ERDC/GSL TR-08-7

Geotechnical and Structures Laboratory

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Quality Assurance for Rapid Airfield Construction

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Final report

Prepared for Headquarters, U.S. Army Corps of Engineers
Washington, DC 20314-1000

Abstract: This investigation was conducted to formalize a quality assurance (QA) process for rapid airfield construction. The specific aspects of QA that were addressed included compaction operations and the assessment of strength for both soil and stabilized soil layers. The QA for compaction relies on the construction of a test section for determining optimum number of compaction coverages and target soil properties. The essential pieces of equipment for the compaction QA process include a microwave oven, a Clegg hammer, and tools necessary to conduct a volume-replacement density test for in-place soil. This density test, which was developed during this investigation, involves the use of steel shot as the volume replacement material. The use of steel shot, instead of a conventional sand cone apparatus, was found to make the test both simpler and quicker. The Clegg hammer results are the primary means of judging compaction; thus, the requirements for density tests are minimized through a stepwise acceptance procedure. Statistical criteria for evaluating Clegg hammer and density measurements are also included herein. For estimating soil strength in terms of California bearing ratio, the conventional use of the dual-mass dynamic cone penetrometer is recommended. For estimating the strength of cement-stabilized soil and cement plus fiber-stabilized soil, a correlation between Clegg hammer results and unconfined compressive strength was developed.

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Preface

The Quality Assurance for Contingency Airfield Construction project is a part of the Joint Rapid Airfield Construction (JRAC) program. The JRAC program was a comprehensive, 6-year, demonstration-based research and development program executed by the U.S. Army Engineer Research and Development Center (ERDC) during fiscal years 2002–2007. The JRAC program was sponsored by Headquarters, U.S. Army Corps of Engineers.

This publication was prepared by personnel of the ERDC Geotechnical and Structures Laboratory (GSL), Vicksburg, MS. The findings and recommendations presented in this report are based upon studies conducted at the ERDC in Vicksburg, MS. The required laboratory and field testing was conducted at various times from January 2003 through December 2006. The physical testing team consisted of Dr. Reed B. Freeman, Travis A. Mann, L. Webb Mason, and Vernon M. Moore, Airfields and Pavements Branch (APB), GSL. Freeman, Mann, and Chad A. Gartrell, APB, prepared this publication under the supervision of Dr. Gary L. Anderton, Chief, APB; Dr. Larry N. Lynch, Chief, Engineering Systems and Materials Division; Dr. William P. Grogan, Deputy Director, GSL; and Dr. David W. Pittman, Director, GSL.

COL Richard B. Jenkins was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

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Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
inches	0.0254	meters
miles per hour	0.44704	meters per second
pounds (force) per foot	14.59390	newtons per meter
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.45359237	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square yards	0.8361274	square meters

Summary

As part of the Joint Rapid Airfield Construction (JRAC) program, personnel of the U.S. Army Engineer Research and Development Center, Vicksburg, MS, developed quality assurance procedures for contingency airfield construction. This effort was executed as part of the “Enhanced Airfield Construction Productivity” component of JRAC. The investigation included equipment evaluations, comparisons, and selections, which involved both laboratory and field studies. The products include guidance for test procedures, testing frequencies, data reduction, and construction decisions. More specifically, the findings can be summarized as follows:

1. The standard microwave test procedure (ASTM D 4643) is recommended for measuring the moisture content of soil. The direct heating method (ASTM D 4959) is recommended as a backup procedure.
2. A volume replacement method was recommended for measuring the in-place density of soils. This test method, which was named the “steel shot density test,” is a hybrid between the sand cone method (ASTM D 1556) and a simpler sand replacement test (ASTM D 4914). The steel shot density test, which involves 3/16-in. stainless steel balls, is fast, easy, and sufficiently accurate. Step-by-step procedures are explained herein.
3. The dynamic cone penetrometer (DCP), ASTM D 6951, is recommended for estimating the strength of soil. Standard procedures for reducing DCP data and converting these data to California bearing ratio (CBR) are reviewed herein.
4. The Clegg hammer (ASTM D 5874) is recommended for estimating the strength of cement-stabilized layers (with or without fibers). Two equations are recommended for converting Clegg impact value (CIV) to unconfined compressive strength (UCS) in units of pounds (force) per square inch:

$$\log(\mathrm{UCS}) = 0.081 + 1.309 \cdot \log(\mathrm{CIV})$$

$$\mathrm{UCS} = 12.51 \cdot (\mathrm{CIV}) - 285.9$$

The first equation is conservative, and the second equation provides estimates of “likely” values. Together, they provide a range of probable

unconfined compressive strengths. These equations are limited to CIVs that are greater than or equal to 32, which corresponds to a UCS value of approximately 100 psi for both equations. (The difference between UCS estimates increases with increasing CIV.)

5. Because of its simplicity and speed, the Clegg hammer is also recommended as a backup tool for estimating the strength of soil. The recommended equation for converting CIV to CBR (percent) is

$$\text{CBR} = 0.05 \cdot \text{CIV}^2 + 0.53 \cdot \text{CIV}$$

   This equation is limited to CIVs less than or equal to 40, which corresponds to a CBR of approximately 100%.
6. The compaction procedures recommended herein are highly dependent on the results of a compaction test section. The test section serves several purposes, among which are identifying the optimum number of compactor coverages and obtaining target material properties. This process involves the Clegg hammer as the primary tool and the steel shot density test as the secondary tool.
7. For convenience and simplicity, the lot size for JRAC operations is flexible and is defined as being as close to 500 yd^2 as possible and preferably between 400 and 600 yd^2. Smaller lots are allowed to prevent a lot from including more than 1 day's placement. Testing includes moisture content, density, smoothness, and strength (Clegg hammer).
   a. Four moisture contents are required for each lot to ensure that the compaction is accomplished near optimum moisture content (OMC). The average moisture content must be within -1% to +2% of the target OMC, and no single measured moisture content can be outside of the range -2% to +3% of the target OMC.
   b. Density tests are time-consuming, so a stepwise approach is recommended where as few as two tests may be required for each lot. Warning and action limits are established for both the mean value and any single test, based on results of the compaction test section.
   c. Smoothness testing is conducted with a 12-ft straightedge wherever smoothness appears to be questionable. Deviations from the straightedge in excess of 3/8 in. shall be corrected by removing material and replacing with new material, or by reworking and recompacting existing material.
   d. Because of the simplicity and speed of the Clegg hammer, 20 tests are required for each lot. The Clegg hammer is the primary device for ensuring quality and consistent construction in a JRAC operation.

> Warning and action limits are established for both the mean value and the lower tail of the distribution of Clegg hammer results. The warning and action limits are based on results of the compaction test section. The mean comparison ensures adequate central tendency for a lot. The lower tail comparison ensures that there are no exceptionally weak areas within the lot.

This report presents the findings of the investigation in the following order:

- Chapters 2 and 3 describe recommended test procedures for moisture and density measurements for soils.
- Chapters 4 and 5 describe recommended test procedures for estimating the strengths of in situ soil and stabilized soil.
- Chapter 6 describes additional uses for the Clegg hammer.
- Chapters 7 and 8 provide guidance for soil compaction.
- Chapter 9 describes a quality assurance program for compaction operations.
- Chapter 10 summarizes the findings from this investigation.

In addition, Appendix A details the calculations of the *t*-Distribution, and Appendix B presents step-by-step procedures for a simplified density test that was developed during this investigation.

1 Introduction

In order to combat widely distributed pockets of terrorist activities, the modern U.S. military must be capable of quick and efficient deployments of people and equipment anywhere in the world. Cargo aircraft will play a key role in this effort, both during the initial projection of forces and during sustainment operations. The airfield infrastructure in many countries is inadequate for large U.S. cargo aircraft. Even when an adequate airfield is initially available, it often requires repairs prior to occupation due to lack of maintenance or battle damage. Therefore, the rapid construction of semi-prepared airfields for cargo aircraft is a critical component to the U.S. military meeting future force projection goals.

To meet this challenge, the U.S. Army Engineer Research and Development Center (ERDC) conducted a 6-year research effort titled "Joint Rapid Airfield Construction (JRAC)." The primary objective of this program was to develop improved tools, methods, and technologies for the U.S. military, toward its endeavor of airfield construction in contingency environments. These developments can be categorized into three technical thrust areas:

1. Remote, optimized site selection
2. Enhanced airfield construction productivity
3. Rapid, innovative soil stabilization

The investigation and findings presented in this report are part of the "Enhanced Airfield Construction Productivity" thrust area. These findings, which were produced by several different work units over a 3-year period, represent only a portion of the products developed within this thrust area.

The collective components of this report seek to address the unique challenges of quality assurance for JRAC operations. Quality assurance, as it is addressed in this report, includes the necessary precautions for ensuring adequate compaction as well as rapid, low-logistics materials testing for remote environments. Relative to private construction and Department of Defense (DoD) civil construction projects, quality assurance for JRAC operations is unique in that the owner and the contractor are the same entity, that is, the U.S. military.

The specific objectives of this investigation for JRAC were to

- Identify test procedures for the purpose of measuring both soil moisture content and in-place soil density
- Identify test procedures for the purpose of estimating the strength of both in situ and stabilized soil
- Provide guidance for soil compaction operations, including the execution of a compaction test section
- Provide a quality assurance program for compaction operations

The scope of this report is limited to the stated objectives. This report does not address several other aspects of JRAC's "Enhanced Airfield Construction Productivity," which includes the use of global positioning system (GPS) controlled construction equipment, the incorporation of modern pulverizer technology, and the development of advanced tools such as the automated dynamic cone penetrometer (DCP).

2 Measuring Soil Moisture Content

Test devices considered

The equipment to be used for estimating soil moisture content for JRAC operations needed to be accurate, fast, and rugged. It also needed to be an acceptable method for handling a wide range of soil types. The following devices were considered for moisture testing:

- Drying oven
- Nuclear density gauge
- Standard microwave
- Computer-controlled microwave
- Source of direct heat
- Calcium carbide gas pressure tester
- Various types of electronic moisture probes

The drying oven method of obtaining moisture content for soil and aggregates (ASTM D 2216) [American Society for Testing and Materials 2006] has long been the standard for geotechnical engineering. For example, the sand cone specification for measuring soil density (ASTM D 1556) requires users to "determine the water content in accordance with Test Method D 2216, D 4643, D 4944, or D 4959. Correlations to Test Method D 2216 will be performed when required by other test methods." According to ASTM D 2216, the drying oven must be thermostatically controlled, preferably a forced-draft type, and it must meet the requirements of ASTM E 145. For this test, any lid on the soil sample container should be removed, and the soil is dried to a constant mass at a temperature of 110±5°C.

In most cases, drying a test specimen overnight (about 12 to 16 hr) is sufficient and sand may require only about 4 hr. In cases where the necessary time needs to be confirmed, drying should be continued until the change in mass after two successive measurements (greater than 1 hr) is an insignificant amount (less than about 0.1%) (ASTM D 2216). Intermediate mass measurements can be obtained while the sample is hot, but the final mass is obtained after the sample has cooled enough to be handled with bare hands; cooling should be accomplished in a desiccator. Single-operator coefficient of variation has been found to be 2.7%, which means that two

companion measurements should not be considered suspect unless they differ by more than 7.8% of their mean (ASTM C 670).

For estimating the moisture contents of soil in pavement engineering, nuclear density gauges (Photo 1) are used in accordance with ASTM D 3017, “Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth).” Nuclear moisture content testing includes a fast neutron source (typically a sealed isotope material such as americium-beryllium or radium-beryllium) and a slow neutron detector (Coleman 1988). Estimation of moisture content relies on the thermalization or slowing of fast neutrons. Both the neutron source and the detector are located on the bottom of the gauge, near the surface of the soil, so the measurement involves “backscatter” of neutrons through the soil. The moisture content in mass per unit volume of the material under test is determined by comparing the detection rate of thermalized or slow neutrons with previously established calibration data (ASTM D 3017).

Photo 1. Nuclear density gauge.

Drying by microwave oven (ASTM D 4643) offers the advantage of speed over a convection oven. The microwave may cause some aggregate particles to shatter, thus promoting material loss from the container. Shattering is caused by steam explosions or thermal stresses. Larger particles (e.g., larger than the No. 4 sieve) and particles that are porous and/or brittle are

particularly susceptible to shattering (ASTM D 4643). The microwave should not be used when highly accurate results are needed or when the dried soil will be used for any other tests, such as plastic limit testing. Microwave heating can cause differential heating within a sample, and the sample can easily become overheated (heated to over 115°C). For this reason, the microwave can give higher moisture contents than the convection oven, on the order of 0.25%-0.6% (ASTM D 4643).

To overcome the overheating problem with microwaves, Gilbert (1998) invented a computer-controlled microwave with a built-in scale (Photo 2). The computer, along with soil temperature measurements, allows for cyclic heating in order to avoid overheating the soil samples. By preventing overheating, this automatic microwave ensures that the measured moisture contents are closer to those measured by the convection oven. The automatic microwave also minimizes particle shattering.

Photo 2. Computer-controlled microwave oven.

The determination of moisture content by direct heating can be used as a substitute for ASTM D 2216 when more rapid results are desired and slightly less accurate results are acceptable (ASTM D 4959). This is a popular method for field work. The source of heat can be anything that will raise the specimen temperature to 110°C. Commonly used sources of heat include electric, gas, butane or oil-fired stoves, and hotplates. Similar to

the oven procedure, the soil sample is repeatedly stirred, heated, and weighed until two consecutive mass determinations for the dry soil change by 0.1% or less.

The calcium carbide (Speedy®) gas pressure tester (ASTM D 4944) offers the advantage of immediate results (Photo 3). In this test method, calcium carbide reacts with water in the soil to produce acetylene gas. Steel balls in a sealed, metal chamber help to break up soil clumps, thus exposing free water. As moisture content increases, gas production increases, and the pressure in the sealed chamber increases. A dial gauge on the device allows measurement of the pressure inside the chamber. Prior to a construction job and for each relevant soil type, a calibration curve must be established for converting pressure to moisture content of soil. The calibrations are accomplished by measuring both pressure and oven-dry moisture (ASTM D 2216) for companion samples of soil that represent the range of moisture contents to be encountered in the field. This test method is not appropriate for particles larger than the No. 4 sieve, and it is not appropriate for highly plastic soils that are sticky when wet and form hard clods when dry. One additional disadvantage for this test method is that the acetylene gas is flammable.

Photo 3. Calcium carbide gas pressure tester.

Several types of electronic moisture testers were considered, including

1. The Aqua-Spear Moisture Indicator by Mastrad Quality and Test Systems
2. The Field Scout Soil Moisture Meter, Model TDR 300, by GENEQ, Inc.
3. The Soil Moisture Resistivity Probe by Kessler Soils Engineering Products, Inc.
4. The Soil Moisture Meter by ELE International, Inc.

The electronic moisture meters operate on various principles concerning the effects of moisture on the electrical properties of soil. Literature on the Aqua-Spear (Photo 4) mentions the importance of the dielectric constant of soil; the Field Scout (Photo 5) relies on time-domain reflectometry (ASTM D 6565); and the Kessler Resistivity Probe (Photo 6) and the Soil Moisture Meter (Photo 7) both rely on the effects of moisture on electrical resistance of either the soil itself or a “dummy” gypsum block. With exception for the resistivity probe, the other electrical moisture meters were developed for the agricultural industry. Therefore, previous experimentation has involved loose, weak surface soils. All these agricultural probes have difficulty with hard soils because they cannot simply be inserted into the ground. Holes must be drilled; however, this can then lead to problems with contact between the probe and the sidewall of the hole. The Kessler

Photo 4. Aqua-Spear Moisture Indicator.

Photo 5. Field Scout Moisture Meter,
Model TDR 300.

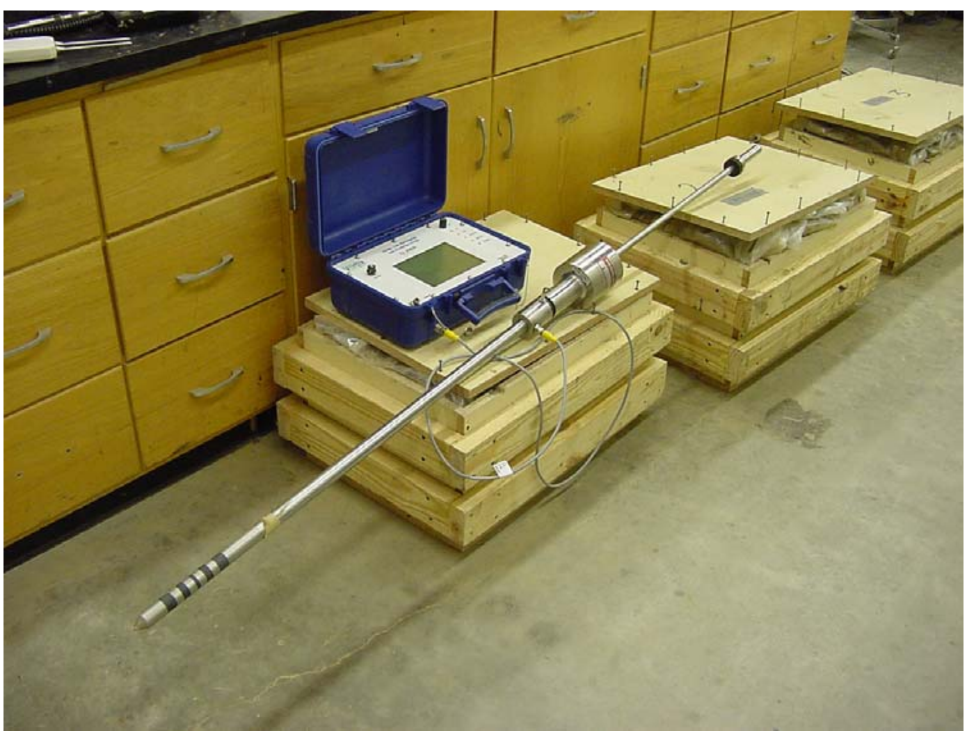

Photo 6. Soil Moisture Resistivity Probe.

Photo 7. Soil Moisture Meter.

Resistivity Probe has the advantage of being developed to be incorporated with DCP technology. The moisture probe shaft can be inserted into a hole previously produced by a DCP soil strength test. The most severe disadvantage for all these probes is that the electrical properties of soil have strong correlations with volumetric moisture content. Volumetric moisture content (w_v) can only be converted to gravimetric moisture content (w_g) if the dry density of soil is known. This would mean that each soil moisture test would have to be accompanied by a soil density test.

$$w_g = \frac{\rho_w}{\rho_d} \cdot w_v \quad (1)$$

where:

ρ_w = density of water
ρ_d = dry density of soil.

Devices eliminated from consideration

Convection drying ovens require too much time (4 to 16 hr) to be used for the purpose of quality control during the rapid construction of a JRAC scenario. Typically, the moisture content of soil needs to be estimated

prior to compaction to ensure that the moisture content is sufficiently close to the soil's optimum moisture content for compaction.

The nuclear density gauge was originally considered the most preferred gauge because of its versatility and ease of use. However, the nuclear gauge was eliminated from consideration due to the likely difficulties of in-theater operations meeting the regulations that are enforced by the U.S. Nuclear Regulatory Commission (NRC). According to these regulations, the U.S. military would need to keep a sufficient number of properly trained Radiation Safety Officers to ensure conformance with all NRC regulations and to maintain records that would be available for audits by the NRC. Shipping of nuclear density gauges would be tedious due to their radioactive components. Any person operating a nuclear gauge would need to have completed a Nuclear Gauge Certification Class (or Refresher Class) within 2 years. Finally, ground transportation, storage, and leak testing of nuclear density gauges would need to adhere to NRC requirements (Troxler 1997).

The computer-controlled microwave was judged too delicate and too specialized for JRAC operations. This piece of equipment could be made more rugged and transportable, but not in the time frame necessary for JRAC. If this piece of equipment were to have a problem in the field, few people know the intricacies of this device well enough to fix the problem. For JRAC, it would make more sense to rely on equipment that can be readily replaced or fixed.

The calcium carbide (Speedy®) gas pressure tester is fast, easy, and rugged. However, it was eliminated for two reasons. It is not appropriate for highly plastic soils, and JRAC requires versatile equipment that is able to handle almost all situations. Second, this device requires pre-calibration for each soil type relating gas pressure to moisture content.

All the electronic moisture probes were eliminated for one or both of the following reasons. First, several of the devices were designed for soft agricultural soils, so they could not be easily inserted into hard soils. Second, each device required a conversion from volumetric moisture content to gravimetric moisture content. This would be necessary because almost all soil compaction experience in geotechnical engineering has been documented using gravimetric moisture contents.

After these eliminations, the remaining moisture content tests are the standard microwave oven and the use of direct heat.

Recommended test procedures

The standard microwave was selected as the primary device for determining the moisture content of soil. The determination of moisture content by direct heating is considered a secondary or backup method. The use of the microwave for JRAC operations follows the procedures outlined in ASTM D 4643. Recommended sample sizes are shown in Table 1. If the soil is clay and relatively dry, any clumps should be cut into sizes smaller than about 1/4 in. in order to speed drying and to minimize temperature differentials. The proper setting and timing for heating soils in a microwave will be determined with experience; however, the "high" setting is generally satisfactory. Also, a good starting point for timing the heat increments includes a 3-min initial duration and 1-min intervals thereafter. The soil should be stirred and weighed between intervals. Heating is finished when the change in soil mass is 0.1% or less. The final mass should be obtained immediately after the heating cycle or after cooling in a desiccator. A typical standard deviation for single-operator measurements is about 1% moisture (ASTM D 4643).

Table 1. Test specimen masses for determining moisture content by standard microwave (after ASTM D 4643).

Largest Sieve with a Cumulative Percent Retained of Not More than About 10%	Recommended Mass of Moist Specimen, g
2.0 mm (No. 10)	100 to 200
4.75 mm (No. 4)	300 to 500
19 mm (3/8 in.)	500 to 1000

Similar to the microwave method, the direct heating method for determining moisture content cannot be described as a precise procedure. The behavior of a soil when subjected to direct heating depends on its mineralogical composition, so the appropriate number and duration of heating increments between mass measurements will vary from soil to soil. Also, similar to the microwave procedure, the soil sample is repeatedly stirred, heated, and weighed until two consecutive mass determinations for the dry soil change by 0.1% or less. Recommended sample masses are shown in Table 2. There are no precision statements for this test method.

Table 2. Test specimen masses for determining moisture content by direct heating (after ASTM D 4959).

Largest Sieve with a Cumulative Percent Retained of Not More than About 10%	Recommended Mass of Moist Specimen, g
2.0 mm (No. 10)	200 to 300
4.75 mm (No. 4)	300 to 500
19 mm (3/8 in.)	500 to 1000

Comparative tests

A small laboratory experiment was conducted to validate that the standard microwave test could provide accurate moisture content measurements for a wide range of soil types and moisture contents. The experiment included three soil types: silty sand, silt, and high plasticity clay. According to the Unified Soil Classification System (USCS), ASTM D 2487, these soils classified as silty sand (SM), lean silt (ML), and high plasticity clay (CH), respectively. Moisture contents ranged from approximately 1% to 60%. Each soil was prepared at three different moisture contents, and they were compacted into plastic-lined boxes using an asphalt Marshall hammer and 4-in. loose lifts (Photo 8). The compacted soil was sealed and left alone for 2 days, thus allowing the moisture within the soil to "equilibrate" (Photo 9).

The coefficients of variation between replicate moisture measurements for each of the drying methods are listed in Table 3. Both drying methods showed good repeatability with coefficients of variation less than 7%. The one exception was microwave testing at the absolute lowest moisture content (average moisture = 1%). Finding relatively high coefficients of variation for relatively low mean values is not unusual because the mean value assumes the denominator in this calculation.

$$\text{coefficient of variation} = \frac{s}{\bar{x}} \cdot 100\% \tag{2}$$

where:

s = sample standard deviation
$\bar{x}$ = sample mean.

Photo 8. Compacting soil for the moisture content tests.

Photo 9. Wet soils are sealed for the equilibration period.

Table 3. Soil densities and coefficients of variation for moisture content tests.

USCS Soil Type	Box	Dry Density[a] pcf	Moisture Content, %	Coefficient of Variation Between Replicates, %	
				Convection Oven	Microwave Oven
SM	1	110.6	1.0	6.0	13.3
	2	115.2	5.4	1.1	2.2
	3	129.0	7.7	1.3	3.4
ML	1	99.8	6.5	2.3	1.5
	2	99.3	14.1	1.4	1.1
	3	100.7	18.8	1.6	1.5
CH	1	no data	29.5	2.7	4.7
	2	73.2	37.4	2.3	1.4
	3	61.9	61.7	6.1	6.2

[a] Measured with a Troxler nuclear density gauge.

The SM and ML soils were compacted to dry densities that would be reasonable for pavement engineering (Table 3). The CH soil had relatively low dry densities in boxes 2 and 3 because the moisture contents were well above optimum. The moisture content for box 1 was closer to optimum, so its dry density would have been higher. However, density was not measured for that particular box.

The measured moisture contents are shown in Figure 1. Each bar represents the average of three replicates. The difference between moisture determinations from the microwave and the convection oven was generally less than 1%. The exceptions were the two highest moisture contents for the CH, and these high moisture contents are outside the normal construction range. For each soil, the two drying methods were compared over the three moisture contents using a paired *t*-test (Freund and Wilson 1993). The two drying methods were paired for each moisture content. For each soil type, the method of drying was found to have no significant influence on the measured moisture. The calculated *p*-values for SM, ML, and CH were 0.67, 0.53, and 0.44, respectively. A *p*-value gives the probability of being incorrect if the two drying methods are stated as providing different results. For engineering applications, a paired *t*-test is considered to have found a significant difference between test methods if the *p*-value is 0.05 or less.

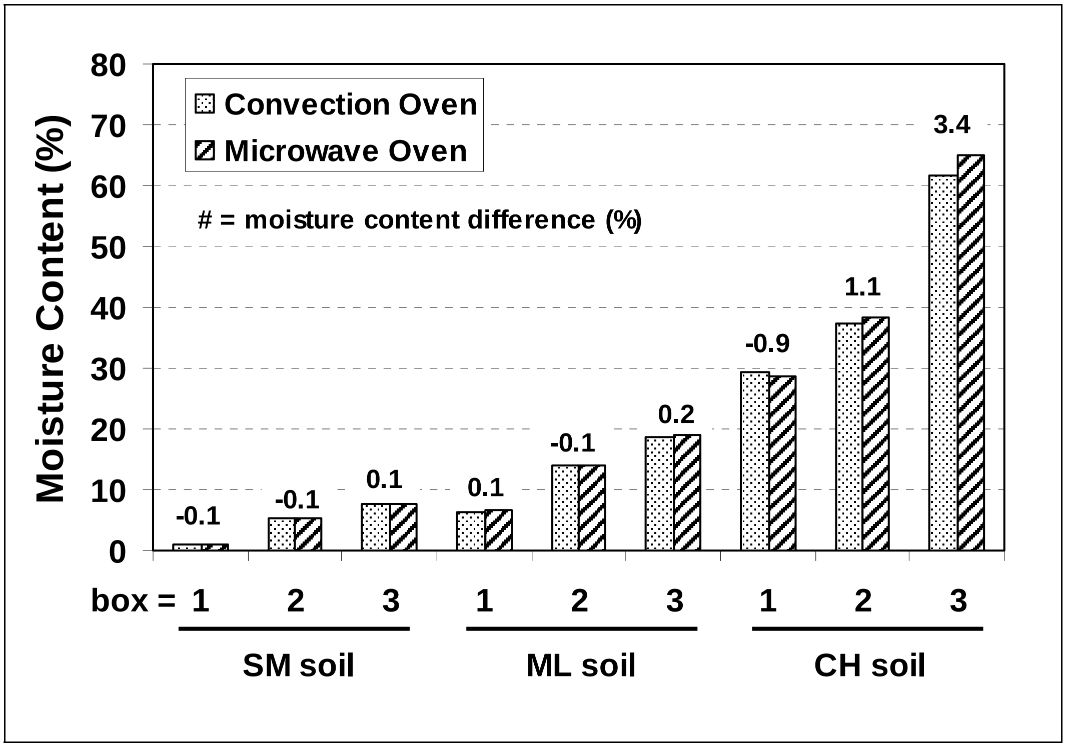

Figure 1. Average measured moisture contents.

