Report No. CDOT-2008-11
Final Report

IMPROVING QUALITY ASSURANCE OF MSE WALL AND BRIDGE APPROACH EARTHWORK COMPACTION

Michael A. Mooney, Christopher S. Nocks, Kristi L. Selden,
Geoffrey T. Bee, Christopher T. Senseney

October 2008

COLORADO DEPARTMENT OF TRANSPORTATION
DTD APPLIED RESEARCH AND INNOVATION BRANCH

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
CDOT-2008-11		

4. Title and Subtitle IMPROVING QUALITY ASSURANCE OF MSE WALL AND BRIDGE APPROACH EARTHWORK COMPACTION	5. Report Date 6. Performing Organization Code
7. Author(s) Michael A. Mooney, Christopher S. Nocks, Kristi L. Seldon, Geoffrey T. Bee, Christopher T. Senseney	8. Performing Organization Report No. CDOT-2008-11
9. Performing Organization Name and Address Colorado School of Mines 1500 Illinois Street Golden, Colorado 80401	10. Work Unit No. (TRAIS) 11. Contract or Grant No. 80.24
12. Sponsoring Agency Name and Address Colorado Department of Transportation - Research 4201 E. Arkansas Ave. Denver, CO 80222	13. Type of Report and Period Covered 14. Sponsoring Agency Code

15. Supplementary Notes
Prepared in cooperation with the US Department of Transportation, Federal Highway Administration

16. Abstract
The objective of the study was to investigate the efficacy of new devices for quality assurance (QA) of Class 1 backfill in MSE wall and bridge approach earthwork compaction. Extensive testing at two construction sites, an MSE wall project near Golden, CO, and multiple MSE wall/bridge approach projects near Wheat Ridge, CO, revealed that the light weight deflectometer (LWD), dynamic cone penetrometer (DCP), and Clegg Hammer are all capable of assessing the compacted state of Class 1 backfill. Field test data also revealed that the current CDOT practice of single position nuclear gauge (NG) testing is inadequate. An evaluation of field data demonstrated that target values (TVs) exist for the LWD modulus (E_{LWD}), Clegg Impact Value (CIV), and DCP penetration index that could serve as surrogates for the current 95% compaction requirement. The DCP exhibited two key limitations: moisture sensitivity and penetration resistance from placed geogrid (MSE walls) and geofabric (bridge approach). Because the LWD and Clegg Hammer do not possess the same limitations, these two devices were deemed most suitable for QA of MSE wall and bridge approach earthwork compaction.

Implementation:
We recommend CDOT implement a pilot study using the light weight deflectometer and Clegg Hammer in conjunction with nuclear gauge testing with the following objectives: (1) identify E_{LWD} and CIV target values for various soils, site & moisture conditions, seasons, etc.; (2) evaluate if/how target values change with these conditions; (3) populate a database of target values; and (4) allow CDOT inspectors, consultants, and contractors to evaluate the devices.

17. Keywords	18. Distribution Statement
Class 1 backfill, light weight deflectometers (LWD), Clegg Hammers, dynamic cone penetrometers (DCP), nuclear gauges (NG), QA, mechanically stabilized earth retaining walls	No restrictions. This document is available to the public through the National Technical Information Service Springfield, VA 22161; www.ntis.gov

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	77	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

IMPROVING QUALITY ASSURANCE OF MSE WALL AND BRIDGE APPROACH EARTHWORK COMPACTION

by

Michael A. Mooney, Ph.D., P.E., Associate Professor
Christopher S. Nocks, 1LT, USAF, Former Graduate Student
Kristi L. Selden, Undergraduate Student
Geoffrey T. Bee, Undergraduate Student
Christopher T. Senseney, P.E., MAJ, USAF, Graduate Student

Report No CDOT-2008-11

Prepared by
Colorado School of Mines
Division of Engineering
1500 Illinois Street
Golden, Colorado 80401

Sponsored by the
Colorado Department of Transportation
In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

October 2008

Colorado Department of Transportation
DTD Applied Research and Innovation Branch
4201 E. Arkansas Ave.
Denver, CO 80222
(303) 757-9506

ACKNOWLEDGEMENTS

The authors wish to thank CDOT's DTD Applied Research and Innovation Branch for funding this study, and the panel including C.K. Su, Dr. H.C. Liu, Mark Vessely, Dr. Trever Wang, Lynn Croswell, Matt Greer, and Dr. Aziz Khan for investing their time to oversee the project. We are extremely grateful to the many CDOT personnel and consultants at project sites that helped during this study. We would particularly like to acknowledge Mr. Martin Herbaugh, Project Manager for the SH40 - I70 project and Mr. Rene Valdez, Project Manager for the SH58 - I70 project, without whose support this project would not have been possible.

EXECUTIVE SUMMARY

This report presents the findings from CDOT Study 80.24, Improving Quality Assurance of MSE Wall and Bridge Approach Earthwork Compaction. The objective of the study was to investigate the efficacy of new devices for quality assurance (QA) of Class 1 backfill in MSE wall and bridge approach earthwork compaction. The report documents the preliminary assessment of a number of potential earthwork QA devices, and recommends further investigation through field testing for the dynamic cone penetrometer (DCP), light weight deflectometer (LWD), and Clegg Hammer.

Field testing was conducted at two sites – an MSE wall construction project at the intersection of I-70 and State Highway (SH) 40 near Golden, CO, and multiple MSE wall/bridge approach construction projects at the intersection of I-70 and SH58 near Wheat Ridge, CO. DCP, Clegg Hammer, LWD, and nuclear gage (NG) tests were performed on numerous test beds at these sites.

Analysis of field data revealed that the current CDOT practice of single position NG testing is inadequate. The uncertainty in single position NG density was found to be equivalent to ± 3-4 percent compaction (%C). The orientation of the device alone (e.g., north, south, east, west) often determined whether passing (> 95 %C) or failing (< 95 %C) density was achieved. Moreover, soil is heterogeneous, and soil density, shear strength and modulus vary spatially. This heterogeneity should be accounted for through a statistically-based QA approach. To better represent the variability in soil density and to minimize uncertainty in reported data, we recommend that CDOT increase the required number of tests per lot (evaluation area). A specific number is not recommended here as it was not the focus of this study; however, an approach similar to that recommended in Section 5.2.3 could be pursued. At an absolute minimum, CDOT could improve the current procedure with no additional time/cost by modifying the current inspection practice from a single position 4 minute reading to a four position NG test (i.e., north, south, east, west) with each position as a 1-minute reading. A revised approach could be investigated within the pilot implementation of new devices as described below.

Extensive testing on MSE wall and bridge approach earthwork compaction sites revealed that the LWD, DCP, and Clegg Hammer are all capable of reflecting the compacted state of Class 1 structure backfill soil. E_{LWD} (modulus of elasticity or soil stiffness as measured by LWD in MPa), CIV (Clegg impact value of soil stiffness as measured by Clegg Hammer without units), and $\overline{\mathrm{DPI}}$ (average DCP penetration index as measured in mm/blow) are much more sensitive to changes in compaction than density. While dry density ranged by 20% from typical uncompacted to fully compacted states, E_{LWD}, CIV and $\overline{\mathrm{DPI}}$ were found to vary by 500%, 400% and 1000% respectively. Testing with all devices revealed that adequate compaction is not being achieved within 1 m (3 ft) of MSE wall faces. This is attributed to the compaction procedure where contractors are reluctant to use vibratory rollers within 1 m of the wall. The vibratory plates used in this zone are not providing adequate compactive effort. Inadequate compaction in this zone is exacerbated by the measurement restriction of the NG within 1.6 m (5 ft) of the wall.

An evaluation of the data reveals that target values (TVs) exist for E_{LWD}, CIV and $\overline{\mathrm{DPI}}$ that could serve as surrogates for the current 95 %C density requirement. Over multiple sites, test beds and Class 1 backfill soils, consistent TVs emerged. The observed TVs for E_{LWD}, CIV and $\overline{DPI}$ were found to be 32.5 MPa, 11.9 and 10.2 mm/blow. Within the scatter of the data, the TVs did not vary across the different Class 1 backfill soils. $\overline{DPI}$ values appear to be sensitive to moisture while E_{LWD} seem to be insensitive to moisture for these soils tested. The moisture sensitivity of CIV was inconclusive. Moisture sensitivity constitutes a limitation to implementing the TV approach in the absence of NG testing because moisture would need to be measured. Currently, LWD, DCP, and Clegg Hammer devices do not include moisture measurement.

The LWD, DCP, and Clegg Hammer are all capable of evaluating soil properties within 0.3 m (1 ft) of the wall face. This is a significant advantage over the 1.5 m (5 ft) wall proximity restriction of the NG particularly in light of the inadequate compaction observed and measured within 1 m (3 ft) of MSE walls. E_{LWD} is most closely aligned with design parameters (e.g., modulus) for pavements, while CIV is an index parameter that is currently not linked to design parameters. The DCP exhibited two key limitations. Because the DCP test involves penetration through the soil, the DPI is influenced by the placed geogrid (MSE walls) and geofabric (bridge approach). In addition, the moisture sensitivity of $\overline{DPI}$ values requires consideration of moisture in developing TVs and evaluating acceptance. When compared to the LWD and Clegg Hammer results where moisture sensitivity was not clearly observed, DCP implementation requires additional effort.

Implementation Statement

The LWD and Clegg Hammer are both deemed suitable QA devices for MSE wall and bridge approach Class 1 structure backfill. They were found to be equally effective in capturing the degree of compaction. The Clegg Hammer is less expensive and easier to use than the LWD. Conversely, the LWD produces a modulus that can be tied to design and there is significant momentum nationally towards LWD use. To move towards CDOT-wide formal implementation, we recommend that CDOT implement a pilot study using the LWD and Clegg Hammer in conjunction with NG testing on 5-10 MSE wall and/or bridge approach construction sites. The objectives of the pilot program are multiple: (1) identify E_{LWD} and CIV target values (TVs) for the various soils, site & moisture conditions, seasons, etc., observed in practice; (2) evaluate if/how TVs change with soil type, moisture, season, and from site to site; (3) populate a database of TVs; (4) allow a range of CDOT inspectors, consultants and contractors to evaluate all aspects of the devices, e.g., handling, operation, durability, portability. Details of the recommended pilot implementation procedure are provided in Chapter 5.

The pilot implementation will reveal and confirm the efficacy of the LWD and Clegg hammer as a supplement or replacement for the NG in earthwork QA. In addition, the results of the pilot study combined with the findings herein will lead to specific guidelines for TVs, number of required tests, statistical approach to data analysis and acceptance criteria, lot size, etc. A specification can then be written to replace the appropriate sections of the CDOT Bridge Project Special Provisions, the Standard Specifications for Road and Bridge Construction, and the Field Materials Manual.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1: OVERVIEW

1.1 Introduction

Proper earthwork compaction is a *critically important factor* for satisfactory performance of highways, bridges and mechanically stabilized earth walls. Inadequate compaction of soils and aggregates leads to bridge approach settlement and poor performance of mechanically stabilized earth walls (lateral movement, settlement). CDOT is acutely aware of the importance of earthwork preparation. Many mechanically stabilized earth walls (MSE) and bridge approach failures in Colorado can be attributed to lack of adequate and uniform compaction. Current CDOT earthwork quality assurance (QA) exclusively involves nuclear gage (NG) density testing. NG testing has limitations. The NG is a 10-15 minute test, and therefore, inspection is performed infrequently at discrete locations. As a result, the vast majority of the earth structure remains untested. The NG can not be used within 5 feet of an MSE or abutment wall due to obstruction-induced false readings, thus leaving this *critical area untested*. The NG is inaccurate when large size particles are present. Finally, regulatory constraints of using a nuclear based device make operating and maintaining the NG burdensome and costly.

A number of new and proven devices, e.g., the light weight deflectometer (LWD), Clegg Impact Hammer and the dynamic cone penetrometer (DCP), are capable of *more rapid* inspection of materials *with large size aggregates* to *depths of 24 inches or more* (for DCP). A positive evaluation of these devices would enable CDOT to adopt improved QA techniques for MSE wall and bridge approach QA. The objectives of this study were to:

I. Identify the most appropriate device(s) and the appropriate methodology for QA of compacted Class 1 structure backfill material in bridge approach and MSE wall projects.

II. Outline the steps to adopt the new devices and methodologies in CDOT QA in two stages:
   - Short-Term: One or more of the new devices could initially be adopted to supplement NG testing. This will allow the continued use of the NG in concert with these devices.
   - Long-Term: As CDOT personnel become more comfortable with the methods, NG testing could be phased out over a number of years.

1.2 Current CDOT Practice for QA of MSE Wall and Bridge Approach Earthwork

MSE wall design and construction for CDOT projects are governed primarily by Section 504 of the Bridge Project Special Provisions (last revised in 2002). Subsection 504(b) of the *Material* section states that backfill materials for MSE walls shall conform to the Structure Backfill (Class 1) requirements defined by Section 703.08 of the 2005 Standard Specifications for Road and Bridge Construction. Section 703.08 defines Structure Backfill (Class 1) as follows:

703.08 Structure Backfill Material

(a) Class 1 structure backfill shall meet the following gradation requirements

Sieve Size	Mass Percent Passing Square Mesh Sieves
50 mm (2 inch)	100
4.75 mm (No. 4)	30-100
300 µm (No. 50)	10-60
75 µm (No. 200)	5-20

In addition this material shall have a liquid limit of 35 or less and a plasticity index of 6 or less when determined in conformity with AASHTO T 89 and T 90 respectively.

In certain cases in mountainous areas, CDOT allows a maximum particle size of 4 inches to be used in structural backfill for MSE walls (Trevor Wang, personal communication, 2006).

The *Construction Requirements* section, specifically subsection 504(e) *Excavation and Backfill*, requires that MSE wall backfill material be compacted to a density of at least 95% of the maximum modified Proctor density determined in accordance with AASHTO T-180. There are no specified moisture requirements for Class 1 structural backfill. Subsection 504(e) also states that compacted lifts shall not exceed 8 inches in thickness.

Bridge approach earthwork design and construction is governed by Section 206 of CDOT's 2005 Standard Specifications for Road and Bridge Construction. Like MSE wall material, bridge approach material must conform to the Class 1 structure backfill requirements of 703.08, and must be compacted to a density of at least 95% of the maximum modified Proctor density determined via AASHTO T-180. There are no moisture specifications. Test frequency for both MSE wall and bridge approach earthwork is addressed in the CDOT 2007 Field Materials Manual, Item 203, *Compaction*. Under the *Field Tests* subsection of Item 203, it states that a minimum of one density test be taken for each 2,000 cubic yards of embankment material placed.

1.3 Current Approach to CDOT QA: Nuclear Gauge

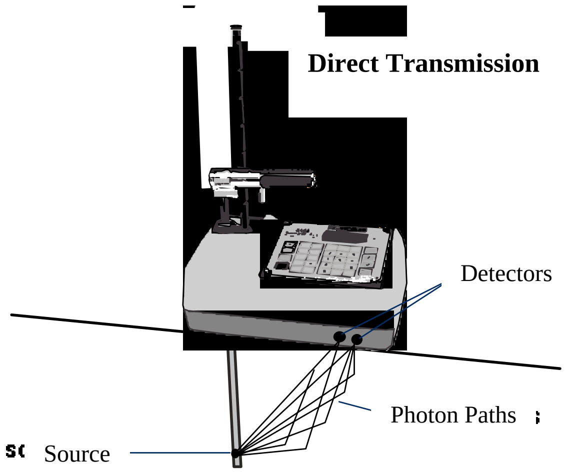
While four approved field testing methods are listed in Item 203 of the CDOT 2007 Field Materials Manual for confirming the compacted density of embankment material, CP 80 (nuclear method) is the approach that is used almost exclusively on CDOT projects. This method uses the NG to determine the compacted dry density and moisture content of the emplaced material. A recent survey of other State Departments of Transportation showed that all but one state, Minnesota, are using the NG almost exclusively for this purpose. This device, however, has limitations when used in this capacity. Before discussing the limitations that research and experience have identified, it would be beneficial to briefly review the principle of operation for this device.

The NG measures density using a gamma ray emitter and receiver (see Figure 1-1). The emitter, or source rod, sends out photons that interact with the soil mass; the receiver counts the number of photons returned. The theory behind this process is that a high density soil will contain a higher number of electrons for the photons to interact with and thus a lower number of photons will be returned to the receiver. Therefore there is an inverse relationship between the density of the soil and the returned photon count rate. Most commercial devices are capable of operating in two modes: direct transmission and backscatter. The direct transmission mode involves inserting the gamma ray source rod into the soil to depths of 2 to 12 inches in 1 or 2 inch intervals. This method requires up to 4 minutes of gauge processing time, but returns accurate density results (+/- 0.11 pcf) (Troxler, 2006). Independent testing by Ayers and Bowen (1988), using more dated devices, showed that gauge accuracy in direct transmission mode was around (+/- 0.80 pcf). The backscatter method uses the same device, but is a non-intrusive, surface method. Instead of inserting the rod into the material (typically asphalt or concrete), the gauge is placed on the ground surface and gamma rays are emitted and collected. The backscatter method is slightly less accurate (+/- 0.25 pcf) and only characterizes density to a depth of approximately 4 inches (Troxler, 2006).

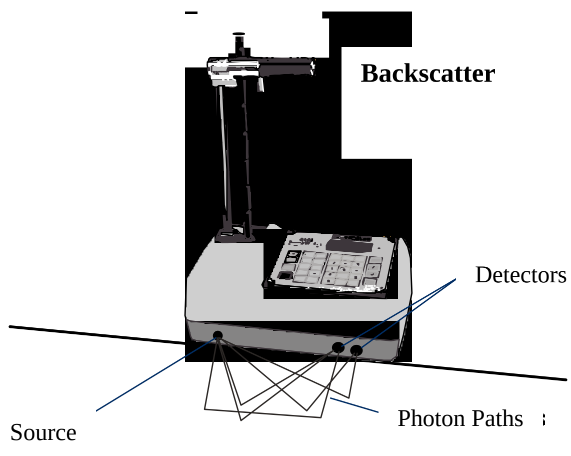
Most NGs also measure moisture content using a neutron emission device (see Figure 1) in a manner similar to the backscatter method discussed above. To estimate moisture content, the device uses a different radioactive source that emits fast neutrons. These fast neutrons interact with hydrogen atoms, which slow their velocities, and the slowed neutrons are then returned to a receiver in the device only capable of counting these slowed neutrons. The device counts the number of slow neutrons received back, and then estimates the number of hydrogen atoms that the radiation interacted with. Since hydrogen atoms are present both in the minerals that make up the soil, as well as the water entrained in the soil, it is necessary to conduct calibration procedures to determine what ratio of the hydrogen atoms that the neutrons interacted with are attributed to the water content, as opposed to the mineral composition of the soil. Once calibrated, the device automatically calculates and returns the estimated moisture content that is accurate to ±2.1% (Troxler, 2006).

The depth to which the NG estimates moisture content ranges from about 4 to 8 inches. This depth is dependent on complex relationships with soil composition, degree of saturation, and other site conditions. Typically density and moisture content measurements are conducted simultaneously by the device. It should be noted, however, that in direct transmission mode with full insertion of the source rod, density is characterized to a depth of 12 inches, whereas moisture content may only be estimated for the top 4 inches of soil. Due to the way in which this device operates, there are certain inherent limitations that trouble QC inspectors with respect to the monitoring of compaction for MSE wall and bridge approach backfills.

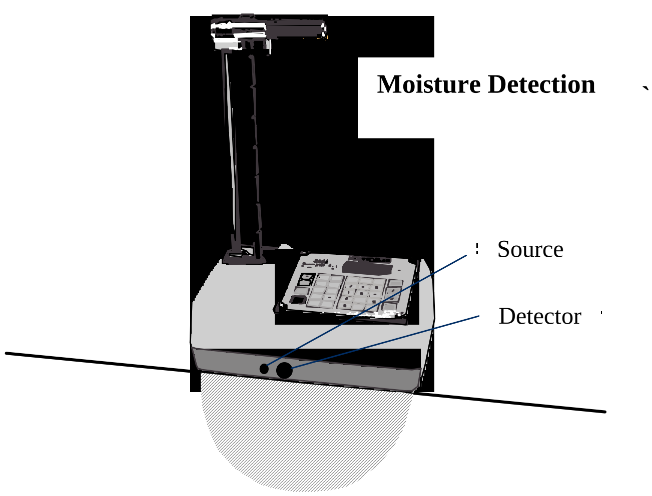
- **Limited depth of measurement –** the NG is capable of estimating material densities and moisture contents to depths of 4 to 8 inches. For density, this depth is controlled by the depth in which the source rod is inserted into the soil, and most devices only have an 8 inch source rod.



