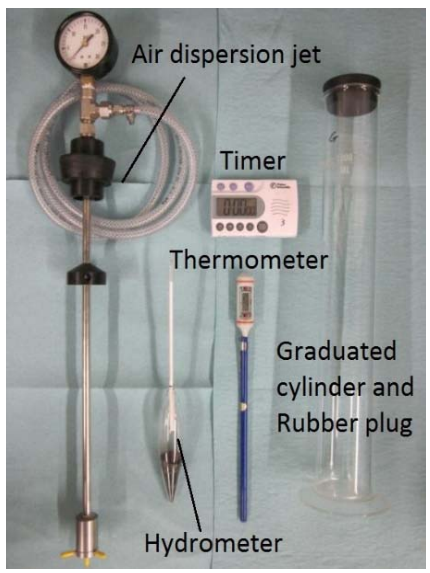
Laboratory Test	Test Standard
Standard Test Method for Particle-Size Analysis of Soils	ASTM D422-63
Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft3(600 kN-m/m3))	ASTM D698-07
Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer	ASTM D854-10
Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3))	ASTM D1557-02
Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils	ASTM D1883-05
Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)	ASTM D2487-06
Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	ASTM D3282-09
Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	ASTM D4318-10

Particle Size Analysis

Particle-size analysis was conducted in accordance with ASTM D422-63 (ASTM 2007). In preparing the soil samples, representative bulk samples obtained from field were air dried in room temperature and processed through a rapid soil processor to pulverize the soil (Figure 13a). The pulverized and air-dried soil was then carefully mixed and divided using a splitter multiple times in accordance with ASTM D422-63, to obtain representative samples for fine and coarse sieve analysis (Figure 13b) and hydrometer test (Figure 13c). An air dispersion jet was used in this study to disperse the fine particles in the hydrometer.

(a)

(b)



(c)

Figure 13. (a) Rapid soil processor (b) Sieve shaker and (c) Hydrometer devices

Atterberg Limits Test

Atterberg limits tests were performed according to ASTM D4318-10 (ASTM 2010) to determine a soil's liquid limit (LL), plastic limit (PL), and plasticity index (PI). Dry preparation method was used to prepare the samples. LL tests were performed using the multipoint method with at least three points for each material.

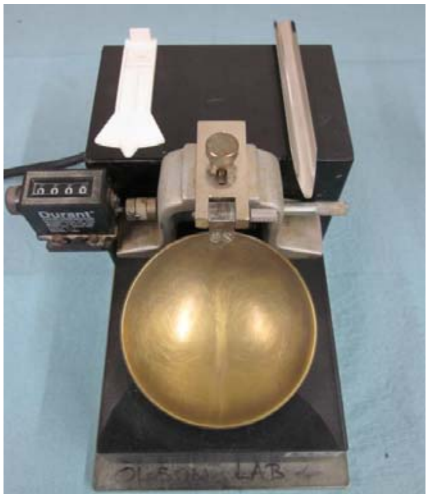


Figure 14. Atterberg limits test

Soil Classification

The particle-size analysis test results and Atterberg limits test results were used to classify materials in accordance with ASTM D2487-06 (ASTM 2006) Unified Soil Classification System (USCS) and ASTM D3282-09 (ASTM 2009) AASHTO Soil Classification System.

Specific Gravity

ASTM D854-10 (ASTM 2010) test method was used to determine the specific gravity of embankment materials. The sample passing the No. 4 sieve (4.75 mm) was used, and tests were conducted on moist-specimen in accordance with method A of the test standard.

Proctor Compaction Tests

The moisture-dry unit weight relationships of embankment materials were determined in accordance with ASTM D698-07 (ASTM 2007) and ASTM D1557-02 (ASTM 2002). Base on the grain size distribution of the soils, method A was applicable for all the materials. A calibrated mechanical hammer was used to conduct the Proctor tests (Figure 15). The tests were performed for a minimum of five different moisture contents and the optimum moisture-density characteristics were obtained based upon Li and Sego fit parameter curves (Li and Sego 1999, 2000a, and 2000b) that were fit to the data. Equation 9 shows the relationship and relevant parameters.

Figure 15. Mechanical Proctor setup

$$\gamma_d\ (w) = \frac{G_s\,\gamma_w}{(1+\frac{w\,G_s}{S_m - S_m\,(\frac{w_m - w}{w_m})^{n+1}\,(\frac{w_m^n + p^n}{(w_m - w) + p^n})})} \qquad (10)$$

where

S_m = Maximum saturation

n = shape factor

p = the parameter which influences the width of the upper portion of the curve

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; and

w_m = moisture content at S_m;

S_m is the maximum degree of saturation which can be determined from the points on the wet side of the compaction curve that are parallel to the zero air void curve (ZAVC) (Figure 16). S_m usually remains constant and does not change as the compaction effort changes, and W_m is the water content associated with the S_m. The boundary on the dry side of optimum is the dry density (γ_{dd}). The parameters n and p are parameters which determine the shape and width of the compaction curve. When n is increased, the dome of the curve becomes sharper, and when n is decreased the curve tends to flatten. The parameter p influences the width of the upper portion of the curve without changing its shape factor (n) and boundary conditions (S_m and γ_{dd}). To obtain the best fit curve for the Proctor test points, 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 value is achieved.

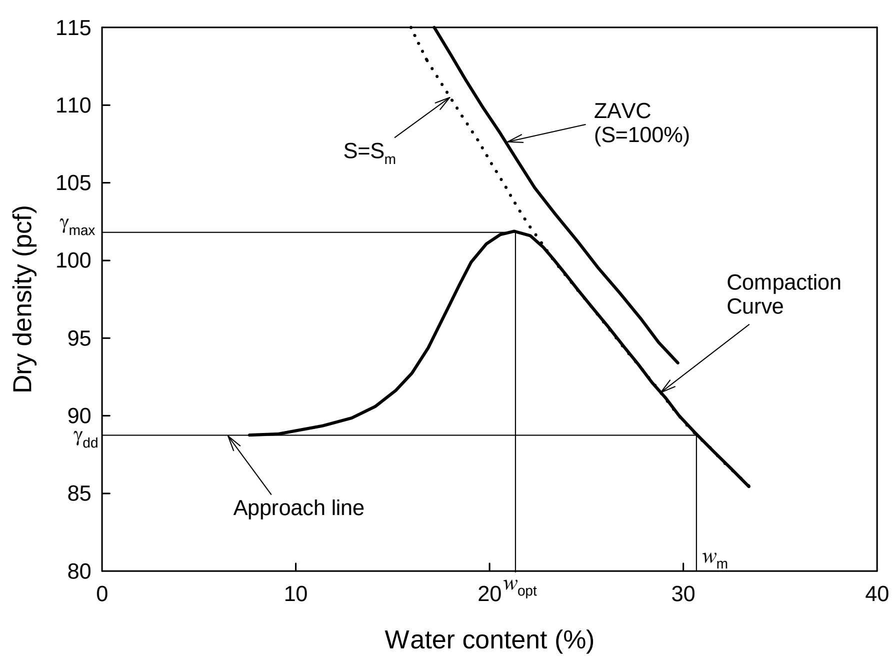


Figure 16. Density curve (reproduced from Li and Sego 1999)

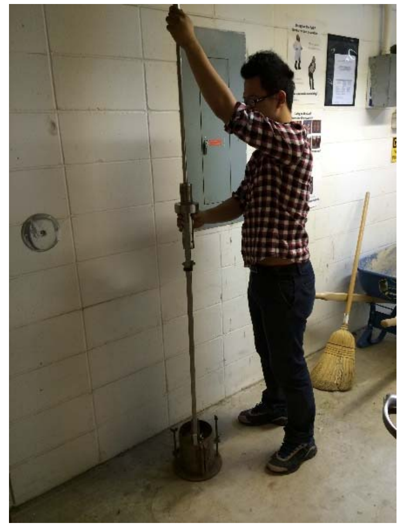
California Bearing Ratio Test

The California bearing ratio (CBR) of laboratory compacted soils were determined in accordance with ASTM D1883 (ASTM 2005). This test method is primarily intended for but not limited to evaluating the strength of cohesive materials. In this study, western Iowa loess samples were are compacted using five different compaction energies (Table 20) to obtain different target unit weights above and below the optimum moisture. The objective of the study was to evaluate the moisture-dry unit weight-CBR relationships. Pictures of the compaction procedure are shown in Figure 17a-c. The compaction energy was determined using Equation 11 (Proctor 1948).

$$Energy_{impact} = \frac{\binom{number\ of\ blows}{per\ layer} X \binom{number\ of}{layers} X \binom{weight\ of}{hammer} X \binom{height\ of}{drop\ hammer}}{Volume\ of\ mold} \tag{12}$$

(a)

(b)



(c) (d)

Figure 17. (a) Std. and Mod. Proctor with 6 inch CBR mold; (2) Std. Proctor used to compact materials in CBR mold; (3) CBR test; (4) DCP test in the mold

Table 20. Experimental plan to conduct laboratory CBR test

Test	γ_{dmax} (lb/ft^3)	w (%)	Rel. to w_{opt}	Compaction Energy (lb-ft/ft^3)	Lifts	Blows /Lift	Hammer Weight (lb)	Drop Height (in.)
1	95.1	20.0	+2 Std.	SSS 4850	3	24	5.5	12
2	95.1	14.0	-4 Std.	SS 7425	3	37	5.5	12
3	98.5	14.0	-4 Std.	S 12400	3	61	5.5	12
4	100.2	16.0	-2 Std.	S12400	3	61	5.5	12
5	101.1	18.0	0 Std.	S 12400	3	61	5.5	12
6	99.8	20.0	+2 Std.	S 12400	3	61	5.5	12
7	96.6	22.0	+4 Std.	S 12400	3	61	5.5	12
8	104.1	13.0	-2 Mod.	SM 34650	5	38	10	18
9	109.4	13.0	-2 Mod.	M 56000	5	61	10	18
10	110.8	15.0	0 Mod.	M 56000	5	61	10	18
11	106.7	18.0	0 Std.	M 56000	5	61	10	18

After the CBR test, DCP test was conducted in the CBR mold with a 10.1 lb (i.e., half the weight of the original hammer), to determine DCP-CBR in accordance with Equation 3. The DCP-CBR results were then compared with the laboratory determined CBR values.

CHAPTER 4. MATERIALS

This chapter presents the soil index properties of the 28 embankment materials collected from 9 field projects in this study, and of western Iowa loess used in the laboratory study. The field project materials were obtained from Polk, Warren, Linn, Pottawattamie, Mills, Woodbury and Scott Counties in Iowa. Embankment materials were obtained from multiple test beds at each project sites. Gradation, Atterberg limits, specific gravity, and compaction properties were tested for each material.

Table 21 to Table 26 provide a summary of the parent materials, particle size analysis, Atterberg limits, specific gravity, soil classification, and Proctor compaction test results. The grain size distribution curves of the embankment materials are separated by each project and shown in Figure 18 to Figure 26. The embankment materials consisted of cohesive soils with glacial till at three project sites and with western Iowa loess at four project sites. On one project site, granular material consisting of alluvial sand from the Missouri river flood plain. Of the 25 cohesive materials collected, 6 classified as select, 18 classified as suitable, and one classified as unsuitable per Iowa DOT specification section 2102 soil classification(DOT 2012). The three granular soils collected were classified as suitable, per Iowa DOT specification section 2102.

Table 21. 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	3.0	2.6	1.8
Sand content (%) (4.75 mm – 75 μm)	9.7	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(27)	A-7-5(9)	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} (pcf)	103.9	104.0	110.0	110.6
Mod. Proctor, w_{opt} (%)	16.0	13.6	11.5	11.5
Mod. Proctor, γ_{dmax} (pcf)	112.3	120.0	122.0	123.0

Table 22. Soil index properties of embankment materials obtained from Warren County and Linn County #1

Parameter	Warren County TB1	Warren County TB2	Warren County TB3 (Grey)	Warren TB3 County (Brown)	Linn County #1
	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)	0.9	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(10)	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	15.0	21.0	17.0	13.5
Std. Proctor, γ_{dmax} (pcf)	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} (pcf)	123.9	128.5	115.5	125.0	130.8

Table 23. Soil index properties of embankment materials obtained from Linn County #2

Parameter	Linn County #2 TB1	Linn County #2 TB2	Linn County #2 TB3	Linn County #2 TB4	Linn County #2 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)	0.9	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} (pcf)	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} (pcf)	130.8	129.5	131.0	132.1	130.0

Table 24. 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(21)	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} (pcf)	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} (pcf)	117.5	117.5	109.2	105.0	110.0

Table 25. 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)	0.0	1.0	2.0	0.0	2.1
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} (pcf)	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} (pcf)	118.0	122.5	131.0	117.2	119.5

Table 26. Soil index properties of embankment materials obtained from Woodbury County US 20

Parameter	Woodbury County (US20) TB1	Woodbury County (US20) TB2	Woodbury County (US20) TB3	Woodbury County (US20) TB4
	9/26/2014	9/26/2014	10/18/2014	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(10)	A-6(10)	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} (pcf)	110.0	106.0	106.7	110.5
Mod. Proctor, w_{opt} (%)	12.4	14.0	14.0	13.0
Mod. Proctor, γ_{dmax} (pcf)	120.0	117.0	117.5	119.6

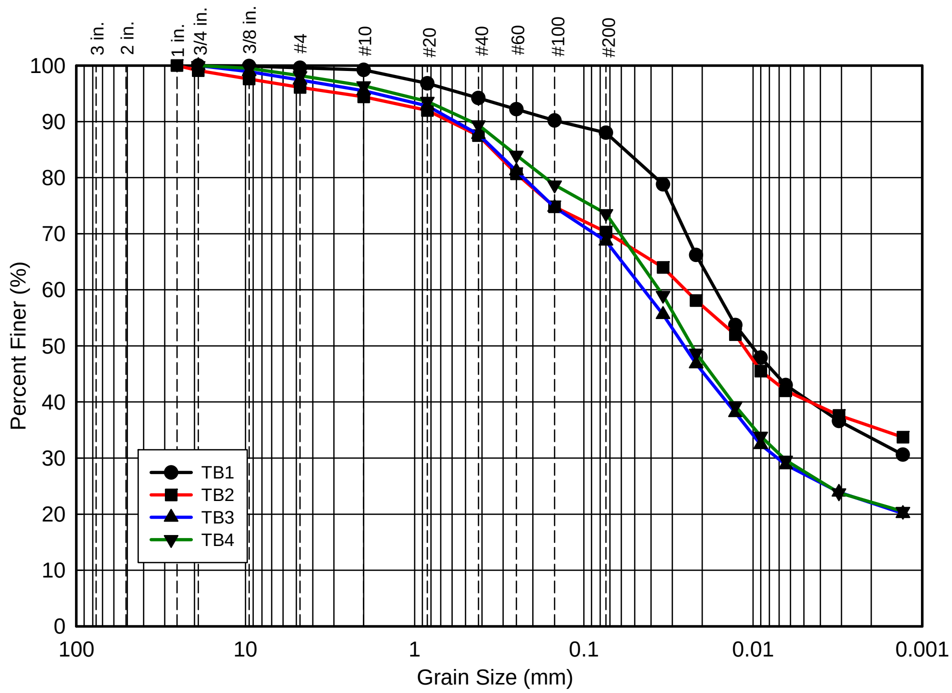


Figure 18. Grain size distribution of embankment materials obtained from Polk County

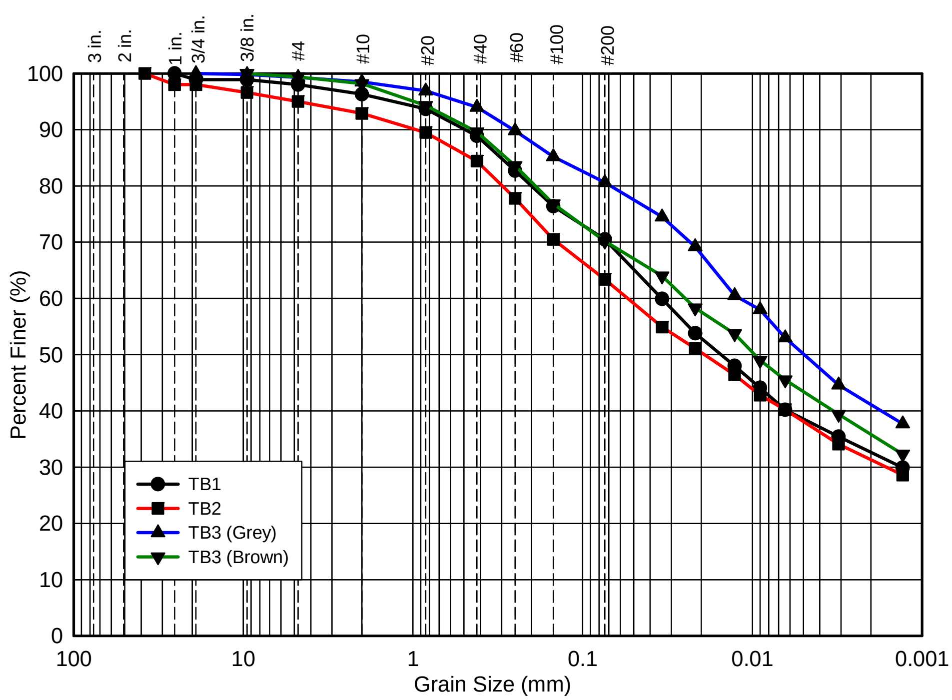


Figure 19. Grain size distribution of embankment materials obtained from Warren County

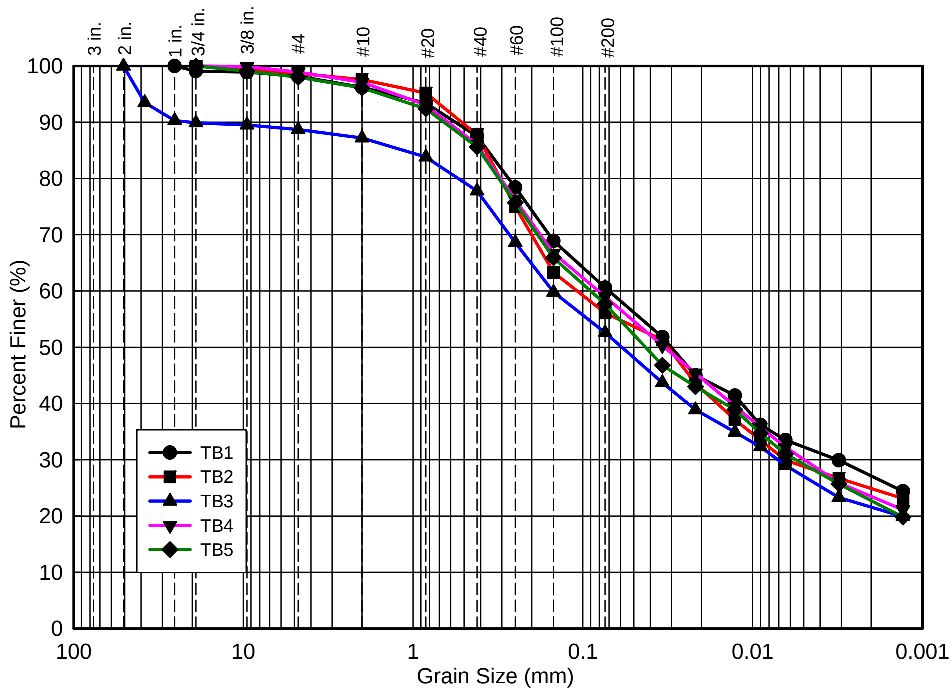


Figure 20. Grain size distribution of embankment materials obtained from Linn County #2

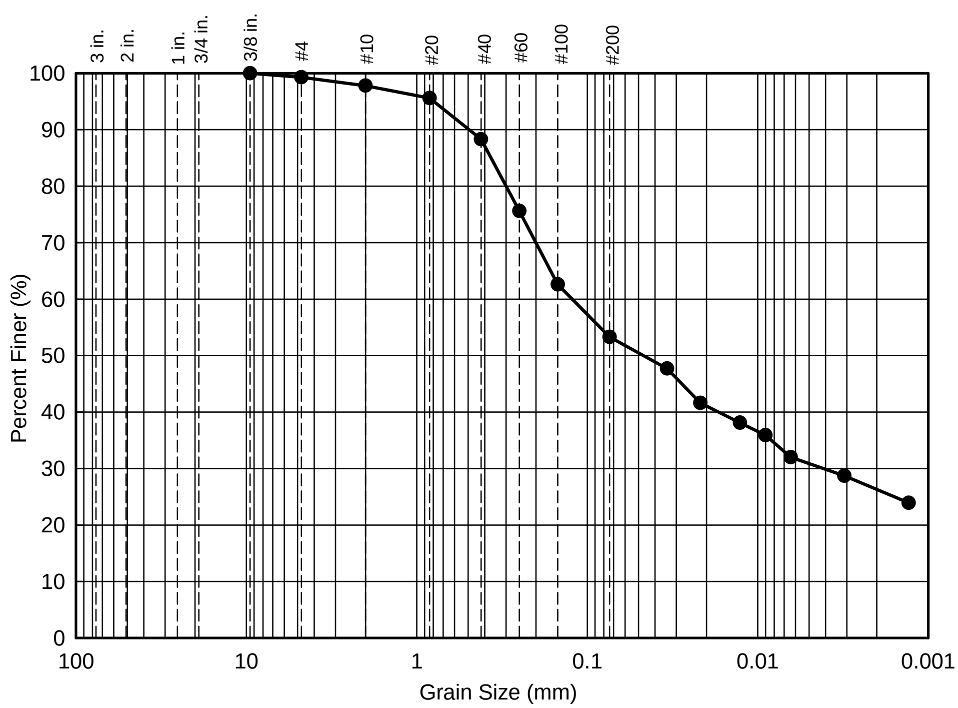


Figure 21. Grain size distribution of embankment materials obtained from Linn County #1

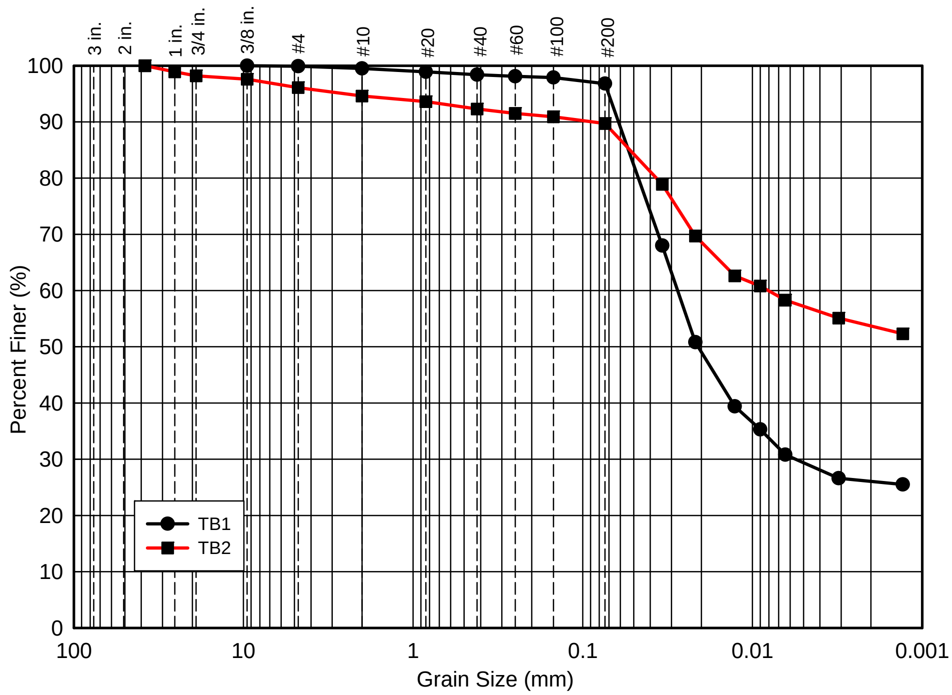


Figure 22. Grain size distribution of embankment materials obtained from Mills County

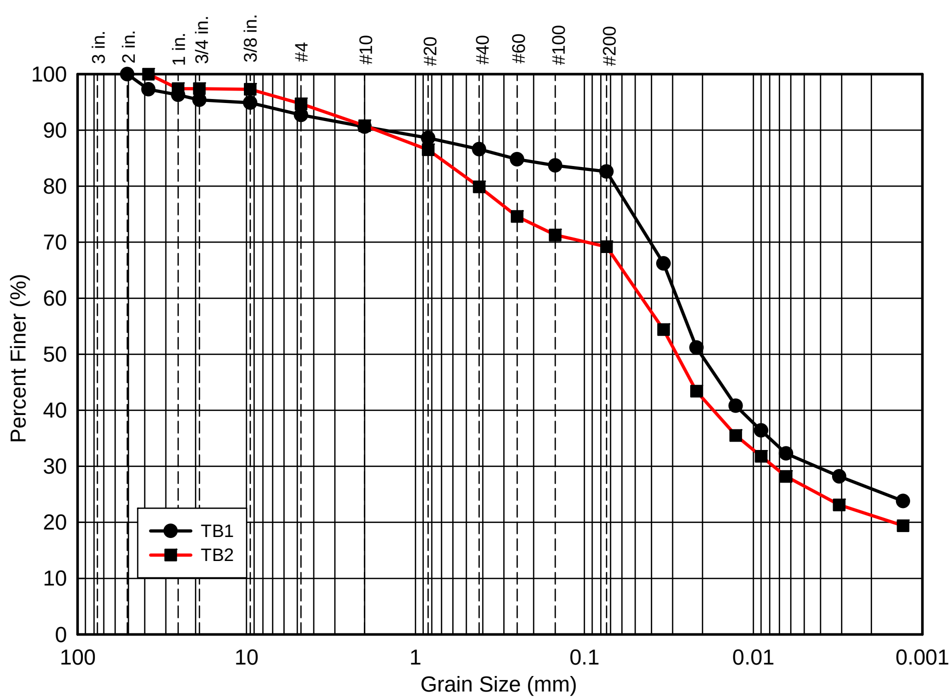


Figure 23. Grain size distribution of embankment materials obtained from Pottawattamie County

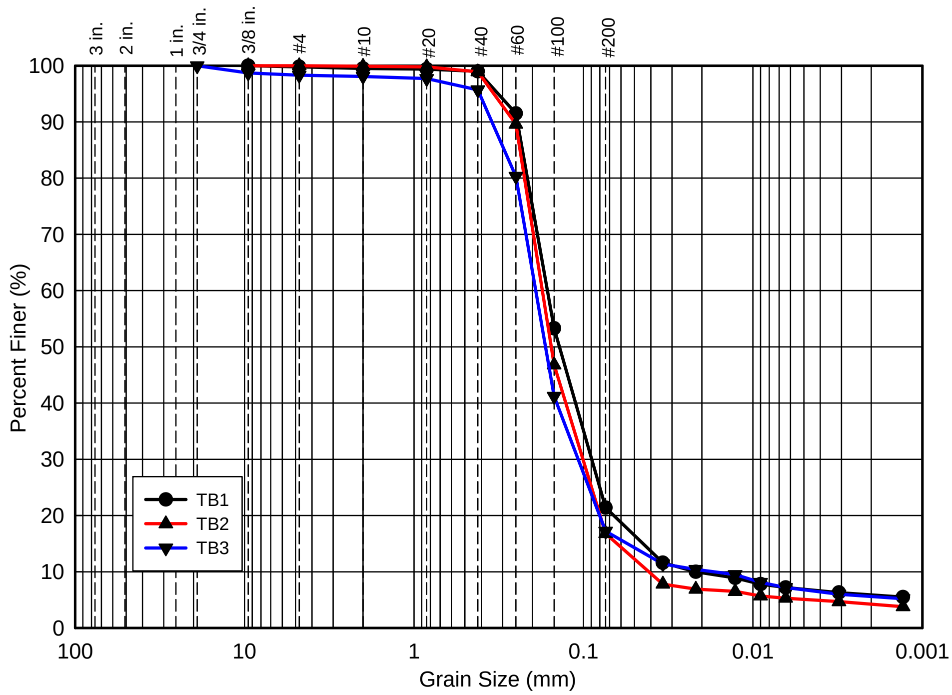


Figure 24. Grain size distribution of embankment materials obtained from Woodbury County I-29

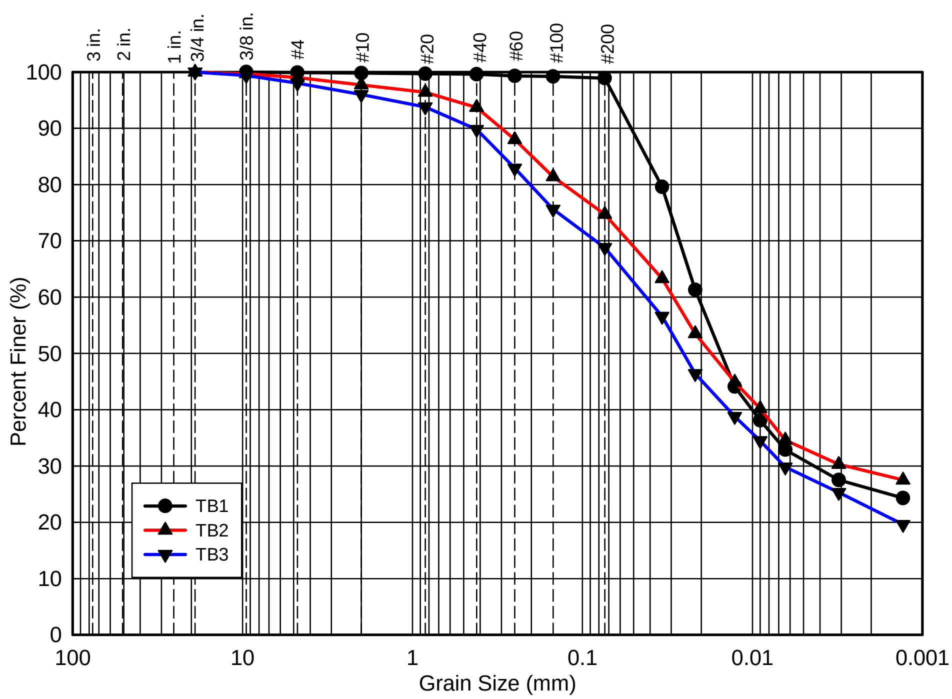


Figure 25. Grain size distribution of embankment materials obtained from Scott County

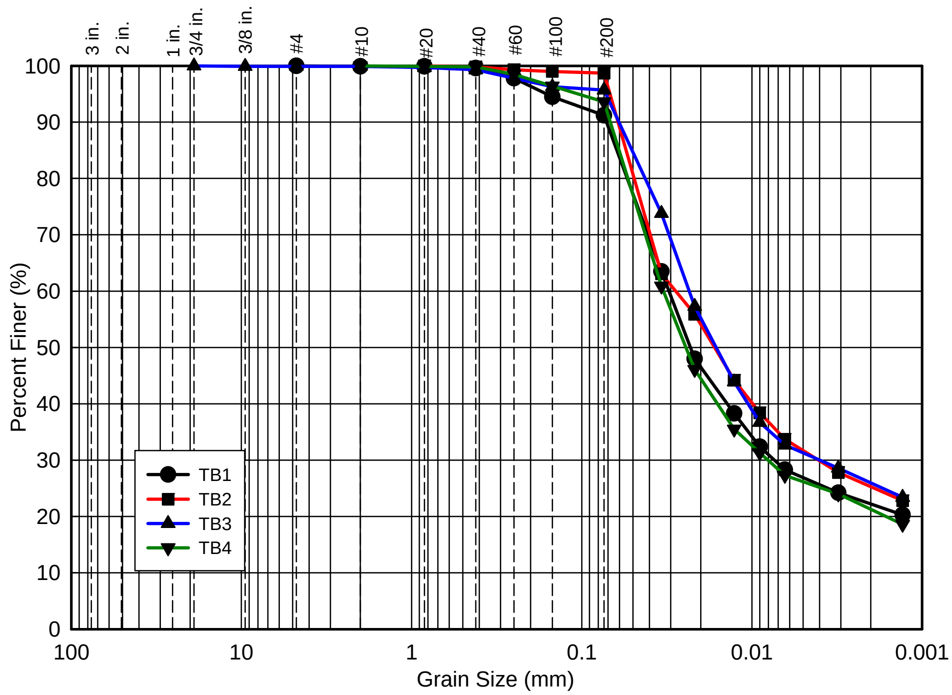


Figure 26. Grain size distribution of embankment materials obtained from Woodbury County US 20

Western Iowa loess material was used in this study to evaluate CBR-moisture-dry unit weight relationships. Index properties of the material are summarized in Table 27, and the material grain size distribution curve is shown in Figure 27.

Table 27. Material index properties of western Iowa loess

Parameter	Western Iowa loess
Parent Material	Loess
Gravel content (%) (> 4.75 mm)	0.0
Sand content (%) (4.75 mm – 75 μm)	2.9
Silt content (%) (75 μm – 2 μm)	97.1
Clay content (%) (< 2 μm)	6.5
Liquid limit, LL (%)	29
Plastic limit, PL (%)	23
Plastic Index, PI (%)	6
AASHTO classification	A-4(0)
USCS classification	CL-ML
USCS Description	Silty-clay

Table 27. Continued

Parameter	Western Iowa loess
Iowa DOT Material Classification	Suitable
Soil Color	Olive Brown
Specific Gravity, Gs	2.72
Std. Proctor, w_{opt} (%)	18.6
Std. Proctor, γ_{dmax} (pcf)	101.1
Mod. Proctor, w_{opt} (%)	15.7
Mod. Proctor, γ_{dmax} (pcf)	111.3

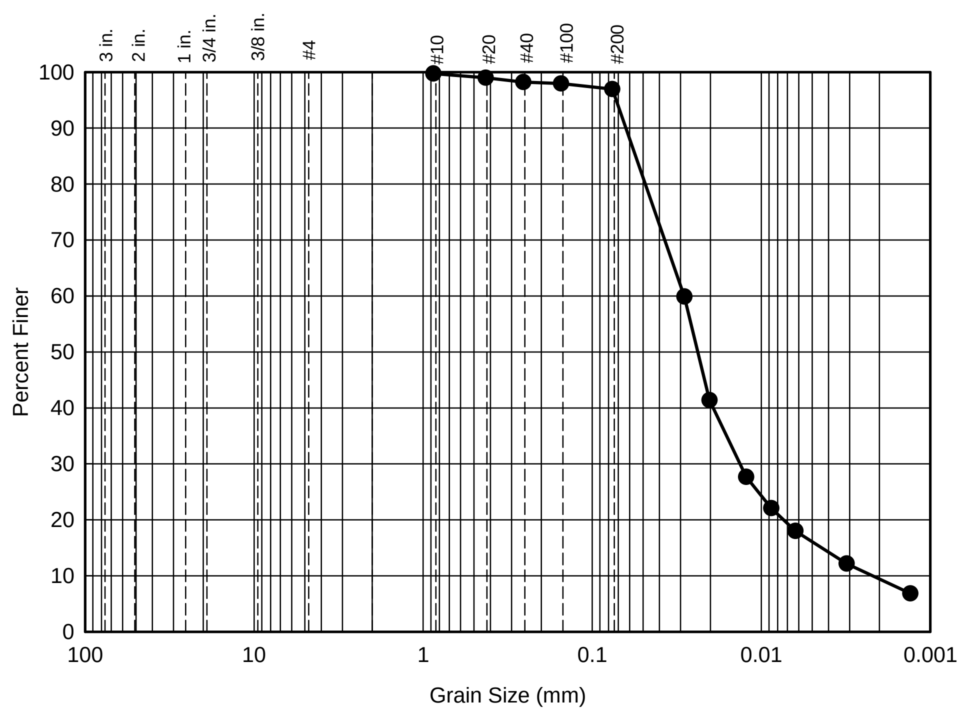


Figure 27. Particle size distribution curve of western Iowa loess

CHAPTER 5. RESULTS AND DISCUSSION

This chapter presents results from field projects and of laboratory evaluation to establish target CBR values for field QC/QA.

FIELD PROJECT RESULTS

A summary of field projects is provided in Table 28, which includes information of each projects, dates of ISU testing, field testing conducted at project site, and the availability of QC/QA data at the time of ISU testing. QC data was testing performed by the contractor or the contractor representative, while QA data was testing performed by the Iowa DOT or the DOT representative.

In the following sections, a project overview and field observations, ISU laboratory and field test results in comparison with QC/QA test results (where available), and a summary of key findings from each project are provided.

Table 28. Summary of field projects

Project Number	Project ID	Description	County	Field Testing		QC data during ISU Testing	QA data during ISU Testing
1		Northeast side of Intersection between I-35 and Grand Ave, Polk, IA	Polk	TB1: 5/29/14	15 DC, 5 DCP	NA	NA
2	IM-035-2(365)67--13-77	Northeast side of Intersection between I-35 and Grand Ave, Polk, IA	Polk	TB2: 6/7/14	NA	NA	NA
3		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

Table 28. Continued

Project Number	Project ID	Description	County	Field Testing		QC data during ISU Testing	QA data during ISU Testing
4	IM-035-2(365)67--13-77	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
5	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
6		Beside I-35, Hoover St, and NW 97th St, Warren, IA	Warren	TB2: 7/22/14	15 DC, 5 DCP	w	NA
7		Intersection between I-35 and Hwy 92, Warren, IA	Warren	TB3: 8/4/14	15 DC, 5 DCP	w	NA
8	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
9		New constructed Collins Rd near Old Ferry Rd, Linn, IA	Linn	TB2: 7/8/14	NA	w	NA
10		New constructed Collins Rd near Covington Rd, Linn, IA	Linn	TB3: 7/15/14	20 DC, 8 DCP	w	NA
11		New constructed Collins Rd near Covington Rd, Linn, IA	Linn	TB4: 8/1/14	15 DC, 5 DCP	w	NA
12		New constructed Collins Rd near Old Ferry Rd, Linn, IA	Linn	TB5: 9/8/14	15 DC, 5 DCP	w	NA
13	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

Table 28. Continued

Project Number	Project ID	Description	County	Field Testing		QC data during ISU Testing	QA data during ISU Testing
14	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
15		East side of Intersection between I-29 and Platteview, Mills, IA	Mills	TB2: 6/26/14	15 DC, 6 DCP	NA	NA
16	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
17		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
18	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	7 DCP	w	w
19		Southeast side of Intersection between I-29 and 260th st, Woodbury, IA	Woodbury I-29	TB2: 7/10/14	6 DCP	w	w
20		Southeast side of Intersection between I-29 and 260th st, Woodbury, IA	Woodbury I-29	TB3: 8/7/14	5 DCP	w	w
21	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
22		Northwest side of Intersection between I-74 and E 67th st, Scott, IA	Scott	TB2: 7/31/14	15 DC, 5 DCP	NA	NA
23		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 28. Continued

Project Number	Project ID	Description	County	Field Testing		QC data during ISU Testing	QA data during ISU Testing
24	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
25		Northeast side of Intersection between US 20 and Minnesota Ave, Woodbury, IA	Woodbury (US20)	TB2: 9/26/14	15 DC, 5 DCP	NA	NA
26		Northwest side of Intersection between US 20 and Jasper Ave, Woodbury, IA	Woodbury (US20)	TB3: 10/18/14	15 DC, 5 DCP	NA	NA
27		Northeast side of Intersection between US 20 and Minnesota Ave, Woodbury, IA	Woodbury (US20)	TB4: 10/18/14	15 DC, 5 DCP	NA	NA

Notes: DC – Drive Core Cylinder;
DCP – Dynamic cone penetrometer;
GPS measurements were obtained at each test location;
NA – Not available

Project 1. Polk County I-35 Reconstruction

Project overview and observations

The project site was visited four times. Figure 28 to Figure 33 shows pictures of the construction equipment used on the site during embankment construction. Caterpillar MT-35 scrapers and Caterpillar 740B dump trucks were used to collect fill materials from cuts and borrow areas for placement in fill areas. Caterpillar 143H motor grader was used to level the embankment surface. A disk was used to dry embankment materials. A Caterpillar D6T dozer was used to spread loose lift material. A pull behind sheepsfoot roller was used for soil compaction. Some other field observations with seepage observed at the toe of the embankment, where a borrow source was located, is shown in Figure 34, and geogrid placed at the bottom for the embankment toe is shown in Figure 35.