3 Measuring In-Place Soil Density

Test devices considered

The devices that were considered for density testing included

- Nuclear density gauge
- Electrical density gauge
- Sand cone
- Rubber balloon
- Drive cylinder
- Alternative volume replacement methods

For estimating the density of soil in pavement engineering, nuclear density gauges are used in accordance with ASTM D 2922, "Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)." This test method covers the determination of the total or wet density of soil and soil-rock mixtures by the attenuation of gamma radiation where the source and detector(s) remain on the surface (Backscatter Method) or the source or detector is placed at a known depth up to 300 mm (12 in.) while the detector(s) or source remains on the surface (Direct Transmission Method). The density in mass per unit volume of the material underneath the testing device is determined by comparing the detected rate of gamma radiation with previously established calibration data (ASTM D 2922).

Electrical (or non-nuclear) density gauges have emerged as potential substitutes for nuclear density gauges. Two such gauges were evaluated as part of this study: the Electrical Density Gauge™ (EDG) marketed by Humboldt Manufacturing Company (Photo 10) and the Moisture + Density Indicator (M+DI) marketed by Durham Geo Slope Indicator (Photo 11). Both devices were demonstrated for use at the ERDC in Vicksburg, MS. The principle of operation for EDG involves measuring the electrical dielectric properties of compacted soil, including its moisture, using high radio frequency between a pair of electrodes (GENEQ 2007). Four electrodes are driven into the ground so that four readings can be obtained: two directions between each opposing pair. The standardized ASTM test method for this procedure was still under development at the time of this study. The M+DI gauge uses time domain reflectometry to measure the

Photo 10. Electrical density gauge marketed
by Humboldt Manufacturing Company.

Photo 11. Electrical density gauge marketed by Durham Geo Slope Indicator.

travel time of an electromagnetic step pulse traveling through four spikes that have been driven into the ground (Durham Geo Slope Indicator 2007). This procedure is in accordance with ASTM D 6780, "Water Content and Density of Soil in Place by Time Domain Reflectometry (TDR)."

The sand cone test is conducted in accordance with ASTM D 1556, "Density and Unit Weight of Soil in Place by the Sand-Cone Method." This test relies on the principle of volume replacement where soil is removed from a hole. The mass and moisture content of the soil is then measured, and the volume of the hole is estimated by filling it with standard sand. The typical hole size has a diameter of approximately 6 in.; however, smaller and larger sand cones are available to accommodate smaller and larger hole sizes, respectively. The sand is provided in a container and is dropped from a standard height that is established by a funnel (Photo 12). The sand is standardized to ensure that it flows easily from the container and that it conforms to the shape of the hole. The sand must be clean, dry, uniform in density and grading, uncemented, and durable (ASTM D 1556). Rounded sand is preferred to ensure that it is free-flowing. The sand must conform to the following grading to ensure that it is not susceptible to segregation during handling or clumping in humid environments (ASTM D 1556):

Photo 12. Sand cone.

- Maximum particle size less than 2.0 mm (No. 10) sieve
- Less than 3% by mass passing the 0.25 mm (No. 60) sieve
- Uniformity coefficient (C_u) less than 2.0, where:

$$C_u = D_{60}/D_{10}$$

D_{60} = nominal particle size at which 60% of the aggregate is finer
D_{10} = nominal particle size at which 10% of the aggregate is finer.

The rubber balloon test (Photo 13) is conducted in accordance with ASTM D 2167, "Density and Unit Weight of Soil in Place by the Rubber Balloon Method." This is another test that relies on the principle of volume replacement. In this case, the volume of the hole is estimated by filling it with water. The necessary water is contained within a reservoir on the rubber balloon device and a flexible, thin membrane balloon allows the water to fill the hole in the ground. A small pressure (typically 5 psi) is supplied to the water reservoir to ensure that the balloon assumes the shape of the hole. The recommended minimum volume of test holes for soils with 1/2-in. maximum particle size is 0.05 ft^3, which can be accomplished with either a 4-in.-diam hole that is 8 in. deep or a 6-in.-diam hole

Photo 13. Rubber balloon.

that is 4 in. deep. This test is not recommended for soils that contain aggregate particles with sharp edges. This test is also not recommended for soft soils that could be deformed by the 5-psi pressure (ASTM D 2167).

The drive-cylinder test method (Photo 14) is conducted in accordance with ASTM D 2937, "Density of Soil in Place by the Drive-Cylinder Method." The test method involves obtaining a relatively undisturbed soil sample by driving a thin-walled cylinder into the ground. This test method is appropriate for near-surface measurements; deeper measurements would require ASTM D 1587, "Thin-Walled Tube Sampling of Soils for Geotechnical Purposes." Once the cylinder is driven into the ground, it is removed be excavating around the cylinder, thus ensuring that the soil within the cylinder is not compressed. After trimming and cleaning the cylinder so that the only soil remaining is within the volume of the cylinder, the unit weight of soil can be determined by knowing the volume of the cylinder and the weight of the soil within the cylinder. Moisture content is then determined by drying a sample of soil that is obtained from the middle of the cylinder. Typical cylinder sizes include 2-7/8-in. diam × 3 in. tall and 3-3/4-in. diam × 5 in. tall. This test is not appropriate for sampling the following types of soils (ASTM D 2937):

Photo 14. Drive cylinder.

- Organic soils that can compress during sampling
- Very hard natural soils and heavily compacted soils that cannot be easily penetrated by the drive sampler
- Soils of low plasticity that will not be readily retained in the cylinder
- Soils that contain appreciable amounts of gravel (particles coarser than 3/16 in. or USCS Sieve No. 4)

There are simple alternatives to volume replacement techniques that could replace the sand cone and rubber balloon tests described earlier. For example, ASTM (2006) includes two procedures for pouring materials into holes in the ground in order to determine hole volume:

- ASTM D 4914, "Density of Soil and Rock in Place by the Sand Replacement Method in a Test Pit"
- ASTM D 5030, "Density of Soil and Rock in Place by the Water Replacement Method in a Test Pit"

Both of these tests are intended for excavated pits with relatively large volumes: 1 to 6 ft^3 for ASTM D 4914 and 3 to 100 ft^3 for ASTM D 5030. Pits of this size would be required when particle sizes in the soil are relatively large. ASTM D 4914 is used with maximum particle sizes of 3 to 5 in., and ASTM D 5030 is used with maximum particle sizes greater than 5 in. In both test methods, the outline of the area to be excavated is secured with a metal template. Although the materials in JRAC would not have particles of the sizes suggested for these test methods, the procedures are worth mentioning because they offer guidance on the development of alternative volume replacement techniques.

The procedure for sand replacement is summarized as follows (from ASTM D 4914):

> The ground surface at the test location is prepared and a template (metal frame) is placed and fixed into position. The volume of the space between the top of the template and ground surface is determined by filling the space with calibrated sand using a pouring device. The mass of the sand required to fill the template in place is determined and the sand removed. Material from within the boundaries of the template is excavated forming a pit. Calibrated sand is then poured into the pit and template; the mass of

> sand within the pit and the volume of the hole are determined. The wet density of the in-place material is calculated from the mass of material removed and the measured volume of the test pit. The moisture content is determined and the dry unit weight of the in-place material is calculated.

The template for this method is constructed with 1-1/2-in. angle iron and it has the shape of a square with 24-in. sides. The sand pouring device can take many forms, although the device must have a spout that will reach into a field test pit so that the drop distance from the end of the spout to the sand surface can be maintained at about 2 in. The inside diameter of the spout must also be large enough to allow free flow of the sand without clogging (ASTM D 4914). The loose unit weight of the sand is calibrated by using the designated pouring device to pour sand into a mold of known volume that has a size and shape similar to the field test pits (ASTM D 4914).

The procedure for water replacement (ASTM D 5030) is similar to the procedure for sand replacement (ASTM D 4914), except the template is round with a diameter of 6 ft. Also, since water is used as the filler, a liner is required. A suggested liner includes two plastic sheets with thicknesses of 4 to 6 mils. The amount of water used to fill the ring and pit can be measured by mass or volume. If the water is measured by mass, its temperature must be measured for proper conversion of mass to volume.

Devices eliminated from consideration

The nuclear density gauge was originally considered the most preferred gauge due to its accuracy, versatility, and ease of use. However, the nuclear gauge was eliminated from consideration for the same reasons as those presented in the section on moisture testing.

The electrical (non-nuclear) test devices were eliminated from consideration for this study for two reasons:

1. For each soil of interest, they rely on pre-calibration of electrical measurements versus traditional measurements of moisture and density.
2. Their use required substantial training and efficient sharing of data, which would become tedious for multiple military groups and frequent personnel changeovers.

Essentially, these devices rely on comparisons between electrical measurements in the field with measurements obtained for similar soil samples for which moisture and density were estimated by traditional methods. The EDG relies primarily on an electronic database that contains comparisons between previous field measurements of soil moisture and density, that is, EDG data versus nuclear or sand cone data. The M+DI method recommends prior calibration of the device for each specific soil using laboratory compaction molds. Finally, each of these tests involves driving four probes into the soil. It is the opinion of the authors that the act of driving these probes can affect density measurements, and the magnitude of these effects would change with soil type, moisture, and density.

The sand cone has shown to be more precise and accurate for estimating dry density of soil, as compared to the nuclear gauge, rubber balloon, and drive cylinder (Coleman 1988). However, the sand cone was eliminated from consideration for JRAC operations for the following reasons:

1. It is a time-consuming test, and its proper execution would require a relatively high degree of training and practice.
2. Calibrating the loose unit weight of sand requires a setup that can accommodate the funnel. Typically, a 6-in. Proctor mold would be appropriate, but this equipment would not be readily available.
3. Sufficient standard sand would have to be transported to the site. Reuse of sand would require additional calibrations for loose unit weight.
4. Even standard sand is susceptible to changes in flow and loose unit weight when it gets moist, such as would be the case in humid environments.

The rubber balloon test would offer the advantages of simplicity and speed. Also, the accuracy would likely be sufficient for JRAC operations. This test was eliminated from consideration, however, because the apparatus is delicate. The apparatus used in this study presented some operational problems with the components used for changing the pressure in the water reservoir.

Similarly, the drive cylinder would offer the advantages of simplicity and speed. The drive cylinder apparatus offers the additional advantage of ruggedness. The drive cylinder was eliminated from consideration, however, because its use is restricted to a small range of soil types. The drive cylinder can only be used for fine-grained soils that have sufficient cohesiveness to remain intact in the cylinder during excavation and trimming.

Also, proper removal of the cylinders from the ground surface requires digging the soil from around the sides of the cylinder and undercutting several inches below the bottom of the cylinder (ASTM D 2937). This process is fairly destructive, leaving a hole that would be larger than sand cone or rubber balloon holes.

The alternative volume replacement techniques, as described earlier (ASTM D 4914 and D 5030), could be eliminated quickly because they are intended for large volume holes (>1 ft^3), which would only be necessary for testing soils with large particles (>3 in.). However, the concept of getting sufficient accuracy in soil density estimates via a simple method of volume replacement was considered the most appropriate test method for JRAC. The development of a specific procedure for this purpose will be described in the next section.

Development of a test procedure

The method developed for measuring density during JRAC operations involves volume replacement with stainless steel balls. The method is called the "steel shot density test." The accuracy acquired with this test was regarded as sufficient for JRAC operations where maximum dry density is estimated based on soil physical properties to an accuracy of ±5%.

At the beginning of development for this test, plastic balls were considered because they had the advantage of being lightweight, and it was expected that they could be dried in the microwave oven. Unfortunately, the surfaces of the plastic balls became rougher as the balls were repeatedly used, thus changing their ability to pack under free-fall. Also, they partially melted when they were dried in the microwave.

Stainless steel was selected as the material for the density test because these balls are resistant to corrosion, thus allowing them to be washed and reused. Also, stainless steel is still magnetic, thus allowing the use of magnets for retrieving balls from a hole. Selecting the size for the steel balls involved a compromise between small sizes that could conform to irregularities in soil surfaces and large sizes that would be easier to handle. The balls had to be uniform in size in order to leave no opportunity for changing gradations, which could occur with segregation. For similar reasons, the sand used in the sand cone test must meet uniformity requirements (as presented earlier). The optimum diameter for the stainless steel balls was identified as 3/16 in. (4.8 mm).

The prescribed stainless steel balls for this test method are American Iron and Steel Institute Type 440C, as specified by ASTM A 276, "Stainless Steel Bars and Shapes." The stainless steel has a yield strength of 275,000 psi and a modulus of elasticity of 29,000,000 psi. The stainless steel has a density of 480 pcf (specific gravity = 7.69) and minimum Rockwell hardness of 58 HRC (Brinell hardness = 285), ASTM A 370, "Mechanical Testing of Steel Products." The balls used for this investigation were purchased from National Precision Ball Group of Mechatronics Corporation, located in Preston, WA, at a cost of approximately $13 per 1,000 balls.

Although the weight of these balls was a disadvantage for their transportability, these balls offered two critical advantages:

1. Reuse without recalibration of unit weight
2. Consistent packing during free-fall

The tendency for these balls ("steel shot") to pack consistently was verified with three small experiments. For the first experiment, a large metal cup (5-3/4 in. tall with tapered walls, approximately 4 in. wide at bottom and 5 in. wide at top) was filled by various techniques with both steel shot and standard 20-30 sand (ASTM C 778) (Photo 15). The sand met the requirements for the sand cone density test (ASTM D 1556). The volume of the cup (1206.4 mL) was measured by filling it with water and knowing the unit weight of water for a given temperature. Then, the cup was filled by three methods:

1. Pouring from the top of the cup
2. Pouring from 6 in. above the cup
3. Fill and refill while vibrating for 3 min using a vibrating table

When poured from near the top of the cup, the bulk unit weights for the sand and steel shot were 97.2 and 289 pcf, respectively. The increases in bulk unit weights, caused by higher drop and vibration, are shown in Figure 2. Relative to the standard 20-30 sand, the bulk unit weight of the steel shot was found to be much less sensitive to changes in the method of placement.

Photo 15. Filling metal container with steel shot.

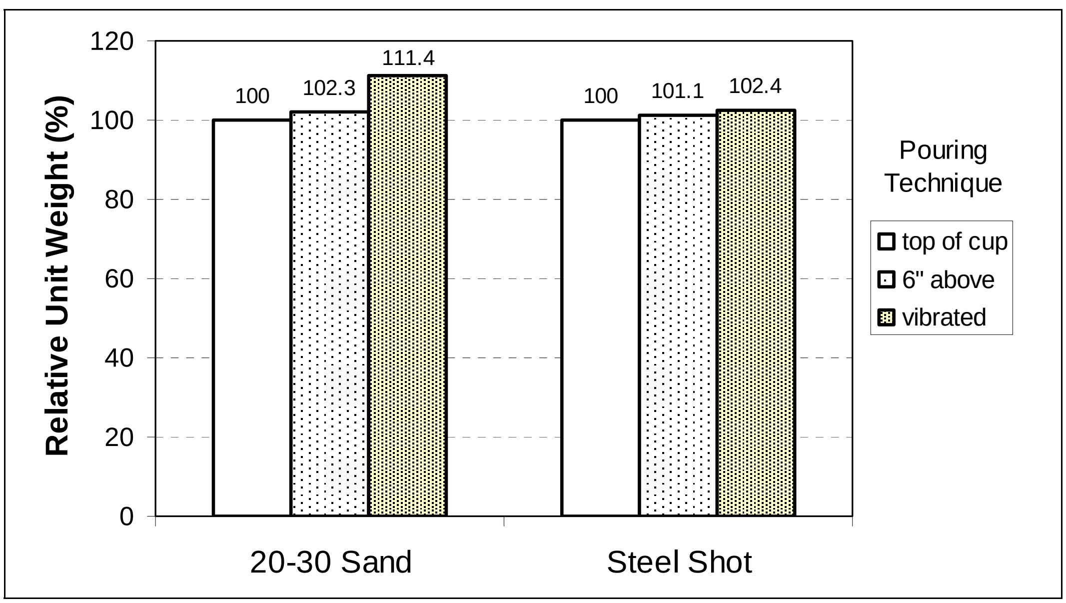

Figure 2. Relative final unit weights for different filling procedures.

When estimating the bulk unit weight that could be expected in the field, the authors were concerned that the bouncing action of the balls falling into the metal cylinder might affect this estimate. Therefore, the authors conducted a second small experiment using the same small metal scoop to pour the steel shot into plastic molds of various sizes, including a 3- by 6-in. plastic cylinder mold for concrete and a larger plastic beaker, which

had a volume similar to that which would be required for the sand cone density test (approximately 0.05 ft^3 according to ASTM D 1556) (Photo 16). The volume of the cylinder mold was 695 mL, and the volume of the beaker was 1390.1 mL, as measured by filling them with water and knowing the unit weight of water for a given temperature. The bulk unit weight of steel shot in these containers was consistently found to be 283 to 285 pcf, so the default bulk unit weight for this stainless steel shot as it is poured into a hole was established as 284 pcf.

Photo 16. Plastic containers for determining bulk density of the steel shot.

The third small experiment, which was conducted to confirm the consistent packing tendencies of steel shot, involved the same plastic beaker from above and a plastic graduated cylinder. The graduated cylinder, which had a measuring volume of 1000 mL (Photo 17), was chosen because it was anticipated as being helpful for the steel shot density test. The graduated cylinder was filled with steel shot by pouring the shot from a small metal scoop (Photo 18). The cylinder was filled to 1000 mL, where the measurement was identified as the top of a smooth and level surface of steel shot (Figure 3). Using two consecutively filled graduated cylinders, the beaker was filled with steel shot by pouring the shot into the beaker as if it were a hole in the ground (Photo 19). While pouring the steel shot, the open end of the graduated cylinder was held within 2 in. of the top of the beaker. After overfilling the beaker slightly, the excess steel shot was removed by screeding (Photo 20). The volume of the beaker was then estimated by the bulk volume of steel shot that was used to fill it, where bulk volume was determined from the graduated cylinder measurements

Photo 17. Plastic graduated cylinder with 1000-mL capacity.

Photo 18. Filling the graduated cylinder
with steel shot.

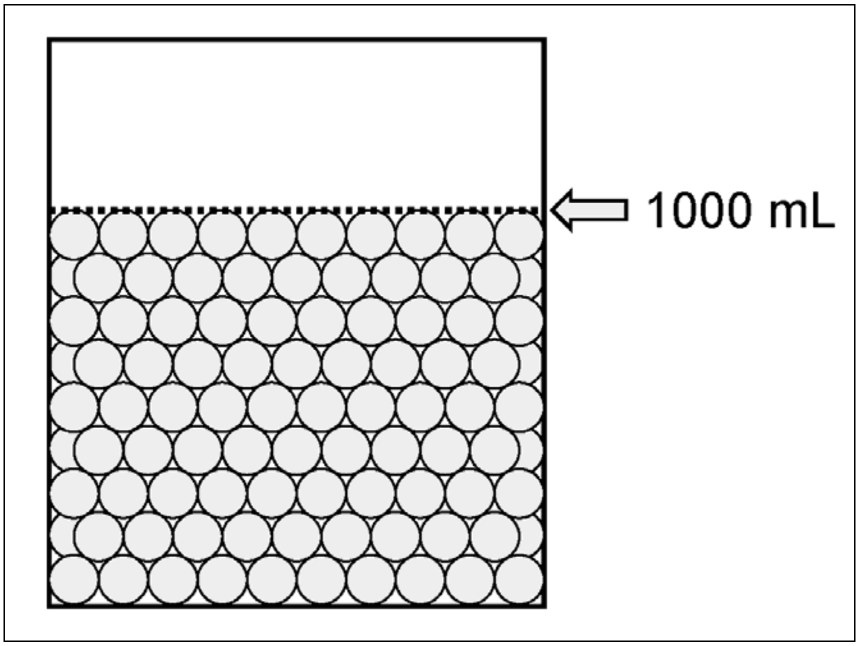

Figure 3. Schematic drawing of the graduated cylinder filled with balls up to the 1000-mL mark.

Photo 19. Pouring balls from graduated cylinder into beaker with known volume.

Photo 20. Screeding off beaker after overfilling slightly.

(Photo 21). The total mass of steel shot that was required to fill the beaker was also measured to allow the calculation of bulk unit weight of steel shot. This process was conducted four times. Results are summarized in Table 4. The volume of the beaker was estimated within 1.5% of the volume estimated by water replacement. The bulk unit weight of the steel shot was estimated within 0.5% of the default value proposed earlier.

Photo 21. Beaker filled with steel shot.

Table 4. Measured beaker volumes and measured bulk unit weights of steel shot.

Replicate	Volume of Beaker,[a] mL	Percent Error[b]	Bulk Unit Weight of Steel Shot,[c] pcf	Percent Error[d]
1	1410	1.4	283.5	-0.2
2	1400	0.7	284.0	0.0
3	1400	0.7	285.1	0.4
4	1400	0.7	284.7	0.2

[a] Determined by steel shot.
[b] Relative to the beaker volume measured by water replacement (1390.1 mL).
[c] Measured in the beaker after pouring from the graduated cylinder.
[d] Relative to the bulk unit weight of 284 pcf, proposed earlier as the default value.

This experiment showed that the steel shot packed within the graduated cylinder to a bulk unit weight that was within approximately 1% of the bulk unit weight attained in a beaker with a volume and shape similar to the hole that would be dug for a density test. Therefore, the graduated cylinder could be incorporated into the test method without complications. Also, the volume assumed by steel shot could be determined directly from graduated cylinder measurements, rather than by estimating the weight of steel shot used to fill a hole and dividing that weight by a predetermined bulk unit weight, as is done in the sand cone method (ASTM D 1556).

The steel shot density test is presented as a step-by-step procedure in Appendix B.

Comparative tests

To validate the ability for the steel shot test to provide accurate density measurements, comparative density tests were conducted on multiple test sections during March–May 2005. The test sections constructed during this time were built primarily with SM surfaces. However, there was a single test section with an available CL layer (Table 5). The table includes percent differences between the densities measured by the steel shot test and a standard test. The sand cone is the preferred standard test, so the comparison is between steel shot and sand cone when possible. However, the Troxler nuclear gauge showed good agreement with the sand cone when testing the SM soil, so the sand cone was not used on several test sections. In these cases, the percent difference in Table 5 was calculated between the steel shot test and the nuclear gauge. Percent differences are all less than 2%, and they are generally 1% or less. Also, given that the percent differences were evenly distributed between positive and negative

Table 5. Dry densities measured by the steel shot method versus sand cone or nuclear gauge.

Soil USCS[a]	Date	Dry Density, lb/ft³				Percent Difference[b]
		Nuclear Gauge	Sand Cone	Rubber Balloon	Steel Shot	
SM	17 Mar 05	130.5	131.2	131.6	132.3	0.9
		130.7	131.0	No data	No data	No data
SM	30 Mar 05	128.8	No data	No data	131.1	1.8
		129.4	No data	No data	130.1	0.5
SM	04 Apr 05	130.7	No data	134.4	130.5	-0.2
		129.0	No data	133.5	128.7	-0.2
SM	05 May 05	133.1	No data	134.1	131.4	-1.3
		131.4	No data	132.5	130.1	-1.0
SM	17 May 05	124.2	No data	128.4	125.5	1.0
		124.3	No data	127.1	125.0	0.6
		123.9	No data	128.2	126.0	1.7
CL	20 Apr 05	No data	102.4	100.5	103.8	1.4
		No data	102.8	102.9	101.9	-0.9
		No data	99.6	96.0	98.1	-1.5
		No data	102.9	102.8	103.8	0.9
		No data	101.1	97.3	100.5	-0.6

[a] Unified Soil Classification System (ASTM D 2487).
[b] Steel shot versus sand cone or nuclear gauge (sand cone preferred if available).

values, the steel shot test does not appear to impose a bias in its measurement of density. Given that the target density (maximum dry density) is estimated to within 5 pcf for the JRAC scenario, these experiments validate that the steel shot test offers sufficient accuracy.

The rubber balloon tests are included in Table 5 as confirmation that this test shows more error relative to the standard tests (sand cone and nuclear gauge), as compared to the steel shot test. The drive cylinder was also used on a few of the test sections. The drive cylinder worked well for the lean clay (CL) soil, but the SM soil did not have sufficient cohesiveness to stay in the cylinder during the extraction, trimming, and weighing process.

4 Estimating Soil Strength

Test devices considered

For the purposes of this report, soil strength is considered to be California bearing ratio (CBR). The CBR for a soil can be measured directly using the field CBR apparatus or it can be estimated from either penetration resistance or soil stiffness values. Due to difficulties with soil stiffness measurements, which will be explained in the next paragraph, the equipment for estimating strength was narrowed down to

- Direct measurement by the field CBR test
- Indirect measurement by a static cone penetrometer
- Indirect measurement by the dynamic cone penetrometer

Field measurements of soil stiffness include both static, high-amplitude measurements, and high-frequency/low-amplitude measurements. The static tests involve prohibitively tedious, large-scale equipment, such as that used by the static plate load test (ASTM D 1196). In contrast, devices that are used for high-frequency/low-amplitude measurements, such as the Humboldt GeoGauge, are portable. However, based on a study by Phillips (2005), the authors decided that correlation relationships between high-frequency soil stiffness and CBR were insufficiently accurate and/or troublesome. Phillips (2005) showed that useful relationships between soil stiffness and CBR could only be accomplished for fine-grained soils having CBR values less than about 10. Phillips (2005) also found that some of the equipment intended for measuring surface modulus was heavily dependent on having a smooth soil surface.

The first of the soil strength tests to be considered, the field CBR test, is conducted in accordance with CRD-C 654 (USACE 1995). The CBR test has the obvious advantage of being a direct measurement; that is, CBR would be measured using the equipment with which the strength parameter was originally developed (Photo 22). The CBR test begins by obtaining a flat testing surface and applying surcharge weights, which consist of steel annular disks that will surround the CBR test piston. The surcharge weights are 10 in. in diameter, and some have a slot removed so they can be added with the piston in-place. A 5-lb weight is used during piston seating, and then additional weights are added to a total of at least 10 lb.