The gamma source is positioned at a specific depth within the test material by insertion into an access hole. Gamma rays are transmitted through the test material to detectors located within the gauge. The average density between the gamma source and the detectors is then determined. Errors resulting from surface roughness and chemical composition of the test material are greatly reduced, and gauge accuracy is improved. Direct transmission is used for testing lifts of soil, aggregate, asphalt, and concrete up to 12 inches in depth.



Backscatter is rapid and nondestructive. The gamma source and detectors remain inside the gauge which rests on the surface of the test material. Gamma rays enter the test material and those scattered through the material and reaching the detectors are counted. Backscatter is primarily used to determine density on layers of asphalt and concrete approximately 4 inches thick



The moisture measurement is nondestructive with the neutron source and detector located inside the gauge just above the surface of the test material. Fast neutrons enter the test material and are slowed after colliding with the hydrogen atoms present. The helium3 detector in the gauge counts the number of thermalized (slowed) neutrons, which relates directly to the amount of moisture in the sample.

Figure 1-1. Illustration of Nuclear Gauge Operation (from Troxler, 2006)

- **Burdensome handling and operating costs** - a NG requires stringent handling procedures and safeguards. NGs must be secured in locked cases when not in use. Individual users must also undergo DOT training on the transport of hazardous material every three years. In Colorado, NG licensing is required by the Colorado Department of Public Health and Environment, Hazardous Materials and Waste Management Division. Companies that operate NGs must license each facility from which they operate these devices. At the time of this writing, licensing fees in Colorado include an initial application fee of $1300 and an annual fee of $1850. These licenses only permit the use of NGs in areas under the jurisdiction of the State of Colorado, meaning that users need to obtain further licenses to operate in other states or on federal lands. NG operators who hold licenses in other states must pay an annual reciprocity fee to operate their gauges in Colorado (i.e., 75% of the annual fee = $1400). These costs are passed on to CDOT.

- **Inaccuracy while measuring density near walls –** walls present a unique challenge to estimating compacted density with a NG. When the NG emits photons, the photons typically interact with electrons in the soil and a certain number are then returned to the receiver. Nearby walls, however, tend to reflect a significant number of photons back to the receiver, and produces an artificially higher returned photon count rate, which the device interprets as a lower soil density. To the authors' knowledge, the detailed relationship between under-registration of density, distance from the wall, and soil type has not been investigated. However, Humboldt suggests taking density readings no closer than 5 feet from a wall. In MSE wall and bridge approach construction, this leaves a vital portion of the wall system left untested.

- **Coarse aggregate –** Class 1 structure backfill can contain aggregates as large as 2 inches, and even as large as 4 inches in certain cases. To create a void that allows the radioactive source rod to be inserted into the soil, a rod is driven into the ground and then removed leaving behind a cylindrical void in which the source rod can be placed. Should this drive rod, which is typically driven with a moderately sized hammer, encounter a large aggregate, it will undoubtedly displace the aggregate in some manner and create an additional void space. When the source rod is then placed in the soil, it is unable to obtain an intimate contact with the soil due to the void spaces created by the displacing of aggregate, resulting in inaccurate density measurements.

- **Density is not necessarily indicative of performance –** the objective of MSE wall and bridge approach earthwork compaction is to achieve suitable levels of shear strength and stiffness in the soil and to minimize compressibility. While density is somewhat proportional to soil strength and stiffness and inversely proportional to compressibility, density is only a surrogate for these engineering properties. A more direct measurement of shear strength (e.g. friction angle) and stiffness (e.g. resilient modulus) would enable the use of performance based QC/QA specifications. In this regard, density measurement and thus the nuclear density gauge are somewhat limited.

In summary, the NG is capable of estimating densities of uniform materials under favorable site conditions. With a required offset of 5 feet near a wall, use limitations due to particle size, and noted user costs and handling issues, the NG has limitations as a QA device for MSE wall and bridge approach earthwork. In addition, material density is not analogous to or indicative of soil stiffness or shear strength, both of which are integral engineering properties.

1.4 Summary of Report

This report contains five chapters. Chapter 2 presents a literature review of QA devices and best practices for QA of MSE wall, bridge approach and other related earthwork. Chapter 2 also summarizes the preliminary evaluation of many potential QA devices, and the recommendations for follow-up field testing. Chapter 3 presents the field testing overview and fundamental analysis of NG density and moisture testing, as well as LWD, DCP, and Clegg devices. Chapter 4 presents field assessment of CDOT's MSE and bridge approach earthwork QA procedures, and the analysis of target values for the various devices. Chapter 5 presents conclusions and recommendations for CDOT.

CHAPTER 2 LITERATURE REVIEW AND BEST PRACTICES

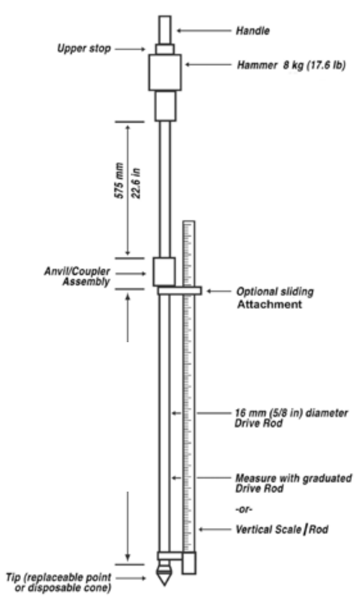
2.1 Literature Review of QA Devices

From the start, a broad approach was taken to gain a general understanding of what earthwork QA devices were available, how each device operates, and what level of performance would be expected when used. Table 2-A summarizes candidate devices that meet the requirements of this study. Capability overviews of each device, developed from an extensive literature review, are presented in Section 2.1. Best practices of these devices are presented in Section 2.2. A summary of Phase I findings and recommendations for Phase II field evaluations are presented in Section 2.3.

Table 2-A. Potential Devices and Soil Properties Measured

Device	Soil Property Measured
Dynamic Cone Penetrometer (DCP)	Shear Strength
Clegg Hammer	Shear Strength, Modulus
Soil Stiffness Gauge (aka GeoGauge)	Low Strain Modulus
Time Domain Reflectometry (TDR)	Density, Moisture
Light Weight Deflectometer (LWD)	Modulus
Dirt Seismic Property Analyzer (DSPA)	Low Strain Modulus

2.1.1 Dynamic Cone Penetrometer



Standard Specification:

ASTM D6951 (2003)

States with Specs and are using this Device:

IL, IN, MN

Price:

Manual $600- $1,000

Automated data acquisition $2,500

Fully automated $10,000

Figure 2-1. DCP Illustration (from MN/DOT spec 5-692.255 mod)

Figure 2-2. Manual, Automated Data Acquisition, and Fully Automated DCPs (right two photographs from Vertek, 2006)

The standard DCP is comprised of a 16mm diameter drive rod that can be as long as 1.2 m, an 8 kg drop (hammer) mass, and a 60º cone located at the tip of the drive rod with a diameter of 20 mm (see Figure 2-1). The DCP works by holding the device perpendicular to the soil surface and dropping the 8 kg mass from a pre-specified height of 575 mm on top of the anvil. The impact causes the probe cone to penetrate into the soil. The depth of penetration per hammer blow is recorded. A Dynamic Penetration Index (DPI) is developed in units of mm/blow and is recorded versus depth. This process is repeated at a test location and creates a profile of DPI values to depths of up to 1.2 m, allowing the DCP test to profile multiple layers at a test site. Based on these results, different subsurface strata and pockets of loose material can be identified. Many correlations have been developed to relate the average DPI over a defined depth (hereafter referred to as $\overline{\text{DPI}}$) to more universally accepted engineering parameters such as California Bearing Ratio (CBR), resilient modulus, and friction angle.

Of the devices researched for this report, the DCP has the most extensive and documented track record. However, little success has been achieved linking $\overline{\text{DPI}}$ to dry unit weight of a given material. Salgado and Yoon (2003) found that a general relationship does exist, as the dry unit weight increases, the $\overline{\text{DPI}}$ decreases. Unfortunately, they were not able to develop a numerical relationship, a setback cited in much of the literature. Instead of adopting density or stiffness based DCP correlations, Mn/DOT has adopted specifications that require a maximum allowable

$\overline{\text{DPI}}$ for a variety of soil materials. This eliminates the uncertainties involved with trying to correlate DPI measurements to density and stiffness values and allows for easy “in-the-field” determination of whether or not a material meets the specification requirements. Multiple reports stated that DCP testing is sensitive to soil moisture conditions, and Salgado and Yoon (2003) showed that the $\overline{\text{DPI}}$ for a given material slightly increases with increasing moisture content. Mn/DOT has taken this into account and adjusted $\overline{\text{DPI}}$ target values based on moisture content. Mn/DOT’s procedures for using the DCP are summarized in Section 2.2.

Salgado and Yoon (2003) noted that soils containing gravel can cause the DCP to produce unrealistic DPI measurements, likely the cause of the DCP coming in contact with a large aggregate. This causes some concern with using the DCP as a QA device on MSE wall and bridge approach earthwork, especially when particle sizes approaching 4 inches are permitted.

2.1.2 Clegg Impact Tester

Standard Specification:

ASTM D 5874 (2002)

States Using Device:

None

States Reviewing Device:

IN, NM

Price:

$3,000 - $3,500

Figure 2-3. Clegg Hammer Models (Lafayette, 2006)

The Clegg Impact Soil Tester, commonly referred to as the Clegg Hammer, is a device that uses a standard weight, or hammer, that is dropped from a height of 18 inches. The primary components of this device are a flat-ended cylindrical hammer and a guide tube, through which the mass is dropped. In the U.S., the Clegg Hammer is produced and marketed by Lafayette

Instruments in Lafayette, Indiana. Hammer masses of 0.5 kg, 2.25 kg, 4.5 kg, 10 kg and 20 kg are available for different applications of the device (see Figure 2-3). As shown in Table 2-B, only the 4.5 kg, 10 kg, and 20 kg Clegg Hammers are intended for use with structural earthwork compaction.

Table 2-B. Available Clegg Hammer Packages (from Lafayette, 2006)

Hammer Mass (kg)	Hammer Diameter (mm)	Recommended Applications
0.5	50	Soft turf, sand, golf greens
2.25	50	Natural or synthetic turf (athletic fields)
4.5	50	Pre-constructed soils, trench reinstatement, bell holes, foundations
10	130	Flexible pavement, aggregate road beds, trench reinstatement, bell holes, foundations
20	130	

The measurement element of the Clegg Hammer is an accelerometer attached to the top of the hammer that measures deceleration upon impact with the soil. The device electronically reports a Clegg Impact Value (CIV), which is the peak deceleration value measured in tens of gravities. ASTM D5874 requires that four consecutive drops be carried out at the same location, and that the value returned from the peak drop be reported as the measured CIV. In the past, the 4.5 kg Clegg Hammer has been calibrated for a site using Proctor compacted samples at optimal density and moisture content. By using the Clegg Hammer on these samples, a target CIV is established and can be used in the field to ensure proper compaction. This approach is not possible with larger 10 or 20 kg models. Assuming that elastic plate bearing theory applies to the Clegg Hammer operation, this device is capable of investigating compaction to a depth of 1 to 1.5 hammer diameters, or up to 26 cm (10 inches) for the 10 and 20 kg hammers (Mooney and Miller 2008). Optional accessories to this device also allow for integrated GPS locating, moisture testing, and data collection and storage.

Unlike the DCP, the Clegg Hammer has not been extensively studied and reported on in literature. The results of the survey of State Departments of Transportation show that no states are currently using the device. New Mexico and Indiana are actively studying the device but have not conducted enough tests to draw any conclusions. A report prepared by the Gas Technology Institute (GTI, 2004) investigated the Clegg Hammer and other QC/QA devices to evaluate its usefulness in monitoring backfill compaction in utility trenches. This report developed no correlations for this device, but did make a few important qualitative observations. GTI (2004) found that in weak materials the CIV is more a measure of shear strength, whereas in stronger materials the CIV is more a measure of stiffness. They also found that CIV readings increase with increasing moisture content to a maximum value (wetter than optimum) and then decrease at higher moisture contents. Soils tested included silty clay, sand, and gravel base. GTI (2004) found that the Clegg Hammer produced results that were independent of factors related to the operator. Adams (2004) criticized the Clegg Hammer's inability to track changes in density and moisture content, a criticism common among devices that measure soil strength and stiffness. Erchul and Meade (1994) developed acceptance criteria for the Clegg Hammer for use in QA of utility trench backfill compaction in Chesterfield County, Virginia. Using the Clegg Hammer on four different soil materials common to the area, Erchul and Meade (1994) noted that for soils returning similar CIVs, the depth to which the impact hammer penetrated the soil was often quite different. In order to track the depth of penetration for each test, Erchul and Meade (1994) added a penetration scale, in units of tenths of an inch, to the handle of the Clegg Hammer. Following the fourth drop of the hammer, the depth of penetration was recorded. In comparing the CIV and penetration data with density and moisture data collected using the nuclear density gauge, Erchul and Meade developed graphical acceptance criteria for soils encountered in Chesterfield County (see Figure 2-4).

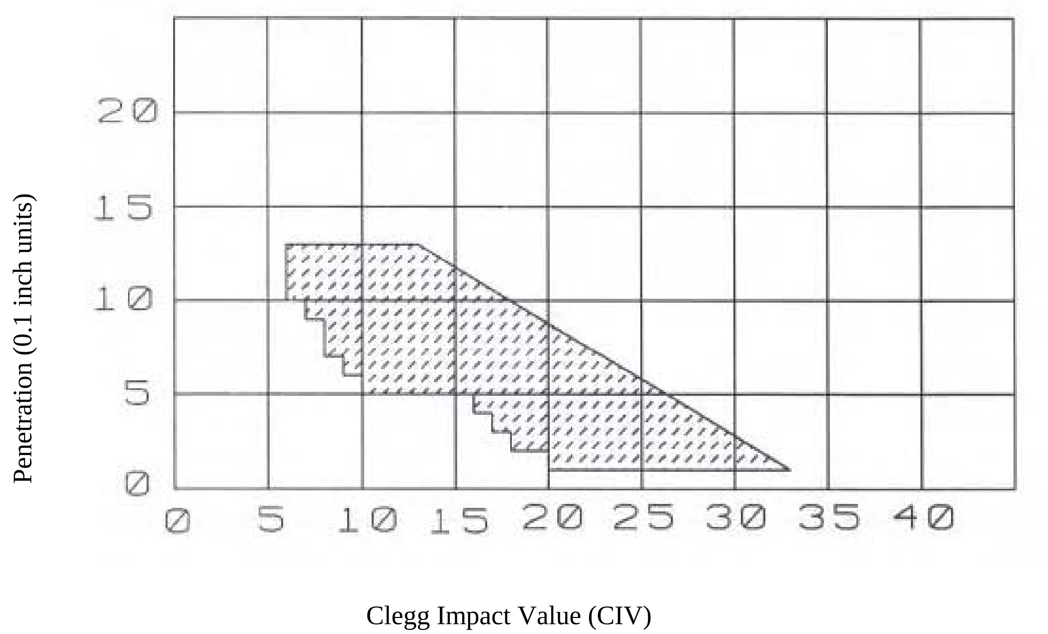


Figure 2-4. Acceptance Criteria for Clegg Hammer (from Erchul and Meade, 1994)

2.1.3 GeoGauge (or Soil Stiffness Gauge)

<u>Standard Specification:</u>

ASTM D6758 (2002)

<u>States using device:</u>

None

<u>States Reviewing Device:</u>
FL, ID, IL, MD, MI, MT, NM, NC, WY

<u>States that Previously Studied Device:</u>
GA, KS, LA, MD, MN, MS, NH, NJ, NY, OK, OR, PA, TX, WA, WV

<u>Price:</u>

$5,000 - $5,500

Figure 2-5. GeoGauge Photo (from Humboldt, 2006)

The Soil Stiffness Gauge (SSG), also referred to as the Humboldt Stiffness Gauge (HSG) or GeoGauge^{TM}, measures in-place stiffness of soil, a mechanical property (Lenke et al., 2001), which can translated to modulus using elastic half-space theory. One of the most concerning findings discovered while researching this device is the number of states that have studied the GeoGauge and concluded that it failed to satisfactorily meet their requirements. A survey conducted of State Departments of Transportation showed that 15 states were unhappy with the results produced by the GeoGauge. At the time of this writing, no states have adopted the GeoGauge for use in any specification. A complaint nearly universally voiced by states who have studied the device is that the GeoGauge results are extremely inconsistent and no correlation to other forms of testing could be developed. The cause of these inconsistencies, which was not speculated on in the survey, could be rooted in either operator-induced error or inherent shortcomings of the device. However, the fact that so many states have reported negative findings for the GeoGauge causes much doubt in its prospect as a QA device. While investigating the device for New Mexico State Highway and Transportation Department, Lenke et al. (2001) noted that while the GeoGauge showed promise as an earthwork QA device, developing specifications using lab-derived target values was difficult. Boundary effects apparent while using the GeoGauge on common lab-sized compaction molds produced unusable data. Lenke et al. (2001) suggested that a shift from lab-derived target values to target values established during construction of control strips would be required to incorporate this device into QA specifications. Lenke et al. (2001) reported that the GeoGauge was sensitive to moisture content, and than the optimum moisture content based on stiffness readings did not coincide with the optimum moisture content obtained from lab density tests. As a result of these complications, New Mexico decided against implementing the GeoGauge as a QA device.

2.1.4 Time Domain Reflectometry

Standard Specification:
ASTM D6780 (2005)

States Using Device:
None

States Reviewing Device:
FL, MN, NJ, NC

States that Previously Studied Device:
IN

Price:
$3,000 - $5,000

Figure 2-6. TDR Device Photo (from Durham, 2006)

Time domain reflectometry is a method that uses electromagnetic wave propagation to estimate moisture content and dry density. Many commercial TDR devices exist on the market, and each is slightly different in terms of components and geometry. Overall, all devices consist of multiple metal probes connected to a single voltage source. Typical rod lengths are about 8 inches and common rod configurations involve 3 rods in a triangular configuration. The rod spacing is only 2.5 inches; therefore, this presents a problem for large particle sizes. The probes are inserted into the soil and voltage pulses are sent down, the time required for the pulse to travel from the voltage source to the end of the probe and back is measured. This travel time is principally dictated by the moisture content of the soil. Therefore the TDR device is capable of directly measuring the volumetric water content and the apparent dielectric constant for the soil. Procedures do exist, however, to calculate the dry density and gravimetric water content. Depth of investigation for this device is dictated by the length of the rods with typical rod lengths around 8 inches for commercially available devices. Siddiqui and Drnevich (1995) established a method that uses field calibration of site soils to determine gravimetric water content and dry density. After extensive laboratory and field testing showed that this method returned sufficiently accurate results, it was accepted by ASTM and designated ASTM D6780 in 2003. The authors cautioned that the TDR method of determining density and water content is only applicable to soils where less than 30% of the sample by weight has particle sizes the exceeding 4.75 mm (No.

4) sieve and the maximum particle size passes the 19 mm (3/4 in) sieve. The method cannot be used on fat clays or soils with high moisture contents due to the fact that no significant secondary reflections of the voltage pulses can be observed, which does not allow for an apparent dielectric constant to be calculated.

2.1.5 Light Weight Deflectometer (LWD)

Standard Specification:

ASTM E2583 - 07

States Using Device:

MN

States Evaluating Device:

IN, KS, MN (other uses), MO, MS

Price:

$8,000 - $15,000

Figure 2-7. Dynatest 3031 LWD (left) and Zorn ZFG 2000 LWD (right)

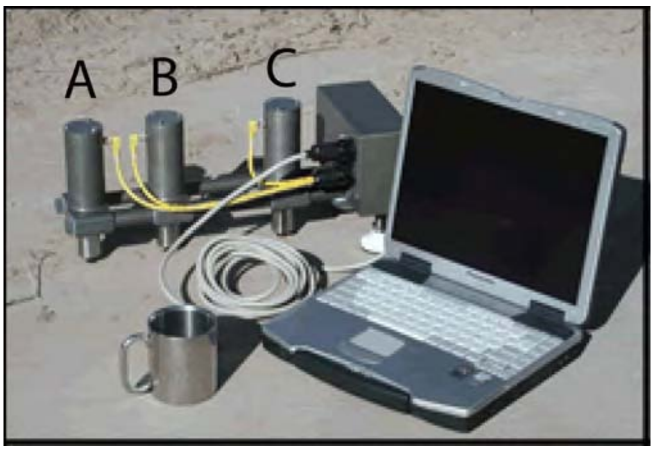
The Light Weight Deflectometer (LWD) was developed to measure the in-situ elastic modulus of soils. A typical LWD weighs approximately 15 to 25 kg, can be operated by 1 person, and requires approximately 2 minutes per test. There are currently three LWDs commercially available in the U.S.: the Dynatest 3031 LWD, Zorn ZFG 2000 and the Carl Bro PRIMA 100. The concept of each device is simple; an impulse load is imparted via drop weight onto a load plate and into the soil. That load is either measured (in the case of the Dynatest) or assumed constant based upon the drop height, loading weight, and damper (in the case of the Zorn). The Dynatest imparts an impulse load with an approximate magnitude and duration of 9.8 kN and 20 ms (Hoffmann et al. 2003). According to German specifications, the Zorn LWD imparts an impulse load not to exceed 7.07 kN with duration of 18 ± 2 ms (Adam et al. 2004). The impulse magnitude and duration are meant to approximately replicate a traffic and/or construction load (Fleming et al. 2007).

