NCHRP SYNTHESIS 456

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Non-Nuclear Methods for Compaction Control of Unbound Materials

A Synthesis of Highway Practice

TRANSPORTATION RESEARCH BOARD
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Non-Nuclear Methods for Compaction Control of Unbound Materials

A Synthesis of Highway Practice

CONSULTANT
Munir Nazzal
Ohio University
Athens, Ohio

SUBSCRIBER CATEGORIES
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TRANSPORTATION RESEARCH BOARD

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NOTE: The Transportation Research Board of the National Academies, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program, conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council.

The members of the technical panel selected to monitor this project and to review this report were chosen for their special competencies and with regard for appropriate balance. The report was reviewed by the technical panel and accepted for publication according to procedures established and overseen by the Transportation Research Board and approved by the Governing Board of the National Research Council.

The opinions and conclusions expressed or implied in this report are those of the researchers who performed the research and are not necessarily those of the Transportation Research Board, the National Research Council, or the program sponsors.

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Cover figure: Nuclear and non-nuclear devices are being used for compaction control of unbound materials in a highway construction project in Iowa.
Credit: Iowa State University (2008).

FOREWORD

Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice*.

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

By Jo Allen Gause
Senior Program Officer
Transportation Research Board

Proper compaction of unbound materials, such as soils, aggregate, and recycled materials, is a critical component in the performance of highway pavements and embankments. The most commonly used device to test for proper compaction is the nuclear density gauge. However, due to the costs associated with regulatory compliance and radiation safety training, there is an increased effort to find acceptable non-nuclear devices. This synthesis documents information on national and international experience with non-nuclear devices and methods for measuring compaction of unbound materials.

Information used in this study was gathered through a literature review, a survey of state departments of transportation (DOTs) and Canadian provincial transportation agencies, and interviews with selected state DOTs.

Munir Nazzal, Ohio University, Athens, collected and synthesized the information and wrote the report. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable with the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

NON-NUCLEAR METHODS FOR COMPACTION CONTROL OF UNBOUND MATERIALS

SUMMARY

Proper compaction of unbound materials is one of the most critical components in the construction of pavements, airfields, and embankments to ensure their adequate performance, durability, and stability. Currently, field density is used as an indicator to assess the quality of compaction and construction of those structures. The nuclear density gauge is the main device used for measuring the field density of compacted layers of unbound materials. However, the use of this device entails extensive regulations and prohibitive costs associated with its handling, storage, calibration, and maintenance and the transportation of radioactive materials. Although the commonly used density-based quality control specifications are relatively straightforward and practical, they do not reflect the engineering properties of unbound materials required to ensure the materials' optimal performance and stability. In addition, the design of pavements and embankments is based on stiffness, strength parameters, or both. Thus, there is a missing link between the design process and construction quality control practices of unbound materials. To address this problem and help speed the construction process, as well as reduce costly construction oversight, federal and state transportation agencies have investigated the use of compaction control specifications for unbound materials that are based on a criterion that closely correlates to the performance measurements used in the design, such as stiffness and strength.

Different non-nuclear devices have been proposed and evaluated during the past years. Although some of the devices measure density and moisture content, others assess the in situ stiffness- and strength-related parameters of various unbound materials. Several studies have evaluated the performance of these devices to compare and correlate their results with those obtained using the conventional nuclear density gauge. Nevertheless, non-nuclear devices have not been adopted or widely implemented by state departments of transportation (DOTs). This report synthesizes useful knowledge and information from a variety of sources on national and international experiences and practices using non-nuclear devices and methods for compaction control of unbound materials. The information collected by this synthesis includes:

- Types of compaction control testing devices used by state DOTs, including construction specifications;
- Non-nuclear devices that have been evaluated by state DOTs and those under consideration, including proposed specifications;
- Various types of non-nuclear devices available and comparison of these devices with nuclear devices;
- Correlation of non-nuclear device measurement results to material properties (e.g., density, modulus, stiffness, moisture content);
- Issues with non-nuclear devices, such as accuracy, precision, ease of use, reliability of data, safety, test time, level of expertise required, Global Positioning System compatibility, calibration, durability, costs, and compatibility with various unbound materials; and
- The advantages, disadvantages, and limitations of the various compaction control devices.

Information in this synthesis was collected through a comprehensive literature review, surveys of U.S. DOTs and Canadian provincial transportation agencies, as well as interviews

with select state DOTs. A total of 41 transportation agencies (40 state DOTs and the Ontario Ministry of Transportation) responded to the survey questionnaire, which corresponds to a response rate of 80.4%. The main findings of the studies conducted to assess various non-nuclear devices are summarized in this report and used to compare their performance and identify their advantages and limitations. In addition, current DOT practices and procedures for compaction control of unbound materials are reviewed and documented. Finally, gaps in knowledge and current practices along with research recommendations to address these gaps are highlighted.

Analysis of survey responses indicated that the majority of DOTs are using field density and/or moisture content measurements obtained by the nuclear density gauge for compaction control of various types of unbound materials. However, the DOTs are interested in having a non-nuclear device that could replace the nuclear density gauge and could be used in compaction control of unbound materials. Some DOTs have evaluated non-nuclear density devices, including the electrical density gauge, the time domain reflectometry-based moisture density indicator, and the soil density gauge. The results of the literature review indicate that the non-nuclear density devices have some advantages over the nuclear density gauge, such as not requiring special licensing to operate them; however, these devices were found to be more difficult to use and require longer testing time. This may explain the consensus among survey respondents on not recommending the use of any of the available non-nuclear density devices.

There are several non-nuclear devices that have been used to measure the in situ moisture content of unbound materials; however, limited studies have been conducted to evaluate most of these devices. The speedy moisture tester and field microwave are the most common non-nuclear devices used to measure the in situ moisture content of unbound materials. According to the survey conducted in this study, 13 states have recommended their use but four did not. The main limitation of both devices is that they cannot be used for all types of unbound materials. The speedy moisture tester cannot be used for highly plastic clayey soil or coarse-grained granular soil. Furthermore, the field microwave is suitable only for materials consisting of particles smaller than 4.75 mm (0.19 in.).

Several DOTs have also assessed the performance of various in situ devices that measure stiffness/strength and can be used for compaction control of unbound materials. The evaluated in situ devices include Briaud compaction device (BCD), dynamic cone penetrometer (DCP), the Clegg hammer, GeoGauge, the light weight deflectometer (LWD), and the portable seismic property analyzer (PSPA). Among these devices, the DCP, GeoGauge, and LWD were the most evaluated by DOTs. The DCP and LWD have been implemented by some DOTs for compaction control of unbound materials. Previous studies indicated that all devices except the PSPA might have difficulties in establishing target field value in the laboratory because of boundary effects on their measurement accuracy. Therefore, several DOTs have attempted to establish those values based on pilot projects or by constructing control strips along a project. Some devices also have limitations on the type of unbound materials they can test. In addition, those devices apply different load magnitudes during the test, resulting in different measurement results. Although the results of in situ stiffness/strength devices were found to be affected by moisture content, none of these devices have the ability to measure it. The considered devices were reported to possess different influence depth. Thus, careful consideration should be given when analyzing their results and using them for compaction control. Several correlations were developed between the in situ test devices' measurements and design input parameters, such as the resilient modulus and California bearing ratio (CBR). However, those correlations are to be used with caution because they can be applied only to certain types of unbound materials and were developed for specified configurations of these devices. In general, no strong correlation was found between in situ stiffness/strength measurements and in-place density because their relationship continuously changes, depending on the moisture content.

According to the survey results, the majority of DOTs are interested in implementing stiffness- and strength-based specifications for compaction control of unbound materials, yet few DOTs have developed such specifications. This was attributed mainly to the lack of trained personnel and funds, the need for new testing equipment, and the unfamiliarity of contractors with such specifications. Only the Indiana and Minnesota DOTs have widely implemented stiffness- and strength-based specifications for compaction control using DCP and LWD measurements. Both states reported that they had positive experiences using the DCP as a tool for compaction control of unbound materials. Other states, such as Missouri, have used the DCP in compaction control but only for a specific type of unbound material.

CHAPTER ONE

INTRODUCTION

INTRODUCTION AND BACKGROUND

Compaction is defined as the process by which particles of soil and unbound aggregates, hereafter referred to as "unbound materials," are rearranged and packed together into a denser state by applying mechanical energy. As unbound materials reach a denser state through compaction, their shear strength and stiffness are enhanced. This makes them capable of resisting more stress with less deformation and thus prevents or reduces the development of detrimental excessive settlement during service. In addition, compaction helps to decrease the susceptibility of the unbound materials to environmental changes, especially those caused by frost heave, swelling, or shrinkage (Holtz et al. 2010). Therefore, proper compaction of unbound materials is one of the most critical components in the construction of pavements, airfields, and embankments to ensure their adequate performance, durability, and stability over time. Compaction control is used to ensure that proper degree of densification is achieved.

Appropriate control of the compaction process depends on compaction specifications. There are two main types of such specifications: (1) method or procedure specifications, and (2) end-product specifications. Both types have similarities in site preparation requirements and peripheral construction requirements, such as site drainage and runoff control, hours of work, and other contractual details. However, there are several differences between the two. With method or procedure specifications, the type, weight, and number of passes of compaction equipment, as well as the lift thicknesses (or maximum allowable material volume), are specified based on prior knowledge of the materials or field test sections. On the other hand, with end-product specifications, sometimes called performance specifications, the contractor is required to compact the soil layer to achieve a target density or stiffness value.

As shown in Table 1, most state departments of transportation (DOTs) currently employ end-product specifications for compaction control of unbound materials in pavement layers, subgrade, and embankments. DOTs assess the quality of compaction of those materials by comparing their field density measurements to a target dry density value. The target density value typically is determined by conducting a specified laboratory standard compaction test, such as the AASHTO T99 or AASHTO T180, on the same material compacted in the field. The nuclear density gauge (NDG) is the device used by most state DOTs for measuring the field density of compacted layers of unbound materials (Tutumluer 2013). However, this device contains radioactive materials that can be hazardous to the operators' health. Thus, its use entails extensive handling, storage, calibration, maintenance, and transportation regulations. All operators are required to undergo radiation safety training and know all applicable safety procedures and regulations. In addition, dosimeters (or film badges) are required to monitor the radiation when using the nuclear density device. Annual calibration and routine safety procedures are also needed to maintain the gauges. Use of the NDG also requires strict licensing and relicensing, record keeping, and storage. The costs associated with owning, operating, licensing, transporting, and maintaining NDGs can be prohibitive. Table 2 provides a summary of those costs (Cho et al. 2011). In addition, improper disposal of NDGs has resulted in environmental contamination incidents costing, in some cases, several million dollars in cleanup (TransTech Systems, Inc. 2008).

Because of the regulatory, safety, and economic burden associated with NDG use, as well as the Department of Homeland Security's desire to reduce the amount of radioactive material in common use, several non-nuclear devices for density measurement have been proposed and evaluated during the past decade. These devices use electrical methods and are based on new technologies, such as time domain reflectometry (TDR) and dielectrics. Various agencies, including FHWA and state DOTs, have evaluated the devices in laboratory and field studies and compared their performance with that of the conventional NDG. Nevertheless, the use of such devices has not been adopted or widely implemented by state DOTs.

Although the compaction control density method widely practiced by state DOTs is simple and relatively straightforward, it nonetheless presents a number of challenges for inspectors and designers. For example, the AASHTO T99 or AASHTO T180 test is limited in that it determines the required density of a variable material in only a very small sample. More compaction tests could be performed to increase the statistical confidence, but this approach is impractical because these tests are time consuming (Davich et al. 2006). In addition, the energy specified in the standard Proctor test method, first developed more than seven decades ago, does

TABLE 1
SUMMARY OF COMPACTION CONTROL SPECIFICATIONS OF STATE DOTs

State Earthwork Specification	Compaction Control Method	Type	Minimum Compaction Requirements	Loose Lift Thickness	Moisture Control Requirements	Alternative Methods
Alabama (2012)	Specified density	Embankment	95% RC (AASHTO T99: Method A for 10% passing or less; Method C for more than 10% retain on No. 4; Method D for 20% or more retained.)	8 in. (loose)	Strict moisture control will not be required.	N/A
		Modified or improved roadbed layer	100% RC (AASHTO T99: Method A for 10% passing or less; Method C for more than 10% retain on No. 4; Method D for 20% or more retained.)	—	OMC ± 2%	
		Base				
Alaska (2004)	Specified density	Embankment	95% RC (AASHTO T180 and AASHTO T224)	8 in. (loose)	OMC ± 2%	Compact until embankment does not rut under the loaded hauling equipment
		Subgrade				N/A
		Base	98% RC (AASHTO T180 and AASHTO T224)			
Arizona (2008)	Specified density	Embankment	100% of RC for top 6 in and beneath approach and anchor slabs, 95% RC for other (ADOT Material testing manual)	8 in. (loose)	At or near OMC	N/A
		Lime/cement treated subgrade	100% of RC (ADOT Material Testing Manual)	8 in. (loose)		
		Base				
Arkansas (2003)	Specified density	All type	95% Relative density (AASHTO T99: Method A for 10% passing or less; Method C for more than 11–30% retain on #4; and AASHTO T180 Method D for more than 30% or more retained)	10 in. (loose)	At OMC	N/A
		Subgrade		—		
		Cement treated base	95% of RC (AASHTO T134)	—	±5% of OMC	
California (2010)	Specified density	All types	95% RC (California Test 216 or 231)	8 in. (loose)	Suitable moisture content such that required density can be obtained and embankment stable	N/A
		Where 95% RC not required	90% RC (California Test 216 or 231)			
		Aggregate base/subbase	95% RC (California Test 216 or 231)			
Colorado (2011)	Specified density	Embankment	100% RC (AASHTO T99) or 95% RC (AASHTO T180	8 in. (loose)	OMC 2% (dry side)	Proof rolling
		Lime treated subgrade	95% RC (AASHTO T99)		±2% of OMC	—
		Aggregate base	95% RC (AASHTO T180)	—	—	
Connecticut (2004)	Specified density	Embankment	95% RC (AASHTO T180 Method D)	12 in. (loose)	At OMC	N/A
		Subgrade				
		Processed aggregate base				
Delaware (2001)	Specified density	Embankment	95% RC (AASHTO T99 Method C, Modified)	8 in. (loose)	±2% of OMC	N/A
		Subgrade				
		Aggregate base	98% RC (AASHTO T99 Method C, Modified)			
District of Columbia (2009)	Specified density	Embankment	95% RC full depth and top 6 in. of subgrade (AASHTO T180 Method D)	6 in. (loose)	Not specified	N/A
		Subgrade	95% RC (AASHTO T180 Method D)			
		Aggregate base	95% RC (AASHTO T180 Method D)			Control strip
Florida (2013)	Specified density	Embankment	100% RC (AASHTO T99 Method C)	12 to 6 in. (compacted)	Suitable moisture content such that required density can be obtained	N/A
		Subgrade	98% RC (AASHTO T180 Method C)	12 in. (compacted)		
		Base		4 to 8 in. (compacted)		

TABLE 1
(*continued*)

State Earthwork Specification	Compaction Control Method	Type	Minimum Compaction Requirements	Loose Lift Thickness	Moisture Control Requirements	Alternative Methods
Georgia (2010)	Specified density	Embankment	Full depth and 100 ft from bridge edge: 100 RC (GDT 7)	8 in.	At OMC	N/A
			Full depth 95% RC; top 1 ft 100% (GDT 7)			
		Subgrade	95% RC (GDT 7)			
		Base				
Hawaii (2005)	Specified density	Embankment	95% RC (AASHTO T180 Method D and T224)	9 in. (loose)	±2% of OMC	N/A
		Subgrade		N/A		
		Base				
Idaho (2012)	Specified density	Embankment	95% RC (AASHTO T99 Method A or C)	8 in. (loose)	+2/-4% of OMC	N/A
		Subgrade				
		Granular base				
Illinois (2012)	Specified density	Embankment	Height < 1-1/2 ft 95% RC (AASHTO T99 Method C and T224)	8 in. (loose)	Top 2 ft not more than 120% OMC such that adequate compaction is achieved.	N/A
			1-1/2 ft < height < 3 ft First lift 90% RC; remainder 90% RC (AASHTO T99 Method C and T224)			
			Height > 3 ft Lower 1/3 of the embankment to 90% RC; first lift above lower 1/3 to 93% with remainder to 95% RC (AASHTO T99 Method C and T224)			
	Specified density and DCP	Subgrade	95% RC (AASHTO T99 Method C and T224) with IBV based on DCP = 8 (Illinois Test Procedure 501)			N/A
	Specified density	Granular base	100% RC (AASHTO T99 Method C and T224)	8 in.		N/A
Indiana (2012)	Specified density or stiffness/ strength	Embankment	95% RC (AASHTO T99 Method C and T224) DCP or LWD target value	8 in. (loose)	-2% to +1% OMC -3% to 0% OMC for loessial soils	Proof rolling
		Subgrade	100% RC (AASHTO T99 Method C and T224) or DCP target value			
		Aggregate base	100% RC (AASHTO T99 Method C and T224) or LWD target value			
Iowa (2012)	Specified density and DCP	Embankment	95% RC (AASHTO T99 Method C) and DCP stability and uniformity limits	Variable, such that adequate compaction is achieved	Variable such that adequate compaction is achieved	N/A
	Roller walkout		Compacted a minimum of 1 pass per 1 in. of loose fill until the tamping feet penetrate 3 in. or less into an 8-in. lift	Variable	Variable	
	Specified density	Natural subgrade	95% RC (test method Iowa No. 103)	—	—	
		Base				
Kansas (2007)	Specified density	Embankment	As specified within construction plans	8 in. (loose)	Specified on construction plans unless approved by engineer	N/A
		Cement/fly ash treated subgrade	95% RC (AASHTO T99)		OMC ± 3%	
		Granular base	95% RC (AASHTO T99)		OMC ± 3%	
Kentucky	Specified density	Embankment	95% RC (KM 64-511)	12 in./3ft	OMC ± 2%	N/A
		Subgrade				
	Control strip	Base	98% RC, single point 95% (KM 64-511)	8 in.	—	
Louisiana (2006)	Specified density	Embankment	95% RC (DOTD TR415 or TR418)	Nonplastic material 15 in.	±2% of OMC	N/A
		Subgrade				
		Base		—		

(*continued on next page*)

TABLE 1
(*continued*)

<table>
<tr><th>State Earthwork Specification</th><th>Compaction Control Method</th><th>Type</th><th colspan="2">Minimum Compaction Requirements</th><th>Loose Lift Thickness</th><th>Moisture Control Requirements</th><th>Alternative Methods</th></tr>
<tr><td rowspan="3">Maine (2002)</td><td rowspan="3">Specified density</td><td>Embankment</td><td colspan="2" rowspan="2">90% of max density (AASHTO T180, Method C or D)</td><td>8 in. (loose)</td><td rowspan="3">Proper to maintain compaction and stability</td><td rowspan="3">N/A</td></tr>
<tr><td>Subgrade</td><td>—</td></tr>
<tr><td>Aggregate base and subbase</td><td colspan="2">95% RC (AASHTO T180, Method C or D)</td><td>—</td></tr>
<tr><td rowspan="3">Maryland (2008)</td><td rowspan="3">Specified density</td><td>Embankment</td><td colspan="2">1 ft below top of subgrade 92%; top 1 ft 97% (AASHTO T180)</td><td>8 in. (loose)</td><td rowspan="2">±2% of OMC</td><td rowspan="3">N/A</td></tr>
<tr><td>Subgrade</td><td colspan="2">97% RC (AASHTO T180)</td><td></td></tr>
<tr><td>Base</td><td colspan="2">97% (AASHTO T180)</td><td></td><td></td></tr>
<tr><td rowspan="3">Massachusetts (2012)</td><td rowspan="3">Specified density</td><td>Embankment</td><td colspan="2">95% Relative density (other than rock) (AASHTO T99, Method C)</td><td>12 in. (loose)</td><td rowspan="2">at OMC</td><td rowspan="3">N/A</td></tr>
<tr><td>Subgrade</td><td colspan="2" rowspan="2">95% RC (AASHTO T99, Method C)</td><td></td></tr>
<tr><td>Gravel base</td><td></td><td></td></tr>
<tr><td rowspan="2">Michigan (2012)</td><td rowspan="2">Specified density</td><td rowspan="2">Embankment</td><td>Cohesive material</td><td rowspan="2">95% RC (Michigan Density Testing and Inspection Manual)</td><td>9 in. (loose)</td><td>OMC to 4% OMC</td><td rowspan="2">Twelve-inch layer method</td></tr>
<tr><td>Granular material</td><td>15 in. (loose)</td><td>OMC to 5% OMC</td></tr>
<tr><td rowspan="4">Minnesota (2005)</td><td rowspan="4">Specified density or stiffness/ strength</td><td rowspan="2">Embankment</td><td>Upper 3 ft of embankment or portions adjacent to structures</td><td>100% RC (Minnesota Grading and Base Manual) DCP or LWD target limit</td><td>8 in. (loose)</td><td>65%–102% OMC</td><td rowspan="4">Quality compaction method; DCP for granular material</td></tr>
<tr><td>Below upper 3 ft and not adjacent to structures</td><td>95% RC (Minnesota Grading and Base Manual) DCP or LWD target limit</td><td>12 in.</td><td>65%–115% OMC</td></tr>
<tr><td>Subgrade</td><td colspan="2" rowspan="2">100% RC (Minnesota Grading and Base Manual)
DCP or LWD target limit</td><td rowspan="2"></td><td></td></tr>
<tr><td>Base</td><td>65% of OMC to OMC</td></tr>
<tr><td rowspan="4">Missouri (2013)</td><td rowspan="4">Specified density</td><td rowspan="2">Embankment</td><td>Embankments more than 50 ft below the top of finished subgrade, within 100 ft of structures, or within 18 in. of subgrade</td><td>95% Relative Compaction (AASHTO T99, Method C)</td><td rowspan="2">8 in. (loose)</td><td>Such that adequate compaction is achieved</td><td rowspan="3">DCP index for Type 7 base</td></tr>
<tr><td>All other embankments unless otherwise noted</td><td>90% Relative Compaction (AASHTO T99, Method C)</td><td>0% to +3% OMC for loessial soils</td></tr>
<tr><td>Subgrade</td><td colspan="2">95% RC (AASHTO T99, Method C)</td><td></td><td>MC for adequate compaction is achieved</td></tr>
<tr><td>Aggregate Base</td><td colspan="2">95% (AASHTO T99, Method C)</td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Mississippi (2004)</td><td rowspan="2">Specified density</td><td>Embankment</td><td colspan="2">95% to 98% of maximum density</td><td>8 in. (loose)</td><td>Such that adequate compaction is achieved</td><td rowspan="2">N/A</td></tr>
<tr><td>Base</td><td colspan="2">Average of five at least 93% of maximum density, no single density below 89% of maximum density</td><td></td><td></td></tr>
<tr><td rowspan="3">Montana (2006)</td><td rowspan="3">Specified density</td><td>Earth embankment B including all backfills</td><td colspan="2">90% of maximum density (AASHTO T99)</td><td>8 in. (loose)</td><td rowspan="2">±2% of OMC</td><td rowspan="3">N/A</td></tr>
<tr><td>Subgrade</td><td colspan="2">95% of maximum density (AASHTO T99)</td><td></td></tr>
<tr><td>Aggregate base</td><td colspan="2">98% of maximum density (MT-230)</td><td></td><td></td></tr>
<tr><td rowspan="3">Nebraska (2007)</td><td rowspan="3">Specified density</td><td>Embankment</td><td colspan="2">95% RC (NDR T99)</td><td>8 in. (loose)</td><td></td><td rowspan="3">N/A</td></tr>
<tr><td>Stabilized subgrade</td><td colspan="2">100% RC (NDR T99)</td><td></td><td></td></tr>
<tr><td>Base</td><td colspan="2">—</td><td></td><td></td></tr>
<tr><td rowspan="2">Nevada (2001)</td><td rowspan="2">Specified density</td><td>5 ft or less in height</td><td colspan="2">90% RC (test method Nevada No. T101)</td><td rowspan="2">8 in. (loose)</td><td rowspan="2">Not specified</td><td rowspan="2">N/A</td></tr>
<tr><td>All other (including subgrade and aggregate base)</td><td colspan="2">95% RC (test method Nev. No. T101)</td></tr>
</table>

TABLE 1
(*continued*)