Photo 22. Field California bearing ratio test.

Higher total weights may be used to simulate eventual pavement overburden. The CBR test is performed by pushing a steel cylindrical piston into the ground at a rate of 0.05 in. per minute. The piston has a diameter of 1.95 in. and a cross-sectional area if 3 in.^{2} Load readings are obtained at least at the following depths of penetration (inches): 0.025, 0.050, 0.075, 0.100, 0.125, 0.150, 0.175, 0.200, and 0.300. After correcting the stress-versus-penetration curve for any upward concavity (see CRC-C 654), the applied pressures are identified for penetration values of 0.1 in. and 0.2 in. The applied pressures at 0.1 in. and 0.2 in. of penetration are divided by the standard pressures of 1000 psi and 1500 psi, respectively. The CBR values are calculated by multiplying these ratios 100 percent. The CBR is usually selected at the 0.1-in. penetration. If the ratio percentage at 0.2-in. penetration is greater, the test should be rerun. If the second test provides similar results, the CBR at 0.2-in. penetration is used.

Static cone penetrometers are pushed into the soil at a slow and steady pace (Figure 4). They have the advantage of being very rapid tests compared to the CBR test. Portable cone penetrometers are pushed into the soil manually. Common portable cone penetrometers include the trafficability cone penetrometer and the airfield cone penetrometer. The airfield penetrometer was considered for this program because it is longer than the trafficability cone penetrometer and it has a stiffer proving ring, so that it can estimate CBR for stronger soils. This device includes a straight shaft

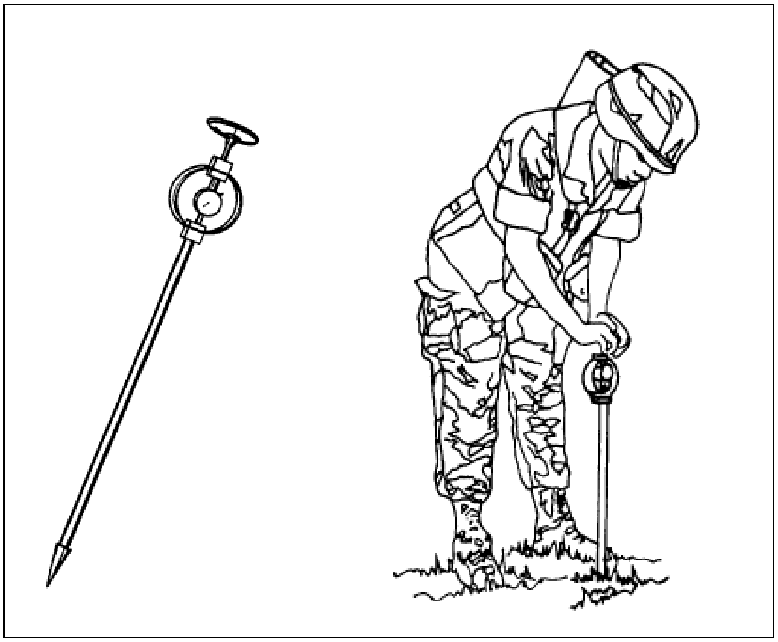

Figure 4. Cone penetrometer (DA 1994a).

with a 30-deg right circular cone, base diameter of 1/2 in., at its end. A proving ring indicator provides readings directly in terms of an airfield index (AI), which has a second-degree polynomial relationship with CBR (Figure 5).

The airfield cone penetrometer can be pushed into a soil to a depth of 24 in. or until the force required exceeds 150 lb, whichever occurs first. As the cone gets deeper, however, skin friction can affect readings, especially when the soil is clay. If soil strength measurements are needed below 24 in. or below the depth where the required force exceeds 150 lb (CBR = 18), a pit must be dug to the desired depth.

The DCP test is conducted in accordance with ASTM D 6951. The DCP is portable and rapid (Photo 23). It can estimate CBR up to 100%, and it can estimate soil strength to a depth of 28 to 37 in. below ground surface, depending on the model. The entire DCP kit with protective case and extra cone tips weighs approximately 60 lb. Similar to the static cone penetrometers, the DCP includes a shaft with a cone at its end. The angle of the cone is 60 deg, and the diameter at the base of the cone is 0.790 in., which is 0.16 in. larger than that of the rod to minimize the potential for frictional resistance along the length of the rod. Unlike the static cone penetrometers, the DCP cone is driven into the ground by impact loads that are caused by a falling weight. The weight is lifted by hand, and it slides down

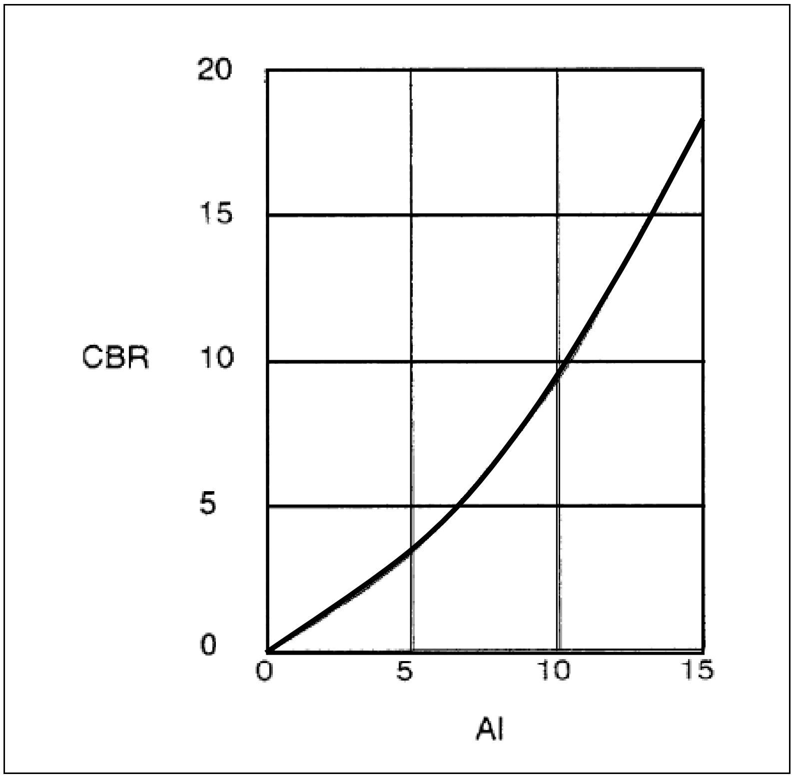

Figure 5. Predicting CBR from an airfield index (DA 1997).

Photo 23. Dynamic cone penetrometer.

a rod over a distance of about 22-1/2 in. until it impacts an anvil, which transfers the energy to the lower rod to which the cone is connected. The cones can be either removable or secured by threads. The falling weight can be adjusted to either 17.6 lb or 10.1 lb. The significance of this adjustment will be explained below.

Data collected for the DCP involves measuring the average penetration per drop (millimeters/blow) in increments of drop numbers that provide penetrations between 20 and 35 mm. Penetration is measured as a relative value from the ground surface using a ruler (Photo 23). Although many users round penetration measurements to the nearest 5 mm, it is just as quick to read to the nearest millimeter, so 1-mm precision is preferred. Converting these measurements to CBR involves relationships derived from several years of testing (Webster et al. 1992; 1994), as shown in Figure 6. The predictive equations for the three trends shown in Figure 6 follow:

1. If the soil is USCS (ASTM D 2487) CH:

$$\text{CBR (\%)} = \frac{348.3}{\text{PR}} \tag{3}$$

where PR is the penetration rate (millimeters/blow).

2. If the soil is USCS CL with CBR < 10:

$$\text{CBR (\%)} = \frac{3452}{\text{PR}^2} \tag{4}$$

3. For any other type of soil:

$$\text{CBR (\%)} = \frac{292}{\text{PR}^{1.12}} \tag{5}$$

The original investigations toward developing the predictive equations (Eq. 3, 4, and 5) were conducted with the 17.6-lb dropping weight. On soft soils, this heavy weight can cause large penetrations, however, thus reducing the resolution and accuracy of measurements. In other words, a single blow could cause a penetration of several inches, well outside the guidance for keeping the penetration between 20 and 35 mm for each blow

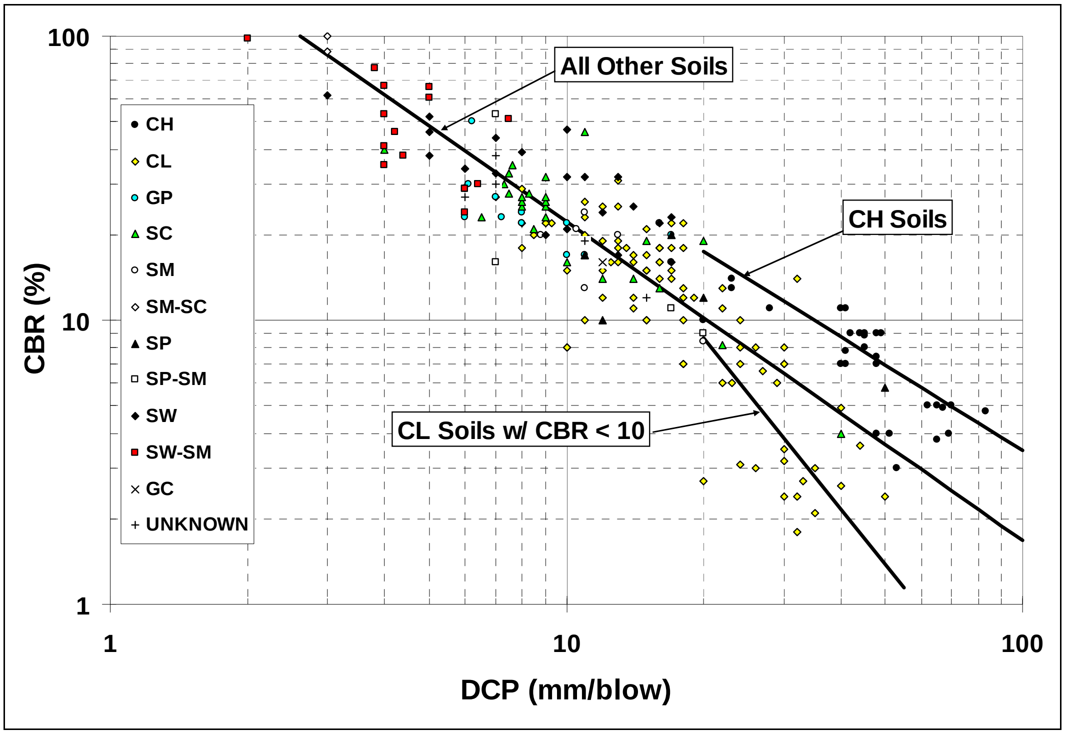

Figure 6. Conversion of DCP measurements to CBR (after Webster et al. 1994).

increment. Therefore, a 10.1-lb dropping weight was selected for weak soils. The magnitude of 10.1 lb was selected because, on the average, it caused one-half the penetration of the 17.6-lb hammer. Thus, the light hammer improved the penetration increments for weak soils, and the same predictive CBR equations could be used simply by doubling the measured penetration rate (millimeters/blow) as measured by the light hammer. Most DCP kits include both hammer weights and are called "dual-mass dynamic cone penetrometers."

Devices eliminated from consideration

The field CBR test, which would be conducted in accordance with CRD-C 654 (USACE 1995), was determined to be too time-consuming; each test requires about 30 min. Also, the test only characterizes the strength of soil at the surface. To quantify strengths at any depths below the surface, a pit would have to be dug to each depth for which an estimate of soil strength was needed.

The airfield penetrometer was found to be limited to only relatively weak soils. Given that the airfield penetrometer relies on being pushed into soil manually, it is only appropriate for clay, silt, and sand. The device is not appropriate for most hardened surfaces (i.e., cemented surface crust) or

dense sand-gravel blends. The correlation of AI to CBR is only valid up to the realistic maximum applied force of 150 lb, which corresponds to a CBR of about 18% (DA 1994b).

Recommended test procedure

The recommended test for measuring strength of soil and soil-aggregate blends in JRAC operations is the DCP, which is marketed by Kessler Soils Engineering Products, Inc. As a supplement to the earlier technical introduction to this device, the accuracy of CBR predictions will now be discussed. Predictions for CBR based on DCP measurements are compared to true measurements of CBR in Figures 6 through 8. The correlation coefficients associated with these figures are summarized in Table 6. Also included in the table are *p*-values for the correlations and coefficients of determination (R^2) for the predictive equations. The *p*-values represent the probability of being incorrect if the correlation is said to be significant. For engineering applications, a correlation is usually considered significant if the *p*-value is less than 0.05. The R^2 values can be interpreted as the proportion of variability in CBR that is explained by the DCP measurement.

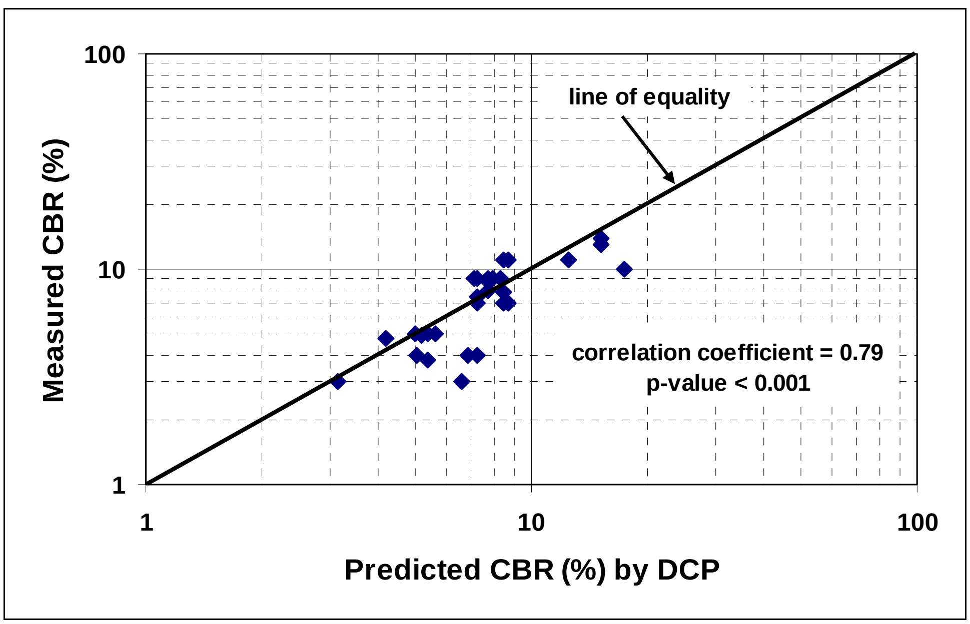

Figure 7. Comparison between predicted and measured CBR for CH soil.

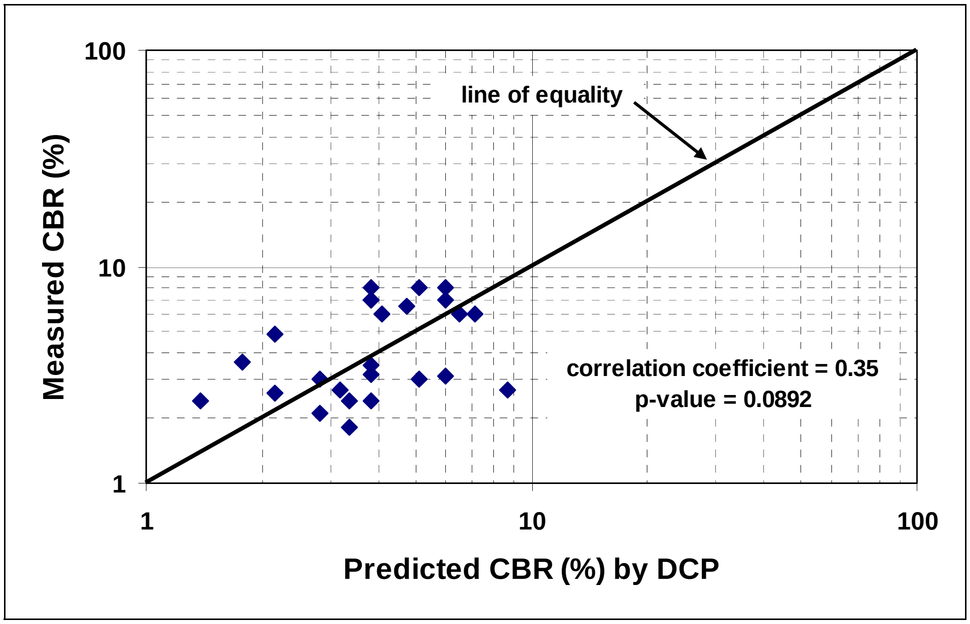

Figure 8. Comparison between predicted and measured CBR for CL soil with CBR <10.

Table 6. Correlation statistics between DCP-estimated CBR and field CBR results.

Soil Type	Correlation Coefficient, ρ	*P*-value for the Correlation	Coefficient of Determination, R^2
CH	0.79	<0.001	0.62
CL with CBR <10	0.35	0.0892	0.12
All Other Soils	0.88	<0.001	0.77

The correlations for both CH and "all other soils" were significant, and the predictive equations for DCP measurements explained 60 to 80% of the variability in true CBR. The correlation for weak CL soils, however, was not significant, and the predictive equation for DCP measurements explained only 12% of the variability in true CBR. The predictive equations were investigated in this manner to demonstrate the uncertainty associated with estimating CBR with DCP measurements. Also, if the DCP must be used in weak CL soils, the estimated CBR should be used with caution.

One of the advantages of the DCP is its ability to estimate soil strength with depth, up to a maximum depth of 28 to 37 in. below ground surface, depending on the model. Webster et al. (1994) developed general guidance for determining layers of soil thickness as a function of changing CBR. They recommend that a layer be delineated when there is an increase or decrease in CBR in excess of 25% over a depth interval of 4 in. Webster et al. (1994) reported the ability to define layer thickness to within an

accuracy of 1 in. for layers of significantly different material strength, i.e., base course over a natural subgrade (Tingle and Jersey 2007). Figures 9–11 show the delineation of layers in typical CBR profile plots (Tingle and Jersey 2007).

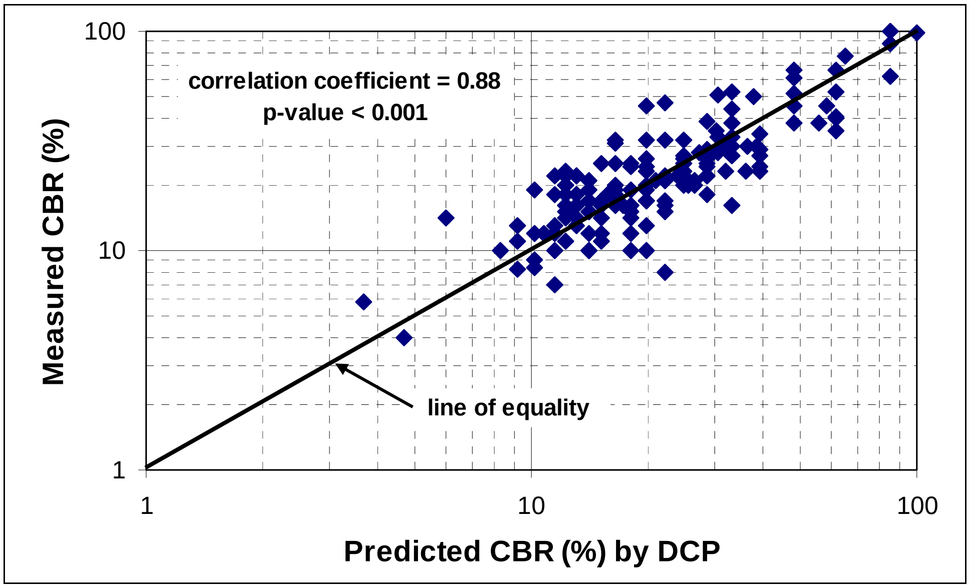

Figure 9. Comparison between predicted and measured CBR for all other soils.

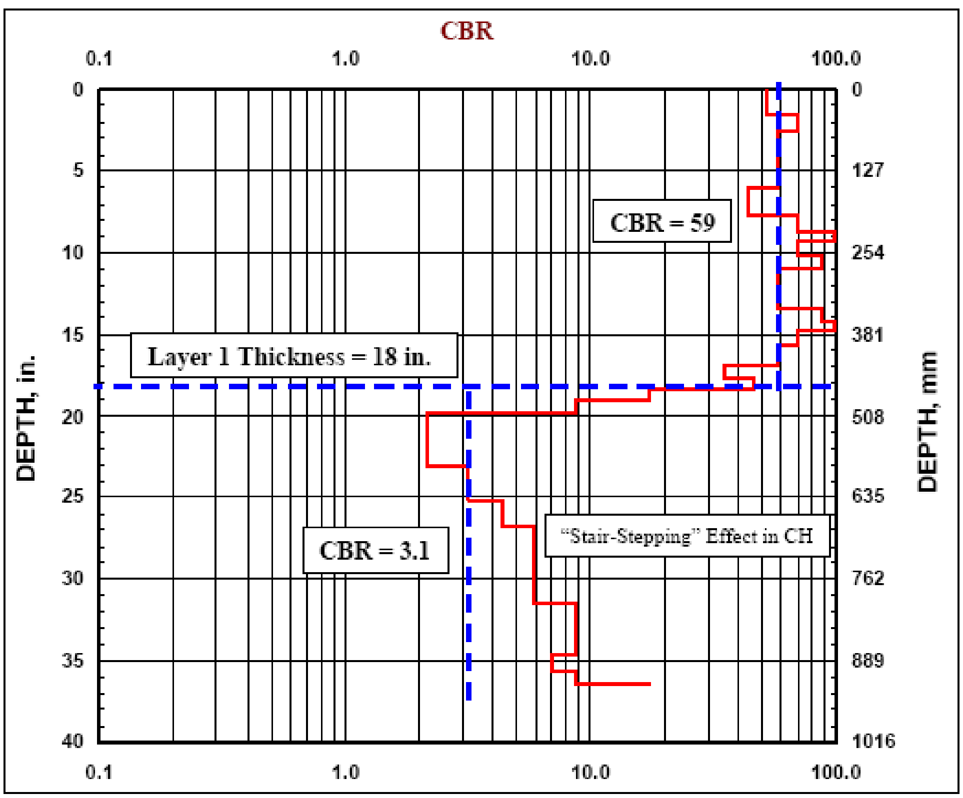

Figure 10. Thickness determinations for distinct layers (Tingle and Jersey 2007).

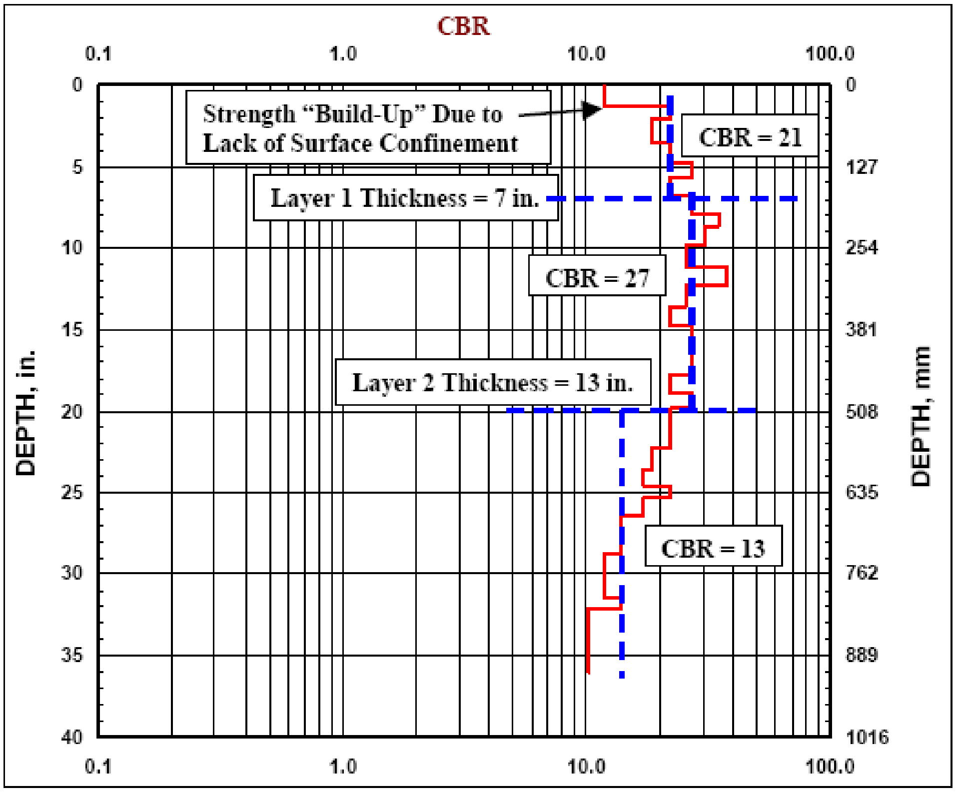

Figure 11. Thickness determinations for indistinct layers (Tingle and Jersey 2007).

5 Estimating Strength of Stabilized Layers

Test devices considered

Strength testing for stabilized layers was invariably accomplished by obtaining a stiffness measurement of stabilized soil and relating that measurement to strength. Direct strength measurements through coring, cutting beams, or pullout testing were judged to be prohibitively destructive and/or time-consuming. Also, attempts to use the DCP led to penetration refusal, that is, no measurable penetration after more than 10 blows with the heavy hammer. Refusal would prevent any capability to detect changes in strength over time. The devices that were considered for stiffness testing included

- The GeoGauge™
- The portable falling weight deflectometer (PFWD)
- The Clegg hammer

The GeoGauge (Photo 24) operates by vibration loading, in accordance with ASTM D 6758, "Standard Test Method for Measuring Stiffness and Apparent Modulus of Soil and Soil-Aggregate In-Place by an Electro-Mechanical Method." The vibratory load is applied to the soil through a single ring-shaped foot, which has an outside diameter of 4.5 in. and an inside diameter of 3.5 in. (Lenke et al. 1999). The typical load pulse is 5 to 10 ms with forces on the order of 2 lbf and displacements on the order of 0.05 mils. (Sawangsuriya et al. 2002a; 2002b). The GeoGauge™ is marketed by Humboldt Manufacturing Company.

The PFWD (Photo 25) is marketed in several alternative designs that are offered by several different manufacturers. The specific device considered for this study was the Prima 100, which is marketed by Dynatest International (Photo 25). The PFWD works on the concept of measuring soil surface deflection at the center of a falling mass; deflection is measured by a geophone that is placed within a loading plate. Although multiple configurations of the PFWD are possible, the PFWD used for this evaluation included falling weights of 22, 33, and 44 lbf. The bearing plate to be impacted by the falling weight included diameters of 5.9 and 11.8 in. Impact was accomplished via four rubber buffers. The PFWD considered in this study was capable of a peak impact load on the order of 225 to

Photo 24. Humboldt GeoGauge (after Phillips 2005).

Photo 25. Prima 100 portable falling weight deflectometer with laptop computer (after Phillips 2005).