Field observations indicated that the fill material obtained from the borrow area was relatively wet. Pumping was observed under construction traffic. Field testing was conducted on 5/29/2014, 6/7/2014, 8/5/2014, and 8/19/2014 on test sections that were passed either on the same day or the previous day by the Iowa DOT field inspector. Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 5 locations, during the 1st, 3rd, and 4th visit. No testing was performed during the 2nd visit due to rain.

Figure 28. Caterpillar MT-35 scraper used to collect and place loose fill materials

Figure 29. Caterpillar 740B dump truck used to place loose fill materials

Figure 30. Caterpillar 143H motor grader used to level the embankment surface

Figure 31. Disk used to dry embankment materials

Figure 32. Caterpillar D6T Dozer used to control lift thickness

Figure 33. Sheepsfoot roller used for soil compaction

Figure 34. Seepage near embankment toe

Figure 35. Geogrid placed near embankment toe

Field and laboratory test results

Laboratory Proctor test results for materials collected from the four test beds are presented in Figure 36, Figure 38, Figure 39, and Figure 41. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} and γ_{dmax} that was being used by the DOT for QC/QA; (c) ISU field test results from dive core testing; and (d) contractor QC test results (only on TB4).

Based on 15 drive core tests performed from TB1, only 4 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 0.2% to +7.2% of w_{opt}, with an average of about +2.6% from all test points. The dry unit weight of the material varied from 95% to 101.6% of standard Proctor γ_{dmax}, with an average of about 97.8% from all test points (Figure 36).

DCP-CBR profiles and cumulative blows with depth profiles from the four test beds are presented in Figure 37, Figure 40, and Figure 42. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 1.4 in the top 8 in. and 12 in, which is considered as very poor according to SUDAS. The CBR value for top 9 in. of point 1 is extremely low which can be indicated as uncompacted fill. Point 4, 7 and 13 have very low CBR values even when the depth goes deeper, that indicate this test bed have poor quality (Figure 37).

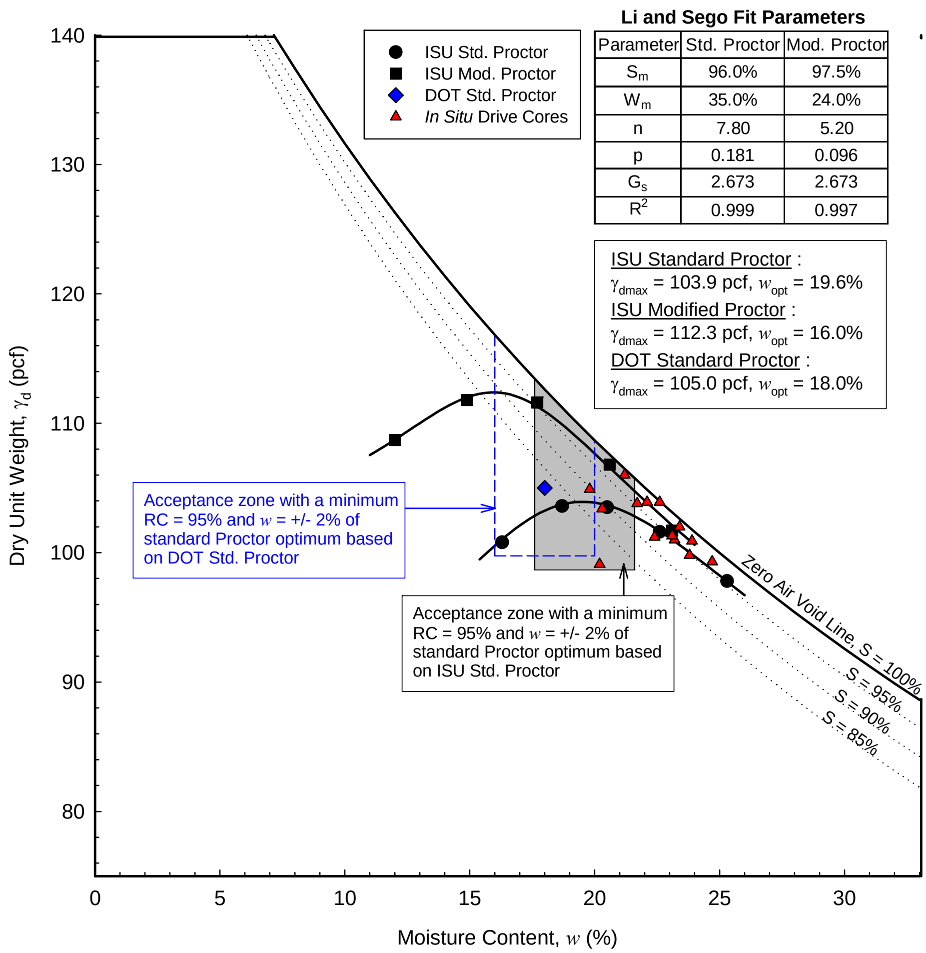


Figure 36. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Polk County Project TB1

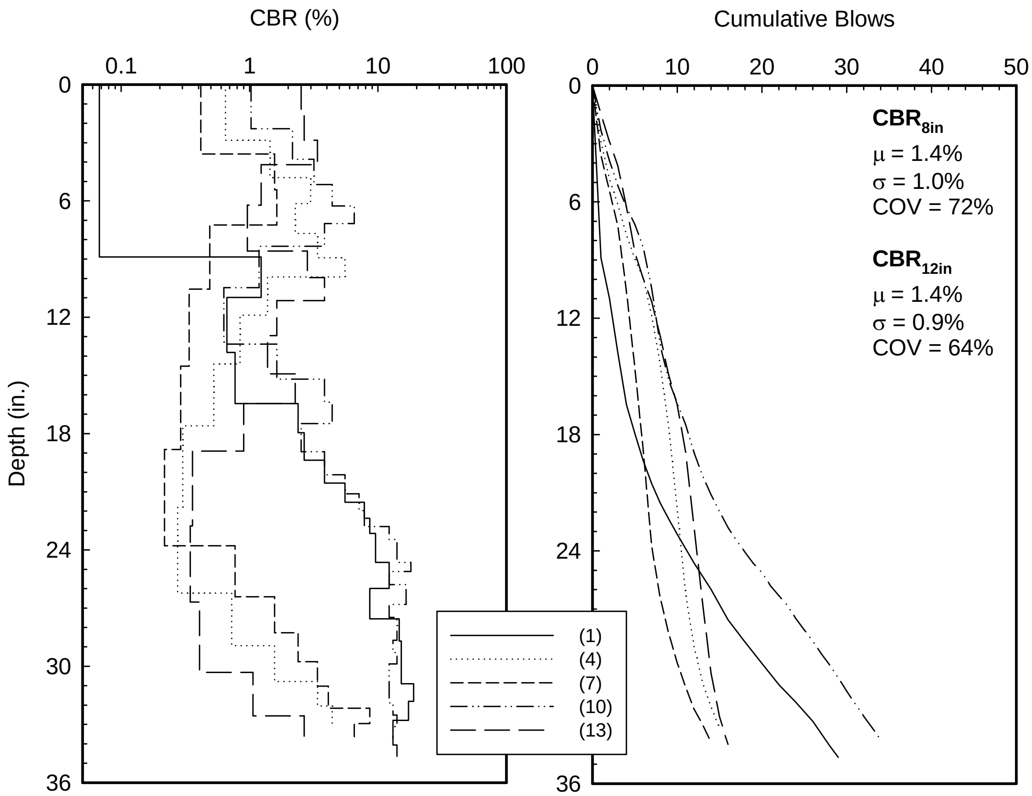


Figure 37. DCP-CBR profile at Polk County Project TB1

TB2 does not have in situ drive core data (Figure 38).

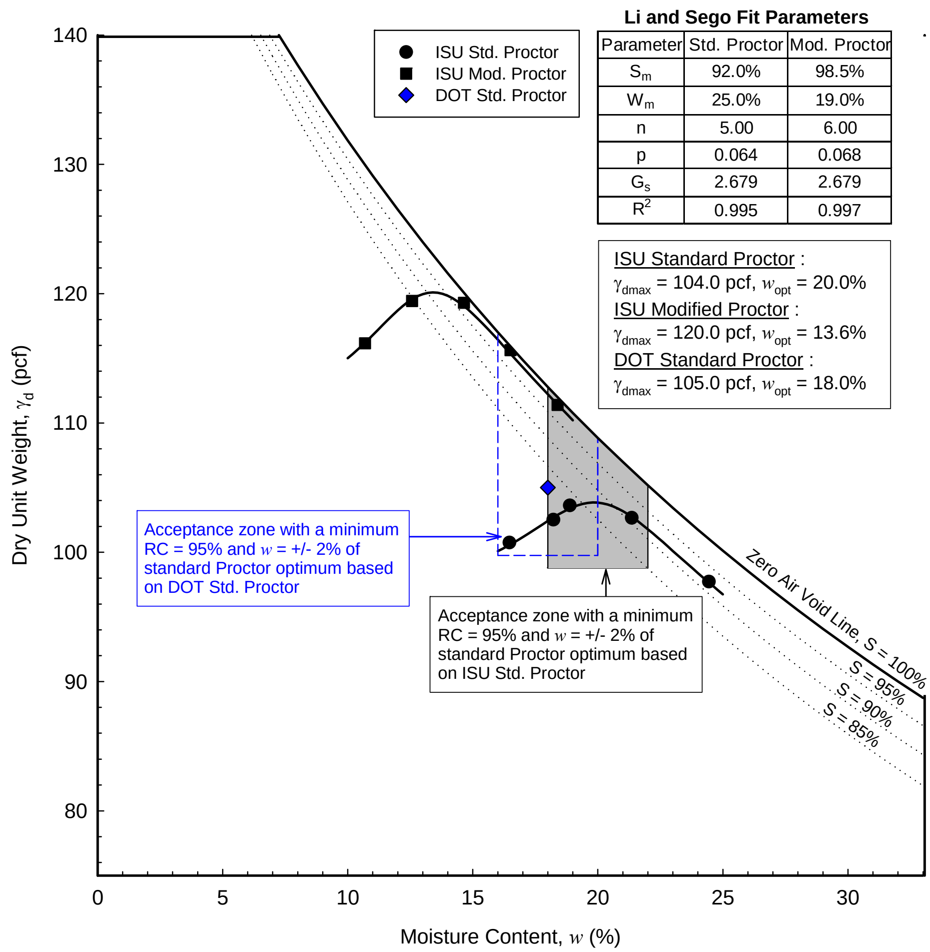


Figure 38. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Polk County Project TB2

Based on 15 drive core tests performed from TB3, all 15 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 1.5% to +0.5% of w_{opt}, with an average of about -0.7% from all test points. The dry unit weight of the material varied from 99.1% to 105.1% of standard Proctor γ_{dmax}, with an average of about 103% from all test points (Figure 39).

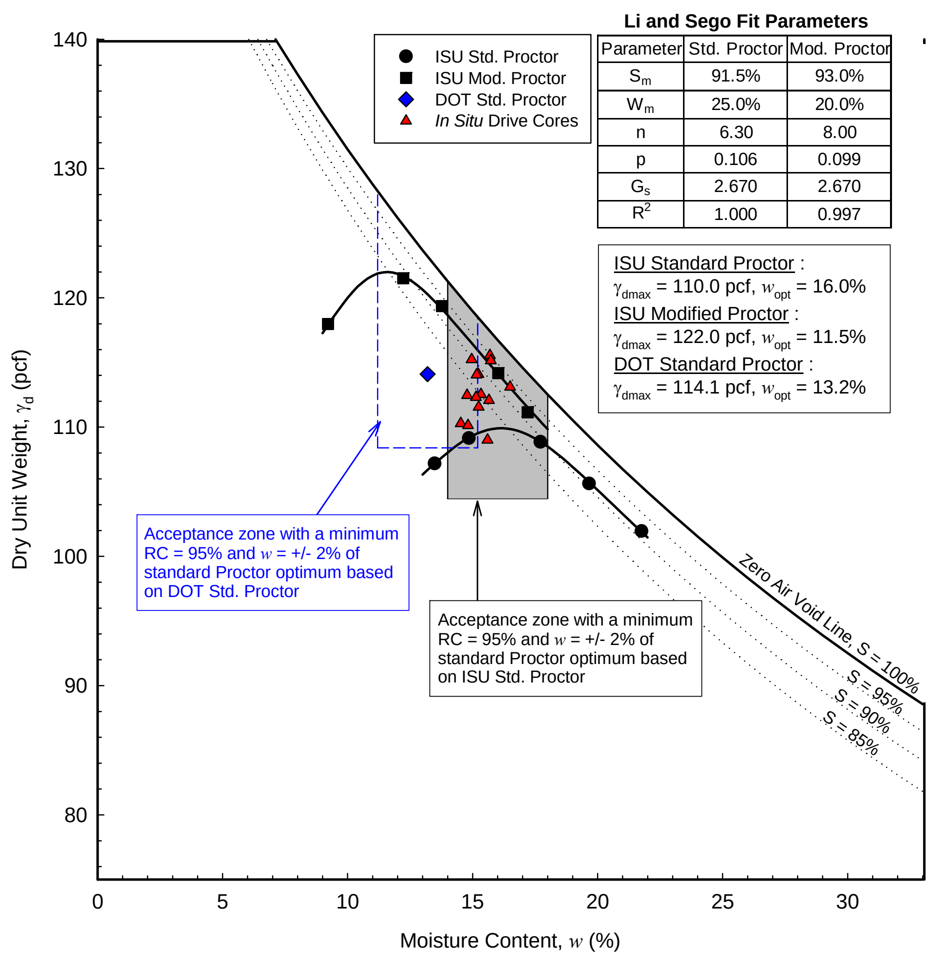


Figure 39. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Polk County Project TB3

The results from TB3 showed an average DCP-CBR of 8.2 in top 8 in. and 8.6 in top 12 in, which is considered as poor according to SUDAS. Point 3 have a very low CBR from 7 to 14 inch, and point 9 have a very low CBR from 19 to 29 inch which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 40).

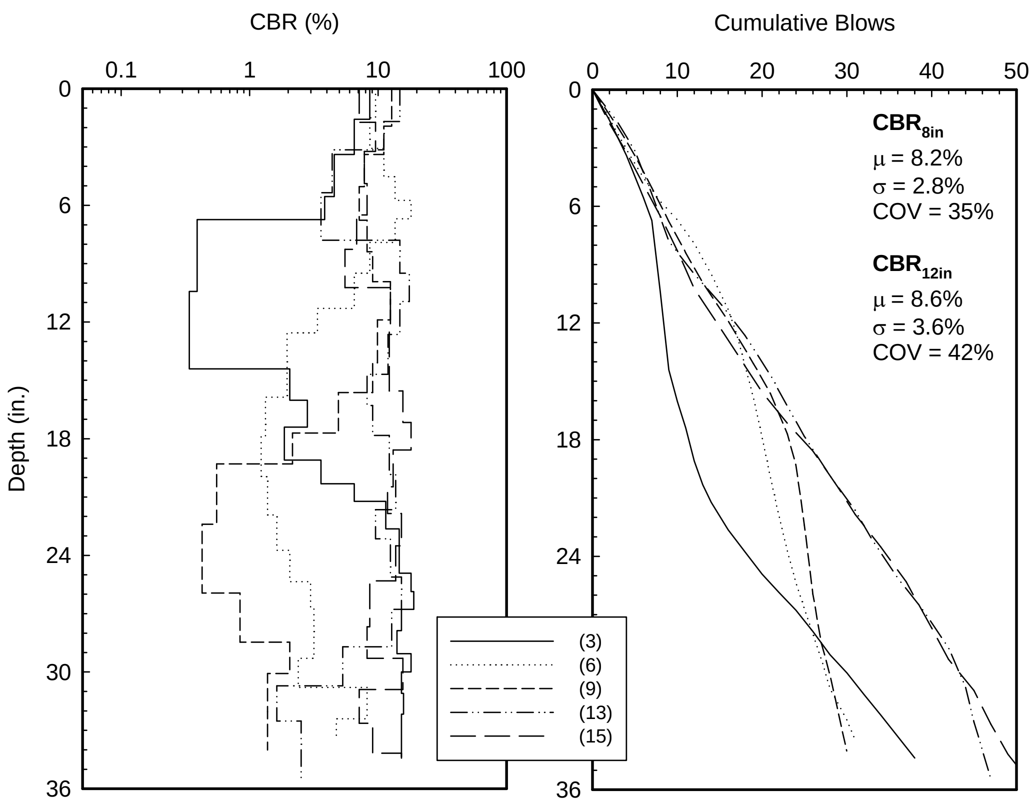


Figure 40. DCP-CBR profile at Polk County Project TB3

Based on 15 drive core tests performed from TB4, only 2 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 3.4% to +4.8% of w_{opt}, with an average of about +3.0% from all test points. The dry unit weight of the material varied from 94.2% to 105.1% of standard Proctor γ_{dmax}, with an average of about 96.8% from all test points. The QC test results which marked as a yellow star are qualified for Iowa DOT standard. (Figure 41Figure 39).

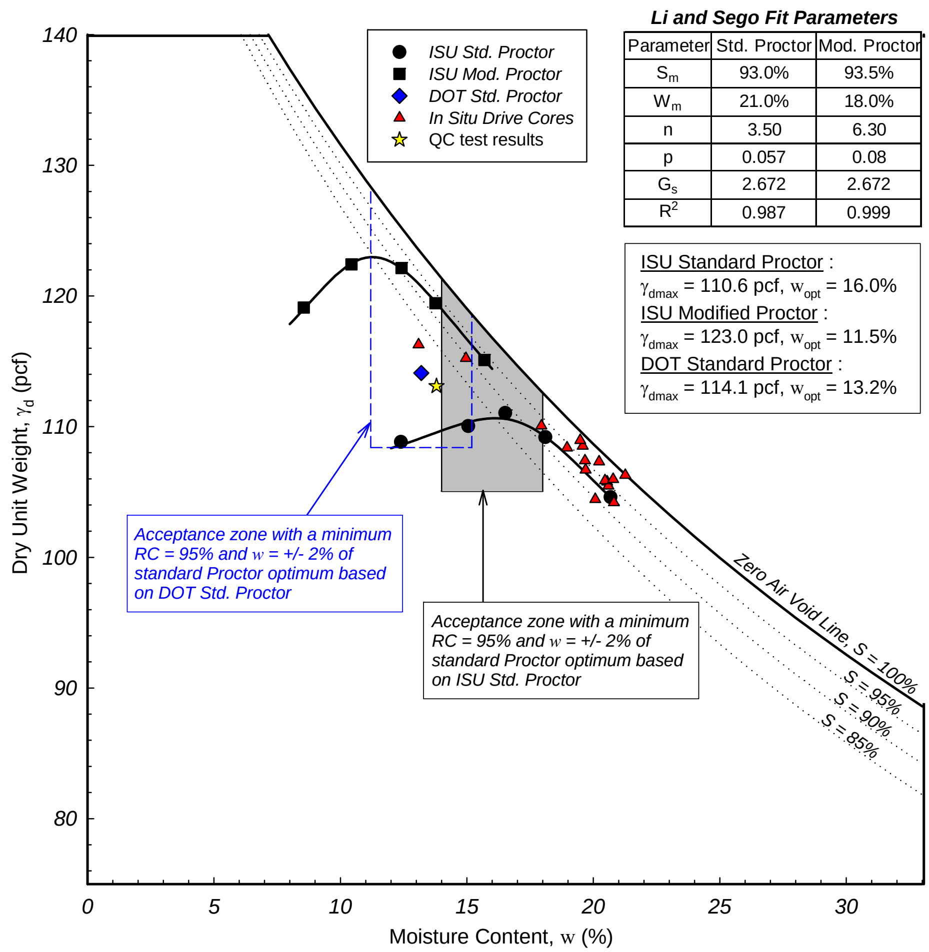


Figure 41. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Polk County Project TB4

The results from TB4 showed an average DCP-CBR of 0.6 in top 8 in. and 3.4 in top 12 in, which is considered as poor according to SUDAS . Point 2 and top 5 inch of all points have very low CBR value which is an indication of “uncompacted” fill effect, the CBR value of point 6 and 10 varies a lot which indicate the compaction is poor and not uniform (Figure 42).

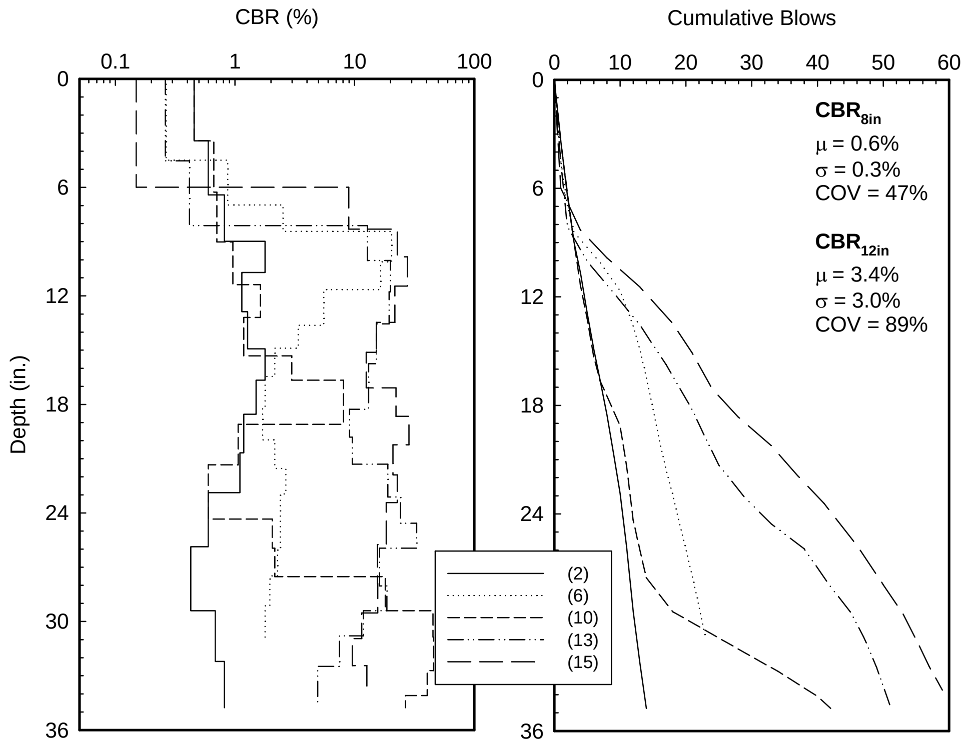


Figure 42. DCP-CBR profile at Polk County Project TB4

Summary of results

Table 29 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill materials placed were relatively wet and pumping was noticed under construction traffic.
- The moisture content of the material in TB1 varied from - 0.2% to +7.2% of w_{opt}, with an average of about +2.6% from all test points. The dry unit weight of the material varied from 95% to 101.6% of standard Proctor γ_{dmax}, with an average of about 97.8% from all test points.

- The results from TB1 showed an average DCP-CBR of about 1.4 in the top 8 in. and 12 in, which is considered as very poor.
- The moisture content of the material in TB3 varied from - 1.5% to +0.5% of w_{opt}, with an average of about -0.7% from all test points. The dry unit weight of the material varied from 99.1% to 105.1% of standard Proctor γ_{dmax}, with an average of about 103% from all test points.
- The results from TB3 showed an average DCP-CBR of 8.2 in top 8 in. and 8.6 in top 12 in, which is considered as poor.
- The moisture content of the material in TB4 varied from - 3.4% to +4.8% of w_{opt}, with an average of about +3.0% from all test points. The dry unit weight of the material varied from 94.2% to 105.1% of standard Proctor γ_{dmax}, with an average of about 96.8% from all test points.
- The results from TB4 showed an average DCP-CBR of 0.6 in top 8 in. and 3.4 in top 12 in, which is considered as poor.

Table 29. Summary of field results for 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
Relative Compaction				
Average (%)	97.8	N/A	103.0	97.1
Range (%)	95 to 101.6	N/A	99.1 to 105.1	94.2 to 105.1
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

Table 29. Continued

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
Standard Deviation (%)	1.92	N/A	0.49	1.97
COV (%)	73	N/A	-73	65
$CBR_{8\text{ in.}}$				
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
$CBR_{12\text{ in.}}$				
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

Project 2. Warren County

Project overview and observations

The project site was visited three times. Figure 43 to Figure 45 shows pictures of the construction equipment used on the site during embankment construction. Caterpillar D6T Dozer used to spread loose lift material, Caterpillar MT-35 scraper used to collect fill materials from cuts and borrow areas for placement in fill areas, a pull behind sheepsfoot roller was used for soil compaction.

Field observations indicated that the fill material obtained from the borrow area was in a suitable humidity. Field testing was conducted on 6/3/2014, 7/22/2014, and 8/4/2014 on test sections that were passed either on the same day or the previous day by the Iowa DOT field inspector. Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 5 locations, during the visit.

Figure 43. Caterpillar D6T Dozer used to control lift thickness

Figure 44. Caterpillar MT-35 scraper used to collect and place loose fill materials

Figure 45. Sheepsfoot roller used for soil compaction

Field and laboratory test results

Laboratory Proctor test results for materials collected from the three test beds are presented in Figure 46, Figure 48, Figure 50, and Figure 51, and two soil type were presented from TB 3. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} and γ_{dmax} that was being used by the DOT for QC/QA; and (c) ISU field test results from dive core testing.

Based on 15 drive core tests performed from TB1, 12 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 2.5% to +11.3% of w_{opt}, with an average of about -0.1% from all test points. The dry unit weight of the material varied from 85.5% to 105% of standard Proctor γ_{dmax}, with an average of about 99% from all test points (Figure 46).

DCP-CBR profiles and cumulative blows with depth profiles from the three test beds are presented in Figure 47, Figure 49, and Figure 52. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 5.6 in the top 8 in. and 12 in, which is considered as poor according to SUDAS. The CBR value does not varies a lot for each point, which indicate the compaction is relatively uniform (Figure 47).

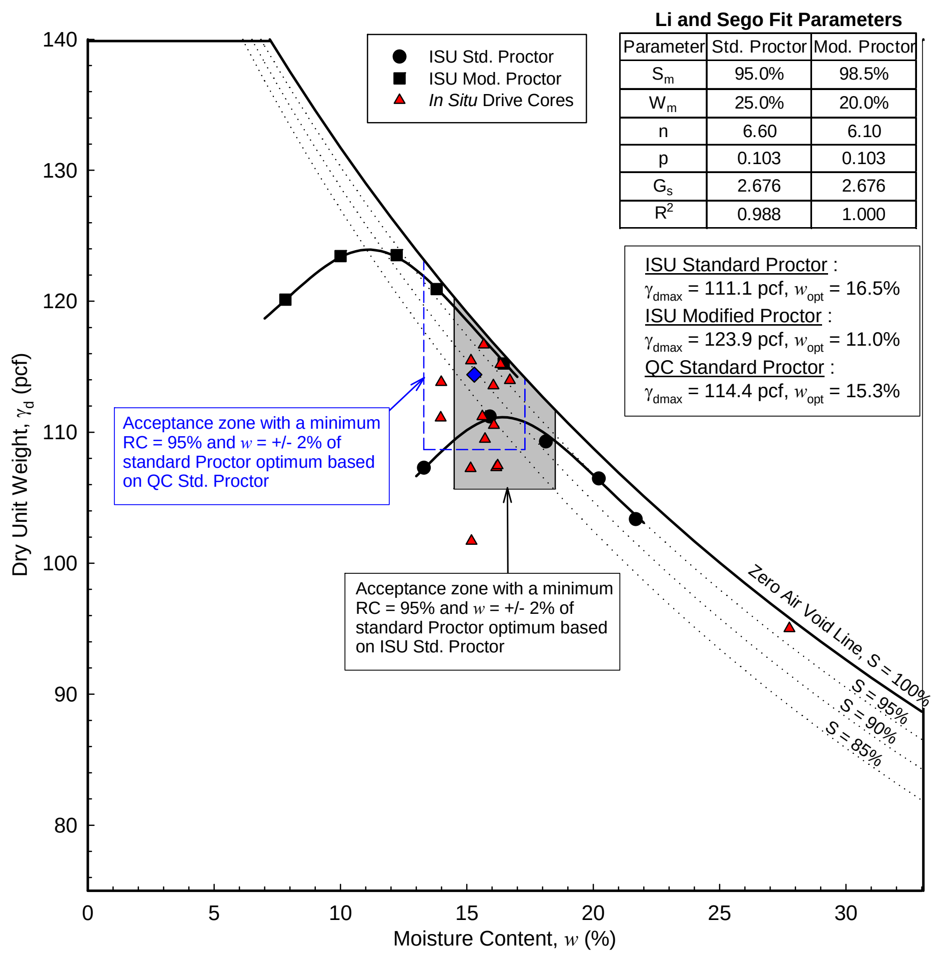


Figure 46. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Warren County Project TB1

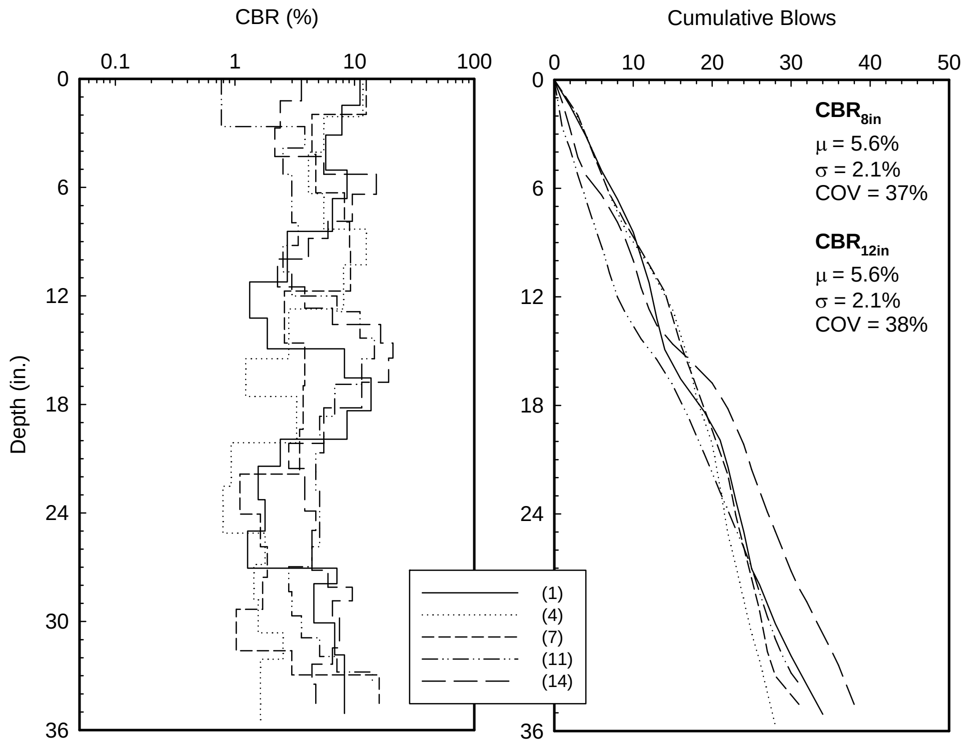


Figure 47. DCP-CBR profile at Warren County Project TB1

Based on 15 drive core tests performed from TB2, 9 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from -1.4% to +0.5% of w_{opt}, with an average of about -0.4% from all test points. The dry unit weight of the material varied from 91.5% to 102.7% of standard Proctor γ_{dmax}, with an average of about 97.5% from all test points (Figure 48).

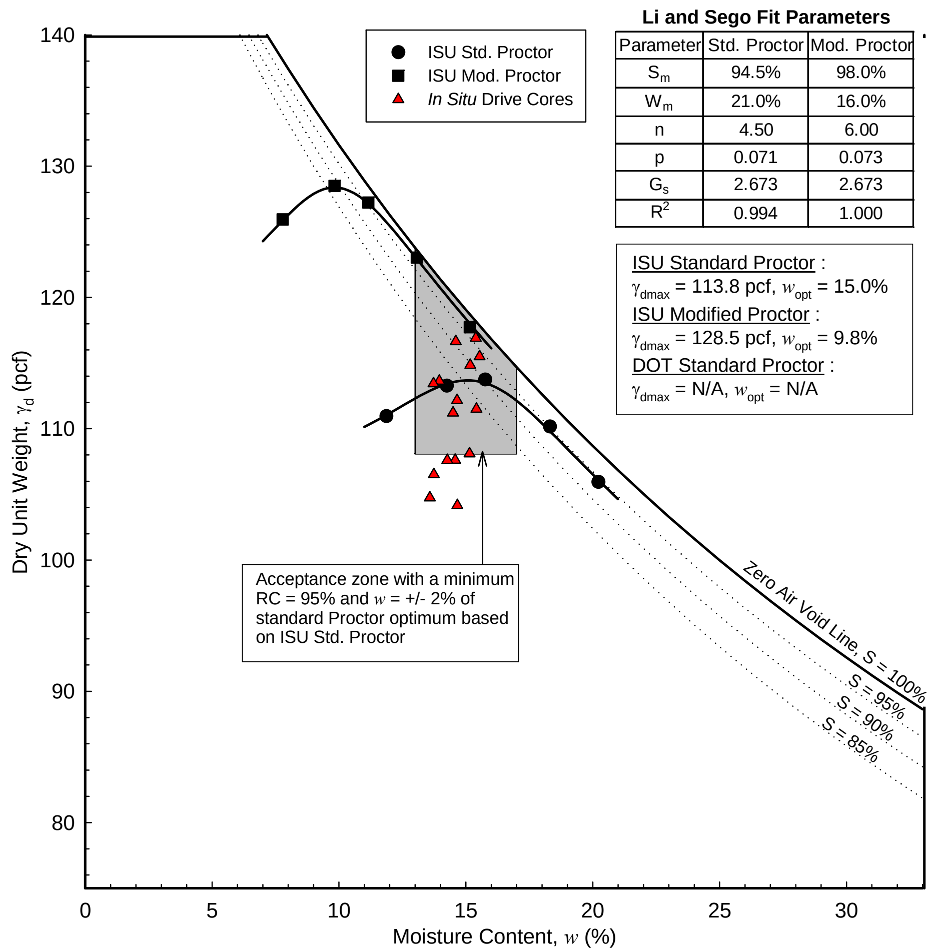


Figure 48. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Warren County Project TB2 (N/A: not available)

The results from TB2 showed an average DCP-CBR of 5.7 in top 8 in. and 5.6 in top 12 in, which is considered as poor according to SUDAS. Point 1, 4, and 7 have a low CBR below 20 in. deep, which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 49).

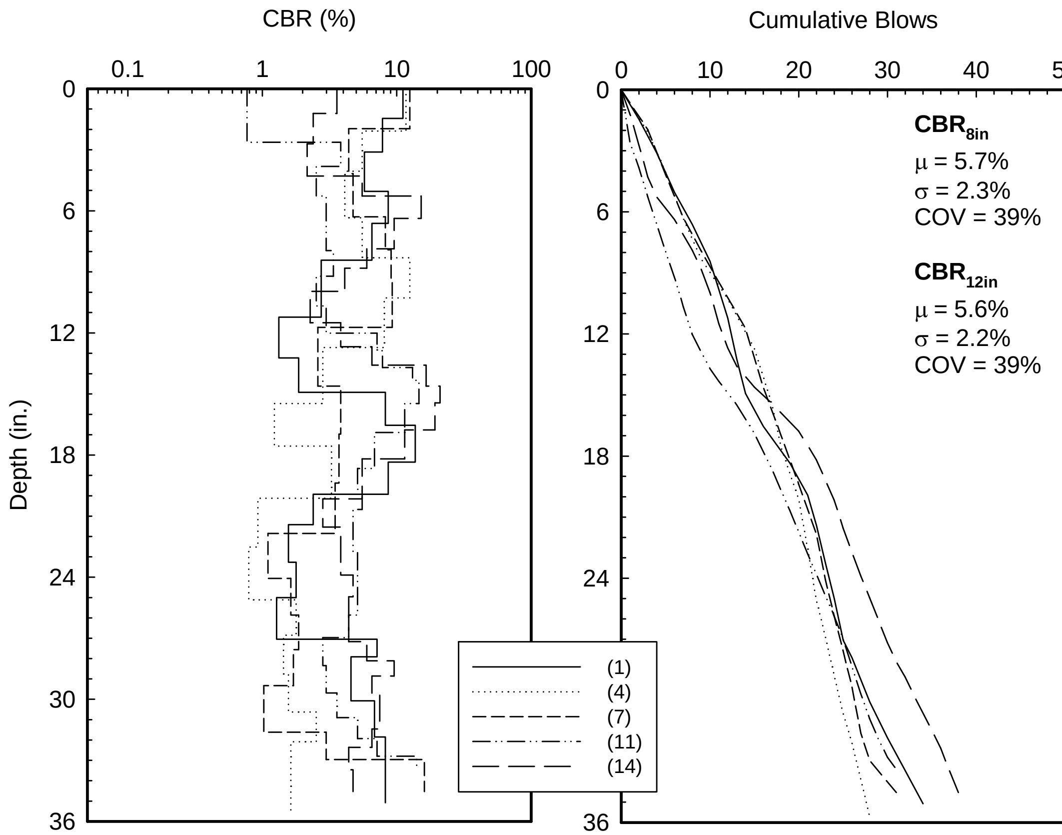


Figure 49. DCP-CBR profile at Warren County Project TB2

There are two types of soil in TB3, one is gray clay and the other one is brown clay. Based on 15 drive core tests, 4 out of 8 drive core results fell in the acceptance zone identified from ISU Proctor testing for brown clay. 0 out of 7 fell in the acceptance zone identified from ISU Proctor testing for grey clay. The moisture content of the material in situ varied from -3.2% to +9.4% of w_{opt}, with an average of about +3.3% from all test points. The dry unit weight of the material varied from 84.1% to 107.0% of standard Proctor γ_{dmax}, with an average of about 93.6% from all test points (Figure 50 and Figure 51).