During the LWD test, when the drop weight impacts the housing and plate, the response of the plate or ground surface is measured with a geophone (Dynatest) or an accelerometer (Zorn). The output from the geophone or accelerometer is used to determine the displacement time history of the ground surface via numerical integration. The maximum applied force and maximum displacement are determined and used to estimate soil modulus (E_{LWD}). Per ASTM E2583, three seating drops of the hammer are followed by three measurement drops. E_{LWD} reflects the average values from the measurement drops.

In current practice, modulus is typically estimated via rigid or flexible plate on elastic half-space theory (Adam et al. 2004, Hoffmann et al. 2003). Both LWDs offer multiple loading plate diameters (typically 100 mm, 200 mm, and 300 mm). The most common plate diameter is 300 mm. However, the plate diameter can be reduced to ensure the deflection values can be accurately measured; deflections too small or too large may introduce unwanted errors (Lin et al. 2006). Many research studies have been conducted on the LWD. Good agreement has been shown between moduli measured by several LWDs and modulus values from other in-situ tests (Fleming et al. 2007, Siekmeier et al. 2000, Abu-Farsakh et al. 2004). The LWD has been recommend as a good in-situ testing device by multiple researchers (White et al. 2004, Kremer & Dai 2004, Abu-Farsakh et al. 2004). The LWD also provides rapid assessment of layer stiffness and identification of localized weak spots (Lin et al. 2006). The measurement depth of the Dynatest LWD with a 200 mm diameter loading plate has been estimated to be 270-280 mm deep (Abu-Farsakh et al. 2004). Mooney and Miller (2008) and Fleming et al. (2007) estimate the measurement depth to be 1-1.5 times the plate diameter, thus extending to 450 mm deep for the 300 mm diameter LWD plate and 300 mm for the 200 mm LWD plate.

No known studies have been performed on the use of the LWD during MSE wall construction; however, multiple studies have been successfully conducted on LWD use on coarse base aggregate (Von Quintas et al. 2005, Kremer & Dai 2004, White et al. 2004, Lin et al. 2006). While LWD modulus estimates on coarse gravel produced higher standard deviations than other soils, they were deemed to be within an acceptable level and likely due to uneven contact surface and heterogeneity of the soil (Lin et al. 2006). There are no protruding or penetrating aspects of the LWD, therefore, there should be no interference created by large particle sizes. There have been no investigations into the influence of a rigid wall on LWD results. Using the common 2:1 stress distribution with depth rule of thumb, the LWD should be able to operate within 200 mm of a wall without influencing the results. Moisture content can influence modulus reading. As moisture content increases, modulus generally decreases. Mn/DOT incorporated this relationship into their LWD pilot specification as displayed in Table 2-E.

2.1.6 Surface Seismic Testing



Standard Specification:
TxDOT (pending)

States using device:
None

States Reviewing Device:
FL, TX

Price:
$30,000, includes Toughbook laptop computer

Figure 2-8. Dirt Seismic Pavement Analyzer (DSPA) from Geomedia Research & Development Services

Surface waves are stress waves traveling along the free surface of a material, similar to waves propagating on the surface of water. In soils, the velocity of these surface waves is mainly dependent on the skeleton stiffness of the particles (modulus), the porosity or dry density, and the degree of water saturation. Specifically, surface wave velocity increases as soil modulus increases. Since soil compaction involves the increase in dry density and soil modulus as moisture remains constant (in theory), surface wave velocity measurement is a good technique to assess compaction. Surface seismic methods rely on one or several receivers (typically accelerometers or geophones) arranged at known distances from each other on the surface and an impulse or vibrating source that generates seismic waves at one or several locations. Figure 2-8 illustrates the DSPA (Geomedia Research & Development) that uses two accelerometers and one impulse source. Conversely, one can use an instrumented hammer and one other receiver (see

Figure 2-9). The surface wave velocities are calculated from the relative time difference between signals recorded at different locations along the surface. In general the measured velocities become more accurate and repeatable with more distances between receivers as shown in Figure 2-9.

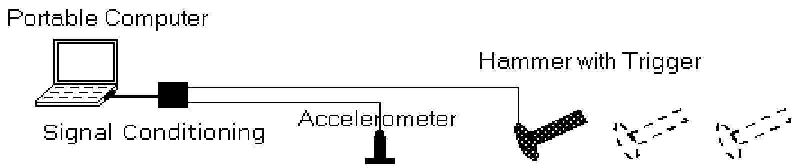


Figure 2-9. Seismic Surface Wave Testing Using One Receiver and One Source

The main disadvantage with seismic techniques is that the nature of wave propagation, required equipment, and data processing can all become relatively complex compared to standard test methods. For this reason there is no ASTM standard on surface wave testing and only a limited number of commercially available devices which are also constrained to certain applications.

2.2 Best Practices for QA of MSEW, Bridge Approach, or Related Earthwork

The vast majority of states are still almost exclusively using the NG for QA of earthwork compaction. Minnesota is the only state not using the nuclear density gauge, and has taken a very progressive stance on implementing cutting edge devices for earthwork QA. Two other states, Indiana and Illinois, have adopted new technologies but are still relying heavily on the NG for QA testing. Many states are reviewing or have reviewed a variety of devices, but have yet to incorporate these devices into standard practice. Table 2-C summarizes what devices are being or have been studied by different states, and which of these devices have been adopted into practice.

Table 2-C. Summary of Non-nuclear Devices Being Used or Studied by Various States

	Devices					
	LWD	**DCP**	**GeoGauge**	**Clegg Hammer**	**Time Domain Reflectometry**	**Dirt Seismic Property Analyzer**
In use; spec. developed	MN[1]	IL[1], IN[1], MN[1]				
Currently Under Review	IN, KS, MN[2], MO, MS	FL, IA, IL[2], IN[2], LA	FL, ID, IL, MD, MI, MT, NC, NM WY	IN, NM	FL, MN, NJ, NC	FL, TX
Previously Studied and Not Adopted into Practice			GA, KS, LA, MD, MN, MS, NH, NJ, NY, OK, OR, PA, TX, WA, WV		IN	

[1]Only pilot specifications at the time of this writing
[2]Device still being reviewed for use in other applications

2.2.1 Mn/DOT DCP Specification

The DCP is currently being used in earthwork QA at Mn/DOT. In their 2002 Grading and Base Manual (modified in 2006), Mn/DOT published specification 5-692.255, which describes the use of the DCP for granular materials. In this specification, testing procedures are described for using the DCP on base course, granular subgrades, and aggregate filter materials. Maximum allowable penetration indices at specified moisture contents have been developed by Mn/DOT through their experience with this device. Table 2-D shows the Mn/DOT DCP specification. Grading

Number is calculated using Equation 1 in Section 2.2.2. Percent Dry of Optimum is the percentage less than optimum of the field moisture content as compared to laboratory optimum moisture content. Maximum allowable seat is the depth of penetration of the first 2 blows. Maximum allowable $\overline{\mathrm{DPI}}$ is the average penetration of blows 3, 4, and 5. Approximate test layer is the depth below the surface to which the DCP tip evaluates.

Table 2-D. Mn/DOT DCP Penetration Requirements

Grading Number	Gravimetric Moisture	Maximum Allowable Seat	Maximum Allowable $\overline{\mathrm{DPI}}$	Approximate Test Layer
GN	%	mm	mm/blow	mm
3.1-3.5	< 5.0	40	10	100-150
	5.0-8.0	40	12	
	> 8.0	40	16	
3.6-4.0	< 5.0	40	10	100-150
	5.0-8.0	45	15	
	> 8.0	55	19	
4.1-4.5	< 5.0	50	13	100-150
	5.0-8.0	60	17	
	> 8.0	70	21	
4.6-5.0	< 5.0	65	15	125-175
	5.0-8.0	75	19	
	> 8.0	85	23	
5.1-5.5	< 5.0	85	17	150-300
	5.0-8.0	95	21	
	> 8.0	105	25	
5.6-6.0	< 5.0	105	19	175-300
	5.0-8.0	115	24	
	> 8.0	125	28	

2.2.2 Mn/DOT LWD Pilot Specification

Mn/DOT has a pilot specification (Pilot Specification 2105) for the LWD, permitting its use as an earthwork QA device. The specification includes specific requirements for QA of bridge approach earthwork. Under this specification, Mn/DOT requires contractors to construct a control strip to develop compaction target values for the LWD. LWD readings are recorded at three locations on designated proof layers in the control strip between passes of compaction equipment. Once LWD values appear to reach a maximum value, the values at the three locations are recorded and control strip construction moves on to the next layer. Multiple proof layers are required to fully characterize the influence moisture content has on LWD modulus values. The compaction target values are then determined by averaging the maximum LWD modulus readings at the three designated locations on each proof layer and then averaging the average maximum LWD modulus values from all proof layers constructed under the same moisture conditions.

Mn/DOT recognizes construction of control strips can be cumbersome and impractical for certain earthwork projects. After 2008, they are hoping to eliminate control strips and establish target values for QA specification. As opposed to LWD modulus target values, Mn/DOT plans to adopt LWD deflection target values for field QA (John Siekmeier, personal conversation, 2008). This is a more conservative approach as the LWD specifically measures deflection. The LWD makes some assumptions to determine modulus including contact stress and Boussinesq linear half-space. Mn/DOT will utilize deflection values from Table 2-E, or a slightly modified version, for determination of target values. The selected target value would need to be verified. This methodology will be determined after verification of these numbers during the 2008 construction season. Note that Table 2-E is based on a load of 6.28 kN and a drop height of approximately 0.52 m because Mn/DOT employs an LWD with a shorter drop arm in the interest of portability.

Mn/DOT uses Grading Number (GN), similar to Fineness Modulus in concrete mix design, to classify granular materials. GN uses percent passing values from a traditional sieve analysis and is calculated using Equation 1. A GN of 3.1-3.5 approximately corresponds to a GW material while a GN of 5.6-6.0 corresponds to an SM or SC.

$$GN = \frac{1" + \frac{3}{4}" + \frac{3}{8}" + \#4 + \#10 + \#40 + \#200}{100} \quad (1)$$

Where: 1” = % passing 1” sieve, #4 = % passing #4 sieve, etc.

As Mn/DOT does not permit use of the NG, either the Burner Method or Speedy Method are specified to determine moisture content. The Burner Method uses traditional oven drying by comparing the weight of a wet sample to the weight of a dry sample. The Speedy Method uses a cast aluminum pressure bottle to measure pressure generated inside the bottle when combining the sample soil with calcium carbide. Moisture content is read directly off a gauge on the bottle.

Table 2-E. Mn/DOT Predicted Zorn LWD Deflection and Modulus Values
(from Mn/DOT Pilot Spec 2105)[1]

Grading Number	Moisture Content	Target Zorn LWD Modulus	Target Zorn LWD Deflection
GN	**%**	**MPa**	**mm**
3.1-3.5	5-7	80	0.38
	7-9	67	0.45
	9-11	50	0.60
3.6-4.0	5-7	80	0.38
	7-9	53	0.56
	9-11	42	0.71
4.1-4.5	5-7	62	0.49
	7-9	47	0.64
	9-11	38	0.79
4.6-5.0	5-7	53	0.56
	7-9	42	0.71
	9-11	35	0.86
5.1-5.5	5-7	47	0.64
	7-9	38	0.79
	9-11	32	0.94
5.6-6.0	5-7	42	0.71
	7-9	33	0.90
	9-11	29	1.05

[1]200mm load plate, 10kg hammer, 0.52m drop height, F=6.28kN

There is precedent beyond Mn/DOT for establishing target LWD values. Austria, as well as other European countries, uses the LWD for QA of earthwork and pavement subgrade, subbase, and base courses. Table 2-F shows the Austrian pavement specification from the International Society of Mechanics and Geotechnical Engineering (ISSMGE, 2005). The Zorn LWD Modulus values correlate well to Mn/DOT Pilot Spec 2105. Interestingly, Austria does not factor in moisture content into their modulus criteria. One possible explanation is that LWD modulus in coarse grained materials is less affected by moisture than in cohesive materials.

Table 2-F. ISSMGE Zorn LWD Modulus Required Values (ISSMGE, 2005)[1]

Level	Zorn LWD Modulus (MPa)
1-m Below Subgrade	18 (cohesive); 24 (cohesionless)
Top of Subgrade	30 (cohesive); 38 (cohesionless)
Top of Subbase	58 (rounded); 68 (angular)
Top of Base	70 (rounded); 82 (angular)

[1]300mm load plate, 10kg hammer, 1m drop height, F=7.07kN

2.3 Phase I Findings and Phase II Field Evaluation Recommendations

The published technical literature on the DCP sheds favorable light on the ability of this device to perform well as a QC/QA device. Its simplicity of design and operation, robust construction, and portability make it attractive for use in assessing compaction conditions of MSE walls and bridge approach earthwork. The success that Mn/DOT has had using maximum DPI readings for inspecting the quality of earthwork compaction suggests that, regardless of how well DPI readings correlate to other engineering properties, the DCP is quite capable of identifying areas of inadequate compaction. The depth to which the DCP can assess soil conditions is also beneficial for use in MSE wall and bridge approach earthwork, where several feet of compacted fill is normal. One concern, however, in using the DCP with select backfill material, such as CDOT Class 1 structural backfill, is the effect large aggregates have on DPI measurements. Table 2-G lists the strengths and limitations of the DCP as reported in the literature.

Table 2-G. DCP Strengths and Limitations

Strengths	Limitations
Simple design, robust construction, good portability	Manual operation may require 2 personnel (one to operate, one to take readings)
Capable of assessing soil conditions to a depth of 1.2 m	Sensitive to moisture conditions (though mostly in cohesive soils)
Well studied and documented track record, strong correlation with CBR	DPI measurements do not correlate well with dry density readings
Shallow testing can be done quickly, 1 to 5 min/location	Large aggregate may cause erroneous test results
Successfully being used (in MN)	Deeper testing in dense material can take up to 10 to 15 min/location

The Clegg Hammer appears to be a very capable device for use in QA of MSE wall and bridge approach earthwork. Good correlations have been developed for Clegg CIV, albeit for the 4.5 kg hammer. The manufacturer cautions that even though all Clegg Hammer models report CIVs, these values are dependent on hammer weight and geometry, thus two different weight hammers will report different CIVs for the same material. Therefore, more correlation equations will need to be developed in order to accommodate the 10 kg and 20 kg hammers. These heavier hammers seem better suited for MSE wall and bridge approach earthwork due to the depths to which they are able to measure. Table 2-H lists the strengths and limitations of the Clegg Hammer.

Table 2-H. Clegg Hammer Strengths and Limitations

Strengths	Limitations
Simple operation, portable design, integrated data acquisition	Sensitive to moisture conditions
Nondestructive, non-intrusive	Possibility of boundary effects when calibrating device using Proctor molds
Developed correlations with CBR values	Weak correlation to density measurements
Quick testing, < 1 min/location	Different weight hammers report different CIV values
Optional accessories allow integrated GPS positioning and moisture content testing via integrated moisture probe	

After reviewing the literature and canvassing other State Departments of Transportation, the viability of the GeoGauge as a QA device was deemed questionable. It appears that the complications of inconsistent testing results stem from the difficulty in obtaining a proper foot/soil contact. Table 2-I identifies some strengths and limitations of this device. Further consideration of this device is deemed unnecessary.

Table 2-I. GeoGauge Strengths and Limitations

Strengths	**Limitations**
User-friendly operation, portable design	Extremely sensitive to seating conditions
Capable of calculating soil stiffness from direct measurements of force and displacement	Questionable correlations due to inconsistencies in testing data
Quick testing, 1-2 min/location (after proper seating established)	No correlation to density measurements
Non-destructive and non-intrusive	Sensitive to moisture conditions
	Unfavorable findings by multiple State DOT's

Time Domain Refrectometry cannot be used on soils containing particle sizes larger than ¾ inch or on material containing more than 30% (by weight) of particles coarser than the No. 4 sieve make it unacceptable for use as a QA device for MSE wall and bridge approach earthwork with Class 1 backfill. Further analysis of this device is thus deemed unnecessary.

The LWD seems a very capable device for MSE wall and bridge approach earthwork QA. The device would likely be best utilized when supported by less frequent testing with the DCP or nuclear density gage to provide a comprehensive understanding of the subsurface stratum and soil properties. Strengths and weaknesses of the LWD are summarized in Table 2-J.

Table 2-J. LWD Strengths and Limitations

Strengths	Limitations
Easy to transport and simple to operate	Modulus is moisture-dependent and the LWD does not measure moisture
Quick test – 1 min/test	Sensitivity to changes in compaction not well developed
Multiple size loading plates enable measurement over a range of modulus values and different depths	
Seemingly no influence caused by large aggregate	
No interference with MSE wall reinforcement	
Measurement depth (up to 0.5 m) allows assessment of multiple layers	

At this time, surface seismic wave testing appears too complex for QA inspection. The approach is fundamentally sound, but further analysis is deemed unnecessary due to complexity and fragility of existing systems. This approach should be considered as simpler systems are developed. Table 2-K summarizes the strengths and limitations.

Table 2-K. Surface Wave Testing Strengths and Limitations

Strengths	Limitations
Measures a fundamental soil property (small strain stiffness modulus) which can also be measured in the laboratory	Complexity and accuracy is dependent on the layer profile
Can resolve different layers and hence measure a material property of each layer that is independent of the complete layer profile and surface condition	Can be time consuming and can require complex data processing to resolve different layers
Samples a large volume of the material	Currently no ASTM procedure
Sensitive to changes in compaction	Measurements can be affected by the surrounding geometry such as an MSE wall
Is not influenced by large aggregate materials	Fragile equipment components

In summary, the DCP, Clegg Impact Hammer, and LWD were selected for field evaluation.

CHAPTER 3: OVERVIEW OF FIELD TESTING AND EVALUATION OF DEVICE UNCERTAINTY

3.1 Test Sites and Procedures

Field testing with the DCP, LWD, Clegg Hammer, and NG was performed at two construction sites (see Figs. 3-1 and 3-2):

(1) <u>SH40 – I70 Hogback Park and Ride MSE Wall, Golden</u>.

(2) <u>SH58 – I70 Multi-MSE Wall and Bridge Approach Intersection, Golden</u>.

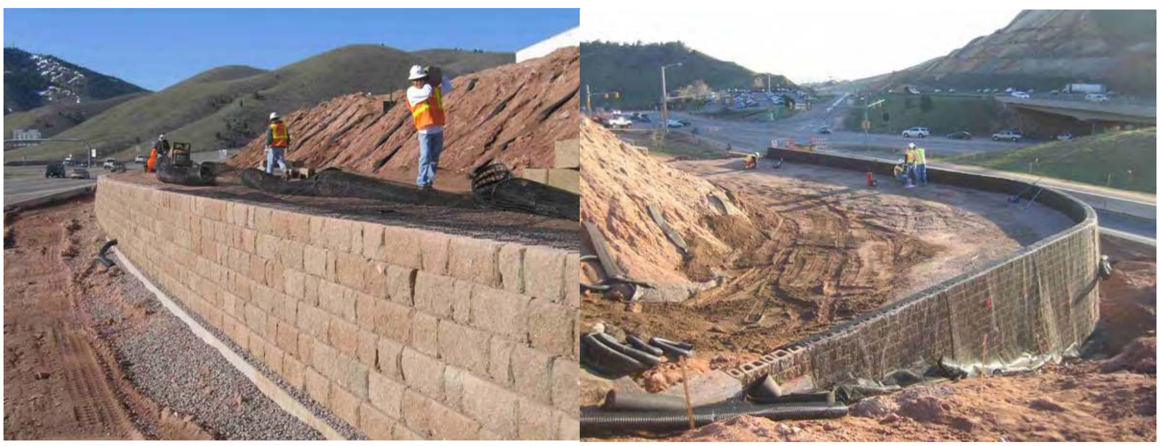


Figure 3-1. SH40 – I70 Hogback MSE Wall

Figure 3-2. SH58 – I70 MSE Wall (Parallel to SH58)

Field testing with the DCP, LWD, and Clegg Hammer was performed at numerous discrete locations within active construction sections at each site (see Figs. 3-3 and 3-4). At most test locations, NG testing was also performed by the QA representative at each site. This allowed direct comparison of DCP, LWD, and Clegg data to moisture-density data. Figure 3-5 illustrates a typical single-position testing configuration.