3400 lb, with a total load pulse of 15 to 20 ms (Phillips 2005). The depth of influence for this device is up to 15 in. (Dynatest International 2004). The Prima 100 also offers an alternative for measuring multiple deflections at various offsets from center-of-mass, but this alternative was immediately eliminated from consideration due to the cumbersome setup required and the resulting delicateness of the apparatus. The use of a PFWD invariably requires either a portable computer or a handheld device for data collection (Phillips 2005). The PFWD is sufficiently bulky and heavy to require a hand truck for moving it more than a few feet.

The Clegg hammer (Photo 26) was developed in Australia in the 1970s by Dr. Baden Clegg. The Clegg hammer is marketed by more than one company in countries including England, Australia, and the United States. The device used by the authors was purchased from Lafayette Instrument Company in Lafayette, IN. The Clegg hammer operates under a similar principle as the PFWD in that its measurements are related to soil response to the impact of a falling mass. However, the Clegg hammer is available in various configurations, ranging from bulky (like the PFWD), to more compact and portable. The Clegg hammer is available with hammer masses of 45 lb (20 kg), 10 lb (4.5 kg), 5 lb (2.25 kg), and 1 lb (0.5 kg).

Photo 26. Clegg hammer (10-lb).

Similar to the PFWD, the 45-lb Clegg hammer requires the use of a hand truck. The lightest Clegg hammer (1 lb) is intended for very soft soil, such as that found in agricultural applications. The 5-lb Clegg hammer has been used for measuring shock attenuation characteristics of natural and artificial playing surfaces. The 10-lb Clegg hammer is the original size and is still considered the "standard." The 10-lb Clegg hammer was used in this study because it offered a good compromise between light and portable, yet heavy enough to have a zone of influence on the order of a compacted lift of soil. The digital readout for Clegg hammer results is conveniently attached to the hammer (Photo 26).

Devices eliminated from consideration

Although the influence depths for measurements by the GeoGauge has been shown to be on the order of 7 in. for some soils, the depth of influence decreases with soil stiffness (Nazzal 2003). This finding, as well as the fact that the GeoGauge is not recommended for soils with stiffnesses greater than approximately 30 ksi (Phillips 2005), caused the authors to eliminate the GeoGauge from consideration for estimating the strength of stabilized layers.

The authors considered the PFWD to offer much potential for quality control and assurance for pavement subgrades and base layers. However, the device was judged too bulky for the JRAC scenario. Relative to other instruments, the PFWD would require more space for shipment. Also, the use of a hand truck to maneuver around an airfield that is busy with construction equipment would be awkward. Finally, the fact that the electronics are attached to the PFWD by a cord necessitates two operators.

Recommended test procedures

The standard 10-lb Clegg hammer was selected as the tool for estimating the strength of stabilized soil layers. The Clegg hammer would accomplish this task via measurements of soil stiffness under impact. The authors also presumed that the Clegg hammer would be useful for supplementing the steel shot density test during compaction operations. The usefulness of hammer impact for monitoring soil density in compaction operations is explained in ASTM D 5874, "Determination of the Impact Value (IV) of a Soil."

In its standard configuration, the Clegg hammer is 2 in. (5 cm) in diameter and is dropped from a height of 18 in. (45 cm). As the hammer drops, it is guided through a steel tube. The output for the Clegg hammer is Clegg impact value (CIV). The CIV is actually the peak deceleration of the hammer on impact in units of 10 gravities (g) with the output below units of 10 gravities truncated (ASTM D 5874). The hammer is equipped with an accelerometer and is instrumented with a peak-hold electronic circuit. The circuitry is filtered electronically to remove unwanted frequencies (ASTM D 5874).

The standard procedure is to smooth the surface of the soil for testing. The hammer is dropped four times at the same location, and the highest reading is taken as the measurement. This procedure eliminates low values that often occur during the first couple of drops when loose particles on the surface of the soil are compacted into the bulk of the lift. The Clegg hammer model used in this study was equipped with a digital readout that retains its reading for 15 sec. If the next drop is within that time period and it gives a higher CIV, the new CIV will be displayed. If the next drop gives a lower CIV, the first CIV is retained. After 15 sec the display turns off, and the process starts over again at the next testing location. Newer models will store the test data and display the last four CIV readings and the peak reading.

The Clegg hammer used by the authors can be carried in a cloth bag, which includes a pocket for the cable that connects the hammer handle to the digital display unit. The hammer can be locked with a pin to prevent movement. The digital display and the verification ring are stored in a foam-lined plastic case. The verification ring is a rubber ring that fits inside the hammer sleeve. To verify that the hammer is working properly, the hammer is periodically dropped onto the ring while the ring is sitting on a concrete slab. The reading should be approximately the same for each of these tests. For the authors, this reading was approximately 30. Based on information supplied in ASTM D 5874, the units for this reading is $m/s^2 \times 10^2$, so a display of 30 implies a peak deceleration of 3000 m/s^2.

A single study was found in which the Clegg impact value was correlated with compressive strengths of cement-stabilized soils (Okamoto et al. 1991). Six soil types were stabilized with various amounts of cement in a laboratory setting to provide compressive strengths ranging from 5 to 940 psi, with an average of 275 psi. The Clegg hammer was performed on

blocks at curing ages ranging from 1 to 17 days. Companion strengths were measured using 2.8- by 5.6-in. cylinders (ASTM D 1632) that were compacted to the same density as the blocks and were cured in similar moist conditions. Unconfined compressive strength in units of pounds (force) per square inch is calculated from Clegg impact value as follows:

$$\log(\mathrm{UCS}) = 0.081 + 1.309 \cdot \log(\mathrm{CIV}) \qquad R^2 = 0.90 \tag{6}$$

Validation tests for predicting the strength of stabilized soil

Validation tests for using the Clegg hammer for predicting the strength of stabilized soil involved both a laboratory study and a field study. In the laboratory study, the Clegg hammer test was performed on 4-in.-diam laboratory-produced samples in order to validate or modify existing CIV versus strength relationships. In the field study, the Clegg hammer was used to monitor strength gain over a period of 10 days.

The laboratory study involved producing 4- by 6-in. samples of soil and cement-stabilized soil using a gyratory compactor (Photo 27). The compaction was designed to produce soil with density similar to that which would be attained in the field. This compaction process was the same as that used during the original development of promising stabilized soils for JRAC scenarios. In this process, samples were air-cured at 50% relative humidity and approximately 77°F. Then, at specified ages, three replicate samples were tested in unconfined compression at a loading rate of 0.1 in./min.

Photo 27. Laboratory soil sample produced with a gyratory compactor.

For the purpose of developing a CIV versus strength relationship, companion cylinders were produced for Clegg hammer testing. At each designated age for compression testing, three replicate companion cylinders were tested with the Clegg hammer. Clegg hammer testing involved placing the cylinders inside 4-in.-diam plastic concrete cylinder molds. The bottoms of the molds were removed, and the molds were cut vertically at one location (Photo 28). The molds were tightened to a snug fit around the cylinders using screw clamps, as shown in Photo 29. The snug fit was intended to represent the soil that would surround the Clegg hammer testing location in the field.

Similar to the field testing procedure, the hammer was dropped four times on the top of the cylinder, and the highest recorded CIV was used as the test result. Clegg hammer and unconfined compression tests were conducted on both a SM and a cement-stabilized SM soil. The cement-stabilized soil included 4% by mass of ASTM C 150 Type I portland cement. Ages for testing were designated as 1, 3, and 28 days.

Photo 28. Plastic concrete cylinder mold to be used for simulating confinement for the soil sample.

Photo 29. Laboratory Clegg hammer test for a confined soil sample.

The variability between replicate measurements provides an indication of repeatability for a test. Figures 12 and 13 compare the coefficient of variation for the laboratory Clegg hammer test against the coefficient of variation for the UCS test.

$$\text{coefficient of variation (\%)} = \frac{\text{standard deviation}}{\text{mean}} \cdot 100\% \qquad (7)$$

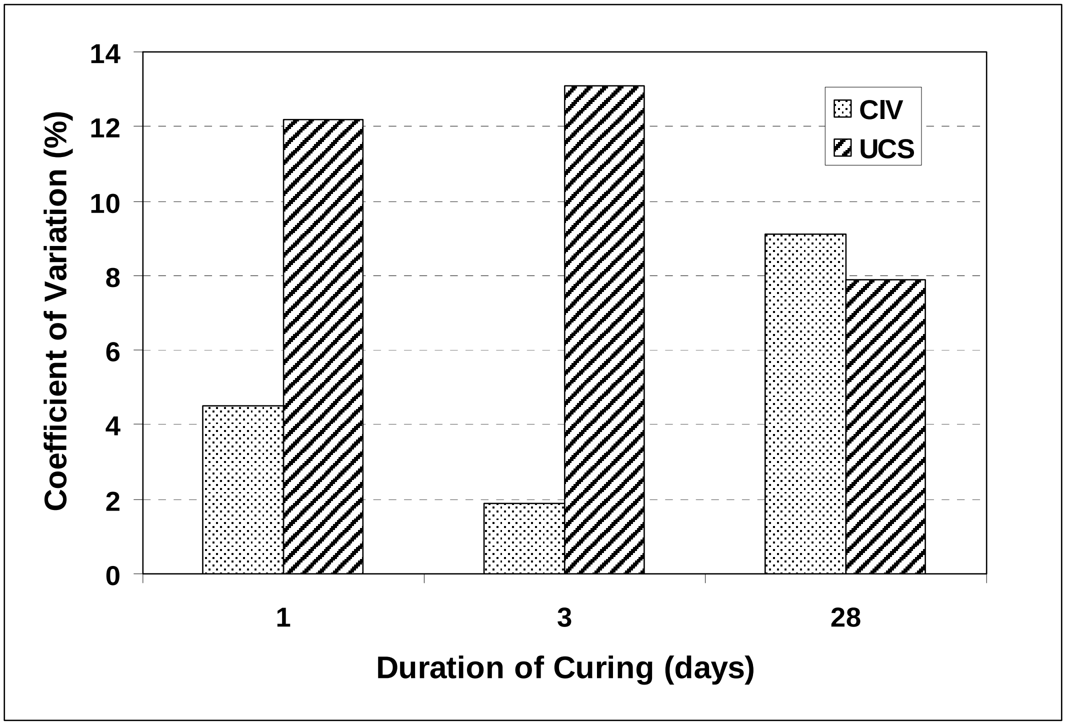

Figure 12. Test replicate variability for SM soil.

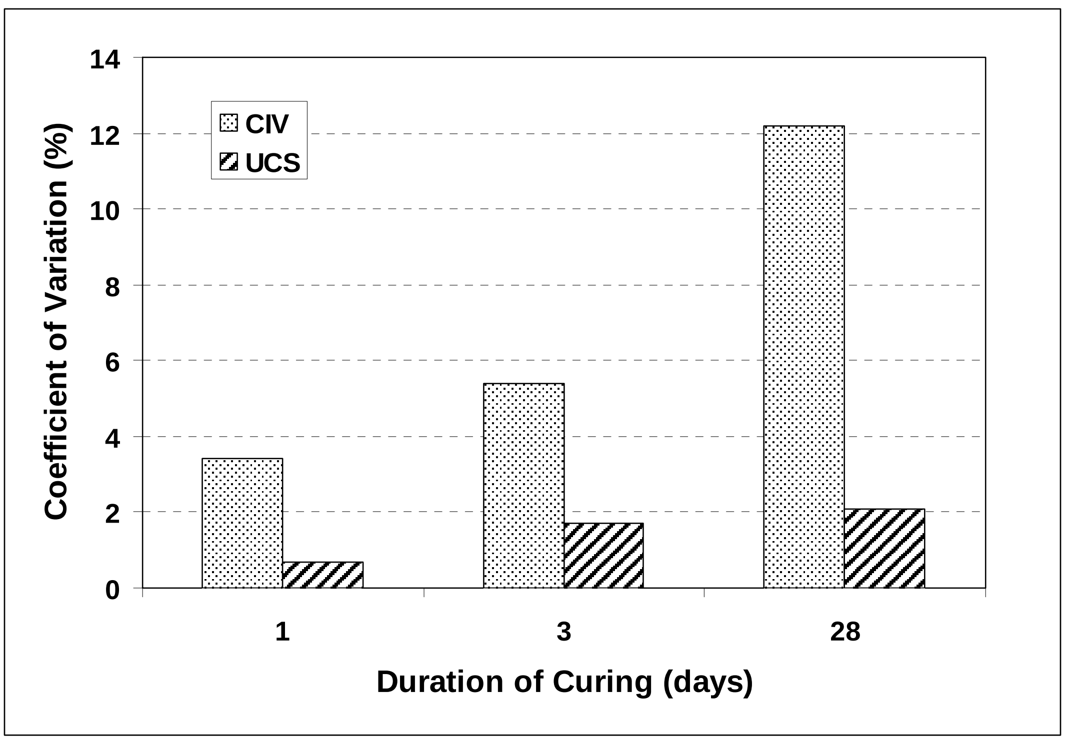

Figure 13. Test replicate variability for cement-stabilized SM soil.

Within-laboratory coefficients of variation for ASTM test methods are generally on the order of 2 to 6%. This is true for unconfined compressive strength tests for soil (ASTM D 2166), soil-cement (ASTM D 1633), and concrete (ASTM C 39). This is also true for flexural strengths of soil-cement (ASTM D 1635) and concrete (ASTM C 78).

Figures 14 and 15 show that the CIV measurements for SM soil and the UCS measurements for stabilized SM soil both conform to these typical coefficients of variation. There are two particular cases in which the tests had relatively high variability, on the order of 12 to 13%:

1. When testing UCS for low-strength (<100 psi) unstabilized SM soil
2. When conducting the Clegg hammer test for high-strength (>500 psi) cement-stabilized SM soil

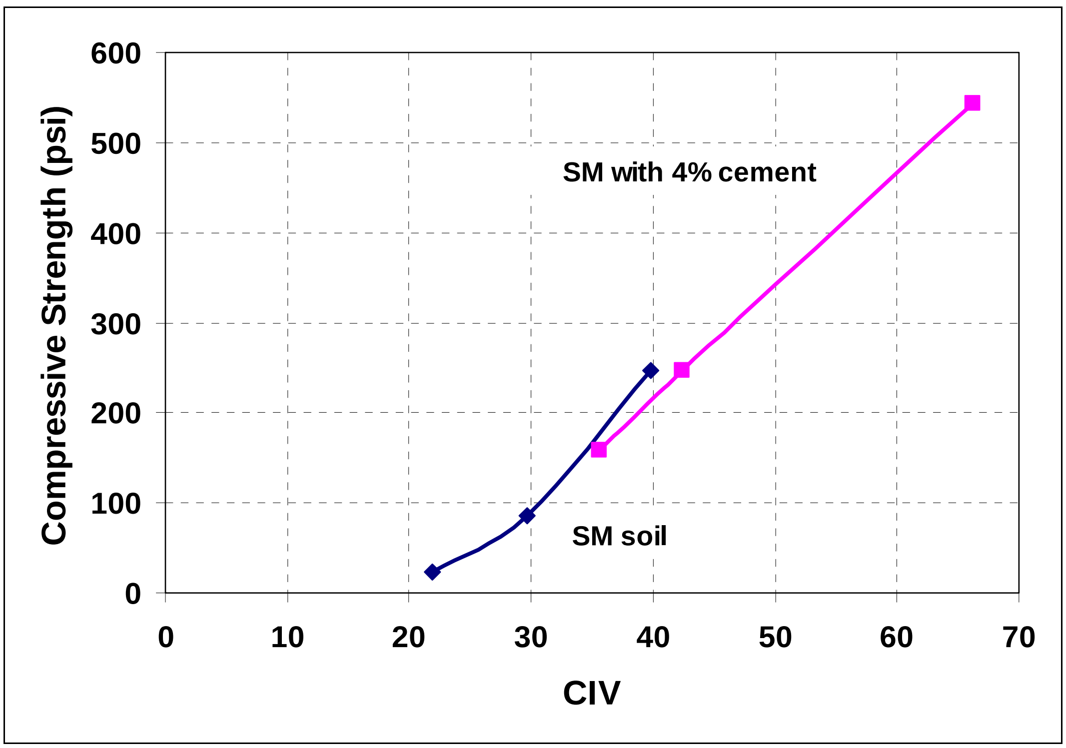

Figure 14. Measured CIVs and unconfined compressive strengths at test ages of 1, 3, and 28 days.

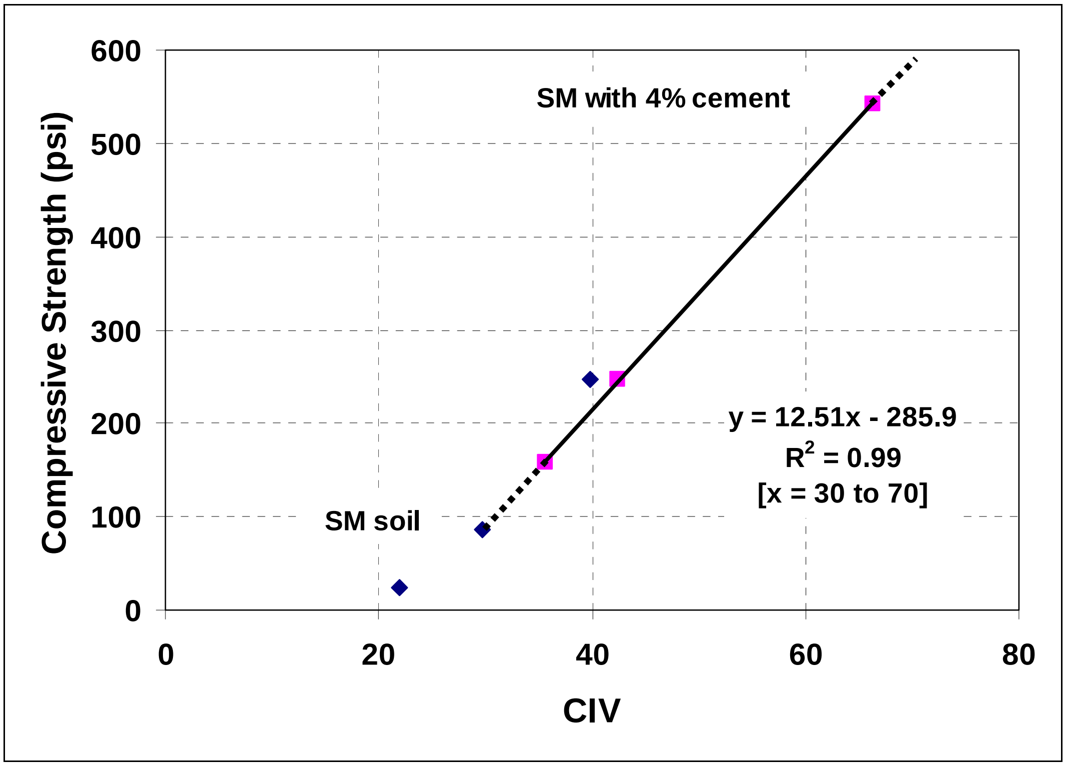

Figure 15. Predictive equation for estimating the compressive strength of cement-stabilized soil.

Users of these test methods need to be aware of these cases where variability will be relatively high. More importantly, this finding indicates that the variability of Clegg hammer tests in the field will likely be on the order of 10 to 15% for stabilized soils with strengths in excess of 500 psi. Total variability in the field would include this test variability as well as spatial variability in the true material properties.

Clegg hammer data (CIVs) are compared to unconfined compressive strengths in Figure 16. The trend is linear, and the data for SM soil fall in line with the data for cement-stabilized soil. The cement-stabilized soil data were then used to develop a linear predictive equation (Figure 16), which uses CIV as the independent variable for predicting UCS.

$$\text{UCS (psi)} = 12.51\,(\text{CIV}) - 285.9 \qquad (8)$$

The equation is considered to be valid for a range of CIVs approximately equal to the range of CIV data that were available in this study: 30 to 70. This range of CIV input predicts UCS from approximately 100 psi to approximately 600 psi.

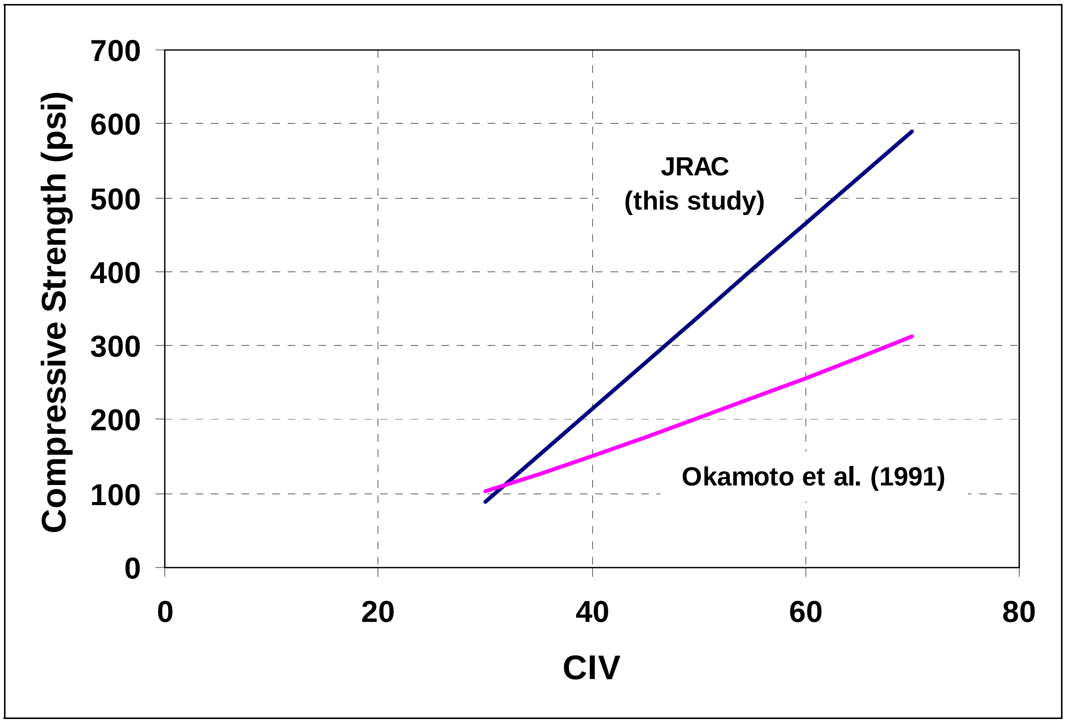

Figure 16. Comparison between equations for predicting the compressive strength of cement-stabilized soil.

In Figure 16, the predictive equation for UCS developed in this study is compared to the published equation by Okamoto et al. (1991). Okamoto et al.'s equation, which was developed using several soil types and several percentages of cement, is more conservative. The equation developed in this study involved a single soil type (SM) and a single percentage of cement (4% by mass); however, it has the advantage of having been developed for a specific stabilized soil that is likely to be encountered in a JRAC scenario. As a preliminary solution, the predictive equation developed herein will be considered the best estimate for strength. The Okamoto et al. equation will be considered a lower bound of possibilities for predicted strength.

As a precautionary note, the authors experienced a case in which the Clegg hammer became damaged. This involved testing of SM soil that had been stabilized with 7% cement and had cured outside for several months. The CIVs were in the range of 150 to 200. After approximately 20 tests, or about 80 hammer drops, the Clegg hammer stopped working because some electronics in the hammer malfunctioned. Future use of the Clegg hammer should be limited to stabilized soils that give CIV values less than 100. This corresponds to stabilized soils with unconfined compressive strengths less than about 950 psi according to Equation 8.

As further validation for using the Clegg hammer to monitor the strength gain of cement-stabilized soil, the Clegg hammer was used on two cement-stabilized airfield test sections that were constructed at the ERDC in Vicksburg, MS. Both test sections were constructed with SM soil. The first test section was stabilized with 3% Type III (ASTM C 150) portland cement, and the second test section was stabilized with a combination of 3% Type III cement and 3% Soil Sement®, which is a polymer emulsion product.

The coefficients of variation for the CIV measurements on the test sections are shown in Figure 17. Each coefficient of variation was calculated from 10 Clegg hammer tests, which were distributed around the test section area. Coefficients of variation for the cement-stabilized test section range from 10 to 15%, and coefficients of variation for the cement-polymer-stabilized soil ranged from 10 to 35% (Figure 17). All these values can be considered typical when compared with historical variability data, as summarized by Freeman and Grogan (1997). Freeman and Grogan (1997) reported coefficients of variation of 10 to 35% for field tests performed on compacted soil or compacted subbase pavement layers.

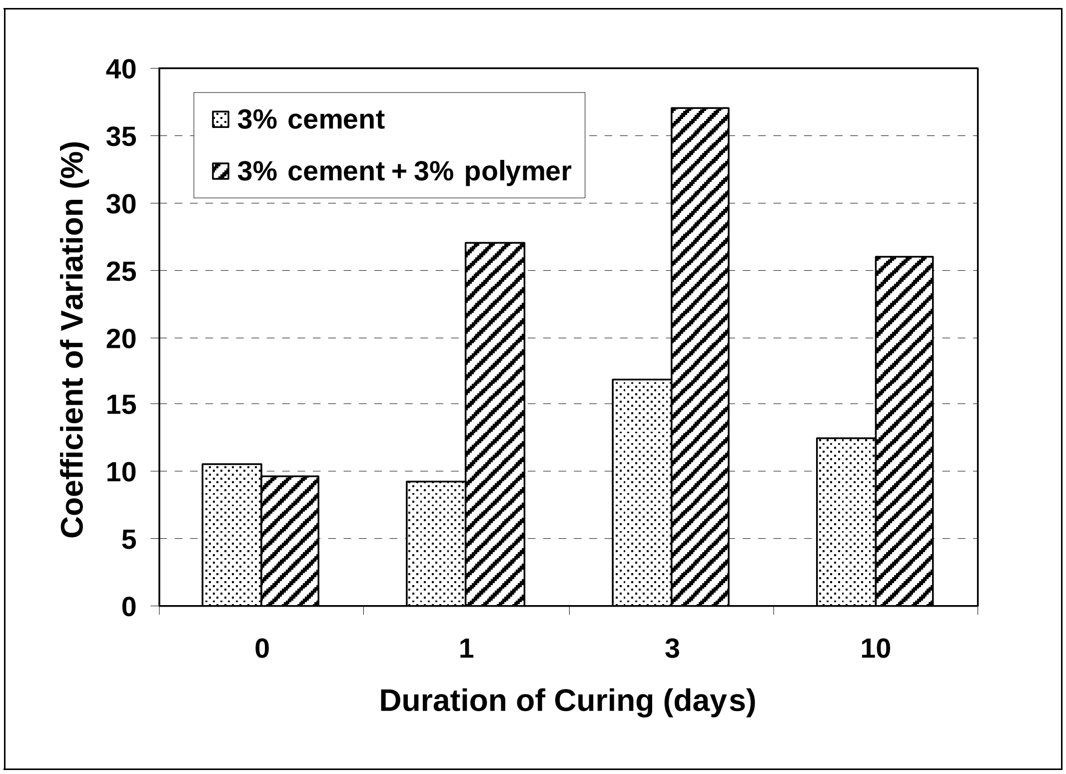

Figure 17. Variability of stabilized soil test sections based on Clegg hammer results.