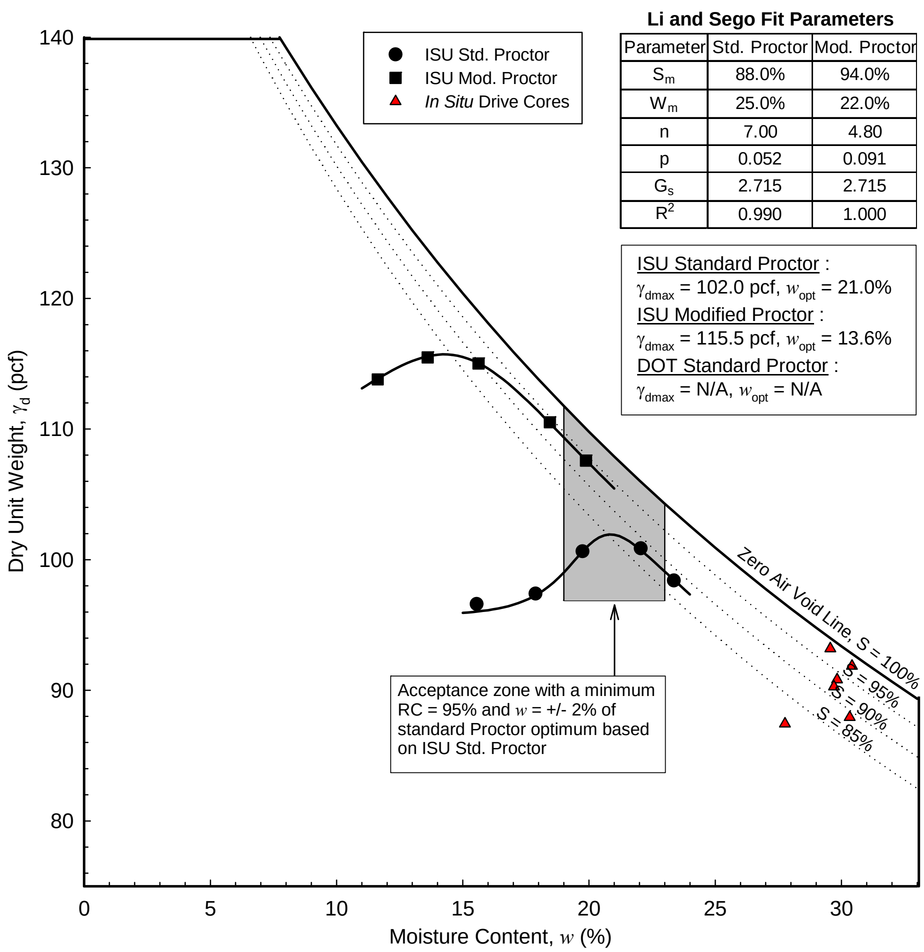


Figure 50. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Warren County Project TB3 (Grey soil) (N/A: not available)

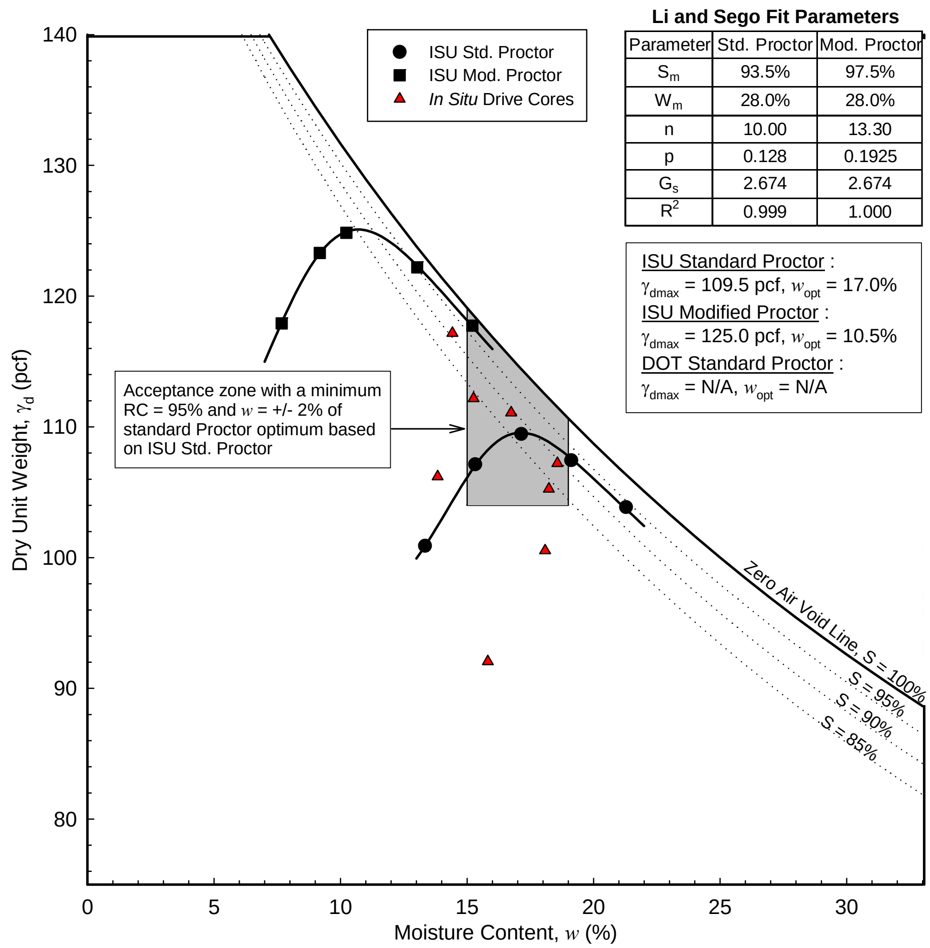


Figure 51. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Warren County Project TB3 (Brown soil) (N/A: not available)

The results from TB3 showed an average DCP-CBR of 4.9 in top 8 in. and 4.5 in top 12 in, which is considered as very poor according to SUDAS. Point 7 have a very low CBR from 7 to 16.5 in. deep, which is an indication of “uncompacted” fill or the oreo-cookie effect (Figure 52)

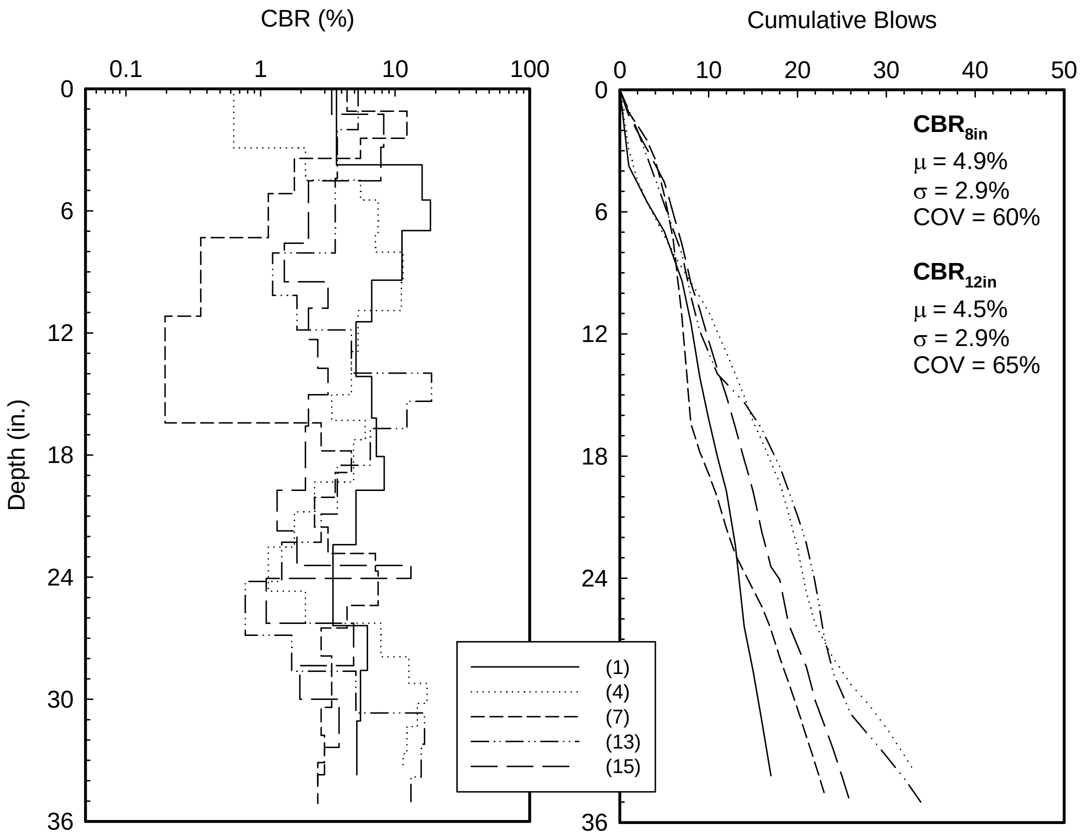


Figure 52. DCP-CBR profile at Warren County Project TB3

Summary of results

Table 30 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was in a suitable humidity
- The moisture content of the material in TB1 varied from - 2.5% to +11.3% of w_{opt}, with an average of about -0.1% from all test points. The dry unit weight of the material varied from 85.5% to 105% of standard Proctor γ_{dmax}, with an average of about 99% from all test points.

- The results from TB1 showed an average DCP-CBR of about 5.6 in the top 8 in. and 12 in, which is considered as poor.
- The moisture content of the material in TB2 varied from -1.4% to +0.5% of w_{opt}, with an average of about -0.4% from all test points. The dry unit weight of the material varied from 91.5% to 102.7% of standard Proctor γ_{dmax}, with an average of about 97.5% from all test points.
- The results from TB2 showed an average DCP-CBR of 5.7 in top 8 in. and 5.6 in top 12 in, which is considered as poor.
- The moisture content of the material in TB3 varied from -3.2% to +9.4% of w_{opt}, with an average of about +3.3% from all test points. The dry unit weight of the material varied from 84.1% to 107.0% of standard Proctor γ_{dmax}, with an average of about 93.6% from all test points.
- The results from TB3 showed an average DCP-CBR of 4.9 in top 8 in. and 4.5 in top 12 in, which is considered as very poor.

Table 30. Summary of field results for Warren County

Parameter	**Warren County TB1**	**Warren County TB2**	**Warren County TB3**
	6/3/2014	7/22/2014	8/4/2014
Relative Compaction			
Average (%)	99.0	97.5	93.6
Range (%)	85.5 to 105	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 (%)	-0.1	-0.4	3.3
Range (%)	-2.5 to +11.3	-1.4 to +0.5	-3.2 to +9.4

Table 30. Continued

Parameter	Warren County TB1	Warren County TB2	Warren County TB3
	6/3/2014	7/22/2014	8/4/2014
Standard Deviation (%)	3.25	0.65	4.78
COV (%)	-2828	-161	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

Project 3. Linn County #1

Project overview and observations

The project site was visited five times. Figure 53 to Figure 57 shows pictures of the construction equipment used on the site during embankment construction. Caterpillar 390D were used to collect fill materials from cuts and borrow areas for placement in fill areas, Caterpillar D6R Dozer was used to spread loose lift material, a disk was used to dry embankment materials, a pull behind sheepsfoot roller was used for soil compaction, Caterpillar 14M motor grader was used to level the embankment surface. Some other field observations with seepage observed in the construction site is shown in Figure 58.

Field observations indicated that the fill material obtained from the borrow area was relatively wet. Field testing was conducted on 6/6/2014, 7/8/2014, 7/15/2014, 8/1/2014 and 9/8/2014 on test sections that were passed either on the same day or the previous day by the

Iowa DOT field inspector. Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 5 locations, during the 1st, 4th, and 5th visit, and 20 test locations and DCP testing at 8 locations during 3rd visit. No testing was performed during the 2nd visit due to rain.

Figure 53. Caterpillar 390D excavated materials from borrow source

Figure 54. Caterpillar D6R Dozer used to control lift thickness

Figure 55. Disk used to dry embankment materials

Figure 56. Sheepsfoot roller used for soil compaction

Figure 57. Caterpillar 14M motor grader used to level the embankment surface

Figure 58. Seepage occurred in the construction site

Field and laboratory test results

Laboratory Proctor test results for materials collected from the five test beds are presented in Figure 59, Figure 61, Figure 62, Figure 64, and Figure 66. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} and γ_{dmax} that was being used by the DOT for QC/QA; and (c) ISU field test results from dive core testing.

Based on 15 drive core tests performed from TB1, all 15 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 1.5% to +1.3% of w_{opt}, with an average of about -0.5% from all test points. The dry unit weight of the material varied from 96.5% to 107% of standard Proctor γ_{dmax}, with an average of about 103.5% from all test points. TB1 is not a fresh compacted embankment test bed, it is a final grade embankment section (Figure 59).

DCP-CBR profiles and cumulative blows with depth profiles from the four test beds are presented in Figure 60, Figure 63, Figure 65, and Figure 67. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 7.6 in the top 8 in. and 6.9 in top 12 in, which is considered as poor according to SUDAS. The CBR value does not varies a lot for each point, which indicate the compaction is relatively uniform (Figure 60).

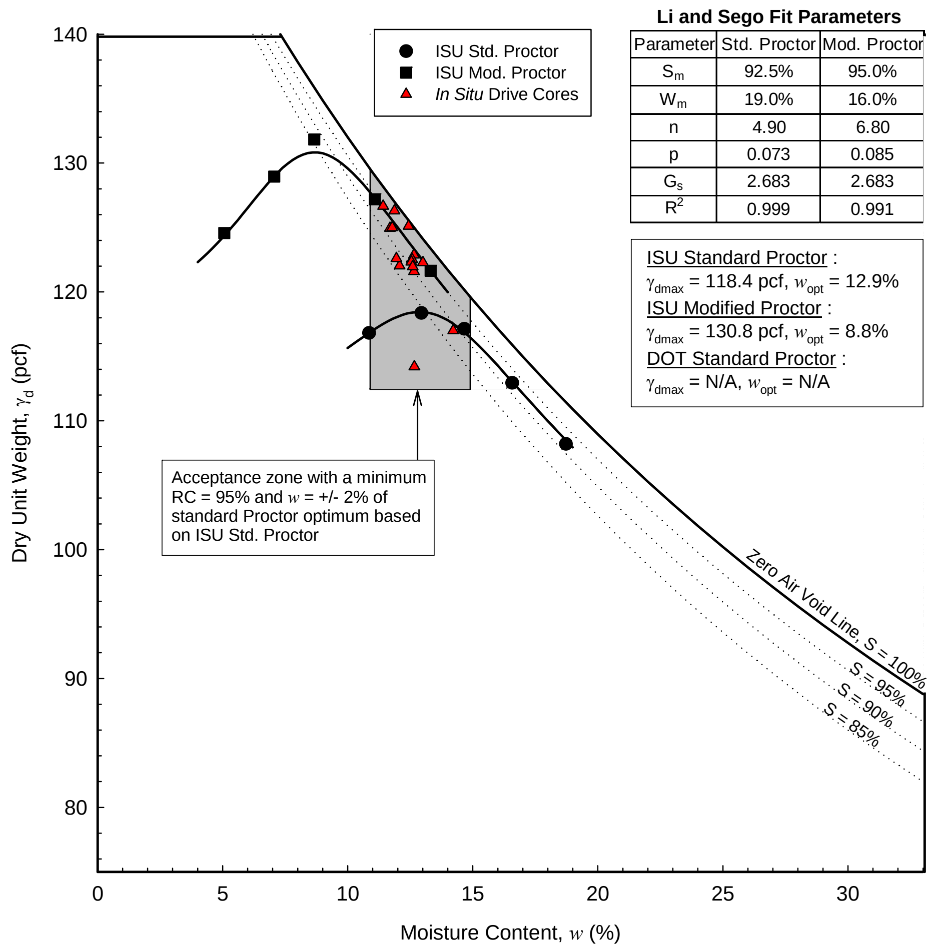


Figure 59. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Linn County #1 Project TB1

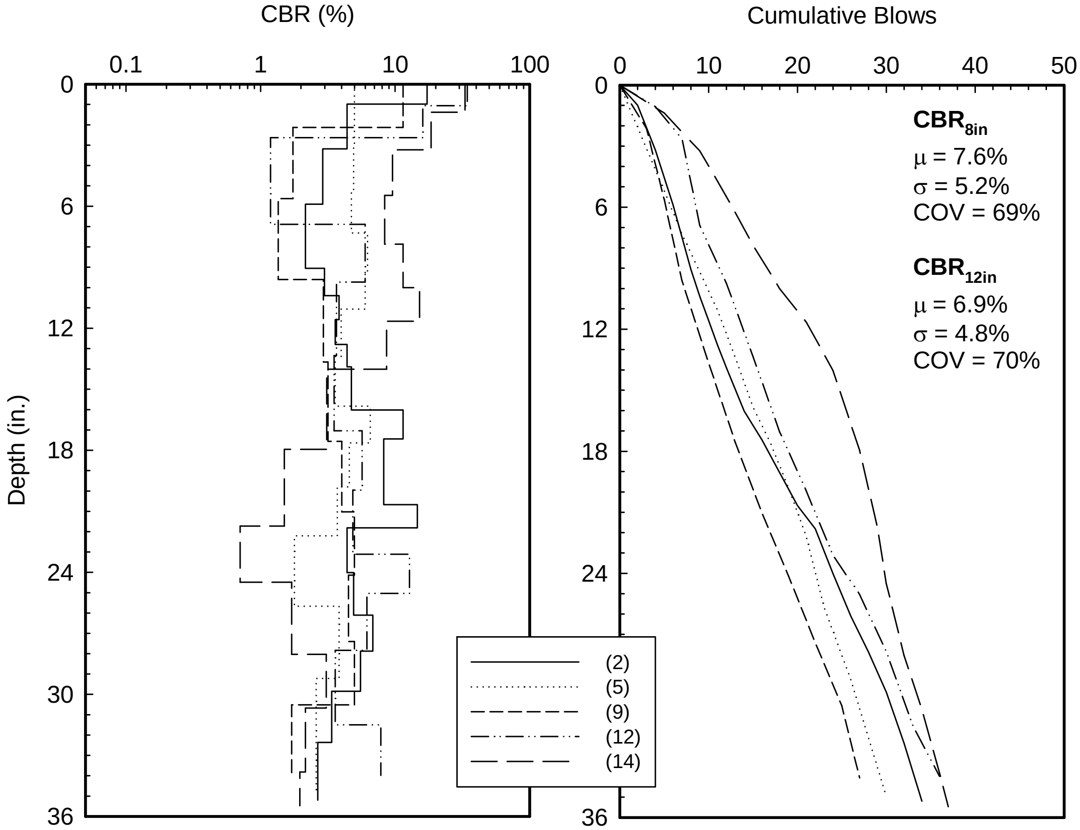


Figure 60. DCP-CBR profile at Linn County #1 Project TB1

No in situ drive core test have been conducted at TB2 (Figure 61).

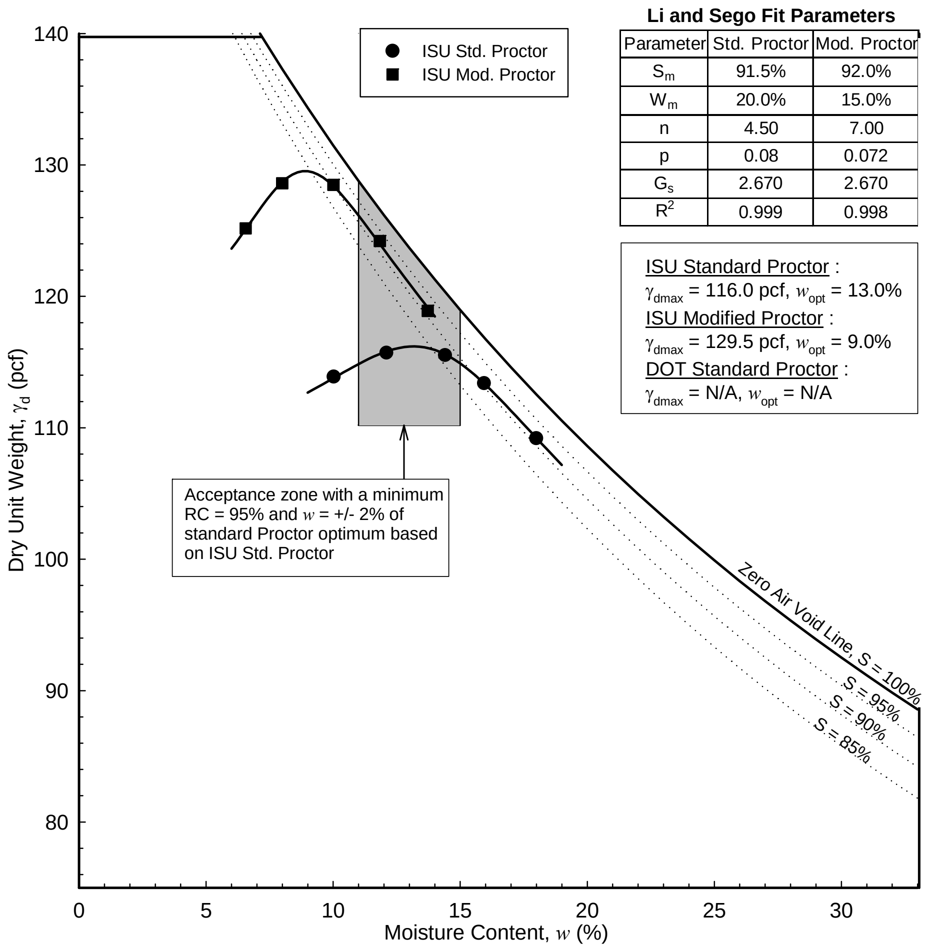


Figure 61. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Linn County #1 Project TB2

Based on 20 drive core tests performed from TB3, 17 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 1.8 to +2.8% of w_{opt}, with an average of about +0.6% from all test points. The dry unit weight of the material varied from 92.5% to 104% of standard Proctor γ_{dmax}, with an average of about 99.2% from all test points (Figure 62).

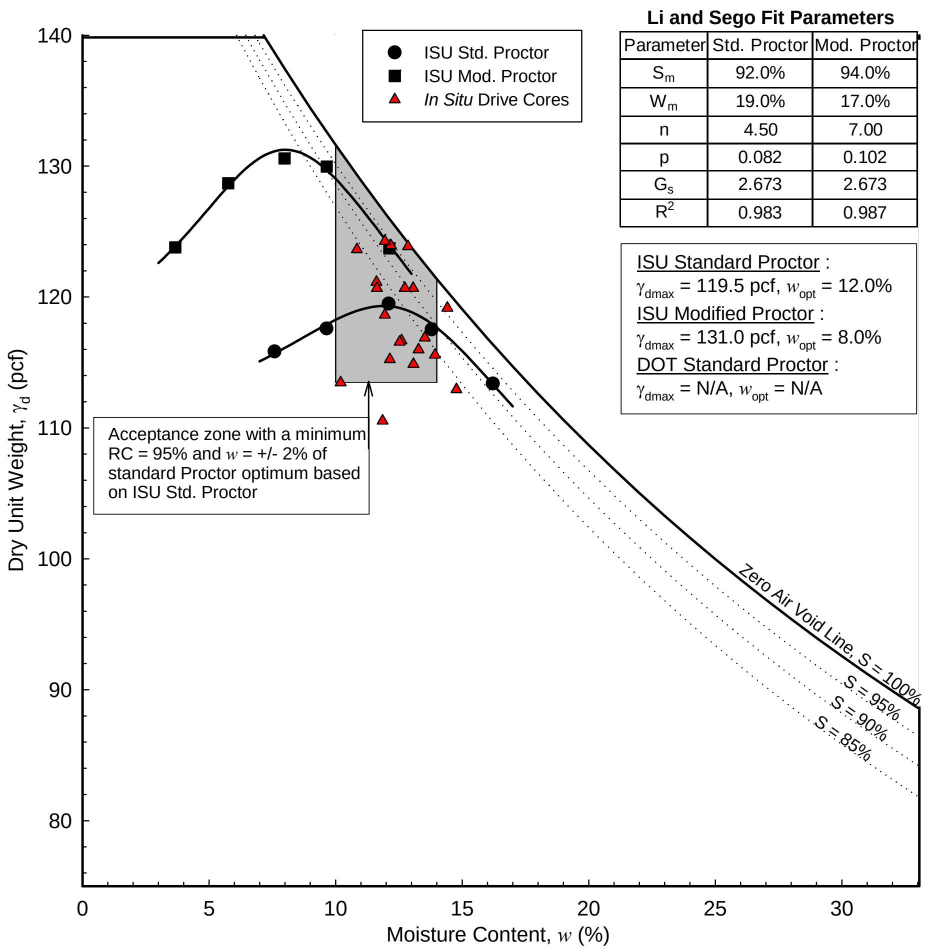


Figure 62. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Linn County #1 Project TB3 (N/A: not available)

The results from TB3 showed an average DCP-CBR of about 4.3 in the top 8 in. and 3.4 in top 12 in, which is considered as very poor according to SUDAS. The CBR value varies a lot while depth goes deeper which indicate this test bed is not uniformly compacted. Every DCP point have a low CBR value one or multiple times, which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 63).

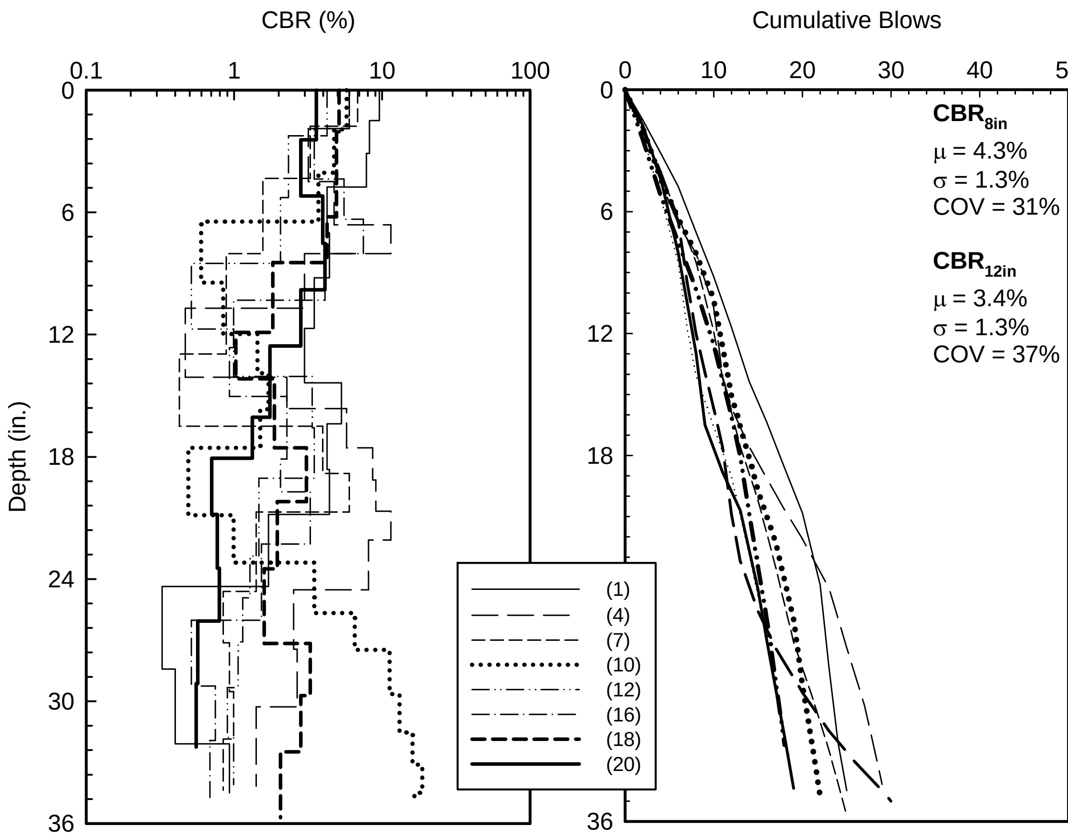


Figure 63. DCP-CBR profile at Linn County #1 Project TB3

Based on 15 drive core tests performed from TB4, 9 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from -0.9% to +10.1% of w_{opt}, with an average of about +2.5 % from all test points. The dry unit weight of the material varied from 87.8% to 103.2% of standard Proctor γ_{dmax}, with an average of about 98.8% from all test points (Figure 64).

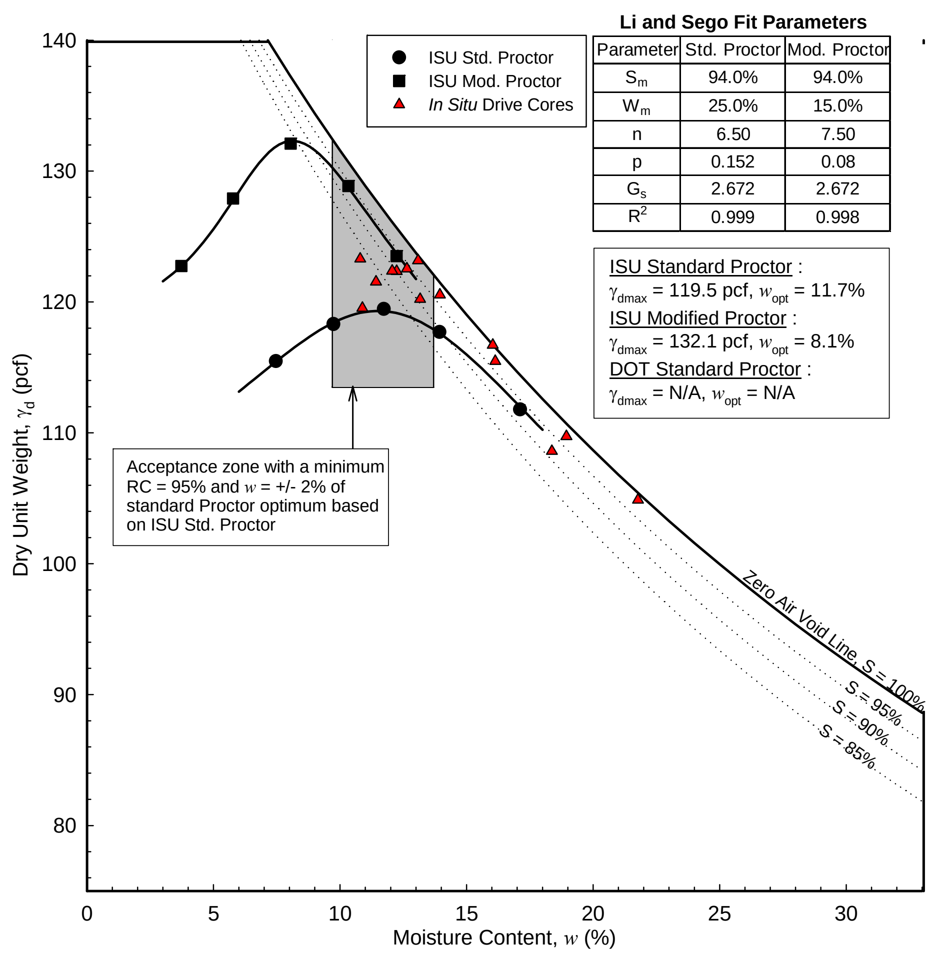


Figure 64. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Linn County #1 Project TB4 (N/A: not available)

The results from TB4 showed an average DCP-CBR of about 3.0 in the top 8 in. and 3.5 in top 12 in, which is considered as very poor according to SUDAS. The CBR value varies a lot while depth goes deeper which indicate this test bed is not uniformly compacted. Point 13 have a very low CBR from 19.5 to 29 inch, point 15 have a very low CBR from 10 to 13.5 inch, and point 9 have a very low CBR from 26.5 to 30 inch, which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 65).

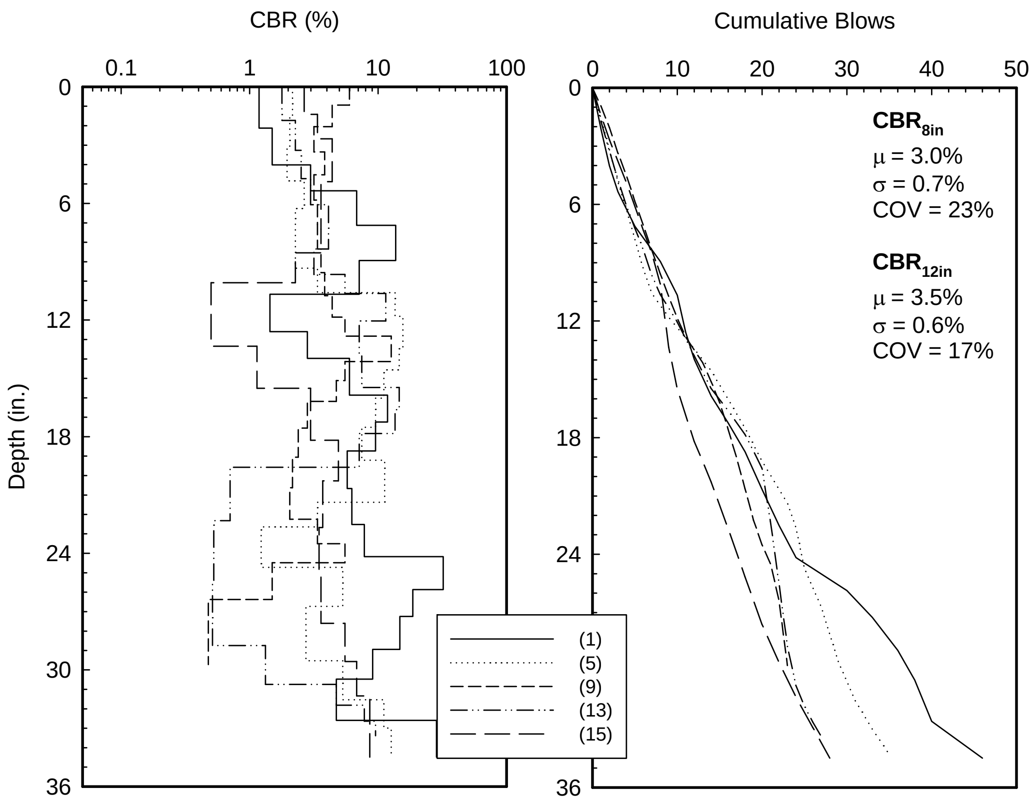


Figure 65. DCP-CBR profile at Linn County #1 Project TB4

Based on 15 drive core tests performed from TB4, all 15 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 0.2% to +1.1% of w_{opt}, with an average of about +0.6 % from all test points. The dry unit weight of the material varied from 99.0% to 103.5% of standard Proctor γ_{dmax}, with an average of about 101.4% from all test points (Figure 66).

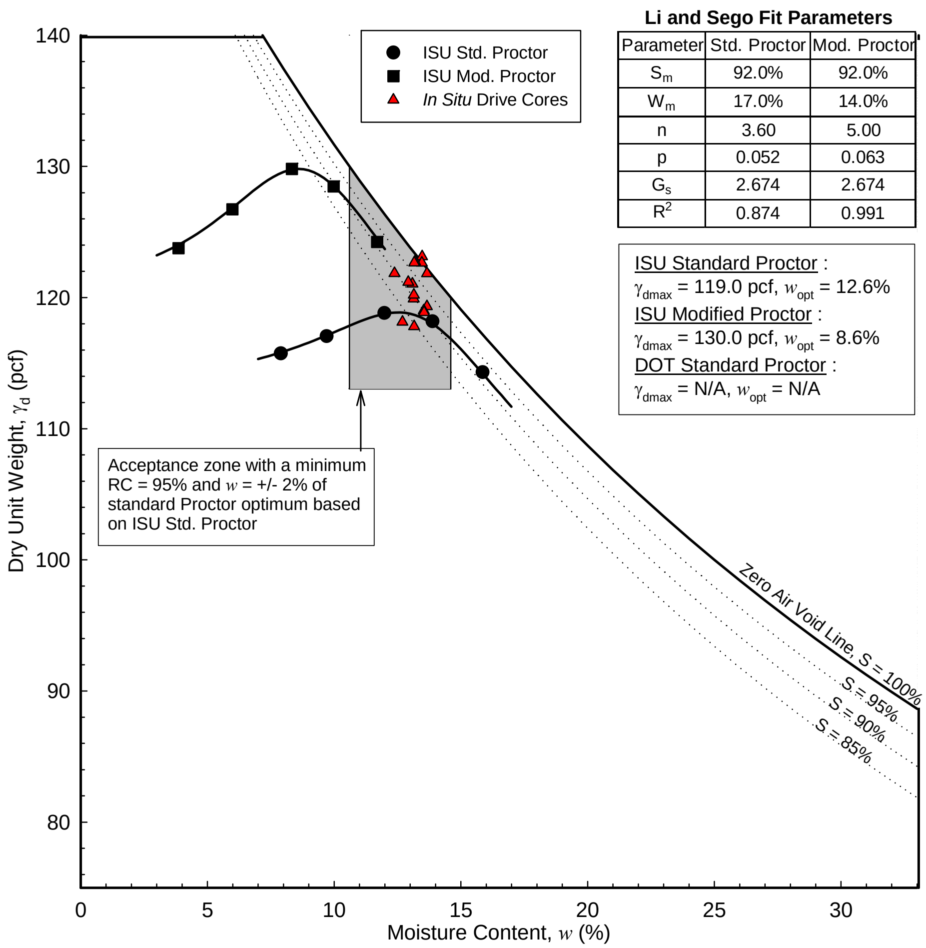


Figure 66. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Linn County #1 Project TB5 (N/A: not available)

The results from TB5 showed an average DCP-CBR of about 2.3 in the top 8 in. and 2.6 in top 12 in, which is considered as very poor according to SUDAS. The CBR value does not varies a lot for each point, which indicate the compaction is relatively uniform (Figure 67)

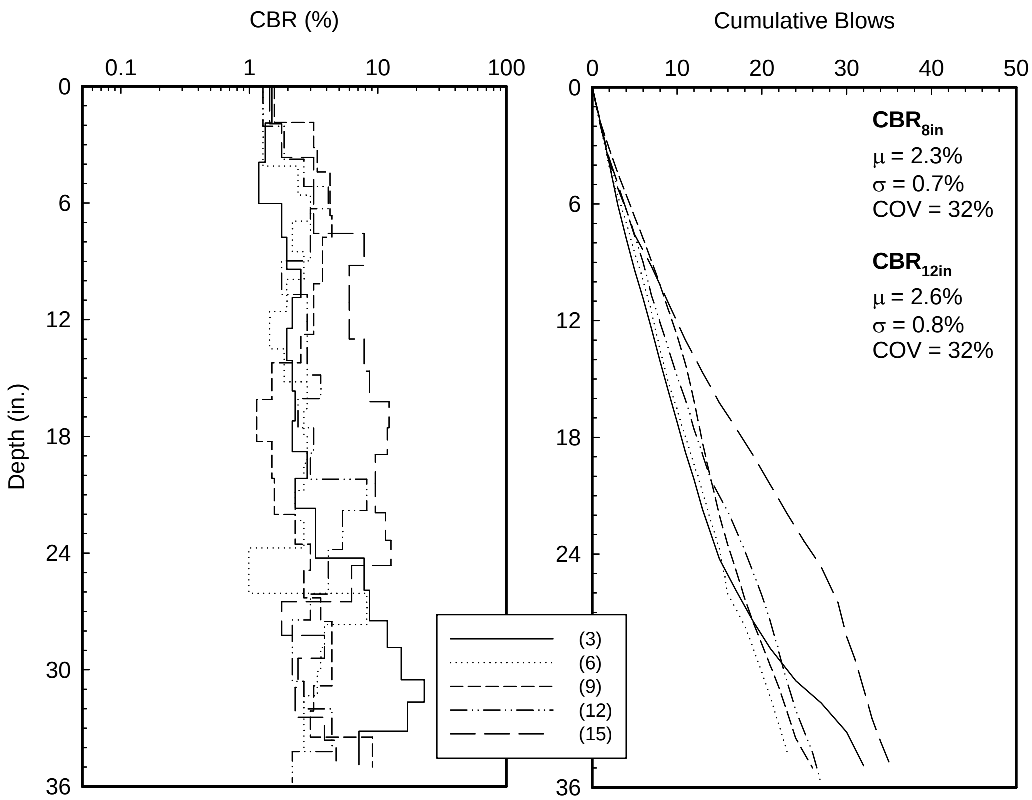


Figure 67. DCP-CBR profile at Linn County #1 Project TB5

Summary of results

Table 31 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was relatively wet.