Figure 3-3. DCP and Clegg Impact Hammer Operation

Figure 3-4. Seating the LWD Plate and Administering a NG Test

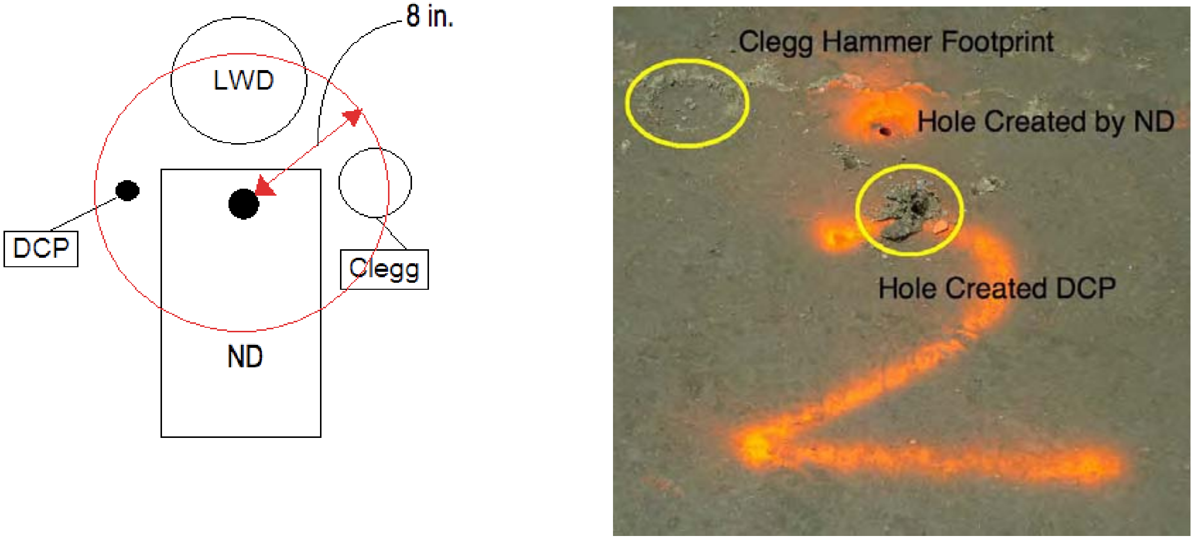


Figure 3-5. Typical Layout Adopted for Single Location Testing

At each site, test beds were identified as single lift areas where multiple spot tests could be performed, typically within 1-2 hours. Test beds for MSE wall were generally 10m (30ft) wide, the width of the backfill, and 30m (100ft) to 60m (200ft) long. Test beds for bridge approaches were generally 10m (30ft) wide and 7m (20ft) long. Tables 3-A and 3-B summarize the modified Proctor compaction results for the 5 soils encountered and the 30 test beds, respectively.

Table 3-A. Summary of Modified Proctor Compaction Results

Soil #	Max Dry Density kg/m^3 (lb/ft^3)		Optimum Moisture (%)
1	2171	135.5	4.6
2	2249	140.4	4.6
3	2263	141.3	5.9
4	2171	135.5	7.9
5	2204	137.6	7.4

Table 3-B. Summary of 30 Test Beds

Test Bed	Location	Size m x m (ft x ft)	Soil #	Test Date	Number of Test Locations DCP	LWD	Clegg	NG
1	SH-40 Hogback MSE	61x8 (200x25)	1	3/22/07	24	0	24	24
3	SH-40 Hogback MSE	30x8 (100x25)	1	3/26/07	12	12	18	18
4	SH-40 Hogback MSE	30x8 (100x25)	1	3/28/07	20	20	20	20
5	SH-40 Hogback MSE	50x8 (160x25)	2	4/5/07	21	21	21	21
6	SH-40 Hogback MSE	5x8 (15x25)	2	4/18/07	0	5	0	5
7	Bridge App. SH-58	12x6 (40x20)	3	7/3/07	10	0	10	10
8	Bridge App. SH-58	12x6 (40x20)	3	7/5/07	5	0	0	0
9	Bridge App. SH-58	12x6 (40x20)	3	7/5/07	5	0	2	5
10	Bridge App. SH-58	12x6 (40x20)	3	7/6/07	5	0	0	5
11	MSE SH-58	17x6 (55x20)	4	7/9/07	11	0	11	11
12	Bridge App. SH-58	12x6 (40x20)	3	7/9/07	3	0	3	3
13	MSE I-70	15x4 (50x12)	4	7/9/07	6	0	5	6
14	MSE I-70	30x8 (100x25)	4	7/10/07	9	0	9	9
15	MSE SH-58	8x6 (25x20)	4	7/10/07	3	0	3	3
16	MSE SH-58	15x5 (48x16)	5	7/11/07	6	0	6	6
17	MSE SH-58	30x8 (100x25)	5	7/11/07	5	0	2	6
18	MSE SH-58	46x8 (150x25)	5	7/17/07	15	9	15	6
19	MSE SH-58	37x8 (120x25)	5	7/18/07	15	15	15	15
20	MSE SH-58	24x8 (80x25)	3	7/23/07	10	10	10	10
21	MSE SH-58	24x8 (80x25)	3	7/23/07	4	11	11	11
22	MSE SH-58	15x8 (50x25)	3	7/24/07	0	10	10	10
23	MSE SH-58	18x8 (60x25)	3	7/24/07	0	6	6	6
24	MSE SH-58	30x8 (100x25)	3	7/25/07	0	11	11	11
25	MSE SH-58	30x8 (100x25)	3	7/30/07	0	14	14	14
26	MSE SH-58	30x8 (100x25)	3	7/31/07	0	15	15	15
27	MSE SH-58	30x8 (100x25)	3	8/1/07	0	10	10	10
28	MSE SH-58	30x8 (100x25)	3	8/6/07	0	12	12	12
29	MSE SH-58	30x8 (100x25)	3	8/7/07	0	10	10	10
30	Bridge App. SH-58	12x3 (40x10)	3	7/17/08	15	0	15	0

Hogback MSE Wall Testing – Test beds 1-6 were performed at the SH40 – I70 Hogback MSE wall construction project. The Hogback MSE wall was approximately 100 m (300 ft) in length and included sections with wall heights varying from 3-12 m (10-40 ft) (see Figure 3-1). The modular MSE wall included 200mm (8 in) tall bricks and geogrid reinforcing placed every 600 mm (24 in) in elevation. Compaction QA requirements included 200 mm (8 in) lift thickness and achievement of 95% modified Proctor maximum dry density. Per CDOT requirements, there was no moisture specification when using Class 1 backfill.

SH58 – I70 Interchange Testing – Test beds 7-30 were conducted within four structures on the SH58 – I70 construction site. The earthwork structures included a westbound bridge approach to I70, the MSE wall parallel to SH58, the MSE wall parallel to I70 ramping into the flyover, and the bridge approach connecting the I70 parallel MSE wall to the flyover. The MSE wall parallel to SH58 was approximately 200 m (660 ft) long and ranged from 2 to 6 m (6 to 20 ft) in height. The modular MSE walls were constructed with 200 mm (8 in) tall pin locking blocks and included geogrid reinforcing placed between each lift. The length of the geogrid was varied from approximately 1.5 m to 5 m (5 to 16 ft) alternating with each 200 mm lift. Each lift included a coarse gravel placed against the MSE wall for drainage which extending approximately 0.1 - 0.3 meters (0 - 1ft) and Class 1 backfill. Compaction QA requirements included 200 mm (8 in) lift thickness and achievement of 95% modified Proctor maximum dry density. Per CDOT requirements, there was no moisture specification when using Class 1 backfill.

3.2 Uncertainty in Device Measurement

This section characterizes the precision uncertainty of each device, the local variability measured with each device, and the appropriate statistical representation of data based on these findings. For any experimental test, accuracy and precision of the test device are key characteristics that define uncertainty. Accuracy – defined as the difference between the true and measured values – is unknown for the LWD, Clegg Hammer, or DCP because the true value cannot be measured. The accuracy of the NG was quantified by Troxler to be ±1.0% for density (Troxler, 2006). The precision of a measured value reflects the degree of reproducibility or repeatability. Precision is often reported in terms of the standard deviation (σ) and the associated confidence intervals, i.e., $\pm$ 1σ implies 68% confidence, $\pm$ 2σ implies 95% confidence. This chapter presents results to quantify precision uncertainty.

The geotechnical properties of earthwork (e.g., density, moisture, modulus, strength) vary spatially. It is critically important to understand the local variability of these properties, particularly when comparing data from two devices that measure over different volumes of soil. Here we present the results of local variability studies for the NG, LWD, DCP, and Clegg Hammer. The chapter concludes with recommendations for statistical representation of data that is then used throughout the report.

3.2.1 Device Precision

To quantify the working precision of each device, repeatability testing was performed using two approaches. For both approaches, the device was placed at a given location and an initial test was performed (i.e., one minute reading from nuclear gage (NG), average of 3 LWD drops, peak of 4 Clegg Hammer drops). For the ‘in place’ approach, 5-10 repeated tests were performed without removing or re-placing the device. For the ‘re-place’ approach, the device was removed and re-placed between each test.

Nuclear Gauge - A direct transmission nuclear gage (NG) test was performed at a 200 mm (8 in) probe depth thirty two times in the same location. The first sixteen times were run as ‘in-place’ tests where the device was left in exactly the same position. The following sixteen tests were run as ‘re-place’ tests where the device was removed from the ground and replaced in the same location. The location for each re-place test was identified by the indentation the gage left in the soil. The data is presented in Figure 3-6. The standard deviation (σ) of dry density and moisture were 6.1 kg/m^3 (0.38 pcf) and 0.16% respectively. The corresponding COVs were 0.30% and 4.13%. For 95% confidence (2σ), the uncertainty was ±0.60% (density) and ±8.27% (moisture).

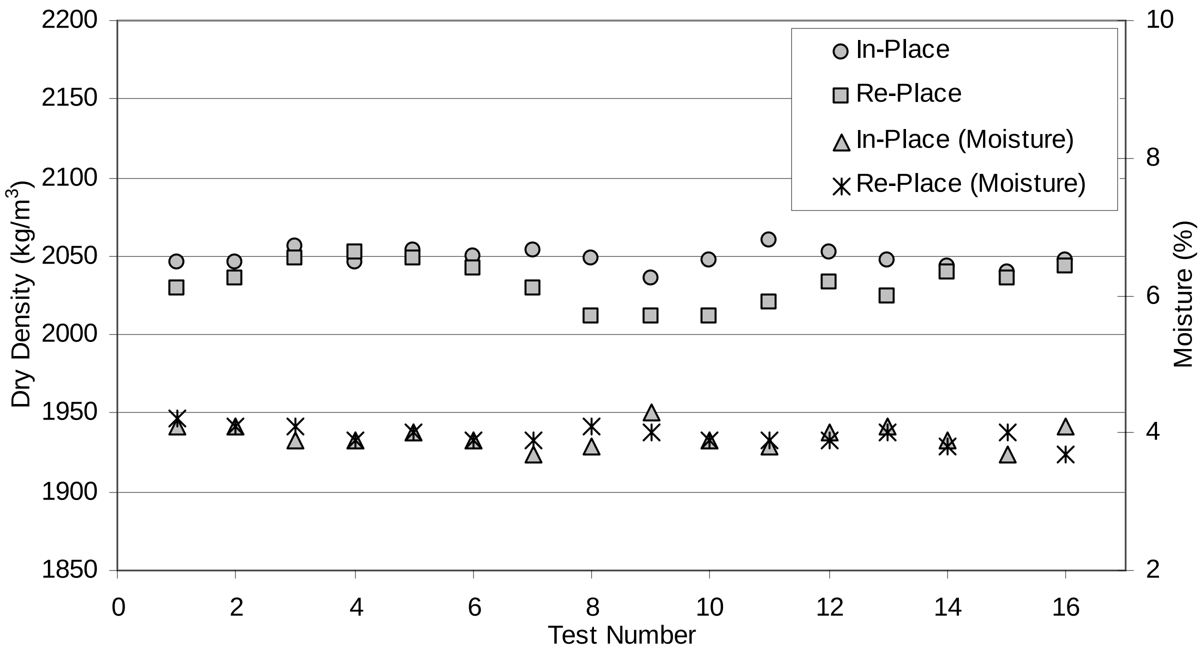


Figure 3-6. NG Repeatability Data in Density and Moisture

LWD – Shown in Figure 3-7 are plots of the data collected from five LWD repeatability tests conducted over a range of soil stiffness. These LWD repeatability tests were conducted over a range of soil stiffness in order to determine the effects of stiffness on precision uncertainty. After 3 seating drops, 3 measurement drops were performed to determine the first value of E_{LWD} (test 1). An additional 9 tests (3 drops per test) were performed. Series 4 and 5 in Figure 3-7 represent re-place tests were the LWD was picked up and re-placed in the same location (easy to do because the plate outline was visible in the soil). Testing was conducted on an MSE wall and stiffness was varied by starting close to the wall and moving perpendicularly away from it. In each case the data was de-trended in order to remove the effects of compaction where multi drop tests were conducted.

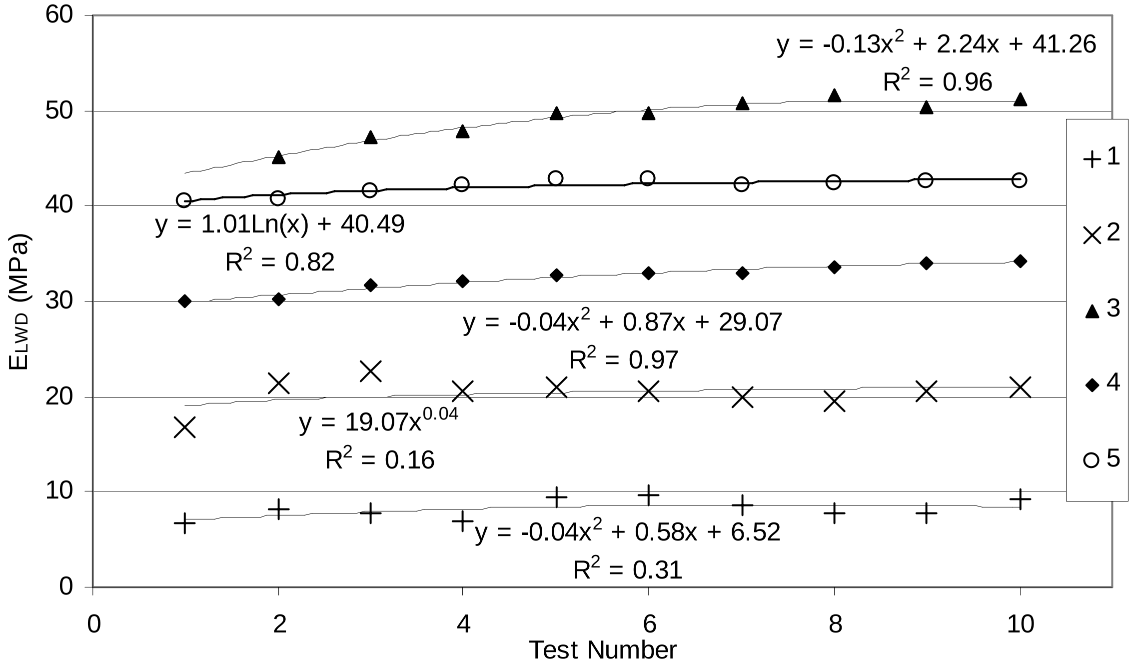


Figure 3-7. Repeatability Tests Conducted on Varying Soils
(increase in E suggests some compaction continued)

The LWD data from Figure 3-7 is presented in Table 3-C and reveals that precision uncertainty is strongly dependent on the value of E_{LWD}. Precision uncertainty improves from ±12.5% at a modulus of 20 MPa to less than ±2% at values greater than 30 MPa. As will be shown in Chapter 4, E_{LWD} > 30 MPa is desirable, and for this level, precision uncertainty is adequate.

Table 3-C. Data Summary of Figures 3-6

Series	Offset From MSE Wall	μ E_{LWD} (MPa)	2σ-Uncertainty (MPa)	2σ-Uncertainty (%)
1	0.3 m (0.98 ft)	8.21	1.73	21.02
2	0.8 m (2.62 ft)	20.40	2.54	12.43
3	3.0 m (9.84 ft)	49.25	0.85	1.72
4	NA	32.72	0.50	1.52
5	NA	42.19	0.70	1.66

Clegg Hammer - Figure 3-8 shows the results from three in-place repeatability tests conducted with the Clegg Hammer. Results from these tests were de-trended and presented in Table 3-D.

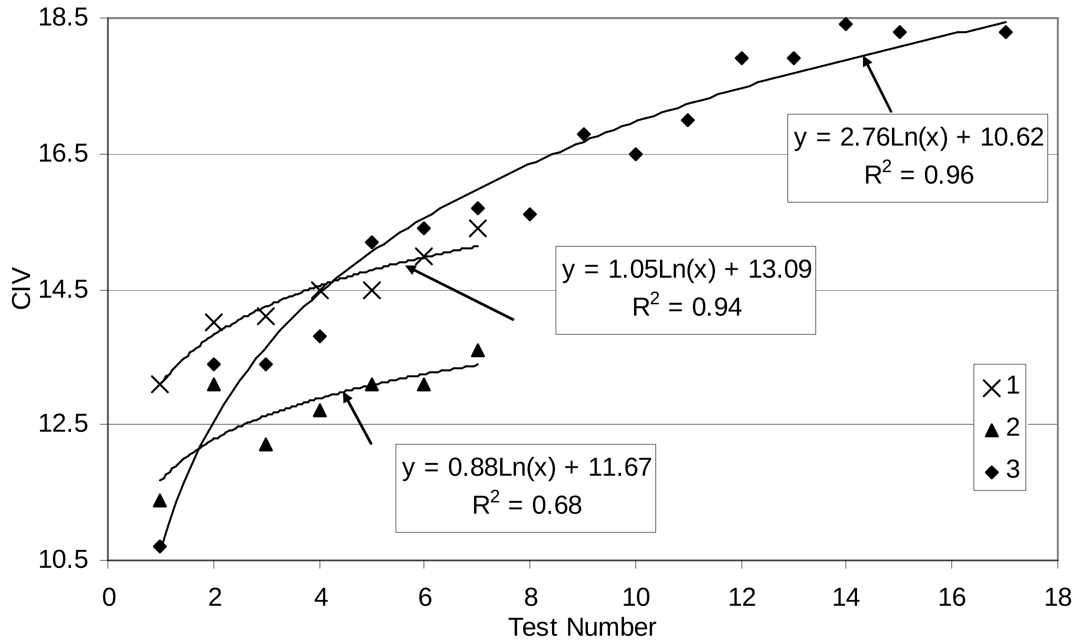


Figure 3-8. In-Place Test Results for Clegg Hammer

Table 3-D: Uncertainty in CIV

Series	µ (Clegg)	2σ Uncertainty (CIV)	2σ Uncertainty (%)
1	14.37	0.37	2.58
2	12.74	0.83	6.54
3	16.05	0.85	5.28
Average		0.68	4.80

Table 3-D indicates that the Clegg Hammer's precision uncertainty is less impacted by different soil stiffness with an average precision uncertainty of ±4.8%. It should be noted that series one brings the average precision uncertainty down with a values twice as low as the other two series. It is also important to note that the best precision achieved by the Clegg of 2.6% is 1% higher than the precision achieved by the LWD on soils with moduli over 30MPa.

Table 3-E summarizes the precision uncertainty of the tested devices. DCP precision can not be obtained due to the destructive nature of the test. The precision of the LWD changes dramatically with stiffness and therefore an equation has been listed instead of a value. The plot in Figure 3-9

shows the relationship between E_{LWD} and precision uncertainty. It can be seen that a low value in E_{LWD} leads to a high degree of imprecision. For example, an E_{LWD} value of 16 MPa corresponds to a precision of ±15%, while a value of 40 MPa corresponds to a precision of ±1.6%. The NG determined density has the lowest precision uncertainty however both the LWD and Clegg Hammer achieve less uncertainty than the NG determined moisture.