Figure 17 shows that the Clegg hammer detected greater variability for the cement-polymer-stabilized soil, as compared to the cement-only stabilized soil. This greater variability would be expected because the cement-polymer-stabilization process is more complicated and is thus more susceptible to variations.

The average measured CIV values for the two test sections, over 10 days of curing, are shown in Figure 18. The test section with cement only never gained much strength. This test section was actually a pulverization study, so attaining sufficient strength was not a big concern. The test section with cement only could have had insufficient water. The test section with cement-polymer-stabilized soil, however, showed large improvements in strength through the entire 10 days. The polymer likely contributed to strength and, in addition, the polymer was supplied as an emulsion. The water provided by the emulsion likely contributed to the success of the cement.

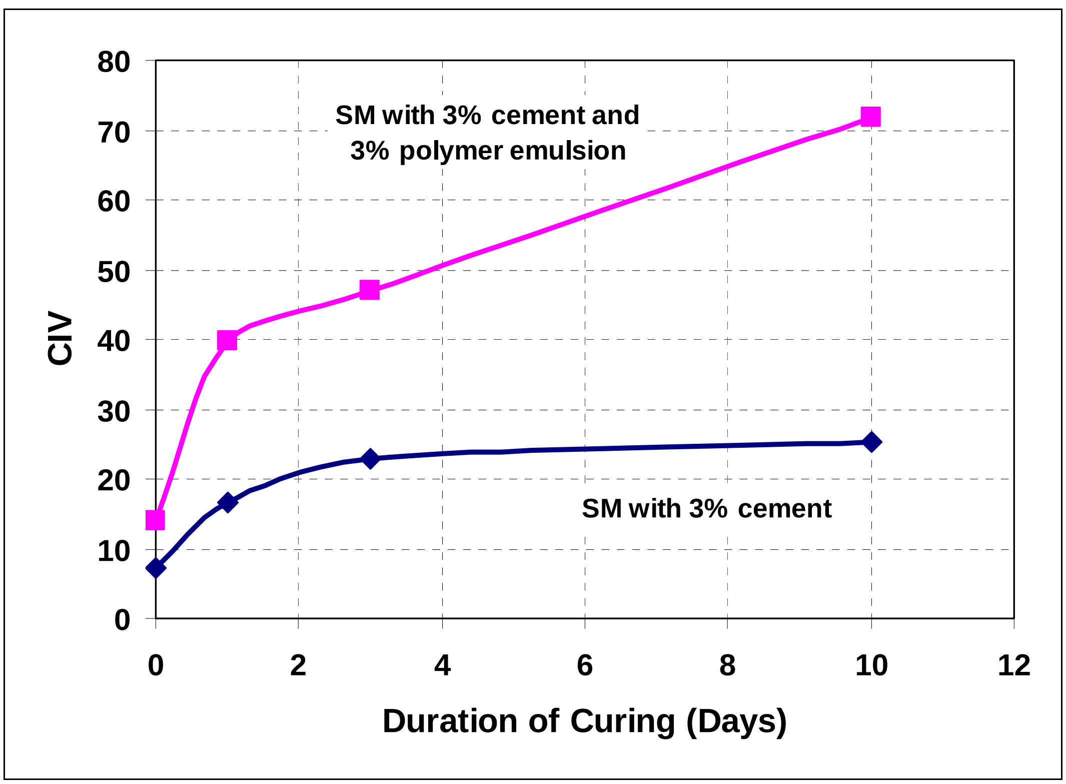

Figure 18. Strength gain over time for stabilized soil, based on Clegg hammer results.

The strength prediction relationships presented earlier were proposed as being valid for CIVs equal to or higher than 32. The test sections with cement only did not meet this requirement and would therefore be tested as a soil, as will be presented later. The cement-polymer-stabilized soil test section exceeded the CIV of 32 at 1 day. Therefore, both strength prediction equations presented earlier were applied to these CIV data (Figure 19). If a military unit were waiting for the test section to reach 200 psi, it would likely have attained that strength after 1 day and would have certainly attained that strength after 4 days. If a military unit were waiting for the test section to reach 300 psi, it would likely have attained that strength after 3 days and would have certainly attained that strength after 9 days.

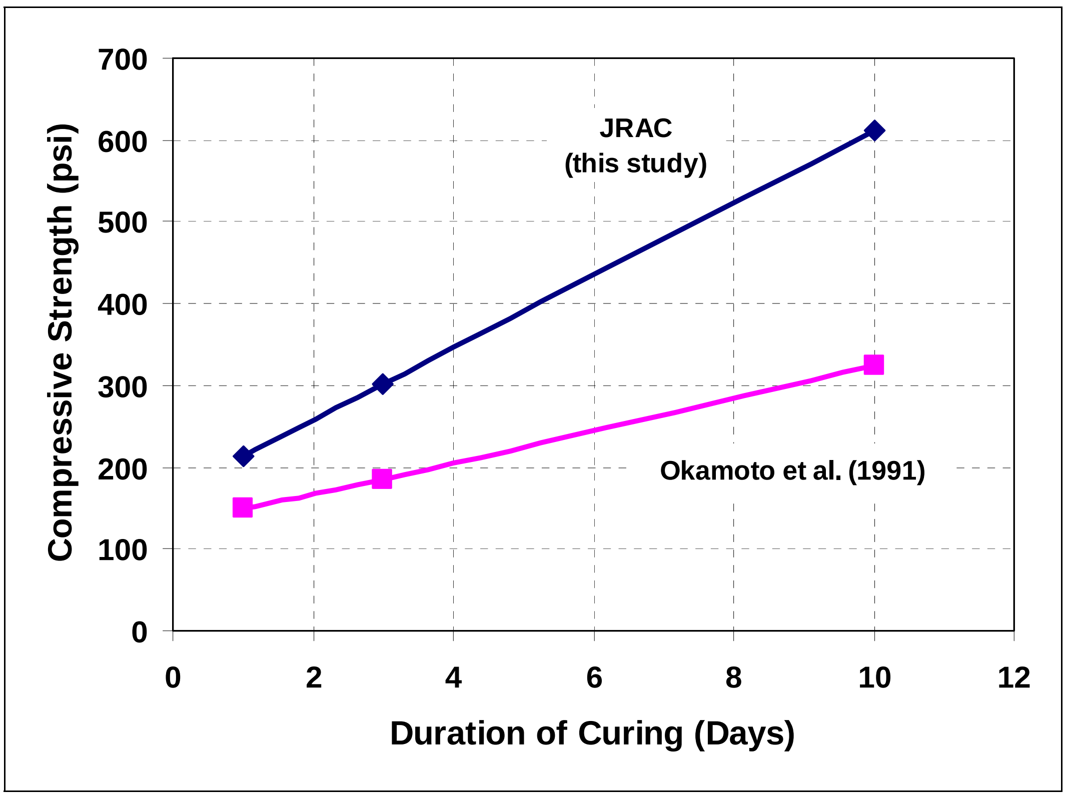

Figure 19. Predictions of compressive strength for a stabilized soil test section (cement + polymer) based on Clegg hammer results.

It must be noted that while the predictions in Figure 19 are for a soil stabilized with both portland cement and polymer, the predictive equations were originally developed on soils that were stabilized with only portland cement. The authors are assuming that the accuracies of these strength predictions for stabilized soil are not affected substantially by the addition of polymer to the stabilization process. This assumption is reasonable given that the Clegg hammer predicts strength from impact stiffness and

the majority of impact stiffness would be provided by the cement component of the stabilization. It is assumed that while the portland cement controls impact stiffness and strength, the polymer contributes more to the toughness (i.e., flexibility) of the system. These theories, however, were not proven as part of this study.

6 Additional Uses for the Clegg hammer

The Clegg hammer was originally purchased and studied for estimating the strength of stabilized soil. However, there are two additional possible uses for the Clegg hammer in the JRAC scenario that will be presented in this section. These uses include estimating CBR for unstabilized soils and monitoring compaction operations.

Estimating strength (CBR) for unstabilized soil

In the JRAC scenario, the DCP is the preferred method for estimating CBR for soil. The DCP has a long tradition of use in the U.S. Army, and it offers the great advantage of estimating strength of soil with depth. However, there is a lot of published information related to using the Clegg hammer for estimating CBR (%) for unstabilized soils. Several relationships are shown below, all taking a similar algebraic form. This section of text presents information necessary for considering the possibility of using the Clegg hammer as a backup device for estimating soil strength, albeit limited to a surface measurement.

1. Clegg (1980):

$$\begin{aligned} \mathrm{CBR} &= 0.07 \cdot (\mathrm{CIV})^{2.0} \\ R^2 &= 0.79 \end{aligned} \tag{9}$$

1. Clegg (1986):

$$\begin{aligned} \mathrm{CBR} &= \left[0.24 \cdot (\mathrm{CIV}) + 1\right]^{2.0} \\ R^2 &= 0.92 \end{aligned} \tag{10}$$

2. Mathur and Coghlans (1987):

$$\begin{aligned} \mathrm{CBR} &= 0.109 \cdot (\mathrm{CIV})^{1.86} \\ R^2 &= 0.79 \end{aligned} \tag{11}$$

3. Al-Amoudi et al. (2002):

$$\begin{aligned} \text{CBR} &= 1.35 \cdot (\text{CIV})^{1.01} \\ R^2 &= 0.85 \end{aligned} \tag{12}$$

4. Al-Amoudi et al. (2002) generalized model:

$$\begin{aligned} \text{CBR} &= 0.169 \cdot (\text{CIV})^{1.7} \\ R^2 &= 0.85 \end{aligned} \tag{13}$$

In the generalized model (Eq. 13), Al-Amoudi et al. (2002) combined their data with data from Clegg (1980) and Mathur and Coghlans (1987). The coefficient of determination (R^2) remained high, but the generalized model had the advantage of the largest number of data points, which can effectively increase R^2 (Figure 20).

A few field experiments were conducted as part of this study to validate the capability for the Clegg hammer to estimate soil strength. The Clegg hammer was performed at ten sites where the field CBR test was also conducted. Soil types included SM, SP, CL, and CH. The CIVs were converted to CBRs using the two models that were developed with the most data and experience, which are Clegg's 1986 model (Eq. 10) and Al-Amoudi et al.'s 2002 generalized model (Eq. 13). Comparisons between measured CBR and estimated CBR, based on these models, are shown in Figure 19. For the Clegg model (1986), predictions were different from the measured CBR by a factor as high as 3. For the generalized model (Al-Amoudi et al. 2002), some CBR estimates were different from the measured CBR by a factor as high as 4. While the Clegg model tended to be unconservative, the generalized model tended to be conservative. Therefore, the most accurate estimates for CBR could be obtained by averaging these two models. The average of the two models can be approximated simply by

$$\text{CBR} = 0.05 \cdot \text{CIV}^2 + 0.53 \cdot \text{CIV} \tag{14}$$

Comparisons between measured CBR and estimated CBR, based on this "combination model," are shown in Figure 21.

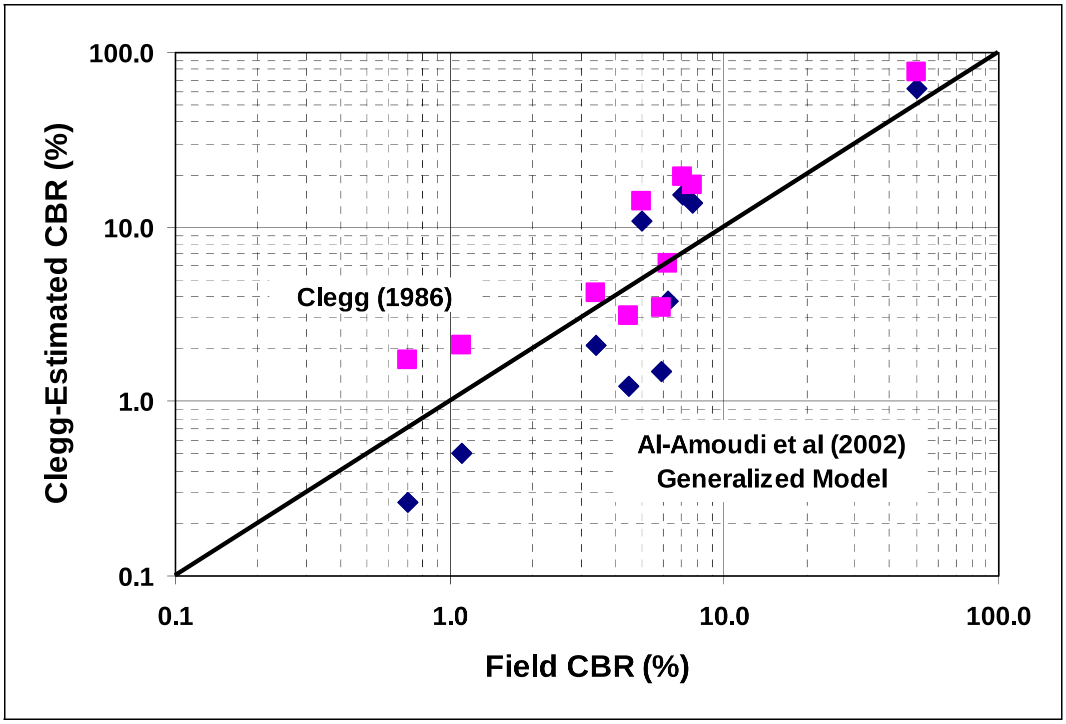

Figure 20. Clegg hammer-estimated CBR versus measured field CBR for various soils.

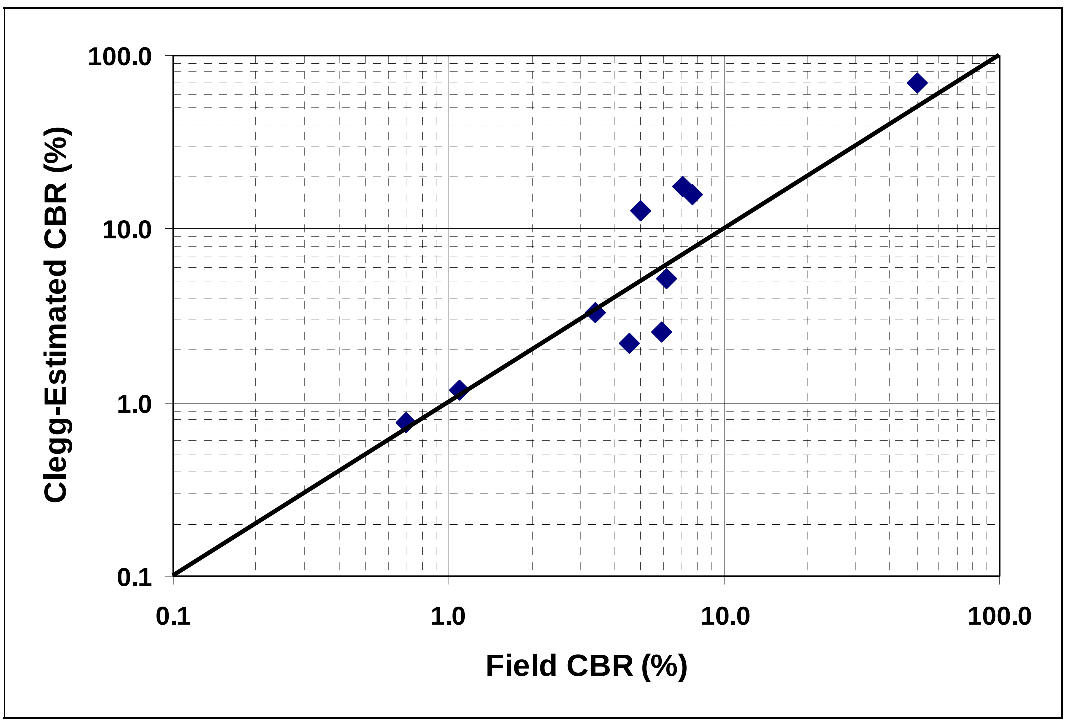

Figure 21. Clegg-estimated CBR, based on the "combination model," versus measured field CBR.

In summary, if the Clegg hammer must be used for estimating soil strength (CBR) for JRAC operations, the combination model (Eq. 14) is the preferred model. This model should be used only up to a CIV of 40 for unstabilized soil because 40 corresponds to a CBR of approximately 100%. The DCP should be used, however, if time permits and if the noise generated by the falling mass does not cause security problems.

Seeking optimum coverages for compaction

Given that the Clegg hammer will be available for JRAC operations, another potential use is monitoring changes in soil during compaction trials. The purpose for this would be in the determination of the optimum number of roller passes for a given construction situation. The advantage of the Clegg hammer in this regard is that it is a quick and easy test, so taking separate measurements over an area of soil at varying roller pass levels is reasonable. Attempting to conduct density tests after each couple of passes would become tedious. The Clegg hammer could identify when density tests are necessary, thus minimizing the number of density tests that must be performed for a compaction trial.

The opportunity for demonstrating the Clegg hammer in this regard involved a concurrent study at the ERDC in Vicksburg. The objective of this concurrent study was to quantify the ability of a pulverizer to mix soil uniformly with depth, given various soil additives. The study included two test sections, both constructed with an SM soil. One test section was called the “dry” test section because the pulverizer was used simply as a mixer; it did not inject any additives during mixing. The second test section was called the “wet” test section because, while the pulverizer blended soil, it also injected an emulsified polymer at a rate of 3% by mass. The Clegg hammer was conducted on two separate test items within each SM test section. The dry test section included test items that were treated as follows:

- In Item 1, the pulverizer blended in a 1-1/2-in. layer of crushed limestone aggregate.
- In Item 2, the pulverizer blended in 2-in.-long fibrillated fibers (0.4% by mass).

Similarly, the wet test section included two test items that were treated as follows:

- In Item 1, the pulverizer blended in the emulsified polymer (3% by mass) only.
- In Item 2, the pulverizer blended in both the emulsified polymer and Type III portland cement (3% by mass).

In all cases, the pulverizer blended to a depth of 12 in. After pulverization and mixing, compaction was not intended to accomplish any target density; vibratory rollers were used simply to densify the soil to a state that allowed destructive sampling. The dry test section was compacted with a Caterpillar® CS-433E weighing 15,000 lb, and the wet test section was compacted with a Caterpillar® CS-563D weighing 24,000 lb. On both test sections, the drums were vibrated during rolling.

The following discussion concerning Clegg hammer tests during compaction is intended for two purposes:

- To demonstrate that CIVs can reflect changes in soil during compaction
- To verify the potential for using CIV results to identify the optimum number of coverages for compaction

Clegg hammer results for the wet test section with polymer emulsion and portland cement are shown in Figure 22. The Clegg hammer test was conducted after each coverage or each two coverages up to 10 coverages. Each average CIV shown in Figure 22 represents the average of eight measurements distributed around the test section. The coefficient of variation for these eight measurements is also shown in the figure. The average CIV increased each time it was measured. After 10 coverages, the roller was still not able to complete compaction because the lift was 12 in. thick. The coefficient of variation was consistently lower than 15%.

Clegg hammer results for the wet test section with only polymer emulsion are shown in Figure 23. In terms of CIV attained, this modified soil did not compact as easily as the soil with cement, perhaps due to the excess water. Similar to the previous test item, CIV was still increasing at 10 coverages. The coefficient of variation was consistently lower than 20%.

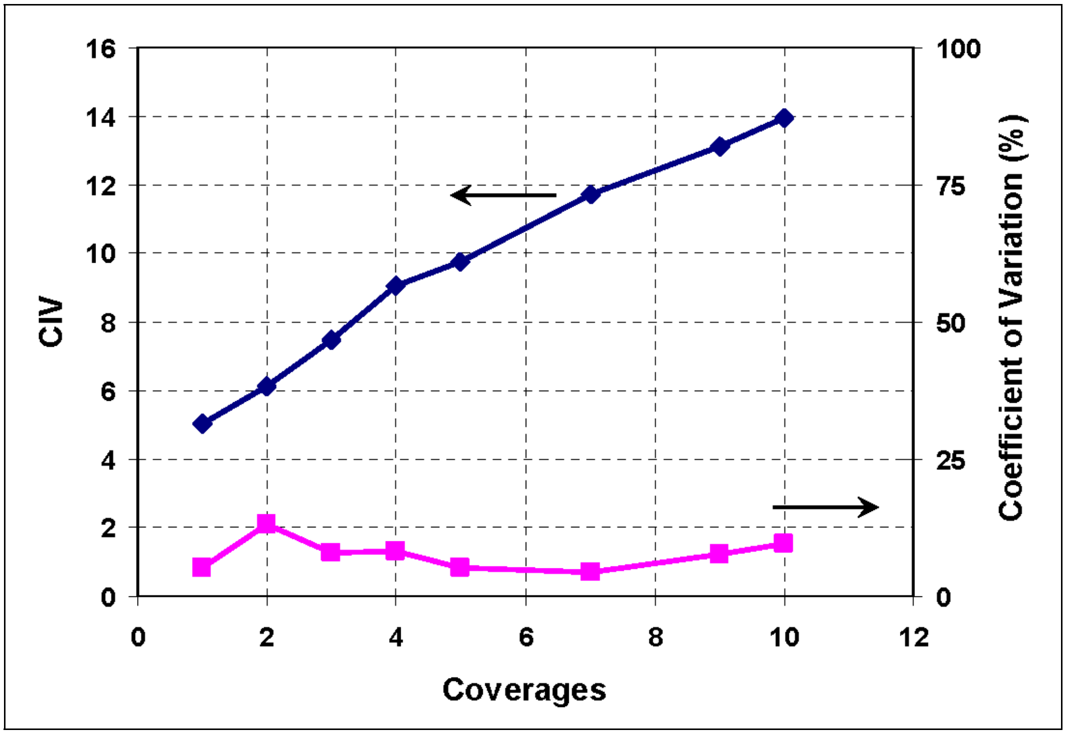

Figure 22. Clegg hammer results for SM soil stabilized with polymer emulsion and portland cement.

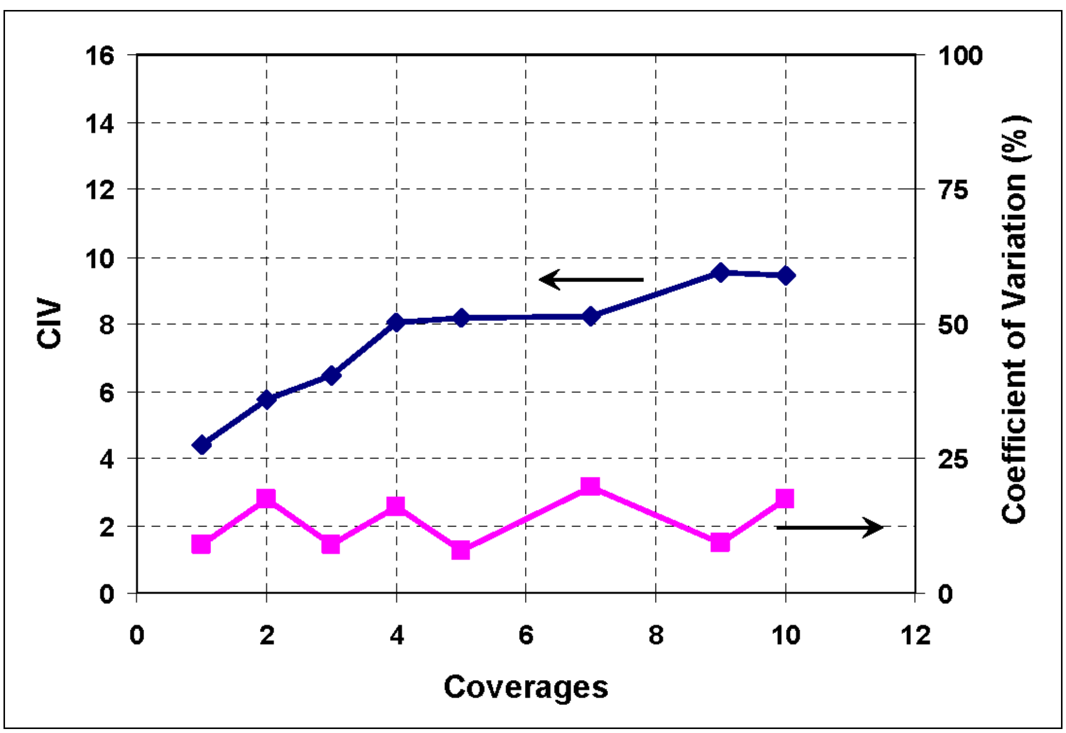

Figure 23. Clegg hammer results for SM soil stabilized with polymer emulsion.

The dry test section was compacted with the lighter compactor, so the CIVs attained were generally lower. Figure 24 shows Clegg hammer results for the dry test section with blended crushed limestone aggregate. Because of the lighter compactor, the CIV was leveling off after only 7 coverages.

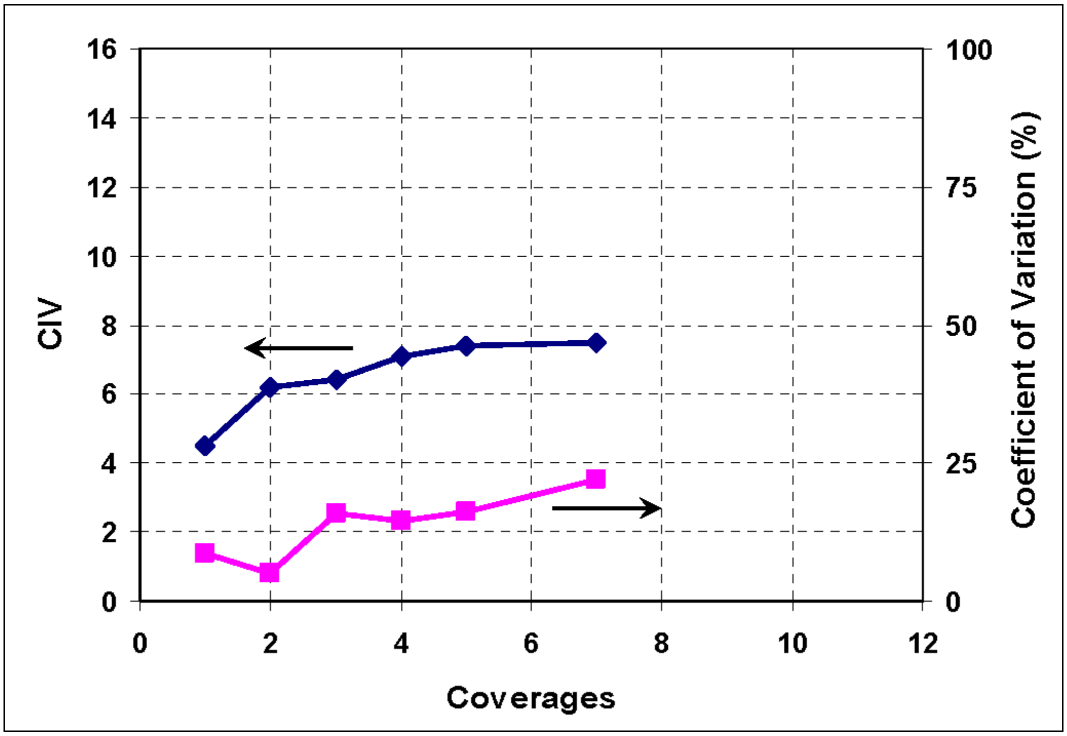

Figure 24. Clegg hammer results for SM soil stabilized with limestone aggregate.