State Earthwork Specification	Compaction Control Method	Type	Minimum Compaction Requirements		Loose Lift Thickness	Moisture Control Requirements	Alternative Methods
New Hampshire (2010)	Specified density	Embankment	98% RC (beneath approach slab and 10 ft back of a structure); all other 95% (AASHTO T99)		12 in. (loose)	Not specified	N/A
		Subgrade	95% Relative density (AASHTO T99 or control strip)				
		Base			8 in.		
New Mexico (2007)	Specific density	Embankment	95% Relative density (AASHTO T99 Method C)		8 in. (loose)	OMC to OMC –5%; for soil with PI > 15 OMC to OMC +4%	N/A
		Subgrade	Top 6 in. 100%; soil with PI³15: 95% (AASHTO T99, Method C)				
		Base	95% of maximum density (AASHTO T180)		6 in.		
New Jersey (2007)	Specified density	Embankment	95% Relative density (AASHTO T99, Method C)		12 in. (compacted)	Not specified	End dumping method, control fill method, direct method
		Subgrade					Proof rolling for subgrade
	Control strip	Base	Q³ 0.36 (where Q = (Average lot density– 0.95 * Maximum Density)/Range of Lot Density				
New York (2008)	Specified density	Embankment	90% minimum; 95% minimum for subgrade of embankment (AASHTO T99)		Depends on compaction device	Not specified	N/A
		Subgrade					
		Soil cement base	95% of maximum density (AASHTO T99)				
North Carolina (2012)	Specified density	Embankment	95% RC [AASHTO T99 (state modified)]		10 in. (loose)	Not specified	Proof rolling
		Subgrade	100% RC [AASHTO T180 (state modified)]		8 in.		
		Base	Aggregate base	100% RC [AASHTO T99 (state modified)]			N/A
			Cement treated	97% RC [AASHTO T99 (state modified)]		+1.5% of OMC	
			Lime treated	97% RC [AASHTO T99 (state modified)]		+ 2% of OMC	
North Dakota (2008)	Specified density	Embankment	95% RC (AASHTO T99) / 90% (AASHTO T180)		12 in. (loose)	-4% of OMC to +5% of OMC (AASHTO T99) / OMC to +5% of OMC (AASHTO T180)	N/A
		Subgrade			—		
	Specification method	Base	—	—	—	—	
Ohio (2013)	Specified density	Embankment	γ_{max} (pcf)	% RC	8 in. (loose)	Suitable moisture content	N/A
			90 to 104.9	102			
			105 to 119.9	100			
			120 and more	98			
		Subgrade	AASHTO T99, T272 and supplemental specification 1015)				
	Test strip	Aggregate base	98% RC of test section			±2% of OMC	
Oklahoma (2009)	Specified density	Embankment	95% RC (AASHTO T99 Methods C or D)		8 in. (loose)	±2% of OMC	N/A
		Subgrade					
		Aggregate base	98% (Type A); 95% for Types B, C, and D				
Oregon (2008)	Specified density or Deflection	Embankment	95% RC (AASHTO T99)		8 in. (loose)	-4% of OMC to OMC +2%	Proof rolling using ODOT TM 158
		Subgrade	—				
		Aggregate base	100% RC				
Rhode Island (2010)	Specified density	Top 3 ft of embankment	90% RC (AASHTO T180 and T224)		12 in. (loose)	Not specified	N/A
		Reminder up to subgrade and subgrade	95% RC (AASHTO T180 and T224)				
		Aggregate base	95% RC (AASHTO T180 and T224)				

(*continued on next page*)

TABLE 1
(*continued*)

State Earthwork Specification	Compaction Control Method	Type	Minimum Compaction Requirements	Loose Lift Thickness	Moisture Control Requirements	Alternative Methods
South Carolina (2007)	Specified density	Embankment	95% RC (SC-T-29)	8 in.	Suitable moisture content	N/A
		Subgrade				
		Base	100% RC (SC-T-29)		±2% of OMC	
South Dakota (2004)	Specified density	Embankment	95% or greater (AASHTO T99)	8 in. (loose)	For OMC 0–15% = > OMC -4% to OMC +4%; for OMC > 15% => OMC -4% to OMC +6%	Ordinary compaction method
		Subbase and base	Base 97% RC; subbase 95% (D 104, Method 4 and SD 105 or SD 114)		Not specified	N/A
Tennessee (2006)	Specified density	Embankment	95% RC (AASHTO T99 and T224)	10 in. (loose)	± 3% of OMC	N/A
		Base	Aggregate base: 100% RC, 92 (single, minimum) (AASHTO T99 and T224)			
		Subgrade (lime treated)	Average per lot 96% (minimum), single determination 92% (minimum) (AASHTO T99 and T224)		±3% of OMC	
Texas (2004)	Specified density	Embankment	PI ≤ 15 = > density ≥ 98%; 15 < PI ≤ 35 => density ≥ 98% and ≤ 102% ; W ≥ Wopt PI > 35 => density ≥ 95% Da and ≤ 100% Da; W ≥ Wopt. (Tex-114-E)	16 in. (loose)	15 < PI ≤ 35 => D W ≥ Wopt PI > 35 => W ≥ Wopt	N/A
		Subgrade	Lime treated or fly ash treated: 95% of maximum density (Tex-114-E);			
		Lime treated	98% of maximum density (Tex-114-E)			
Utah (2012)	Specified density	Embankment	Average per lot 96% (minimum), single determination 92% (minimum) (AASHTO T180 Method D for A-1 soils; AASHTO T99 Method D for all other soils)	12 in. (loose), may be reduced if unsatisfactory density	Appropriate moisture	N/A
		Untreated base course	Average 97% RC (minimum); single point 94% RC (AASHTO T180 Method D for A-1 soils; AASHTO T99 Method D for all other soils)		OMC ± 2%	
Vermont (2006)	Specified density	Embankment	Top 24 in. immediately above subgrade 95%; rest 90% relative density (AASHTO T99, Method C)	8 in. (loose)	OMC + 2%	N/A
		Subgrade	95% of maximum density (AASHTO T99, Method C)			
		Subbase and base	95% RC (AASHTO T180, Method D)		Not specified	
Virginia (2007)	Specified density	Embankment	95% RC	8 in. loose	±2% of OMC	N/A
		Subgrade	% Retained No. 4 / RC 0–50 / 100 51–60 / 95			
		Base	61–70 / 90		±2% of OMC	Control strip
Washington (2012)	Specified density	Embankment	Method A: Not specified Method B: 95% RC; height < 2 ft 90% RC Method C: 95% RC	8 in.	Method A: suitable moisture content Method B: +3% of OMC; Method C: ±3% of OMC	N/A
		Subgrade	95% RC (% retained on No. 4: if 30 maximum (AASHTO T99 Method A); if more than 30% on # and 30% or less retained on 3/4 sieve (T180 Method D); if 30% or more on 3/4 sieve (WSDOT Method No. 606)	6 in.		
		Aggregate base				
West Virginia (2010)	Specified density	Embankment	95% RC (WV specifications: MP 717.04.21, MP 207.07.20, MP 700.004.2, etc.)	12 in. (loose) (40% or higher passing 3/4 in. sieve)	-4% of OMC to +3% of OMC (40% passing 3/4 in. sieve), OMC (elastic soils)	N/A
		Subgrade and base				

TABLE 1
(*continued*)

State Earthwork Specification	Compaction Control Method	Type	Minimum Compaction Requirements		Loose Lift Thickness	Moisture Control Requirements	Alternative Methods
Wisconsin (2013)	Specified density	Embankment	Embankments less than or equal to 6 ft or within 200 ft of a bridge abutment	95% RC (AASHTO T99, Method C)	8 in. (loose)	Such that the material does not rut excessively and such that the material can be compacted properly	N/A
			Embankments higher than 6 ft	6 ft below subgrade: 90% RC; within 6 ft from top: 95% RC (AASHTO T99, Method C)			
		Subgrade	Final 6 in.	95% RC (AASHTO T99, Method C)			
Wyoming (2010)	Specified density	Embankment	95% RC (Materials Testing Manual WY DOT and AASHTO T180)		8 in. (loose)	-4% of OMC to +2% of OMC	N/A
		Subgrade					
		Base					

RC = relative compaction; OMC = optimum moisture content; γmax: = maximum dry density, N/A = not available.

not accurately represent the compaction energy levels currently applied in the field (Davich et al. 2006).

Other problems with the current density-based compaction control method arise from the design and performance perspective. Although compaction of unbound materials results in increased density, the main purpose of compaction is to improve the materials' engineering properties, not only their density. The key functional properties of unbound layers are their stiffness and strength, which are measures directly related to their structural performance. Stiffness or strength parameters of unbound materials typically are used in the design of different transportation structures, such as pavements, but are not evaluated during the compaction process. Consequently, there is a missing link between the design and compaction quality control processes. To address this problem, federal and state transportation agencies have investigated the use of compaction control procedures for unbound materials that are based on a criterion that closely correlates to the performance parameters used in the design, such as stiffness and strength. This effort was also motivated by the development and implementation of the *Mechanistic–Empirical Pavement Design Guide* (MEPDG). Currently, several in situ test devices are reported to measure the stiffness or strength properties of compacted unbound materials and are robust and accessible to different construction sites. These include the Briaud compaction device (BCD), Clegg hammer (CH), dynamic cone penetrometer (DCP), GeoGauge, light weight deflectometer (LWD), soil compaction supervisor (SCS), and portable seismic property analyzer (PSPA).

TABLE 2
COSTS ASSOCIATED WITH OWNING AND OPERATING
A NUCLEAR DENSITY GAUGE

Item	Cost
Cost of nuclear gauge	$6,950
Radiation safety and certification class	$750
Safety training	$179
Hazardous materials certification	$99
RSO training	$395
TLD Badge Monitoring	$140/year
Life of source capsule integrity	15 year
Maintenance and recalibration	$500/year
Leak test	$15
Shipping	$120
Radioactive materials license	$1,600
License renewal	$1500/year
Reciprocity	$750

Source: Cho et al. 2011.

At present, few state DOTs have included these devices in their specifications for compaction control of unbound materials or have implemented their use in field projects.

A national synthesis of previous research is needed to evaluate and compare the performance of various available non-nuclear methods and devices that can be used in compaction control of unbound materials. This will allow state DOTs to understand the different non-nuclear methods that exist and know their advantages and limitations, which will help in implementing those methods and devices. This report synthesizes useful knowledge and information from a variety of sources on the national and international experience in using non-nuclear methods for compaction control of unbound materials. The information collected by this synthesis includes the following:

- Types of compaction control testing devices used by state DOTs, including construction specifications;
- Non-nuclear devices that have been evaluated by state DOTs and those under consideration, including proposed specifications;
- The various types of non-nuclear devices available and comparisons with nuclear devices;
- Comparison of measurements of non-nuclear device results to material properties (e.g., density, modulus, stiffness, moisture);
- Issues with non-nuclear devices, such as accuracy, precision, ease of use, reliability of data, safety, test time, level of expertise required, Global Positioning System (GPS) compatibility, calibration, durability, costs, and compatibility with various unbounded materials; and
- Advantages, disadvantages, and limitations of the various compaction control devices.

STUDY OBJECTIVES AND SCOPE

The main objective of this synthesis is to compile and summarize all available information on the various non-nuclear devices and methods that have been used for compaction control of unbound materials. The synthesis focuses on non-nuclear devices that measure density, as well as those that evaluate in situ stiffness- and strength-related properties of unbound materials that can be used to examine the quality of construction. Information on the accuracy, repeatability, ease of use, test time, cost, GPS compatibility, calibration, compatibility with the various unbound materials, and depth of influence of the different non-nuclear compaction control devices is documented and discussed. In addition, the main advantages, disadvantages, and limitations of those devices are identified. All correlations between the measurements of the considered devices and density, as well as the resilient modulus or any other input parameter for designing transportation and geotechnical structures, are provided. The synthesis presents a review of stiffness- and strength-based specifications that have been developed and implemented by state DOTs for compaction control to unbound materials. Finally, it highlights gaps in knowledge and current practices, along with research recommendations to address these gaps.

STUDY APPROACH

Various methods were used to collect information in this synthesis. They include a comprehensive literature review, a survey of materials/geotechnical engineers from state DOTs, and selected interviews. The following sections describe those methods.

Survey Questionnaire

A survey questionnaire was prepared and distributed to the materials/geotechnical engineers from all state DOTs and the Ontario Ministry of Transportation in Canada. The questionnaire was designed to be both comprehensive and brief in an attempt to increase response rate. More details about the steps followed in conducting this survey are provided in chapter two.

Literature Review

A comprehensive literature review of all published national and international materials and ongoing research projects focusing on the performance of various non-nuclear devices and methods used for compaction control of different unbound materials used in pavements, embankments, and foundations was conducted. The literature search included standard methods, such as TRB's Transportation Research Information System (TRIS), COPENDEX, National Technical Information Service (NTIS), as well as consulting with domestic and national experts in the field. Research reports on studies conducted by the FHWA, U.S. Army Corps of Engineers, and state DOTs on non-nuclear methods and devices for compaction control were reviewed. In addition, information was obtained from the state DOTs construction specification books and manuals. The literature review results are presented in chapters three through five of the synthesis report.

Interviews

Interviews were performed over the phone and by e-mail with selected survey respondents to seek additional details about their experience with using non-nuclear methods for compaction control of unbound materials and clarify any discrepancies found in the questionnaire. In addition, those interviews have helped in developing the case examples for the implementation of stiffness- and strength-based specifications for compaction control of unbound materials in some states. Those are presented in chapter five of this report.

REPORT ORGANIZATION

This report is organized into six chapters. Chapter two presents the description, the respondents' information, and the results of the survey questionnaire conducted as part of this synthesis. This information is included in chapter two before other chapters primarily to provide an overview of current state DOTs' practices for compaction control of unbound materials and to give a picture of how state DOT engineers perceive the available non-nuclear compaction control devices. Chapter three discusses current state DOT specifications and practices for compaction control of various unbound materials that are based on density measurement. In addition, this chapter describes the various available non-nuclear density devices and provides a summary of the finding of studies that were conducted to evaluate them. Chapter four provides a description of various in situ test methods that measure stiffness or strength properties and have been used in compaction control of unbound materials. In addition, it summarizes the findings of the studies conducted to evaluate those devices and highlights the main advantages and limitations of those devices. Chapter five presents a review of stiffness- and strength-based compaction control specifications that have been developed or are being developed by state DOTs. Finally, chapter six summarizes the key findings and main conclusions from the literature and survey information compiled in chapters two through five. It also provides recommendations for future study and additional research needs.

CHAPTER TWO

CURRENT PRACTICES FOR COMPACTION CONTROL OF UNBOUND MATERIALS

INTRODUCTION

This chapter presents the pursued approach, summary of the responses, and key findings of the survey conducted in this study to collect information from state DOTs and Canadian provincial transportation agencies on their practices related to compaction control of unbound materials and their experiences with different non-nuclear devices considered in this synthesis.

SURVEY OVERVIEW

A survey questionnaire was prepared and distributed to the materials/geotechnical engineers from all state DOTs and the Ontario Ministry of Transportation in Canada. The main objectives of this survey were to (1) identify the current practices of various DOTs with respect to compaction control of unbound materials, and (2) learn about the DOTs' experiences with different non-nuclear density, as well as stiffness- and strength-based compaction control methods and devices that they have evaluated, used, or implemented. The survey consisted of 33 close-ended, multiple choice-type questions, which were implemented using the TRB survey software for distribution as an online survey. A copy of the survey is provided in Appendix A. A unique link was then created and e-mailed on January 23, 2012, to materials/geotechnical engineers from 50 state DOTs and the Ontario Ministry of Transportation in Canada. The survey was kept open until March 4, 2013. Those who did not start the survey after three weeks were contacted by phone and asked to complete it. A weekly reminder e-mail was sent to participants who did not respond. A few respondents were subsequently contacted by phone to clarify any discrepancies found in their questionnaire answers or to obtain additional information.

SURVEY RESULTS

The survey was sent to 50 state DOTs and the Ontario Ministry of Transportation in Canada, and a total of 41 responses (80.4%) were received. In the survey analysis presented in this chapter, the percentages were computed based on the 41 responses received. Salient survey findings are presented here; additional survey details can be found in Appendix C.

Figures 1 through 3 present the number of respondents that dealt with different types of unbound materials in compacted subgrade soils, base course layers, and embankments, respectively. Respondents were allowed to choose more than one type of unbound materials. Therefore, the overall numbers do not add up to the total number of respondents (i.e., 41). The majority of state DOTs have encountered low and high plasticity clay and silt, as well as sands, in their subgrade soils and embankments. In addition, the majority of states have used gravel, limestone aggregate, and recycled hot-mix asphalt (HMA) and portland cement concrete (PCC) in their base course layers. Approximately 40% (17 of 41) of respondents have used recycled HMA and PCC in their embankments.

Based on the results of the survey, Figure 4 presents a review of state DOT practices related to field compaction and construction quality control of unbound materials. More than 75% of responding state DOTs indicated that they require a minimum relative compaction higher than 90% in accepting compacted subgrade soils, unbound base course, and embankment layers. Most of these DOTs use AASHTO T99 and AASHTO T180 or a modified version of those standards to establish the target field density value. However, three state DOTs (Delaware, Ohio, and South Dakota) indicated that they use the one-point Proctor test based on the family of curves that they have developed to determine the target field density value. Although about half of respondents indicated that they specify moisture content limits in their compaction control acceptance criterion for base and subgrade soil, more than 65% include those limits for embankments.

Most of those DOTs require that field moisture content be within ±2% of the optimum moisture content. The staff of two state DOTs (Minnesota and Indiana) mentioned that they use DCP and/or LWD in their compaction control specifications for subgrade soils and base course layers, but only Indiana DOT uses DCP for compaction control of embankment layers. Furthermore, three state DOTs (Rhode Island, Utah, and Wisconsin) indicated that they do not have formal acceptance criteria for compacted base course layers and embankments. Finally, although five respondents do not have any compaction control requirements for subgrade soils, some state DOTs, such as those of New Jersey and Oregon, use proof rolling to examine the quality of compacted subgrade soils.

Figure 5 presents the results of survey questions related to the use of state DOTs for intelligent compaction in field projects. Although 24 states evaluated or demonstrated the

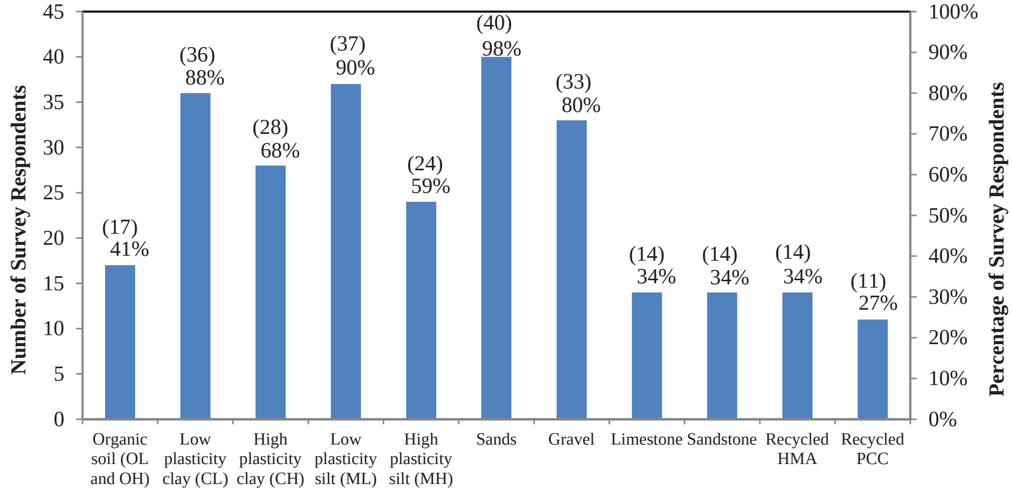


FIGURE 1 Survey results for types of unbound materials used in compacted subgrade soils.

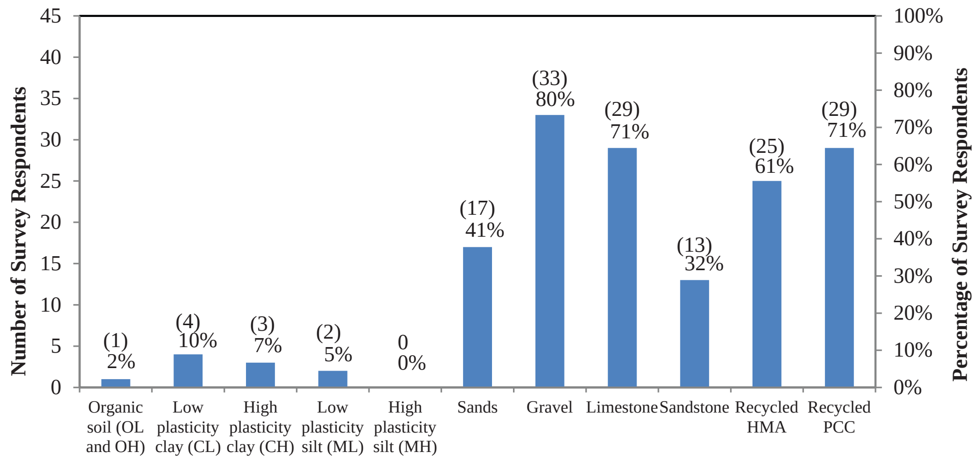


FIGURE 2 Survey results for types of unbound materials used in base course layers.

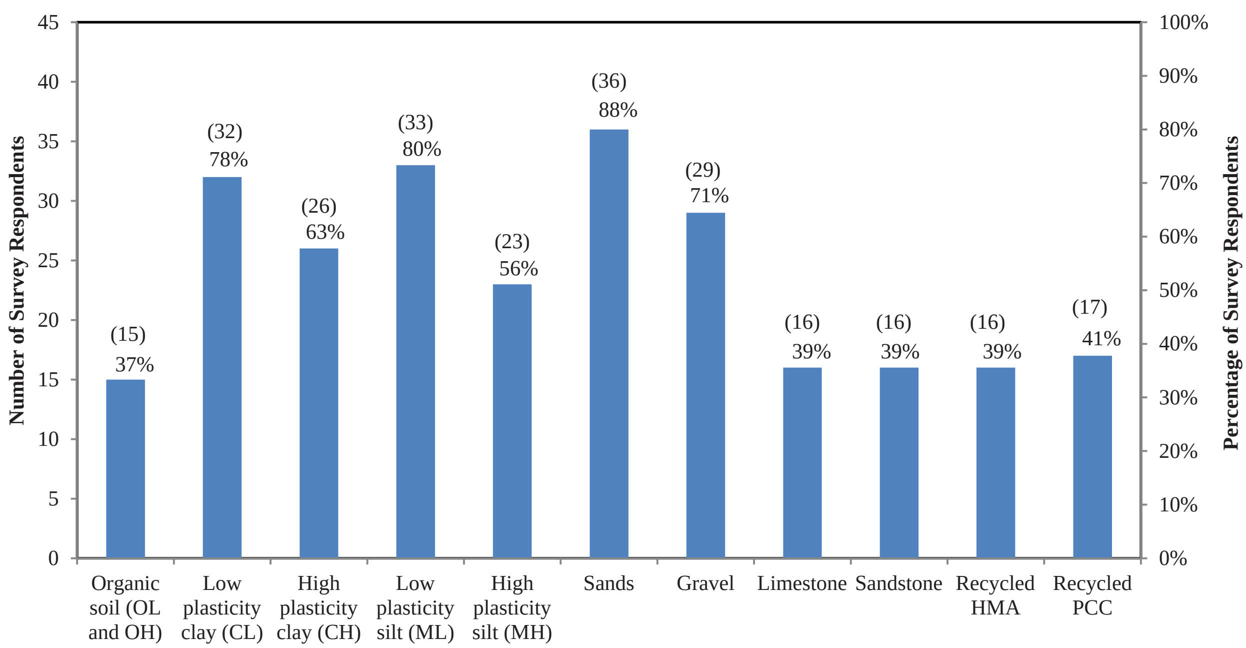


FIGURE 3 Survey results for types of unbound materials used in embankments.

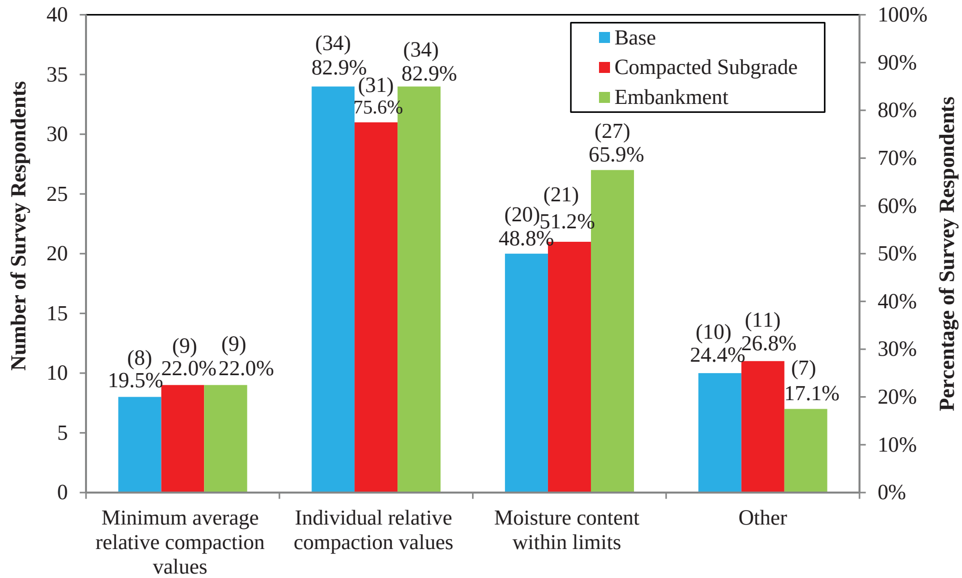


FIGURE 4 Survey results for type of compaction control method.

use of intelligent compaction, only two state DOTs (Indiana and Minnesota) have actually implemented its use in field projects. Furthermore, 13 states indicated that they plan to use it in the future.

Figure 6 presents survey results regarding DOTs' interest in using non-nuclear density devices. Only four DOTs indicated that they are not interested in using non-nuclear density devices. The rest indicated interest but also noted obstacles that could stop or impede the implementation of such devices. According to Figure 7, slightly more than 50% of respondents (21 of 41) indicated a need for new non-nuclear density devices. However, 12 DOTs cited lack of confidence in the performance and reliability of currently available devices as a prime reason impeding their implementation. In addition, 20 DOTs found it problematic that contractors tended to be unfamiliar with available non-nuclear density device technology.

Figure 8 presents the number of DOTs that have evaluated or used each of the available non-nuclear density devices. The majority of respondents (29 of 41) have not used or evaluated any such device. The electrical density gauge (EDG) was the most evaluated device among respondents. This evaluation was mainly done through in-house research studies. Figure 8 also shows that less than 15% of respondents evaluated the moisture density indicator (MDI) and soil density gauge (SDG).