Table 3-E. 95% Confidence Interval for Precision of Each Device

Device	Precision[1] (%)
LWD	$P = 0.015(E_{LWD})^2 - 1.360(E_{LWD})+31.866$
Clegg	4.80
NG - Density	0.60
NG - Moisture	8.27
DCP	NA

[1] Precision reported as % based on the device's mean value and reflects 95% confidence

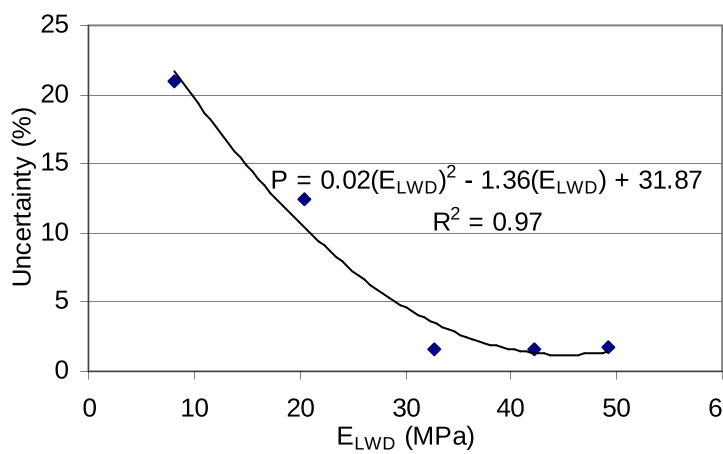


Figure 3-9. Precision Corresponding to Measurement Value

3.3 Assessment of Local Variability

Local variability in soil properties also contributes to uncertainty in single or multiple measurements. The characterization of local variability is particularly important when comparing values between devices. As shown in Figure 3-10, each device provides an average measurement over a volume of soil. The measurement volumes vary considerably across devices. The implication of local variability on earthwork QA will first be shown by considering NG density and moisture.

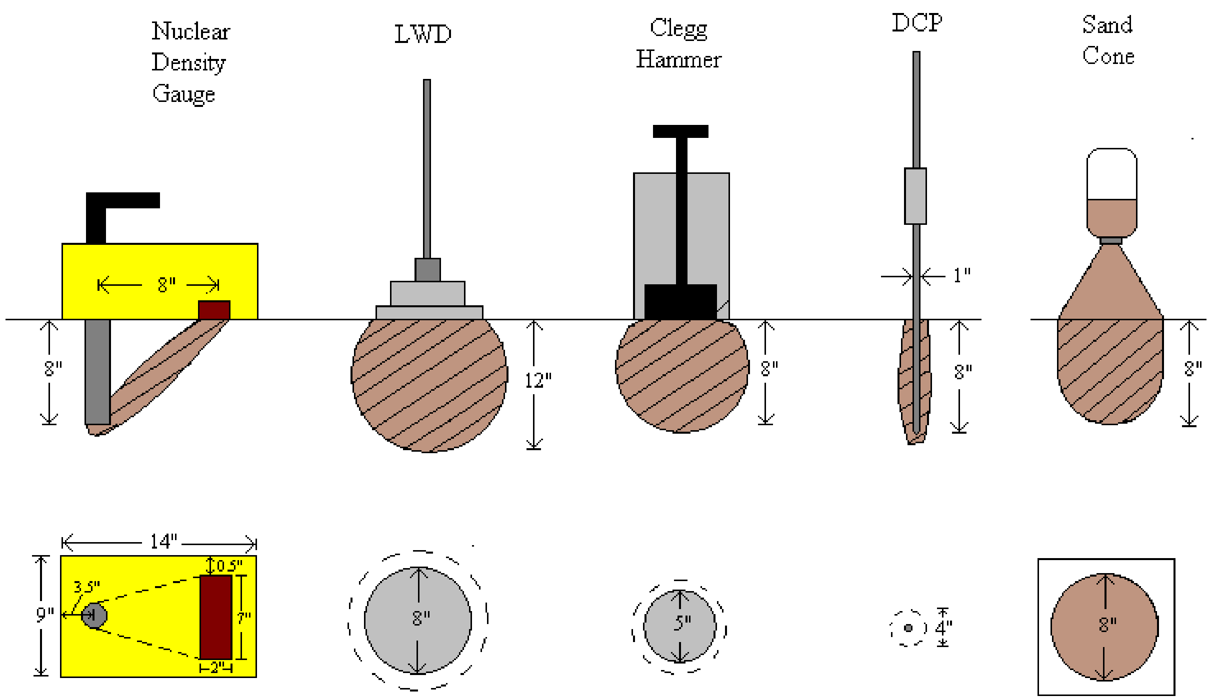


Figure 3-10. Comparison of QA Device Measurement Volumes

A four-position test (see Figure 3-11) was used to explore the local variability in density and moisture of soil with the NG. This configuration was selected because this measurement volume is roughly equivalent to that evaluated with the LWD test. One hole was driven as marked by the X in Figure 3-11. A one minute test was conducted beginning in position 1. The NG was rotated 90 degrees to position 2 and another one minute test was conducted. This procedure continued through position 4. The volume of soil tested with this configuration is difficult to precisely define, however, it is assumed to be a 3D clover shape originating at the probe depth of 8 inches and expanding upward and outward to the surface detector location (see Fig. 3-10).

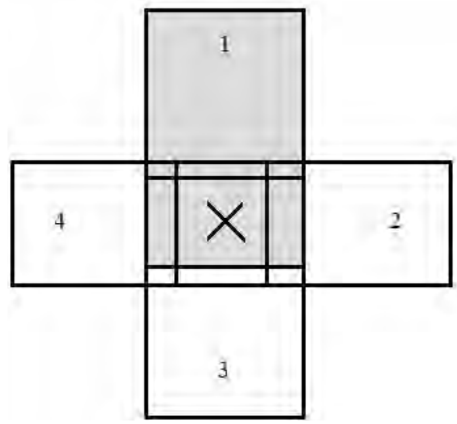


Figure 3-11. NG Testing Pattern for Local Variability

Twelve 4-position tests were conducted on a well compacted high quality aggregate base material. Figure 3-12 presents the values recorded at each position for each of the 12 points (tests). The circle for each point (test) reflects the average of the four positions. Figure 3-11 shows that at one-half of the points, the positional orientation of the device dictated whether the required density was met or not met. This is pertinent because current CDOT NG practice uses a single-position approach to testing. Based on these results, acceptance or failure is often determined by orientation (position) of the NG in CDOT practice.

Figure 3-13 presents histograms of the variability of the Figure 3-12 data. Specifically, the 4-position density or moisture range (R) normalized by the 4-position density or moisture average (× 100) yields a measure of variability (as a percentage of the density). The mean R/μ (× 100) in dry density was found to be 4%, implying that a single position measurement of dry density carries an uncertainty due to local variability of ± 4%. The corresponding uncertainty in percent compaction (%C) is ± 3-4%. This single position uncertainty in NG density is significant when one considers that the entire compaction process induces a %C change of ≈ 10 % (i.e., soil is typically placed at %C ≈ 85 % and acceptance is based on achieving %C ≥ 95%). Therefore, the compaction process induces a %C change of ≈ 10 % yet a single position NG density carries an uncertainty of ± 3-4%. The uncertainty in density can be decreased significantly by adopting the 4-position NG test. This is described further below and in Chapter 5.

The local variability within the 4-position area depicted in Figure 3-10 is also important when considering correlations between density and E_{LWD}, CIV, and DPI. For example, one LWD test produces an E_{LWD} value that represents an average value over a volume roughly similar to the four-position volume of the NG setup. To this end, the appropriate density to compare with E_{VIB} should be the average of the four positions. Considerable variability induced error can result from using a single-position NG density when comparing to E_{LWD}.

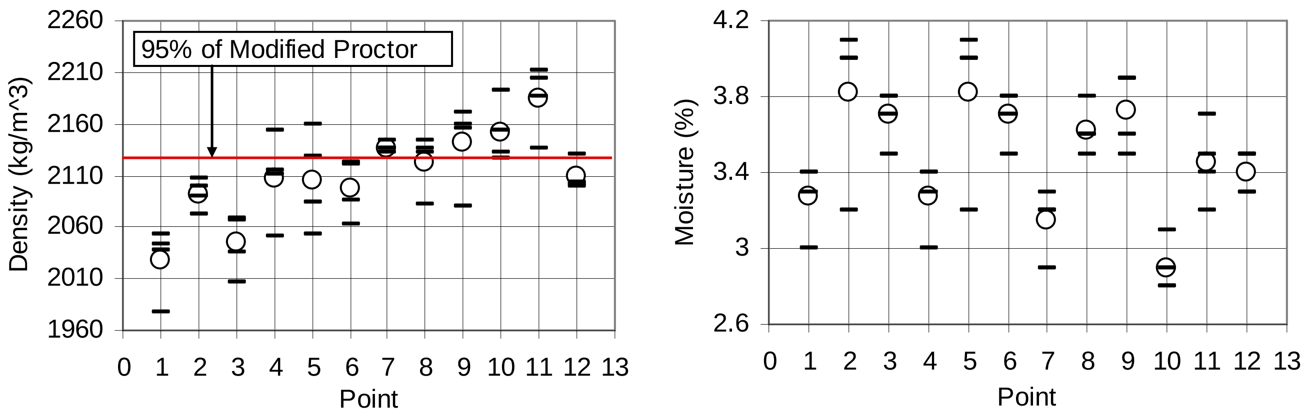


Figure 3-12: 4-Position and Average Density and Moisture at 12 Locations (Points)

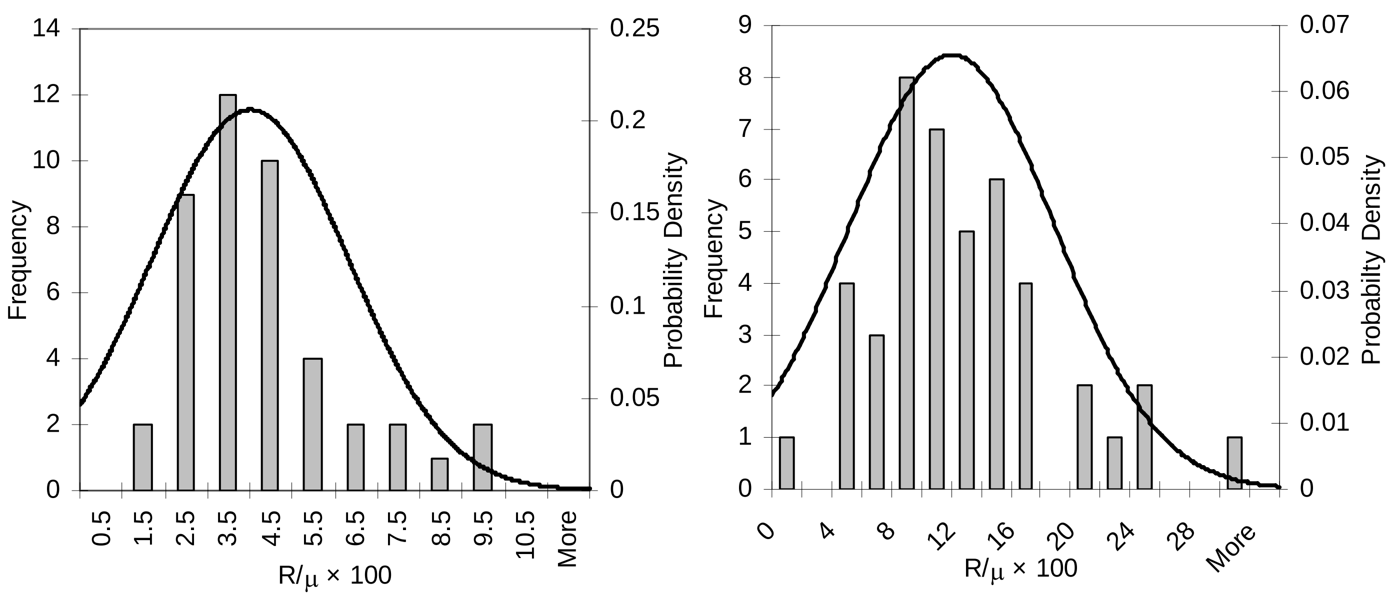


Figure 3-13: Variability in Density and Moisture Determined by 4-Position NG Test

The variability of soil properties should be considered when implementing QA procedures. The R/μ of LWD, Clegg Hammer, DCP and NG data collected on 4 test beds were assessed to characterize local variability. As shown in Figure 3-14, NG density varied by 5-10%. This is reasonably consistent with the results presented above. The R/μ for E_{LWD}, CIV and $\overline{DPI}$ were much greater, and are consistent with the findings of Nazarian et al. (2005) who found that modulus exhibits much higher variability than density. These results illustrate that all soils have sufficient variability in density, modulus and shear strength that should be captured in QA procedures. The importance of capturing variability is more critical for modulus and strength.

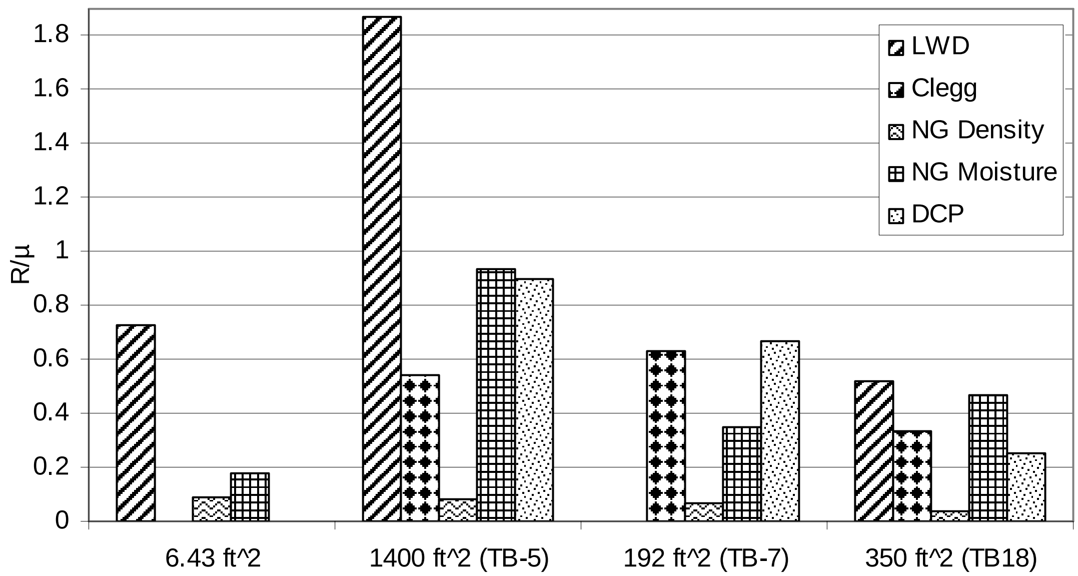


Figure 3-14: Precision Uncertainty for Each Device on Various Test Beds

3.4 Conclusions

In this chapter, the uncertainty in NG, DCP, LWD, and Clegg Hammer data were examined. Precision uncertainty was evaluated through repeatability testing. Uncertainty due to local variability was also examined over comparative measurement volumes and over larger test bed areas. The precision uncertainty of each device was considered acceptable for QA testing. Uncertainty is much greater when soil is poorly compacted (e.g., E_{LWD} is low). Evaluation of NG testing over an area equal to the measurement volume of the LWD test revealed significant variability. Specifically, by rotating the NG around the same source rod location, %C varied by ±

3-4%. This implies that within a small area (less than 2 sq. ft), NG dry density and thus acceptance or failure, is often dictated by orientation (position) of the NG. Because of this large relative uncertainty (± 3-4% compared to the approximate 10% change in %C typical as material is compacted) in single-position NG testing, we recommend CDOT modify the current NG approach to better account for local variability. This is addressed in Chapters 4 and 5.

An assessment of variability in NG, LWD, DCP, and Clegg Hammer data over typical test bed areas revealed 5-10% R/μ results for NG density and much greater R/μ values for E_{LWD}, DPI, and CIV. These results indicate that soil property variability must be statistically accounted for in QA. The methodology to account for variability is addressed in Chapters 4 and 5.

CHAPTER 4: FIELD TEST RESULTS

As described in Chapter 3, field testing with the NG, LWD, DCP, and Clegg Hammer was performed on 30 test beds and five soils. The objectives were two-fold: (1) assess the compaction of MSE wall and bridge approach earthwork with all four devices; and (2) evaluate the ability of each of the proposed devices to characterize compaction. This effort included evaluating the ability of each device to evaluate soil compaction close to the MSE and abutment walls, and to identify if the resulting E_{LWD}, CIV and $\overline{DPI}$ data suggests that target values from these devices can be used for QA criteria (as a supplement to or replacement for NG density).

4.1 Characterization of MSE Wall and Bridge Approach Compaction

The general ability of each device to assess compacted state was investigated on numerous MSE wall and bridge approach earthwork sections (test beds). Testing was performed in discrete locations beginning at the MSE or bridge approach wall face (i.e., X = 0 as shown in Figure 4-1) and extending out to the edges of Class 1 backfill. LWD, DCP, and Clegg Hammer testing were successfully performed within 0.3 m (1 ft) of the wall face. To fit into active construction projects with disrupting the contractor, most testing was performed on completed (compacted) sections.

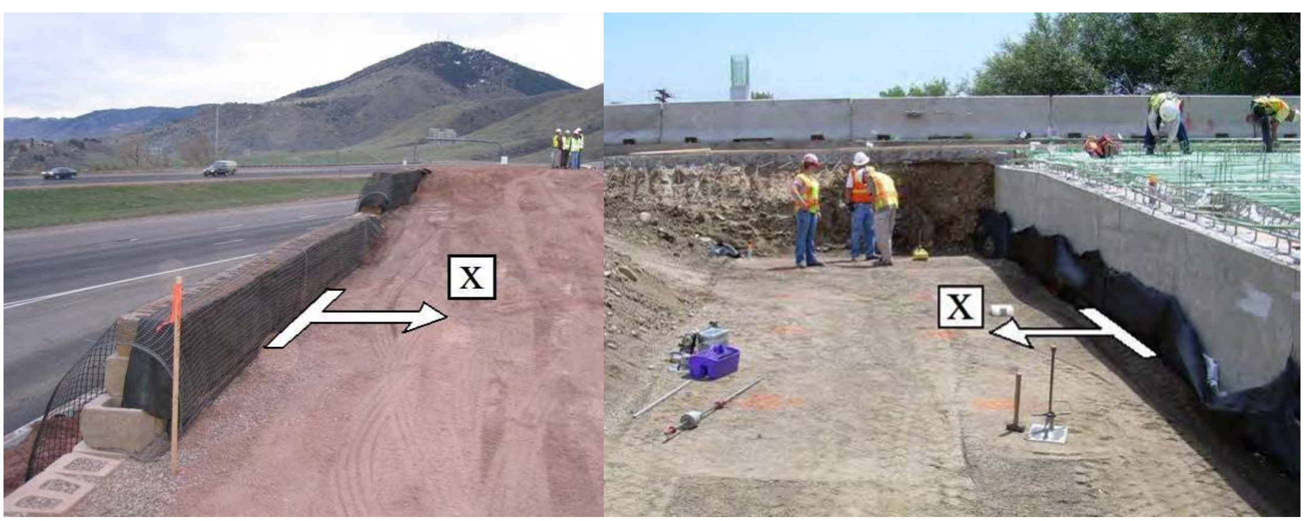


Figure 4-1. Offset X for MSE Wall and Bridge Approach Testing

Figure 4-2 presents dry density (NG), E_{LWD}, CIV and $\overline{DPI}$ data collected at 18 test locations on test bed 3 as a function of offset X. The X scale is presented in 0.3 m increments for visual interpretation in ft (0.3 m = 1 ft). Compared to the 95 %C acceptance shown in Figure 4-2, the area within approximately 1 m (3-4 ft) of the MSE wall face has not met the required 95 %C. All other density values indicate acceptable compaction has been achieved. The E_{LWD}, CIV and $\overline{DPI}$ data points each produce trends similar to NG density. For example, E_{LWD} is very low (5-8 MPa) within 1 m of the wall face but is relatively constant (20 MPa) for X > 1 m. It is important to note that these E_{LWD} values were determined using a 300 mm diameter load plate, whereas the target values in Section 4.2 were determined using a 200 mm diameter load plate.