In a real compaction trial scenario, additional coverages would be applied to ensure that the maximum possible degree of compaction had indeed been accomplished. The coefficient of variation was consistently lower than 25%.

Figure 25 shows Clegg hammer results for the dry test section with 2-in.-long fibrillated fibers. This test item exhibited an odd decrease in CIV from coverage 5 to coverage 7. Soil with fibers is known to be "spongy" and difficult to compact. Perhaps once the fiber-modified soil has densified as much as it will allow, additional coverages are disruptive. In a compaction trial, additional coverages would be advised to ensure that the optimum is 5. The coefficient of variation was consistently lower than 20%.

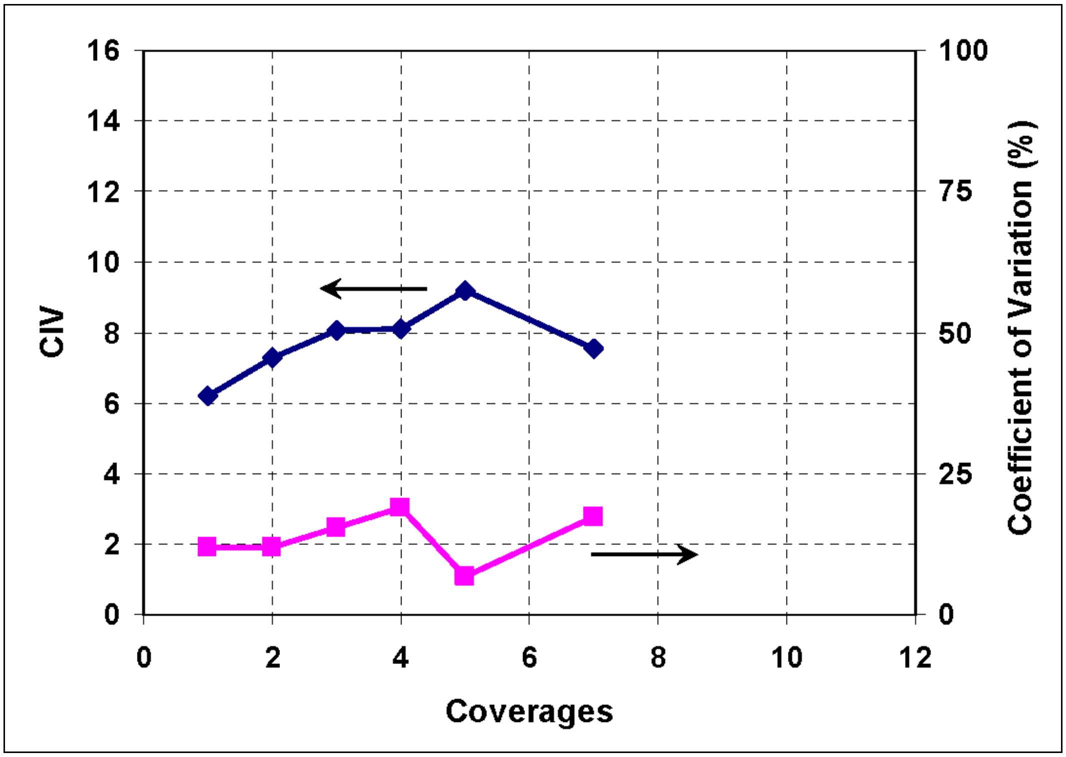

Figure 25. Clegg hammer results for SM soil stabilized with fibrillated fibers.

While actual densities were not measured between compaction coverages because the primary purpose of the test section was to study soil blending, the data support the observation that the Clegg hammer measurements were influenced by changes in soil that occurred during compaction. The authors surmise that the most influential change during compaction was soil density. Therefore, the authors propose that during compaction trials on a construction project, the Clegg hammer can be used to identify the coverage levels for which density measurements should be attained. During construction, the Clegg hammer can be used as a quick way of checking the adequacy and thoroughness of compaction efforts.

7 Guidance for Compaction Operations

For the purposes of this guidance, the compaction equipment that is assumed to be available for JRAC scenarios includes the Caterpillar® 433E and 563D. Both of these compactors are available with either a smooth drum or a padded foot drum. Pertinent information for these pieces of equipment for compaction operations is given in Table 7.

Table 7. Specifications for compactors expected to be available for JRAC scenarios.

	Light Compactors		Heavy Compactors	
	Smooth Drum	Pad-Foot Drum	Smooth Drum	Pad-Foot Drum
Model	CS-433E	CP-433E	CS-563D	CP-563D
Applicable Footnotes	a,c,d	a,d	a,b,c,d	a,b,d
Speeds	2 forward/ 2 reverse	2 forward/ 2 reverse	2 forward/ 2 reverse	2 forward/ 2 reverse
Maximum Speed (mph)	7.1	7.1	7.8	8.1
Working Speed (mph)	3.4	3.4	4.0	4.0
Weight (lb): Operating Shipping	 14,635 14,036	 15,170 14,580	 23,975 23,243	 24,856 24,123
Drive	Drum/ Rear Wheel	Drum/ Rear Wheel	Drum/ Rear Wheel	Drum/ Rear Wheel
Vibratory System: Frequency (vpm) High amplitude (in.) Low amplitude (in.)	 1,915 0.066 0.033	 1,915 0.061 0.030	 1,915 0.067 0.033	 1,915 0.067 0.033
Centrifugal Force (lb): High amplitude Low amplitude	 30,000 15,000	 30,000 15,000	 60,000 30,000	 60,000 30,000
Drum Dimensions (ft-in.): Width Diameter	 5'6" 4'0"	 5'6" 4'0"	 7'0" 5'0"	 7'0" 5'1"
General Dimensions (ft-in.): Overall width w/ blade Overall width w/o blade Overall height Wheel to drum Overall length Curb clearance	 6'11" 5'11" 9'7" 8'6" 16'3" 1'3"	 6'11" 5'11" 9'7" 8'6" 16'3" 1'3"	 8'0" 7'6" 9'11" 9'6" 18'1" 1'7"	 8'0" 7'6" 9'11" 9'6" 18'1" 1'9"

Source: Caterpillar, Inc. (2002).
[a] Leveling blade available.
[b] Drum conversion kit available (smooth to padded or padded to smooth).
[c] Shell kit available (smooth shell or padded shell).
[d] Variable frequency vibration available (1400 to 1915 vpm).

Generally, a heavy compactor is preferred for compaction operations because the number of passes for accomplishing a desired density can be reduced. However, a lighter compactor has the advantage of easier shipment. Therefore, light construction units in the military often have to rely on the lighter compactors.

Recommendations on drum type and static versus dynamic compaction are summarized in Table 8. The recommendations are based on soil type. At the extremes of soil types are coarse materials without fines at one end and clays at the other end. In general, coarse materials benefit from vibration during compaction because the vibrations allow particles to reorient into their most dense packing state. Clays do not benefit from vibration because the vibratory forces would simply be transferred to pore water pressure, without affecting the packing density of particles. Coarse materials are best compacted by a smooth drum because the smooth drum can promote "tightening" of the compacted surface. If a pad-foot drum were used for coarse materials, the feet would promote particle crushing and would continually disturb the surface of the compacted layer, without net benefit. Clays are best compacted by a pad-foot drum because clays require a kneading action for efficient densification.

Table 8. Recommendations for type of compactor based on soil characteristics.

Description of Soil	Soil Examples Using the USCS[a,b]	Type of Compactor		
		Smooth Drum Vibratory	Pad-Foot Vibratory	Pad-Foot Static
Coarse Gravel without Fines	GW, GP	Best	Fair	Poor
Fine Gravel or Coarse Sand without Fines	GW, GP, SW, SP	Best	Good	Fair
Sand/Gravel Mix with Little Fines	GW-GM, GW-GC GP-GM, GP-GC SW-SM, SW-SC SP-SM, SP-SC	Good	Best	Good
Coarse-Grained Soils with Clay Binder	GM, GC, GC-GM, SM, SC, SC-SM	Fair	Best	Best
Fine Sands and Silts with Clay	SM, SC, SC-SM CL-ML	Fair	Best	Best
Silty and Sandy Clays	CL, CL-ML, ML	Poor	Good	Best
Clays	CH, MH, CL	Poor	Fair	Best

Source: Caterpillar, Inc. (2006).

[a] USCS = Unified Soil Classification System (ASTM D 2487).

[b] Column added by the authors – not part of the Caterpillar® reference.

For rapid airfield construction (i.e., JRAC), lift thicknesses should be limited to 6 in. after compaction. This thickness is recommended for three reasons:

1. This thickness would be advisable for most situations, especially if a military unit is limited to using one of the light compactors.
2. If thicker lifts are considered, equipment operators may be apt to attempt to err on the higher side of lift thickness. Excessive thicknesses may result in low densities near the bottom of lifts.
3. The steel-shot method of conducting soil density tests is currently limited to a testing depth of 6 in. Density testing should include the entire lift thickness.

During compaction, rollers should operate at the working speeds shown in Table 7. The speeds of 3.4 mph and 4 mph can be converted to approximately 300 ft/min and 350 ft/min. If compaction is to include vibrations, the frequency and amplitude can be adjusted on-site, within the machine capabilities listed in Table 1. However, these vibration parameters should remain unchanged once the test section is started and through construction. Generally, lower frequencies and higher amplitudes are needed for large aggregates and/or thick compaction lifts. Rapid airfield construction should not involve large aggregates or thick lifts, so initial recommended values are on the order of 1,915 vpm and 0.033 in.

The process for determining the optimum number of roller passes will be discussed in the next section. The discussion will include the use of the term “coverage.” Coverage is the minimum number of roller passes received by any point on the surface of soil. It is generally assumed that the roller passes are applied in a manner that attempts to distribute the compactive effort uniformly across the soil. The process is complicated slightly, however, by overlapping compactor drums. If an area to be compacted requires six compaction lanes (including drum overlap), then two coverages would require 12 roller passes. The recommended range of coverages for compacting soil is 2 to 10. Even numbers provide for convenient coverages because they allow for rollers to travel “up and back” in the same lane, thus minimizing transverse position shifts. If more than 10 coverages are required, the following decisions need to be reviewed:

- Type of roller drum
- Static or dynamic compaction

- Weight of roller
- Lift thickness

Estimates for compaction productivity are important for planning purposes. Productivity is typically measured as cubic yards per hour. Productivity is generally improved by

- Minimizing roller passes
- Maximizing roller speed
- Maximizing drum width
- Maximizing lift thickness
- Minimizing changes in roller direction

Roller passes are minimized by using the heaviest available and suitable equipment. Roller speed and drum width are limited by the equipment that is available for JRAC scenarios, as presented in Table 7, and the lift thickness for JRAC is limited to 6 in. Changes in roller direction can be minimized by utilizing the longest construction lanes possible. However, the practical length of compaction lanes is limited by soil drying. All compaction for an area of soil needs to be completed before the soil dries to a moisture content of 1% below the predetermined optimum moisture content (OMC).

The calculation for typical productivity assumes that, on the average, each change in direction for a roller is equivalent to traveling 45 ft at working speed. This calculation also assumes that the effective width for a compaction drum is equal to actual width minus 1 ft. The effective width allows for the necessary lane overlap.

$$Productivity\ (\mathrm{yd}^3/\mathrm{hr}) = \frac{V}{T \cdot C} \cdot \left(\frac{\mathrm{yd}^3}{27\ \mathrm{ft}^3}\right) \cdot \left(\frac{3600\ \mathrm{s}}{\mathrm{hr}}\right) \tag{15}$$

where:

V = volume of soil (ft^3) per lane (Eq. 16)
T = time (s) per pass for one lane (Eq. 17)
C = number of compaction coverages needed

$$V = L \cdot t \cdot (w - 1) \tag{16}$$

where:

L = compacted length (ft) of each lane
t = lift thickness (ft)
w = drum width (ft)

$$T = \frac{(L + 45)}{S} \tag{17}$$

where:

S = working speed (ft/s) of the roller
$= \frac{\text{miles}}{\text{hr}} \cdot \left(\frac{5280\ \text{ft}}{\text{miles}}\right) \cdot \left(\frac{\text{hr}}{3600\ \text{s}}\right).$

The final elements of compaction to be covered here are related to moisture control and target density. A unique part of the JRAC scenario is that time and facilities are not available for measuring OMC and maximum dry density (MDD), as would normally be determined by conducting laboratory procedures to produce moisture-density curves (e.g., ASTM D 1557). The OMC and MDD, which are related to soil characteristics and compactive effort, must be surmised based on measured physical properties of the soil and a database of laboratory test results for soils. The expected accuracies of the predictions for OMC and MDD are ±2% and ±5 pcf, respectively.[1] The uncertainty associated with the absence of a laboratory-produced moisture-density curve makes the construction and evaluation of a compaction test section absolutely mandatory for the JRAC scenario. The objectives and procedures for accomplishing a compaction test section will be presented in the next two chapters.

[1] Personal communication. 2007. Dr. Ernest S. Berney IV, Research Civil Engineer. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

8 Test Sections for Compaction

Objectives

The objectives of a compaction test section are to

- Confirm the process for achieving proper pre-compaction soil moisture contents
- Optimize the type of compactor drum
- Confirm the speed of compactors
- Optimize the number of compaction coverages
- Identify the target soil properties for a properly compacted lift
- Confirm the target optimum moisture content
- Confirm the loose lift thickness requirement

Confirming pre-compaction soil moisture contents simply involves ensuring that the procedures used for wetting the soil produce moisture contents within allowable limits of a target moisture content. The soil must be within 1% and plus 2% of the target value, which is the estimated optimum moisture content (DA 1997; DoD 1997a).

The type of compactor (smooth drum or padded) may be decided based on soil type, as explained in the previous section. However, if the type of compactor is still under debate, the test section provides an opportunity to determine the best drum configuration.

The speed of compactors should remain at the values recommended in the previous section (300 ft/min for 433E models and 350 ft/min for 563E models). The test section provides an opportunity to ensure that the drivers can maintain these speeds, with exception for turning or changing directions.

The number of coverages should remain within the ranges defined in the previous section, that is, 2 to 10. However, the test section permits the opportunity to determine the optimum number of coverages, which is the minimum number of coverages that accomplishes maximum possible soil density (and stiffness) given the situation.

Identification of target soil properties for a properly compacted lift is necessary because these targets will be used for the remainder of the compaction operations for the airfield feature. The soil properties will include Clegg impact value and soil dry density, that is, if dry density can be measured. Density will not be measured directly for soil that is stabilized with fibers.

Confirmation of the target OMC is necessary because the OMC was estimated based on soil physical properties, rather than by producing moisture-density curves in a laboratory. This confirmation will involve primarily a visual assessment, as described in the next section. This confirmation is necessary because striving for a target moisture content that is not the OMC will result in lower soil densities in the compacted lift.

Confirmation of the loose lift thickness requirement simply involves ensuring that the resulting compacted lift in the test section is 6 in. ± 0.25 in. The initial estimate for appropriate loose lift thickness will be 8 in. for soils in the JRAC scenario. This thickness may need to be adjusted slightly to obtain the proper compacted lift thickness. This confirmation is necessary because thin lifts will reduce productivity and thick lifts may not have uniform density throughout their thickness.

Test section layout

The test section can be inside the actual construction area. However, in this case, any soil that is not compacted with the identified minimum number of coverages would have to be reworked and recompacted. In the case of chemically stabilized soil, these areas would have to be trimmed away, removed, and discarded. Examples of such soil include the soil areas outside of the test section length, where rollers decelerate, accelerate, and turn to line up for compaction lanes. Reworking of non-chemically stabilized soil will need to be accomplished in accordance with the compaction guidelines developed during the test section process.

The test section width must be at least three times the roller drum width. The test section length must be sufficient to allow a distance of at least 90 ft where rollers travel straight and at constant velocity. All turning, deceleration, and acceleration must be outside of this length.

The test section will be compacted in three lanes, as shown in Figure 26. Compaction will start with the outside lanes and will conclude with the center lane. The lanes should overlap by approximately 1 ft. Each lane will receive two passes at a time, as exemplified by the pass numbers in Table 9. Therefore, after each six passes of the compactor, at least two coverages of compaction are accomplished for the entire test section.

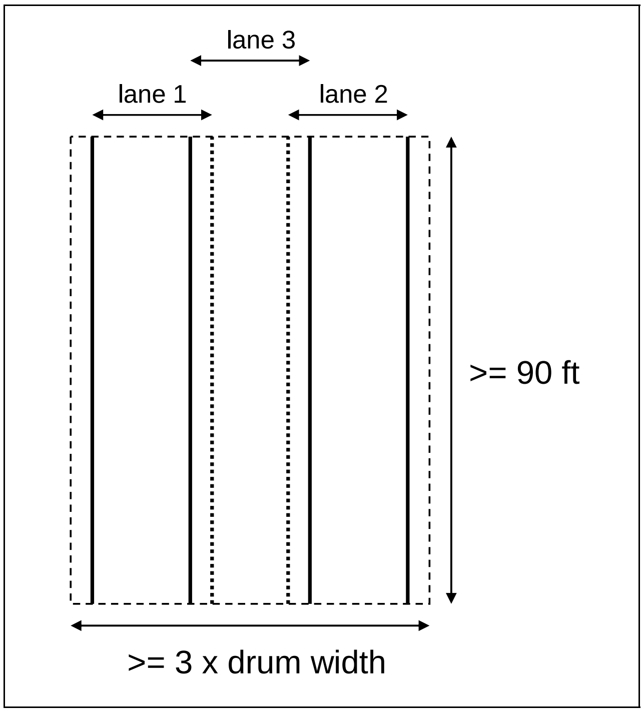

Figure 26. Test section size and compaction lanes.

Execution

The first objective for test sections was listed previously as confirming precompaction soil moisture contents. After the loose soil is spread in a manner to provide sufficient test section layout, soil samples should be obtained from four widely spaced locations within the 90-ft test section length. These "widely spaced" sample locations are identified visually; they are not identified using a formal randomization process. In the JRAC scenario, when maximizing speed of construction is critical, the statistical benefits of the randomization process are not sufficient to warrant its use.

If the average measured moisture content differs from the target moisture content by more than -1% or +2%, or if any of the measured moisture contents differs from the target moisture content by more than -2% or +3%, the soil wetting and/or processing procedure needs to be evaluated

Table 9. Relationship between pass sequence and compaction coverages.

Compactor Pass Number	Completed Coverages	Lane 1	Lane 3	Lane 2
1		X		
2		X		
3				X
4				X
5			X	
6	2		X	
7		X		
8		X		
9				X
10				X
11			X	
12	4		X	
13		X		
14		X		
15				X
16				X
17			X	
18	6		X	
19		X		
20		X		
21				X
22				X
23			X	
24	8		X	
25		X		
26		X		
27				X
28				X
29			X	
30	10		X	

and corrected. Compaction of a test section should not start until this moisture content requirement can be met. The average, or mean value ($\overline{x}$), is calculated as follows:

$$\overline{x} = \frac{\sum_{i=1}^{n} x_i}{n} \qquad (18)$$

where:

$\overline{x}$ = mean of variable x
x_i = i^{th} measurement of variable x
n = number of replicates for x.

The second objective for test sections, as listed previously, was to optimize the type of compactor (smooth drum or padded). This is only necessary if the decision was not finalized based on soil type. A comparison between the two types of compactors would simply involve accomplishing two test sections, one with each type of compactor. The two test sections can be compared in terms of minimum assigned compaction coverages, final densities, and final stiffnesses as measured by the Clegg hammer. The optimum compactor would be that which accomplished final density with the minimum number of coverages, that is, unless the compactor requiring more coverages offered a significantly higher density or stiffness. The decision as to whether the improved soil properties offered engineering significance would require engineering judgment.

The third objective for test sections, as listed previously, was to confirm that the drivers of the compactors could maintain proper speeds (300 ft/min for 433E models and 350 ft/min for 563E models). During the test section procedures that follow, a person should be assigned to measure travel times for compactors within known lengths of the constant velocity test section. The measured length should be as long as possible within the available 90+ ft. The drivers should be able to keep their speeds within 5% of the target values (i.e., the working speed of the roller). This equates to keeping measured times within 5% of the calculated target time to traverse the known length. Target time can be calculated as follows:

$$\text{target time (s)} = \frac{\text{known length (ft)}}{\text{target speed (ft/s)}} \tag{19}$$

The remaining four objectives for test sections, as listed previously, will be noted during the following explanation of test section compaction and testing procedures. JRAC operations can involve any of the four compaction scenarios shown in Table 10. Test section procedures for these scenarios will be discussed in the order shown in the table.

Table 10. Compaction scenarios for JRAC.

	Compactor has a Smooth Drum	Compactor has a Pad-Foot Drum
Soil or chemically stabilized soil without fibers	1	3
Soil or chemically stabilized soil with fibers	2	4

For scenario number 1 (Table 10), 10 Clegg hammer tests should be conducted after each pair of compaction coverages. An example of well-distributed test locations is shown in Figure 27. No tests should be conducted within the zones of compactor drum overlap (i.e., where compaction lanes overlap). As mentioned in reference to the moisture content tests, the location of testing would typically be determined randomly in ordinary construction situations. However, for a JRAC scenario, the cost in terms of time for measuring out randomly generated test locations would outweigh the statistical benefit. Also, contrary to the typical construction relationship between an owner and a contractor, the owner and the contractor are the same entity in a JRAC scenario. Therefore, while a contractor quality control program may err toward areas with seemingly high-quality materials or workmanship, the person conducting the Clegg tests for JRAC could be encouraged to err toward weak areas.

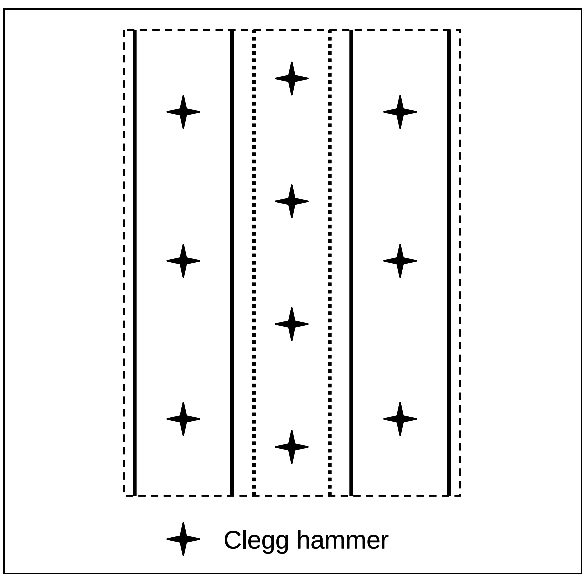

Figure 27. Example of well-distributed Clegg hammer tests.

The purpose of conducting Clegg hammer tests is to identify the maximum number of productive coverages by the designated compactor. The purpose of conducting the test after each pair of coverages is to allow the convenience of compacting “up and back,” prior to shifting compaction lanes.

Analyzing the changes in Clegg hammer results requires engineering judgment. The reason that this analysis is not amenable to a statistical process will become apparent in the ensuing presentation. To facilitate the process of making this judgment, at least 12 compaction coverages should be applied to the test section. There are two common “idealistic” trends for average Clegg hammer results:

- Diminishing densification (Figure 28)
- Densification followed by disruption (Figure 29)

The optimum number of compaction coverages for the idealistic trends is shown in each figure.

Figures 27 and 28 also show seven sets of reasonable average Clegg hammer results for the two idealistic trends, given typical test section variability. For these plots, the coefficient of variation for test results throughout the test section was assumed to be 20%, and each average CIV was assumed to have been calculated with ten Clegg hammer test replicates.

The purpose of these plots is to demonstrate the different types of results that can be expected for average CIV even if the true trend during compaction fits one of the idealistic models. When attempting to pick the optimum number of coverages for compaction, it is difficult to identify as a single number, so the engineer must err toward the higher value. For the idealistic trends in Figures 27 and 28, the optimum number of coverages is 10 and 8, respectively. To pick 10 coverages for diminishing densification, given any of the plausible results, the engineer needs to be aware of the variability that will accompany test results. The engineer must view the entire plot and look at the overall shape of the curve. A comparison of any single pair of test results, which differ in compaction by two coverages, may be misleading.

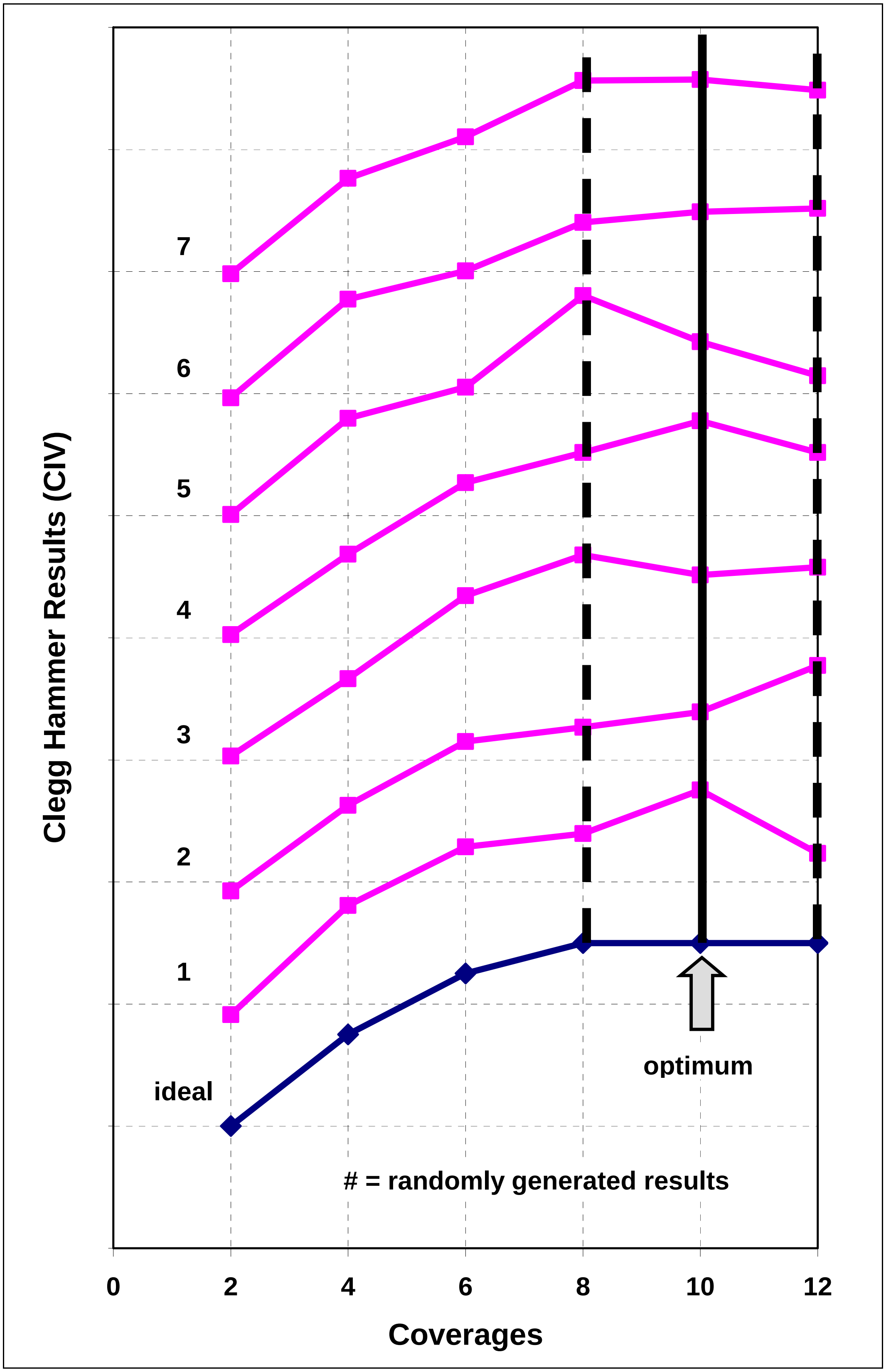

Figure 28. Reasonable Clegg hammer results for a test section with diminishing densification during compaction.