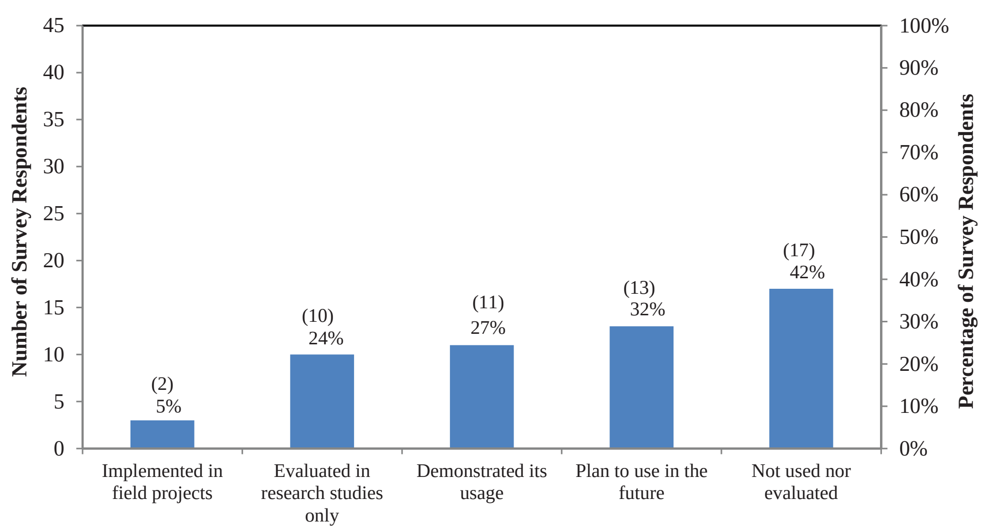


FIGURE 5 Survey results regarding intelligent compaction.

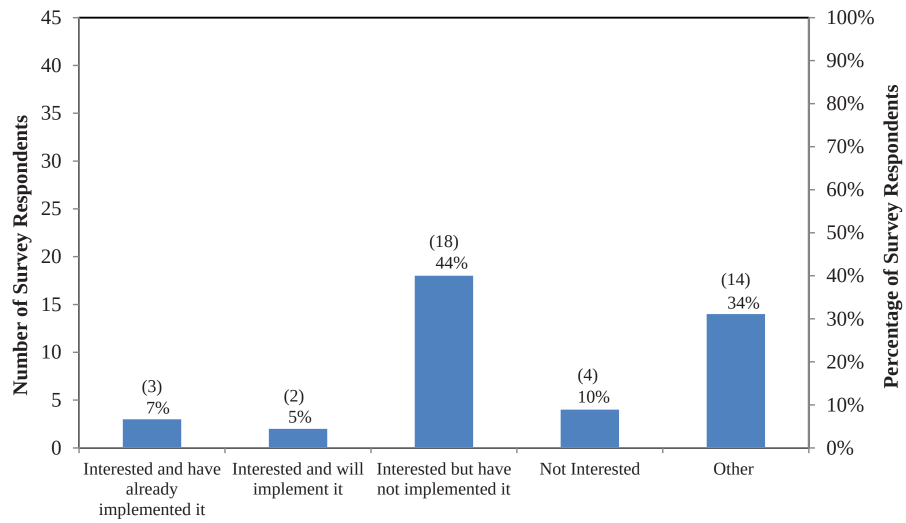


FIGURE 6 Respondents' interest in using non-nuclear density devices.

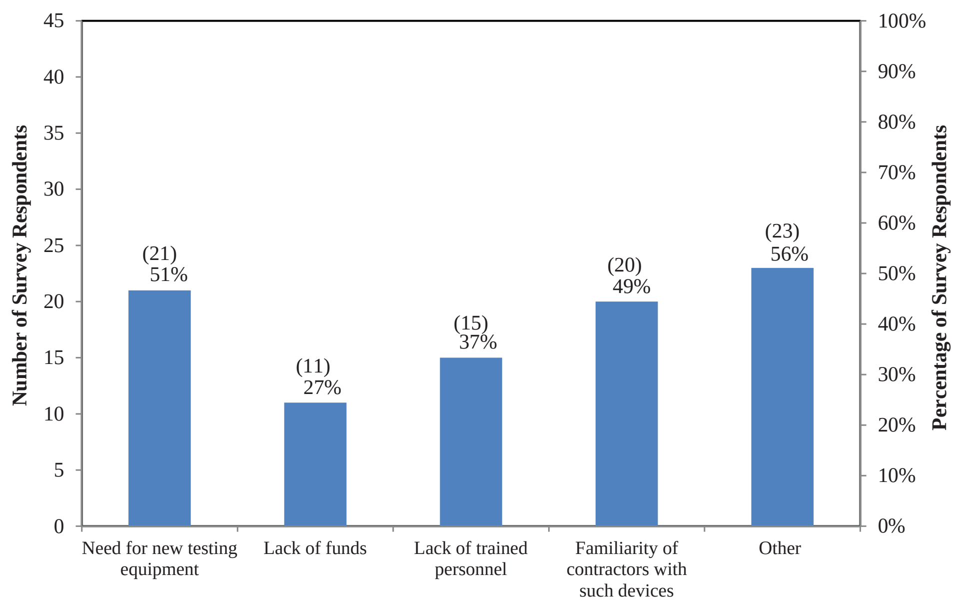


FIGURE 7 Main obstacles impeding implementation of non-nuclear density devices.

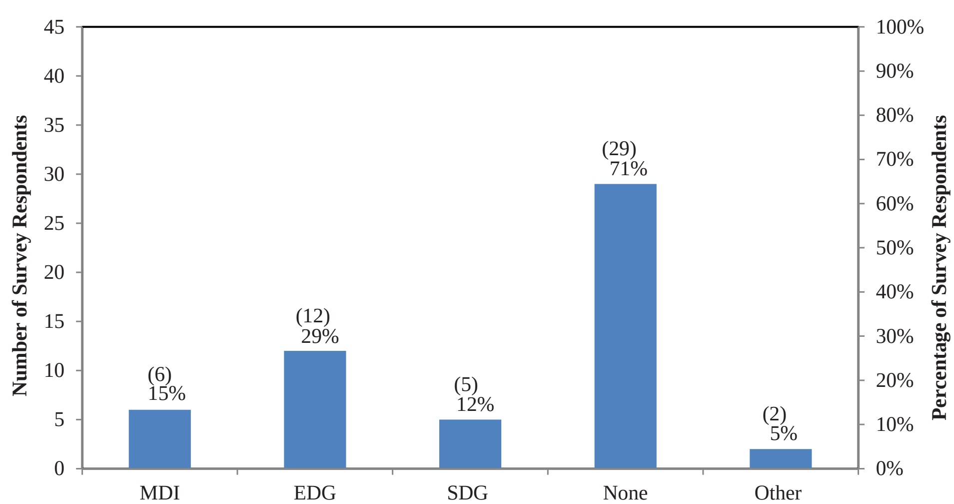


FIGURE 8 Number of respondents that used or evaluated non-nuclear density devices.

Figure 9 presents survey results on the experience of respondents with non-nuclear density devices. More than half of the respondents who evaluated the EDG, MDI, and SDG devices indicated that those devices are slightly complex or complex compared with the NDG. In general, state DOTs that evaluated the MDI, EDG, and SDG devices found their calibration procedure to be time consuming. In addition, whereas the majority of them (4 of 5) found the testing time of the SDG to be moderately short, there was no consensus about the testing time of the MDI and EDG. It is clear that most respondents who evaluated the EDG (9 of 12), MDI (4 of 6), and SDG (5 of 6) devices thought that intermediate or high-level expertise was required to operate those devices. Although most state DOTs did not have information about the cost, durability, and GPS compatibility of the MDI, EDG, and SDG, all of them agreed that those devices required fewer safety precautions than did the nuclear density device. Most DOTs that evaluated the MDI, EDG, and SDG devices found the accuracy and repeatability of those devices to be fair or poor, suggesting a lack of confidence in their reliability. This may explain the consensus among respondents of not recommending the use of any of the available non-nuclear density devices.

Figure 10 presents the DOTs' responses with respect to devices that measure strength- or stiffness-related properties and have been used or evaluated for compaction control of unbound materials. Among the currently available in situ test devices, the DCP (20), GeoGauge (19), and LWD (13) are the most evaluated and used devices among respondents. These evaluations were done through research conducted in house, as well as by universities and consultants. In addition, whereas only one DOT evaluated the BCD and PSPA, six DOTs evaluated the Clegg hammer. Finally, 15 of the 41 DOTs did not evaluate any of the listed devices. A few DOTs indicated that they have developed or currently are developing specifications for the DCP (5) and LWD (3). However, only two state DOTs (Indiana and Minnesota) have implemented the use of the DCP and the LWD in field projects.

Figures 11 through 13 present survey results regarding DOTs' experiences with different in situ stiffness devices. In terms of ease of use, the majority of respondents who evaluated the Clegg hammer (5 of 6), GeoGauge (12 of 18), DCP (16 of 19), and LWD (7 of 12) indicated that those devices were easy or moderately easy to use. In addition, while at least 50% of respondents who evaluated the Clegg hammer and DCP indicated that little experience was needed to operate those devices, the majority found that intermediate-level experience was needed to perform LWD tests. There was no consensus on the level of experience needed for the GeoGauge.

The majority of DOTs (12 of 20) found the calibration of the DCP to be simple and quick, but there was no consensus about the LWD, GeoGauge, or Clegg hammer. According to Figure 11, most of the respondents who evaluated the DCP, LWD, GeoGauge, and Clegg hammer found the testing time

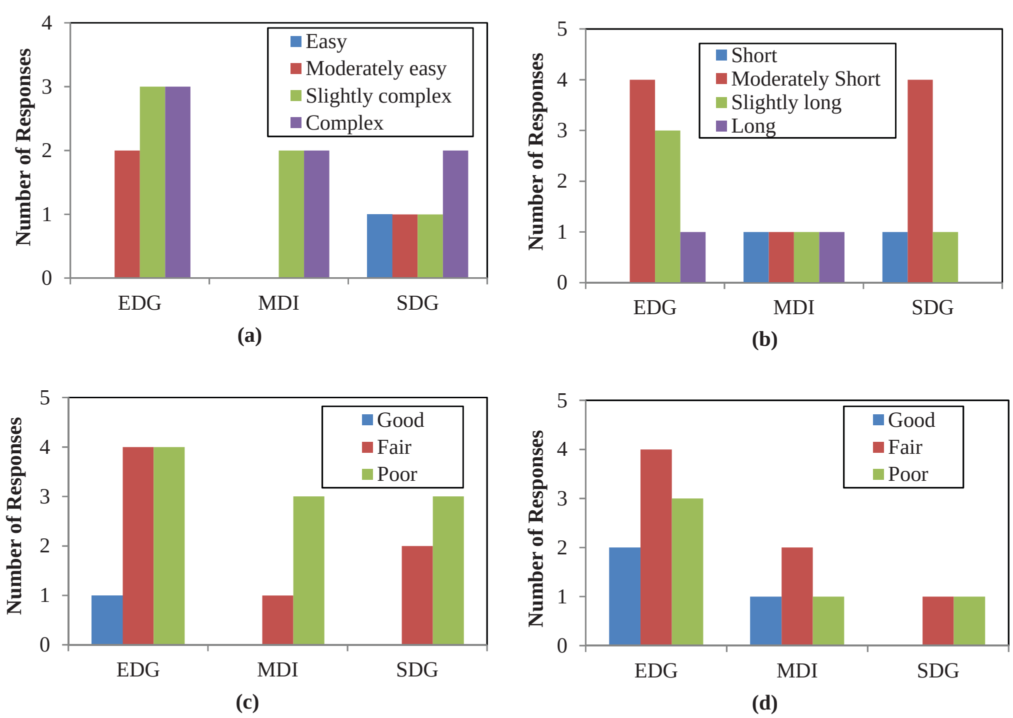


FIGURE 9 Experience of DOTs with the non-nuclear density devices: (*a*) ease of use, (*b*) calibration, (*c*) accuracy, and (*d*) repeatability.

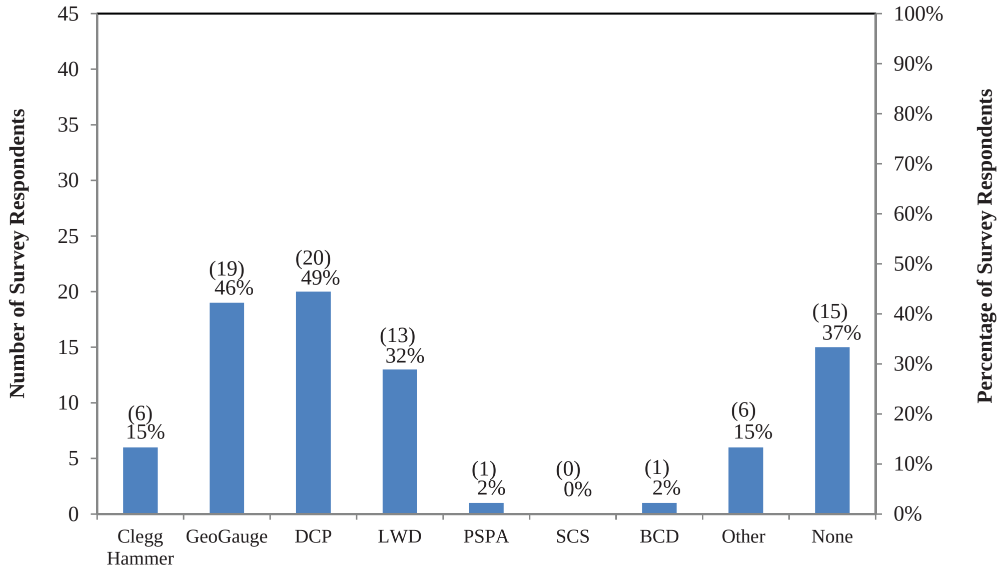


FIGURE 10 In situ devices used or evaluated for compaction control of unbound materials.

of those devices to be short or moderately short. In terms of cost, at least 60% (12 of 20) of DOTs that evaluated the DCP found that it was inexpensive. Furthermore, the majority of the DOTs that used the LWD found it was expensive or moderately expensive. Finally, about 30% of DOTs reported that the GeoGauge (7 of 19) and Clegg hammer were moderately expensive.

In terms of durability, at least half of respondents that evaluated the DCP (14 of 20), LWD (7 of 13), and Clegg hammer (3 of 6) indicated that the durability of those devices was good or very good. Roughly one-third (6 of 19) indicated that the GeoGauge had good durability. The majority of state DOTs considered the DCP (15 of 20), GeoGauge (12 of 19), LWD (8 of 13), and Clegg hammer (3 of 6) to be safe or moderately safe devices. With regard to GPS compatibility, whereas none of the state DOTs indicated that the GeoGauge and Clegg hammer were GPS compatible, a limited number had GPS-compatible DCP (1 of 20) and LWD (4 of 13) devices.

As for repeatability and accuracy, around 50% of respondents who evaluated the DCP (12 of 20) and the LWD (6 of 13) indicated that those devices were very good or good. By contrast, about half of the DOTs that evaluated or used the GeoGauge found it to have poor to fair accuracy and repeatability. Finally, there was no agreement among state DOTs about the repeatability and accuracy of the Clegg hammer.

According to Figure 13, more than 50% of respondents who evaluated the DCP (11 of 20) and LWD (9 of 13) recommended using them for compaction control of unbound materials. Two respondents (Colorado and Florida) recommended the Clegg hammer; however, no one recommended using the GeoGauge.

As shown in Figure 14, survey results indicate that more than 50% of the state DOTs that evaluated or used the DCP and LWD found them to be compatible with various types of unbound materials. However, a lower percentage indicated that the DCP is compatible with unbound base materials as compared with sand and fine-grained materials. In addition, fewer DOTs found the LWD to be compatible with fine-grained soils as compared with the other listed types of unbound materials. According to Figure 14, higher percentages of DOTs that evaluated the GeoGauge indicated that it is compatible with fine-grained soil (37%) and sands (42%) as compared with unbound base materials (21%).

Figure 15 presents the results of the survey question on the level of interest for implementing stiffness- and strength-based compaction control specifications. The majority of respondents (27 of 41) are interested in implementing stiffness- and strength-based specifications for compaction control of unbound materials. However, only two state DOTs (Indiana and Minnesota) have implemented such specifications.

As for the reason that may stop or impede implementation, 18 DOTs indicated that such reasons include the need for new testing equipment, lack of trained personnel, and familiarity of contractors with such devices. In addition, 10 DOTs indicated lack of funds as a reason that might stop the implementation of stiffness devices. Some DOTs also indicated that the effect of moisture on in situ stiffness/strength measurements must be addressed to implement a stiffness-based specification. Currently, four state DOTs (Missouri, Indiana, Illinois, and Minnesota) have stiffness- and strength-based production specifications for compaction control. Four additional states have developmental or experimental specifications. The staff of the Indiana and Illinois

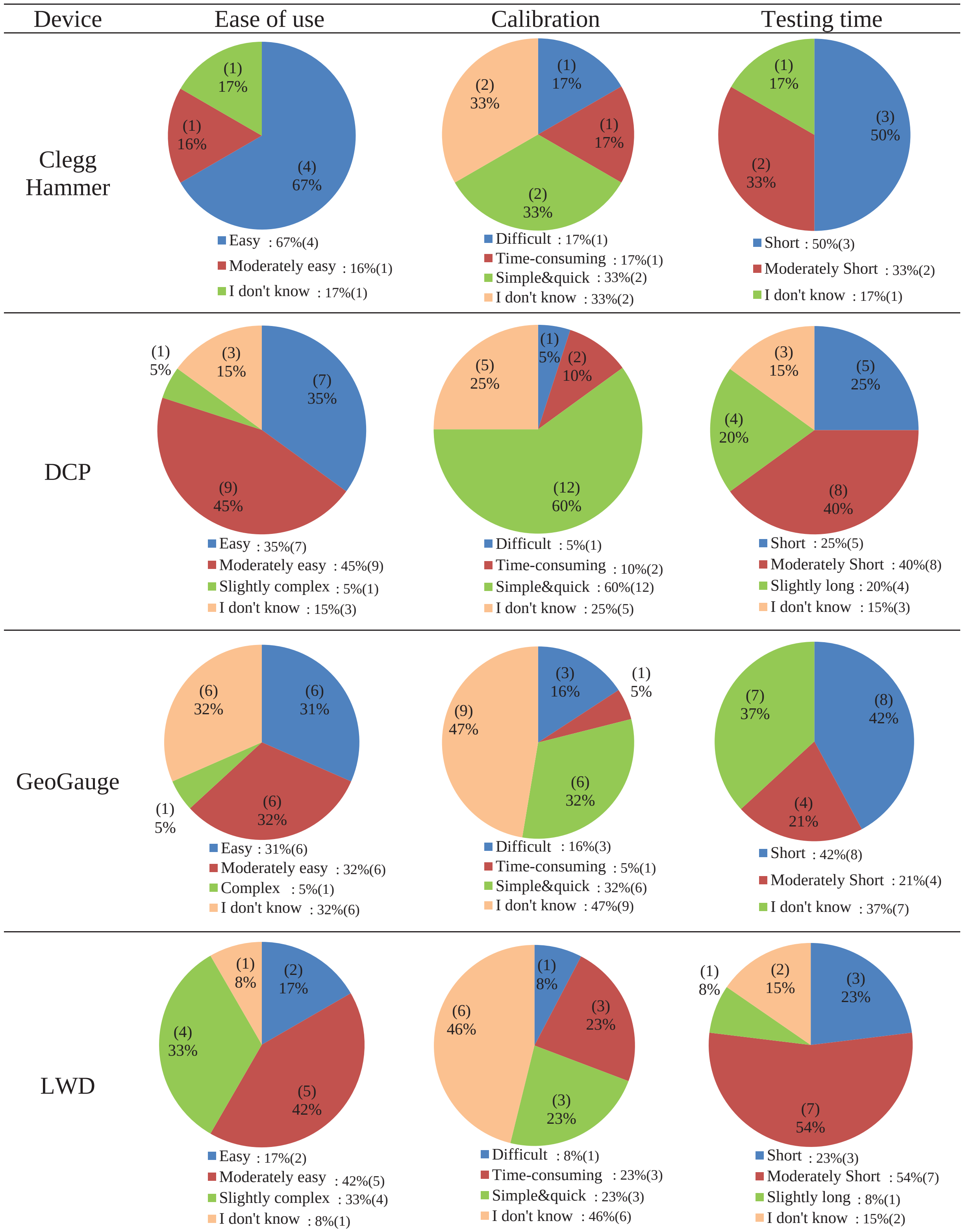


FIGURE 11 Experience of DOTs with in situ devices related to use, calibration, and testing.

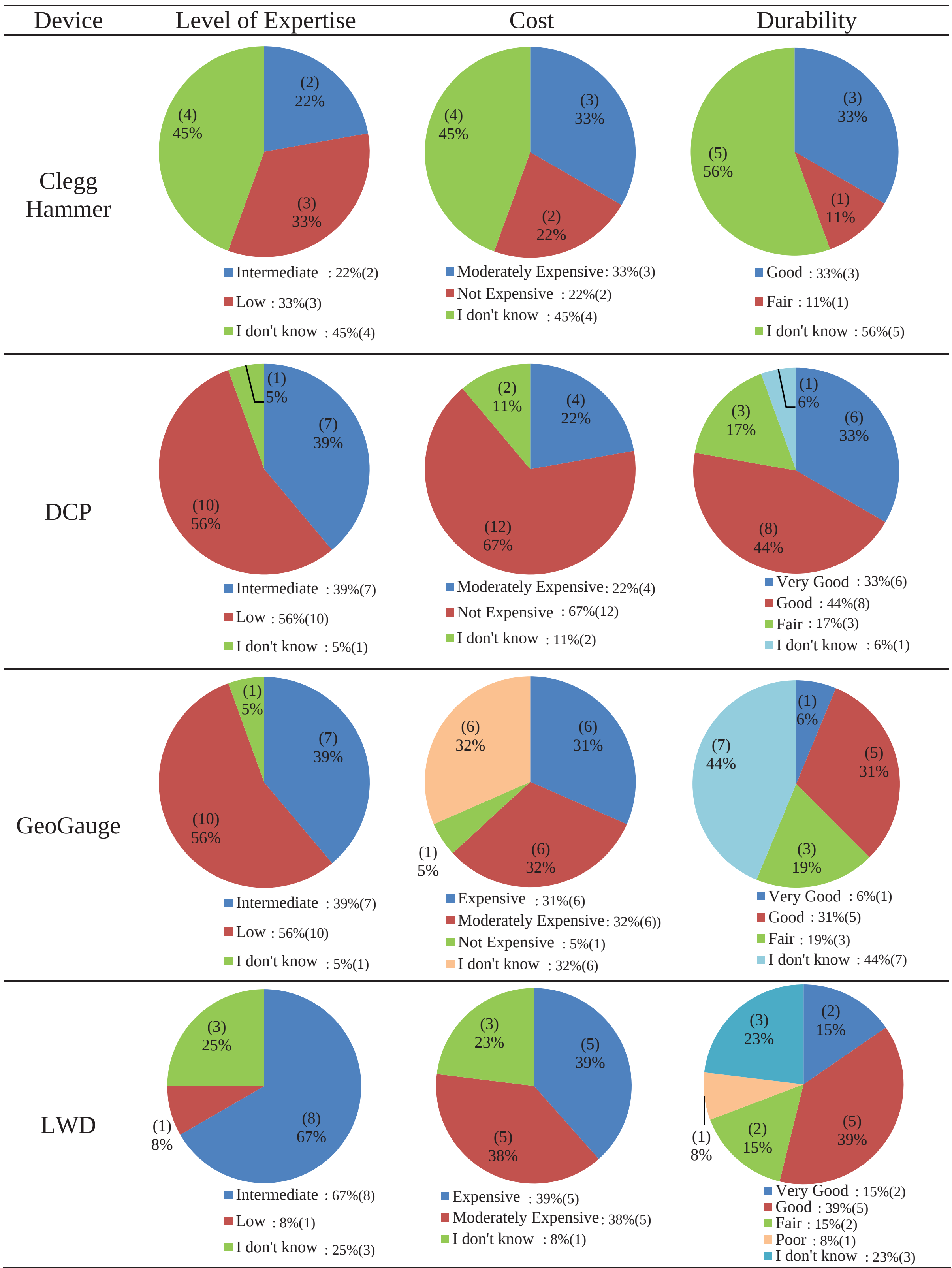


FIGURE 12 Experience of DOTs with in situ devices related to level of expertise, cost, and durability.