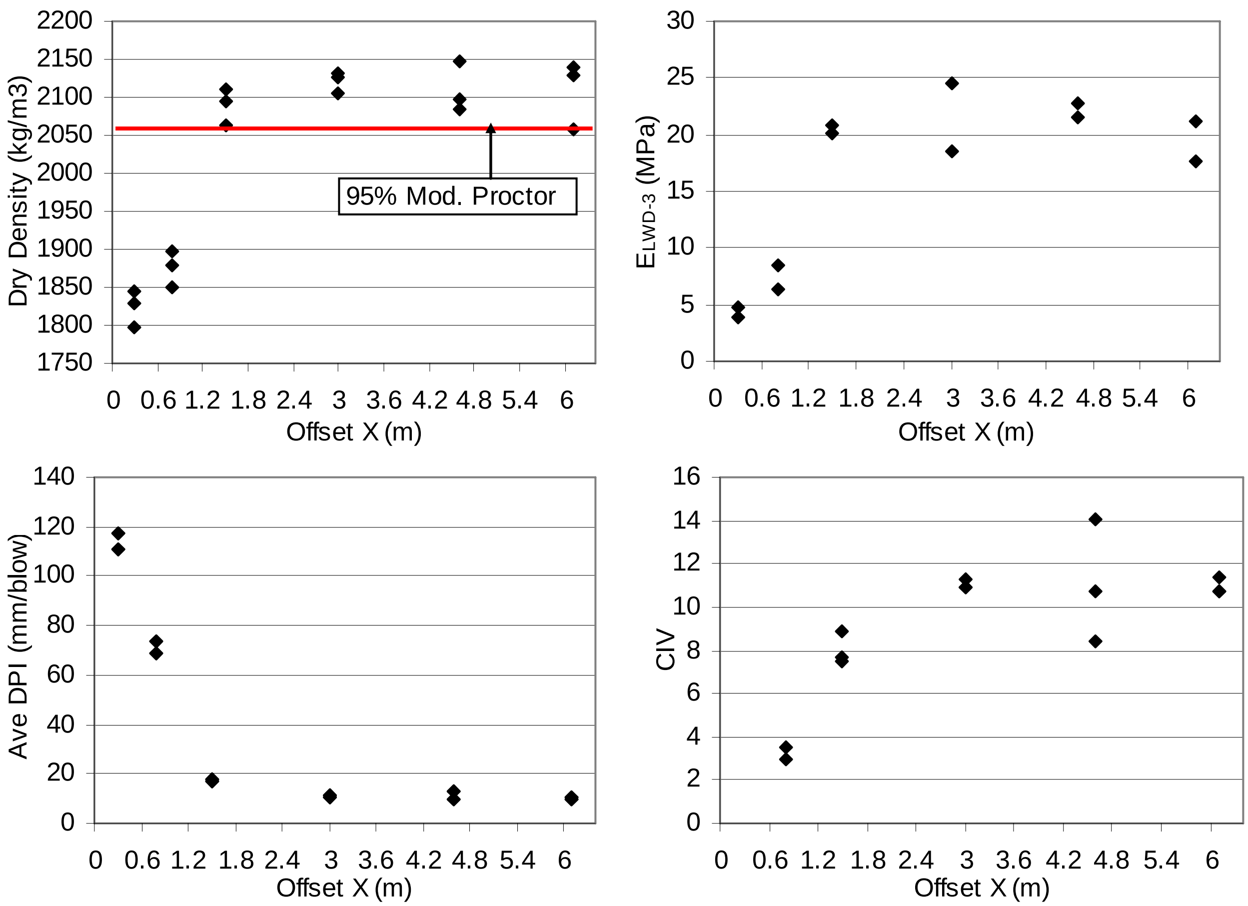


Figure 4-2. MSE Wall Soil Characteristics as Determined by all Devices
($\overline{DPI}$ is denoted as Ave DPI)

The $\overline{DPI}$ data is similarly clear. Here, a high $\overline{DPI}$ indicates softer material and low $\overline{DPI}$ indicates stiffer material. For X > 1 m, $\overline{DPI}$ is reasonably constant (10 mm/blow). The CIV data exhibits a similar trend with X; however, there is greater scatter particularly at X = 4.5 m.

The greater variability in E_{LWD}, CIV and $\overline{DPI}$ as compared to dry density is evident by the changes in values in Figure 4-2. From X = 0-6 m, dry density varies by 14% while E_{LWD}, CIV and $\overline{DPI}$ vary by 500%, 400% and 1200%, respectively. These data suggest that soil modulus and shear strength undergo much greater changes during compaction than does dry density.

The $\overline{DPI}$ data in Figure 4-2 stems from the DPI profile created from each DCP test. Figure 4-3 presents complete DPI records for 6 locations ranging from X = 0-6 m in test bed 3. The $\overline{DPI}$ presented in Figure 4-2 reflects the mean value over 200 mm (8 in), less seating drops 1 and 2, and reflects the lift thickness. This definition can be modified to any lift thickness or depth up to 1 m. Figure 4-2 illustrates how inadequate compaction is achieved even in lifts below the current one being assessed (e.g., depths of 0.8 m in Fig. 4-2). In contrast to the other devices and NG, the DCP enables a QA inspector to evaluate compaction of underlying lifts to a depth of 1.2 m.

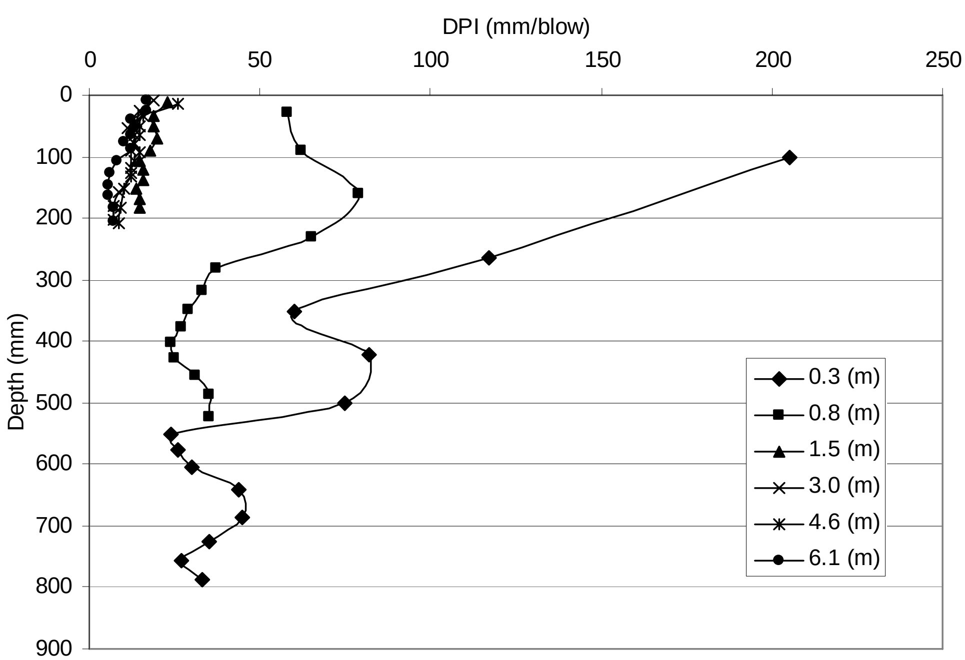


Figure 4-3. DPI Profiles at Varying Distances from Wall

One limitation observed during DCP testing stems from the DCP tip interaction with geogrid and geofabric. With geogrid and geofabric placed at 200 mm (8 in) spacings, the DCP tip must penetrate through the grid or fabric. Similar to the influence of large rocks that artificially alter the DPI readings, the geogrid and fabric provides some resistance during DCP testing. This was noticeable to the DCP operator during testing. This is a limitation of the DCP device for MSE wall and bridge approach earthwork where geogrid and geofabrics are always present.

Dry density, E_{LWD}, CIV and $\overline{DPI}$ data from test beds 1, 3 and 5 are presented together in Figure 4-5. While greater scatter in data exists, the trend is similar across all 3 MSE wall test beds. First, adequate compaction is not being achieved within 1.2 m (4 ft) of the wall face. Second, the LWD, DCP, and Clegg Hammer values mimic the dry density results, and therefore compaction.

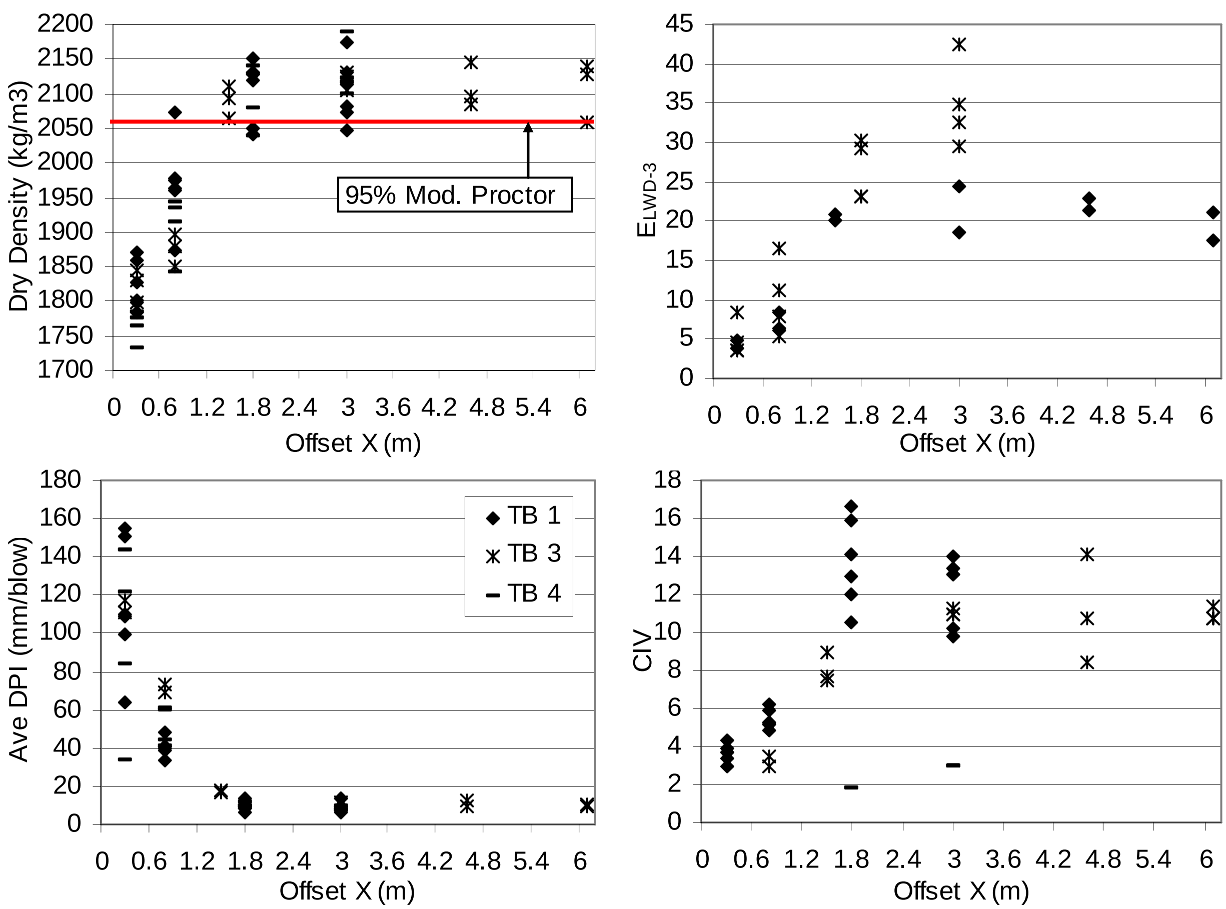


Figure 4-4. Data from Test Beds 1, 3, 4 ($\overline{DPI}$ is denoted as Ave DPI)

In test bed 30, a similar assessment of compaction at various X offsets from a bridge abutment was performed. The data collected from these 15 locations are presented in Figure 4-5 and reveal an increase in compaction for increasing X. Here, NG testing was not performed, and therefore it is difficult to definitively identify if there is an offset range where adequate compaction was not achieved. It is worth noting that the X < 1 m values of CIV and $\overline{\text{DPI}}$ indicate much better compaction than the similar MSE wall values shown in Figure 4-4.

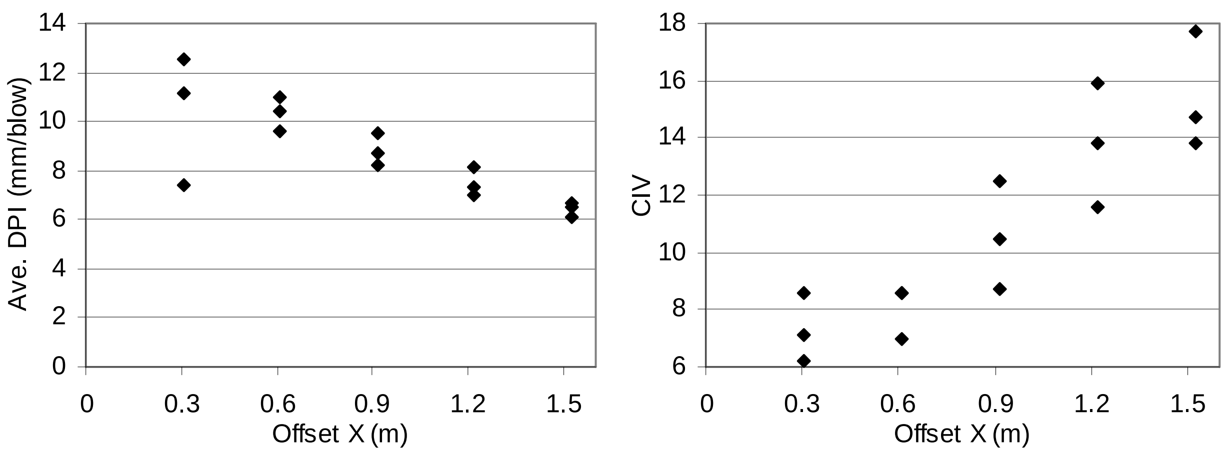


Figure 4-5. Test Bed 30 – CIV and DCP Data ($\overline{\text{DPI}}$ is denoted as Ave DPI)

The inadequate compaction within 1 m of MSE wall faces and the improved near bridge abutment compaction can be attributed to compaction methods. As shown in Figure 4-6, vibratory roller compactors are used for X > 1 m and plate compactors are used for X < 1 m of MSE walls. In contrast, vibratory roller compactors are used for earthwork against abutment walls. Per the results presented here, the vibratory plate compactors being used are not able to provide adequate compaction. The inadequate compaction within 1 m of the wall face largely goes unnoticed because of the near-wall limitation of the NG. These results suggest that CDOT should revisit compaction equipment requirements for near wall situations. Heavier plate compactors and/or vibratory trench compactors (see Fig. 4-6) may provide the required compactive effort and thus might be mandated.

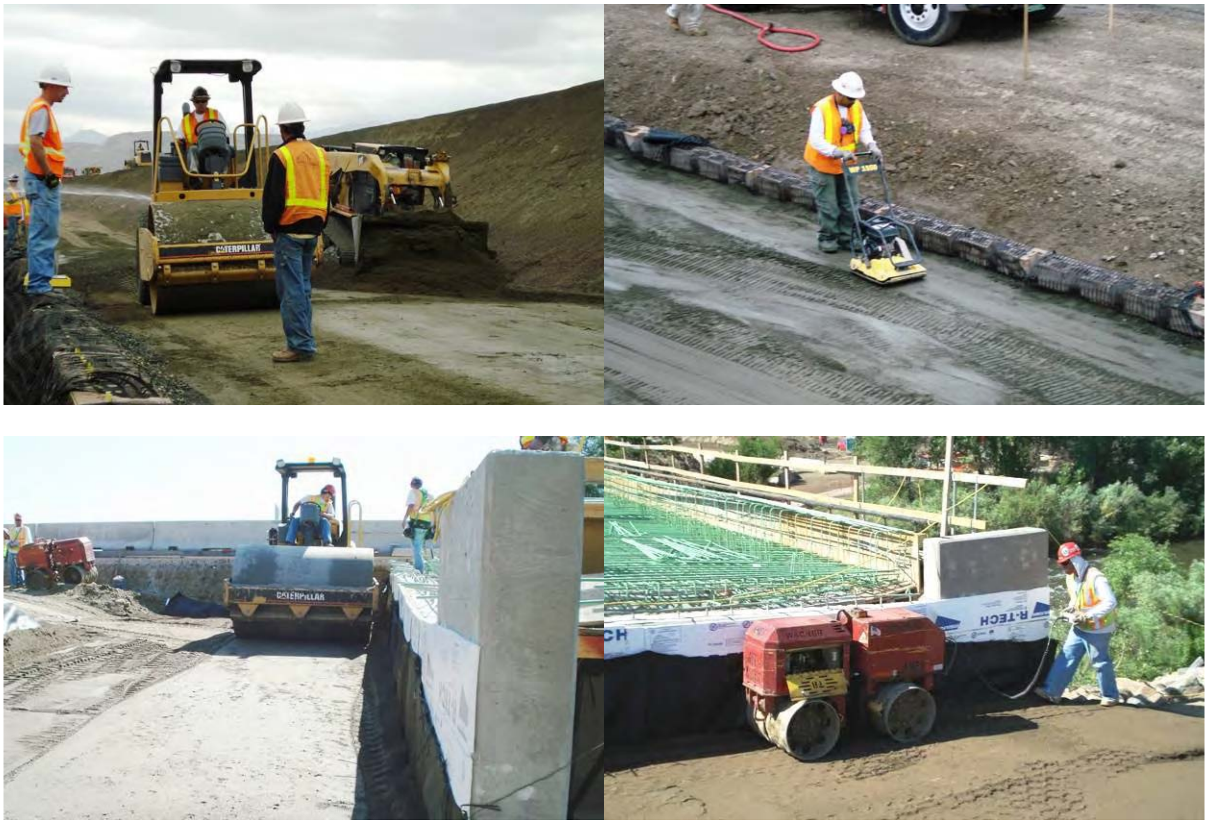


Figure 4-6. Near Wall Compaction Techniques

4.2 Identification of Target Values

To implement LWD, DCP, and/or Clegg Hammer based testing in earthwork QA, test results must reveal E_{LWD}, CIV and DPI values that correlate to the required dry density requirements. Here, we investigate whether such E_{LWD}, CIV and $\overline{DPI}$ "target values" are evident. Before examining the collected data for target values, it is beneficial to introduce the proposed specification. The proposed QA methodology is similar in concept to the Mn/DOT specification (see Section 2.2), and would involve comparison of E_{LWD}, CIV or DPI data collected during QA against an E_{LWD}, CIV or $\overline{DPI}$ target value.

The specification would require that the average of spot test data ($\hat{\mu}$) within a test area or lot minus the standard error (SE) of the data be greater than the target value (TV) as shown in Equation 2. SE is the standard deviation of the average, and is an unbiased estimate of uncertainty in a sample average. SE is quantified in Equation 3 using standard deviation (σ) and number of test points (n) (Bevington and Robinson 2003).

$$\hat{\mu} - SE > TV \tag{2}$$

$$SE = \frac{1}{\sqrt{n}}\sigma \tag{3}$$

The form of Equation 2 has statistical significance. Assuming a Gaussian distribution in the data, there is 68% confidence that the true test bed average value is within the window $\hat{\mu} \pm SE$. Recall that $\hat{\mu}$ is a sample average, and an estimate of the true average. Consequently, there is 84% confidence that the true test bed average is greater than $\hat{\mu} - SE$. Hence, the requirement in Equation (2) provides 84% confidence that the sample average $\hat{\mu}$ is greater than the TV. The specification is discussed further in Chapter 5 and only introduced here to provide some backdrop for the evaluation of TVs.

E_{LWD}, $\overline{DPI}$ and CIV TVs were determined by examination of all data sets collected, and by using the average %C as an indication of a passing or failing test bed. As shown below, the approach involves trial and error setting of the TV in an attempt to minimize false positives and true negatives when compared to the existing 95 %C specification.

E_{LWD} *Target Value* – To develop an E_{LWD} TV for Class 1 structure backfill, data from 12 test beds (18-29) was analyzed. There were generally 8 to 12 LWD tests and locations in each test bed. Tests were conducted in locations exhibiting a distribution of points passing and failing 95 %C as determined by the NG. E_{LWD} from all test locations within each test bed were averaged. Table 4-A presents LWD data from test beds 18 to 29, including number of points passing 95 %C (n_p), number of points failing 95 %C (n_f), average %C, average moisture (w), average E_{LWD} ($\hat{\mu}_{E-LWD}$), standard deviation of E_{LWD} data (σ), range of E_{LWD} (R), SE of E_{LWD} and $(\hat{\mu} - SE)_{E-LWD}$.