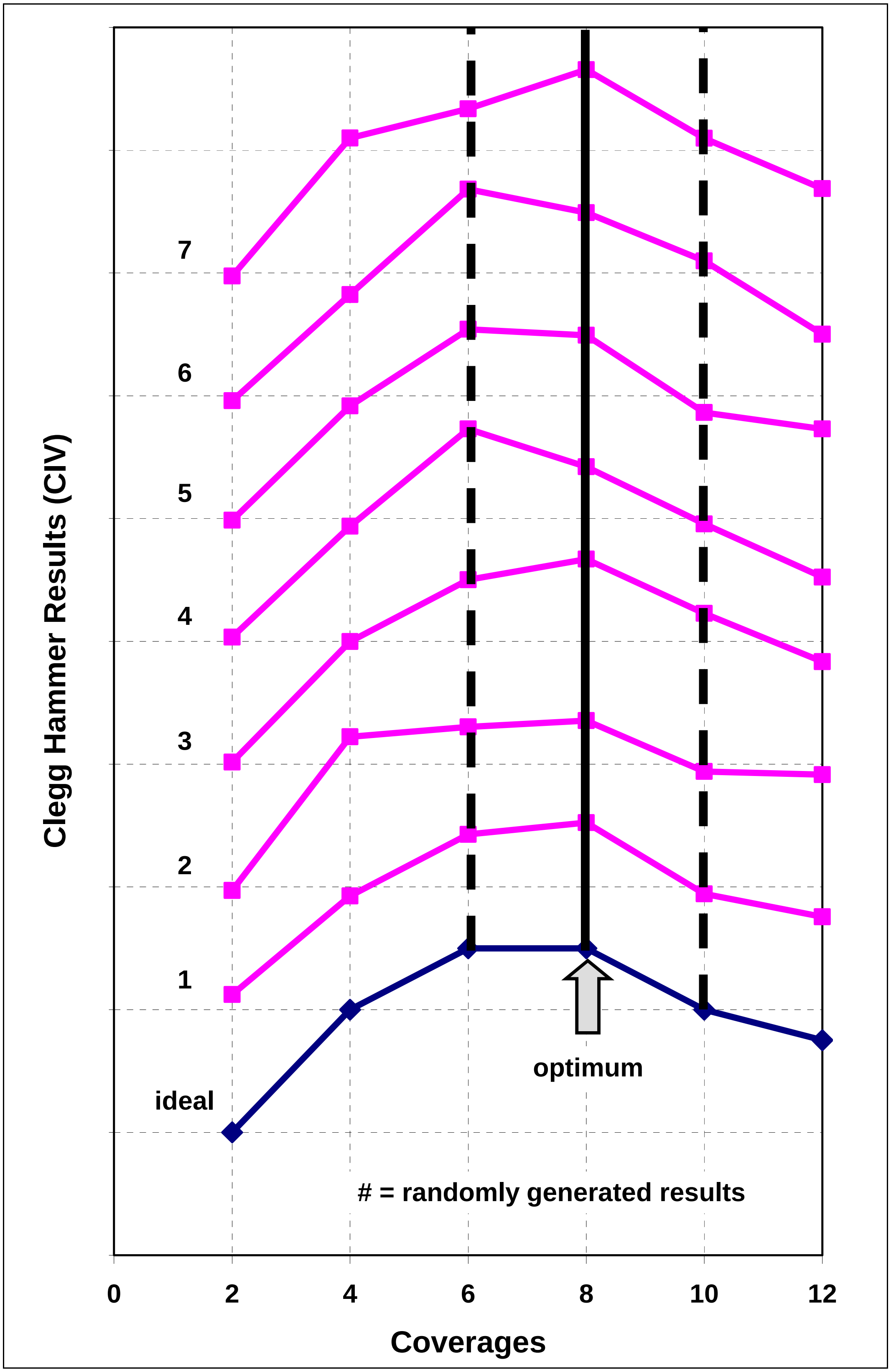

Figure 29. Reasonable Clegg hammer results for a test section with densification followed by disruption during compaction.

For the case of diminishing densification, the optimum number of coverages may be preceded by a slight decrease in average CIV (Figure 27, trend 5) or it may be followed by a slight increase in average CIV (Figure 27, trend 2). Sufficient initial identified numbers for optimum coverages are 8 to 12, as shown by the dashed lines. The optimum will continue to be scrutinized during compaction.

Similar statements can be made for the process of picking 8 as the optimum number of coverages for a test section with densification followed by disruption (Figure 29). Figure 29 demonstrates that the two or more consecutive decreases in average CIV are indicative of disruption. Sufficient initial identified numbers for optimum coverages are 6 to 10, as shown by the dashed lines. The optimum will continue to be scrutinized during compaction.

Meanwhile, the Clegg hammer data are also used to calculate the coefficient of variation for CIV. If the coefficient of variation exceeds 25% during test section operations, the engineer should investigate and correct the cause of variable compaction.

As the test section is compacted and tested and once a coverage level is identified as a possible optimum, two density tests should be conducted. Similar to the Clegg hammer tests, the density tests should be well distributed around the test section, as exemplified in Figure 30. For each pair of coverages that follow, two additional density tests are performed. If planned properly, the maximum total number of density tests should be six.

The final product of the test section is to provide target statistics for the measured material properties. These target statistics are the fifth test section objective, as listed previously. Target statistics for density and moisture (measured prior to compaction) are the mean values. The target mean for moisture should be the value that permitted maximum densification. The target mean for density should be the highest average among the various pairs of tests.

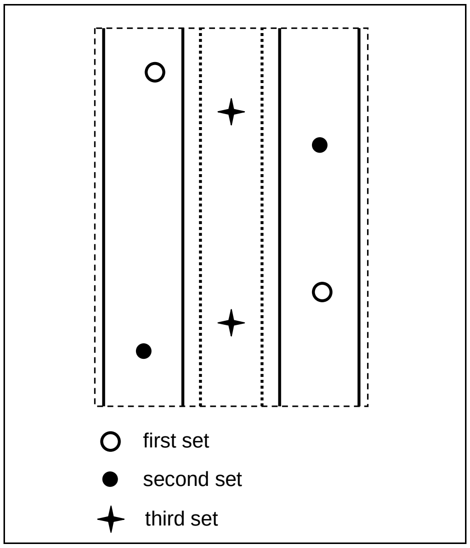

Figure 30. Example of well-distributed density tests.

The target statistics for Clegg impact values are then calculated from the data obtained at the selected optimum number of coverages. The target statistics for CIV include the mean value, variance (s^2), and standard deviation (s).

$$s^2 = \frac{\sum_{i=1}^{N} \left(x_i - \overline{x}\right)^2}{n - 1} \tag{20}$$

$$s = \sqrt{s^2} \tag{21}$$

Using the mean value, the standard deviation, and the sample size, Student's t-distribution can be used to establish a lower limit CIV below which only 10% of the test section CIV values fall. The lower limit is termed LL_{10} to signify the 10%.

$$LL_{10} = \overline{x} - t_{(\alpha,\nu)} \cdot s \tag{22}$$

where:

$t_{(\alpha,\nu)}$ = t – statistic from Appendix A, Table A1
α = level of significance = 10% = 0.10
ν = degrees of freedom = n-1.

Also, from standard deviation and the known sample size (n), the standard error of the mean, $\bar{s}$, is calculated as follows. The next section of text will explain how to use these statistics for quality assurance.

$$\bar{s} = \frac{s}{\sqrt{n}} \tag{23}$$

The sixth and seventh objectives of test sections are performed after compaction is completed. The sixth objective is to confirm that the target OMC was close to the actual OMC. Recall that, in the JRAC scenario, the target OMC would have been estimated using soil physical properties and a database of information. If the OMC has been estimated well, the measured MDDs in the test section should be within 5 pcf of the predicted values. Similar to the target OMC, the predicted values for MDD would have been obtained using soil physical properties and a database. A second method for ensuring that the target OMC was close to the actual OMC involves a visual inspection. If a handful of compacted soil at OMC is squeezed, the soil particles should stick together. However, if excess water drains from the soil in your hand, the target OMC was too high. If the soil in your hand has no ability to retain shape, the target OMC was too low. As an additional visual clue, if any water has ponded on the test section, the target OMC was too high. If any soil blows away with wind, the target OMC was too low.

The seventh objective for test sections is to confirm that the compacted lift meets the thickness requirement of 6 in. ± 0.25 in. In the case of processing in-place material, the assumption can be made that tilling to a depth of 6 in. will produce a compacted lift 6 in. thick. Proper execution of tilling 6 in. will require reliance on the modern pulverizer machinery. In the case of building on top of a ground surface with processed material, the lift thickness can be estimated by first surveying the ground surface prior to scarifying. The elevation should be measured at the ten locations shown for Clegg hammer testing in Figure 26. After the test section is completed, the new elevation can be measured at these same ten locations. The average thickness should be within 6 in. ± 0.25 in. If thickness control is adequate, there should not be any measured thickness outside of the range of 6 in. ± 0.5 in.

JRAC compaction scenario number 2 (Table 10) involves soil (or chemically stabilized soil) with fibers and compaction with a smooth drum

roller. The test section procedure would be the same as for scenario 1, except density testing would not be possible. The presence of intermixed fibers will prevent the digging of density testing holes.

JRAC compaction scenarios 3 and 4 (Table 10) are similar to scenarios 1 and 2, respectively, except compaction is performed with pad-foot drums. The test section procedures for scenarios 3 and 4 would be the same as for scenarios 1 and 2, respectively, except the Clegg hammer testing would not start until after the roller has "walked out" of the compacted lift. Of course, coverages that occur during the "walking out" process are still counted, and these coverages are included in defining "coverage A" and "coverage B." An additional difference is that the surface of the soil may be rough with pad-foot indentations. The surface must be smooth in areas for testing and final lift thickness measurements.

9 Quality Assurance for Compaction Operations

Once the test section has been completed and has facilitated determination of the number of necessary compaction coverages and the target test results, construction of the airfield feature (e.g., apron) can begin. During construction, as many as four test procedures will be required to ensure adequate quality for the final product:

1. Moisture content
2. Smoothness (straightedge)
3. Clegg hammer
4. Density

Density tests can only be conducted if the soil does not include fibers. Many additional construction procedures must be performed properly to ensure adequate quality. For example:

1. In situ soil must be consistent throughout the airfield feature, that is, there must not be substantial changes in soil characteristics.
2. Grade throughout the airfield feature must be established properly.
3. Processed soils must be blended and must be placed without segregation,
4. Fibers and or cement must be spread uniformly prior to mixing, and they must be added at the designated rate (percent by weight).
5. Proper compacted lift thickness will require reliance on the GPS-controlled construction equipment for accomplishing the proper loose lift thickness that was confirmed during the test section operations.
6. Compaction must be accomplished in accordance with the procedures established by the test section.

This presentation assumes that the aforementioned procedures were performed correctly. In reference to the first example above, if the natural soil within a feature is not uniform, the situation can be treated in either of these ways:

1. Remove the soil to sufficient depth for processing and replacement.
2. Treat areas of different soils separately; thus, require separate test sections, etc.

Frequency of testing

In conventional construction, projects are divided into "lots" of material for the purpose of controlling contractor payment and/or penalization. Lots are typically divided into four equal-sized "sublots" for the purpose of ensuring that tests are distributed spatially throughout the lot (DA 2000). In the JRAC scenario, the contractor and the owner are one and the same. Therefore, the implementation of lots for the purpose of payments is not necessary. However, the implementation of lots for the purpose of controlling testing frequency and for approving a project in increments is still necessary.

According to FM 5-410 (DA 1997), typical lot sizes are 2,000 yd^2 for subbase construction and 1,200 yd^2 for stabilized subgrade construction. Also, according to FM 5-410, in order to statistically evaluate a lot, at least four samples should be obtained and tested. The FM recommends obtaining one sample from each of the four equal-sized sublots, and the FM also recommends random identification of sample locations. The net result is one sample for each 500 yd^2 of subbase and one sample for each 300 yd^2 of stabilized subgrade. This guidance, as well as guidance from several Unified Facilities Guide Specification documents, is summarized for unstabilized and stabilized materials in Tables 11 and 12, respectively.

Table 11. Specified testing areas for unstabilized materials.

Reference	Material or Test Type	Test Area yd^2/test
FM 5-410, "Military Soils Engineering" (DA 1997)	Subbase (any test)	500
UFGS 02721A, "Subbase Courses" (DoD 1997b)	Thickness	500
	Field density	1000
UFGS 02731A, "Aggregate Surface Course" (DoD 1998a)	Thickness	500
	Field density	1000

Table 12. Specified testing areas for stabilized materials.

Reference	Material or Test Type	Test Area yd^2/test
FM 5-434, "Military Soils Engineering" (DA 1997)	Stabilized subgrade (any test)	300
UFGS 02712A, "Lime-Stabilized Base Course, Subbase, or Subgrade" (DoD 1997c)	Thickness	500
	Field density	250
UFGS 02711A, "Portland Cement-Stabilized Base or Subbase Course" (DoD 1998b)	Thickness	500
	Field density	250

As mentioned previously, the concept of dividing a project into “lots” is important to JRAC both for the purpose of controlling testing frequency and for approving compaction effort in increments. For JRAC purposes, a lot should be flexible. For convenience, lot sizes may need to change with project geometry and speed of construction. Also, for simplicity in JRAC operations, no differentiation is made between the frequency of testing for unstabilized and stabilized soils.

The lot size for JRAC is defined as a convenient size as close to 500 yd^2 as possible and preferably between 400 and 600 yd^2. Given that each soil characteristic to be measured will involve a minimum of two test replicates, this provides for more testing frequencies that are commensurate with the stabilized materials in Table 12. Frequent testing was deemed important to JRAC because airfield pavements without hardened surfaces do not leave much room for error (i.e., materials with inferior quality). To help reduce the necessary testing, density measurements will be conducted in a step-wise manner, as will be discussed later.

As an example of establishing convenient lot size, suppose soil is placed in rows that are 70 yd long and 4 yd wide. In this case, two rows could be considered as a lot ($2 \times 280 = 560$ yd^2). Materials can be evaluated and approved in these lot-size increments. Smaller lot sizes may be necessary in cases where a day’s construction had to be stopped prematurely. A single lot should not include work that was performed on different days because this would cause confusion, especially in the case of stabilized materials that rely on curing.

Prior to compaction, four moisture contents will be measured for each lot, both before and after adjusting soil moisture. The soil should be adjusted and blended until the average moisture content is within -1% to +2% of the target OMC and no single measured moisture content is outside of the range -2% to +3% of the target OMC. Recall that the target OMC may have been adjusted during the test section process. If the soil is wetter than its OMC, then it must be aerated by tilling. If the soil is dryer than OMC, water must be added and blended into the soil. The calculation for the volume of water (Vw) necessary for a given volume of soil (Vs) and a calculated deficiency in moisture content (ω), can be accomplished as follows:

$$Vw \ \ (\text{gal}) = \text{MDD} \cdot Vs \cdot \omega \cdot \frac{1 \text{ gal}}{8.33 \text{ lb}} \tag{24}$$

where:

MDD = target maximum dry density (pcf)
Vs = volume of soil (ft^3) to be wetted (= length · width · thickness).

$$\omega = \frac{\text{OMC - mc}}{100\ \%} \quad (25)$$

where:

OMC = optimum moisture content (%)
mc = measured moisture content (%).

During compaction, smoothness testing will be conducted with a 12-ft straightedge wherever smoothness appears to be questionable. Following the examples of the Unified Facilities Guide Specifications listed in Tables 11 and 12, deviations from the straightedge in excess of 3/8 in. shall be corrected by removing material and replacing with new material, or by reworking and recompacting existing material.

Density tests are tedious and time-consuming, so the "rapid" requirements of JRAC will require that these tests be minimized. This is why the Clegg hammer was incorporated into the quality assurance program. Density tests are still necessary, however, in order to help detect any changes that might occur in materials. For example, the particle size distribution of a soil-aggregate blend can become more coarse, thus keeping Clegg values elevated even if relative densities fall. Density tests will be conducted in a step-wise manner, resulting in two to four density tests for each lot (provided the lot passes requirements). Density results during compaction are converted to "percent density," which is calculated with respect to the target average density found during the test section process. The limit requirements for density test results are shown in Figure 31. The average and extreme (i.e., individual) values from two density tests are compared to Figure 31 and evaluated as follows:

1. If neither warning nor action limits are exceeded, no further activity is required; the lot passes density requirements.
2. If a warning limit is exceeded, two additional density tests are required, and the engineer needs to be aware of the possibility for compaction problems or changing soil conditions.

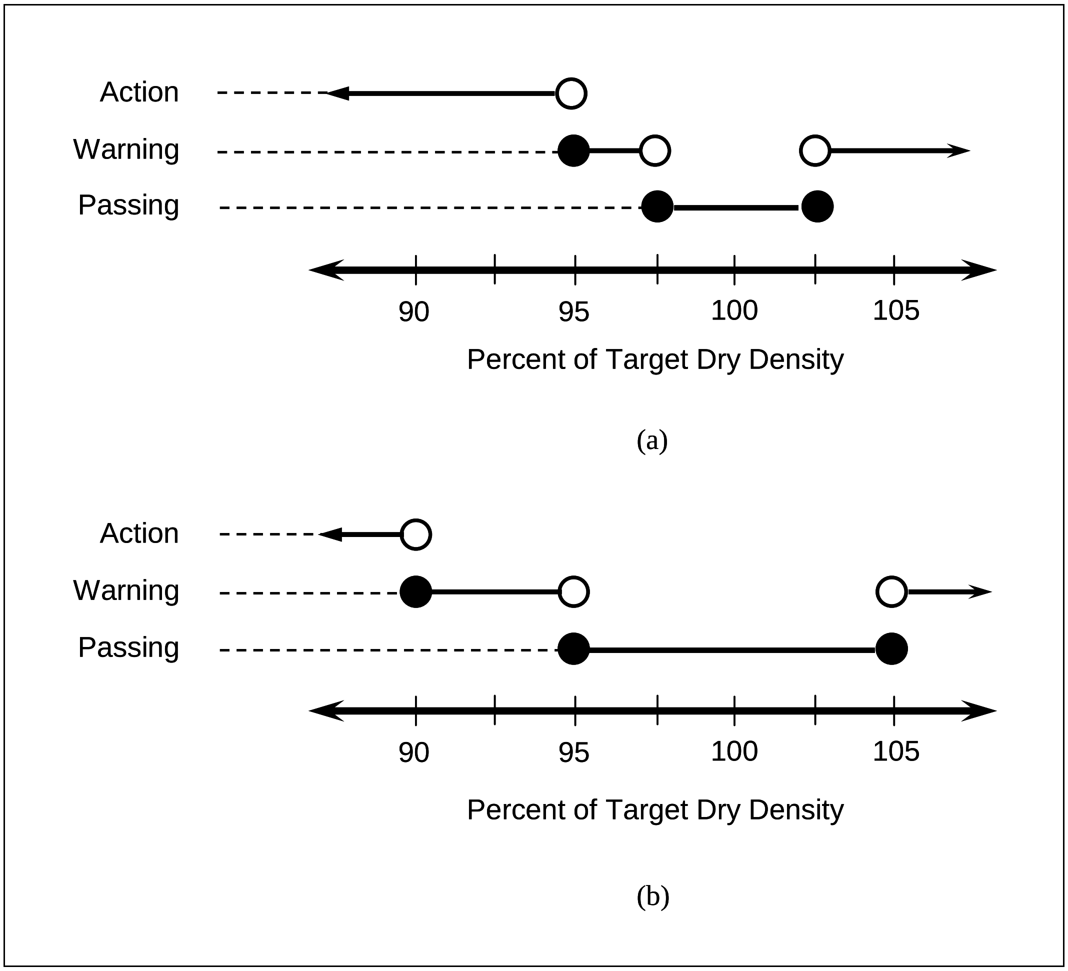

Figure 31. Limit requirements for dry density (a) average and (b) extreme values.

3. If an action limit is exceeded, the pertinent areas of the lot must either receive additional compaction or be investigated for changing soil properties. Construction is stopped until the problem is resolved.

If two additional density tests are required, for a total of four tests, the data are again compared to Figure 31:

1. If neither action limit is exceeded, no further activity is required; the lot passes density requirements.
2. If an action limit is exceeded, the pertinent areas of the lot must either receive additional compaction or be investigated for changing soil properties. Construction is stopped until the problem is resolved.

The line plots in Figure 31 will now be explained to clear up any confusion. A closed circle includes the value at which it rests: less than or equal to (≤) and greater than or equal to (≥). An open circle does not include the value

at which it rests: less than (<) or greater than (>). For each of (a) and (b) in Figure 31, all possible values are addressed. For Figure 31(a), the following rules apply:

1. "Action" is required if the average dry density (ρ_d) is less than 95% of target density.
2. Average dry density passes if $97.5 \leq \rho_d \leq 102.5\%$ of target density.
3. Otherwise, average dry density meets the "Warning" criteria.

For Figure 31(b), the following rules apply:

1. "Action" is required if any individual dry density (ρ_d) measurement is less than 90% of target density.
2. The individual dry densities pass if they are all within $95 \leq \rho_d \leq 105\%$ of target density.
3. Otherwise, the individual dry densities meet the "Warning" criteria.

Analyzing Clegg hammer results

The Clegg hammer is quick and easy to perform, so its testing frequency was established as 20 Clegg hammer tests for each lot, that is, on the order of one test for each 25 yd^2. Clegg hammer results are measured in terms of CIV. The target CIV statistics, which were established during the test section process, included a mean value ($\bar{X}$), the standard error of the mean ($\bar{s}$), and the lower limit below which 10% of soil CIV resides (LL_{10}). The speed of Clegg hammer testing allows it to be used to find areas that need work while the compactor is still nearby and while the soil is still at the proper moisture content. Therefore, similar to moisture content and density, the Clegg hammer results should be analyzed immediately. Similar to the density tests, this analysis includes two components:

1. Investigation of mean value
2. Investigation of extreme values

Each of these components has a separate purpose. The mean value provides an indication of central tendency for the Clegg hammer results, similar to the centroid for a 2-dimensional (2-D) area. Given a lot with an adequate mean value, the investigation of extreme values ensures that the lot was not excessively variable (similar to the moment of inertia for a 2-D area). Excessive variability could lead toward the existence of isolated weak areas in a lot.

When comparing the mean CIV for a lot ($\bar{x}$) to the target mean CIV from the test section ($\bar{X}$), the warning and action limits are shown in Figure 32. In using Figure 32a, the lot mean ($\bar{x}$) is compared to statistics calculated using test section data: $\bar{X}$, $\bar{X} - \bar{s}$, and $\bar{X} - 2\bar{s}$. The percent of the lot CIVs lower than LL_{10} is calculated by comparing a calculated T statistic (Eq. 26) with the standard t-distribution values in Appendix A. The lower limit, LL_{10}, was calculated previously using test section data. The T is calculated using LL_{10} along with $\bar{x}$ and s from the lot in question. When comparing T to the t-values in Table A1, interpolation may be required and is sufficient. This test is considered a "one-tail" test. The objective is to estimate alpha (α), which is the estimate for the percent of the CIV distribution that resides below LL_{10}.

$$T = \frac{\bar{x} - LL_{10}}{s} \tag{26}$$

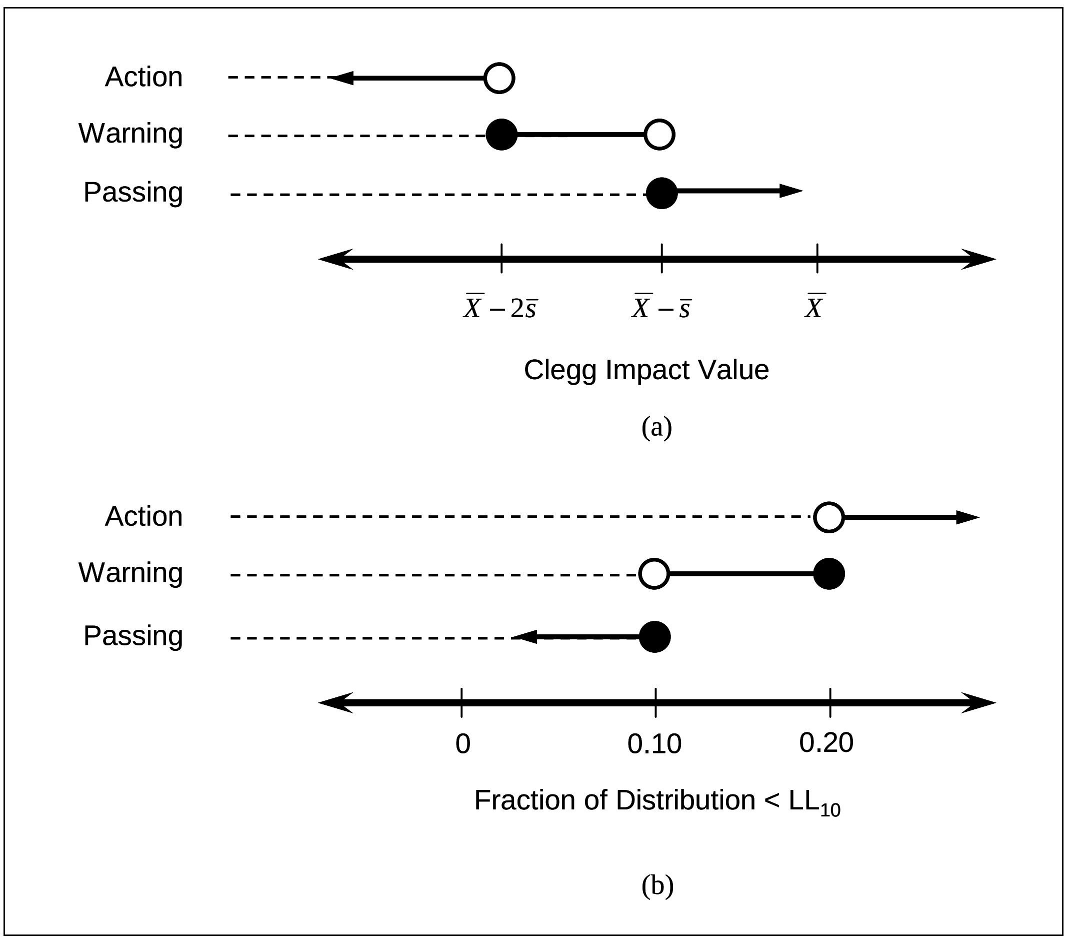

Figure 32. Limit requirements for Clegg impact value (a) average and (b) extreme values.