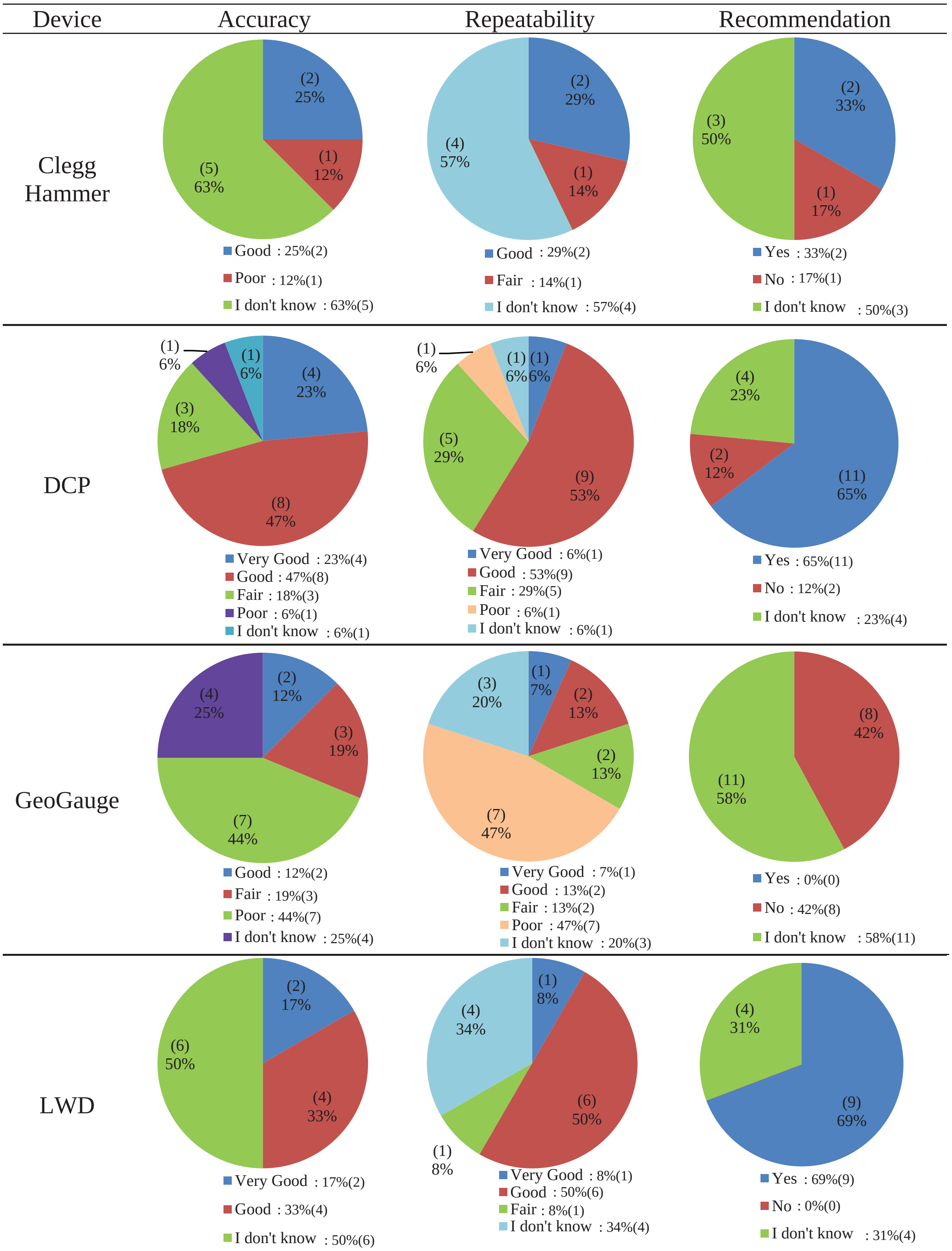


FIGURE 13 Experience of DOTs with in situ devices related to accuracy, repeatability, and recommendation.

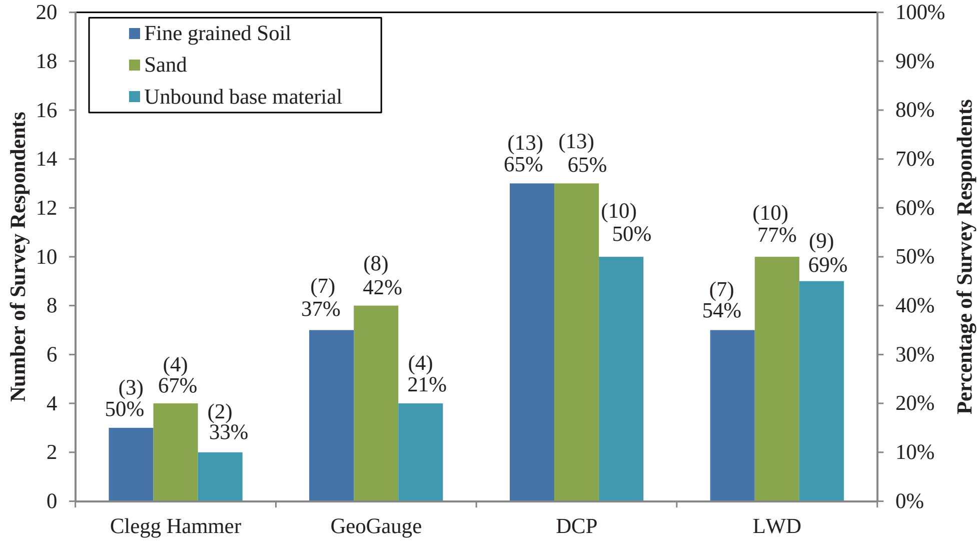


FIGURE 14 Compatibility of in situ devices with different types of unbound materials.

DOTs indicated that they frequently use those specifications. Indiana DOT staff reported that stiffness- and strength-based specifications are used in projects with more than 15 million cubic yards of fill, which represents more than 80% of construction projects in Indiana. At least three states have developed DCP and LWD target values for compaction control of various unbound materials.

With regard to survey results on the use of non-nuclear devices for in situ moisture content measurement, the respondents were most familiar with the speedy moisture tester (22) and the field microwave (14) among all other in situ devices. In addition, most respondents have not evaluated or used the other devices to measure moisture content of unbound materials. Figure 16 presents survey results related to the experience of state DOTs with in situ moisture content measurement. As shown in Figure 16, at least 13 states found the speedy moisture tester and field microwave to have good or very good repeatability and accuracy. Most respondents indicated that the speedy moisture tester and field microwave are easy or moderately easy to use. According to Figure 17, 13 states have recommended using those devices for measurement of in situ moisture content of unbound materials. However, at least four states did not recommend them.

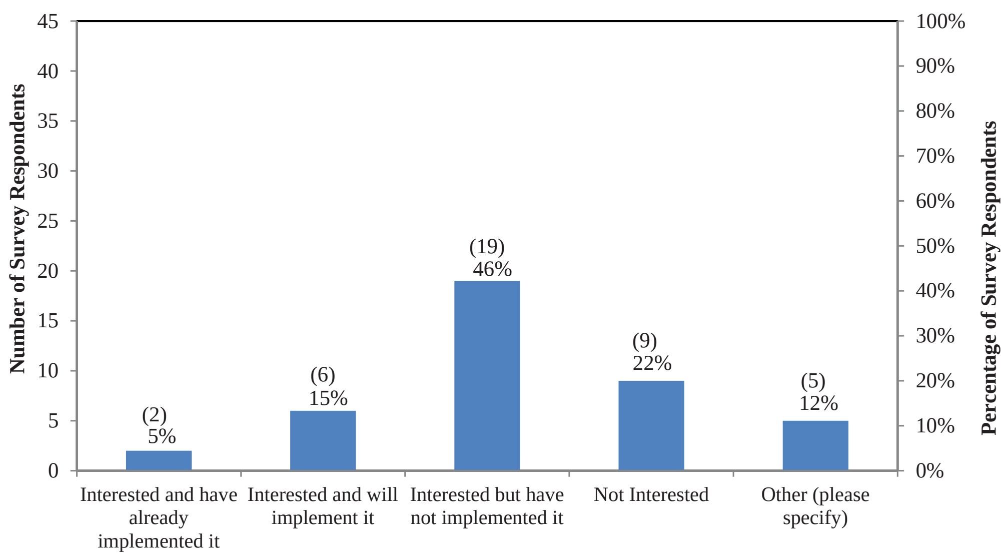


FIGURE 15 Interest in stiffness- and strength-based compaction control specifications.

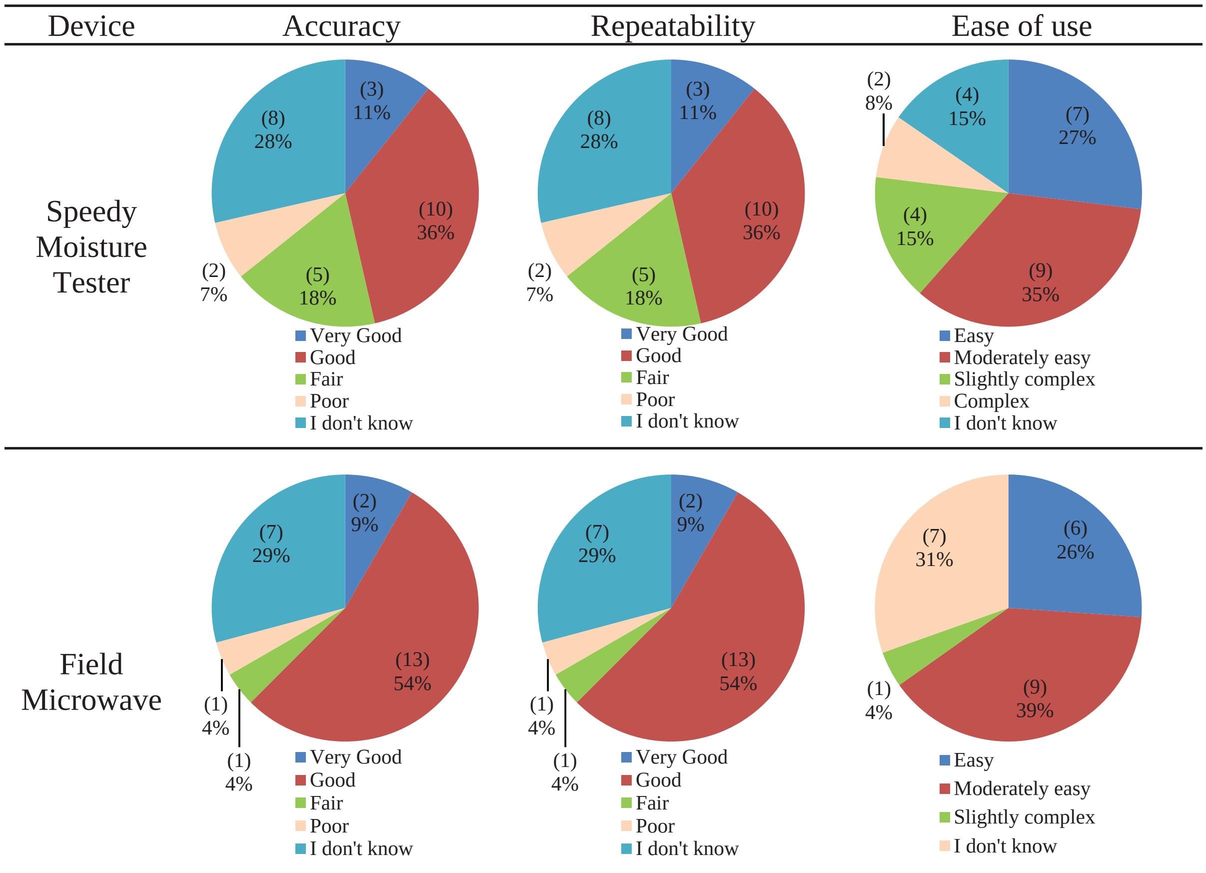


FIGURE 16 Experience of DOTs with in situ devices for moisture content measurement.

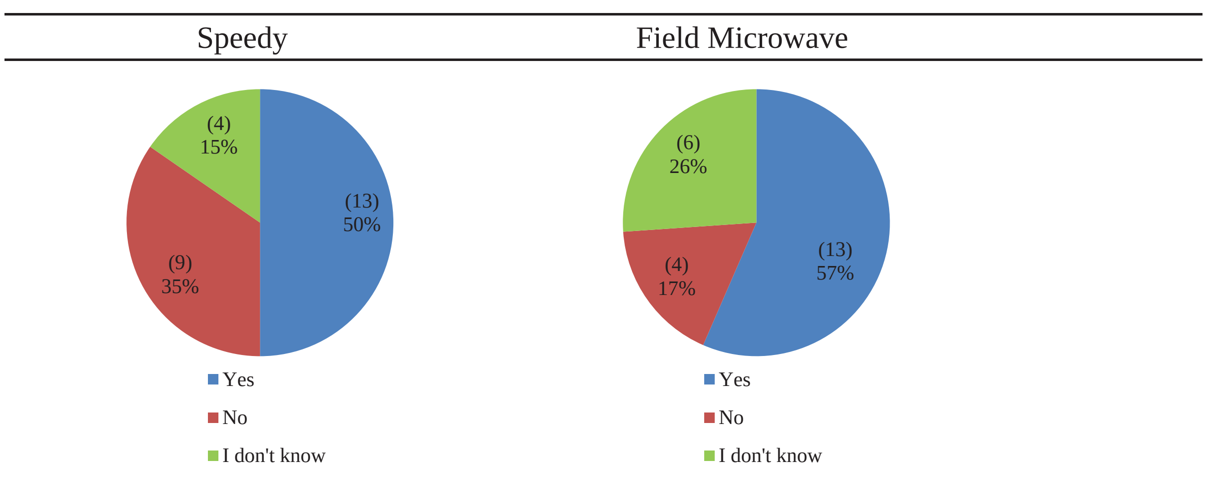


FIGURE 17 Recommendation of DOTs for using in situ devices for moisture content measurement.

CHAPTER THREE

NON-NUCLEAR METHODS FOR DENSITY MEASUREMENTS OF UNBOUND MATERIALS

INTRODUCTION

Until the 1920s compaction of unbound materials was performed largely on a trial-and-error basis. Stanton (1928) was the first to use soil compaction tests to determine optimum moisture content and maximum dry density. Proctor (1933, 1945, 1948) extended this work and studied the effect of soil compaction on shear strength and permeability. He also significantly contributed to the development of the standard laboratory compaction test, commonly known as the Proctor test. Despite the many advances in compaction technologies since then, the Proctor test remains an important component in quality control compaction procedures of unbound materials in the United States. This chapter provides a review of the commonly used density-based compaction control methods. It also describes the non-nuclear density devices that have been evaluated as alternatives to the nuclear density gauge (NDG). Furthermore, it discusses the principles of operation and main advantages and limitations of those devices, thus synthesizing what has been reported in past studies. In the following sections, photographs of non-nuclear density devices from certain manufacturers are provided for demonstration purposes only. This should not be construed as endorsements by this synthesis study of these devices.

CURRENT DENSITY-BASED COMPACTION CONTROL METHODS

The current compaction quality control of unbound materials involves determining the field dry density and moisture content of compacted lifts and comparing them to target density and moisture content values, which usually are determined using laboratory-specified tests performed on the same material used in the field. The ratio between the field density and the target laboratory density value is referred to as relative compaction. State DOTs typically require achieving a minimum relative compaction that varies between 90% and 100% for acceptance of compacted layers of unbound materials. It can be noted that the use of relative compaction requires that the material tested in the laboratory possess gradation and specific gravity similar to that in the field. In addition, the field and laboratory compactive effort imparted to the material must be similar. According to Marek and Jones (1974), if the material type or imparted energy in the field differs significantly from the reference material or compactive effort, the computed relative compaction will not be meaningful and valid. The following sections discuss the different methods that have been used to determine the target field density values for various unbound materials.

Impact Compaction Laboratory Methods

The impact compaction laboratory methods are the ones most commonly used to establish the compaction characteristics of unbound materials. These methods involve compacting a sample of the material to be used in the field in a standard mold using a drop hammer. The compaction energy can be varied by changing the number of hammer blows per layer as well as the weight and drop height of the hammer. Based on the compaction energy applied during the test, the procedure is either a “standard” or “modified” compaction procedure. The modified compaction test uses a compactive effort approximately 4.5 times greater than that of the “standard” test and was developed by the U.S. Army Corps of Engineers to better represent the compaction effort required for airfields to support heavy aircraft (Holtz et al. 2010). ASTM and AASHTO have standard specifications for both standard and modified compaction effort tests. The standard compaction procedure is documented as ASTM D698 or AASHTO T99, whereas the modified compaction procedure is specified as ASTM D1557 or AASHTO T180. Table 3 presents a comparison between the ASTM and AASHTO specified compaction methods. Several state DOTs have developed and used modified versions of the original ASTM and AASHTO specifications. Although these modified versions are somewhat different from the ASTM and AASHTO standards, the basic procedures and principles are the same (Tutumluer 2013). As shown in Table 3, the standard and modified compaction tests differ in terms of the hammer weight and drop height, as well as in the number of layers of unbound materials in the mold. Although the ASTM standards have three methods (Methods A–C) for different mold sizes and maximum particle size of the material to be compacted, the AASHTO standards have four methods (Methods A–D). The selection of the appropriate method in both cases depends on the gradation of the tested material.

The “standard” compaction method was originally developed based on a study done by Proctor (1933). In this study, Proctor (1933) performed “plasticity-needle” penetration resistance tests to determine the indicated saturated penetration resistance (ISPR) for several compacted earth-fill dam sections. Based on laboratory compaction tests, Proctor (1933)

TABLE 3
SUMMARY OF ASTM AND AASHTO STANDARDS FOR IMPACT COMPACTION METHODS

Test	Method	Standard Effort		Modified Effort	
		ASTM (D698)	AASHTO (T99)	ASTM (D1557)	AASHTO (T180)
Mold diameter	A	4 in. (102 mm)		4 in. (102 mm)	
	B	4 in. (102 mm)	6 in. (152 mm)	4 in. (102 mm)	6 in. (152 mm)
	C	6 in. (152 mm)	4 in. (102 mm)	6 in. (152 mm)	4 in. (102 mm)
	D	N/A	6 in. (152 mm)	N/A	6 in. (152 mm)
Mold volume	A	0.0333 ft^3 (944 cm^3)		0.0333 ft^3(944 cm^3)	
	B	0.0333 ft^3 (944 cm^3)	0.075 ft^3 (2,124 cm^3)	0.0333 ft^3 (944 cm^3)	0.075 ft^3 (2124 cm^3)
	C	0.075 ft^3 (2,124 cm^3)	0.0333 ft^3 (944 cm^3)	0.075 ft^3 (2124 cm^3)	0.0333 ft^3 (944 cm^3)
	D	N/A	0.075 ft^3 (2,124 cm^3)	N/A	0.075 ft^3 (2124 cm^3)
Rammer weight	A, B, C	5.5 lbf (24.4 N)		10 lbf (44.5 N)	
	D	N/A	5.5 lbf (24.4 N)	N/A	10 lbf (44.5 N)
Height of drop	A, B, C	12 in. (305 mm)	12 in. (305 mm)	18 in. (457 mm)	18 in. (457 mm)
	D	N/A	12 in. (305 mm)	N/A	18 in. (457 mm)
Layers	A, B, C	3		5	
	D	N/A	3	N/A	5
Blows per layer	A	25		25	
	B	25	56	25	56
	C	56	25	56	25
	D	N/A	56	N/A	56
Compactive effort	A, B, C	12,400 ft-lbf/ft^3 (600 kN-m/m^3)		56,000 ft-lbf/ft^3 (2700 kN-m/m^3)	
	D	N/A	12,400 ft-lbf/ft^3 (600 kN-m/m^3)	N/A	56,000 ft-lbf/ft^3 (2700 kN-m/m^3)
Material	A	Passing No. 4 (4.75-mm) sieve	Passing No. 4 (4.75-mm) sieve	Passing No. 4 (4.75-mm) sieve	Passing No. 4 (4.75-mm) sieve
	B	Passing 3/8 in. (9.5-mm) sieve	Passing No. 4 (4.75-mm) sieve	Passing 3/8 in. (9.5-mm) sieve	Passing No. 4 (4.75-mm) sieve
	C	Passing ¾-in. (19-mm) sieve	Passing ¾-in. (19-mm) sieve	Passing ¾-in. (19-mm) sieve	Passing ¾-in. (19-mm) sieve
	D	N/A	Passing ¾-in. (19-mm) sieve	N/A	Passing ¾-in. (19-mm) sieve
Use	A	≤25% by mass retained on No. 4 sieve	≤40% by mass retained on No. 4 sieve	≤25 by mass retained on No. 4 sieve	≤40% by mass retained on No. 4 sieve
	B	≤25 by weight retained on 9.5-mm sieve	≤40% by mass retained on No. 4 sieve	≤25% by mass retained on 9.5-mm sieve	≤40% by mass retained on No. 4 sieve
	C	≤30% by weight retained on 19-mm sieve	≤30% by weight retained on 9.5-mm sieve	≤30% by weight retained on 19-mm sieve	≤30% by weight retained on 9.5-mm sieve
	D	N/A	≤30% by weight retained on 19-mm sieve	N/A	≤30% by weight retained on 19-mm sieve

N/A = not applicable.

determined the compactive effort required to duplicate a field ISPR value of 300 psi. He suggested that soil samples be compacted in $\frac{1}{20}$-ft containers using "firm blows" of a 5.5-lb tamper. However, as a result of a printing error, the "firm blows" were interpreted as "free-falls" of the tamper. This has led some organizations to assume that instead of striking a minimum length blow of 12 in., the tamper should be dropped a distance of 12 in. in free fall. Proctor pointed out this mistake in several of his published papers (Proctor 1945, 1948). He stated that the objective of compaction in earth fills was to achieve a penetration resistance value of 300 psi; the 12-in. blow was required to ensure accurate determination of this value and was never intended as a "standard" or "optimum" (Proctor 1945). Despite Proctor's strong recommendation against this, the use of soil dry density, instead of a strength-based penetration test, as the standard of soil compaction has been adopted by most organizations.

In addition, the "standard" compaction method originally was intended for fine-grained soils. However, today it is

TABLE 4
TYPICAL RANGES OF MAXIMUM DRY UNIT WEIGHTS AND OPTIMUM MOISTURE CONTENTS USING STANDARD COMPACTION TESTS

AASHTO Classification	Soil Description	Anticipated Performance of Compacted Soil	Typical Ranges of γ_{dmax}		OMC (%)
			pcf	kN/m^3	
A-1-a, A-1-b	Well-graded gravel/sand mixtures	Good to excellent	115–142	18.1–22.3	7–15
A-2-4, A-2-5, A-2-6, A-2-7	Silty or clayey gravel and sand	Fair to excellent	110–135	17.3–21.2	9–18
A-3	Fine sand	Fair to good	100–115	15.7–18.1	9–15
A-4	Sandy silts and silts	Poor to good	95–130	14.9–20.4	10–20
A-5	Elastic silts and clays	Unsatisfactory	85–100	13.3–15.7	20–35
A-6	Silt-clay	Poor to good	95–120	14.9–18.8	10–30
A-7-5	Elastic silty clay	Unsatisfactory	85–100	13.3–15.7	20–35
A-7-6	Clay	Poor to fair	90–115	14.1–18.1	15–30

After Gregg and Woods (1960).

common practice to use impact compaction reference testing of other soil types for which it was not intended. Tutumluer (2013) notes that impact compaction reference testing may not adequately represent compaction characteristics in the field for certain aggregate that have low fines content. Both the ASTM and AASHTO standards can be conducted only on materials below a certain grain size, either 4.75 mm (0.19 in.) or 19.0 mm (¾ in.), depending on the method used. If the material to be tested includes particles larger than these sizes, corrections need to be applied to determine the unbound materials' maximum dry density. The method typically used by state DOTs to perform this correction is AASHTO T224, "Correction for Coarse Particles in the Soil Compaction Test." In this method, density is corrected by computing the weighted average of the density values of materials smaller and larger than the limiting particle size. Although density of the smaller material is determined using AASHTO T99 or T180, density of the larger material is based on knowledge of its bulk-specific gravity. This correction cannot be applied if the tested materials have more than 30% by mass of its particles larger than 19 mm (¾ in.).

Several studies were conducted to evaluate the effect of the soil types and properties on the maximum dry density and optimum moisture content determined by impact compaction methods. Gregg and Woods (1960) reported typical ranges of maximum dry density and optimum moisture content values of different types of soils, which are summarized in Table 4. Several researchers have found that for fine-grained soils, good correlations exist between the maximum dry density, the optimum moisture content, and the Atterberg limits (Woods 1938; Basheer 2001; Gurtug and Sridharan 2002; Omar et. al. 2003; Sridharan and Nagaraj 2005; Sivrikaya 2008; Sivrikaya et al. 2008; Kim et al. 2010). Based on tests results conducted on 102 soil samples in Indiana, Kim et al. (2010) proposed the equations shown in Table 5 to compute the compaction properties of fine-grained soils based on their plastic and liquid limits.

One-Point Proctor Test

The one-point Proctor method is an impact compaction test that was developed to determine the maximum density and optimum moisture content of unbound materials based on only a one-point measurement of density and moisture content. In this method, a sample of the unbound material used in the field is obtained and compacted in a Proctor mold using a standard (AASHTO T99) or modified (AASHTO T180) effort. The dry density and moisture content is measured and plotted on predetermined compaction curves, referred to as the family of compaction curves, with similar shape and geometry for various types of tested soils. An example of such curves is provided in Figure 18. If the measured dry density and moisture content fall on one of the existing family of curves, the maximum dry density and optimum moisture

TABLE 5
RELATIONSHIP BETWEEN γ_{dmax}, PLASTIC LIMIT AND LIQUID LIMIT

Parameters Considered in Developing the Relationship	Relationship	R^2
PL (%), γ_{dmax} (pcf)	$PL = -48.024\ln(\gamma_{dmax}) + 245.82$	0.66
LL (%), γ_{dmax} (pcf)	$LL = -85.018\ln(\gamma_{dmax}) + 434.75$	0.54

Source: Kim et al. (2010).

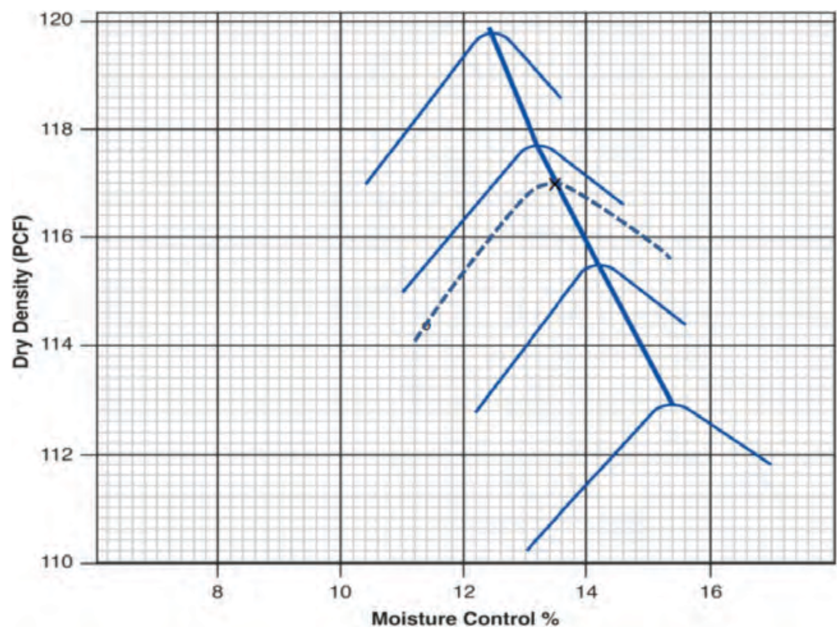


FIGURE 18 An example of the one-point Proctor method (Idaho DOT).

content for that curve will represent those of the tested material. However, if the point (representing density and moisture content) falls in between two curves, a new curve that is parallel to and similar in shape to the nearest existing curve is drawn through it. The maximum dry density and optimum moisture content are then read from the new fitted curve. The one-point Proctor test is typically performed in accordance with the AASHTO T272-04 standard method. This test has been used by several state DOTs, including those of Indiana, Idaho, Iowa, Ohio, South Dakota, and Washington. Typically, each DOT develops its own family of compaction curves based on tests conducted on unbound materials encountered during construction.