Table 4-A. LWD Data Summary

Test Bed	Soil #	n	n_p	n_f	%C	w (%)	E_{LWD} Statistical Parameters				
							$\hat{\mu}$	σ	R	SE	μ - SE
18	5	9	9	0	96.4	4.6	35.6	6.0	18.2	2.0	33.6
19	5	15	12	3	97.1	5.3	40.5	6.4	23.4	1.7	38.9
20	3	10	5	5	94.8	3.4	51.9	9.3	31.0	2.9	49.0
21	3	11	3	8	93.0	5.2	30.8	8.5	35.4	2.6	28.3
22	3	10	7	3	95.4	5.1	42.6	6.8	21.2	2.2	40.4
23	3	6	3	3	95.9	5.0	30.9	11.4	38.1	4.7	26.3
24	3	11	5	6	95.3	5.0	39.1	16.8	59.8	5.1	34.0
25	3	14	12	2	96.4	4.4	38.1	6.3	25.3	1.7	36.4
26	3	15	2	12	91.7	4.6	31.5	8.7	35.9	2.2	29.3
27	3	10	4	6	95.0	3.7	37.1	9.3	28.6	2.9	34.2
28	3	12	6	6	94.0	5.8	35.1	11.1	39.8	3.2	31.9
29	3	10	5	5	93.9	5.1	40.6	5.9	21.1	1.9	38.7

Consistent with the statistical basis of the proposed specification, TV was estimated by evaluating $(\hat{\mu} - SE)_{E-LWD}$ as shown in Figure 4-8. The 95 %C line is highlighted as is the proposed TV E_{LWD} = 32.5 MPa. In Figure 4-7, false positives (FP) are identified as test beds that would fail per the %C criteria but pass per the E_{LWD} TV. Similarly, true negatives (TN) are identified as test beds that would pass the %C criteria but fail per the E_{LWD} TV criteria. The chosen TV produced the fewest number of FPs and TNs. Moreover, the statistical nature of this approach implies that there will be FPs and TNs.

The $\hat{\mu}_{E-LWD}$ and $(\hat{\mu} - SE)_{E-LWD}$ values are highlighted for test beds 18 and 28 to illustrate the influence of SE. For test bed 18, $\hat{\mu}_{E-LWD}$ = 35.6 MPa and $(\hat{\mu} - SE)_{E-LWD}$ = 33.6 MPa, while for test bed 28, $\hat{\mu}_{E-LWD}$ = 35.1 MPa and $(\hat{\mu} - SE)_{E-LWD}$ = 32.0 MPa. The average %C data indicated a passing test bed 18 and a failing test bed 28 (see Table 4-A). Hence, the SE was important in identifying test bed 28 as a failing test bed. It is worth mentioning that SE values ranged from 2-5 (Table 4-A) and constitutes 5-10% of $\hat{\mu}_{E-LWD}$. SE can be reduced by increasing the number of test points.µ

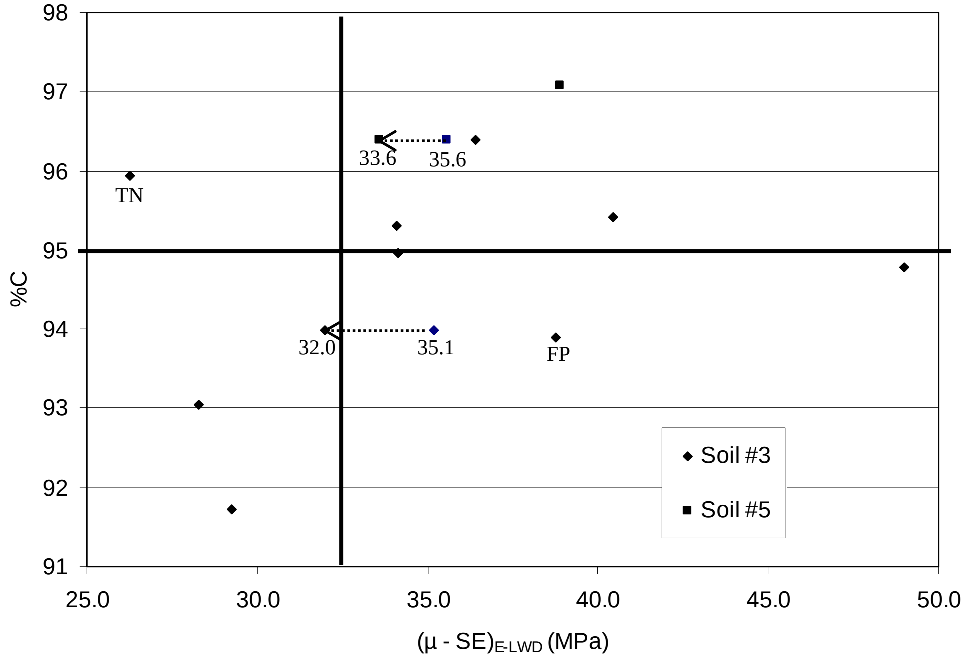


Figure 4-7. $(\hat{\mu} - SE)_{E-LWD}$ **vs. %C**

The possible influence of moisture on E_{LWD} was also evaluated. Average moisture content versus $(\hat{\mu} - SE)_{E-LWD}$ is plotted in Figure 4-8. Figure 4-8 shows no significant trend. The influence of moisture content on E_{LWD} was considered negligible for these Class 1 structure backfill test beds.

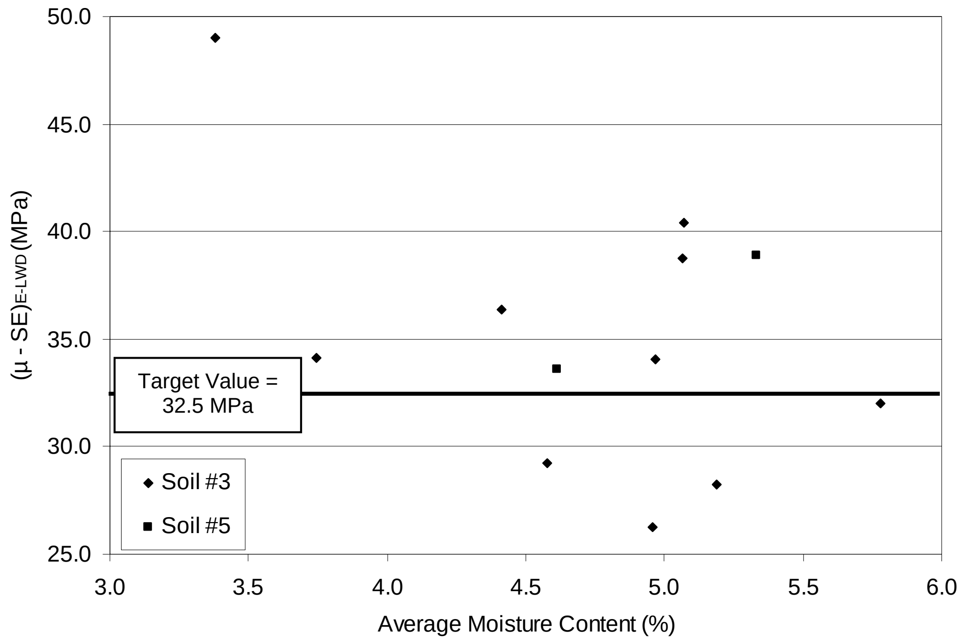


Figure 4-8. Average Moisture Content vs. $(\hat{\mu} - \mathrm{SE})_{\mathrm{E-LWD}}$

It should be noted that E_{LWD} values are device and device parameter specific (e.g., plate diameter, drop mass and height). Therefore, the target values recommended in this study are based on 200 mm plate diameter Zorn device with an impulse of 7.07 kN. To calculate E_{LWD}, a Boussinesq linear half space solution is used assuming Poisson's ratio of 0.5 and uniform stress distribution.

Clegg Hammer Target Value – CIV data from 17 Class 1 backfill test beds were analyzed to develop a TV CIV. The statistics of CIV data are presented in Table 4-B, and test bed $(\hat{\mu} - \mathrm{SE})_{\mathrm{CIV}}$ is plotted versus %C in Figure 4-9. The 95 %C line and proposed CIV = 11.9 TV line are highlighted. The CIV = 11.9 TV results in three FPs and two TNs. Again, these are acceptable given the statistical nature of this approach. The statistics $\hat{\mu}_{\mathrm{CIV}}$ and $(\hat{\mu} - \mathrm{SE})_{\mathrm{CIV}}$ are highlighted for test beds 19 and 26 to illustrate their importance and relationship to the proposed criteria. Moreover, these two test beds can be used to illustrate how the specification would work.

Table 4-B. Clegg Hammer Data Summary

Test Bed	Soil #	*n*	n_p	n_f	%C	*w* (%)	CIV Statistical Parameters				
							μ	σ	*R*	*SE*	*μ - SE*
7	3	10	3	7	93.5	5.1	9.1	1.7	5.7	0.5	8.6
9	3	2	2	0	95.5	6.3	12.4	0.5	0.7	0.4	12.0
12	5	3	3	0	96.3	7.0	12.0	2.8	5.2	1.6	10.4
16	5	6	4	2	95.8	6.3	11.7	1.4	4.1	0.6	11.1
17	5	6	2	4	96.3	6.5	12.1	0.1	0.2	0.1	12.0
18	5	9	9	0	96.4	4.6	13.6	1.1	2.5	0.4	13.2
19	5	15	12	3	97.1	5.3	12.6	1.3	4.1	0.3	12.3
20	3	10	5	5	94.8	3.4	16.9	2.0	5.2	0.6	16.3
21	3	11	3	8	93.0	5.2	15.7	2.9	10.4	0.9	15.2
22	3	10	7	3	95.4	5.1	15.8	2.1	7.5	0.7	15.1
23	3	6	3	3	95.9	5.0	14.8	1.6	4.3	0.7	14.1
24	3	11	5	6	95.3	5.0	15.4	3.8	12.7	1.1	14.3
25	3	14	12	2	96.4	4.4	16.1	1.8	5.7	0.5	15.6
26	3	15	2	12	91.7	4.6	12.2	2.1	6.6	0.6	11.6
27	3	10	4	6	95.0	3.7	14.7	1.7	5.6	0.5	14.2
28	3	12	6	6	94.0	5.8	12.7	2.3	7.9	0.7	12.0
29	3	10	5	5	93.9	5.1	14.2	1.7	5.2	0.6	13.6

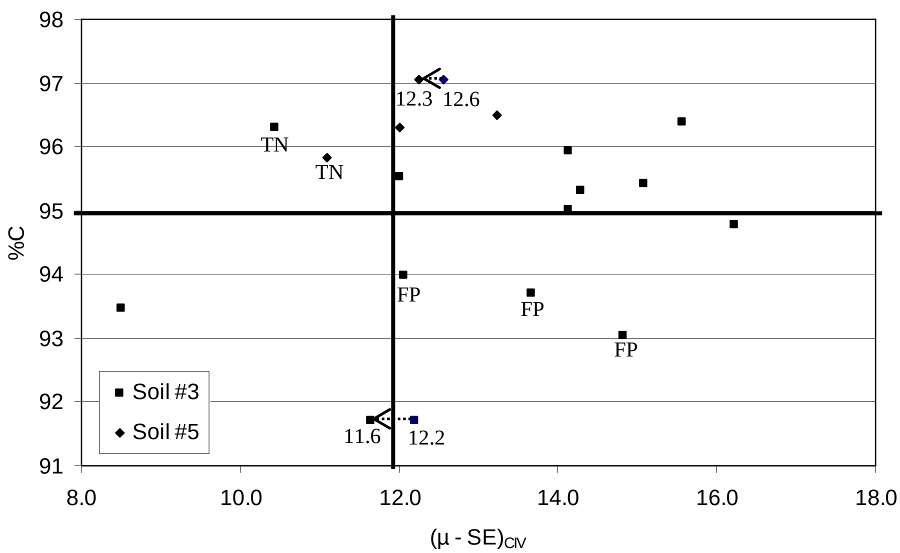


Figure 4-9. $(\hat{\mu} - \mathrm{SE})_{\mathrm{CIV}}$ **vs. %C**

The Clegg Hammer QA specification would be stated as Equation (4).

$$\left(\hat{\mu}-\frac{1}{\sqrt{n}}\hat{\sigma}\right)_{CIV} > 11.9 \tag{4}$$

Assume a QA inspector conducted 15 Clegg Hammer tests on an MSE earthwork section and that the QA test results are identical to test bed 19 data in Table 4-B. The $(\hat{\mu}-SE)_{CIV}$ = 12.3 for test bed 19, and therefore would pass inspection. Conversely if test bed 26 were used, $(\hat{\mu}-SE)_{CIV}$ = 11.6 and it would fail inspection.

The influence of moisture on CIV was also evaluated. The results in Figure 4-10 suggest that CIV decreases with increasing moisture; however, the correlation is not strong. Moving forward with CIV TVs, the moisture influence requires further monitoring.

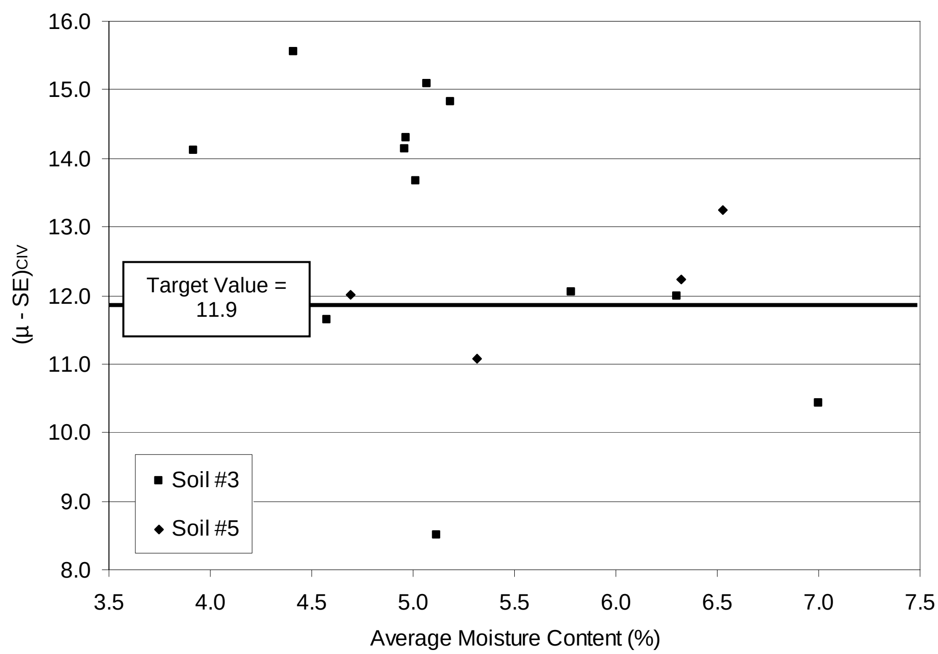


Figure 4-10. Average Moisture Content vs. $(\hat{\mu}-SE)_{CIV}$

DCP Target Value – DCP data collected on 14 test beds was evaluated to help identify a TV $\overline{\text{DPI}}$ (see Table 4-C). For each test bed, 3 to 15 DCP tests were conducted and $\overline{\text{DPI}}$ values were determined. Recall that each $\overline{\text{DPI}}$ reflects the average DPI (less the first two seating drops) over a 200 mm penetration depth to match with lift thickness. This depth of analysis can be modified to accommodate thicker or multiple lifts. Figure 4-11 illustrates the relationship between $(\hat{\mu} - \text{SE})_{\overline{\text{DPI}}}$ and %C. The 95 %C line and proposed $\overline{\text{DPI}}$ TV = 10.2 are highlighted. The TV $\overline{\text{DPI}}$ = 10.2 produced 3 FPs and 2 TNs.

Table 4-C. DCP Data Summary

Test Bed	Soil #	*n*	n_p	n_f	%C	*w* (%)	DCP Statistical Parameters				
							μ	σ	*R*	*SE*	μ - SE
7	3	15	5	10	93.5	5.2	11.5	2.0	7.7	0.5	11.0
9	3	5	5	1	96.4	6.8	16.8	2.6	6.6	1.1	15.7
10	3	5	5	0	97.6	6.5	12.7	2.4	6.3	1.1	11.6
11	4	11	11	0	97.1	6.1	11.2	1.5	5.8	0.8	10.4
12	3	3	3	0	96.3	7.0	11.1	0.4	1.3	0.2	10.9
13	4	6	5	1	96.0	5.7	16.6	1.9	4.6	2.1	14.5
14	4	9	7	2	97.2	5.7	12.9	2.6	6.9	1.3	11.6
15	4	3	1	2	95.3	3.6	15.9	3.5	6.8	3.0	12.9
16	5	6	4	2	95.8	6.3	12.6	1.3	3.4	1.7	10.9
17	5	5	2	3	93.9	5.7	14.3	3.6	8.7	2.7	11.6
18	5	6	6	0	96.5	4.6	10.3	0.8	2.1	1.3	9.0
19	5	15	12	3	97.1	5.3	9.9	1.5	5.5	0.8	9.1
20	3	10	5	5	94.8	3.4	6.9	1.0	2.8	0.3	6.6
21	3	4	1	3	94.2	4.3	8.3	1.1	4.0	0.6	7.7

The influence of moisture on $\overline{\text{DPI}}$ data was also evaluated. Figure 4-12 suggests that a correlation exists between $\overline{\text{DPI}}$ and moisture. As the moisture increases, $\overline{\text{DPI}}$ decreases. These results imply that moisture dependence should be factored into $\overline{\text{DPI}}$ TVs, and that QA personnel must measure moisture along with $\overline{\text{DPI}}$ to evaluate acceptance. From a practical perspective, this constitutes a limitation of DCP testing.

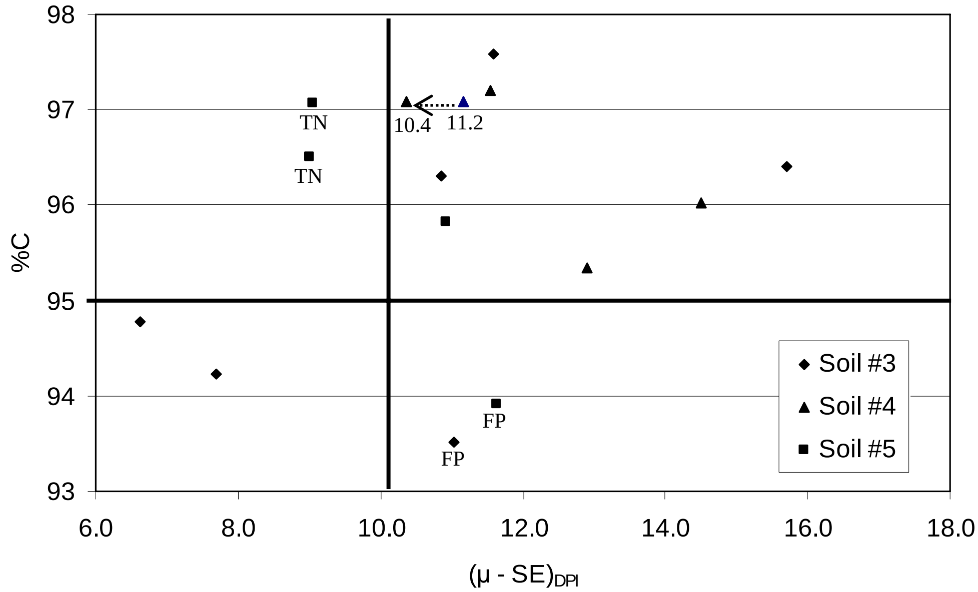


Figure 4-11. $(\hat{\mu} - \mathrm{SE})_{\overline{\mathrm{DPI}}}$ **vs. Percent Compaction**

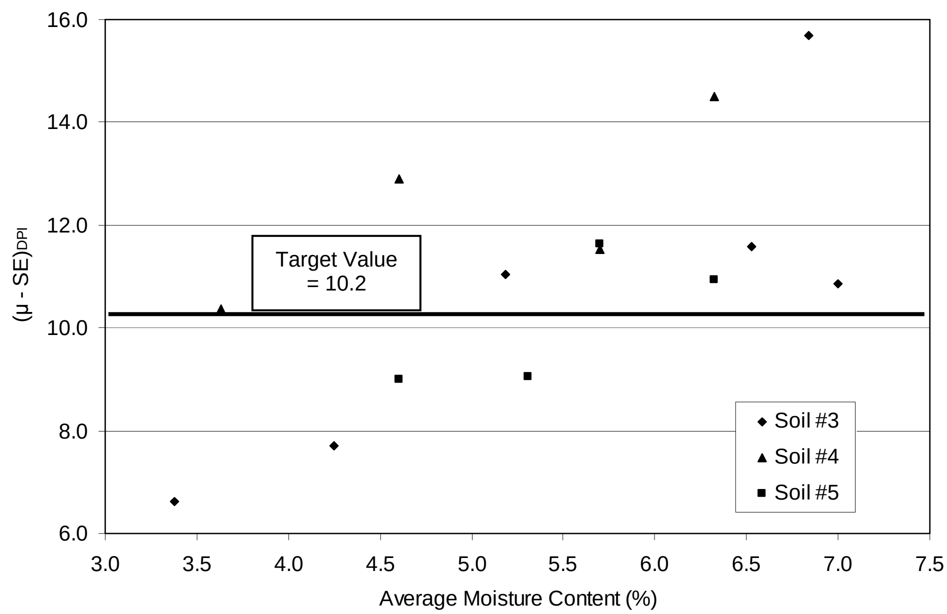


Figure 4-12. Average Moisture Content vs. $(\hat{\mu} - \mathrm{SE})_{\overline{\mathrm{DPI}}}$

4.3 Conclusions

Extensive testing on MSE wall and bridge approach earthwork compaction revealed that LWD, The DCP and Clegg Hammer are both capable of reflecting the compacted state of Class 1 structure backfill soil. In fact, E_{LWD}, CIV and $\overline{DPI}$ are much more sensitive to changes in compaction than density. While dry density ranged by 20% from typical uncompacted to fully compacted states, E_{LWD}, CIV and $\overline{DPI}$ were found to vary by 500%, 400% and 1000% respectively. The LWD, DCP, and Clegg Hammer are all capable of evaluating soil properties within 0.3 m (1 ft) of the wall face. Testing with all devices revealed that adequate compaction is not being achieved within 1 m (3-4 ft) of MSE wall faces. This is attributed to the compaction procedure where contractors are reluctant to use vibratory rollers within 1 m (3 ft) of the wall. The vibratory plates used in this zone are not providing adequate compactive effort. The inadequate compaction in this zone is exacerbated by the measurement restriction of the NG within 1.6 m (5 ft) of the wall.