- The moisture content of the material in TB1 varied from - 1.5% to +1.3% of w_{opt}, with an average of about -0.5% from all test points. The dry unit weight of the material varied from 96.5% to 107% of standard Proctor γ_{dmax}, with an average of about 103.5% from all test points.
- The results from TB1 showed an average DCP-CBR of about 7.6 in the top 8 in. and 6.9 in top 12 in, which is considered as poor.
- The moisture content of the material in TB3 varied from - 1.8 to +2.8% of w_{opt}, with an average of about +0.6% from all test points. The dry unit weight of the material varied from 92.5% to 104% of standard Proctor γ_{dmax}, with an average of about 99.2% from all test points.
- The results from TB3 showed an average DCP-CBR of about 4.3 in the top 8 in. and 3.4 in top 12 in, which is considered as very poor.
- The moisture content of the material in TB4 varied from -0.9% to +10.1% of w_{opt}, with an average of about +2.5 % from all test points. The dry unit weight of the material varied from 87.8% to 103.2% of standard Proctor γ_{dmax}, with an average of about 98.8% from all test points.
- The results from TB4 showed an average DCP-CBR of about 3.0 in the top 8 in. and 3.5 in top 12 in, which is considered as very poor.
- The moisture content of the material in TB5 varied from - 0.2% to +1.1% of w_{opt}, with an average of about +0.6 % from all test points. The dry unit weight of the material varied from 99.0% to 103.5% of standard Proctor γ_{dmax}, with an average of about 101.4% from all test points.

- The results from TB5 showed an average DCP-CBR of about 2.3 in the top 8 in. and 2.6 in top 12 in, which is considered as very poor.

Table 31. Summary of field results for Linn County #1

Parameter	Linn County #1 TB1	Linn County #1 TB2	Linn County #1 TB3	Linn County #1 TB4	Linn County #1 TB5
	6/6/2014	7/8/2014	7/15/2014	8/1/2014	9/8/2014
Relative Compaction					
Average (%)	103.5	N/A	99.2	98.8	101.4
Range (%)	96.5 to 107.0	N/A	92.5 to 104.0	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 (%)	-0.5	N/A	0.6	2.5	0.6
Range (%)	-1.5 to +1.3	N/A	-1.8 to +2.8	-0.9 to +10.1	-0.2 to +1.1
Standard Deviation (%)	0.68	N/A	1.13	3.31	0.36
COV (%)	-138	N/A	204	131	59
$CBR_{8\ in.}$					
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

Project 4. Linn County #2

Project overview and observations

The project site was visited one time. Figure 68 to Figure 70 shows pictures of the construction equipment used on the site during embankment construction. Caterpillar 740B dump trucks were used to collect fill materials from cuts and borrow areas for placement in fill areas. A disk was used to dry embankment materials. A Caterpillar dozer was used to spread loose lift material. A pull behind sheepsfoot roller was used for soil compaction. Some other field observations with Contractors QC, Iowa DOT QA, and ISU researchers were conducting the in situ test, is shown in Figure 71, Figure 72, and Figure 73.

Field observations indicated that the fill material obtained from the borrow area was in a suitable humidity. Field testing was conducted on 6/6/2014 on test sections that were passed in the same day by the Iowa DOT field inspector. Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 5 locations during the visit.

Figure 68. Caterpillar 740 dump truck used to place loose fill materials and Caterpillar dozer was used to spread loose lift material

Figure 69. Sheepsfoot roller used for soil compaction

Figure 70. Disk appear on site used to dry materials

Figure 71. Contractor was conducting QC tests

Figure 72. DOT engineer was conducting QA tests

Figure 73. ISU in situ drive cylinder test

Field and laboratory test results

Laboratory Proctor test results for materials collected from this test bed are presented in Figure 74. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figure. The figure also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} and γ_{dmax} that was being used by the DOT for QC/QA; (c) ISU field test results from dive core testing; and (d) contractor QC and Iowa DOT QA test results.

Based on 15 drive core tests performed from TB1, all 15 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 0.5% to +1.4% of w_{opt}, with an average of about +0.5% from all test points. The dry unit weight of the material varied from 96.7% to 100.9% of standard Proctor γ_{dmax}, with an average of about 98.7% from all test points. 3 out of 4 QC test results and all 3 Iowa DOT QA results fell in the acceptable zone identified from ISU Proctor testing (Figure 74).

DCP-CBR profiles and cumulative blows with depth profiles from this test bed are presented in Figure 75. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 3.7 in the top 8 in. and 4.1 in the top 12 in, which is considered as very poor according to SUDAS. The CBR value varies a lot below 13 in. deep. Point 14 have a low CBR from 15.5 to 27.5 inch which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 75).

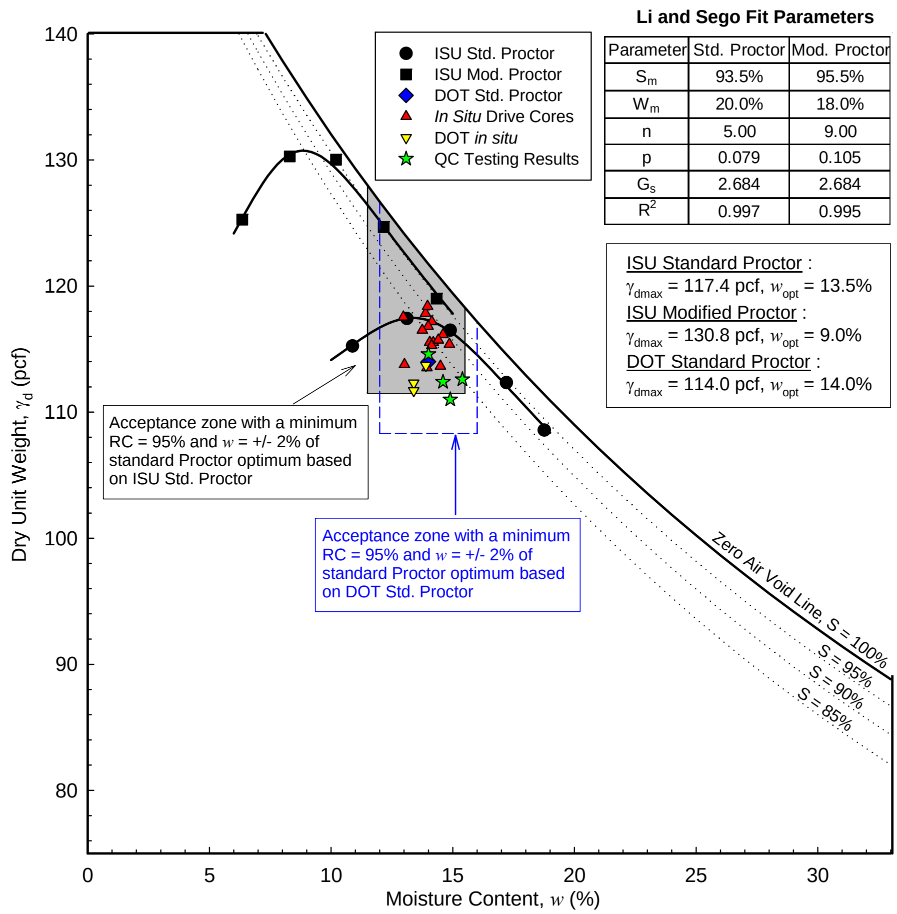


Figure 74. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Linn County #1 Project

From Figure 75, five DCP points shows an average CBR ratio bigger than 1, the CBR value varies a little bit while the depth went deeper. That indicates the overall embankment have a good shear strength and stiffness quality.

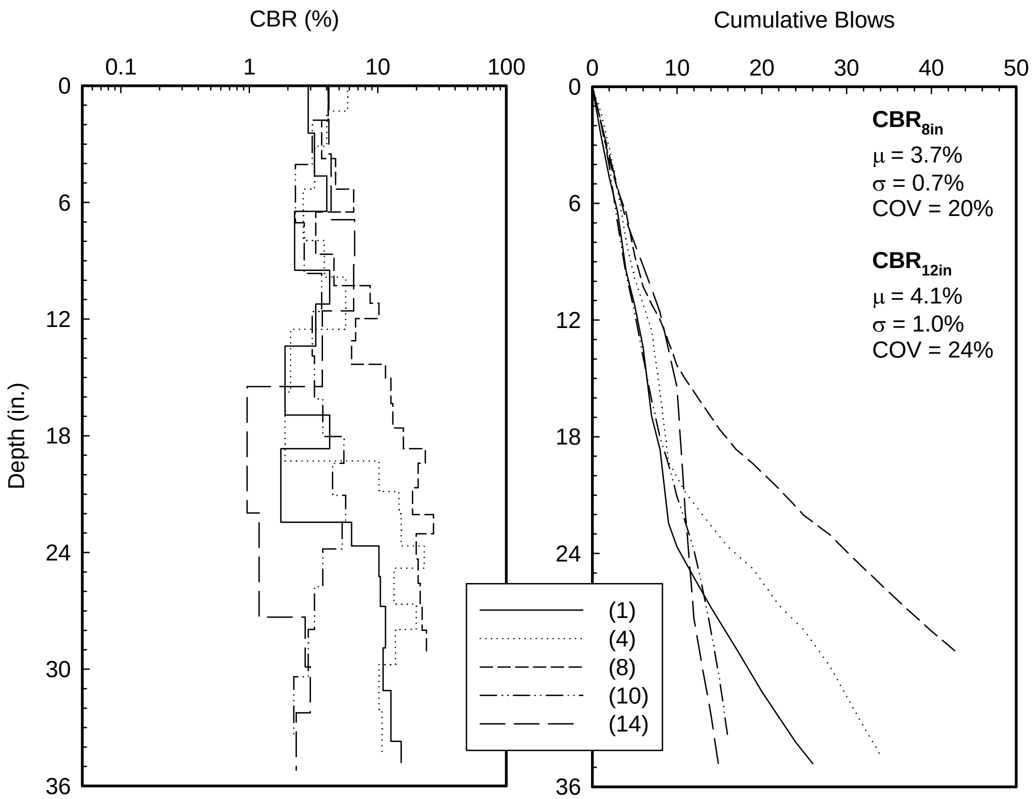


Figure 75. DCP-CBR profile at Linn County #1 Project

Summary of results

Table 32 Summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was in a suitable humidity.
- The moisture content of the material in situ varied from - 0.5% to +1.4% of w_{opt}, with an average of about +0.5% from all test points. The dry unit weight of the material varied from 96.7% to 100.9% of standard Proctor γ_{dmax}, with an average of about 98.7% from all test points.

- The results from TB1 showed an average DCP-CBR of about 3.7 in the top 8 in. and 4.1 in the top 12 in, which is considered as very poor.

Table 32. Summary of field results for Linn County #2

Parameter	Linn County #1
	8/4/2014
Relative Compaction	
Average Relative compaction (%)	98.7
Range of Relative compaction (%)	96.7 to 100.9
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

Project 5. Mills County

Project overview and observations

The project site was visited twice at the same day. Figure 76 to Figure 79 shows pictures of the construction equipment used on the site during embankment construction. Caterpillar 621E scraper were used to collect fill materials from cuts and borrow areas for placement in fill areas. A disk was used to dry embankment materials. A Caterpillar D6R dozer was used to spread loose lift material. A pull behind sheepsfoot roller was used for soil compaction. Some other field observations with extremely wet materials was observed in the middle of the construction site is shown in Figure 80. ISU researchers in situ test process is shown in Figure 81.

Field observations indicated that the fill material obtained from the borrow area was relatively wet. Field testing was conducted on 6/26/2014 on test sections that TB1 passed the previous day and TB2 passed at the same day by the Iowa DOT field inspector. Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 6 locations during the visit. According to the contractor, the site have rained the night before ISU field test.

Figure 76. Caterpillar 621E scraper used to collect and place loose fill materials

Figure 77. Caterpillar D6R dozer used to control lift thickness

Figure 78. Disk presented on site without pulling machine

Figure 79. Sheepsfoot roller used for soil compaction

Figure 80. Very wet materials in the center of the construction site

Figure 81. ISU in situ drive cylinder test

Field and laboratory test results

Laboratory Proctor test results for materials collected from the two test beds are presented in Figure 82 and Figure 84. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} and γ_{dmax} that was being used by the DOT for QC/QA; and (c) ISU field test results from dive core testing.

Based on 15 drive core tests performed from TB1, zero fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from +3.1% to +11.6% of w_{opt}, with an average of about +6.1 % from all test points. The dry unit weight of the material varied from 84.3% to 98.3% of standard Proctor γ_{dmax}, with an average of about 92.4% from all test points. All in situ drive core cylinder results are wetter then acceptable range, which is reasonable due to the raining condition the night before test (Figure 82).

DCP-CBR profiles and cumulative blows with depth profiles from the two test beds are presented in Figure 83 and Figure 85. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 2.9 in the top 8 in. and 2.6 in the top 12 in, which is considered as very poor according to SUDAS. The CBR value does not varies much until 24 in. deep (Figure 83).

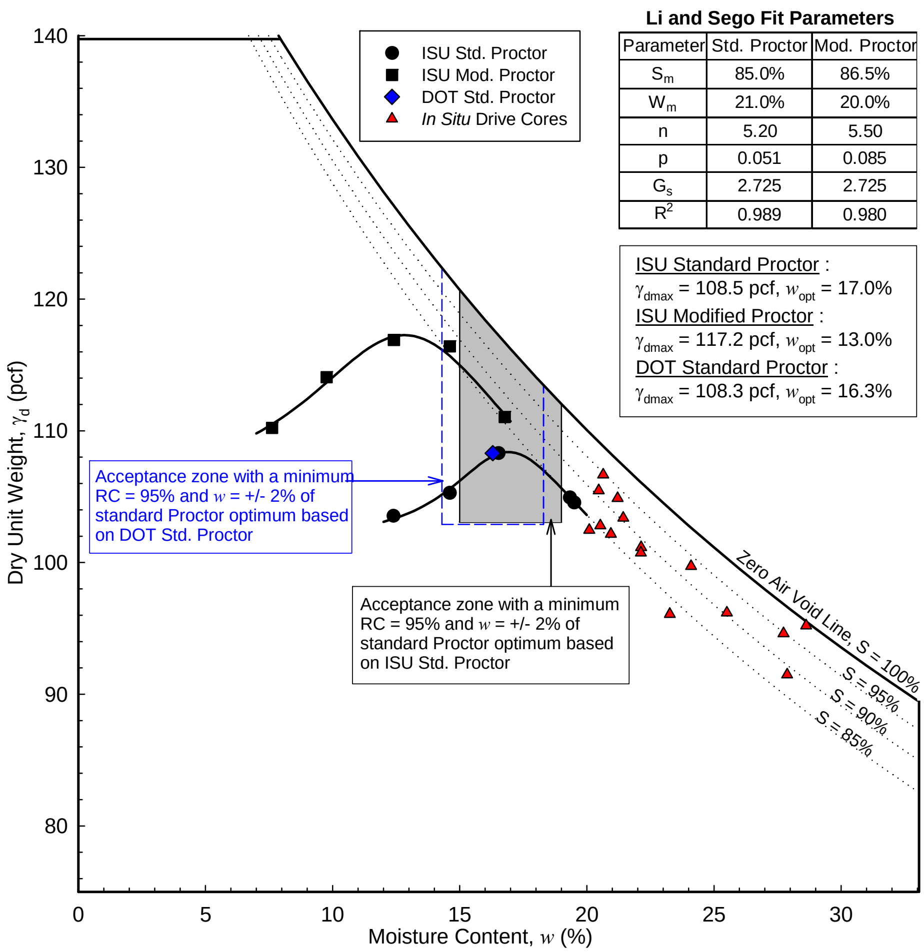


Figure 82. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Mills County Project TB1

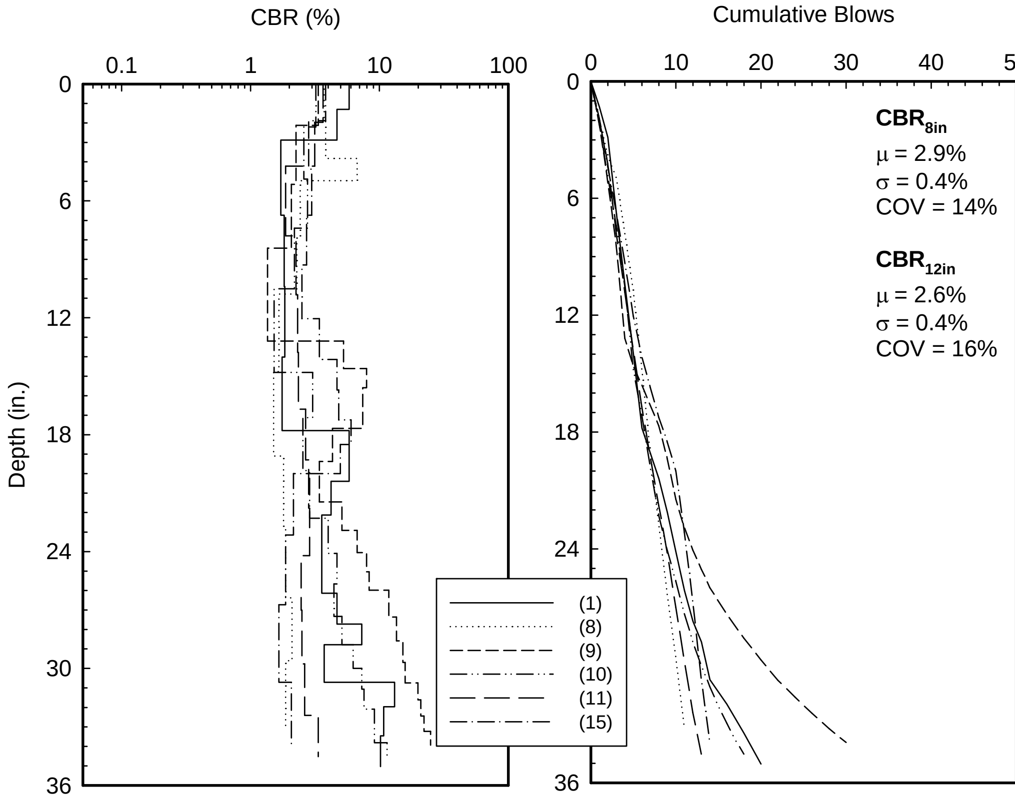


Figure 83. DCP-CBR profile at Mills County Project TB1

Based on 15 drive core tests performed from TB2, 5 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from -4.0% to +5.1% of w_{opt}, with an average of about +1.6 % from all test points. The dry unit weight of the material varied from 94.5% to 101.4% of standard Proctor γ_{dmax}, with an average of about 97.6% from all test points (Figure 84).

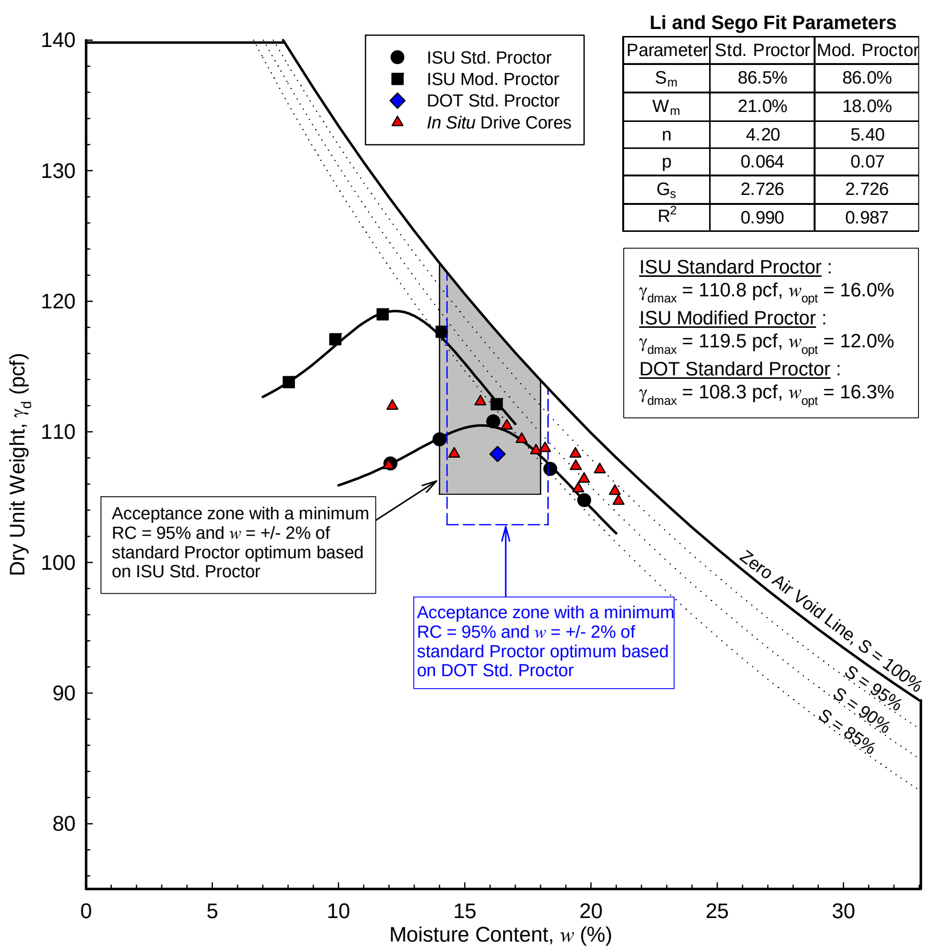


Figure 84. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Mills County Project TB2

The results from TB2 showed an average DCP-CBR of about 6.8 in the top 8 in. and 6.2 in the top 12 in, which is considered poor according to SUDAS. Point 8 have a low CBR value from 10 to 15 inch, and point 12 have a low CBR value from 24 to 28 in., which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 85).

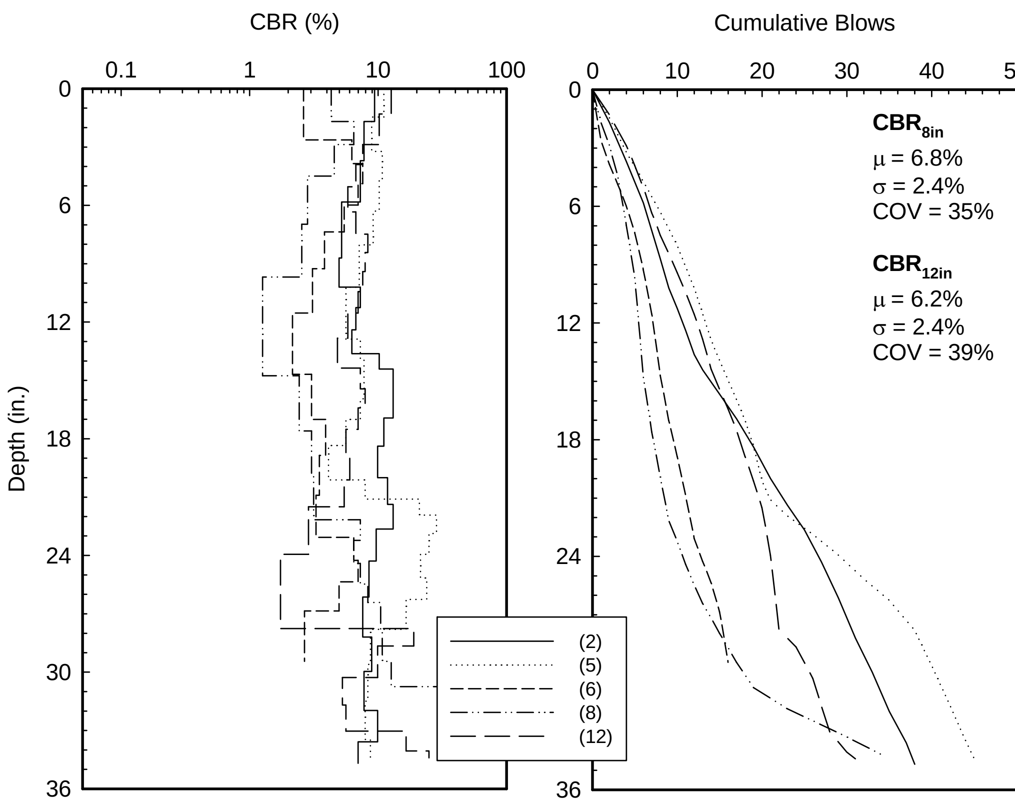


Figure 85. DCP-CBR profile at Mills County Project TB2

Summary of results

Table 33 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was relatively wet.
- The moisture content of the material in TB1 varied from +3.1% to +11.6% of w_{opt}, with an average of about +6.1 % from all test points. The dry unit weight of the material varied from 84.3% to 98.3% of standard Proctor γ_{dmax}, with an average of about 92.4% from all test points.
- The results from TB1 showed an average DCP-CBR of about 2.9 in the top 8 in. and 2.6 in the top 12 in, which is considered as very poor.
- The moisture content of the material in TB2 varied from -4.0% to +5.1% of w_{opt}, with an average of about +1.6 % from all test points. The dry unit weight of the material varied from 94.5% to 101.4% of standard Proctor γ_{dmax}, with an average of about 97.6% from all test points.
- The results from TB2 showed an average DCP-CBR of about 6.8 in the top 8 in. and 6.2 in the top 12 in, which is considered poor.

Table 33. Summary of field results for Mills County

Parameter	Mills County TB1	Mills County TB2
	6/26/2014	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

Table 33. Continued

Parameter	Mills County TB1	Mills County TB2
	6/26/2014	6/26/2014
$\Delta w\% = w_{field}\% - w_{opt}\%$		
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

Project 6. Pottawattamie County

Project overview and observations

The project site was visited twice. Figure 86 to Figure 89 shows pictures of the construction equipment used on the site during embankment construction. A Caterpillar dozer was used to spread loose lift material. A disk was used to dry embankment materials. A 851B dozer and sheepsfoot roller was used for soil compaction. Dynapac CA250-II vibratory smooth drum roller used for soil compaction. ISU researchers in situ test process is shown in Figure 81.

Field observations indicated that the fill material obtained from the borrow area was in a suitable humidity. Field testing was conducted on 7/2/2014 and 7/10/2014 on test sections that were passed either on the same day or the previous day by the Iowa DOT field inspector.

Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 5 locations during two visits.

Figure 86. Caterpillar dozer used to control lift thickness

Figure 87. Caterpillar 851B dozer with sheepsfoot roller wheel used for soil compaction

Figure 88. Dynapac CA250-II vibratory smooth drum roller used for soil compaction

Figure 89. Disk used to dry embankment materials

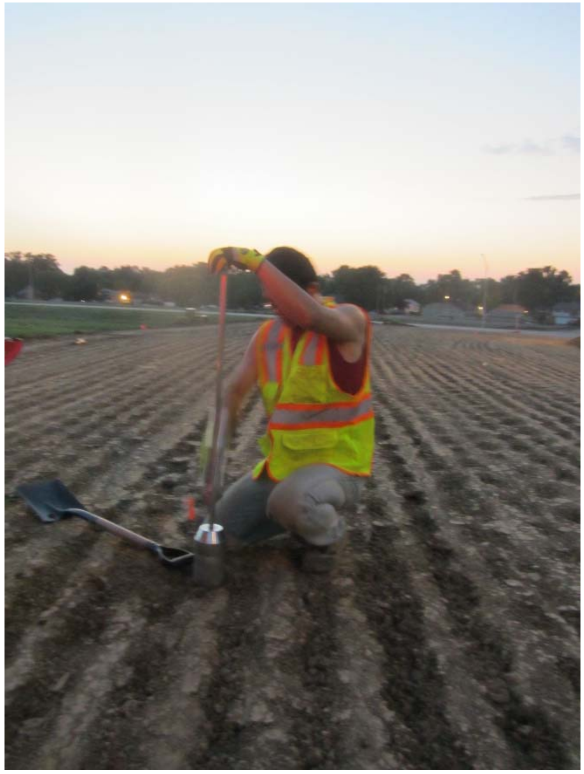


Figure 90. ISU in situ drive cylinder test

Field and laboratory test results

Laboratory Proctor test results for materials collected from the two test beds are presented in Figure 91 and Figure 93. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} and γ_{dmax} that was being used by the DOT for QC/QA; (c) ISU field test results from dive core testing; and (d) contractor QC and Iowa DOT QA test results.

Based on 14 drive core tests performed from TB1, only 7 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from

- 1.6% to +6.1% of w_{opt}, with an average of about +1.4 % from all test points. The dry unit weight of the material varied from 90.3% to 101.7% of standard Proctor γ_{dmax}, with an average of about 96.9% from all test points. 3 out of 5 QC test results and 1 out of 2 Iowa DOT QA results fell in the acceptable zone identified from ISU Proctor testing. (Figure 91).

DCP-CBR profiles and cumulative blows with depth profiles from the two test beds are presented in Figure 92 and Figure 94. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 6.0 in the top 8 in. and 5.4 in the top 12 in, which is considered as poor according SUDAS. CBR value varies a lot between different DCP points while the depth went deeper which indicate this embankment section does not have a uniform compaction, point 2 and point 4 have low CBR value which is an indication of “uncompacted” fill or the oreo-cookie effect (Figure 92).

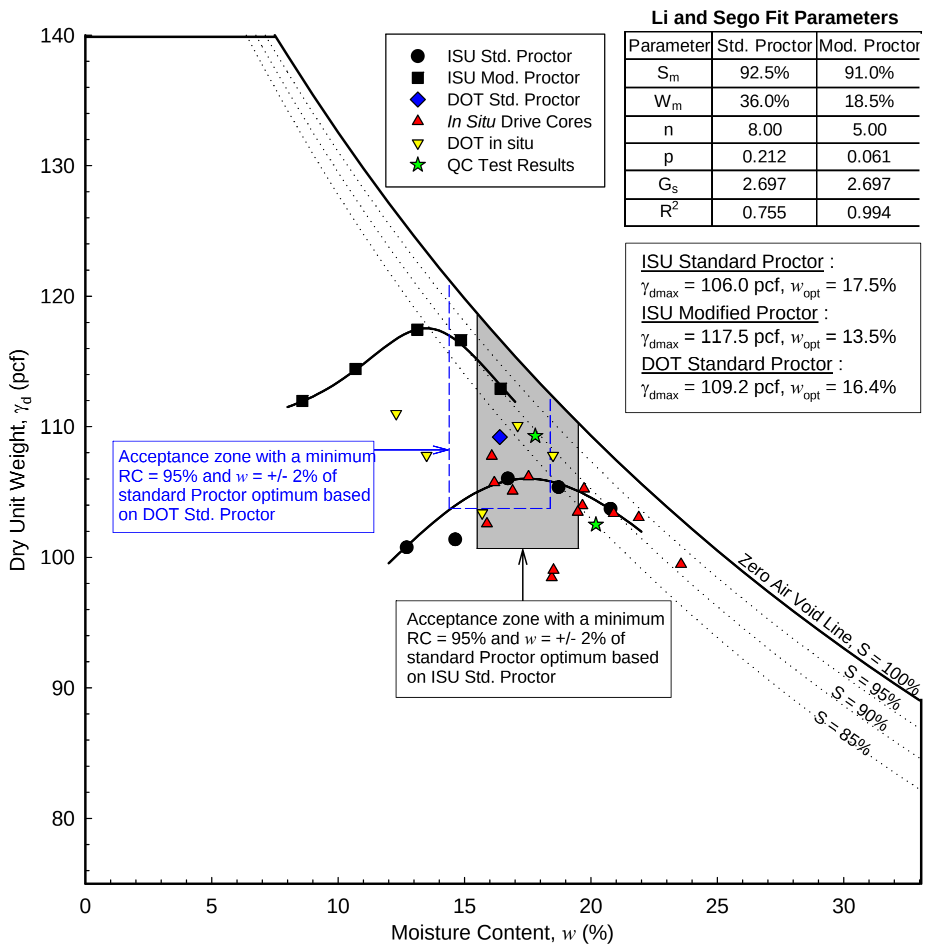


Figure 91. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Pottawattamie County Project TB1

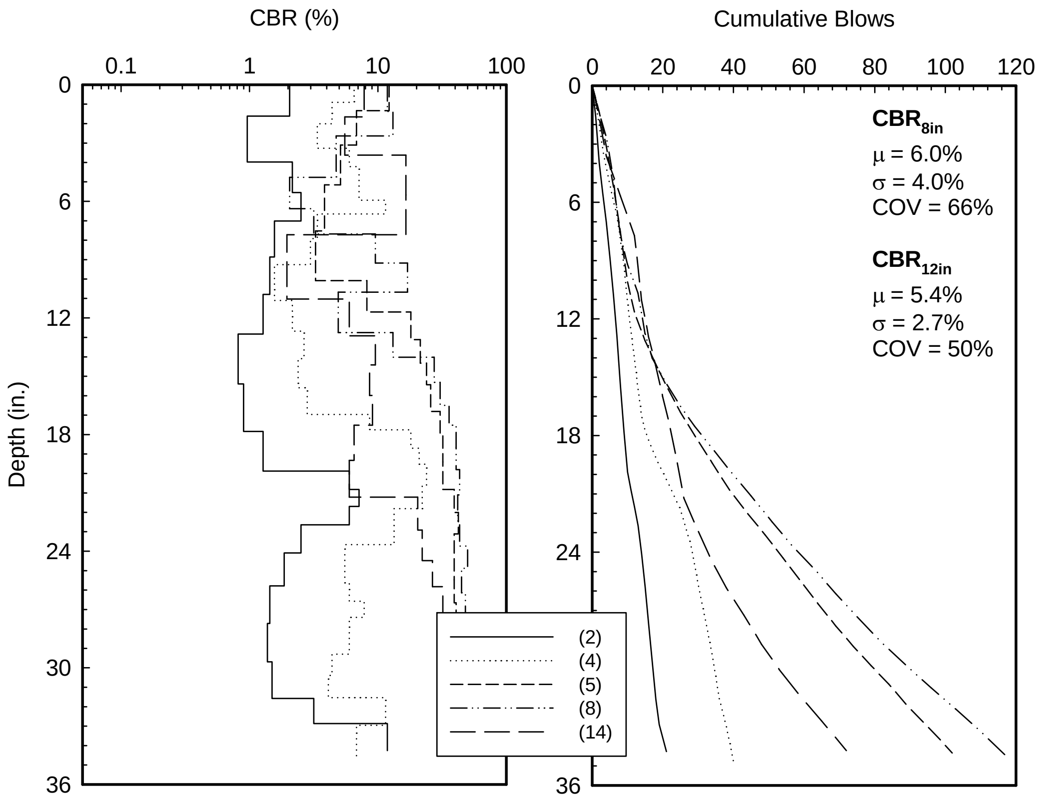


Figure 92. DCP-CBR profile at Pottawattamie County Project TB1

Based on 15 drive core tests performed from TB2, 9 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 1.6% to +6.1% of w_{opt}, with an average of about +1.4 % from all test points. The dry unit weight of the material varied from 90.3% to 101.7% of standard Proctor γ_{dmax}, with an average of about 96.9% from all test points. 3 out of 5 QC test results and 0 out of 5 Iowa DOT QA results fell in the acceptable zone identified from ISU Proctor testing. The Iowa DOT QA test which marked as a yellow triangle are conducted one night after compaction, so is reasonable that the results are drier than the accept range. (Figure 93).