To evaluate the effectiveness of the one-point Proctor test, Wermers (1963) performed 861 compaction tests and compared the results from the one-point Proctor test with the standard Proctor test. He reported that the optimum moisture content found from the one-point Proctor test was on average 0.19% higher than that obtained by the standard Proctor test. Wermers also reported that 92% of the maximum dry density values obtained from the one-point Proctor test were within 4.0 pcf of those obtained from the standard Proctor test.

Static Compaction Laboratory Method

Porter (1930) introduced the static compaction method, in which he used a static pressure of about 13,800 kPa (2,000 psi) to compact granular soil samples of 152 mm (6 in.) in diameter. However, since that time, this method has not been widely used because the application of static pressure was not found to be effective in compacting granular materials (Rodriguez et al. 1988). Thus, currently there is no standard procedure for performing static compaction tests. However, the procedure that was followed in previous studies involved compacting about 2,500 g of moist unbound materials in a Proctor compaction mold [typically a 102-mm (4-in.) mold was used] in one lift by applying static stresses using a hydraulic compression device (Bell 1977; Zhang et al. 2005; White et al. 2007a). A load cell and two linear variable differential transformers (LVDTs) are typically utilized to measure the applied stresses and deformation, respectively. The obtained load versus deformation curve is used to determine the applied static compaction energy using Eq. 1.

$$\text{Energy}_{\text{static}}\left(\text{kN-m/m}^3\right) = \frac{\begin{array}{c}\text{area of load versus}\\ \text{deformation curve }(\text{kN-m})\end{array}}{\text{Volume of mold }(\text{m}^3)} \qquad (1)$$

Aguirre (1964) compared the maximum dry density values obtained using the static and impact compaction methods for 17 different soils. The results indicated that the moisture-density curves for both static and impact compaction tests were similar for coarse sands and gravels. However, the maximum dry density determined using impact compaction for the plastic clay soils evaluated in that study were lower than those obtained in static compaction.

Bell (1977) reported that at given moisture content, the static compaction required less compactive energy than both impact and kneading compaction methods to achieve a target density value. Zhang et al. (2005) and White et al. (2007a) reported that the static compaction energy required to obtain a given dry density value decreased with increasing moisture content. Bell (1977), Zhang et al. (2000), and White et al. (2007a) found that the moisture-density results were similar to impact compaction test results in their studies.

Vibratory Compaction Laboratory Method

In this method, the maximum dry density of unbound materials is determined by applying vibratory compaction forces. This can be done by using a vibratory hammer or a vertically vibrating table. For tests conducted using a vertically vibrating table, there are three main variables that control the compaction energy imparted into the tested material, namely the vibration amplitude and frequency and the weight of the surcharge used. The compaction energy can be adjusted by changing any of those variables based on Eq. 2. Currently, there is an ASTM standard (ASTM D4253) for performing vibratory compaction tests, but no such specification is provided by the AASHTO.

$$\text{Energy}_{vib} = \frac{W \times f \times A \times t}{V} \tag{2}$$

where

Energy_{vib} = vibratory compaction energy (kN-m/m^3),
W = weight of surcharge (kN),
f = frequency of vibration (Hz),
A = amplitude (m),
t = time (s), and
V = volume of mold (m^3).

Lambe (1951) reported that granular free-draining soils do not respond to variations in compacting moisture content and impact energy as cohesive soils because of negligible lubrication; therefore, vibratory compaction should be used. Many researchers reported that vibratory compaction methods produce consistently higher maximum densities for granular materials than does the impact compaction method and also better replicates field densities (Burmister 1948; Felt 1958; Pettibone and Hardin 1964). Although the vibratory compaction method was developed originally for granular soils, some studies indicated that it can be effective in cohesive soils if compacted at low frequencies (Converse 1956; Lewis 1961). According to a recent survey conducted by Tutumluer (2013), only two state DOTs (Kansas and Alabama) reported the use of vibratory compaction methods for unbound aggregate materials.

Gyratory Compaction Laboratory Method

The U.S. Army Corps of Engineers introduced the gyratory compaction test procedure for soils based on extensive testing on silty sand material (Coyle and West 1956; McRae 1965). The test procedure consisted of placing a thoroughly mixed, loose, moist sample in a cylindrical mold and then applying a controlled normal force to both the top and bottom of the sample at a constant gyration rate. The applied normal force was supplemented with a kneading action or gyratory motion at an angle (gyration angle) to compact the material. Currently, there are no standard values available on gyratory compaction parameters, such as pressure, angle, number of gyrations, or gyration rate. Different values of gyratory compaction parameters were used in previous studies. A summary of these values is presented in Table 6.

Smith (2000) reported a good correlation between laboratory and field densities for well-graded crushed stone. For fine sand, Ping et al. (2003) found that the optimum moisture and maximum densities achieved in the field were closer to gyratory compaction results than impact and vibratory compaction. In addition, they concluded that the gyratory compaction method could be used to better simulate compaction conditions in the field. Similar conclusions were reached by Kim and Labuz (2006) and White et al. (2007a) for different types of granular and cohesive soils.

Main Limitations of Laboratory Compaction Methods

The main limitation of using laboratory compaction methods to select the target field density is that the volume of material used in those tests is very small compared with the total volume compacted in the field (Kim et al. 2010). If the compacted materials are highly variable, these methods will yield ambiguous results unless field corrections are made frequently. Another issue arises from the presence of a significant amount of gravel and cobbles in earth fill (Holtz et al. 2010). Although the use of laboratory impact reference testing has been extended beyond fine-grained soils, the AASHTO T99 and AASHTO T180 and their corresponding ASTM standards specify limits on the allowable amount of oversized particles in the tested material. For example, both test procedures are limited to unbound materials that have 30% or less by mass of particles with sizes greater than 19 mm (¾ in.). Because of the problems associated with laboratory compaction methods, control strips (or test fill/strip) have been used by some DOTs to determine field target density value.

Control Strip or "Test Strip" Method

A control or test strip is a section that is compacted before and during fill-placement operations to determine the maximum target density value and the roller type, pattern, and number of

TABLE 6
GYRATORY COMPACTION PARAMETERS USED IN PAST STUDIES

Study	Vertical Stress (kPa)	Gyration Angle	No. of Gyrations	Soil Type
Smith (2000)	1,380	1.0	30–40	Well-graded crushed stone
Ping et al. (2003)	2,000	1.25	90	Fine sand
Kim and Labuz (2006)	6,000	1.25	50	Recycled granular material
White et al. (2007a)	6,000	1.25	50	Granular and cohesive soil

TABLE 7
SUMMARY OF STATE DOTs' CONTROL STRIP SPECIFICATIONS FOR UNBOUND MATERIALS

Agency	Length (minimum)	Width (minimum)	Target Value
Alaska	300 ft	12 ft	95% of the maximum control strip density measured using NDG
District of Columbia	100 ft	At least one lane wide	Not specified
Maryland	100 ft	At least one lane wide	Not specified
Kentucky	500 ft	Full lane width	Five tests must be at least 98% of the target density with no individual measurement less than 95% of the target density
Mississippi	500 ft	12 ft	Same rolling pattern and number of passes used in test strip (only for aggregate drainage layer)
North Carolina	300 ft	Full lane width	Not specified
New Hampshire	100 ft	Full lane width	95% of maximum control strip density measured using NDG
Virginia	300 ft	Full lane width	Average 98%; individual minimum 95% of control strip density
New Hampshire	100 ft	Full lane width	Not specified
Alabama	500 ft	Not specified	100%
Indiana	100 ft	Lane width	Not specified
Minnesota	300 ft	32 ft	90% of IC target value
Michigan	600 ft	Not specified	95% of the maximum unit weight; used for open-graded drainage course base only
New Jersey	400 square yards		$Q = \frac{avg\ lot\ density - 0.95\ of\ ref.max.density}{Range\ of\ lot\ density} \geq 0.36$
South Dakota	500 ft	Not specified	95% of target density
West Virginia	100 ft	Full width	Not specified

passes needed to achieve this density of a particular material. The strip is compacted at a moisture content close to the optimum using the compaction equipment to be utilized by the contractor. Field density and moisture measurements are obtained at three or more randomly selected locations after each pass until no significant increase in density is observed. The average final density of the material from the control strip is defined as the maximum target density for that particular material. Usually agencies specify that lifts must be compacted to a certain percentage of this maximum density. Test sections also are typically constructed every 1,000 to 3,000 m^3 (1,500 to 4,000 yd^3) or where the compacted material changes significantly. It can be noted that the compaction of a control strip should be correlated to previously established compaction results for successful implementation of the control strip method (Tutumluer 2013). Based on the review of state DOT construction specifications and manuals, it was found that several states have specifications for using control strips in their compaction control procedures for unbound materials. Table 7 presents a summary of state DOT control strip specifications for unbound materials.

Solid Volume Density Method

In this method, the target field density is selected as a percentage of solid volume density. The solid volume density represents the density of soil solids in a voidless matrix, which can be obtained by multiplying the specific gravity of the aggregate with the unit weight of water. The constructed layer densities in this method are then expressed as a percentage of the solid volume density, referred to as relative solid density (RSD) or solid relative density (SRD). In this method, the relationship between the achieved densities in the field and solid volume density of the compacted material should be known. Kleyn (2012) reported the application of the solid volume density method in the construction control of G1 base in South Africa. He found that 88% of SRD was equivalent to about 106% of the maximum dry density obtained using the modified compaction method. In addition, he indicated that there are a number of conditions that need to be satisfied before using this method. Kleyn (2012) stated that the aggregates have to be very resistant to general construction impacts and free from contamination or deleterious materials. Only fresh, unweathered, and sound rock could be used in construction of the base layer in this procedure. Currently, the solid volume density method has not yet been used in the United States or Canada.

MEASURING IN-PLACE DENSITY OF CONSTRUCTED UNBOUND MATERIALS

Most state DOTs determine the in situ dry density and moisture content of compacted unbound materials using the nuclear density gauge (NDG) (shown in Figure 19). The device was introduced in the early 1970s and gained popularity after an industrywide calibration standard was developed for it (Kim et al. 2010). The NDG works by emitting gamma radiation into the material to be tested and detecting the reflected rays to determine its wet density. Denser materials contain more electrons with which the photons of the gamma radiation interact; therefore, they reflect a lower number of photons.

The number of detected photons is used to calculate the density of the tested material based on calibrated relationships. In addition to the density measurement, the NDG is capable of measuring the moisture content of soil. The high-speed neutrons emitted from the nuclear gauge source get retarded by the hydrogen atoms present in the moist material. The number of slow-speed neutrons detected by the gauge is used to determine the amount of hydrogen atoms present in the material, which is then used to compute its moisture content (ASTM D6938). Nuclear gauges can be operated in two modes: direct transmission mode and backscatter mode. Winter and Clarke (2002) reported that direct transmission mode yielded a more accurate density measurement than did backscatter mode.

The main advantage of the nuclear gauge test over other conventional tests is that it is relatively fast to perform. In addition, it is accurate and repeatable when properly calibrated. The NDG can be used to measure the density of asphalt concrete as well as layers of unbound materials. It also has the advantage of being able to vary depth of measurements using the direct transmission procedure. The NDG is also one of the few density tests that has the capability to provide both moisture content and density measurements. The main drawback with the gauge is that it uses radioactive materials that necessitate strict compliance with regulatory requirements for handling, storage, maintenance, transport, and monitoring. Furthermore, NDG measurements may be affected by the chemical composition of the soil tested (ASTM D6938). Specifically, moisture content measurements are affected when hydrogen atoms exist in the chemical composition of the soil and other recycled pavement materials commonly used today. Because of these issues, highway agencies, universities, and equipment manufacturers have developed several new methods and devices to replace the NDG. Some studies were performed to evaluate these devices. The following sections provide a review of the different non-nuclear density methods that have been used or evaluated by state DOTs during the past decade. In addition, a summary of the main findings of previous studies on these devices is presented.

Moisture Density Indicator

The moisture density indicator (MDI) uses time domain reflectometry (TDR) to measure the dry unit weight and moisture content of soils. It consists of four metal spike probes that are encased in a probe head, which is connected by a coaxial cable to a TDR pulse generator. The generator is attached to a personal digital assistant (PDA) with proprietary software. The MDI works by sending an electromagnetic wave pulse through the four probes that are driven into the soil in the formation shown in Figure 20 a to imitate a coaxial cable configuration. The center spike acts as the central conductor in the coaxial cable, the three outer probes serve as the shield conductor, and the in situ soil acts as the insulator. Typically the spikes have diameters of 0.75 in. and variable lengths of 4, 6, and 8 in. (Rathje et al. 2006; Jackson 2007). The signal reflected back through probes is recorded and analyzed by the PDA using proprietary software to determine electrical properties of the tested soil. This is used to determine the dry density and moisture content of the tested soil. This test is performed according to ASTM D6780. The MDI costs about $6,000.

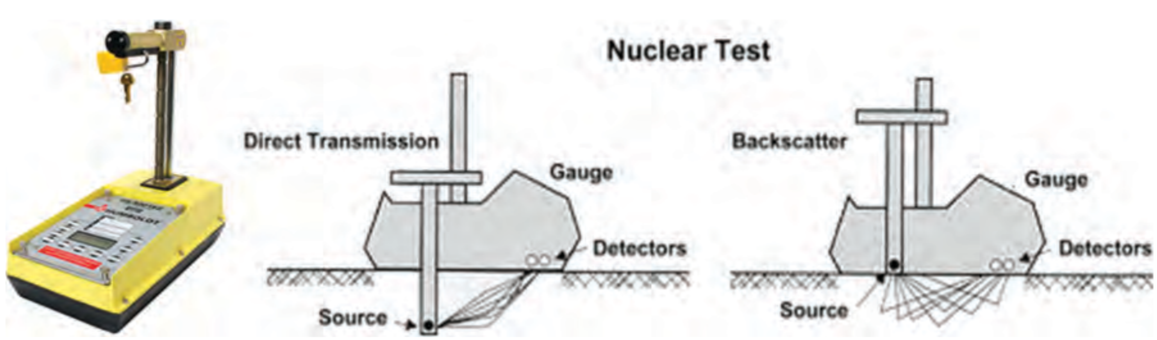
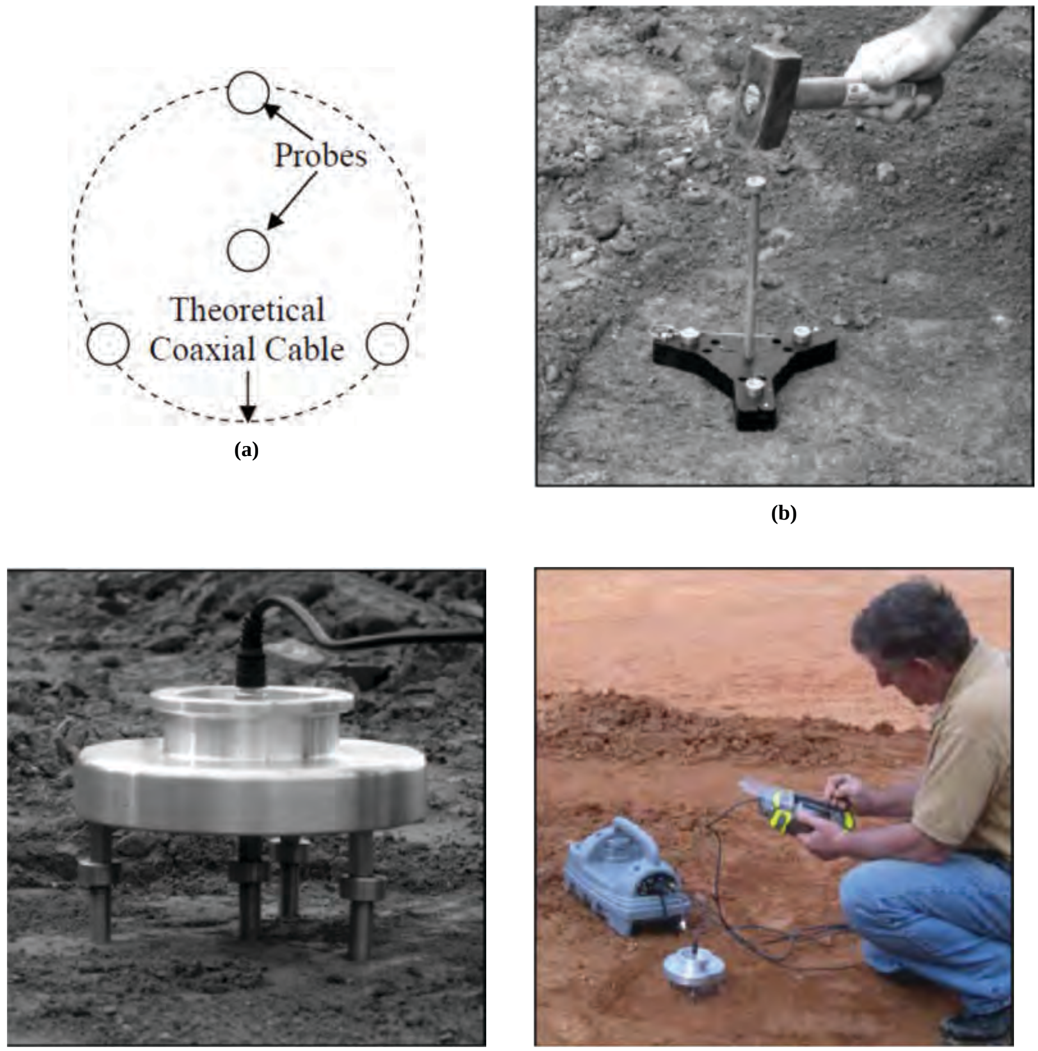


FIGURE 19 Nuclear density gauge (Troxler 2000).



(a)

(b)

(c)

(d)

FIGURE 20 MDI field testing: (*a*) probe configuration, (*b*) probes being driven into the ground, (*c*) coaxial head placed on top of the soil probes, and (*d*) field measurement being taken (Durham 2005).

Two main electrical properties of tested soils are measured using the MDI: the dielectric constant, K_a, and bulk electrical conductivity, EC_b. K_a is calculated using Eq. 3 based on the first and second reflections in the recorded TDR waveform, which represent when the electromagnetic wave enters the soil and when it is reflected from the end of the MDI soil spikes. In addition, the bulk electrical conductivity (EC_b) is found by measuring the source voltage (V_s) and final voltage (V_f) of the TDR waveform using Eq. 3 (Rathje et al. 2006).

$$K_a = \left(L_a/L_p\right)^2 \tag{3}$$

where

L_a = scaled horizontal distance between the first and second reflection points in the TDR waveform, and
L_p = the length of the soil probes.

$$EC_b = (1/C) * \left((V_s/V_f) - 1\right) \tag{4}$$

$$C = \left(2 * \pi * L_p * R_s\right) / \left(\ln\left(d_o/d_i\right)\right) \tag{5}$$

where

V_s = voltage source,
V_f = long-term voltage level,
C = constant related to probe configuration,
R_s = internal resistance of pulse generator,
d_o = outer conductor diameter, and
d_i = inner conductor diameter.

Equations 6 and 7 relate the K_a and EC_b directly to the dry unit weight and moisture content of the soil (Durham 2005; Rathje et al. 2006).

$$(K_a)^{1/2}\left(\rho_w/\rho_d\right) = a + b * w \tag{6}$$

$$(EC_b)^{1/2}\left(\rho_w/\rho_d\right) = c + d * w \tag{7}$$

where

a, b = soil-specific calibration constants for K_a,
c, d = soil-specific calibration constants for EC_b,

ρ_w = unit weight of water,
ρ_d = dry unit weight of soil, and
w = moisture content.

The MDI has two operation modes: one step and two step. In one step mode, the bulk electrical conductivity (EC_b) and dielectric constant (K_a) values are simultaneously measured for a given soil used to determine the dry density and moisture content. In two step mode, the MDI is used to measure the K_a of the soil in place and a soil sample excavated from the field and compacted in a mold. The density of the in situ soil is determined using the density of the soil in the mold and the dielectric constants measured in the field and mold.

Calibration of the MDI requires determining the constants in Eqs. 6 and 7 for the specified soil. This is achieved by measuring the K_a and EC_b for several samples compacted in a Proctor mold at a range of known dry densities and moisture contents. The obtained data are used to develop plots of $(K_a)^{1/2}(\rho_w/\rho_d)$ and $(EC_b)^{1/2}(\rho_w/\rho_d)$ versus moisture content, and a line is fit to the plotted data to determine calibration constants of tested soil.

Repeatability and Accuracy

In general, all studies indicated that the MDI is a repeatable device. The reported coefficient of variation of MDI measurements was in general less than 15% (Rathje et al. 2006). Previous studies showed that MDI moisture content measurements were accurate and very close to those obtained using the oven dry method. The MDI and NDG differed in their dry density measurements (Jackson 2007; Ooi et al. 2010). Ooi et al. (2010) questioned the MDI ability to yield reliable results because of a flaw in its equation formulation; they also suggested that its formulation should be reevaluated.

Main Advantages and Limitations

The MDI has some advantages over the NDG. First, the MDI does not require special licensing to operate (Rathje et al. 2006; Brown 2007). In addition, the MDI operator dependency does not contribute to significant variability in its measurements. The main disadvantage of the MDI is that it is time consuming (15 to 20 min per test). In addition, operating the MDI is more cumbersome than other methods, as many loose parts are required for operation. In addition, it may require excavation of soil from the construction site. Some researchers also indicated difficulty in driving and removing the spikes into the aggregate base or stiff subgrade soils. The MDI also presents limitations on the types of soil it can test. For example, it cannot be used for highly plastic clays because of issues related to electrical conductivity in those clays (Yu and Drnevich 2004). Furthermore, the MDI can test soils with 30% or less, by mass, of its particles smaller than 4.75 mm (0.19 in.) and has a maximum particle size of 19 mm (¾ in.). MDI cannot be used to test frozen soils.

Berney et al. (2013) reported that the installation of the four probes in the seating mold caused it to become wedged into the soil surface. This made the removal of the mold difficult without disturbing the probes, resulting in a loss of contact with the soil along the entire probe length, which led to a very low moisture content reading. This effect was exacerbated in soils with coarser grain sizes. Finally, some studies suggested that the operation of the MDI required an operator with at least moderate knowledge of the device's overall capabilities.

Synthesis of Past Research Studies

New Jersey Jackson (2007) conducted a study for the New Jersey DOT to evaluate the effectiveness of the MDI as a tool for compaction control of dense-graded aggregate base layers. Field testing was conducted on five sites that consisted of dense-graded, aggregate base layers as well as on some New Jersey DOT-designated porous fill materials. The results of this study indicated that both the nuclear gauge and the MDI recorded similar moisture contents. However, differences of as much as 12.53% were observed in the dry density measurements. In general, the dry density values measured by the MDI were less than those obtained using the nuclear gauge. The MDI appeared to be less sensitive to the changes in compaction density measured at different locations at a given site as compared with the NDG. Jackson (2007) indicated that it was difficult to drive and remove the spikes into the aggregate base. A much larger hammer was needed, and it took more than 15 min per test to drive in the spikes. He suggested that spikes of a least 1 in. in diameter be used to enhance penetration of the compacted aggregate base. Jackson (2007) also reported that the MDI data acquisition software froze several times during field and lab testing. Finally, from a job-site efficiency standpoint, researchers found that the transportation of the MDI device was time consuming and cumbersome.

Vermont Brown (2007) reported the results of a study conducted to evaluate the performance of two non-nuclear density devices (EDG and MDI) and compare them with that of the NDG. As shown in Figure 21, the dry density values measured by the MDI had a very good correlation with those of the NDG. However, the moisture content measured by those devices showed high variability, as shown in Figure 22. Brown (2007) indicated that the MDI was time consuming to set up and was not easily transported around the construction site. Its many loose parts required multiple trips to move the device from one spot to another. In addition, Brown (2007) found that MDI setup did not work well in coarse materials because the MDI uses four spikes that are driven through a template in a very concentrated area (of usually about 8 in.). In addition, the spikes were prone to bending when used on aggregate materials or densely compacted subgrade material.

Florida Sallam et al. (2004) presented the results of a study in which the TDR was used to measure moisture contents of A-3 (fine sands) and A-2-4 (sands and gravels with elastic silt fines) at different sites. Figure 23 shows the moisture content

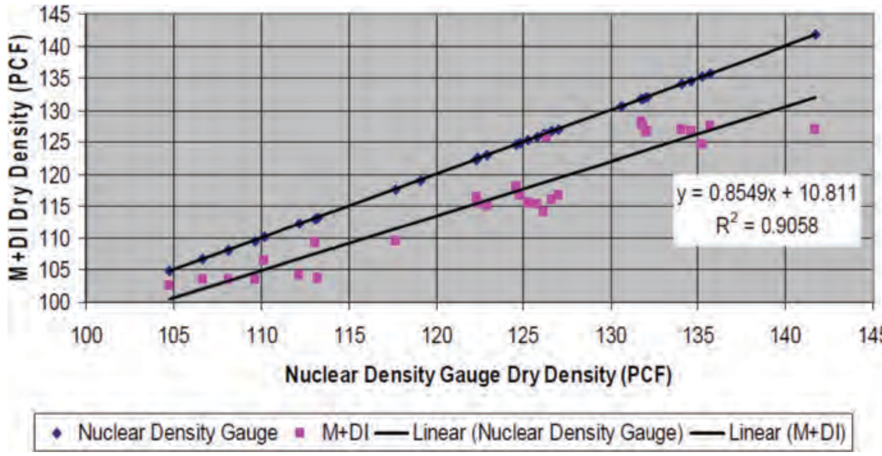


FIGURE 21 MDI dry density compared with NDG dry density (Brown 2007).