If either CIV statistic falls into a warning region, the engineer needs to be cognizant that his lot is just barely passing requirements. Procedures may need to be adjusted for the next lot to ensure adequate quality. If either CIV statistic falls into the action region, the lot needs to be recompacted and retested. If the lot still fails, an investigation is needed to determine whether the problem is related to materials, moisture, or compaction technique. Construction is stopped until the problem is resolved.

10 Conclusions

As part of the JRAC program, personnel of the ERDC, Vicksburg, MS, developed quality assurance procedures for contingency airfield construction. This effort was executed as part of the “Enhanced Construction Productivity” component of JRAC. The investigation included equipment evaluations, comparisons, and selections, which involved both laboratory and field studies. The products include guidance for test procedures, testing frequencies, data reduction, and construction decisions. More specifically, the findings can be summarized as follows:

1. The standard microwave test procedure (ASTM D 4643) is recommended for measuring the moisture content of soil. The direct heating method (ASTM D 4959) is recommended as a backup procedure.
2. A volume replacement method was recommended for measuring the in-place density of soils. This test method, which was named the “steel shot density test,” is a hybrid between the sand cone method (ASTM D 1556) and a simpler sand replacement test (ASTM D 4914). The steel shot density test, which involves 3/16-in. stainless steel balls, is fast, easy, and sufficiently accurate. Step-by-step procedures are explained herein.
3. The dynamic cone penetrometer (ASTM D 6951) is recommended for estimating the strength of soil. Standard procedures for reducing DCP data and converting these data to CBR are reviewed herein.
4. The Clegg hammer (ASTM D 5874) is recommended for estimating the strength of cement-stabilized layers (with or without fibers). Two equations are recommended for converting Clegg impact value to unconfined compressive strength (in units of psi):

$$\log(\mathrm{UCS}) = 0.081 + 1.309 \cdot \log(\mathrm{CIV})$$

$$\mathrm{UCS} = 12.51 \cdot (\mathrm{CIV}) - 285.9$$

The first equation is conservative, and the second equation provides estimates of “likely” values. Together, they provide a range of probable UCS values. These equations are limited to CIVs that are greater than or equal to 32, which corresponds to a UCS value of approximately 100 psi for both equations. (The difference between UCS estimates increases with increasing CIV.)

5. Due to its simplicity and speed, the Clegg hammer is also recommended as a backup tool for estimating the strength of soil. The recommended equation for converting CIV to CBR (%) is

$$CBR = 0.05 \cdot CIV^2 + 0.53 \cdot CIV$$

This equation is limited to CIVs less than or equal to 40, which corresponds to a CBR of approximately 100%.

6. The compaction procedures recommended herein are highly dependent on the results of a compaction test section. The test section serves several purposes, among which are identifying the optimum number of compactor coverages and obtaining target material properties. This process involves the Clegg hammer as the primary tool and the steel shot density test as the secondary tool.
7. For convenience and simplicity, the lot size for JRAC operations is flexible and is defined as being as close to 500 yd^2 as possible and preferably between 400 and 600 yd^2. Smaller lots are allowed to prevent a lot from including more than 1 day's placement. Testing includes moisture content, density, smoothness, and strength (Clegg hammer).
   a. Four moisture contents are required for each lot to ensure that the compaction is accomplished near OMC. The average moisture content must be within -1% to +2% of the target OMC, and no single measured moisture content can be outside of the range -2% to +3% of the target OMC.
   b. Density tests are time-consuming, so a step-wise approach is recommended where as few as two tests may be required for each lot. Warning and action limits are established for both the mean value and any single test, based on results of the compaction test section.
   c. Smoothness testing is conducted with a 12-ft straightedge wherever smoothness appears to be questionable. Deviations from the straightedge in excess of 3/8 in. shall be corrected by removing material and replacing with new material, or by reworking and recompacting existing material.
   d. Because of the simplicity and speed of the Clegg hammer, 20 tests are required for each lot. The Clegg hammer is the primary device for ensuring quality and consistent construction in a JRAC operation. Warning and action limits are established for both the mean value and the lower tail of the distribution of Clegg hammer results. The warning and action limits are based on results of the compaction test section.

The mean comparison ensures adequate central tendency for a lot. The lower tail comparison ensures that that there are no exceptionally weak areas within the lot.

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Appendix A: Statistical Tests Using Student's *t*-Distribution

To determine whether $\bar{x}_1$ is significantly larger than $\bar{x}_2$, calculate T using the data from the two samples:

$$T = \frac{\bar{x}_1 - \bar{x}_2}{\bar{s}_{1,2}}$$

where:

$\bar{x}_1$ = larger of the two sample means
$\bar{x}_2$ = smaller of the two sample means
$\bar{s}_{1,2}$ = pooled standard error of the means

$$\bar{s}_{1,2} = \sqrt{\frac{(n_1 - 1)\cdot s_1^{\,2} + (n_2 - 1)\cdot s_2^{\,2}}{n_1 + n_2 - 2}\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}$$

where:

n_1 = replicates in sample 1
n_2 = replicate tests in sample 2
s_1 = standard deviation for sample 1
s_2 = standard deviation for sample 2.

If $n_1 = n_2$, then

$$\bar{s}_{1,2} = \sqrt{\frac{s_1^{\,2} + \cdot s_2^{\,2}}{n}}$$

Table A1. Student's *t*-distribution.

Degrees of Freedom[a]	Probability (α)[b] of the Calculated *T* Exceeding *t* in This Table (One-Tail Test)[c]					
	α = 0.025	0.05	0.1	0.15	0.2	0.25
v = 1	*t* = 12.706	6.314	3.078	1.963	1.376	1.000
2	4.303	2.920	1.886	1.386	1.061	0.8165
3	3.182	2.353	1.638	1.250	0.9785	0.7649
4	2.776	2.132	1.533	1.190	0.9410	0.7407
5	2.571	2.015	1.476	1.156	0.9195	0.7267
6	2.447	1.943	1.440	1.134	0.9057	0.7176
7	2.365	1.895	1.415	1.119	0.8960	0.7111
8	2.306	1.860	1.397	1.108	0.8889	0.7064
9	2.262	1.833	1.383	1.100	0.8834	0.7027
10	2.228	1.812	1.372	1.093	0.8791	0.6998
11	2.201	1.796	1.363	1.088	0.8755	0.6974
12	2.179	1.782	1.356	1.083	0.8726	0.6955
13	2.160	1.771	1.350	1.079	0.8702	0.6938
14	2.145	1.761	1.345	1.076	0.8681	0.6924
15	2.131	1.753	1.341	1.074	0.8662	0.6912
16	2.120	1.746	1.337	1.071	0.8647	0.6901
17	2.110	1.740	1.333	1.069	0.8633	0.6892
18	2.101	1.734	1.330	1.067	0.8620	0.6884
19	2.093	1.729	1.328	1.066	0.8610	0.6876
20	2.086	1.725	1.325	1.064	0.8600	0.6870
21	2.080	1.721	1.323	1.063	0.8591	0.6864
22	2.074	1.717	1.321	1.061	0.8583	0.6858
23	2.069	1.714	1.319	1.060	0.8575	0.6853
24	2.064	1.711	1.318	1.059	0.8569	0.6848
25	2.060	1.708	1.316	1.058	0.8562	0.6844
26	2.056	1.706	1.315	1.058	0.8557	0.6840
27	2.052	1.703	1.314	1.057	0.8551	0.6837
28	2.048	1.701	1.313	1.056	0.8546	0.6834
29	2.045	1.699	1.311	1.055	0.8542	0.6830
30	2.042	1.697	1.310	1.055	0.8538	0.6828

[a] Degrees of freedom (ν) = $n - 1$, where n = number of replicates.

[b] Alpha (α) is also called the level of significance for a *t*-test.

[c] To use this table for a two-tail test: α(one-tail) = (1/2) x α(two-tail).

Appendix B: Procedure for the Steel Shot Density Test

Scope

This test method may be used to determine the in-place density and unit weight of soils using steel shot as a volume replacement material. The steel shot is used to estimate the volume of a test cavity from which soil has been removed. This test method is applicable for soils with a maximum particle size of ½ in. (12.7 mm). The soil must have physical and mechanical properties that permit the excavation of a stable cavity with a volume on the order of 0.04 to 0.05 ft^3. Consequently, this test method is not applicable for granular soils that are either saturated or loosely deposited.

Referenced documents

ASTM E 11, "Specification for Wire-Cloth and Sieves for Testing Purposes"

ASTM D 1556, "Density and Unit Weight of Soil in Place by the Sand-Cone Method"

ASTM D 2216, "Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass"

ASTM D 4643, "Determination of Water (Moisture) Content of Soil by the Microwave Oven Heating"

ASTM D 4753, "Evaluating, Selecting, and Specifying Balances and Standard Masses for Use in Soil, Rock, and Construction Materials Testing"

ASTM D 4944, "Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester"

Summary of test method

A test cavity is hand excavated in the soil to be tested and all the material from the cavity is saved in a container. The cavity is filled with steel shot of a known bulk density, and the volume is determined. The in-place wet density of the soil is determined by dividing the wet mass of the removed

material by the volume of the cavity. The water content of the material from the hole is determined and the dry mass of the material and the in-place dry density are calculated using the wet mass of the soil, the water content, and the volume of the cavity.

Significance and use

The steel shot test, which is much simpler than the sand cone test (ASTM D 1556), is an available alternative when the testing technician will have little time for training. The steel shot test should only be used when a loss in accuracy of approximately 1%, relative to the sand cone test, is permissible. The steel shot test does not appear to impose any bias in its measurement of density, relative to the sand cone test, so replicate averages will approach the same value.

Apparatus

Steel shot. The prescribed stainless steel balls are 3/16 in. (4.8 mm) in diameter. They are uniform in size in order to leave no opportunity for changing gradations, which could occur with segregation. The balls are American Iron and Steel Institute Type 440C, as specified by ASTM A 276, "Stainless Steel Bars and Shapes." Approximately 0.04 ft^3 of balls is needed for one test, which equates to approximately 12 lb.

Base plate. A square aluminum base plate or template that is 12 in. × 12 in. (30 cm × 30 cm). The plate will be flat on the bottom and it will have sufficient thickness to be rigid; a thickness of 1/8 in. has been found to be sufficient. The plate thickness should be measured prior to testing because the thickness must be considered in calculations. The plate will have edges that are raised to a height 3⁄8 to 1⁄2 in. (10 to 13 mm) above the top surface of the plate. The plate will have a 4-in.-diam hole through its center and it will have a 1/4-in.-diam hole near each of its four corners.

Graduated cylinder. A polypropylene graduated cylinder with a capacity of 1000 mL. The cylinder must be readable to divisions of 5 mL.

Balance or scale. The balance or scale should have a readability of 1.0 g and a capacity of at least 5 kg. This balance or scale will be used both for weighing the soil that is removed from the ground and for weighing moisture content samples. The scale must meet the requirements of ASTM D 4753.

Drying equipment. Equipment for drying soil moisture content samples must conform to ASTM D 2216, D 4643, D 4959, or D 4944. Equipment for drying steel shot must conform to either ASTM D 2216 or D 4959.

Pan. Shallow metal pan to be used when drying steel shot.

Scoop. A small round scoop appropriate for pouring steel shot into the graduated cylinder. A scoop that is 2-1/2 in. wide and 1 in. deep has been found to be appropriate.

Magnet. A handheld magnet is useful for retrieving steel shot. A magnet on the end of a retractable wand is particularly helpful.

Straightedges. The test requires two straightedges: 8 in. long and 12 in. long. The 12-in. straightedge is for scraping the surface of soil and the 8-in. straightedge is for striking off steel shot from the overfilled cavity. Each straightedge must be at least 1 in. wide.

Sieve. A sieve is necessary for cleaning the steel shot. The sieve should conform to ASTM E 11 and it should include both a stainless steel frame and a stainless steel wire screen. A No. 8 sieve (2.36-mm nominal opening) in an 8-in. diam frame is a convenient size.

Miscellaneous equipment. Small mallet (e.g., 3 lb), knife, small pick, chisel, small trowel, screwdriver, or spoons for digging test holes; large nails (e.g., 20d) for securing the base plate; a small paintbrush for collecting loose soil particles; buckets with lids for retaining moist soil samples; small bowls or containers suitable for moisture content determinations (i.e., must be suitable for either convection oven, direct heat, or microwave).

Procedure

Select a location/elevation that is representative of the area to be tested, and determine the density of the soil in-place as follows:

1. Use the small scoop to fill the graduated cylinder with steel shot up to the 1000-mL mark (see Figure 3 of main text). All measurements of steel shot bulk volume will involve flattening the surface of balls in the cylinder and estimating the volume at the top surface of the balls. While filling the cylinder, it may sit vertically or be tilted slightly. The scoop should be held

near the cylinder opening. The cylinder should not be subjected to excessive shaking or vibration.

2. Use a flathead shovel or the 12-in. straightedge to prepare the surface of the soil at the location to be tested. The surface of the soil should be a level plane.
3. Seat the base plate on the plane surface, making sure there is contact with the ground surface around the edge of the center hole. Secure the plate against movement by pushing a nail through each of the four ¼-in. holes.
4. The test cavity volume should be on the order of 0.04 to 0.05 ft^3 (1130 to 1420 cm^3), which equates to digging a 4-in.-diam cavity that is 6 in. deep. For most expedient construction operations, this depth should approximate the thickness of a compacted lift.
5. Dig the test cavity through the center hole in the base plate, being careful to avoid disturbing or deforming the soil that will bound the cavity. The sides of the cavity should slope slightly inward and the bottom should be reasonably flat or concave. The cavity should be kept as free as possible of pockets, overhangs, and sharp obtrusions since these affect the accuracy of the test. Place all excavated soil, and any soil loosened during digging, in a moisture tight container that is marked to identify the test number. Take care to avoid losing any materials. Protect this material from any loss of moisture until the mass has been determined and a specimen has been obtained for a water content determination.
6. Overfill the cavity slightly by pouring the steel shot from the graduated cylinder. This process will take filling the graduated cylinder twice with steel shot because the volume of the cavity should be on the order of 1100 to 1400 mL. While pouring, the open end of the graduated cylinder should be held within 3 in. of the top of the cavity. Use the 8-in. straightedge to strike off the overfilled cavity (Photo 30) and pick up the excess steel shot with the magnet. Place these balls back into the graduated cylinder.

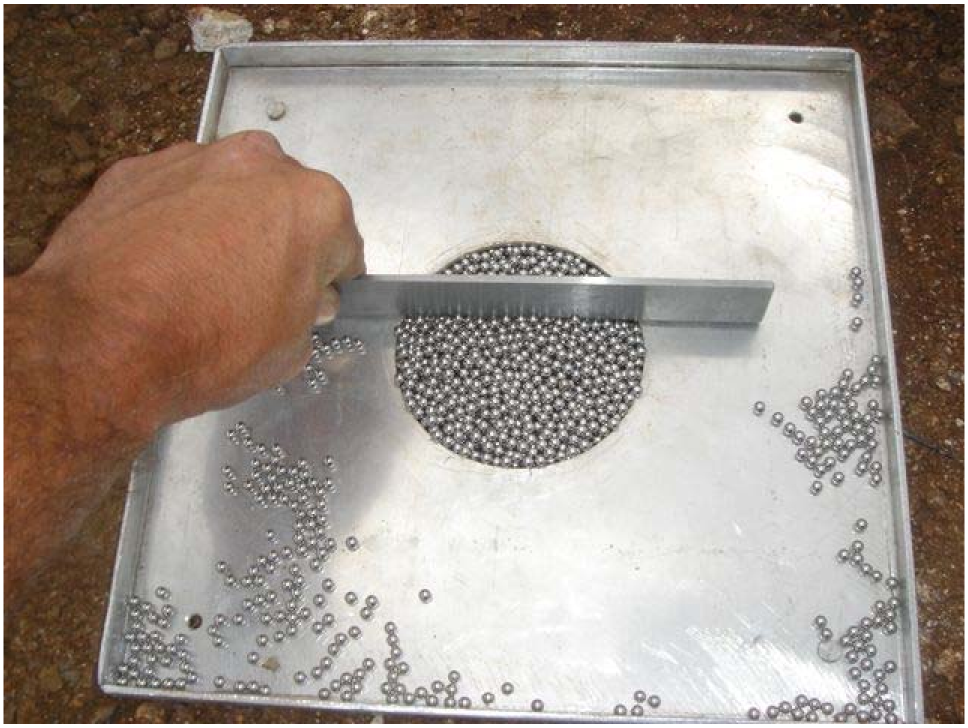

Photo 30. Striking off excess steel shot.

7. Flatten the top surface of the balls in the graduated cylinder and estimate the bulk volume of the remaining balls to the nearest 5 mL (= V_b) (Photo 31). Assuming the cavity is of sufficient size and the graduated cylinder had to be filled twice, the total volume of the cavity (V_t) is

$$V_t(\text{mL}) = 1000 + (1000 - V_b) - 25.7$$

$$V_t(\text{cm}^3) = V_t(\text{mL})$$

   The 25.7 mL is subtracted to account for the volume within the thickness of the plate.
8. Retrieve the steel shot using the small scoop and/or the magnet (Photo 32). Balls that stayed clean during the test can be reused immediately. Balls that have touched soil need to be cleaned as described in the last section of this procedural document.
9. Determine the mass of soil (in grams) removed from the cavity, being sure to exclude the mass of the container used to transport the soil (M_s).
10. Mix the material thoroughly, and obtain a representative specimen for water content determination. The representative specimen should have a mass of at least 250 g.

Photo 31. Steel shot remaining after completing the test
(i.e., after the second pour).

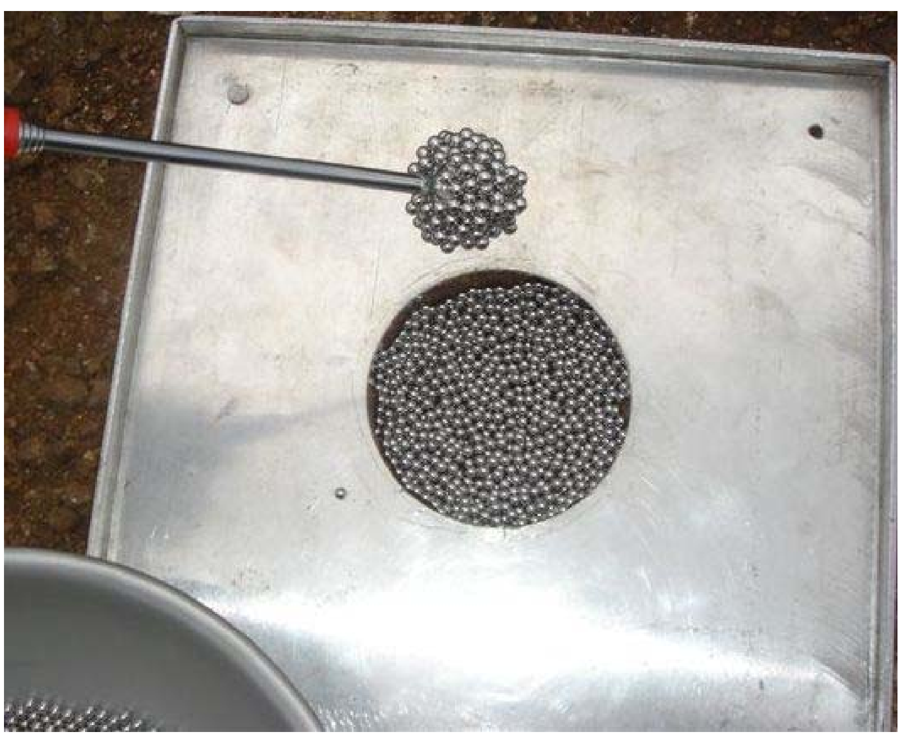

Photo 32. Retrieving steel shot.

11. Determine the water content (w) of the representative specimen in accordance with ASTM D 2216, D 4643, D 4944, or D 4959. Correlations to ASTM D 2216 will be performed when required by other test methods.

$$w\,(\%) = \frac{wet\ soil - dry\ soil}{dry\ soil} \cdot 100\%$$

Final calculations

1. Using mass of soil, M_s (grams), and total volume of cavity, V_t (cm^3), calculate the wet density of soil, ρ_s (g/cm^3):

$$\rho_s = \frac{M_s}{V_t}$$

2. Using the wet density of soil, ρ_s (g/cm^3) and the moisture content, w (%), calculate the dry density of soil, ρ_d (g/cm^3):

$$\rho_d = \frac{\rho_s}{1 + \frac{w}{100}}$$

3. To convert either wet density, ρ_s (g/cm^3), or dry density, ρ_d (g/cm^3), to unit weight of soil (lb/ft^3), multiply the density by the unit weight of water at room temperature (= 62.4 lb/ft^3). For example:
   a. If wet density (ρ_s) = 1.923 g/cm^3, wet unit weight (γ_s) = 120 lb/ft^3 (= 1.923 · 62.4).
   b. If dry density (ρ_d) = 1.763 g/cm^3, dry unit weight (γ_d) = 110 lb/ft^3 (= 1.763 · 62.4).

Reporting

Report, as a minimum, the following information:

1. Test location, elevation, thickness of layer tested, or other pertinent data to locate or identify the test
2. Test cavity volume, cm^3 or ft^3
3. In-place wet density, ρ_s (g/cm^3), or wet unit weight, γ_s (lb/ft^3)
4. In-place dry density, ρ_d (g/cm^3), or dry unit weight, γ_d (lb/ft^3)
5. In-place moisture content of the soil, w (%)

6. If the in-place density or unit weight should be expressed as a percentage of a target value, calculate the "relative density" or the "relative unit weight" as

$$\text{Relative density or relative unit weight} = \frac{\text{measured value}}{\text{target value}} \cdot 100\%$$

Cleaning steel shot

If the steel balls fall into contact with the soil, they must be cleaned prior to reuse. Cleaning the balls simply involves rinsing them with water while they rest on a sieve; a No. 8 sieve has been found to work well for this purpose (Photo 33). Slight agitation of the balls may be required to dislodge some soil particles. Once the balls appear to be clean, they can be dried either by sitting in the sun or through the use of any direct heat source (Photo 34).

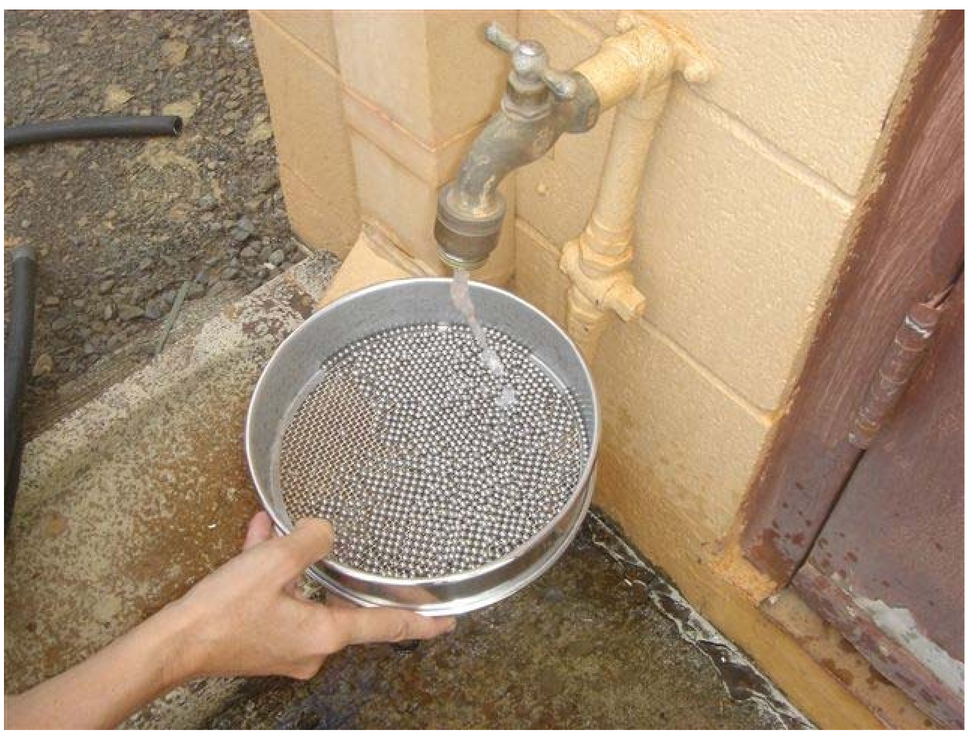

Photo 33. Rinsing balls in the No. 8 sieve.

Photo 34. Drying balls after rinsing.

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*
May 2008

2. REPORT TYPE
Final report

3. DATES COVERED *(From - To)*

4. TITLE AND SUBTITLE
Quality Assurance for Rapid Airfield Construction

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)
Reed B. Freeman, Travis A. Mann, L. Webb Mason, Chad A. Gartrell, and Vernon M. Moore

5d. PROJECT NUMBER

5e. TASK NUMBER

5f. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Engineer Research and Development Center
Geotechnical and Structures Laboratory
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER
ERDC/GSL TR-08-7

9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES)
Headquarters, U.S. Army Corps of Engineers
Washington, DC 20314-1000

10. SPONSOR/MONITOR'S ACRONYM(S)

11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT

This investigation was conducted to formalize a quality assurance (QA) process for rapid airfield construction. The specific aspects of QA that were addressed included compaction operations and the assessment of strength for both soil and stabilized soil layers. The QA for compaction relies on the construction of a test section for determining optimum number of compaction coverages and target soil properties. The essential pieces of equipment for the compaction QA process include a microwave oven, a Clegg hammer, and tools necessary to conduct a volume-replacement density test for in-place soil. This density test, which was developed during this investigation, involves the use of steel shot as the volume replacement material. The use of steel shot, instead of a conventional sand cone apparatus, was found to make the test both simpler and quicker. The Clegg hammer results are the primary means of judging compaction; thus, the requirements for density tests are minimized through a stepwise acceptance procedure. Statistical criteria for evaluating Clegg hammer and density measurements are also included herein. For estimating soil strength in terms of California bearing ratio, the conventional use of the dual-mass dynamic cone penetrometer is recommended. For estimating the strength of cement-stabilized soil and cement plus fiber-stabilized soil, a correlation between Clegg hammer results and unconfined compressive strength was developed.

15. SUBJECT TERMS
Clegg hammer
Joint Rapid Airfield Construction Program
Rapid airfield construction
Rapid quality assurance
Steel shot density test

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			19b. TELEPHONE NUMBER *(include area code)*
UNCLASSIFIED	UNCLASSIFIED	UNCLASSIFIED		122	