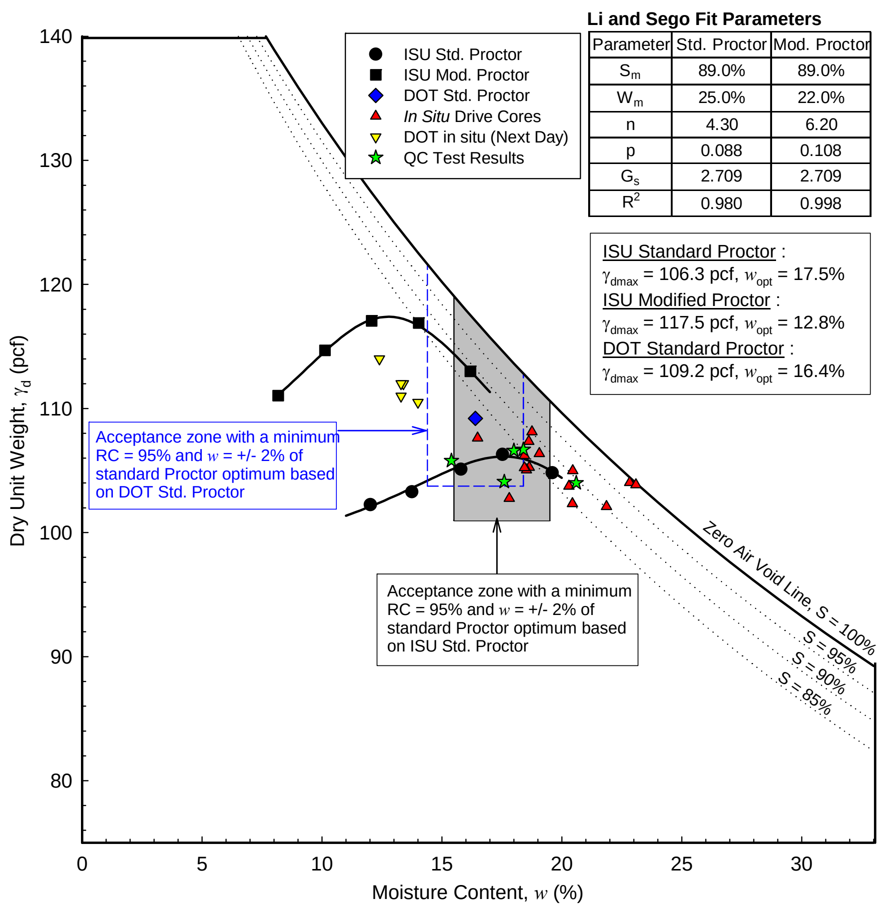


Figure 93. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Pottawattamie County Project TB2

The results from TB2 showed an average DCP-CBR of about 6.0 in the top 8 in., which is considered as poor, and 4.4 in the top 12 in, which is considered as very poor according SUDAS. Point 12 have a very low CBR value from 6.5 to 16 in., and point 1 have a very low CBR value from 5 to 8.5 in., and point 4 have a very low CBR value from 16.5 to 20 in., which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 94)

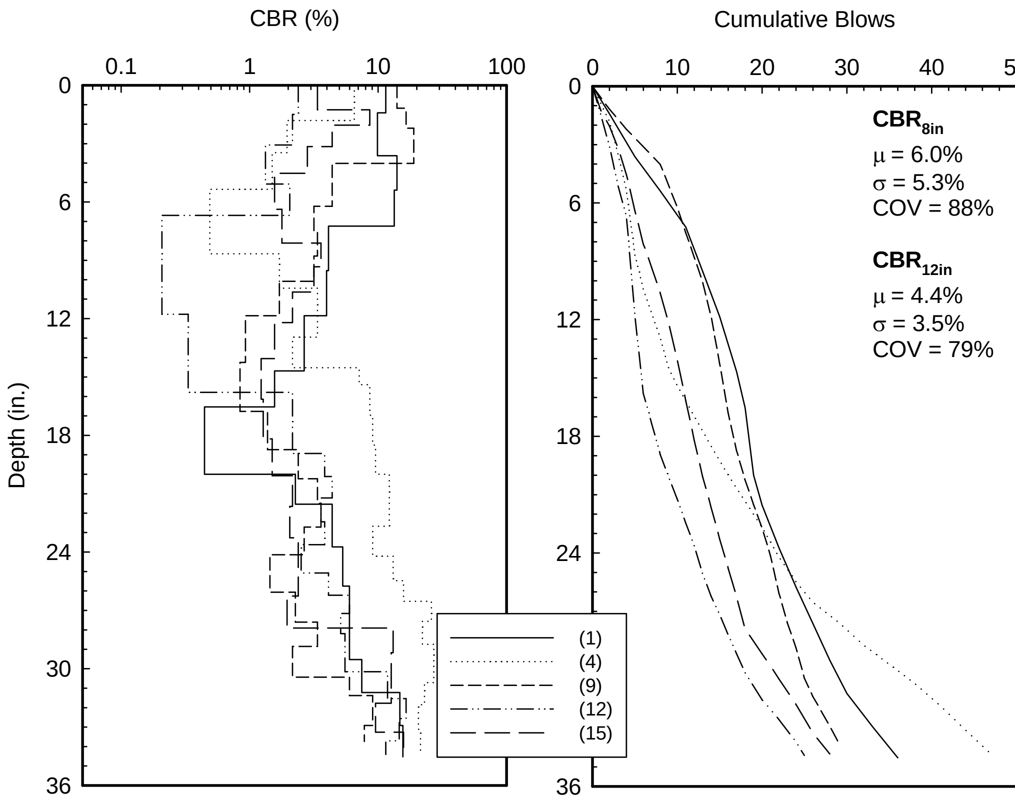


Figure 94. DCP-CBR profile at Pottawattamie County Project TB2

Summary of results

Table 34 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was in a suitable humidity.
- The moisture content of the material in TB1 varied from - 1.6% to +6.1% of w_{opt}, with an average of about +1.4 % from all test points. The dry unit weight of the material varied from 90.3% to 101.7% of standard Proctor γ_{dmax}, with an average of about 96.9% from all test points.
- The results from TB1 showed an average DCP-CBR of about 6.0 in the top 8 in. and 5.4 in the top 12 in, which is considered as poor.
- The moisture content of the material in TB2 varied from - 1.6% to +6.1% of w_{opt}, with an average of about +1.4 % from all test points. The dry unit weight of the material varied from 90.3% to 101.7% of standard Proctor γ_{dmax}, with an average of about 96.9% from all test points.
- The results from TB2 showed an average DCP-CBR of about 6.0 in the top 8 in., which is considered as poor, and 4.4 in the top 12 in, which is considered as very poor.

Table 34. Summary of field results for Pottawattamie County

Parameter	**Pottawattamie County TB1**	**Pottawattamie County TB2**
	7/2/2014	7/10/2014
Relative Compaction		
Average Relative compaction (%)	96.9	98.8
Range of Relative compaction (%)	90.3 to 101.7	96.1 to 101.7
Standard Deviation (%)	0.03	0.02
COV (%)	3	2
$\Delta w\% = w_{field}\% - w_{opt}\%$		
Average Δw (%)	1.4	1.8
Range of Δw (%)	-1.6 to +6.1	-1.3 to +5.3
Standard Deviation (%)	2.23	0.02
COV (%)	162	105

Table 34. Continued

Parameter	Pottawattamie County TB1	Pottawattamie County TB2
	7/2/2014	7/10/2014
$CBR_{8\ in.}$		
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

Project 7. Woodbury County I-29

Project overview and observations

The project site was visited three times. Figure 95 to Figure 97 shows pictures of the construction equipment used on the site during embankment construction. Dump trucks were used to collect fill materials from cuts and borrow areas for placement in fill areas. A Caterpillar D6T dozer was used to spread loose lift material. Caterpillar CS56B vibratory smooth drum roller used for soil compaction. No disk presented while ISU researchers on site. Some other field observations with seepage observed in the construction site is shown in Figure 98, and ISU researchers in situ test process is shown in Figure 99 and Figure 100.

Field observations indicated that the fill material obtained from the borrow area was extremely wet. Dump truck had to be pushed out of the mud by a dozer after it tipped over because the soil was too soft. Field testing was conducted on 7/9/2014, 7/10/2014, and 8/7/2014 on test sections that were passed either on the same day or the previous day by the

Iowa DOT field inspector. Testing involved DCP testing at 5 to 7 locations during three visits.

Figure 95. Dump truck used to place loose fill materials

Figure 96. Caterpillar D6T dozer used to control lift thickness

Figure 97. Caterpillar CS56B vibratory smooth drum roller used for soil compaction

Figure 98. Seepage occurred in the construction site

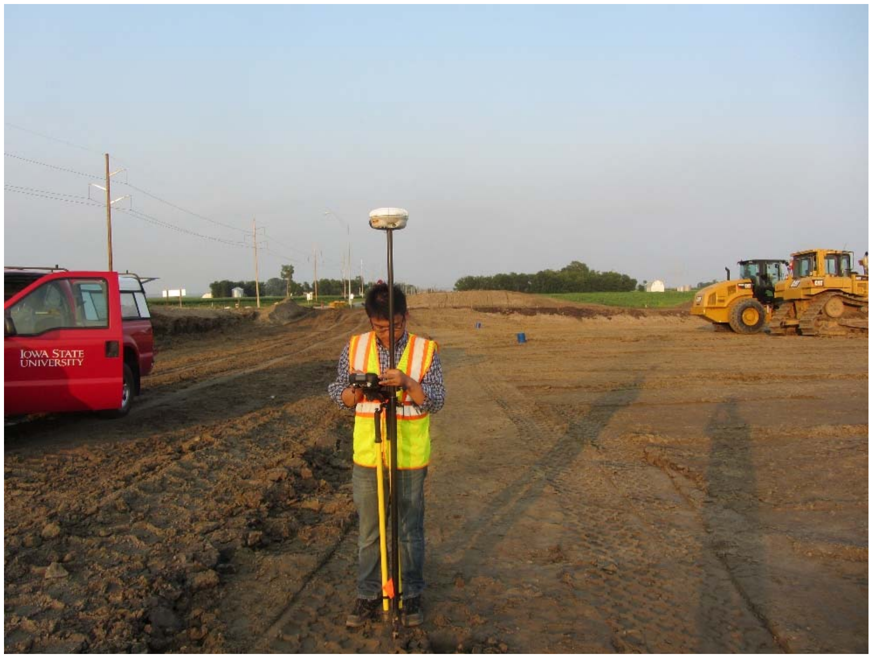


Figure 99. ISU GPS testing

Figure 100. ISU DCP testing

Field and laboratory test results

Laboratory Proctor test results for materials collected from the three test beds are presented in Figure 101, Figure 103, and Figure 105. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; and (b) the w_{opt} that was being used by the DOT for QC/QA.

Based on 15 moisture content tests performed from TB1, the results varied from -4.1% to +11.8% of w_{opt}, with an average of about 3.5% from all test points (Figure 101).

DCP-CBR profiles and cumulative blows with depth profiles from the three test beds are presented in Figure 102, Figure 104, and Figure 106. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 2.6 in the top 8 in. and 3.5 in the top 12 in, which is considered as poor according SUDAS. CBR value improves while the depth goes deeper, and it reaches 10 after 18 in. deep (Figure 102).

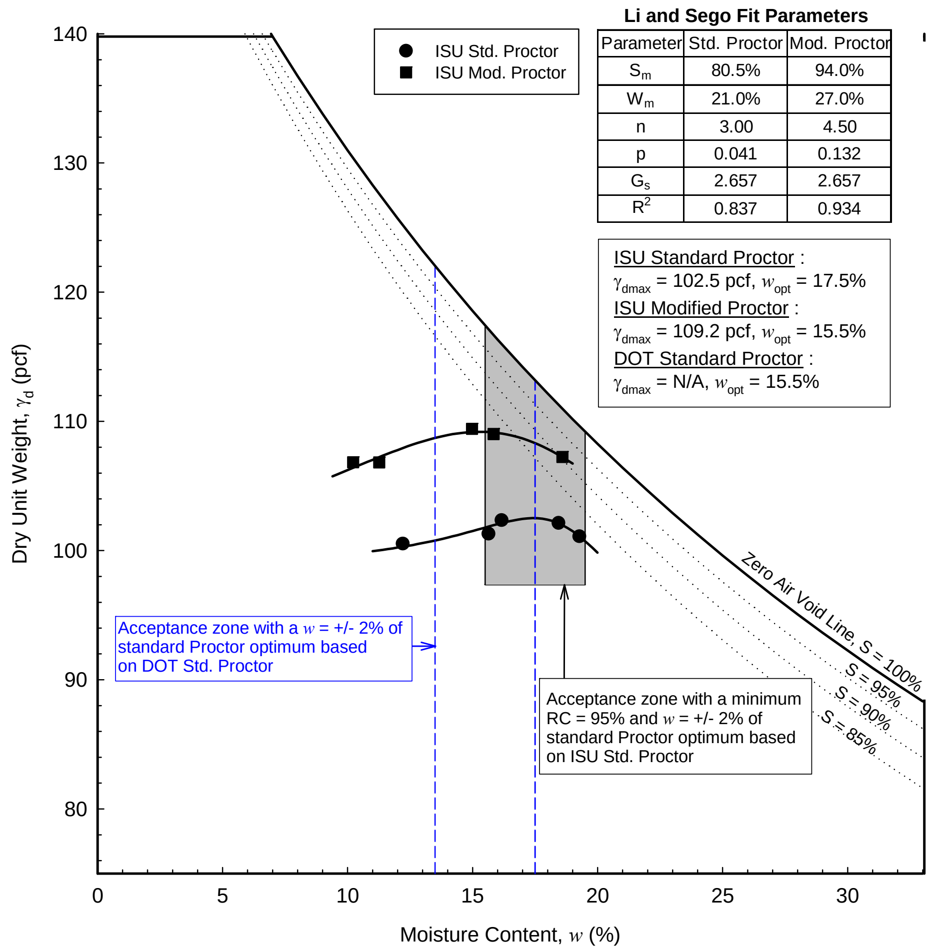


Figure 101. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Woodbury County I-29 Project TB1 (N/A: not available)

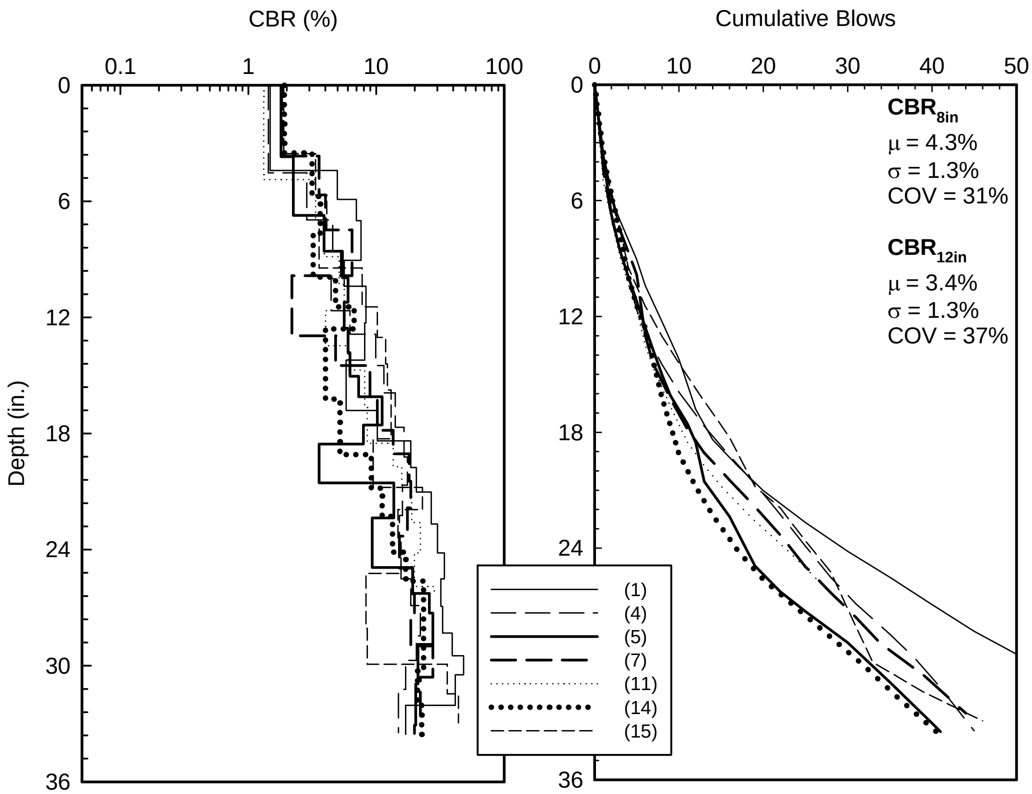


Figure 102. DCP-CBR profile at Woodbury County I-29 Project TB1

Based on 15 moisture content tests performed from TB2, the results varied from +3.9% to +8.9% of w_{opt}, with an average of about +6.9% from all test points (Figure 103).

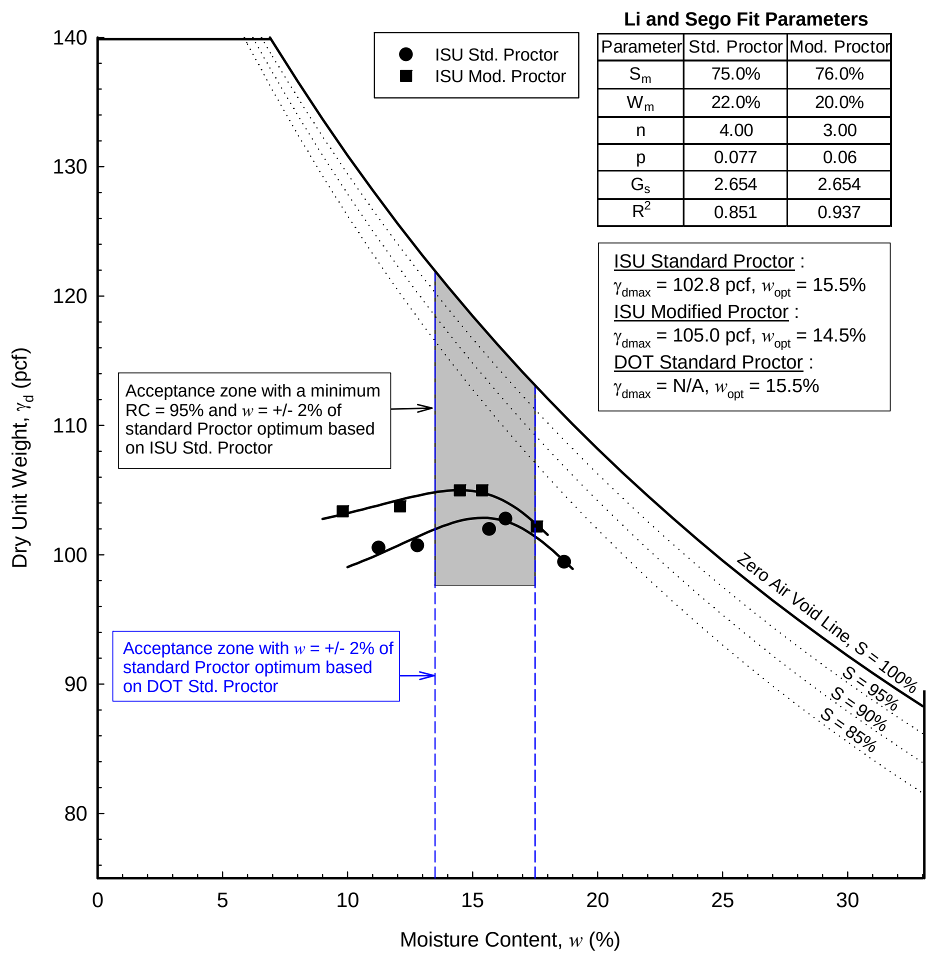


Figure 103. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Woodbury County I-29 Project TB2 (N/A: not available)

The results from TB2 showed an average DCP-CBR of about 1.5 in the top 8 in. and 12 in, which is considered as very poor according SUDAS. Point 5 and 14 have a very low CBR value compare to other points, which indicate the compaction is not uniform in this area (Figure 104).

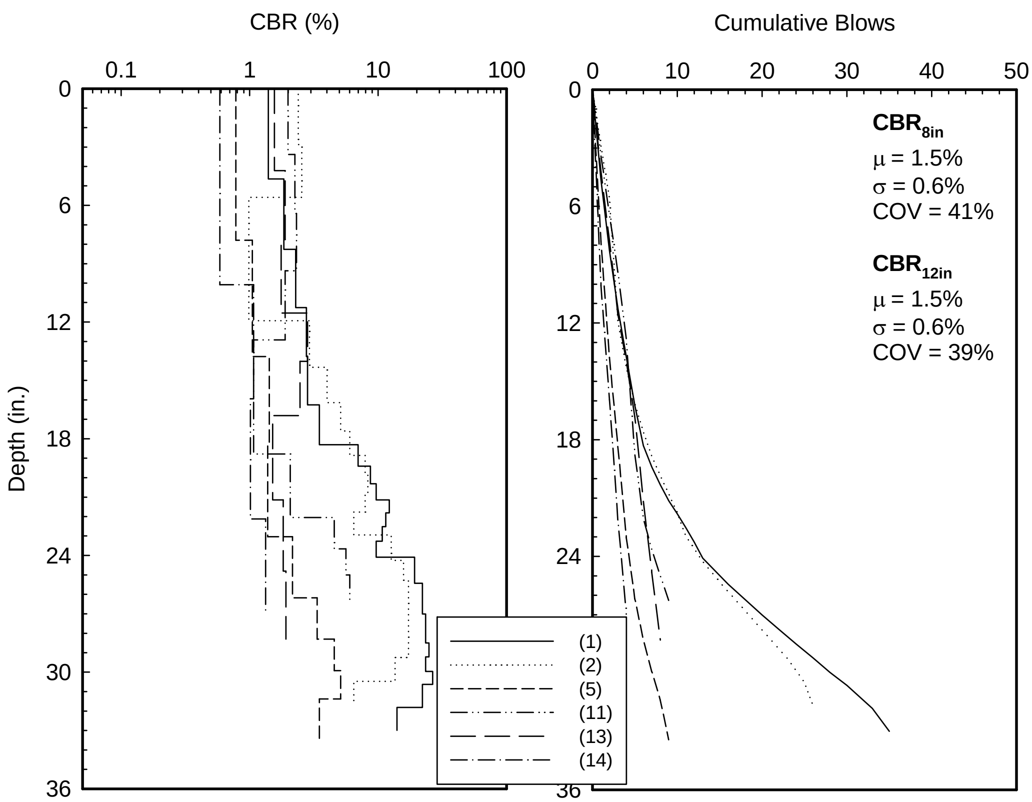


Figure 104. DCP-CBR profile at Woodbury County I-29 Project TB2

Based on 15 moisture content tests performed from TB1, the results varied from -1.1% to +2.1% of w_{opt}, with an average of about +0.2% from all test points (Figure 105).

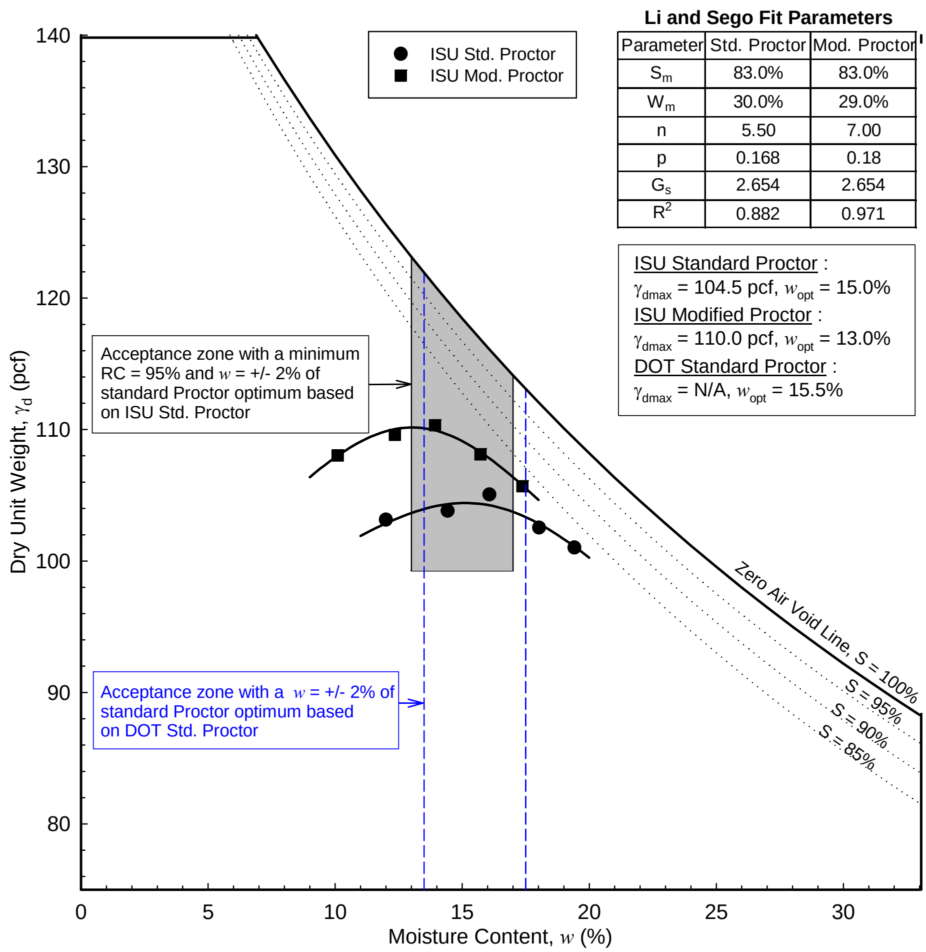


Figure 105. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Woodbury County I-29 Project TB3 (N/A: not available)

The results from TB2 showed an average DCP-CBR of about 3.0 in the top 8 in. and 3.9 in the top 12 in, which is considered as very poor according SUDAS. CBR value varies a lot from 5 to 27 in., which indicate the compaction is not uniform in these layers (Figure 106).

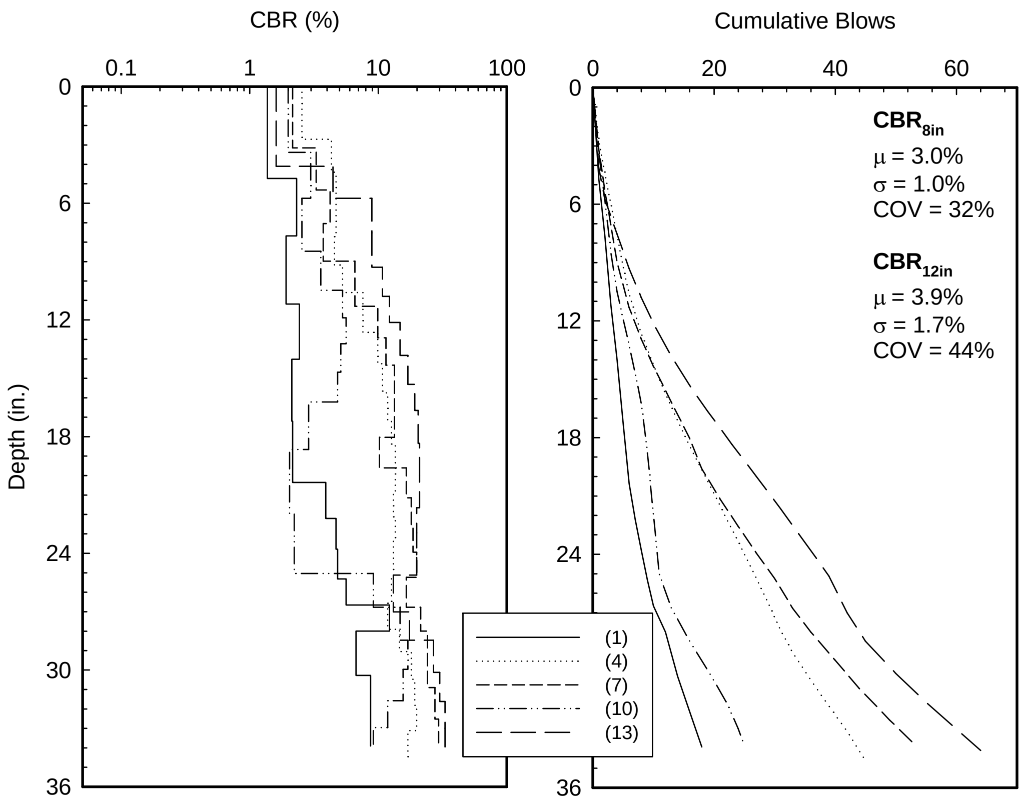


Figure 106. DCP-CBR profile at Woodbury County I-29 Project TB3

Summary of results

Table 35 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was extremely wet.

- Based on 15 moisture content tests performed from TB1, the results varied from - 4.1% to +11.8% of w_{opt}, with an average of about 3.5% from all test points.
- The results from TB1 showed an average DCP-CBR of about 2.6 in the top 8 in. and 3.5 in the top 12 in, which is considered as poor.
- Based on 15 moisture content tests performed from TB2, the results varied from + 3.9% to +8.9% of w_{opt}, with an average of about +6.9% from all test points.
- The results from TB2 showed an average DCP-CBR of about 1.5 in the top 8 in. and 12 in, which is considered as very poor.
- Based on 15 moisture content tests performed from TB1, the results varied from - 1.1% to +2.1% of w_{opt}, with an average of about +0.2% from all test points.
- The results from TB2 showed an average DCP-CBR of about 3.0 in the top 8 in. and 3.9 in the top 12 in, which is considered as very poor.

Table 35. Summary of field results for Woodbury County I-29

Parameter	Woodbury County I-29 TB1	Woodbury County I-29 TB2	Woodbury County I-29 TB3
	7/9/2014	7/10/2014	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 (%)	3.5	6.9	0.2
Range (%)	-4.1 to 11.8	3.9 to 8.9	-1.1 to 2.1
Standard Deviation (%)	4.2	1.4	0.9
COV (%)	20	6	6
$CBR_{8\ in.}$			
Average (%)	2.6	1.5	3.0
Range (%)	2.1 to 3.6	0.8 to 2.2	1.7 to 4.1

Table 35. Continued

Parameter	Woodbury County I-29 TB1	Woodbury County I-29 TB2	Woodbury County I-29 TB3
	7/9/2014	7/10/2014	8/7/2014
Standard Deviation (%)	0.5	0.6	1.0
COV (%)	20	41	32
$CBR_{12\text{ in.}}$			
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

Project 8. Scott County

Project overview and observations

The project site was visited three times. Figure 106 to Figure 111 shows pictures of the construction equipment used on the site during embankment construction. Caterpillar 349E excavate materials from borrow source. A disk was used to dry embankment materials. A Caterpillar dozer was used to spread loose lift material. A pull behind sheepsfoot roller was used for soil compaction. A dynapac pad foot roller used for soil compaction. ISU researchers in situ test process is shown in Figure 112.

Field observations indicated that the fill material obtained from the borrow area was relatively wet. Field testing was conducted on 7/16/2014, 7/31/2014, and 9/19/2014 on test sections that were passed either on the same day or the previous day by the Iowa DOT field inspector. Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 5 locations, during three visits.

Figure 107. Caterpillar 349E excavate materials from borrow source

Figure 108. Caterpillar dozer used to control lift thickness

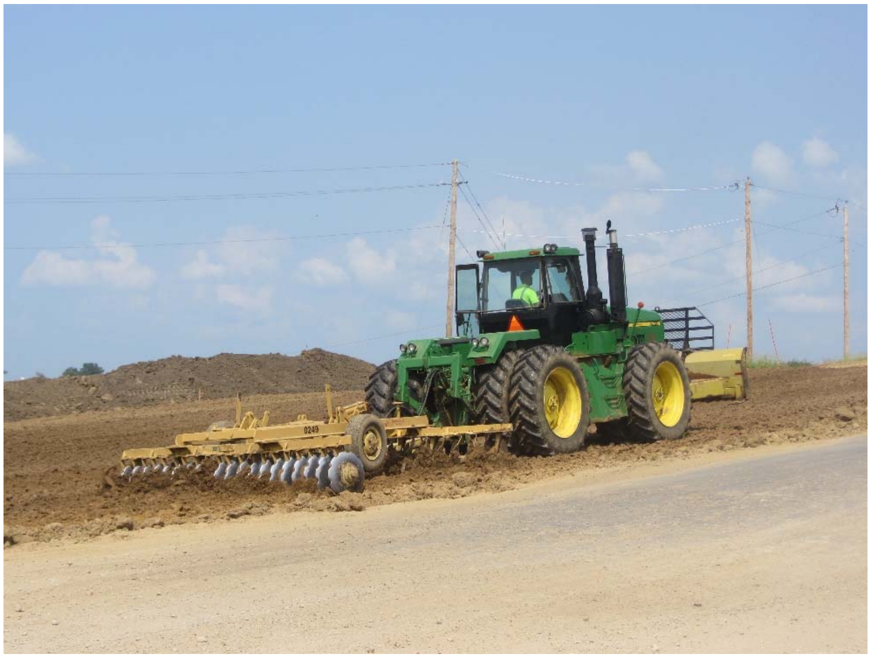


Figure 109. Disk used to dry embankment materials

Figure 110. Sheepsfoot roller used for soil compaction

Figure 111. Dynapac pad foot roller used for soil compaction

Figure 112. ISU in situ drive cylinder testing

Field and laboratory test results

Laboratory Proctor test results for materials collected from the three test beds are presented in Figure 113, Figure 115, and Figure 117. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} and γ_{dmax} that was being used by the DOT for QC/QA; and (c) ISU field test results from dive core testing.

Based on 15 drive core tests performed from TB1, only 5 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 0.4 to +5.5% of w_{opt}, with an average of about +1.8% from all test points. The dry unit weight of the material varied from 92.4% to 102.4% of standard Proctor γ_{dmax}, with an average of about 97.1% from all test points (Figure 36).

DCP-CBR profiles and cumulative blows with depth profiles from the three test beds are presented in Figure 114, Figure 116, and Figure 118. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 7.6 in the top 8 in. and 7.0 in the top 12 in, which is considered as poor according to SUDAS. The CBR value does not varies a lot for each point, which indicate the compaction is relatively uniform (Figure 114).

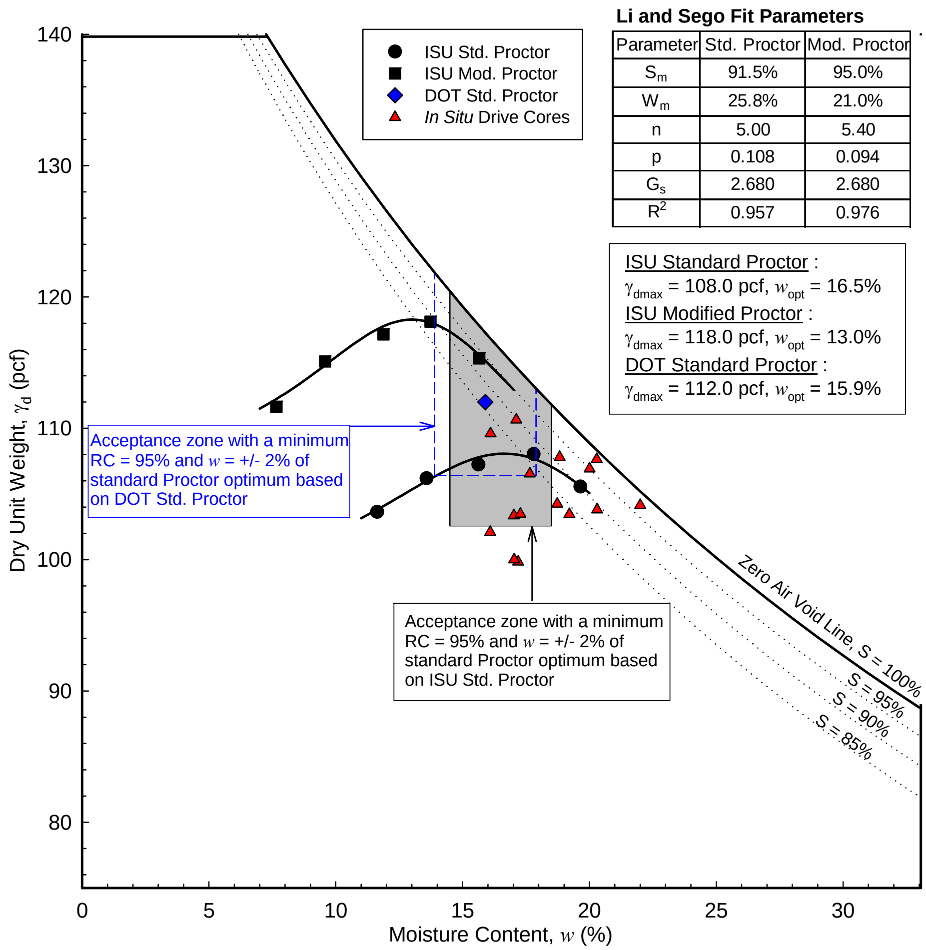


Figure 113. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Scott County Project TB1

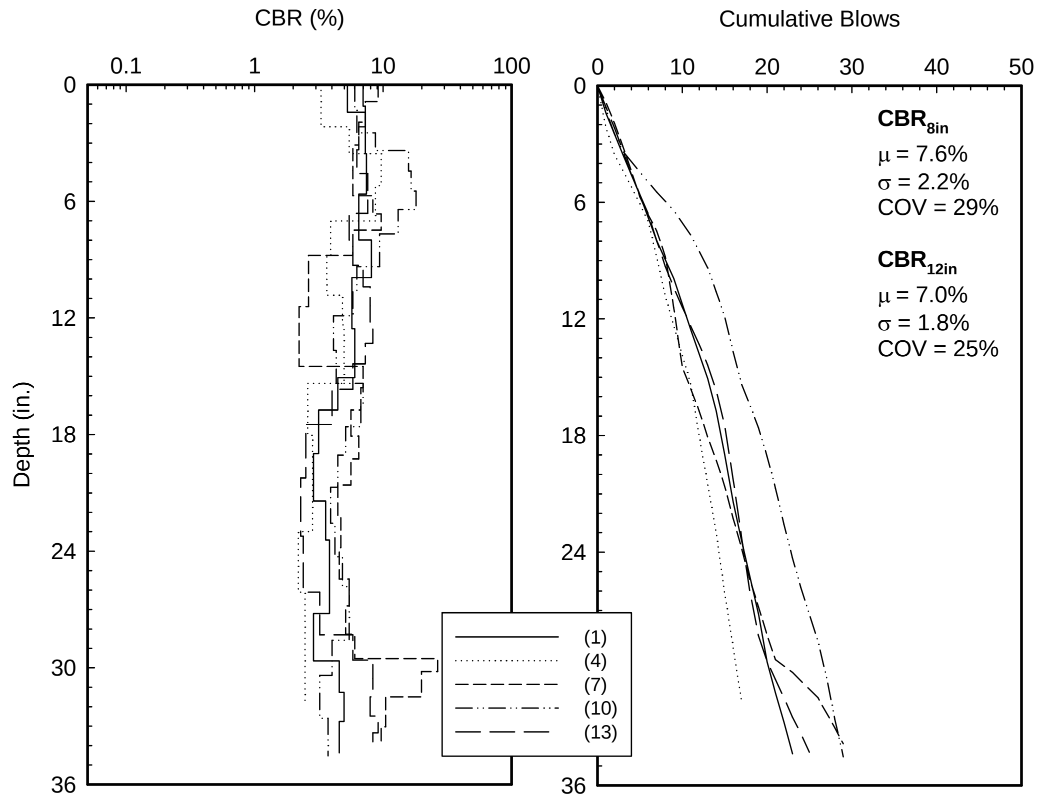


Figure 114. DCP-CBR profile at Scott County Project TB1

Based on 15 drive core tests performed from TB2, only 1 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from +0.7% to +4.6% of w_{opt}, with an average of about +3.3 % from all test points. The dry unit weight of the material varied from 95.3% to 99.4% of standard Proctor γ_{dmax}, with an average of about 97.5% from all test points (Figure 115).

The results from TB2 showed an average DCP-CBR of about 3.1 in the top 8 in. and 2.7 in the top 12 in, which is considered as very poor according to SUDAS. Point 2 have a low CBR value from 20 to 34.5 in., which is an indication of “uncompacted” fill or the oreo-cookie effect (Figure 116).