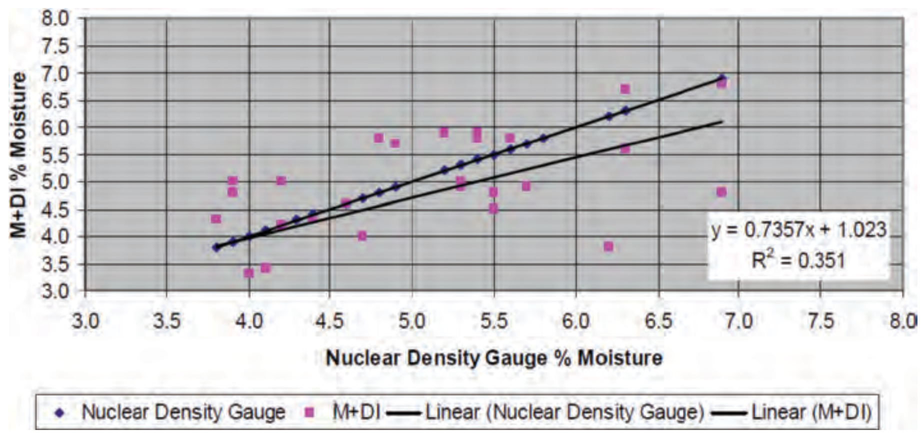


FIGURE 22 MDI moisture content compared with NDG moisture content (Brown 2007).

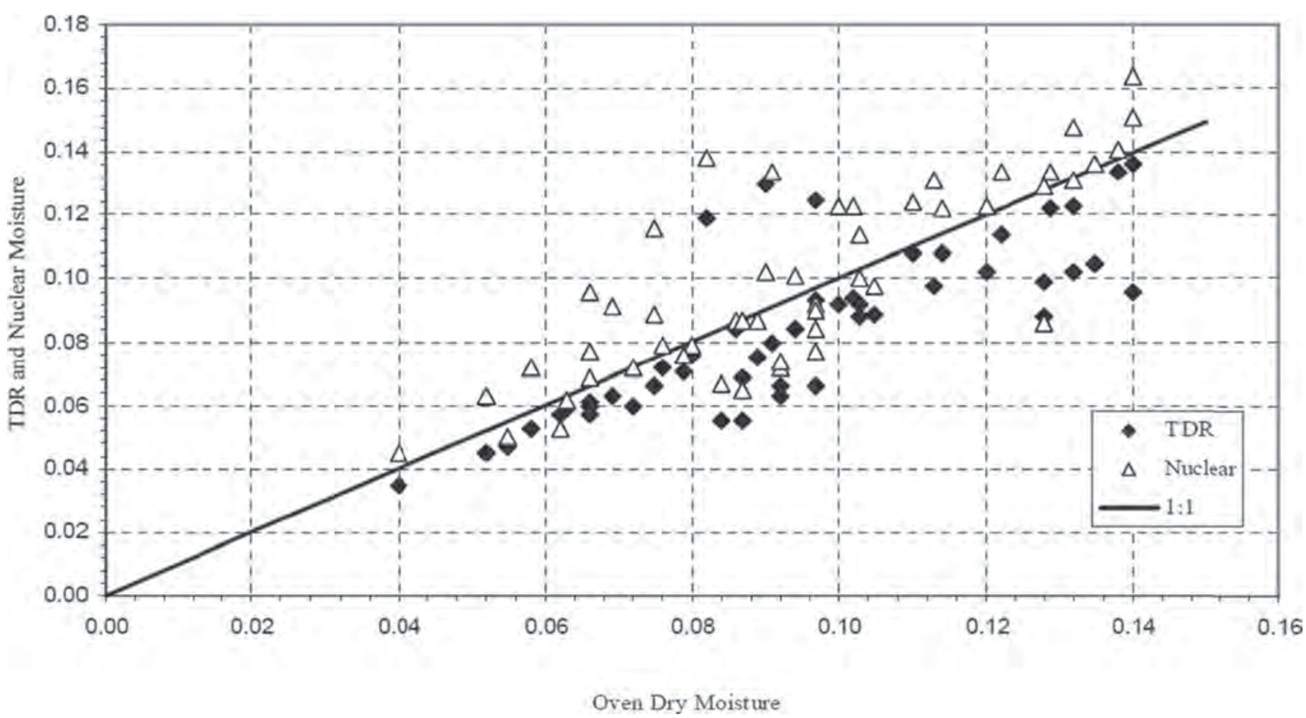


FIGURE 23 MDI moisture content compared with NDG moisture content (Sallam et al. 2004).

data that were obtained in that study. The TDR recorded less scattered and closer moisture content values to the laboratory oven method compared with the nuclear gauge. Thus, it yielded more repeatable and accurate measurements. In general, the TDR consistently underestimated the moisture content, whereas the nuclear density method overestimated it.

Runkles et al. (2006) conducted a study that evaluated the accuracy and repeatability of TDR's one-step method (MDI one step mode) in measuring the dry density and moisture content of sandy soil typically encountered in construction in Florida. The results indicated that the TDR method showed lower variability in the moisture content measurements than did the NDG and could be more accurate if the calibration constants were properly selected. In addition, as shown in Figure 24, the TDR method had a better correlation with the oven dry moisture content measurements than did the nuclear gauge. However, Runkles et al. (2006) reported that the TDR recorded more variable and lower density measurements than did the nuclear gauge.

Hawaii Ooi et al. (2010) reported the results of a study that compared the moisture content and dry density of recycled concrete aggregate (RCA) and reclaimed asphalt pavement (RAP) obtained using nuclear gauge and TDR methods with actual values by compacting these materials in 6-in. lifts and 3-ft diameter bins. The results indicated that the TDR moisture content measurements for RAP and the dry densities for RAP and RCA were reasonably accurate. However, the TDR underestimated the moisture content of the RCA material. The authors also reported that the TDR provided more accurate dry density measurements than did the NDG when the RCA material was tested 10 days after its compaction.

Texas Rathje et al. (2006) conducted field and laboratory testing programs to evaluate the MDI. The field component of the study included determining the density of clayey soils used in Texas for highway embankments or as road subgrade. In addition, the laboratory portion of the study included using the MDI and other devices to test laboratory-compacted samples of poorly graded sand. The results of this study indicated that for clayey soils, the MDI dry density measurements did not agree favorably with those obtained using the nuclear gauge or the rubber balloon. Rathje et al. (2006) also indicated that the MDI dry density values were higher than those of the NDG for high plasticity clays and lower for low plasticity clays. Furthermore, the MDI moisture contents were different than the values measured by the traditional oven drying method and the nuclear gauge. For the laboratory test samples of sand, the MDI showed good agreement with the microwave oven measurements of moisture content. However, it consistently reported the same dry density value for all samples, which did not agree with the rubber balloon measurements.

Electrical Density Gauge

The electrical density gauge (EDG) (Figure 25) uses high radio frequency waves to measure the density and moisture content of soils. It consists of four tapered 6-in. long spike probes, a hammer, a soil sensor and cables, template, temperature probe, battery charger, and hard case. The device works by transmitting high radio frequency waves through the four probes that are driven into the soil in a square formation. Four measurements are obtained at each test location after the probes are inserted. The EDG analyzes the transmitted radio frequency to determine the electrical dielectric properties of the tested

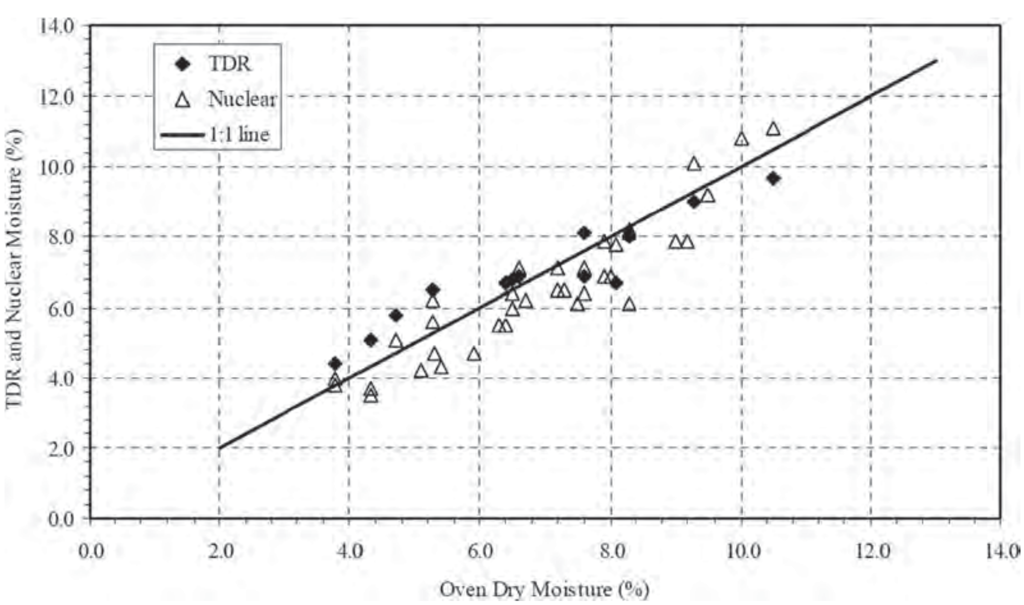


FIGURE 24 MDI moisture content compared with NDG moisture content (Runkles et al. 2006).

FIGURE 25 Electrical density gauge (Berney et al. 2013).

soil, which include resistance (R_s), capacitance (C_s), the quotient (C_s/R_s), and real impedance (Z_s). Those properties are converted to dry density and moisture content measurements by using a soil-specific calibration model, which is developed by taking EDG readings of the soil samples compacted in a laboratory mold at different moisture contents and density combinations to determine dielectric constants for each combination.

The EDG is conducted in accordance with an ASTM D7698 standard. However, a detailed description regarding the theoretical basis for the EDG is not currently available. The price for the EDG device is approximately $9,300 (plus $2,250 for the calibration verifier) (Humboldt Mfg. Co. 2012). The EDG has GPS capabilities as far as 3 meters away from the location of the probes (Humboldt Mfg. Co. 2012).

Repeatability and Accuracy

There is no consensus in the literature on the repeatability and accuracy of the EDG. As part of NCHRP 10-65, Von Quintus et al. (2008) reported that the EDG was a highly repeatable device and had a coefficient of variation (COV) of dry density and moisture content of less than 1% and 5%, respectively. Berney et al. (2013) reached similar findings when testing various types of soils. However, Brown (2007) indicated that there was high variability in EDG moisture content measurements.

In terms of accuracy, some studies showed that the EDG recorded dry density measurements that were similar to those obtained using traditional methods such as NDG (Brown 2007; Von Quintus et al. 2008; Berney et al. 2013). However, this was found to depend mainly on EDG calibration for the considered material (Jackson 2007; Von Quintus et al. 2008; Berney et al. 2013). In addition, Berney et al. (2013) indicated that the relative error in EDG moisture content measurements was approximately 4% when using the lab oven dry method measurements as a reference. On the contrary, Rathje et al. (2006) reported a less favorable agreement between EDG dry unit weight and water content measurements with traditional method measurements when testing poorly graded sand. Cho et al. (2011) found that average error in the EDG dry density and moisture measurements was much higher than that of the NDG.

Main Advantages and Limitations

The main advantage of the EDG over the NDG is that it is safer and does not require special licensing to operate (Rathje et al. 2006; Brown 2007). However, the EDG calibration process was found to be complex and time consuming (Rathje et al. 2006; Berney et al. 2013). Previous studies indicated that the EDG was cumbersome and time consuming to operate in the field because the switching of connectors to and from different probes takes some time. Furthermore, similar to the MDI, the device had many loose parts (template, darts, hammer, and electrodes), making it difficult to transport from one location to another at the site. The probes also could be difficult to drive into stiff soils. Rathje et al. (2006) also suggested that the EDG could not be used for high-plasticity clays and said it relies heavily on other density tests for its calibration.

Synthesis of Past Research Studies

Vermont The results of a study conducted by the Vermont DOT indicated that the EDG's dry density measurement had a strong correlation with that of the NDG (shown in Figure 26) (Brown 2007). However, the EDG moisture content measurement, presented in Figure 27, had a weak correlation with that of the NDG. The authors attributed this to the high variability of moisture contents within various soil types and soil depths.

Texas As part of a study conducted by the Texas DOT, Rathje et al. (2006) evaluated the ability of the EDG to accurately measure the density and moisture content of lab-compacted samples of poorly graded sand. The EDG constantly measured moisture content values of 5% for all samples, whereas the values recorded by the microwave oven ranged between 2.9% and 3.4%. In addition, although the EDG dry density measurements were about 90 pcf for all soil samples, the rubber balloon measurements were 100 to 115 pcf. Rathje et al. (2006) concluded that there was no agreement between EDG density and moisture content measurements and those obtained using traditional methods.

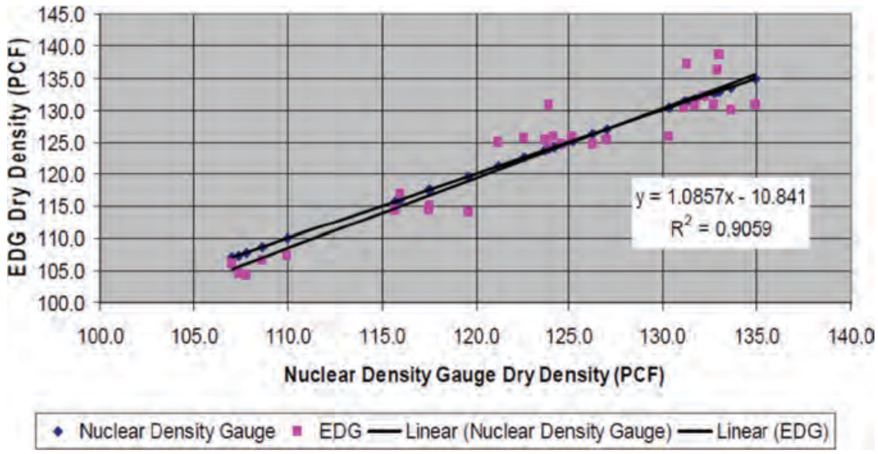


FIGURE 26 EDG dry density compared with NDG dry density (Brown 2007).

Nebraska Cho et al. (2011) reported the results of a study that evaluated the effectiveness of the EDG in measuring in situ moisture content and density. The study included conducting EDG and NDG tests on several soil types at several construction sites in Nebraska. The EDG and NDG measurements were also compared with the standard field dry weight unit measurement determined by taking a sample representative of each measurement area either with a Shelby tube or other method for lab testing. Figure 28 shows the relationships between the EDG and the NDG dry density measurements and those obtained using the drive-cylinder method. In addition, Figure 29 presents the comparisons of the moisture content measurements of the EDG and the NDG to those obtained using the standard oven dry test method. The NDG results had a better correlation than did the EDG results to the moisture content and dry density measurements that were obtained using the standard methods (i.e., oven dry test and drive-cylinder methods). In addition, the average difference between the EDG dry density and moisture content measurements and those obtained using the standard measurement methods were found to be 9.86 pcf and 1.66%, respectively; for the NDG dry density and moisture content measurements, the average difference was 1.71 pcf and 0.22%, respectively. Cho et al. (2011) attributed those results to the nuclear gauge data being corrected using the density and moisture correction factors required by the Nebraska DOT standard test method for the NDG. In addition, they noted that the EDG had similar results to the NDG before the correction factors were applied. Based on the results of life-cycle cost analyses, Cho et al. (2011) indicated that despite the high initial cost of the EDG, it presented an economic advantage over the nuclear gauge when maintenance and operating costs were included.

NCHRP Project 10-65 Von Quintus et al. (2008) reported the results of NCHRP 10-65, in which the repeatability and accuracy of the EDG were evaluated for selected unbound materials. Figure 30 compares the obtained EDG dry density values to those measured with the traditional NDG. There are two different groups of data: one for fine-grained soils and the other for crushed aggregate base materials. In general, as the NDG dry density increased among different materials, the EDG density also increased. However, no apparent correlation exists between EDG and NDG dry density measurements.

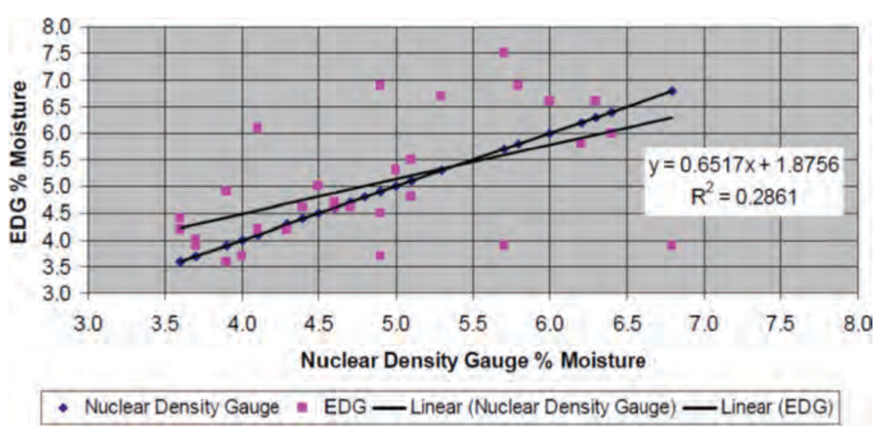


FIGURE 27 EDG moisture content compared with NDG moisture content (Brown 2007).

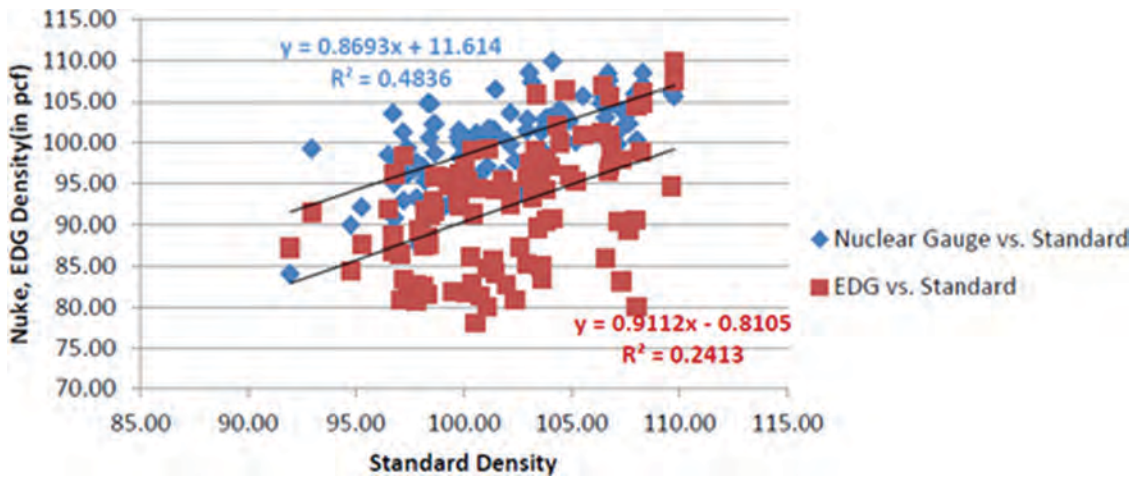


FIGURE 28 Dry densities measured with EDG and NDG (Cho et al. 2011).

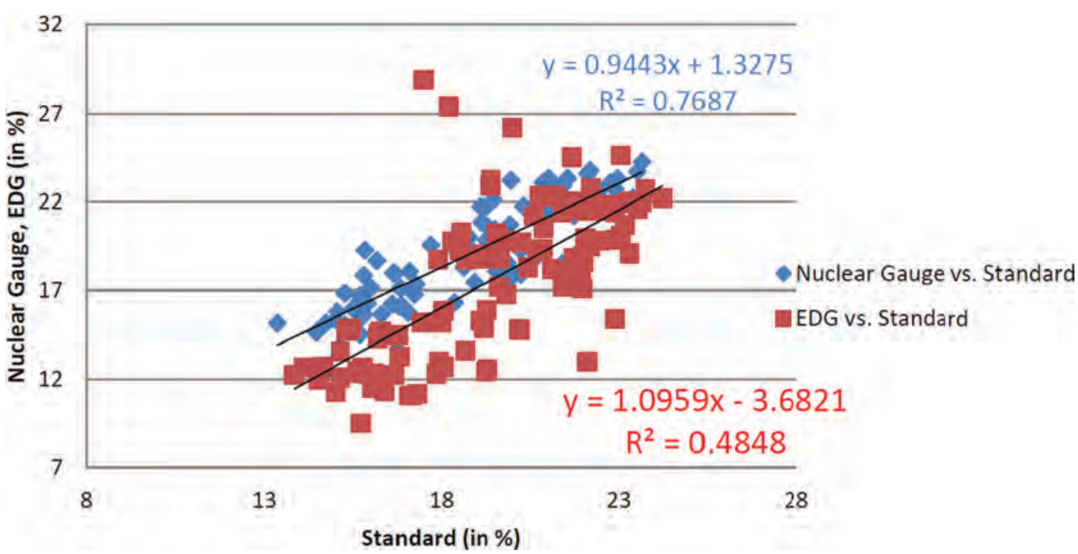


FIGURE 29 Moisture content measured with EDG and NDG (Cho et al. 2011).

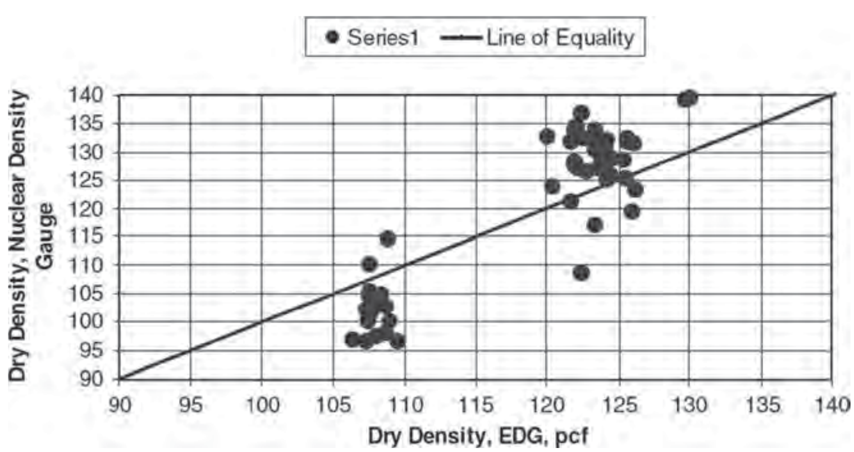


FIGURE 30 Dry densities measured with EDG and NDG (Von Quintus et al. 2008).

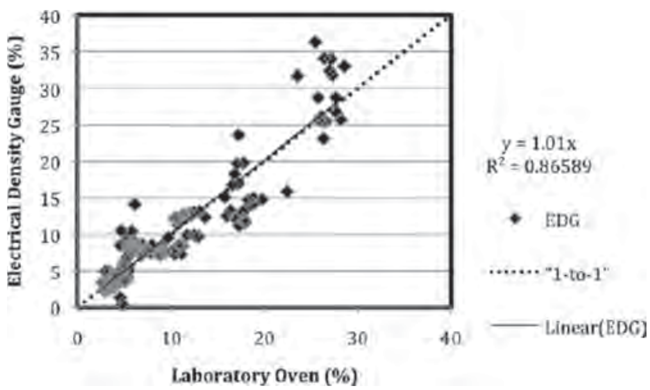


FIGURE 31 Moisture content of EDG and laboratory oven dry method (Berney et al. 2011).

U.S. Army Corps of Engineers Berney et al. (2013) reported a study conducted at the U.S. Army Corps of Engineers Research and Development Center in Vicksburg, Mississippi, to evaluate the effectiveness of various devices, including the EDG, in controlling the compaction of soils used for horizontal construction. The results indicated that the EDG was the second most effective electrical device for measuring the dry density of the various types of soils, having the most accurate precision but only average accuracy when compared with the NDG. Berney et al. (2013) found that the EDG performed better in fine-grained soil than in granular material.

In another study, Berney et al. (2011) examined the repeatability and accuracy of EDG moisture content measurement. Figure 31 presents the data collected in this study. The EDG moisture content had a strong correlation with that of the laboratory oven method. In addition, both methods yielded very close values, as indicated by the slope of the line in Figure 31. This suggests that the EDG had good accuracy and was as repeatable as the laboratory oven method.

Soil Density Gauge

The soil density gauge (SDG) is a self-contained unit that uses electromagnetic impedance spectroscopy (EIS) to measure the density and moisture content of various unbound materials. As shown in Figure 32, the SDG measurement is done through a noncontacting sensor that consists of a central ring and an outer ring. The central ring generates and transmits a radio-frequency–range electromagnetic field into the soil. The response to that field is received by the outer ring and is used to measure the dielectric properties of the tested soil matrix. The SDG performs a calculation on the measured dielectric properties to determine the density and moisture content of the tested soil (TransTech Systems, Inc. 2008). The price of the latest SDG model (SDG 200) starts at $10,000. This model has an advanced GPS system that enables location and independent time logging.

The SDG should be calibrated by compacting a sample of the soil of interest in a Proctor mold and testing it using SDG. The density and moisture content of the compacted soil sample are used as initial condition values for calibration (Gamache et al. 2009). The SDG test consists of taking five individual measurements in a counterclockwise "cloverleaf" pattern at the test site. The first location measured represents the place where density and moisture content is desired to be obtained, and the outline circles of the other four locations must be 1 to 2 in. away from the initial center circle measurement. The surface where the SDG is placed for testing must be flat and free of small stones or debris for the consistency of results (TransTech Systems, Inc. 2008).