An evaluation of the data reveals that target values (TVs) exist for E_{LWD}, CIV and $\overline{DPI}$ that could serve as surrogates for the current 95 %C density requirement. Over multiple sites, test beds and Class 1 backfill soils, consistent TVs emerged. The observed TVs for E_{LWD}, CIV and $\overline{DPI}$ were found to be 32.5 MPa, 11.9 and 10.2 mm/blow. Within the scatter of the data, the TVs did not vary across the different Class 1 backfill soils. $\overline{DPI}$ values appear to be sensitive to moisture while E_{LWD} seem to be insensitive to moisture for these soils tested. The moisture sensitivity of CIV was inconclusive. Moisture sensitivity constitutes a limitation to implementing the TV approach in the absence of NG testing because moisture would need to be measured. Currently, LWD, DCP and Clegg Hammer devices do not include moisture measurement.

The DCP exhibited two limitations. Because the DCP test involves penetration through the soil, the DPI is influenced by the placed geogrid (MSE walls) and geofabric (bridge approach). Second, the moisture sensitivity of $\overline{DPI}$ requires consideration of moisture in developing TVs and evaluating acceptance. When compared to the LWD and Clegg Hammer results where moisture sensitivity was not clearly observed, DCP implementation requires more work.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Findings

A number of devices were investigated for use in MSE wall and bridge approach earthwork quality assurance (QA). If capable, one or more devices would be selected to supplement nuclear gage (NG) testing in the short term, and possible replace NG testing in the long term. Of the six devices initially explored through literature review and a study of best practices nationally, three were selected for field assessment, namely the DCP, LWD, and Clegg Hammer. Field testing with the LWD, Clegg Hammer, DCP, and NG was performed at two construction sites in Colorado and included 30 test beds and 5 Class 1 structure backfill soils.

While not the objective of the study, extensive field testing revealed that the current CDOT practice of single position NG testing is inadequate. The uncertainty in single position NG density was found to be equivalent to ± 3-4 percent compaction (%C). The orientation of the device alone (e.g., north, south, east, west) often determined whether a passing (> 95 %C) or failing (< 95 %C) density was achieved. Moreover, soil is heterogeneous, and soil density, shear strength and modulus vary spatially. This heterogeneity should be accounted for through a statistically-based QA approach.

Extensive testing on MSE wall and bridge approach earthwork compaction sites revealed that the LWD, DCP, and Clegg Hammer are all capable of reflecting the compacted state of Class 1 structure backfill soil. E_{LWD}, CIV and $\overline{DPI}$ are much more sensitive to changes in compaction than density. While dry density ranged by 20% from typical uncompacted to fully compacted states, E_{LWD}, CIV and $\overline{DPI}$ were found to vary by 500%, 400% and 1000% respectively. Testing with all devices revealed that adequate compaction is not being achieved within 1 m (3-4 ft) of MSE wall faces. This is attributed to the compaction procedure where contractors are reluctant to use vibratory rollers within 1 m of the wall. The vibratory plates used in this zone are not providing adequate compactive effort. The inadequate compaction in this zone is exacerbated by the measurement restriction of the NG within 1.6 m (5 ft) of the wall.

An evaluation of the data reveals that target values (TVs) exist for E_{LWD}, CIV and $\overline{DPI}$ that could serve as surrogates for the current 95 %C density requirement. Over multiple sites, test beds and Class 1 backfill soils, consistent TVs emerged. The observed TVs for E_{LWD}, CIV and $\overline{DPI}$ were found to be 32.5 MPa, 11.9 and 10.2 mm/blow. Within the scatter of the data, the TVs did not vary across the different Class 1 backfill soils. $\overline{DPI}$ values appear to be sensitive to moisture while E_{LWD} seem to be insensitive to moisture for these soils tested. The moisture sensitivity of CIV was inconclusive. Moisture sensitivity constitutes a limitation to implementing the TV approach in the absence of NG testing because moisture would need to be measured. Currently, LWD, DCP, and Clegg Hammer devices do not include moisture measurement.

Table 5-A summarizes the key attributes and capabilities of the LWD, DCP, and Clegg Hammer. The LWD, DCP, and Clegg Hammer are all capable of evaluating soil properties within 0.3 m (1 ft) of the wall face. This is a significant advantage over the 1.6 m (5 ft) wall proximity restriction of the NG, particularly in light of the inadequate compaction observed and measured within 1 m of MSE walls. The E_{LWD} is most closely aligned with design parameters (e.g., modulus) for pavements, while CIV is an index parameter that is currently not linked to design parameters. The DCP exhibited two key limitations. Because the DCP test involves penetration through the soil, the DPI is influenced by the placed geogrid (MSE walls) and geofabric (bridge approach). In addition, the moisture sensitivity of $\overline{DPI}$ values requires consideration of moisture in developing TVs and evaluating acceptance. When compared to the LWD and Clegg Hammer results where moisture sensitivity was not clearly observed, DCP implementation requires additional effort.

The LWD and Clegg Hammer are both deemed suitable QA devices for MSE wall and bridge approach Class 1 structure backfill QA. They were found to be equally effective in capturing the degree of compaction. The Clegg Hammer is less expensive and easier to use than the LWD. Conversely, the LWD produces a modulus that can be tied to design and there is significant momentum nationally towards LWD use. As described in Section 5.2, we recommend that CDOT continue to consider these two devices.

Table 5-A. Summary of Device Capabilities

	LWD	Clegg Hammer	DCP
Depth of investigation	30-45 cm (12-18 in) with 300 mm plate diameter; 20-30 cm (8-12 in) with 200 mm plate diameter	14-26 cm (5-10 in)	Any depth up to 120 cm (48 in)
Testing & transport time	4 minutes (1 person)	2 minutes (1 person)	5 to 15 minutes (2 people)
Cost	\$8,000 - \$15,000	\$3,000 - \$3,500	Manual \$600 - \$1,000
Proximity to wall for testing	20 cm (8 in)	13 cm (5 in)	20 cm (8 in)
Strengths	- Provides actual deflection and modulus which could be used in design or performance based QA - Has been implemented in other states (MN) and in Europe - Gaining momentum in DOT community as pavement evaluation and design tool - Multiple size loading plates enable measurement over range of modulus and depth - E_{LWD} was found to be insensitive to moisture for Class 1 structure backfill	- Simplest device to operate & transport around on site - Relatively low cost - CIV was found to be insensitive to moisture for Class 1 structure backfill - CIV has the potential to be linked to design parameters	- Simple design, robust construction, good portability - Well studied and documented track record - Successfully used in other states (MN, TX, IN) and in military
Weaknesses	- Sensitivity to changes in compaction not well developed - Can be difficult to transport - Relatively high cost	- Limited published information and data - CIV is not an engineering property and correlations are limited	- DPI is influenced by geogrid and geofabric of MSE wall and bridge approaches - Deeper testing in dense material is time consuming - Large aggregate may cause erroneous results - DPI was found to be moisture sensitive for Class 1 structure backfill

5.2 Recommended Approach for CDOT Adoption and Further Study

The following three recommendations stem from the findings in this study. If CDOT decides to pursue these recommendations, is important that CDOT personnel play an integral role in carrying out the suggested tasks.

5.2.1 Revise NG Testing Procedure

As described in Section 5.1, single position NG testing is inadequate given the significant variability in earthwork soil properties. To better represent the variability in soil density and to minimize uncertainty in reported data, we recommend that CDOT increase the required number of tests per lot (evaluation area). A specific number is not recommended here as it was not the focus of this study; however, an approach similar to that recommended in Section 5.2.3 could be pursued. At an absolute minimum, CDOT could improve the current procedure with no additional time/cost by modifying the current inspection practice from a single position 4 minute reading to a four position NG test (i.e., north, south, east, west) with each position as a 1-minute reading. A revised approach could be investigated within the pilot implementation of new devices as described below.

5.2.2 Implement LWD and Clegg Hammer Pilot Study to Supplement NG-Based QA

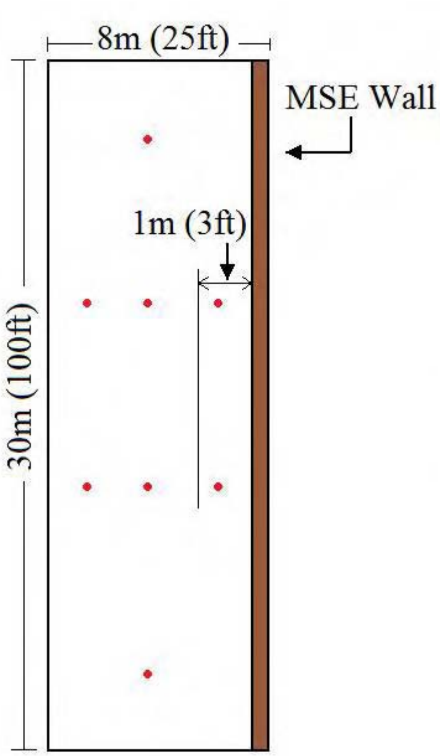
We recommend that CDOT implement a pilot study using the LWD and Clegg Hammer in conjunction with NG testing on 5-10 MSE wall and/or bridge approach construction sites. The objectives of the pilot program are multiple: (1) identify E_{LWD} and CIV target values (TVs) for the various soils, site & moisture conditions, seasons, etc., observed in practice; (2) evaluate if/how TVs change with soil type, moisture, season, and from site to site; (3) populate a database of TVs; (4) allow a range of CDOT inspectors, consultants and contractors to evaluate all aspects of the devices, e.g., handling, operation, durability, portability. Ultimately, CDOT personnel will determine which device best fits their needs. Therefore, it is critical that CDOT personnel buy into and support the pilot study.

The recommended pilot implementation procedure is as follows. LWD and/or Clegg Hammer testing should be performed at a minimum of 8 locations within a lot. Note that LWD and Clegg testing is much faster than NG testing (see Table 5-A). Recommended MSE wall and bridge approach lot sizes are summarized in Table 5-B and illustrated in Figure 5-1. At least 25% of the spot tests should be performed within 1 m (3-4 ft) of the MSE wall or bridge abutment and individual tests should be well-spaced to capture the variability in soil properties.

NG testing should be performed on these lots. The frequency of NG tests may be reduced to a minimum of 4. Each NG test should employ the 4-position approach described in Chapter 4. The average NG density per lot should be used to evaluate acceptance. Similarly, the E_{LWD} and CIV data can be evaluated to identify TVs, etc.

Table 5-B: On-site Testing Protocol of Pilot Study

	MSE wall	Bridge approach
Lot size	Approx. 30m x 8m (100' x 25')	Approx. 12m x 6m (40' x 20')
Number of tests	Minimum of 8	Minimum of 8
Proximity to wall	25% within 1m (3') of wall	25% within 1m (3') of abutment
Proximity of test locations	Tests no closer than 2m (6')	Tests no closer than 2m (6')
Test distribution	Evenly distributed along wall length	Along full lane width



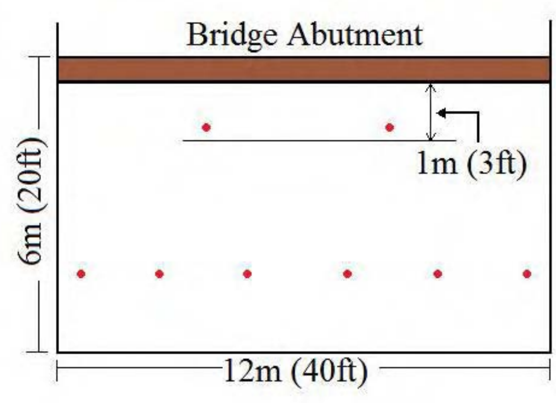


Figure 5-1. Recommended Test Lot for Pilot Study

5.2.3 Develop Specification for LWD and/or Clegg Hammer Use as a Supplement or Replacement for NG

The recommendation detailed in Section 5.2.2 will reveal the efficacy of the LWD and Clegg Hammer as a supplement or replacement for the NG in earthwork QA. In addition, the results of the pilot study combined with the findings herein will lead to specific guidelines for TVs, number of required tests, statistical approach to data analysis and acceptance criteria, lot size, etc. A specification can then be written to replace the appropriate sections of the CDOT Bridge Project Special Provisions, the Standard Specifications for Road and Bridge Construction, and the Field Materials Manual.

REFERENCES

1) Abu-Farsakh, M., Alshibi, K., Nazzal, M., Seyman, E., 2004, "Assessment of In-Situ Test Technology for Construction Control of Base Courses and Embankments," *Louisiana Department of Transportation and Development: Louisiana Transportation Research Center*, Baton Rouge, LA.

2) Adam, D., Adam, C., Kopf, F., 2004, "The dynamic load plate test with the Light Falling Weight Device: Experimental and numerical investigations" In Proc. of The 11th International Conference on Soil Dynamics & Earthquake Engineering (11th ICSDEE) and The 3rd International Conference on Earthquake Geotechnical Engineering (3rd ICEGE), University of California, Berkeley, USA, 7th - 9th January 2004.

3) Adams, A. R., 2004, "Assessment of Rapid Methods for Density Control of MSE Walls and Embankments," [Masters Thesis]: University of Texas at Austin, 178 pp.

4) Ayers, P.D. and Bowen, H.D., 1988, "Laboratory Investigation of Nuclear Density Gauge Operation." *Transactions of the ASAE.* American Society of Agricultural Engineers. March 1988.

5) Bevington, P.R. and Robinson D.K., 2003, Data Reduction and Error Analysis for the Physical Sciences. 3rd Edition, McGraw-Hill, Boston.

6) Colorado Dept of Trans. 2005 Standard Specifications for Road and Bridge Construction. available online at http://www.dot.state.co.us/DesignSupport/Construction/Index.htm

7) Colorado Dept of Trans. 2007 Field Materials Manual. available online at http://www.dot.state.co.us/DesignSupport/Field%20Materials%20Manual/2007/2007%20Field%20Material%20Manual%20Index.htm

8) Colorado Dept of Trans. Bridge Project Special Provisions. Section 504. available online at http://www.dot.state.co.us/Bridge/ProjectSpecials/Project_Specials.htm

9) Durham GeoSlope Indicator Inc., 2006, "Moisture + Density Indicator (M+DI)" product brochure. Stone Mountain, GA. accessed online 28 Nov 06 at http://www.durhamgeo.com/pdf/m_test-pdf/soil/M%2BDI%20Brochure.pdf

10) Erchul, R.A. and Meade, R.B., 1994, "Using an Impact Hammer to Evaluate Compaction." Chesterfield County, VA.

11) Fleming, P.R., Frost, M.W., and Lambert, J.P., 2007, "A Review of the Lightweight Deflectometer (LWD) for Routine Insitu Assessment of Pavement Material Stiffness", Transportation Research Record 2004, Soil Mechanics, pp. 80-87.

12) Gas Technology Institute (GTI), 2004, "Evaluation of Soil Compaction Measuring Devices." GRI Report No. 04/0067. Gas Research Institute. Des Plaines, IL. April 2004.

13) Hoffmann, O., Guzina B., Drescher A., May 2003. "Enhancements and Verification Tests for Portable Deflectometers." Report No. MN/RC – 2003-10, Minnesota Department of Transportation, St Paul, MN.

14) Humboldt Manufacturing Inc., 2006, "GeoGauge" product brochure. Northridge, IL. accessed online 28 Nov 2006 at http://www.humboldtmfg.com/pdf/H-4140GeoGauge.pdf

15) ISSMGE, 2005, "Geotechnics for pavements in transportation infrastructure" *Roller-Integrated Continuous Compaction Control (CCC)*, Technical Contractual Provisions – Recommendations, International Society for Soil Mechanics and Geotechnical Engineering.

16) Kremer, K., and Dai, S., 2004. "Improvement and Validation of Mn/DOT DCP Specification for Aggregate Base Materials and Select Granular Task 3: Field Testing." Minnesota Department of Transportation, Office of Materials, Maplewood, MN.

17) Lafayette Instrument, 2006, "Clegg Hammer Impact Tester" accessed online 28 Nov 06 at http://www.lafayetteclegg.com/

18) Lenke, L. R., McKeen, R. G., Grush, M., 2001, "Evaluation of a Mechanical Stiffness Gauge for Compaction Control of Granular Media," *New Mexico State Highway and Transportation Department Research Bureau*, Albuquerque, NM.

19) Lin, D.F., Liau, C.C., Lin, J.D., 2006. "Factors Affecting Portable Falling Weight Deflectometer Measurements." *J. Geotech. & Geoenv. Engineering*, Vol. 132, No. 6.

20) Minnesota Dept of Trans. 2002 Grading and Base Manual. available online at http://www.mrr.dot.state.mn.us/pavement/gradingandbase/copy%20(2)%20of%20gradingandbase.asp#grading%20and%20base%20manual

21) Minnesota Dept of Trans. *Pilot Spec 2105* "Excavation and Embankment QC/QA – LWD Quality Compaction." available online at http://www.dot.state.mn.us/materials

22) Minnesota Dept of Trans. *Specification 5-692.255* "Modified Dynamic Cone Penatrometer" available online at http://www.dot.state.mn.us/materials

23) Mooney, M.A. and Miller, P.K. "Analysis of Light Falling Weight Deflectometer Test based on In-situ Stress and Strain Response," *J. Geotech. & Geoenv. Engineering*, ASCE, 134(10), 2008, in press.

24) Nazarian, S., Yuan, D., Tandon, V., Arellano, M., 2005, "Quality Management of Flexible Pavement Layers with Seismic Methods," Research Report 0-1735-3, Center for Transportation Infrastructure Systems, University of Texas at El Paso.

25) Salgado, R. and Yoon, S., 2003, "Dynamic Cone Penetration Test (DCPT) for Subgrade Assessment." Joint Transportation Research Program. Proj. No. C-36-45S. Published by Purdue University. West Lafeyette, IN. February 2003.

26) Siekmeir, J.A., Mn/DOT engineer, personal communication, 17 July 2008.

27) Siekmeier, J.A., Young, D., Beberg, D., 2000, "Comparison of the Dynamic Cone Penetrometer with Other Tests During Subgrade and Granular Base Characterization in Minnesota," Nondestructive Testing of Pavements and Backcalculation of Moduli: Third Volume, ASTM STP 1375, S.D. Tayabji and E. O. Lukanen, Eds., American Society for Testing and Materials, West Conshohocken, PA.

28) Siddiqui, S.I., and Drnevich, V.P., 1995, "A new method of measuring density and moisture content of soil using the technique of Time Domain Reflectometry." Rep No.:FHWA/IN/JTRP-95/9, Joint Transportation Research Program, Indiana Dept of Transportation-Purdue University.

29) Troxler Electronic Laboratories Inc., 2006, "RoadReader ™ Nuclear Density Guages." product brochure. Research Triangle Park, NC. accessed online 28 Jul 2008 at http://www.troxlerlabs.com/pdf%20files/3430_40bro.pdf

30) Vertek, a Division of Applied Research Associates Inc., 2006, "Dynamic Cone Penetrometer with Data Acquisition System." product brochure. Randolph, VT. accessed online 28 Nov 06 at http://vertek.ara.com/product%20brochures/dcp%20das.pdf

31) Von Quintus, H.L., Rao, C., Minchin, Jr., R.E., Maser, K.R., Nazarian, S., Prowell, B., 2005. "Nondestructive Testing Technology for Quality Control and Acceptance of Flexible Pavement Construction, Phase II, Part A – Interim Report Task 6." National Cooperative Highway Research Program, Transportation Research Board, National Research Council, NCHRP Project 10-65.

32) Wang, T., CDOT engineer, personal communication, 10 May 2006.

33) White, D.J., Vennapusa, P., Jahren, C.T., 2004, "Determination of the Optimum Base Characteristics for Pavements." Iowa DOT Project TR-482, Iowa Department of Transportation, Ames, IA.