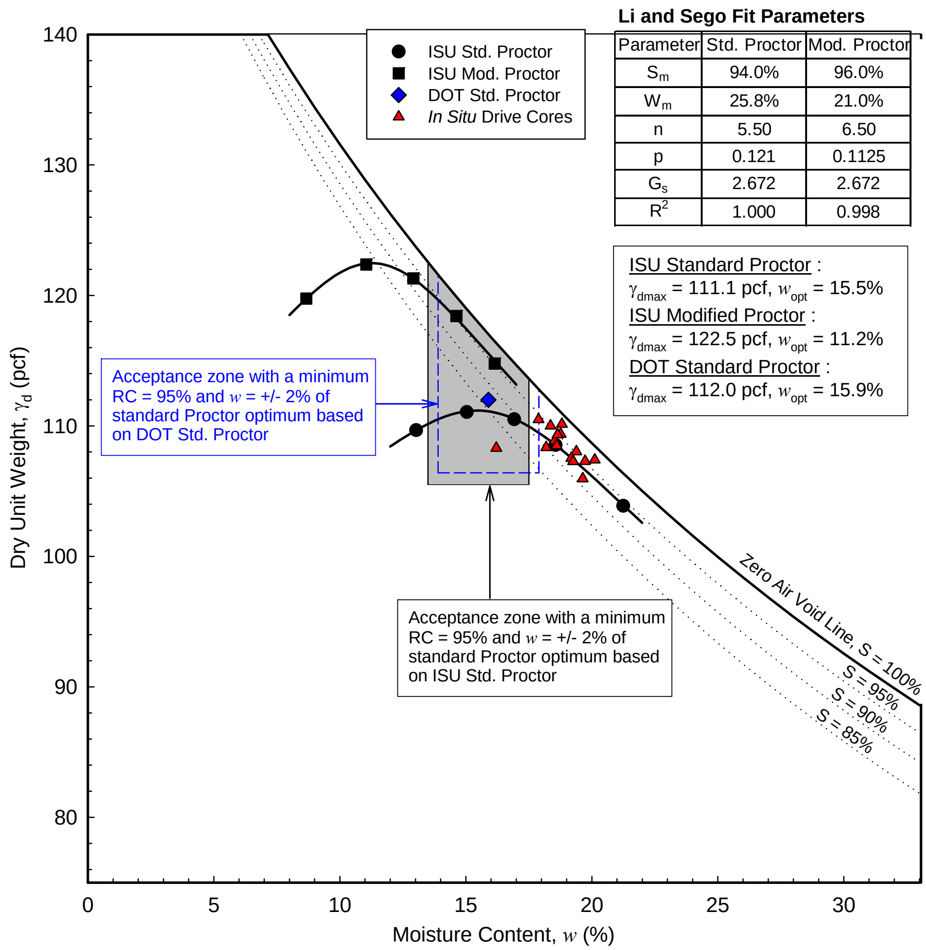


Figure 115. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Scott County Project TB2

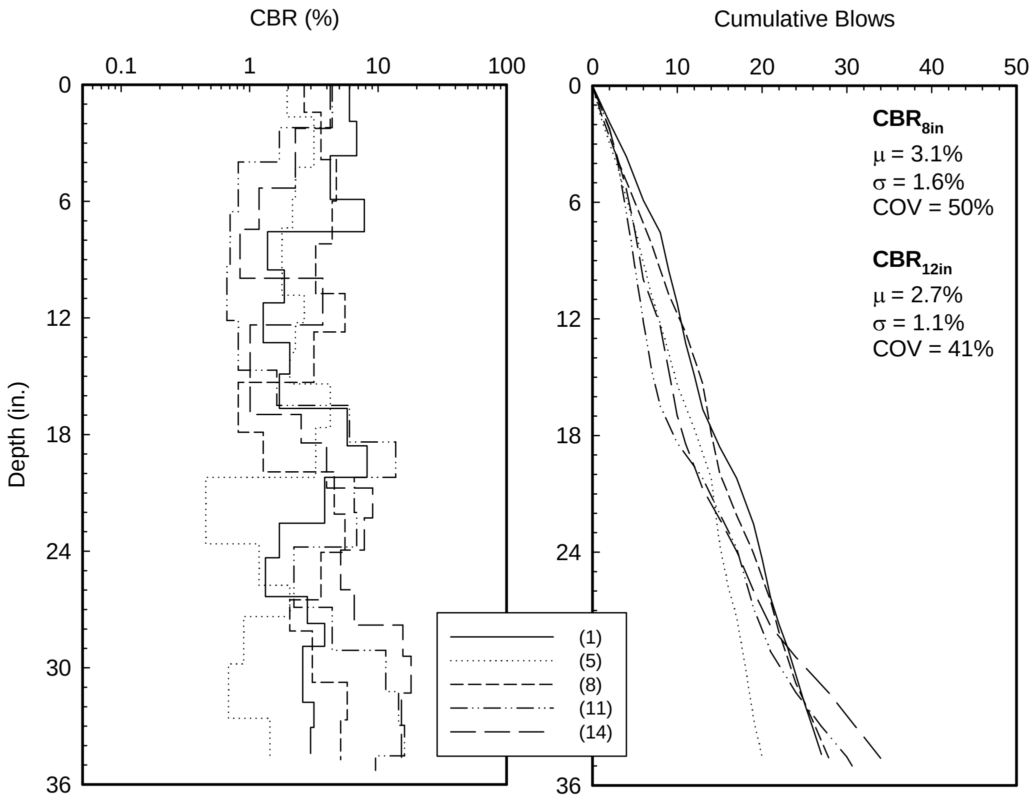


Figure 116. DCP-CBR profile at Scott County Project TB2

Based on 14 drive core tests performed from TB3, 10 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from +0.3% to +7.1% of w_{opt}, with an average of about +2.3 % from all test points. The dry unit weight of the material varied from 92.5% to 100.6% of standard Proctor γ_{dmax}, with an average of about 98.0% from all test points (Figure 117).

The results from TB3 showed an average DCP-CBR of 0.6 in top 8 in. and 0.5 in top 12 in, which is considered as very poor according to SUDAS. The CBR value for top 14 in. of all points are extremely low which can be indicated as uncompacted fill. The CBR value varies a lot, which indicate the embankment does not have a uniform compaction (Figure 118).

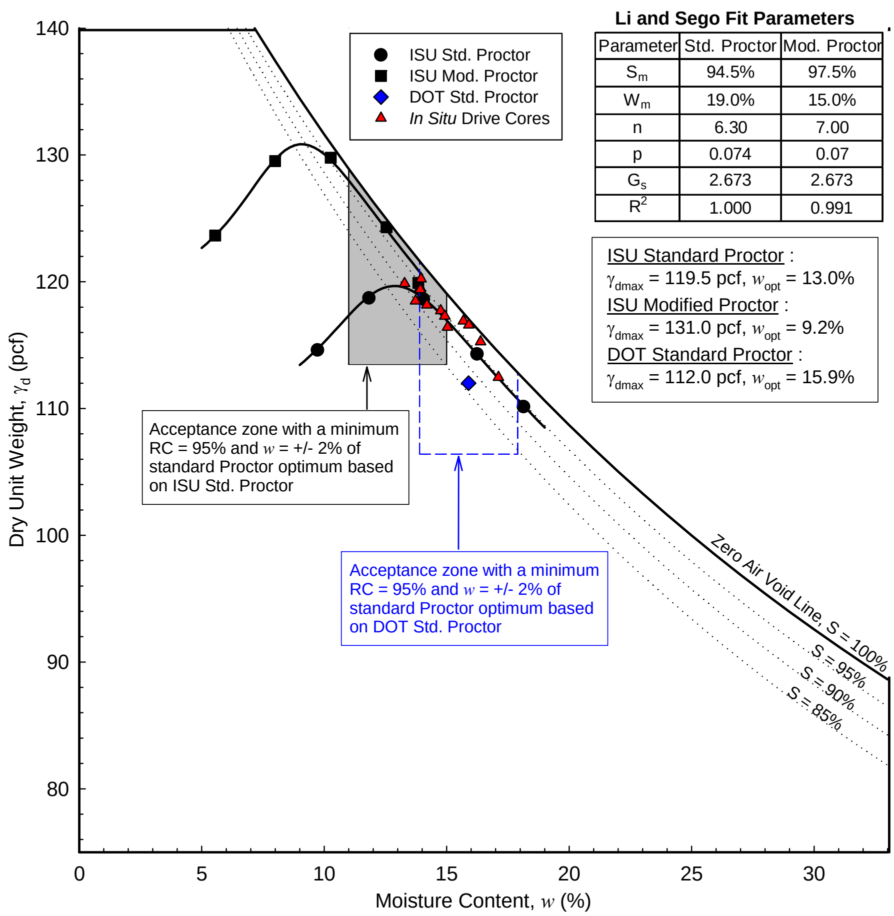


Figure 117. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Scott County Project TB3

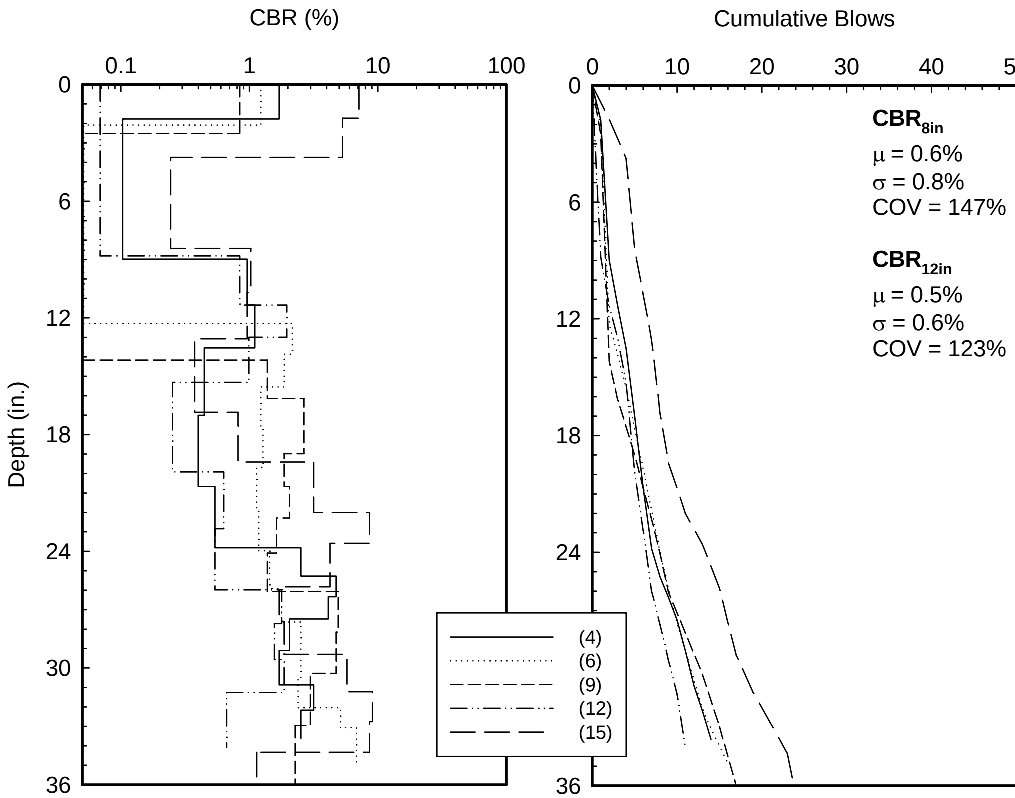


Figure 118. DCP-CBR profile at Scott County Project TB3

Summary of results

Table 36 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was relatively wet.
- The moisture content of the material in TB1 varied from - 0.4 to +5.5% of w_{opt}, with an average of about +1.8% from all test points. The dry unit weight of the material varied from 92.4% to 102.4% of standard Proctor γ_{dmax}, with an average of about 97.1% from all test points.

- The results from TB1 showed an average DCP-CBR of about 7.6 in the top 8 in. and 7.0 in the top 12 in, which is considered as poor.
- The moisture content of the material in TB2 varied from +0.7% to +4.6% of w_{opt}, with an average of about +3.3 % from all test points. The dry unit weight of the material varied from 95.3% to 99.4% of standard Proctor γ_{dmax}, with an average of about 97.5% from all test points.
- The results from TB2 showed an average DCP-CBR of about 3.1 in the top 8 in. and 2.7 in the top 12 in, which is considered as very poor.
- The moisture content of the material in TB3 varied from +0.3% to +7.1% of w_{opt}, with an average of about +2.3 % from all test points. The dry unit weight of the material varied from 92.5% to 100.6% of standard Proctor γ_{dmax}, with an average of about 98.0% from all test points.
- The results from TB3 showed an average DCP-CBR of 0.6 in top 8 in. and 0.5 in top 12 in, which is considered as very poor.

Table 36. Summary of field results for Scott County

Parameter	Scott County TB1	Scott County TB2	Scott County TB3
	7/16/2014	7/31/2014	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

Table 36. Continued

Parameter	Scott County TB1	Scott County TB2	Scott County TB3
	7/16/2014	7/31/2014	9/19/2014
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

Project 9. Woodbury County US 20

Project overview and observations

The project site was visited four times in two days. Figure 119 to Figure 123 shows pictures of the construction equipment used on the site during embankment construction. Caterpillar 631D motor scraper were used to collect fill materials from cuts and borrow areas for placement in fill areas. Caterpillar 140H motor grader was used to level the embankment surface. A Caterpillar D6N dozer was used to spread loose lift material. A pull behind sheepsfoot roller was used for soil compaction. Caterpillar CS56 series vibratory smooth drum roller used for soil compaction.

Field observations indicated that the fill material obtained from the borrow area was relatively wet. Field testing was conducted on 9/26/2014 and 10/18/2014 on test sections that were passed either on the same day or the previous day by the Iowa DOT field inspector.

Testing involved drive cores for density and moisture content determination at 15 test locations and DCP testing at 5 locations during the four visits.

Figure 119. Caterpillar 631D motor scraper used to collect and place loose fill materials

Figure 120. Caterpillar D6N dozer used to control lift thickness

Figure 121. Caterpillar 140H motor grader used to level the embankment surface

Figure 122. Caterpillar CS56 series vibratory smooth drum roller used for soil compaction

Figure 123. Sheeps foot roller wheel used for soil compaction

Field and laboratory test results

Laboratory Proctor test results for materials collected from the four test beds are presented in Figure 124, Figure 126, Figure 128, and Figure 130. The Li and Sego fit parameters for the Proctor curve, w_{opt}, and γ_{dmax} for standard and modified Proctor are identified in the figures. The figures also identify: (a) the acceptance zone for moisture (+/- 2% of w_{opt}) and density (95% of standard Proctor γ_{dmax}) control; (b) the w_{opt} that was being used by the DOT for QC/QA; and (c) ISU field test results from dive core testing.

Based on 15 drive core tests performed from TB1, only 2 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 4.4 to +7.1% of w_{opt}, with an average of about +3.2% from all test points. The dry unit weight of the material varied from 87.8% to 102.4% of standard Proctor γ_{dmax}, with an average of about 96.1% from all test points (Figure 124).

DCP-CBR profiles and cumulative blows with depth profiles from the four test beds are presented in Figure 125, Figure 127, Figure 129, and Figure 131. Summary statistics (i.e., average (μ), standard deviation (σ), and coefficient of variation (COV)) of the CBR of the top 8 in. and top 12 in. are also summarized in the figures. As I mentioned in background section, CBR less than 5 is considered as very poor, CBR value from 5 to 10 is considered as poor, CBR of 10 or more is considered good.

The results from TB1 showed an average DCP-CBR of about 5.3 in the top 8 in. and 6.1 in the top 12 in, which is considered as poor according to SUDAS. Point 6 have a very low CBR from 3 to 16 inch which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 124).

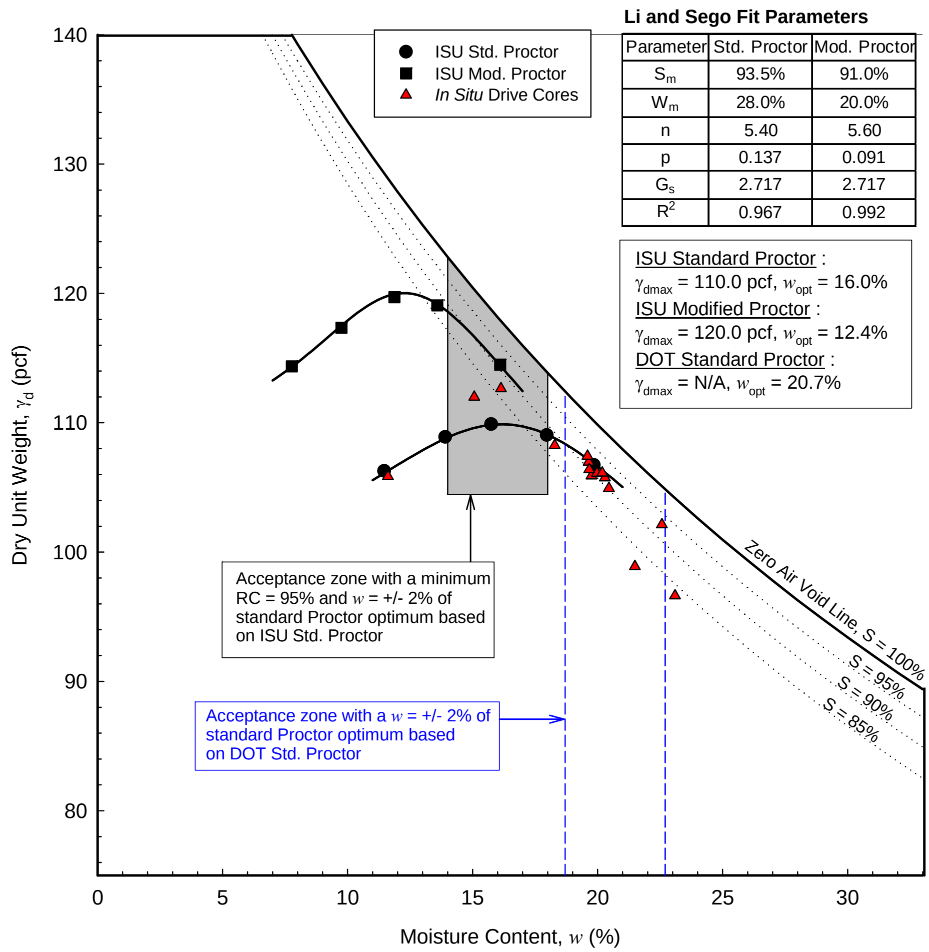


Figure 124. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Woodbury County US 20 Project TB1 (N/A: not available)

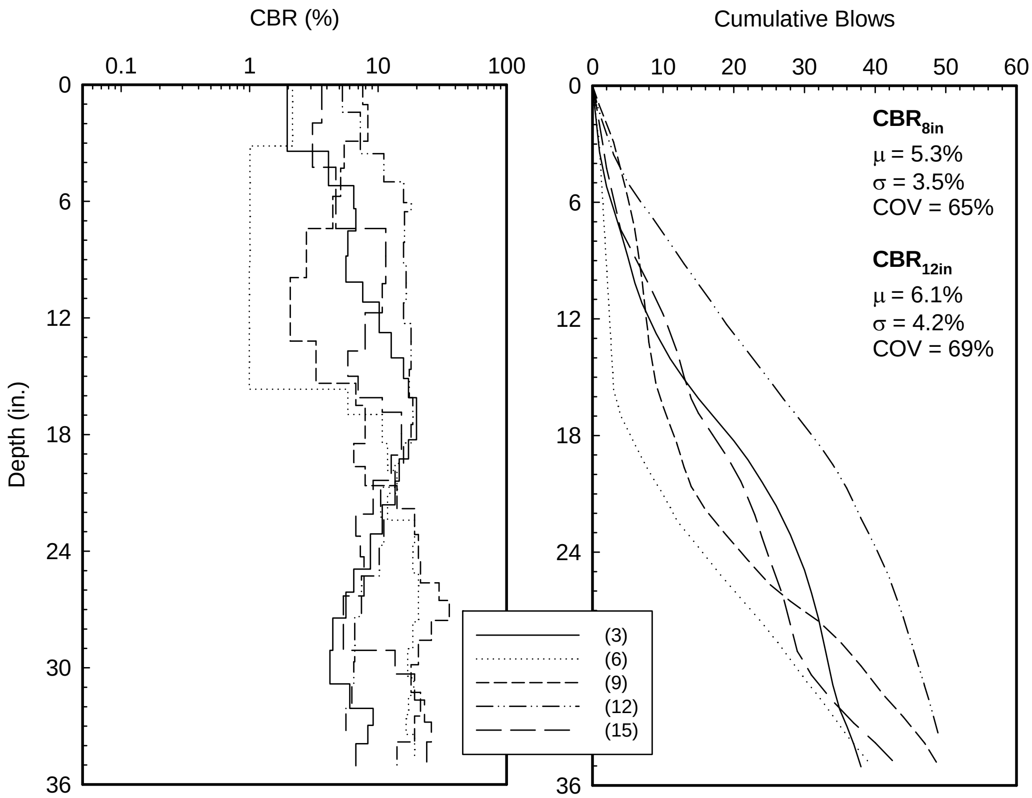


Figure 125. DCP-CBR profile at Woodbury County US 20 Project TB1

Based on 15 drive core tests performed from TB2, only 7 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from +0.6% to +4.4% of w_{opt}, with an average of about +2.4% from all test points. The dry unit weight of the material varied from 95.9% to 101.1% of standard Proctor γ_{dmax}, with an average of about 98.5% from all test points (Figure 126).

The results from TB2 showed an average DCP-CBR of about 2.8 in the top 8 in. and 2.6 in the top 12 in, which is considered as very poor according to SUDAS. Point 7 have a very low CBR from 11.5 to 17 inch which is an indication of “uncompacted” fill or the oreo-cookie effect (Figure 127).

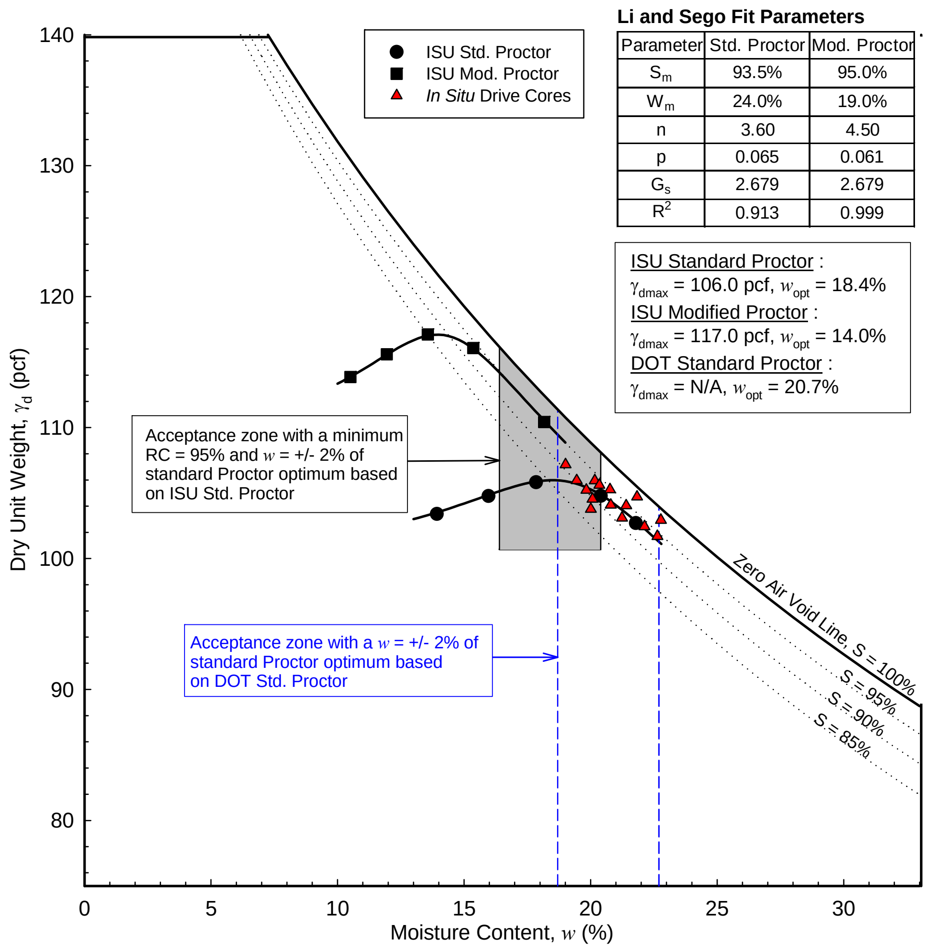


Figure 126. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Woodbury County US 20 Project TB2 (N/A: not available)

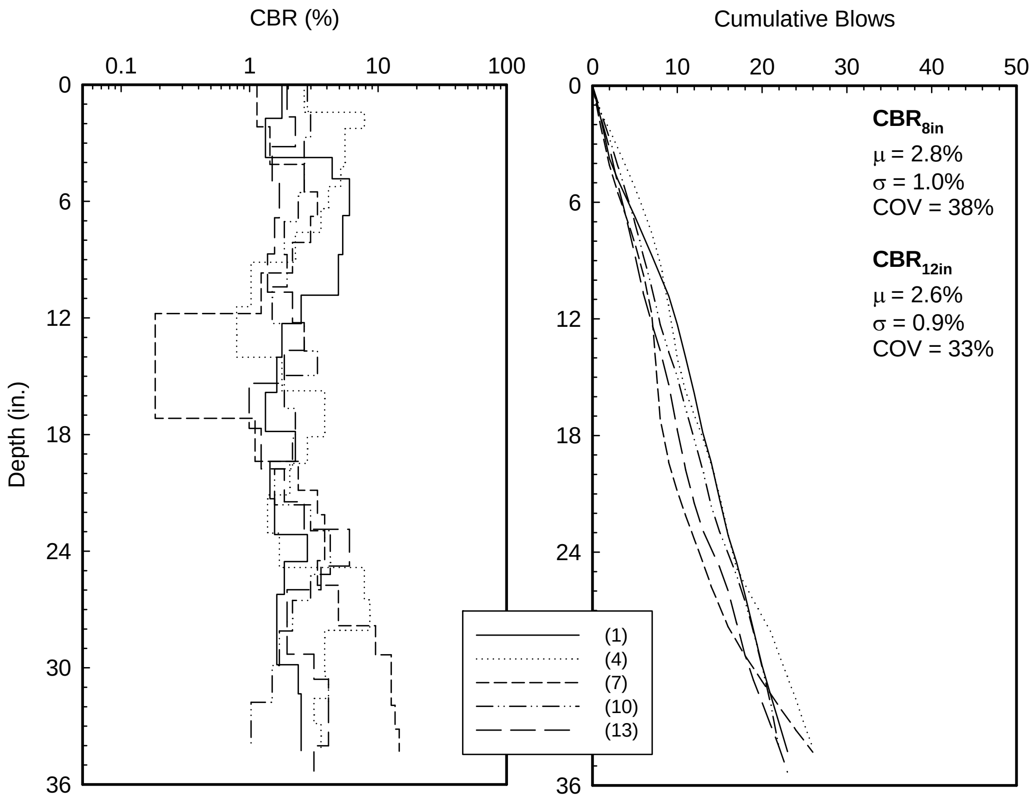


Figure 127. DCP-CBR profile at Woodbury County US 20 Project TB2

Based on 15 drive core tests performed from TB3, 9 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from - 4.4 to +4.4% of w_{opt}, with an average of about +1.4% from all test points. The dry unit weight of the material varied from 94.1% to 109.0% of standard Proctor γ_{dmax}, with an average of about 100.7% from all test points (Figure 128).

The results from TB3 showed an average DCP-CBR of about 4.5 in the top 8 in. and 4.8 in the top 12 in, which is considered as very poor according to SUDAS. The CBR value varies a lot above top 13 in. and below 23 in., which indicate some layer did not compact as uniform as other layers (Figure 129).

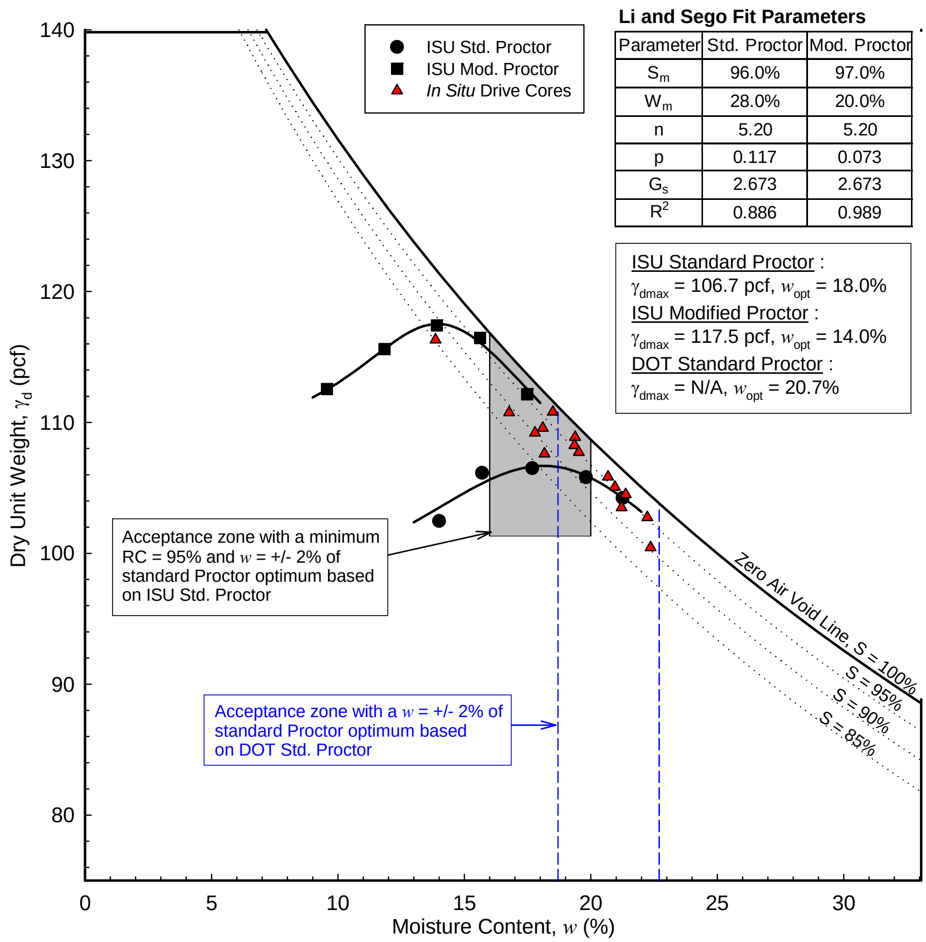


Figure 128. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Woodbury County US 20 Project TB3 (N/A: not available)

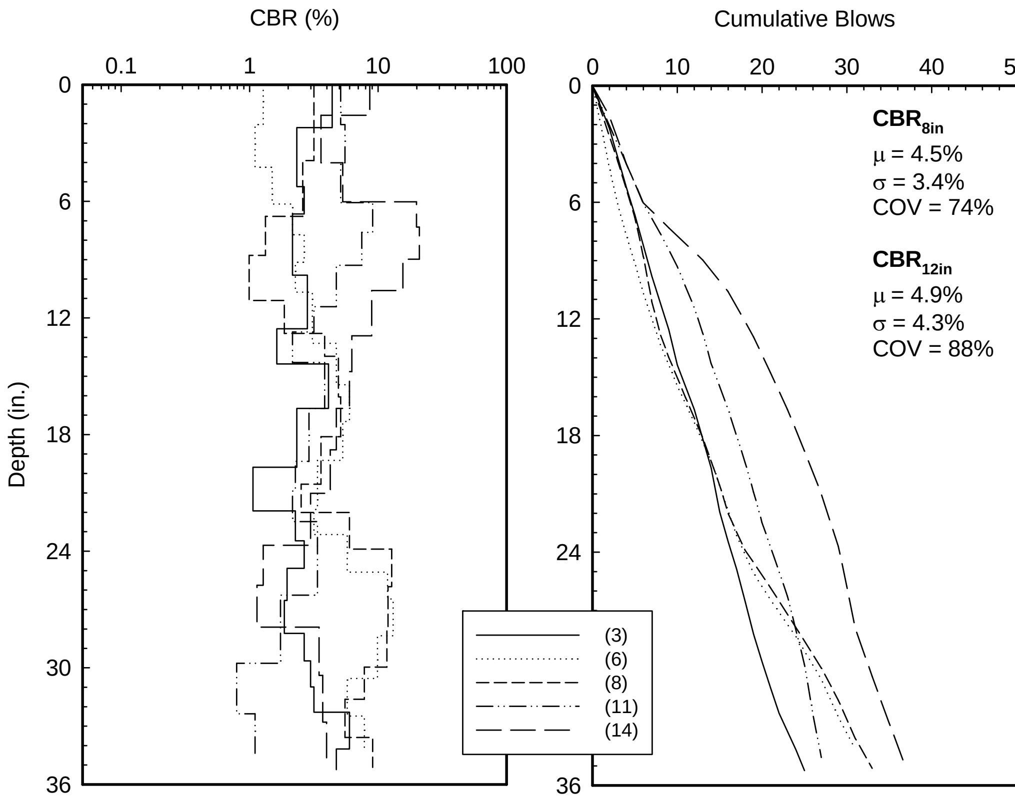


Figure 129. DCP-CBR profile at Woodbury County US 20 Project TB3

Based on 14 drive core tests performed from TB4, 8 fell in the acceptance zone identified from ISU Proctor testing. The moisture content of the material in situ varied from -2.6% to +5.2% of w_{opt}, with an average of about +1.0% from all test points. The dry unit weight of the material varied from 90.8% to 102.0% of standard Proctor γ_{dmax}, with an average of about 97.6% from all test points (Figure 130).

The results from TB4 showed an average DCP-CBR of about 8.1 in the top 8 in. and 7.8 in the top 12 in, which is considered as poor according to SUDAS. Point 2 have a low CBR value from 9 to 21 in., which is an indication of "uncompacted" fill or the oreo-cookie effect (Figure 131).

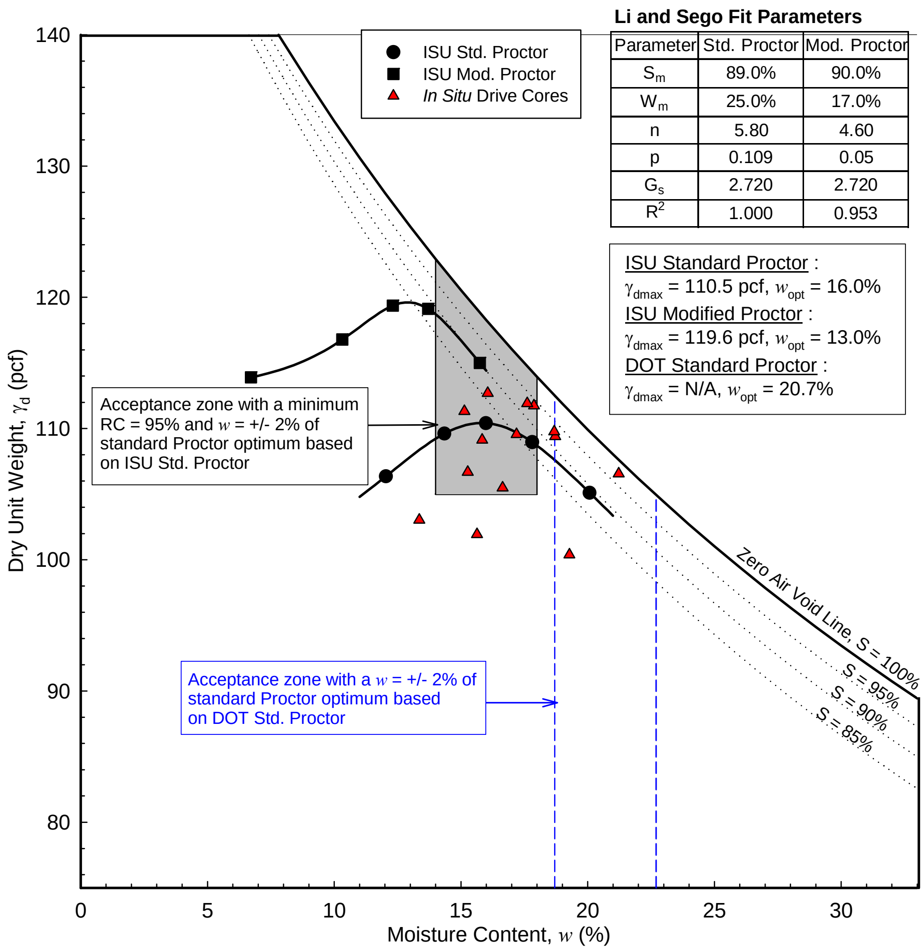


Figure 130. Comparison of in situ moisture-density measurements with laboratory Proctor compaction test results and Iowa DOT acceptance limits for Woodbury County US 20 Project TB4 (N/A: not available)

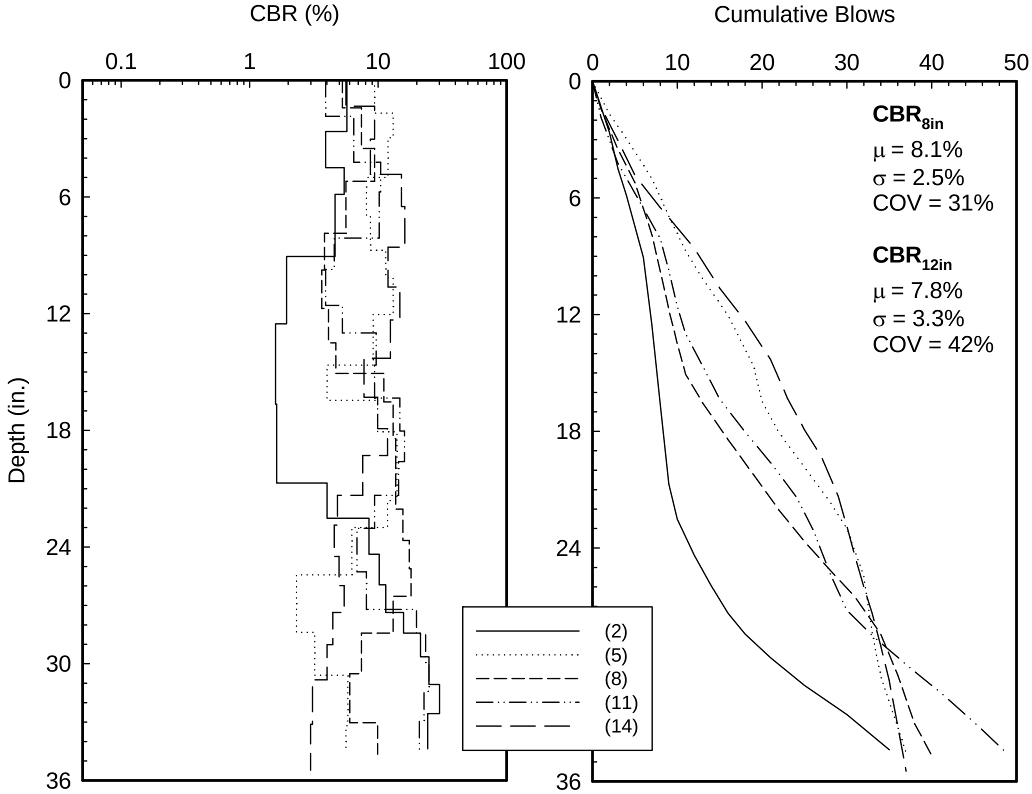


Figure 131. DCP-CBR profile at Woodbury County US 20 Project TB4

Summary of results

Table 37 summarizes the field test results with statistics of relative compaction, moisture content with reference to w_{opt}, and average CBR in the top 8 and 12 in. Key observations from field testing at this project site are as follows:

- The fill material obtained from the borrow area was relatively wet.
- The moisture content of the material in TB1 varied from - 4.4 to +7.1% of w_{opt}, with an average of about +3.2% from all test points. The dry unit weight of the material varied from 87.8% to 102.4% of standard Proctor γ_{dmax}, with an average of about 96.1% from all test points.