Accuracy and Repeatability

In general, previous studies indicated that SDG density and moisture content were repeatable and very close to the NDG values once the linear offset correction was made to the

FIGURE 32 Soil density gauge, SDG200 (Pluta et al. 2008).

device's results (Pluta et al. 2008; Berney et al. 2011). However, Pluta et al. (2008) reported that soil gradation affected the frequency response of the SDG, which could reduce the repeatability of this device because of inconsistencies. Berney et al. (2013) also found an overall coefficient of variation of the corrected SDG to be 0.278 with regard to moisture content.

Advantages and Limitations

The SDG can provide accurate and repeatable moisture content and density measurements when the proper corrections are applied and the gradation of the tested unbound materials is consistent with its calibration. It was suggested that operators of the SDG have extensive knowledge of this device in order to apply the needed corrections (Berney et al. 2011).

Synthesis of Past Research Studies

Pluta et al. (2008) conducted SDG and NDG tests on various types of fine- and coarse-grained soil. Figure 33 presents the obtained wet density measurements in that study. The soil type had a significant effect on the SDG density measurements. To address this issue, the authors applied a linear adjustment based on the specific surface area of the material being tested to correct the SDG soil model's calculation of wet density and moisture. Figure 34 shows the wet density values that were corrected by applying the surface area adjustment. In five of the six types of tested soils, the corrected SDG wet density measurements came much closer to those of the NDG. In addition, the average difference in wet density between the NDG and the SDG was reduced by 119%. Pluta et al. (2008) concluded that the accuracy and precision of the SDG density measurements could be improved by accounting for the specific surface area of the material being tested.

U.S. Army Corps of Engineers

Berney et al. (2011) evaluated the performance of the SDG device in measuring moisture content of various types of soils used by the U.S. Army Corps of Engineers for horizontal construction. As shown in Figure 35, raw SDG moisture content measurements were highly scattered and had a poor relationship with those obtained using the laboratory oven method. Berney et al. (2011) used Eqs. 8 and 9 to correct the SDG measurement by applying a linear offset, which represents the difference between a recorded SDG moisture content measurement and that obtained using the oven method for the soil of interest. Figure 35 shows that the corrected SDG value had much better correlation with the laboratory oven method moisture content measurements. In addition, the corrected SDG values were also closer to those of the lab oven method than the NDG measurements. Based on that, Berney et al. (2011) recommended that the SDG measurements be corrected using Eqs. 8 and 9.

Berney et al. (2013) conducted a study that assessed the performance of the SDG and other non-nuclear density devices. The results indicated that the SDG was the most effective device overall, possessing an optimal combination of accuracy and precision compared with the NDG. Berney et al. (2013) also found that the SDG had better performance in granular soils compared with fine-grained soil. This was attributed to the SDG manufacturer developing its platform using more granular soil types. Therefore, the researchers recommended that the SDG be tuned to capture the density variance in wetter, fine-grained soils.

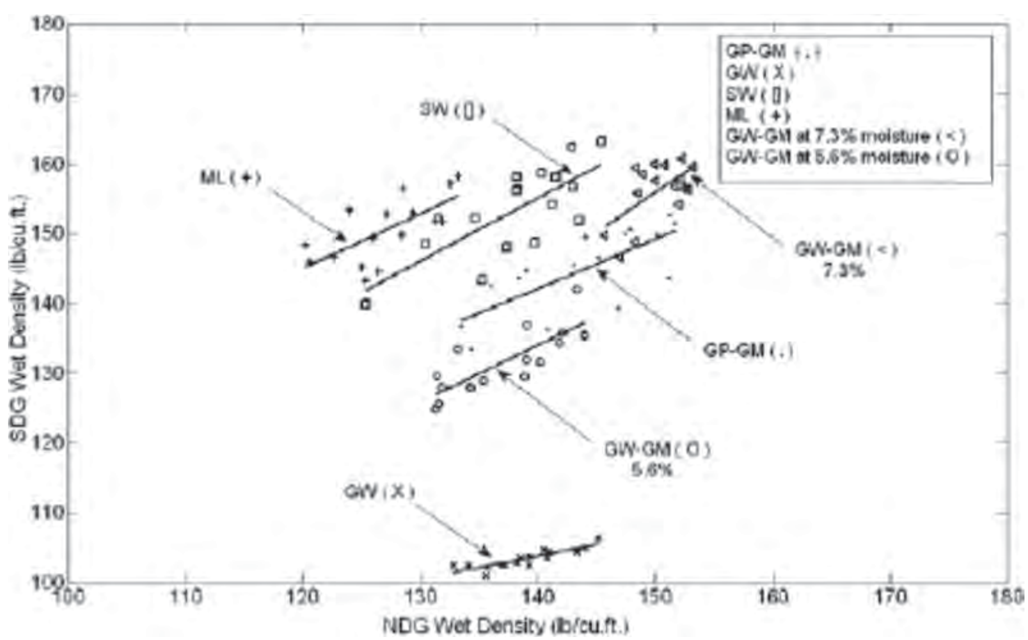


FIGURE 33 SDG wet density compared with NDG wet density (Pluta et al. 2008).

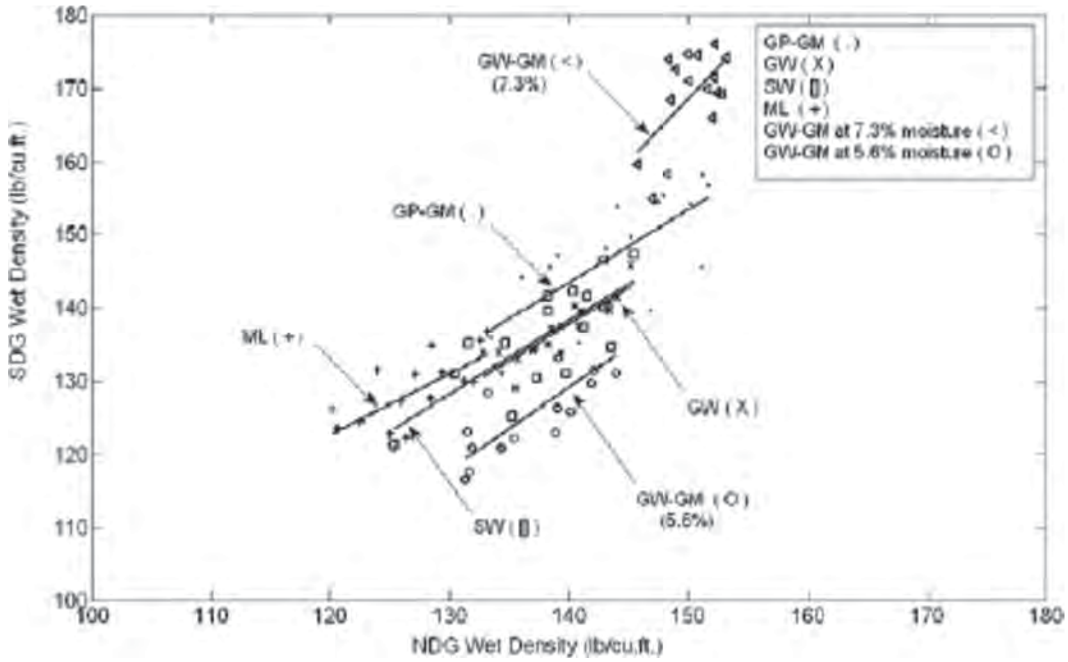


FIGURE 34 Corrected SDG wet density compared with NDG wet density (Pluta et al. 2008).

$$L.O. = STD_{\text{oven}} - MC_{SDG\#1} \quad (8)$$

$$SDG_{\text{Corr}} = MC_{SDG} + L.O. \quad (9)$$

where

$L.O.$ = linear offset,
STD_{oven} = laboratory oven moisture content (standard),
$MC_{SDG\#1}$ = first SDG moisture content for a specific soil,
SDG_{Corr} = corrected moisture content for that soil, and
MC_{SDG} = SDG device reading for moisture content.

MOISTURE MEASUREMENT

Whether measuring density, modulus, or shear strength, moisture content remains a critical parameter in compaction quality control procedures of unbound materials. Therefore, it is essential to obtain rapid, reliable, and accurate moisture content measurements of compacted unbound materials in the field. There are several non-nuclear methods that have been used for measuring the in situ moisture content of unbound materials during construction. The following sections discuss the main methods that have been

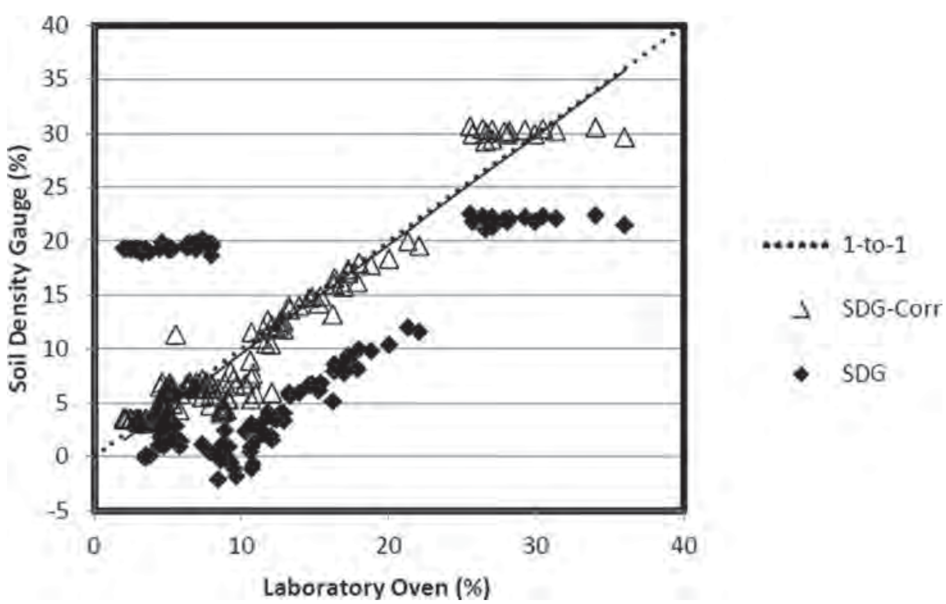


FIGURE 35 Corrected SDG and initial SDG moisture content compared with moisture content obtained using the laboratory oven method (Berney et al. 2011).

evaluated by state DOTs other than those discussed in the preceding sections.

Speedy Moisture Tester

The speedy moisture tester is a commonly used system that measures the moisture content of unbound materials during construction of embankment and pavement layers. It consists of a rugged plastic case containing a low-pressure vessel fitted with a pressure gauge, an electronic scale, steel balls, reagent, and brushes. Figure 36 shows the main components of the speedy moisture tester, which costs about $2,000. The operational principle of this device is based on measuring the amount of gas produced by a reactant material and the free moisture in the soil to determine the soil's moisture content. The test is performed according to the ASTM D4944-04 standard or AASHTO T217, and it involves placing about 20 g of soil along with an equal amount of the reactant material (calcium carbide) in the pressure vessel. After the steel balls are added, the vessel is sealed and inverted to bring the reagent and the soil into contact. The vessel is then shaken for 10 s, followed by a rest period of 20 s. This process is repeated for 1 to 3 min depending on the soil type. Shaking is performed to ensure that all of the moisture reacts with the reagent. Steel balls are used to break up any lumps in the soil sample. Once the reaction is complete, the sample weight and the pressure increase inside the vessel are recorded. Because the pressure in the vessel is proportional to the amount of moisture in the sample, the moisture content can be read directly from the calibrated pressure gauge.

The speedy moisture tester takes less than 5 min to obtain results. The test is easy to perform and requires minimal operator training. However, there are some limitations of this test. First, it cannot be used for highly plastic clayey soil because this type of soil may not mix thoroughly with the reagent; thus, some moisture remains in the soil. In addition, some soils may contain chemical compounds that may react with the reagent, yielding erroneous results. Because of the small size of materials that can be tested in the speedy moisture tester, this test may not provide accurate moisture content measurement of coarse-grained granular soil (Berney et al. 2011). Inaccurate measurements may also result from the use of old calcium carbide reagent, incomplete breakdown of soil lumps, improper sealing of the vessel, insufficient time allotted for the chemical reaction, and the presence of volatile material in the tested material (Petersen et al. 2007). Calcium carbide is considered a hazardous material that requires special handling and environmental considerations. Sallam et al. (2004) indicate that one source of error for this method is the operator's ability to perform the test correctly.

FIGURE 36 Speedy moisture tester (Berney et al. 2011).

Synthesis of Previous Studies

Oman (2004) presented the results of a study in which the moisture content was obtained using the speedy moisture tester and oven dry methods for various types of granular soils in Minnesota. The results indicated that the methods provided comparable moisture content measurements. In addition, as shown in Figure 37, a strong correlation existed between the two methods.

Alleman et al. (1996) found that the speedy moisture tester overestimated the moisture content by 1.25%. However, it was considered reliable once calibration was performed. George (2001) found that the results from the speedy moisture tester were comparable to results from the NDG. Runkles et al. (2006) reported the results of side-by-side tests performed to measure the moisture content of common construction soils in Florida using the ASTM TDR one-step method, nuclear gauge, oven dry, and speedy moisture methods. The results indicated that the speedy moisture method was slightly more variable than the ASTM TDR and nuclear methods. In addition, it had poor correlation with the oven dry measurements. The ASTM TDR and nuclear method measurements had a much stronger correlation with the oven dry measurements.

Berney et al. (2011) reported that the results of a study to evaluate the effectiveness of various devices to measure the moisture content of soil for horizontal construction. The accuracy and repeatability of the considered devices were compared with the standard laboratory oven soil moisture determination. The results of this study indicated that this device had the most effective repeatability; however, it had the worst accuracy among other moisture devices that were used. As shown in Figure 38, Berney et al. (2011) found that the speedy moisture device overestimated the moisture content for all soil types for different reasons. For coarse-grained soils, small sample size (only 20 g) was found to be the reason for the overestimation of moisture content. The sample size tends to contain only fine-grained material, which retains the most available moisture. In addition, for fine-grained soils with high moisture content, the speedy moisture tester required a multiplier to be applied to the charted conversion values. This multiplication increased the overall error in the moisture content results.

Moisture Analyzer

The moisture analyzer is a small drying device with a scale and an overhead ceramic heating element (Figure 39). To use

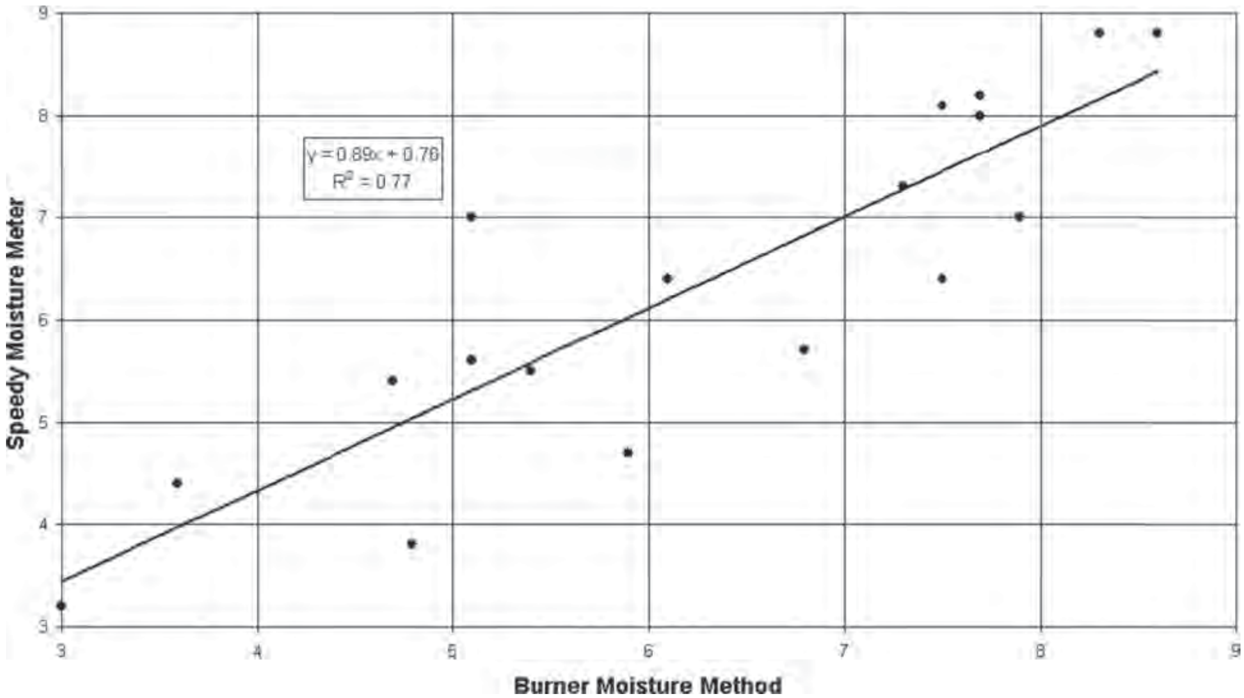


FIGURE 37 Moisture content measured by speedy moisture tester and oven dry method (Oman 2004).

this device, a representative soil sample is placed on a small, disposable aluminum foil dish, and the dish is transferred inside the heating unit. After taking the initial mass, heating of the soil samples is continued until the mass reaches a constant value. The difference between initial and final mass is taken as the mass of water. The gravimetric moisture content is then calculated from mass of water and mass of dry soil sample. The moisture analyzer costs about $1,840 (Sebesta et al. 2013).

Synthesis of Previous Studies

Limited studies were conducted to evaluate this device. Berney et al. (2011) compared the moisture contents obtained using the moisture analyzer with those obtained using the laboratory oven dry method. This comparison is shown in Figure 40. The researchers found that the moisture analyzer underestimated the moisture content compared with the laboratory oven dry method. In addition, they indicated that the small volume size of the tested sample prohibited the use of this device for unbound materials with aggregates exceeding 1 in. in diameter (Berney et al. 2011).

Field Microwave Oven

The field microwave oven provides a fairly reliable moisture measurement in a very short period of time. According to ASTM D4643-08, a soil sample is repeatedly heated and weighed every minute until the readings become steady, which indicates that the sample is completely dry. ASTM specifies that a 700-W microwave oven should be used for

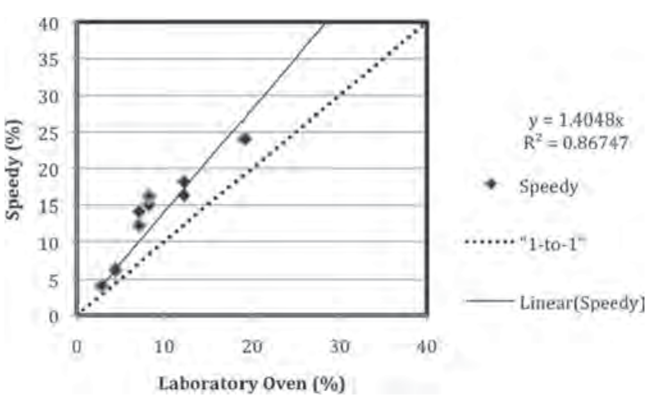


FIGURE 38 Moisture content measured by speedy and oven dry methods (Berney et al. 2011).

FIGURE 39 Sartorious Model MA 150 1,200-g moisture analyzer (Berney et al. 2011).

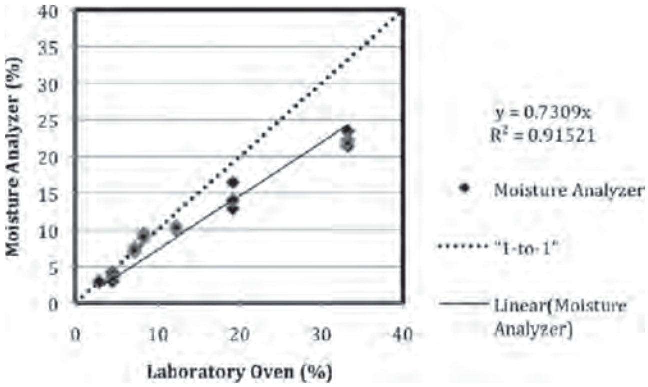


FIGURE 40 Moisture content measured by moisture analyzer and oven dry methods (Berney et al. 2011).

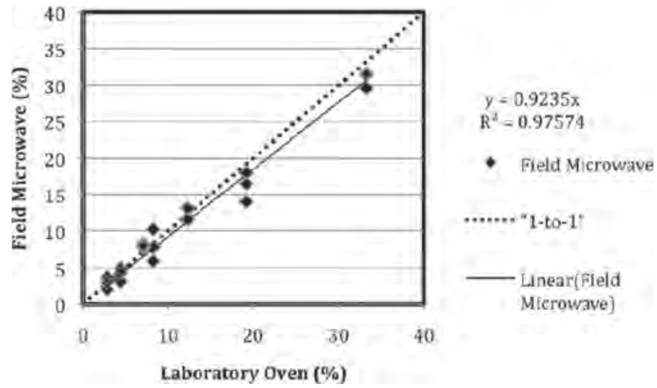


FIGURE 41 Moisture content measured by field microwave oven and laboratory oven dry methods (Berney et al. 2011).

testing. Smaller or larger ovens can be used, but care should be taken to avoid over-drying the soil. Drying times do not have a linear relationship with the wattage of the microwave oven. Rather, drying times decrease exponentially with increased wattage. Because a microwave continues to add energy to the sample, it can drive outbound water in clay minerals if not used according to the ASTM standard, resulting in higher measured values for moisture content. On the other hand, internal studies by the U.S. Army Corps of Engineers have shown that the microwave does not dry out the bound mineral water in gypsum and calcium carbonate soils, making it a superior option than the laboratory oven for those types of soils (Berney et al. 2011).

The field microwave oven method is suitable for materials consisting of particles smaller than 4.75 mm. Care should be taken when testing materials with larger particles because of the increased chance of particle shattering (ASTM D4643-08). The field microwave is dependent on a constant battery source. If the device is not fully charged, the wattage will decrease as the battery loses charge. This could cause considerable error in the measurement (Berney et al. 2011).

Synthesis of Previous Studies

In a study conducted by Berney et al. (2011), the field microwave oven slightly underestimated moisture contents of the tested soils compared with the laboratory oven (Figure 41). In addition, the accuracy of this device deviated for high moisture content silts and clays because of drying of the bound-mineral water in the soil, a water barrier that is not evaporated under constant thermal energy found in a laboratory oven.

DOT600 Roadbed Water Content Meter

The DOT600 Roadbed Water Content Meter (Figure 42) is a portable moisture measuring device in which soil samples collected from field sites are compacted and water content is measured using dielectric permittivity methods. This device is capable of measuring volumetric moisture content and bulk density of soil samples; therefore, gravimetric moisture content also can be calculated. The DOT600 costs about $3,000 (Sebesta et al. 2013).

The device consists of a sample chamber 3 in. in diameter that is retrofitted with a waveguide containing interlaced circuit traces that form a capacitor. The waveguide floats on precision springs. The device's hardware generates and measures a scaled oscillation resonant frequency. Magnetic linear sensors measure sample mass and volume to allow for the determination of gravimetric moisture content. The oscillation

FIGURE 42 DOT600 Roadbed Water Content Meter (Sebesta et al. 2013).

frequency of the circuit decreases as moisture content of the sample increases. This frequency is empirically calibrated to obtain moisture content. The manufacturer provides calibration of this device using soils with a range of textures.

The DOT600 is completely portable, easy to use (minimal training is required), and quickly tests soil moisture content (takes only about 90 s). One of the major limitations of the device is that it cannot be used to test coarser materials. The accuracy of the moisture measurements can be affected by soil type and by soil salinity.

Synthesis of Previous Studies

Minnesota DOT studied the accuracy and effectiveness of the DOT600 for measuring soil moisture content (Minnesota DOT 2012; Hansen and Nieber 2013). In this study, DOT600 measurements were compared with those taken using standard Proctor test for 270 samples. The optimum moisture contents obtained using the DOT600 were consistent with the measurements determined during the standard Proctor test. In addition, researchers found that the variability of moisture measurements using DOT600 was far less than the variability in the optimum moisture constant determined during the Proctor test. Based on overall performance, the Minnesota DOT found that the DOT600 can replace the NDG as well as the sand cone and Proctor tests. However, they suggested some modifications before it could be considered a viable alternative, such as making the device rugged enough for regular field use.

Percometer

The Percometer estimates the moisture content of soils by measuring the dielectric permittivity and conductivity. As shown in Figure 43, the device consists of a 6-cm diameter probe attached to a processor. The probe is designed for insertion into soft materials (a minimum depth of 10 cm is required for accurate measurement) and is ideal for measuring forest soils and road subgrade. When the probe is pressed firmly against the material to be tested, the device emits a small electrical current. The dielectric permittivity and conductivity values are calculated as the current moves through the soil between electrodes on the probe (Camargo et al. 2006). Then the moisture content of the soil is calculated from the measured dielectric permittivity (Camargo et al. 2006). The Percometer costs $7,735.

FIGURE 43 Percometer (Roadscanners 2006).

Synthesis of Previous Studies

Dai and Kremer (2006) reported that the Percometer was moderate in terms of convenience of use. Portability was reported to be moderate as well because the device has no casing. The device's readings were found to be inconsistent if the soil surface was rough and voids on the surface were not completely filled. This tendency was more pronounced with reclaimed materials. The researchers also found that although the dielectric constant increased as the moisture content increased, data were scattered, indicating high variability in test results.

Trident Moisture Meter

The Trident moisture meter uses dielectric permittivity to determine the moisture content of sand, gravel, crushed stone, and other fine and coarse aggregate. The Trident uses its five-pronged sensor, shown in Figure 44, to measure the complex dielectric constant of the material encompassed by the outer four prongs. The manufacturer recommends taking an average of five to 10 readings to ensure an accurate measurement. The integrated microprocessor converts the dielectric constant

FIGURE 44 Trident handheld microwave moisture meter (James Instruments, Inc. 2012).

to moisture content value and displays it as a percentage of dry weight. Material-specific calibration is required for highest accuracy.