- The results from TB1 showed an average DCP-CBR of about 5.3 in the top 8 in. and 6.1 in the top 12 in, which is considered as poor.
- The moisture content of the material in TB2 varied from +0.6% to +4.4% of w_{opt}, with an average of about +2.4% from all test points. The dry unit weight of the material varied from 95.9% to 101.1% of standard Proctor γ_{dmax}, with an average of about 98.5% from all test points.
- The results from TB2 showed an average DCP-CBR of about 2.8 in the top 8 in. and 2.6 in the top 12 in, which is considered as very poor.
- The moisture content of the material in TB3 varied from - 4.4 to +4.4% of w_{opt}, with an average of about +1.4% from all test points. The dry unit weight of the material varied from 94.1% to 109.0% of standard Proctor γ_{dmax}, with an average of about 100.7% from all test points.
- The results from TB3 showed an average DCP-CBR of about 4.5 in the top 8 in. and 4.8 in the top 12 in, which is considered as very poor.
- The moisture content of the material in TB4 varied from -2.6% to +5.2% of w_{opt}, with an average of about +1.0% from all test points. The dry unit weight of the material varied from 90.8% to 102.0% of standard Proctor γ_{dmax}, with an average of about 97.6% from all test points.
- The results from TB4 showed an average DCP-CBR of about 8.1 in the top 8 in. and 7.8 in the top 12 in, which is considered as poor.

Table 37. Summary of field results for Woodbury County US20

Parameter	Woodbury County (US20) TB1	Woodbury County (US20) TB2	Woodbury County (US20) TB3	Woodbury County (US20) TB4
	9/26/2014	9/26/2014	10/18/2014	10/18/2014
Relative Compaction				
Average (%)	96.1	98.5	100.7	97.6
Range (%)	87.8 to 102.4	95.9 to 101.1	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	2.4	1.4	1.0
Range (%)	-4.4 to +7.1	0.6 to +4.4	-4.1 to +4.4	-2.6 to +5.2
Standard Deviation (%)	2.95	1.15	2.27	2.04
COV (%)	93	47	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

SUMMARY OF ALL FIELD TESTING RESULTS

Proctor figures shows that ISU have a different maximum dry density and optimum moisture content with the Iowa DOT results. The possible reason can be the tested material are different.

The Proctor results show the field moisture - density results did not always qualify with the requirements set up by the laboratory standard Proctor results. Because of advances in compaction equipment, some researchers (White et al. 2007), (White et al. 2009) have identified the relationship between laboratory moisture – density and field moisture – density are hard to estimate.

Even though the drive core cylinder results are in the accept moisture and density range, the DCP results indicate that the deeper layers of the embankment are not always qualified.

Figure 132 and Figure 133 shows the frequency of the in situ moisture content and both moisture content and density measurements as percent that were outside the acceptance limits. 100% stands for all samples outside the limits, and 0% stands for all samples that are inside the limits. As you can see, 20 out of 25 projects had a significant percentage of tests that were outside the moisture limits. 17 out of the 22 field projects with 20 to 100% of the moisture and density test results outside the QC/QA acceptance limits.

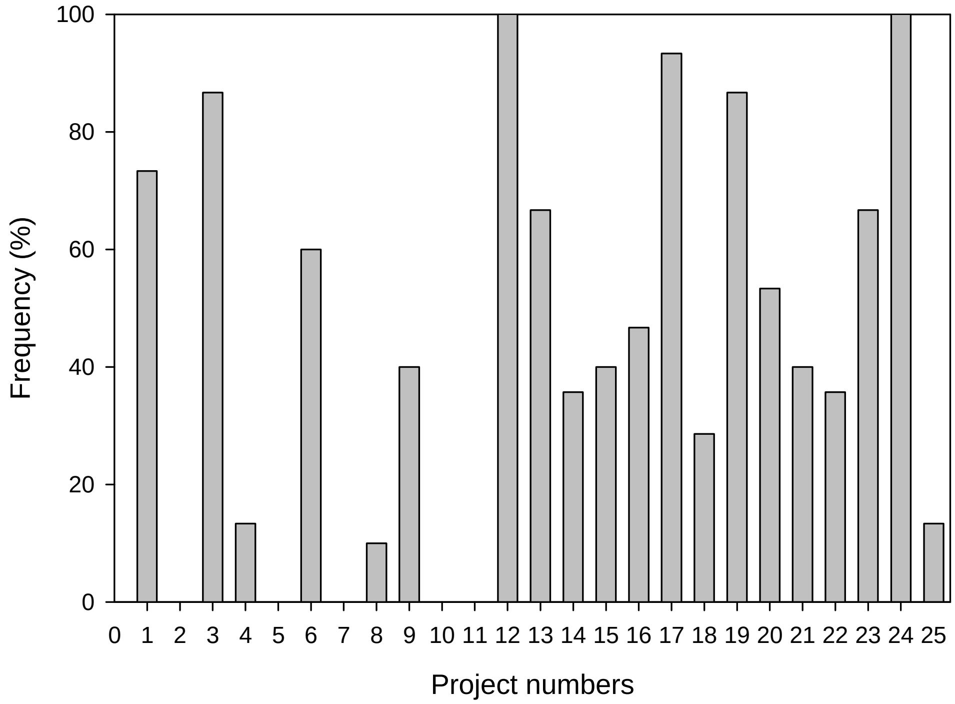


Figure 132. Percentage of test measurements were outside the accepted moisture limits

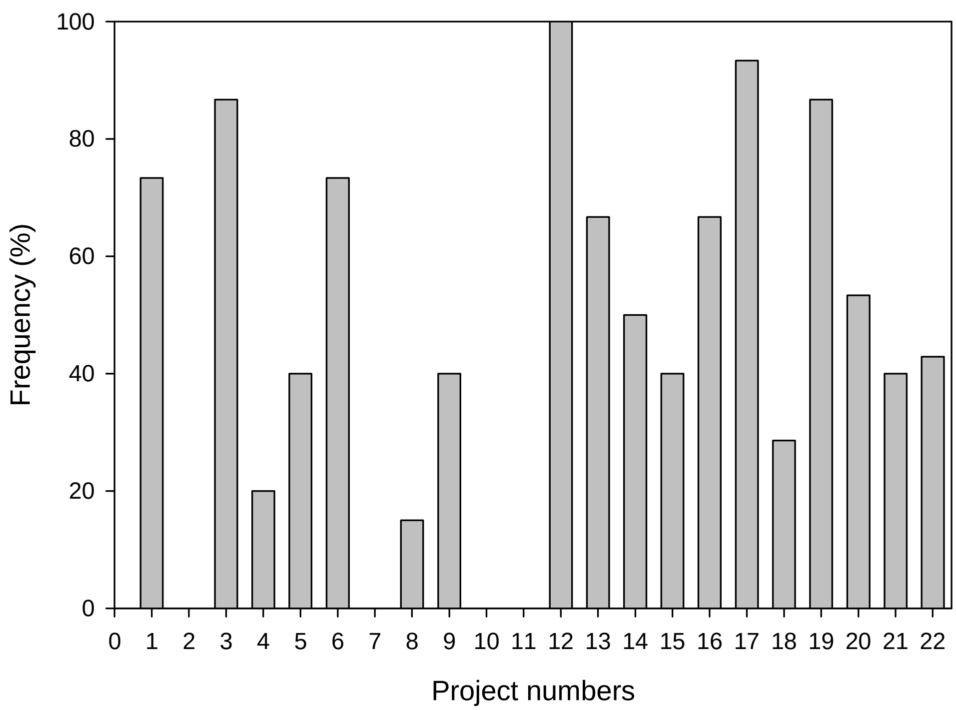


Figure 133. Percentage of test measurements were outside the accepted moisture and density limits

ESTIMATING CBR TARGET VALUES

Field project test results presented above indicated that DCP-CBR testing can be performed relatively quickly and can provide valuable information in terms of the quality of compacted fill material vertically. For using DCP-CBR values as part of QC/QA in lieu of moisture-density testing, a procedure to estimate its target values is needed. In this chapter, analysis of results available in the literature U.S. Army Corps of Engineers (1950) in regards to factors affecting the CBR values and a laboratory test procedure developed to determine the DCP-CBR are presented. CBR test results reported in U.S. Army Corps of Engineers Report are analyzed to assess influence of the mold size (6 in., 7.4 in., and 12 in.) used in CBR testing, soil type, moisture content, and dry unit weight; and to develop a statistical model for predicting CBR.

Factors that Influence Laboratory Determined CBR Values

Description of materials

The U.S. Army Corps of Engineers (1950) report summarized CBR results of five soil types consisting of two cohesive materials (clayey silt and silty clay) and three granular materials (clay gravel, sand, and sandy gravel). Index properties of these soils are summarized in Table 38 and Table 39. Particle size distribution curves for these materials are provided in Figure 134 to Figure 136. Two of the granular materials (clay gravel and sandy gravel) were also tested in modified gradations as identified in the soil index and gradation in Table 38, Table 39, Figure 135, and Figure 136.

Table 38. Soil index properties for clayey silt, silty clay and clay gravel

Soil name	Clayey Silt (Soil type 1)	Silty Clay (Soil type 4)	Clay Gravel (Natural) (Soil type 2)	Clay Gravel (Processed) (Soil type 2)	Clay Gravel (Passing 3/4") (Soil type 2)	Clay Gravel (Passing No.4) (Soil type 2)
Gravel content (%) (> 4.75 mm)	0.0	0.0	60.0	60.01	42.9	0.0
Sand content (%) (4.75 mm – 75 μm)	5.1	2.9	26.6	-	38.6	68.5
Fines content (%) (< 75 μm)	94.9	97.1	13.4	-	18.5	31.5
Liquid limit, LL (%)	40	37	27	27	27	27
Plastic limit, PL (%)	28	23	14	14	14	14
Plastic Index, PI (%)	12	14	12	12	12	12
AASHTO	A-6	A-6	A-2-6	-	A-2-6	A-2-6
USCS classification	CL	CL	GC	-	GC	SC
USCS Description	Lean Clay	Lean clay	Clayey gravel	-	Clayey gravel	Clayey sand
D_{10}	-	-	0.06	-	0.01	-
D_{30}	0.01	0.01	0.54	-	0.25	0.07
D_{60}	0.02	0.02	13.65	7.46	5.48	0.32
D_{85}	0.04	0.03	29.88	13.47	12.63	1.48
D_{100}	-	-	76.35	19.06	19.06	4.76
C_u	-	-	247.73	-	521.96	-
C_c	-	-	0.38	-	1.12	-

Table 39. Soil index property for Clinton sand and sandy gravel (from USCE 1950)

Soil name	**Clinton Sand (Soil type 3)**	**Sandy Gravel (Natural) (Soil type 5)**	**Sandy Gravel (Processed) (Soil type 5)**	**Sandy Gravel (Passing 3/4") (Soil type 5)**	**Sandy Gravel (Passing No.4) (Soil type 5)**
Gravel content (%) (> 4.75 mm)	0.0	48.4	48.4	28.4	0
Sand content (%) (4.75 mm – 75 μm)	80.8	44.3	-	62.3	87.4
Fines content (%) (< 75 μm)	19.2	7.3	-	9.3	12.6
Liquid limit, LL (%)	18	NP	NP	NP	NP
Plastic limit, PL (%)	16	NP	NP	NP	NP
Plastic Index, PI (%)	2	NP	NP	NP	NP
AASHTO	A-2-4	A-3	-	A-3	A-3
USCS classification	SM	GP-GM	-	SP-SM	SM
USCS Description	Silty sand	Poorly graded gravel with silt	-	Poorly graded sand with silt	Silty sand
D_{10}	-	0.21	-	0.11	0.02
D_{30}	0.21	0.45	-	0.31	0.25
D_{60}	0.25	9.12	5.81	2.16	0.42
D_{85}	0.33	24.28	12.13	9.88	2.43
D_{100}	1.67	37.66	18.58	18.58	4.64
C_u	-	44.00	-	19.13	27.81
C_c	-	0.11	-	0.39	10.14

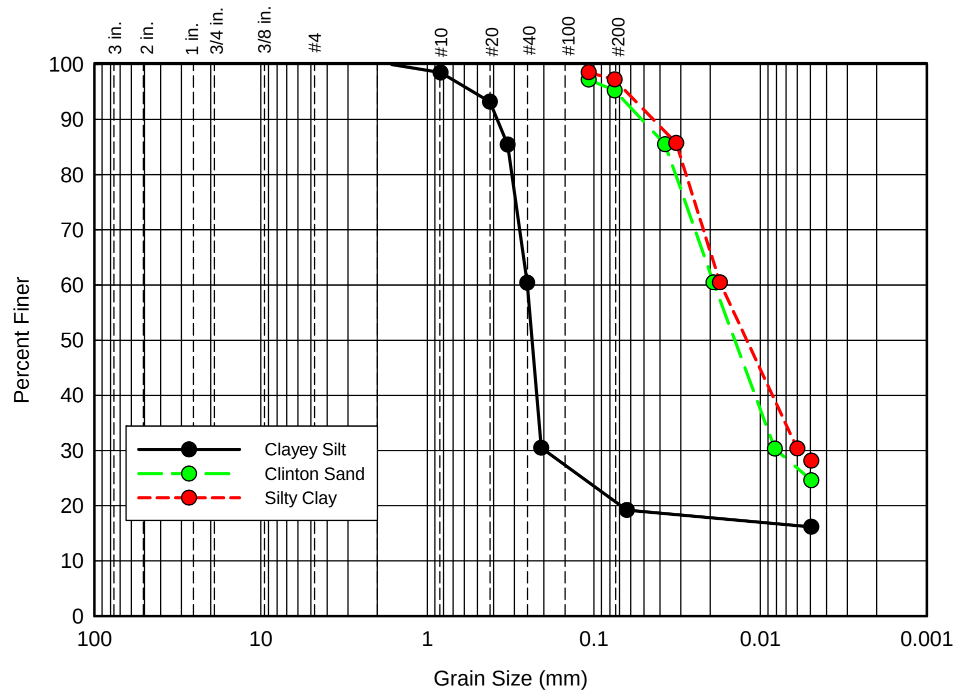


Figure 134. Grain size distribution for soil types 1, 3 and 4

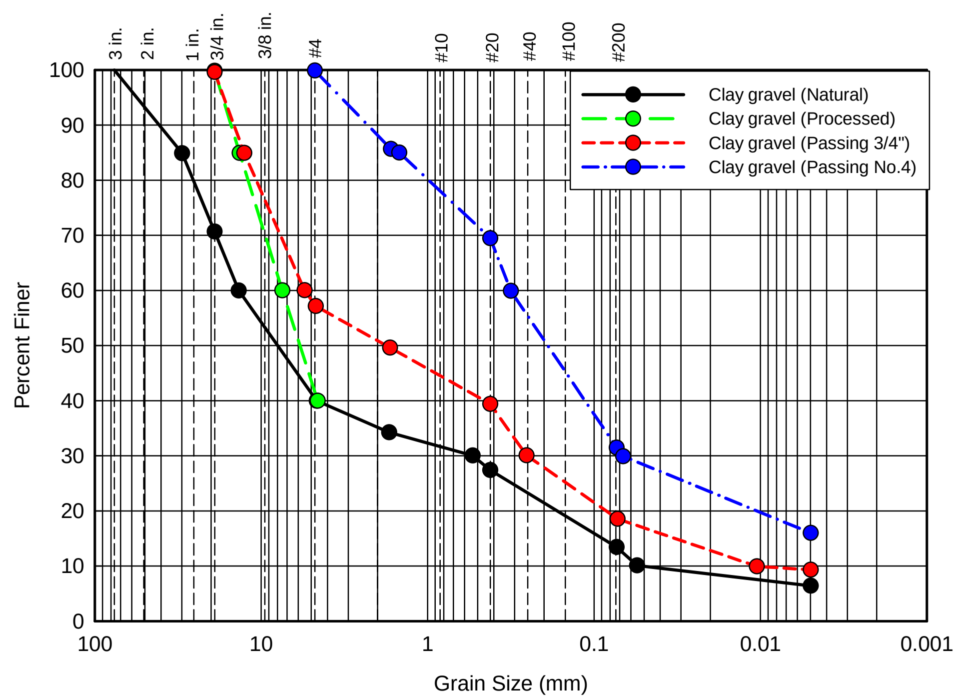


Figure 135. Grain size distribution for soil type 2

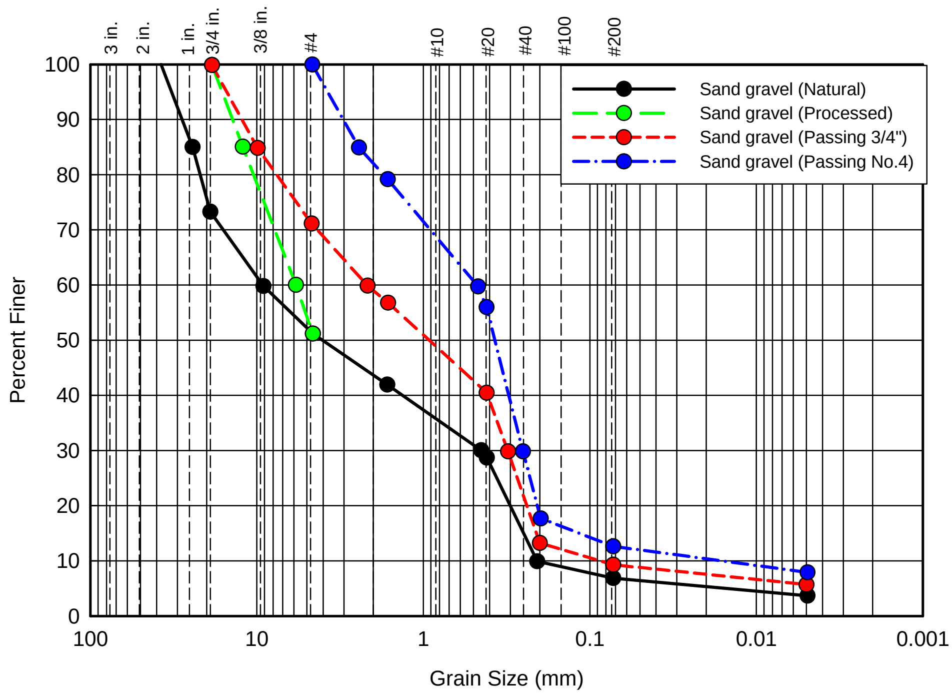


Figure 136. Grain size distribution for soil type 5

Statistical analysis methods

Statistical multiple regression analysis was conducted to assess influence of moisture content (w), dry unit weight (γ_d), mold size (ϕ), soil type, and soil index properties. Analysis was performed using JMP statistical analysis software. The analysis was performed by incorporating the above listed parameters in a linear multiple regression model with and without transformations (using log, exponential, and power functions). The statistical significance of these parameters were assessed using the *t*- and *p*-values associated with each parameter. The selected criteria for identifying the significance of a parameter included p-value ≤ 0.05 is significant, ≤ 0.10 is possibly significant, > 0.10 = not significant, and t-value <-2 or $>+2$ = significant. Higher the t- and p- values, greater is the statistical significance of the parameter.

Results and discussion

Analysis results summarizing the influence of w, γ_d, and ϕ for each soil type are summarized in Table 40 to Table 46. Soil types 2 and 5 with natural gradation and passing No. 4 gradation were not analyzed due to limited data. Combining measurements on all soil types, the influence of w, γ_d, ϕ, soil type, and soil index properties is assessed and the results are summarized in Table 47. Graphs of predicted versus measured values based on the regression models are presented in Figure 137 to Figure 139. For all models, log transformations of all the parameters yielded the lowest root mean squared error (RMSE) and highest coefficient of determination (R^2) values.

Statistical analysis results indicated that w, γd, and ϕ parameters were statistically significant for all soil types, except silty clay and clay gravel material passing the ¾in. sieve. For those two materials, only w, γd parameters were statistically significant. For these seven soil types, RMSE have a range from 7.43 to 23. 79, and R^2 have a range from 0.615 to 0.82. In the prediction expression equation, "-" indicate the CBR value would decrease while the relative parameter increase, and "+" indicate the CBR value would increase while the relative parameter increase. Which means CBR value would increase when w and ϕ parameters decrease, and/or γ_d increase.

Then all materials are combined to assess a common model based on soil index properties, the statistical analysis results indicated that w, γd, ϕ, plastic index (PI), and fines content (F_{200}) parameters were statistically significant. Other soil index parameters (like D30, D60, gravel/sand contents, etc) have been studied, but these parameters are not statistical significant. The RMSE have a relative low value of 11.4, but R2 also reduce to 0.665. In the prediction expression equation, "-" indicate the CBR value would decrease while the relative parameter increase, and "+" indicate the CBR value would increase while the relative parameter increase. Which means CBR value would increase when w, ϕ, and PI parameters decrease, and/or γ_d and fines content increase.

Table 40. Statistical analysis between CBR and moisture, dry density and mold size for clayey silt (soil type 1)

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-811.53	116.53	-6.96	<.0001	.	0.82	7.43
log w	-157.13	12.97	-12.12	<.0001	1.1		
log γ_d	523.24	54.89	9.53	<.0001	1.1		
log mold ϕ	-31.27	7.20	-4.35	<.0001	1.0		
Prediction Expression	CBR = -811.53 - 157.13 x log w + 523.24 x log γ_d - 31.27 x log mold ϕ						

Table 41. Statistical analysis between CBR and moisture, dry density and mold size for processed clay gravel (soil type 2)

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-1646.45	290.25	-5.67	<.0001	.	0.72	12.82
log w	-151.71	12.72	-11.93	<.0001	1.0		
log γ_d	873.84	138.24	6.32	<.0001	1.0		
log mold ϕ	-60.69	12.77	-4.75	<.0001	1.0		
Prediction Expression	CBR = -1646.45 – 151.71 x log w + 873.84 x log γ_d -60.69 x log mold ϕ						

Table 42. Statistical analysis between CBR and moisture, dry density and mold size for clay gravel passing ¾" (soil type 2)

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-1297.24	206.78	-6.27	<.0001	.	0.67	11.50
log w	-132.16	12.45	-10.61	<.0001	1.1		
log γ_d	675.30	99.58	6.78	<.0001	1.1		
Prediction Expression	CBR = -1297.24 - 132.16 x log w + 675.3 x log γ_d						

Note: Mold size is not statistic significant with CBR

Table 43. Statistical analysis between CBR and moisture, dry density and mold size for Clinton sand (soil type 3)

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-1538.06	236.83	-6.49	<.0001	.	0.77	12.49
log w	-119.28	14.54	-8.21	<.0001	1.0		
log γ_d	856.27	113.18	7.57	<.0001	1.0		
log mold ϕ	-91.75	13.97	-6.57	<.0001	1.0		
Prediction Expression	CBR = -1538.06 – 119.28 x log w + 856.27 x log γ_d – 91.75 x log mold ϕ						

Table 44. Statistical analysis between CBR and moisture, dry density and mold size for silty clay (soil type 4)

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-596.77	67.38	-8.86	<.0001	.	0.77	7.82
log w	-149.64	8.89	-16.83	<.0001	1.0		
log γ_d	395.12	34.19	11.56	<.0001	1.0		
Prediction Expression	CBR = -596.77 – 149.64 x log w + 395.12 x log γ_d						

Note: Mold size is not statistic significant with CBR

Table 45. Statistical analysis between CBR and moisture, dry density and mold size for processed sand gravel (soil type 5)

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-2241.31	715.51	-3.13	0.0027	.	0.615	23.79
log w	-159.21	24.36	-6.54	<.0001	1.0		
log γ_d	1205.96	336.58	3.58	0.0007	1.1		
log mold ϕ	-146.91	25.42	-5.78	<.0001	1.1		
Prediction Expression	CBR = -2241.31 – 159.21 x log w + 1205.96 x log γ_d – 146.91 x log mold ϕ						

Table 46. Statistical analysis between CBR and moisture, dry density and mold size for sand gravel passing ¾” (soil type 5)

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-2618.52	403.32	-6.49	<.0001	.	0.767	14.94
log w	-152.55	16.56	-9.21	<.0001	1.0		
log γ_d	1378.80	191.59	7.20	<.0001	1.0		
log mold ϕ	-128.20	16.05	-7.99	<.0001	1.0		
Prediction Expression	CBR = -2618.52 – 152.55 x log w + 1378.80 x log γ_d – 128.20 x log mold ϕ						

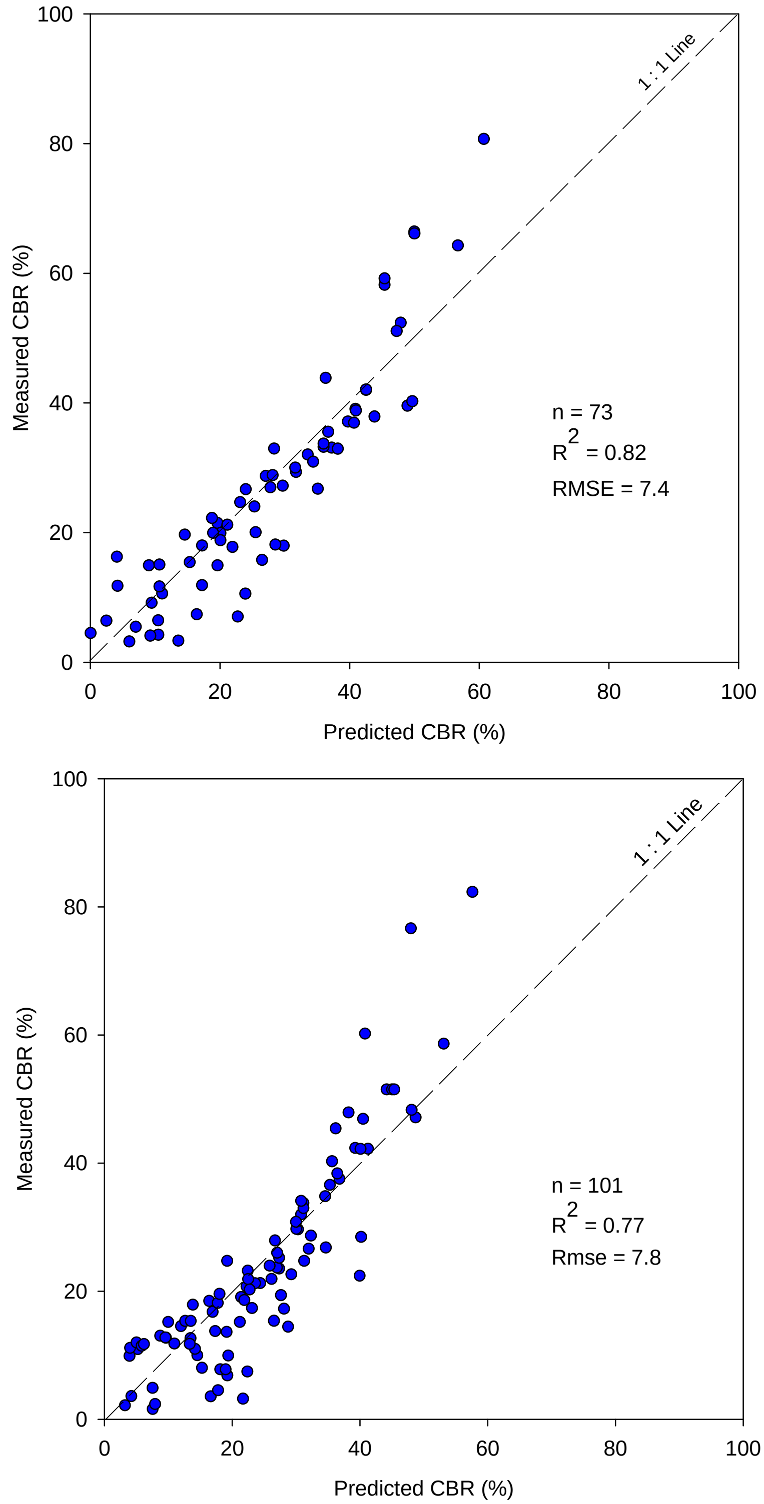


Figure 137. Predicted CBR vs. measured CBR for cohesive soil (clayey silt and silty clay)

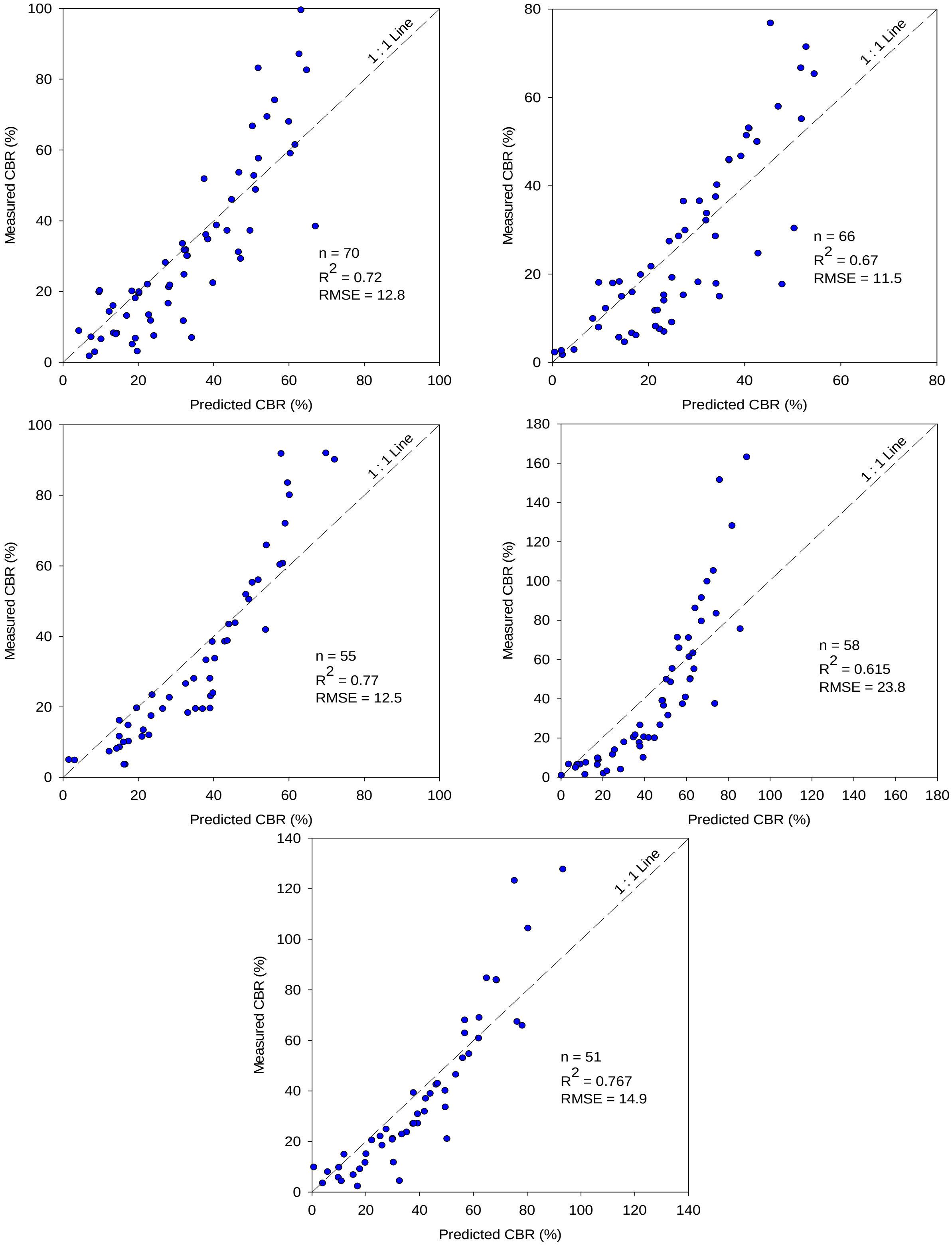


Figure 138. Predicted CBR vs. measured CBR for granular soil (clay gravel, clinton sand and sand gravel)

Table 47. Statistical analysis between CBR and moisture, dry density, mold size, fines content and PI

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-999.19	77.06	-12.97	<.0001	.	0.665	11.4
log w	-131.87	6.43	-20.5	<.0001	4.2		
log γ_d	491.06	34.49	14.24	<.0001	5.2		
log mold ϕ	-25.44	5.23	-4.86	<.0001	1.0		
log PI	-68.53	3.66	-18.74	<.0001	3.2		
log F_{200}	146.17	6.77	21.61	<.0001	13.2		
Prediction Expression	CBR = -999.19 - 131.87 x log w + 491.06 x log γ_d -25.44 x log mold ϕ - 68.53 x log PI + 146.17 x log F_{200}						

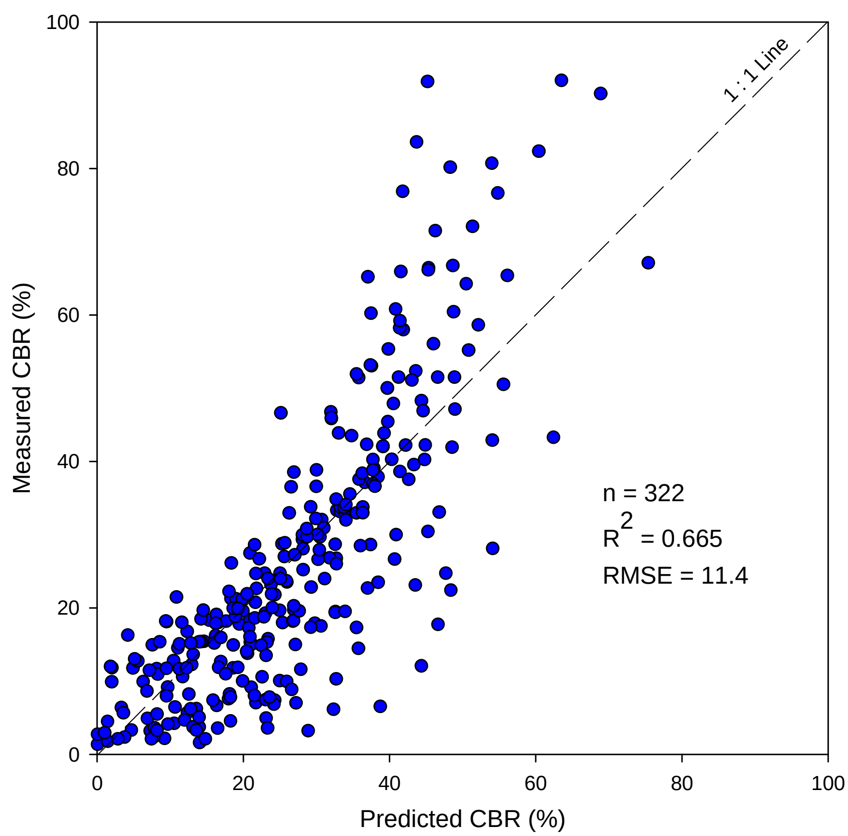


Figure 139. Predicted CBR vs. measured CBR for all soil types

Proposed Procedure for Estimating Field DCP-CBR Target Values

Based on the analysis presented above with U.S. Army Corps of Engineers (1950) data, it is evident that statistically valid models can be developed to estimate target values in relationship with moisture and dry unit weight.

The following procedure is proposed to estimate field DCP-CBR target values:

Determine moisture-dry unit weight relationships in lab using Proctor test

1. Gather enough materials to conduct at least 10-15 Proctor samples. Conduct soil gradation test for the materials.
2. Process these materials in accordance with ASTM D1883 (2005), compact the materials with different compaction energy summarized in Table 48 to determine moisture-density relationship for different compaction energy.
3. Moisture content for each sample should decide based on results from std. Proctor test. (-4, -2, 0, +2, +4 of w_{opt}).

Table 48. Laboratory compaction methods for 6in. mold

Energy Name	Compaction Energy (lb-ft/ft^3)	Lifts	Blows/Lift	Hammer Weight (lb)	Drop Height (in.)	Rel. to w_{opt}
Sub-Sub-Standard (SSS)	5852	3	29	5.5	12	0, +2
Sub-Standard (SS)	7425	3	37	5.5	12	-2, 0, +2
Standard (S)	12400	3	61	5.5	12	(-4, -2, 0, +2, +4)
Sub-Modified (SM)	34650	5	61	5.5	18	-2, 0
Modified (M)	56000	5	61	10	18	-2, 0, +2

Determine DCP-CBR value in CBR mold

4. Conduct DCP test in the CBR mold in accordance with ASTM D6951 (2003).

Develop statistical model relate moisture-density to DCP-CBR

5. Multiple regression analysis should be used to develop a model to relate moisture-density with DCP-CBR.

Develop field target values based on target w and γ_d

6. Establish the relationship between target CBR value and density at certain moisture content.

Example: $w_{opt} = 18\%$, $\gamma_d = 100$pcf, from Std. Proctor

Model: $CBR = b_0 - b_1 \log w + b_2 \log \gamma_d$

DCP-CBR (-2% of w_{opt}, 95% of γ_d) = 10

DCP-CBR (+2% of w_{opt}, 95% of γ_d) = 5

The acceptable CBR value range is ≥ 10 and ≥5 for -2% and +2% of w_{opt}, 95% of γ_d.

QC/QA protocol in situ

7. Measure the in situ moisture content and conduct DCP test, make sure moisture content is in the accept range. If DCP-CBR value reach the target requirements, then the QA have been achieved.
8. If DCP-CBR value does not reach the target requirements, compact the area with two additional passes and redo the DCP test. If the DCP-CBR values achieve the requirement second time, means the QA have been achieved.
9. If the DCP-CBR value still does not reach the target requirements, which means the material is too wet. Disk and compact these area and redo step 7 and 8 until DCP-CBR value achieve the requirement.

Note: That may cost a lot for additional disk and passes because the deeper layer are too wet, so we recommend to do the DCP test for each layer.

Example for estimating target values for western Iowa loess

Western Iowa loess have been studied for laboratory CBR tests. 15 samples have been prepared to determine moisture – density relation through Proctor compaction test. Table 49 lists the dry density and moisture content for each point of the CBR test. The optimum moisture content for std. Proctor and mod. Proctor are 18% and 15%. 6 samples were conducted through standard compaction energy at -4, -2, 0, +1.5, +3, and +4.5 of w_{opt} for std. Proctor, 5 samples were conducted through modify compaction energy at -2, 0, +1, +2, +3 of w_{opt} of mod. Proctor, 1 sample were conducted through sub-sub-standard compaction energy at +2 of w_{opt} of std. Proctor, 2 sample were conducted through sub-standard compaction energy at -4 and 0 of w_{opt} of std. Proctor, 1 samples were conducted through sub-modify compaction energy at -2 of w_{opt} of mod. Proctor. The water content after compaction is lower than the original moisture content. U.S. Army Corps of Engineers (1950) have indicated that for lower water contents, the values before and after compaction are in agreement. When water content increased, free water drains from sample after over compaction due to the reduction in volume of voids, so the water content after compaction is lower. The CBR value have been chosen to use the ratio at 0.2 inch penetration according to ASTM D1883 (ASTM 2005). DCP-CBR data is calculated by equation 3 from DCP test. Figure 140 shows the 15 moisture - density points at 5 different compaction energy levels.

Table 49. CBR at different moisture content and different compaction energy

Test	γ_{dmax} (lb/ft^3)	w (%)	Rel. to w_{opt}	Compaction Energy (lb-ft/ft^3)	CBR (%)	DCP–CBR (%)
1	98.5	17.5	-0.5 Std.	SSS 4850	9.6	12
2	97.2	13.8	-4.2 Std.	SS 7425	18	21
3	101.5	17.0	-1 Std.	SS 7425	19	17
4	100.9	12.4	-5.6 Std.	S 12400	30	32
5	104.2	15.7	-2.3 Std.	S 12400	30	24
6	105.0	17.2	-0.8 Std.	S 12400	25	21
7	106.0	17.8	-0.2 Std.	S 12400	19	18
8	104.4	19.5	+1.5 Std.	S 12400	4.8	10
9	100.6	21.2	+3.2 Std.	S 12400	1.3	4.2
10	109.7	12.6	-2.4 Mod.	SM 34650	65	71
11	113.2	12.4	-2.6 Mod.	M 56000	99	81
12	114.5	13.8	-1.2 Mod.	M 56000	85	51
13	113.8	14.8	-0.2 Mod.	M 56000	47	42
14	111.5	16.4	+1.4 Mod.	M 56000	7.9	20
15	107.9	17.8	-0.2 Std.	M 56000	3.3	13

Then we can establish the relationship between target CBR value and density at certain moisture content, and set up a QC/QA protocol in situ standard. For example, from Figure 140 and Figure 143, we can predict a DCP-CBR value of 19 at -2% of w_{opt} with 95% of γ_d, so the DCP-CBR needs to be higher than 19 for in situ moisture content of 16% to achieve the 95% of γ_d requirements.

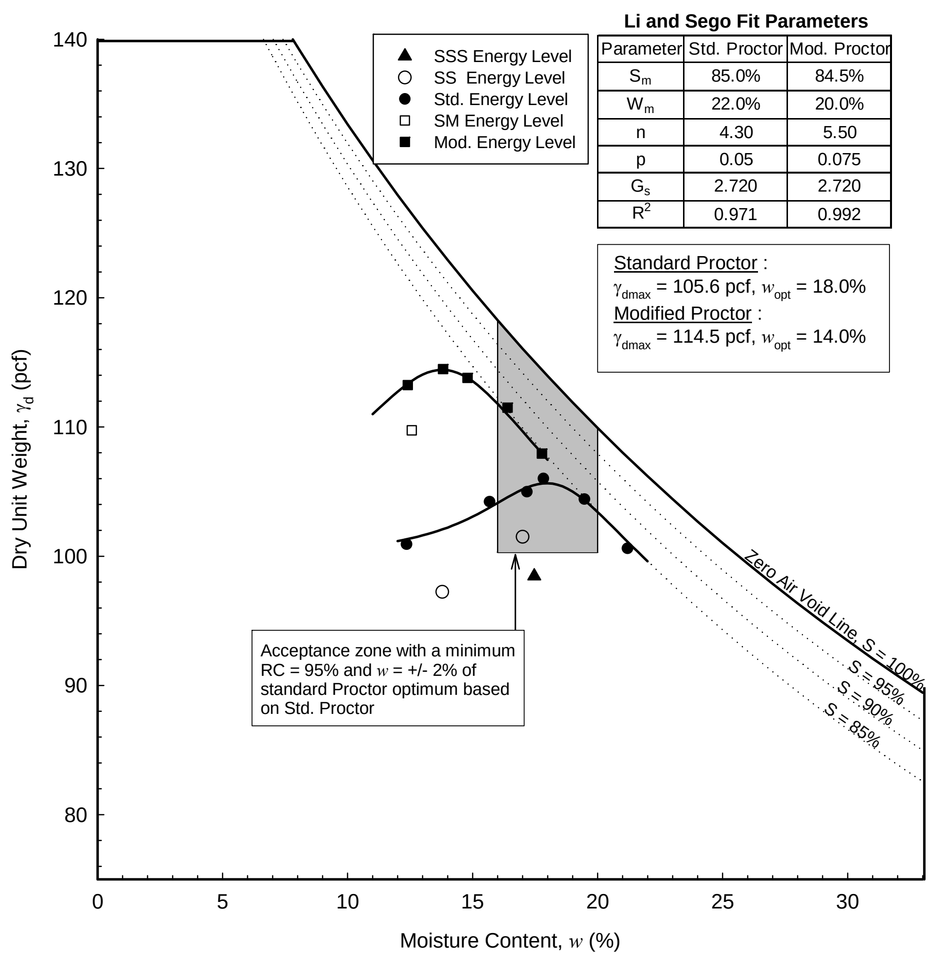


Figure 140. CBR test results with Proctor curve

After compaction, laboratory CBR test and DCP test were conducted in the mold. Figure 141 is the results from DCP test at each CBR mold. The material is about 4 in. deep.

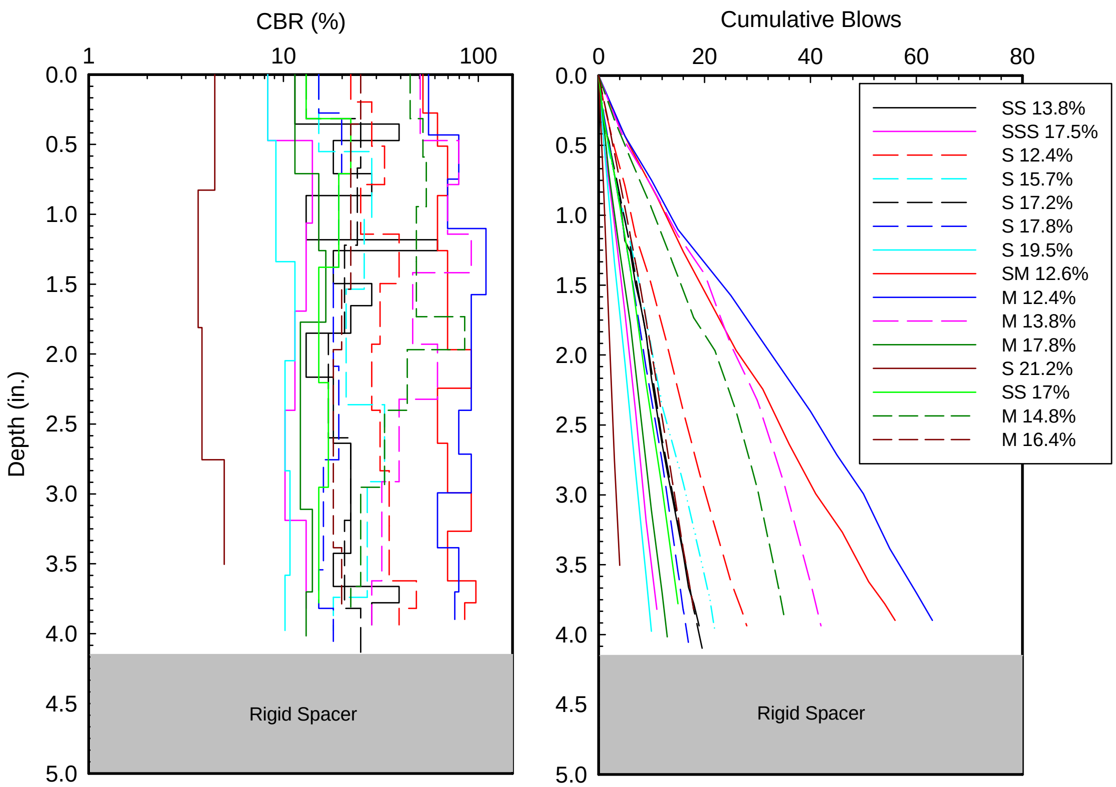


Figure 141. DCP results

Figure 142 and Figure 143 shows the moisture content vs. CBR in different compaction energy. Not like Proctor curve, because of the pore water pressure in the mold, the CBR value after modify compaction energy are lower than the CBR value by standard compaction energy when the moisture content goes higher than 17%.

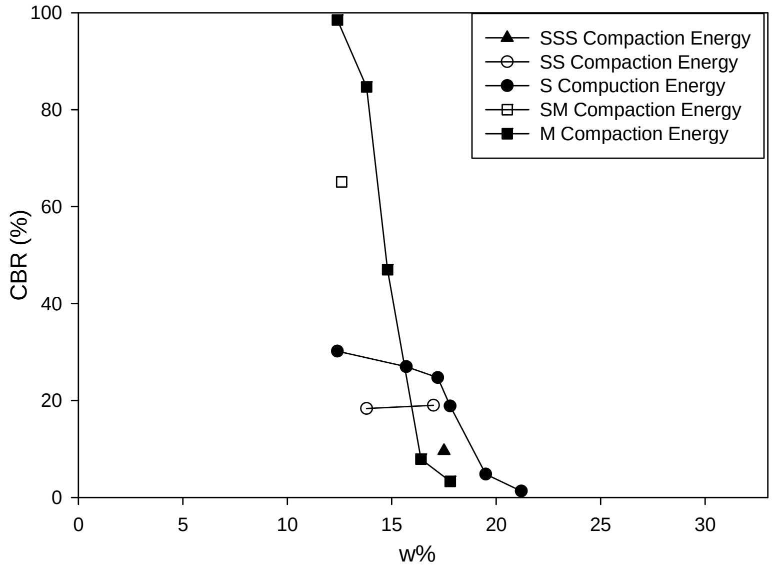


Figure 142. Moisture content vs. laboratory CBR

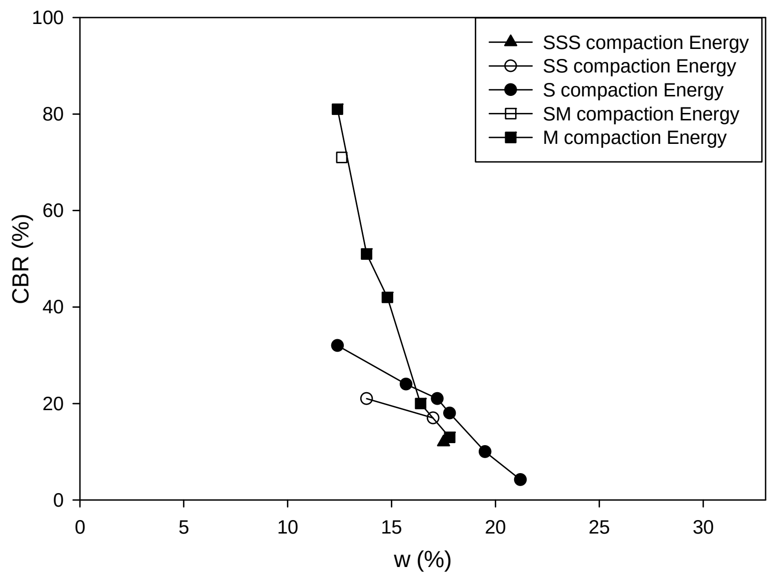


Figure 143. Moisture content vs. DCP – CBR

Figure 144 shows a relationship between the laboratory and field CBR values. This result cannot exactly reflect the relationship between the two groups of CBR values. The laboratory CBR was not equal to the field CBR of the same material.

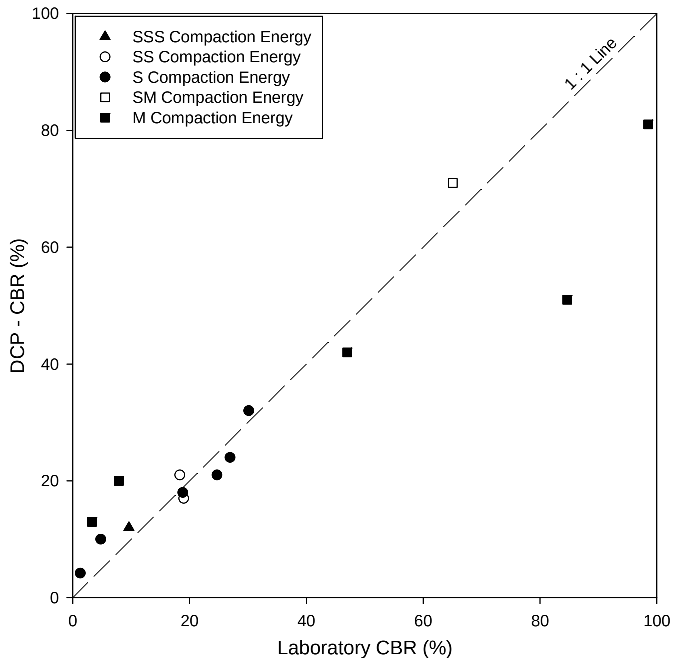


Figure 144. Laboratory CBR vs. DCP – CBR

Statistical multiple regression analysis was conducted through using JMP statistical analysis software to assess influence of moisture content (w), dry unit weight (γ_d). Compaction energy was not statistically significant for this soil type. For both laboratory and DCP CBR, the analysis results summarizing the influence of *w* and γ_d are summarized in Table 50 and Table 51; graphs of predicted versus measured values based on the regression models are presented in Figure 145 and Figure 147; Figure 146 and Figure 148 shows the corresponding moisture and density at different CBR values.

Table 50. Statistical analysis between laboratory CBR and moisture and dry density

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R^2	RMSE
Intercept	-833.59	429.91	-1.94	0.0764	.	0.714	16.2
log w	-255.5	62.8	-4.07	0.0016	1.1		
log γ_d	578.28	197.23	2.93	0.0126	1.1		
Prediction Expression	Laboratory CBR = -833.59 – 255.5 x log w + 578.28 x log γ_d						

Note: Compaction energy is not statistic significant with CBR

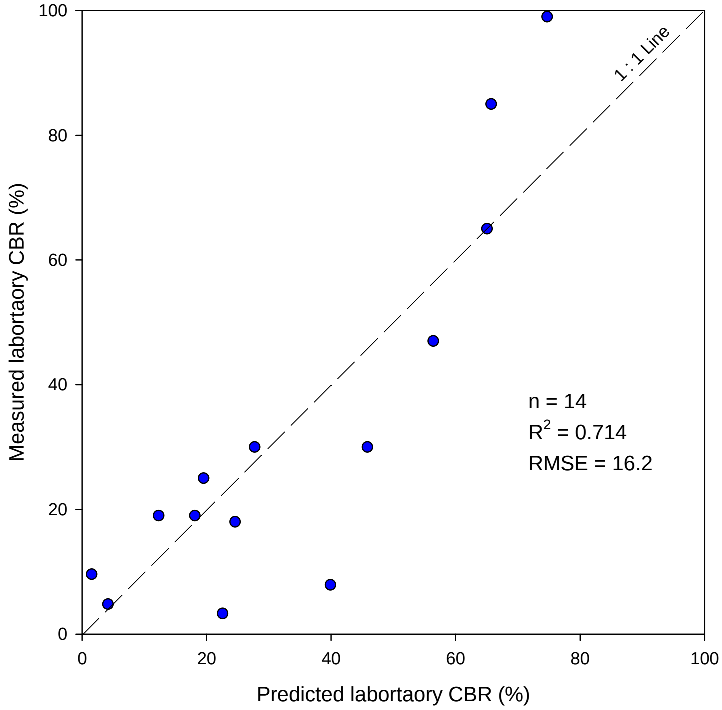


Figure 145. Predicted lab CBR vs. measured lab CBR for western Iowa loess

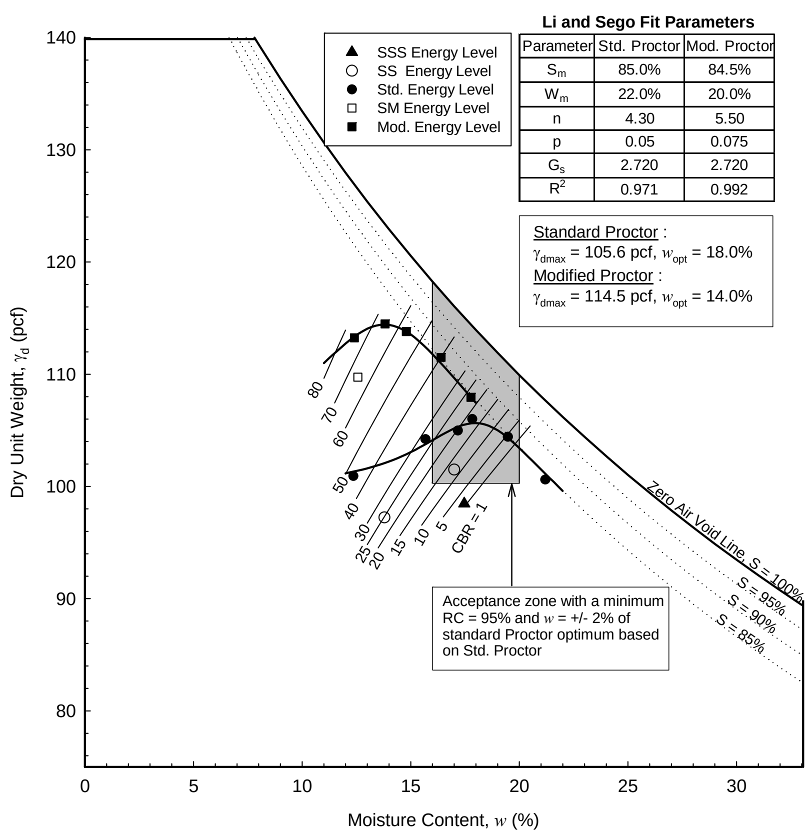


Figure 146. Relationship between w, γ_d, and laboratory CBR for western Iowa loess

Table 51. Statistical analysis between DCP-CBR and moisture and dry density

Parameter Estimates							
Term	Estimate	Std. Error	t Ratio	Prob>|t|	VIF	R2	RMSE
Intercept	-532.77	266.2	-2.00	0.0685	.	0.804	10.0
log w	-213.66	38.89	-5.49	0.0001	1.1		
log γ_d	404.04	122.13	3.31	0.0062	1.1		
Prediction Expression	DCP-CBR = -532.77 – 213.66 x log w + 404.4 x log γ_d						

Note: Compaction energy is not statistic significant with CBR

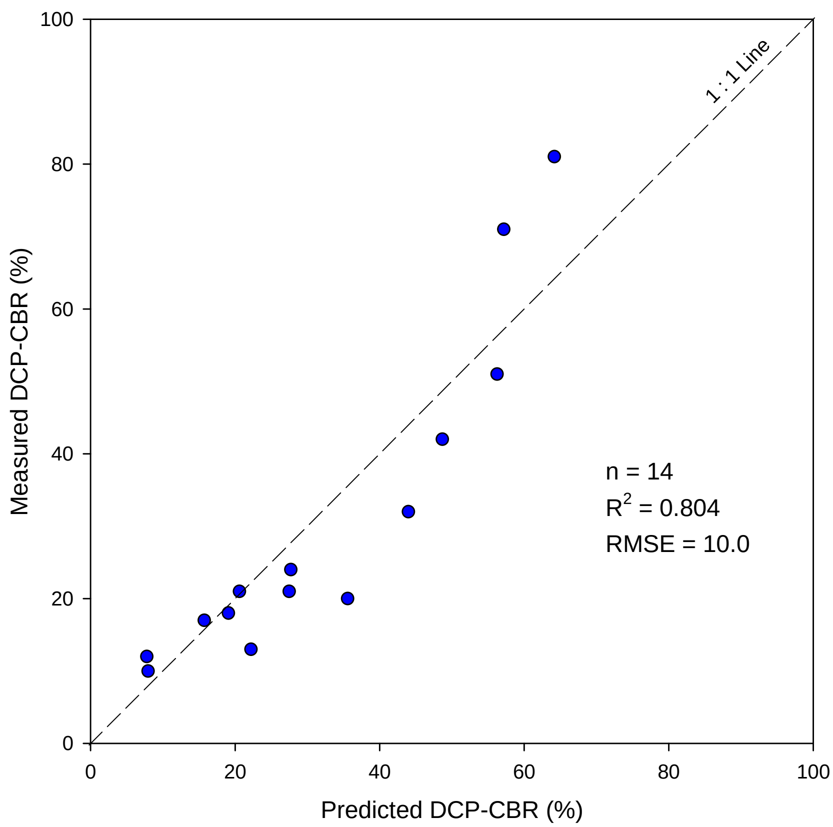


Figure 147. Predicted DCP-CBR vs. measured DCP-CBR for western Iowa loess

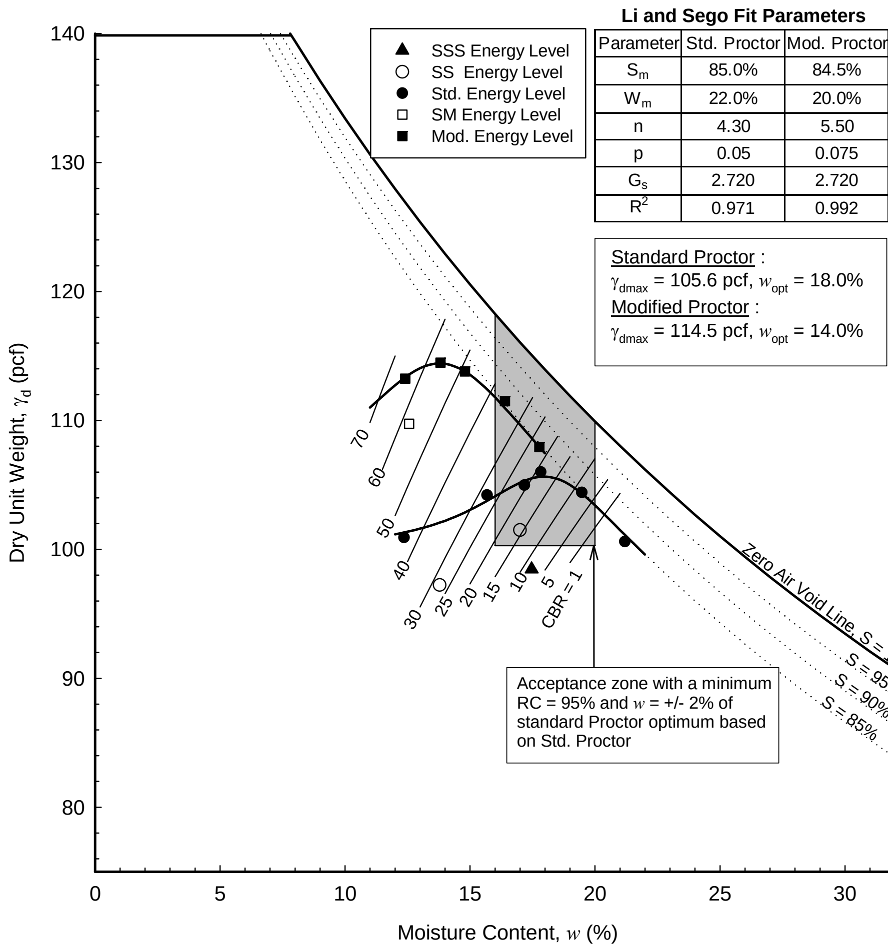


Figure 148. Relationship between w, γ_d, and DCP-CBR for western Iowa loess

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

This chapter presents an overview of the technical merit and scientific value gained from the study and an overview of the lessons learned. The conclusions are presented in three sections, field and lab compaction test results, estimating CBR target values, and followed by recommendations for future research and practice.

FIELD AND LAB COMPACTION TEST RESULTS

Drive core cylinder, Proctor compaction and DCP test have been conducted on embankment materials from 9 project sites to compare the compaction results. Field test results indicated that embankments are frequently constructed outside the QC/QA requirements from Iowa standard specification. 20 out of the 25 projects consisted of 10 to 100% of the data outside the moisture acceptance limits. 17 out of the 22 field projects consisted of 20 to 100% of the data outside the moisture and density acceptance limits.

ESTIMATING CBR TARGET VALUES

One of the most often used parameters to evaluate subgrade/subbase strength for the pavement design is the CBR value. Traditional field CBR testing can be expensive and time consuming. The DCP device is a helpful tool to estimate in situ CBR value. It is known that the CBR value has a relationship with moisture content, dry density, plasticity and fines content, so this report analyzed these parameters to estimate CBR value using multiple linear regression analysis. Data published in the U.S. Army Corps of Engineers (1950) report was used in the analysis, to assess relationships between the parameters listed above, as well as the influence of mold size. The results demonstrated statistical relationships and provided statistical models to predict the CBR values. Results indicated that the relationships between

CBR, moisture, and density are unique to a soil type. Also, mold had a significant effect on the CBR values, especially for granular materials.

This research proposed a practical procedure for estimating target CBR value in reference to moisture and dry density, using laboratory testing and statistical analysis.

SUMMARY OF CONCLUSIONS

The goal of this research was to review grading projects statewide and assess the implementation of compaction with moisture control and contractor quality control operations during embankment construction. Field test results indicated that the majority of the embankment construction projects in Iowa are frequently constructed outside the QC/QA requirements from Iowa standard specification. This research demonstrated that DCP can simply, quickly and inexpensively assess field conditions, and provide a record of shear strength and stiffness profile up to a depth of about 3 ft.

RECOMMENDATIONS FOR FUTURE RESEARCH

The following future work is recommended to build upon the findings from this research:

- Add DCP for field testing method to ensure compliance with moisture control criteria. Specify DCP-CBR value with moisture and density relationship chart or table for different kinds of materials that contractors can follow when they conduct DCP test;
- More CBR tests need to be conducted with multiple soil types and mold sizes, gather and analysis these data to predict CBR value in a more advanced way, and
- Consider the flowchart in Figure 149 for estimating field DCP-CBR target values in QC/QA program.

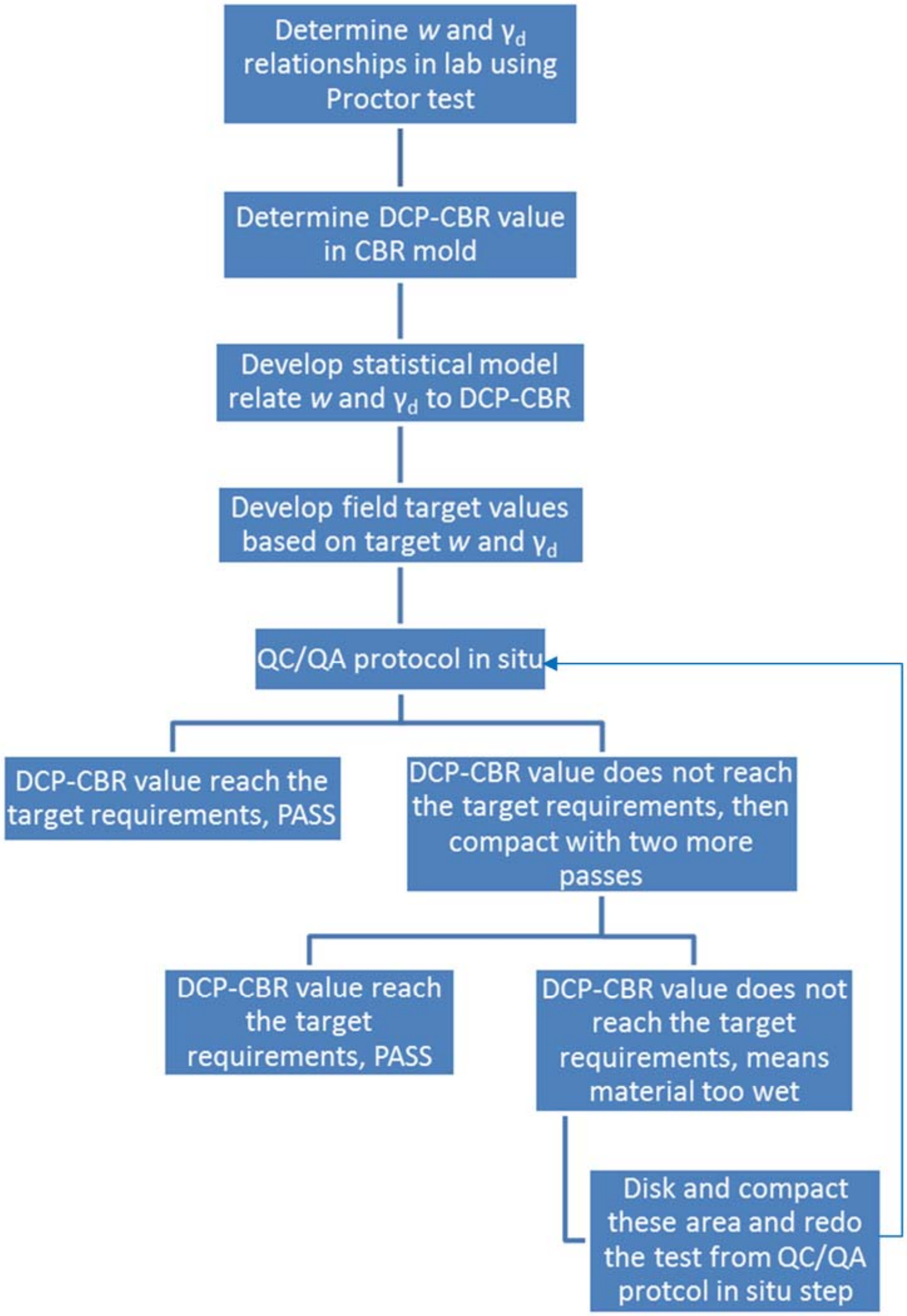


Figure 149. Proposed Iowa DOT flowchart for estimating field DCP-CBR target values in QC/QA program

WORKS CITED

ASTM (2002). "ASTM D1557 - 02 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft." <http://www.astm.org/DATABASE.CART/HISTORICAL/D1557-02.htm>. (April 18, 2015).

ASTM (2003). "ASTM D6951 - 03 Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications." <http://www.astm.org/DATABASE.CART/HISTORICAL/D6951-03.htm>. (April 18, 2015).

ASTM (2005). "ASTM D1883 - 05 Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils." <http://www.astm.org/DATABASE.CART/HISTORICAL/D1883-05.htm>. (April 18, 2015).

ASTM (2006). "ASTM D2487 - 06 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)." <http://www.astm.org/DATABASE.CART/HISTORICAL/D2487-06.htm>. (April 18, 2015).

ASTM (2007). "ASTM D422 - 63(2007)e2 Standard Test Method for Particle-Size Analysis of Soils." <http://www.astm.org/Standards/D422.htm>. (April 18, 2015).

ASTM (2007). "ASTM D698 - 07 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft." <http://www.astm.org/DATABASE.CART/HISTORICAL/D698-07.htm>. (April 18, 2015).

ASTM (2009). "ASTM D3282 - 09 Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes." <http://www.astm.org/Standards/D3282.htm>. (May 3, 2015).

ASTM (2009). "ASTM D6951 / D6951M - 09 Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications." ASTM International, West Conshohocken, PA.

ASTM (2010). "ASTM D854 - 10 Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer." <http://www.astm.org/DATABASE.CART/HISTORICAL/D854-10.htm>. (April 18, 2015).

ASTM (2010). "ASTM D2937 - 10 Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method." <http://www.astm.org/Standards/D2937.htm>. (April 18, 2015).

ASTM (2010). "ASTM D4318 - 10 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." <http://www.astm.org/DATABASE.CART/HISTORICAL/D4318-10.htm>. (April 18, 2015).

ASTM (2011). "ASTM E2583 - 07(2011) Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD)." <http://www.astm.org/Standards/E2583.htm>. (April 18, 2015).

Bandyopadhyay, K., and Bhattacharjee, S. (2010). "Comparative Study Between Laboratory and Field CBR by DCP and IS Method." presented at 2010 Indian Geotechnical Conference, Kolkata, Indian, December.

Bergeson, K., Jaahren, C., Wermager, M., and White, D. J. (1998). "Embankment Quality, Phase I Report," TR-401, Iowa Department of Transportation, Ames, Iowa.

Bergeson, K., and Jahren, C. (1999). "Embankment Quality: Phase II." TR-401, Iowa Department of Transportation, Ames, Iowa.

Burnham, T., and Johnson, D. (1993). "In situ foundation characterization using the dynamic cone penetrometer." Final report, Iowa Department of Transportation, Ames, Iowa..

Choudhary, A. K., Jha, J. N., and Gill, K. S. "Utilization of Plastic Wastes for Improving the Sub-Grades in Flexible Pavements." pp 320-326.

Gill, K. S., Jha, J. N., and Choudhary, A. K. (2010). "CBR Value Estimation Using Dynamic Cone Penetrometer." presented at 2010 Indian Geotechnical Conference, Ludhiana, Indian, December.

Iowa DOT. (2012). "Standard Specifications for Highway and Bridge Construction." Iowa Department of Transportation, Ames, Iowa.

Jung, Y. s., Zollinger, D. G., Cho, B. H., Won, M., and Wimsatt, A. J. (2012). "Subbase and Subgrade Performance Investigation and Design Guidelines for Concrete Pavement." 170p.

Larsen, B. W., White, D. J., and Jahren, C. T. "Embankment Construction QC/QA Using DCP and Moisture Control: Iowa Case History for Unsuitable Soils." Center of Transportation Research and Education, Iowa State University, Ames, IA.

Li, H., and Sego, D.C. (1999). "Soil Compaction Parameters and Their Relationship With Soil Physical Properties." 52nd Canadian Geotechnical Conference Regina, Saskatchewan, Canada, p. 517-524.

Li. H., and Sego. D.C. (2000a). "Equation for Complete Compaction Curve of Fine-grained Soils and Its Applications." Constructing and Controlling Compaction of Earth Fills, ASTM STP 1384 pg. 113-125.

Li. H., and Sego, D.C. (2000b). "The Strength Behavior Along Complete Compaction Curves." 53rd Canadian Geotechnical Conference Montreal, 8, pg. 1195-1202.

Proctor, R. R. (1933). "Fundamental principles of soil compaction." Engineering News Record, 111, 245-248.

Proctor, R.R. (1948). "Laboratory Soil Compaction Methods, Penetration Resistance Measurements and the Indicated Saturated Penetration Resistance." *Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering,* 5, 242-247.

Siekmeier, J., Pinta, C., Merth, S., Jensen, J., Davich, P., Camargo, F., and Beyer, M. (2009). "Using the Dynamic Cone Penetrometer and Light Weight Deflectometer for Construction Quality Assurance." 244p.

SUDAS. (2015). Design Manual Chapter 6 – Geotechnical, 6E – Subgrade Design and Construction, 2013 Edition, Statewide Urban Design and Specifications (SUDAS), Institute of Transportation, Iowa State University, Ames, Iowa.

U.S. Army Corps of Engineers (1950). "Soil Compaction Investigation Report No. 5, Miscellaneous Laboratory Tests."Vicksburg, Mississippi.

Vennapusa, P. K. R., and White, D. J. (2009). "Comparison of Light Weight Deflectometer Measurements for Pavement Foundation Materials." *Geotechnical Testing Journal*, 32(3), pp 239-251.

Walsh, K. D., Houston, W. N., and Houston, S. L. (1997). "Field Implications Of Current Compaction Specification Design Practices." *Journal of Construction Engineering and Management*, 123(4), p. 363-370.

White, D., Vennapusa, P., Zhang, J., Gieselman, H., and Morris, M. (2009). "Implementation of Intelligent Compaction Performance Based Specifications in Minnesota."

White, D. J., Bergeson, K. L., and Jahren, C. T. (2002). "Embankment Quality: Phase III." TR-401, Iowa Department of Transportation, Ames, Iowa.

White, D. J., Larsen, B. W., Jahren, C., and Malama, J. (2007). "Embankment Quality Phase IV: Application to Unsuitable Soils." TR-401, Iowa Department of Transportation, Ames, Iowa.

White, D. J., Rupnow, T. D., and Ceylan, H. (2004). *Influence of subgrade/subbase non-uniformity on PCC pavement performance*. American Society of Civil Engineers, 8 p

White, D. J., Thompson, M. J., and Vennapusa, P. (2007). "Field Validation of Intelligent Compaction Monitoring Technology for Unbound Materials." 412p.

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APPENDIX A. STATE SPECIFICATION FOR EMBANKMENT CONSTRUCTION OF GRANULAR MATERIALS

Table 52. 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" (150 mm).	8 to 12 in.	near w_{opt}	≥ 95% of maximum γd	

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 $\geq$ 95% of maximum γd. Others $\geq$ 90% of maximum γd.	
CO	2011	specify density	NR	less than maximum rock size or 3 ft.	$\leq$ +/-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	$\geq$ 95% of maximum γd	
CT	2008	specify density	NR	maximum 3 ft loess thickness	at wopt	$\geq$ 95% of maximum γd in accordance with AASHTO T 180, Method D.	
DE	2001	NR	NR	maximum 2 ft loess thickness	$\leq$ +/-2% of wopt	$\geq$ 95% of maximum γd by AASHTO T 99 Method C, Modified.	
FL	2015	NR	NR	NR	NR	Compact top 6 in $\geq$ 100% of maximum γd	
GA	2013	NR	Ensure that thickness of the lifts and the compaction are approved by the Engineer.				

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.	

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

State	Spec date	Placement/ compaction method	Disk/Passes	Lift Thickness	w	DD	Other 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	
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 Y99	
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 T180, 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 T99	
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	

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 250psi 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.

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,	$\geq$ 95% of maximum γ_d with $\leq$ +/-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	

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	

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	

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	

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		

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 $\leq$ 15, no moisture content required, density $\geq$ 98% γd		

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.

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 99	

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

Table 53. 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

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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	
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.	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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.	
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	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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	
		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			

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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.	
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 Dynamic Cone Penetrometer, DCP, in accordance with ASTM D 6951

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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.				
		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.	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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 T180, Method C or D,	
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 T99	
MI	2012	specify density	NR	maximum 9 in. loess thickness	≤ +3% of wopt	≥ 95% of maximum γd	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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 percent passing the No. 200 [75 µm] sieve.	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
		95% Relative Density Remaining embankment in the road core					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.	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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.
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.	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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; Pad foot 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	
		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	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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 inch.	
		Compaction Control, Type C.		maximum 8 in. loess thickness	NR	NR	

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 pcf, requires at least 102% maximum dry density compaction energy; if maximum γd from 105 to 119.9 pcf, requires at least 100% maximum dry density; if maximum γd more than 120 pcf, 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.	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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.	
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	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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		
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	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
		Method A		maximum 2 ft loess thickness	NR	The Contractor shall compact each layer by routing loaded haul equipment over its entire width.	
WA	2015	Method B	NR	Top 2 ft, maximum 4 in. loess thickness. Below top 2 ft, maximum 8 in. loess thickness. Up to maximum 18 in. loess thickness after engineer permit	≤ +3% of wopt.	2 ft below finish subgrade ≥ 90% of maximum γd, rest 2 ft to finish subgrade ≥ 95% of maximum γd	
		Method C				≥ 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	
		Standard Compaction				Compact each layer of the embankment until the compaction equipment achieves no further significant consolidation.	
WI	2014	Special Compaction	NR	maximum 8 in. loess thickness	NR	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	

State	Spec date	Placement/compaction method	Disk/Passes	Lift Thickness	w	DD	Other requirements
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		