This device is completely portable, and testing is easy and fast. It provides accurate moisture content measurement and has good data storage capability. However, it requires material-specific calibration and cannot be used for aggregates with a size greater than 25 mm. The Trident costs about $1,900.

Synthesis of Previous Studies

The Trident moisture meter is relatively new, and only a limited number of studies on its use can be found in the literature. Jean-Louis and Gabriel (2010) investigated the correlation between the measurements taken with the Trident and the actual oven moisture contents of selected soil at different levels of compaction. Test results indicated that the Trident moisture content measurements had a strong correlation with those obtained using the traditional oven dry method for both compacted and loose soils.

SUMMARY

This chapter reviews the density-based compaction control methods that have been used by state DOTs. It also provides a comprehensive evaluation of non-nuclear devices used to measure density and/or moisture content based on the results of investigations reported in the literature. Despite that they are used by the majority of DOTs in determining the target density value, the AASHTO T99 and AASHTO T180 standards cannot be used for unbound granular materials that have more than 30% by mass of their particles with sizes greater than 19 mm (¾ in.). Furthermore, past research studies indicated that there are other types of laboratory compaction tests that might be more suitable for granular unbound materials because they may provide better replicates of field densities. Although previous studies indicated that non-nuclear devices have some advantages over the NDG, such as not requiring special licensing to operate, they were found to be more difficult to operate and require longer testing time. Finally, there are several non-nuclear devices that can measure moisture content; however, only limited studies have been conducted to evaluate most of them.

CHAPTER FOUR

METHODS FOR MEASURING THE IN SITU STIFFNESS/STRENGTH OF UNBOUND MATERIALS

INTRODUCTION

Although the density measurement has been long used for compaction control, it does not reflect the engineering properties of unbound materials necessary to ensure their optimal performance. The key functional properties of soil layers are their stiffness and strength, which are considered to be measures of their stability and resistance to deformation under load. Although the stiffness of a material defines its resistance to deformation before failure, the strength is its limiting stress value at failure. Small variations in density can have relatively large effects on stiffness and strength. Therefore, the errors that accumulate during the specified density procedure have the potential to significantly influence the performance of compacted unbound materials (White et al. 2007a). Stiffness and strength are also sensitive to variations in the moisture content, degree of saturation, and state of stress of compacted unbound materials, which all govern the mechanical behavior and response of these materials.

In recent years, the shift from empirical to mechanistic–empirical pavement design procedures has resulted in a growing interest in moving toward compaction control specifications that emphasize stiffness and strength. This has led to the development of several in situ test devices that can measure the stiffness or strength of compacted unbound materials. These devices can be divided into four main groups. The first group consists of impact devices, such as the dynamic cone penetrometer (DCP) and the Clegg hammer (CH). The second category consists of devices that apply static, vibratory, or impact load to the ground, then estimate the stiffness based on the load and displacement measurements (using velocity transducers or accelerometers); these devices include the Briaud compaction device (BCD), the GeoGauge, and the light weight deflectometer (LWD). A third group includes devices that are based on geophysical techniques and includes the portable seismic property analyzer (PSPA), in which surface waves are generated and detected in the tested layer to determine its modulus. Finally, the fourth group consists of sacrificial sensors that are buried in the compacted soil to monitor the growth in amplitude of compression waves during compaction.

In addition to the previous in situ spot tests, technologies that provide continuous assessment of compaction, such as continuous compaction control (CCC) and intelligent compaction (IC), have been investigated by DOTs as a viable tool for controlling the quality of compaction of various pavement layers and subgrade soils.

This chapter summarizes information collected through literature review of the performance of various in situ test devices and methods that have been evaluated by state DOTs to measure stiffness, strength, or any parameter other than density for use in compaction control of unbound materials. For each device, the principle of operation, influence depth, reliability of measurement, and advantages and limitations are first provided. In addition, a summary of the main findings of previously conducted studies is presented. In the preceding sections, pictures of in situ devices from certain manufacturers are provided for demonstration purposes only. Inclusion of photos of these devices should not be construed as endorsements of the devices by this synthesis study.

CLEGG HAMMER

The CH was developed in Australia in late 1960s to measure the stiffness/strength of soils (Rathje et al. 2006). It consists of a flat-end hammer operating within a vertical guide tube. The hammer has a precision accelerometer attached to its end that sends signals to a digital readout unit upon contact with the soil surface. A schematic representation of the CH is presented in Figure 45. The standard hammer has a diameter of 51 mm (2 in.) and weighs 4.5 kg (10 lb). However, CH models with different hammer masses are available. Figure 46 presents photographs of some of these models. The hammer mass used depends on the application. Table 8 presents the various available hammer masses and their applications. Farrag et al. (2005) found that the performance of the 20-kg CH was similar to that of the 10-kg hammer, but larger hammer was less sensitive to small changes in relative compaction. The basic CH system costs approximately $3,000, but the complete system can cost as much as $20,000. This price is the same for all available hammer masses (Rathje et al. 2006).

Principle of Operation

The basic principle behind the CH is to obtain a measure of the deceleration of a free-falling mass from a set height onto a soil surface. The standard method for testing using this device is ASTM D5874. According to this method, the hammer is to be raised 457 mm (18 in.) after placing the device on a compacted lift. The hammer is then released so that it freely

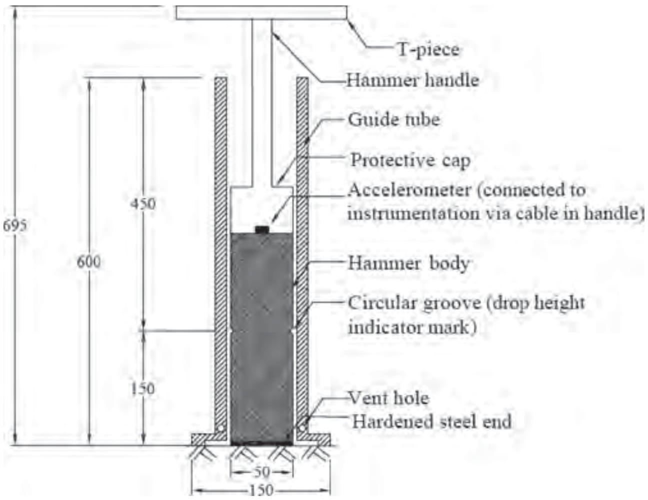


FIGURE 45 Clegg impact hammer (modified after ASTM D5874).

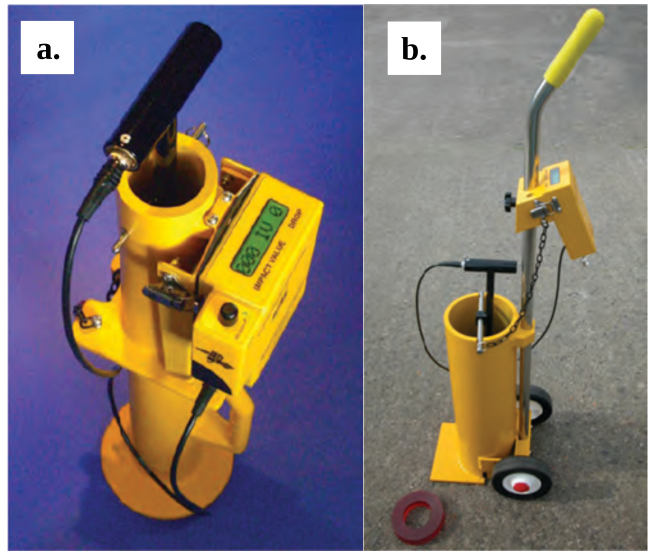


FIGURE 46 Clegg hammers: (*a*) 4.5-kg Clegg hammer, (*b*) 10/20-kg Clegg hammer (Farrag et al. 2005).

falls within the guide tube. During impact, the accelerometer mounted on the hammer produces an electric pulse, which is converted and displayed on the control unit. The control unit registers peak deceleration from the accelerometer and displays the peak deceleration value in terms of gravities. Four consecutive drops should be performed in the same place, according to the ASTM D5874 standard. The Clegg impact value (CIV) is the largest deceleration measured during the four drops. The ASTM D5874 standard states that the first two blows act as a seating mechanism, with CIVs increasing during the first three drops and remaining generally constant after the fourth.

Clegg (1994) proposed equations shown in Table 9 to convert CIVs to the Clegg hammer modulus (CHM) for commonly used Clegg hammers. The equations are derived using double integration of time versus deceleration to determine the deflection, which is then used to compute the elastic modulus based on elastic plate bearing theory.

Use of Clegg Hammer in Compaction Control

In using the CH for compaction control, it is typically required to specify the target CIV for the soil to be compacted. ASTM D5874 describes three laboratory methods for determining the target CIV. The methods involve measuring the CIV at the optimum moisture content, measuring the CIV at a range of moisture contents, or measuring the CIV at a range of dry densities at the optimum moisture content. Each of these methods can use the data obtained from either the standard Proctor or the modified Proctor compaction test.

To determine a target CIV for the optimum moisture content, a soil sample is compacted in a Proctor mold at the optimum moisture content. The CH is then used to measure the CIV, which represents the minimum required value for field compaction. To determine a target CIV from a range of moisture contents, four samples with different moisture contents bracketing the optimum moisture content are compacted in molds at the maximum dry density obtained in the standard or modified Proctor tests. The CIVs are measured for each sample to develop a curve of CIV versus moisture content, and the maximum value is selected as the target CIV in the field. Finally, to set a target CIV from a range of dry densities, four samples are compacted in Proctor molds at the optimum moisture content. Each sample is compacted with a different

TABLE 8
RECOMMENDED APPLICATION FOR VARIOUS AVAILABLE CLEGG HAMMER MASSES

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

Source: Mooney et al. (2008).

TABLE 9
CIV TO MODULUS CONVERSION EQUATIONS

CHM/H (MPa) = 0.23 $(CIV/H)^2$
CHM/S (MPa) = 0.088 $(CIV)^2$
CHM/M (MPa) = 0.044 $(CIV/M)^2$
CHM/L (MPa) = 0.015 $(CIV/L)^2$

*Note: The 20-kg CH is called the "heavy" CH and thus CIV/H and CHM/H to denote the CIV and the CHM of this mass with its diameter and drop height. Likewise, the 2.25-kg CH is known as the "medium" hammer and thus CIV/M and CHM/M. The 0.5-kg CH is called the "light" CH, so the notation is seen as CIV/L and CHM/L. The 4.5-kg CH is considered the "standard" CH. CIV using the standard CH usually is notated simply as CIV, but CHM/S is useful for distinguishing a Clegg hammer modulus derived using the standard CH.
Source: Clegg (1994).

number of blows to produce dry density values ranging from 90% to 100% of the maximum dry density value obtained in standard or modified Proctor tests. The measured CIVs are used to develop a curve of CIV versus relative dry density at the optimum moisture content. The target CIV is selected as the CIV on the developed curve that corresponds to the required percent relative compaction for the site.

Repeatability

In general, for field use, the coefficient of variation (COV) of CIV is 4% for highly uniform working conditions and 20% for highly variable conditions (ASTM D5874). Mooney et al. (2008) evaluated the repeatability of the CH measurement test and found that the hammer's precision had an average precision uncertainty of ±4.8%. Rathje et al. (2006) found that the repeatability was medium for both the 10- and 20-kg hammers.

Influence Depth

Influence depth is the depth in the unbound materials at which the imparted stress by a device becomes negligible. If two or more layers of unbound materials exist within the influence depth of a device, the device measurement will provide a composite value of the two layers rather than the value for the tested layer. Therefore, the determination of the influence depth for test devices is important so that the stiffness/strength value can be associated with the appropriate lift thickness.

Few studies have evaluated the influence depth of the CH. Mooney and Miller (2008) reported that the influence depth ranged between one and one-and-a-half times the hammer diameter and to a maximum of 250 mm (10 in.) for the 10- and 20-kg hammers. White et al. (2007a) found that the influence depth was less than 300 mm (12 in.) for the same size hammers. However, Farrag et al. (2005) reported a lower influence depth of 203 mm (8 in.).

Advantages and Limitations

The CH is simple to use, requires minimum training, has a standard test procedure (ASTM D5874), and can be outfitted with an integrated GPS system (Farrag et al. 2005; Mooney et al. 2008). In addition, its results can be obtained in a short period of time (less than 60 s) and are not operator dependent. Good correlation exists between the CIV and California bearing ratio (CBR) values for different types of soils (Aiban and Aurifullah 2007; Fairbrother et al. 2010). Despite these advantages, several limitations of earlier models of the CH were reported in previous studies (Rathje et al. 2006; Mooney et al. 2008). These included poor portability and mobility, particularly with the heavy, 20-kg hammer. In addition, all CH models were found to have weak connections for field use, which significantly affected their durability. The CH was also found to have limited data storage and downloading capability, which can be an issue when used in large construction projects (Rathje et al. 2006). Farrag et al. (2007) conducted a study to modify and optimize the CH device for soil compaction measurements in the field. The project included physical modifications of the device to reduce its weight and improve its mobility. It also proposed electronic modifications to provide moisture measurement by means of a moisture probe and develop data storage and downloading capabilities. Although the modified CH model appears to address most of these aforementioned limitations, no studies have been done to report on its efficacy in the field.

Mooney et al. (2008) indicated two additional problems with the CH. The first was the inaccuracy of the target CIV obtained by measuring the CIV of a sample compacted in a Proctor mold as a result of boundary effects. The second arose in the testing of soft soils, where the hammer penetrated the soil so quickly that its handle struck the guide tube.

Synthesis of Past Research Studies

Indiana Study

Kim et al. (2010) reported the results of a study that included performing DCP and CH tests on several road sites within Indiana, as well as on clayey soil samples prepared in a test pit and sand samples prepared in a test chamber. The results of this study indicated that the relationship of the CH's CIV with relative compaction exhibited considerable variability. Based on these findings, the authors suggested that the CH could not be used for compaction control of unbound materials.

Maine Study

In a laboratory study in which the CH and LWD were used to test five types of base and subbase aggregates compacted in a test container, Steinart et al. (2005) found that the modulus values obtained using the CH measurements were much lower than the LWD moduli. In addition, as shown in Figure 47, a weak correlation was found between the modulus values of the LWD and the CH. The authors attributed the lower CH moduli to the occurrence of a shallow bearing capacity failure caused by the impact of the Clegg Impact Hammer (CIH). They also found that the modulus values determined from the CH's first drop were less than those obtained from subsequent drops. In addition, the CH moduli tended to increase with each subsequent drop.

Texas Study

Rathje et al. (2006) evaluated the relationships between CIV and the moisture content and dry density for three types of soils: high-plasticity clay, low-plasticity clay, and well-graded sand. This was done by measuring the CIV for soil samples with a range of moisture contents compacted in Proctor molds using standard and modified Proctor compaction efforts, as well as testing soil samples at a constant moisture content and variable dry density. Rathje et al. (2006) indicated that for clayey soils, the CIVs were more affected by the moisture content than the dry density. However, for sandy soils the CIVs generally were affected by moisture content and dry density such that they increased with increasing dry density. The authors also determined the target CIV, which was chosen as the maximum value obtained over the range of moisture contents tested. For samples compacted using the standard Proctor compaction effort, the target CIVs for high-plasticity clay, low-plasticity clay, and well-graded sand were found to be 7.1, 7.9, and 21, respectively.

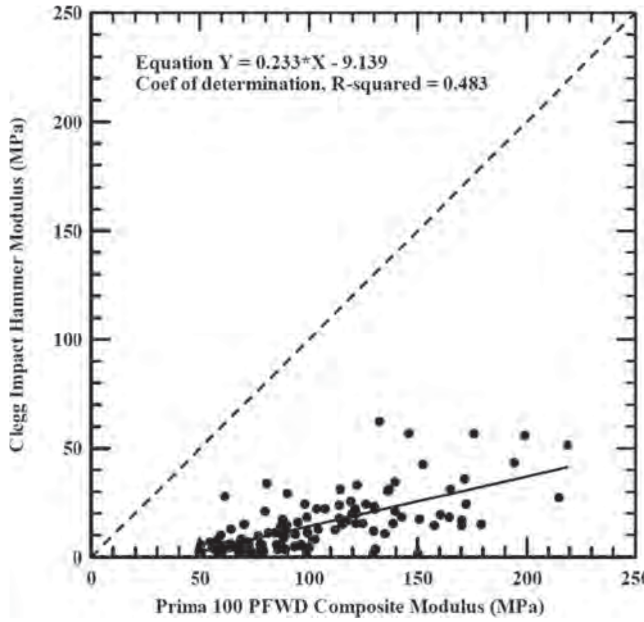


FIGURE 47 Comparison between Clegg hammer and LWD modulus (Steinart et al. 2005).

Virginia Studies

Erchul and Meade (1990) studied the correlation between the dry density and CIV obtained from the CH. They concluded that the CIV was a good indicator of the degree of compaction for granular materials. The authors found that the use of the CH to estimate dry density values required careful calibration for each material under consideration. In another study, Erchul and Meade (1994) added a penetration scale to the handle of the CH to record the depth to which its hammer penetrated the soil. By comparing the CIV and penetration data with density and moisture content measurements obtained using the NDG, the authors developed a graphical acceptance criterion, shown in Figure 48, for utility trench backfill compaction in Chesterfield County, Virginia.

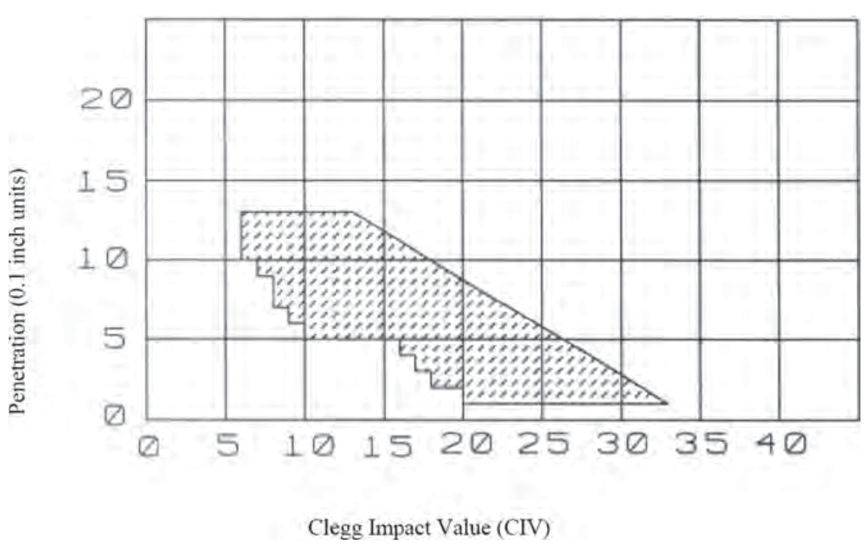


FIGURE 48 Acceptance criterion for Clegg hammer (Erchul and Meade 1994).

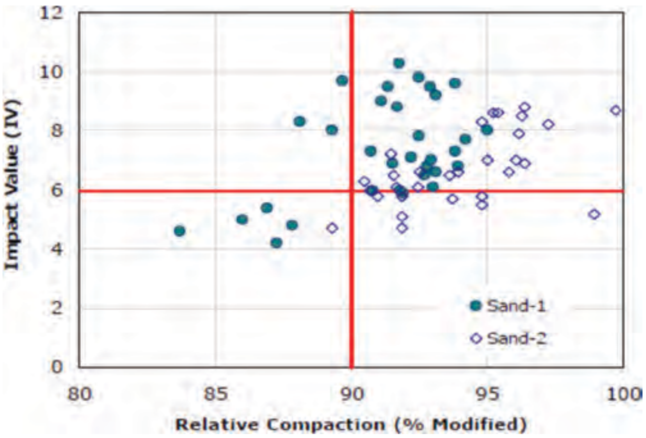


FIGURE 49 The 10-kg hammer CIV compared with relative compaction in sand (Farrag et al. 2005).

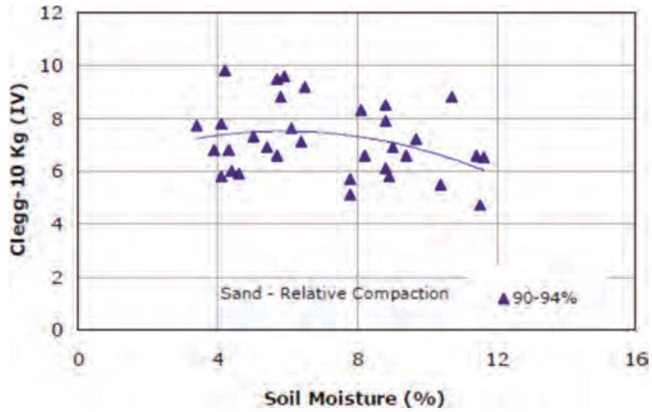


FIGURE 50 Effect of moisture contents on CIV results in sand (Farrag et al. 2005).

New York State Electric & Gas Corporation Study

The New York State Electric & Gas Corporation conducted a field study to compare the CH with the dry density measurement obtained using the NDG (Peterson and Wiser 2003). The study involved obtaining 15 measurements at 12 trench backfill sites in Broome County, New York, consisting of crushed rock and gravel. Readings were taken after each lift was compacted to 90% standard Proctor with a tamper. Target CIVs were based on 90% standard Proctor dry unit weight. The study determined that the CH accurately identified the 90% relative compaction for 84% of the measurements obtained (Peterson and Wiser 2003).

Gas Technology Institute Studies

Farrag et al. (2005) reported the results of a study conducted by the Gas Technology Institute, which included testing trenches constructed with various types of unbound materials using 10-kg and 20-kg CHs. The relationships between the 10-kg hammer CIV, 20-kg hammer CIV, and the relative compactions of sand, silty clay, and stone were investigated in that study. Figure 49 shows the relationship between the 10-kg hammer CIVs and relative compaction of sand. Based on the field test results, the authors concluded that the values from both CH models had weak correlations with the relative density for sand and stone-base materials and better correlations in silty clay soil. The CIVs corresponding to 90% relative compaction found in that study are summarized in Table 10.

The effect of moisture content on 10-kg hammer CIV was also investigated in the same study for all three types of unbound materials. Figure 50 presents the variation of CIVs with the moisture content of tested sand material. The CIVs increased with the increase in the moisture content to a certain point and then decreased at higher moisture contents. Similar results were obtained for the other tested materials. Farrag et al. (2005) also reported that the maximum moisture content values were not necessarily equal to the optimum moisture values obtained from modified Proctor tests. Finally, the study concluded that the performances of the 20-kg and 10-kg hammers were similar.

International Studies

The CH has been evaluated by numerous international studies, the majority of which looked at the correlation between the CIV and the CBR. In general, good correlations were found between CIV and CBR for different types of unbound materials (Clegg 1980; Mathur and Coghlan 1987; Gulen and McDaniel 1990; Pidwerbesky 1997; Al-Amoudi et al. 2002; Aiban and Aurifullah 2007; Fairbrother et al. 2010). Most researchers found the relationship to be exponential. The first correlation, shown in Eq. 10, was presented by Clegg (1980), which was based on laboratory tests done in soils in Australia. Clegg (1987) used the data collected from laboratory and in situ tests conducted on a wide range of soils in Australia, New Zealand, and the United Kingdom to propose a slightly modified correlation, shown in Eq. 11. Al-Amoudi et al. (2002) conducted comprehensive

TABLE 10
CLEGG HAMMER RESULTS CORRESPONDING TO 90% RELATIVE COMPACTION AT OPTIMUM MOISTURE CONTENT

Hammer Type	Sand	Silty Clay	Stone-base
10-kg Hammer (CIV)	6	8	14
20-kg Hammer (CIV)	5	6	9

Source: Farrag et al. (2005).

TABLE 11
SUMMARY OF CBR-CIV RELATIONSHIP PROPOSED BY AL-AMOUDI ET AL. (2002)

Type of Data		Correlations	R^2
Laboratory		CBR = 0.1977 $(\mathrm{CIV})^{1.535}$	0.810
In situ	GM soil	CBR = 0.8610 $(\mathrm{CIV})^{1.1360}$	0.757
	SM soil	CBR = 1.3577 $(\mathrm{CIV})^{1.0105}$	0.845
	GM and SM soils (combined)	CBR = 0.1977 $(\mathrm{CIV})^{1.0115}$	0.846
Laboratory, in situ and literature data		CBR = 0.1691 $(\mathrm{CIV})^{1.695}$	0.850

laboratory and field testing programs to evaluate the CIV–CBR correlation and proposed the correlations shown in Table 11 for different soil types.

$$\mathrm{CBR} = 0.07\,(\mathrm{CIV})^2 \qquad (10)$$

$$\mathrm{CBR} = [0.24(\mathrm{CIV}) + 1]^2 \qquad (R^2 = 0.916) \qquad (11)$$

Based on laboratory testing of steel slag and limestone aggregate base materials, Aiban and Aurifullah (2007) proposed a slightly different model, shown in Eq. 12, than that proposed by Clegg (1980). More recently, Fairbrother et al. (2010) tested 17 subgrade soil samples that were collected from six locations in the East Cape region of New Zealand. Based on those tests, they proposed Eq. 13 to correlate the CIV to the CBR. It can be noted that Fairbrother et al. (2010) recommended that their equation not be used to estimate the CBR of soft subgrade soils because it will overestimate the CBR strength of soils in that condition.

$$\mathrm{CBR} = 0.513(\mathrm{CIV})^{1.417} \qquad (R^2 = 0.94) \qquad (12)$$

$$\mathrm{CBR} = 0.564(\mathrm{CIV})^{1.144} \qquad (13)$$

Few studies have compared the CH's CIV with moduli obtained using other in situ test devices. Whaley (1994) conducted a study that included testing base course materials using the Loadman LWD, standard falling weight deflectometer (FWD), CH, and Benkelman beam.

Figure 51 presents a comparison of measurements obtained from the different devices. Whaley (1994) concluded that poor correlation exists between the CH and the other considered in situ test devices. Pidwerbesky (1997) reached a similar conclusion.

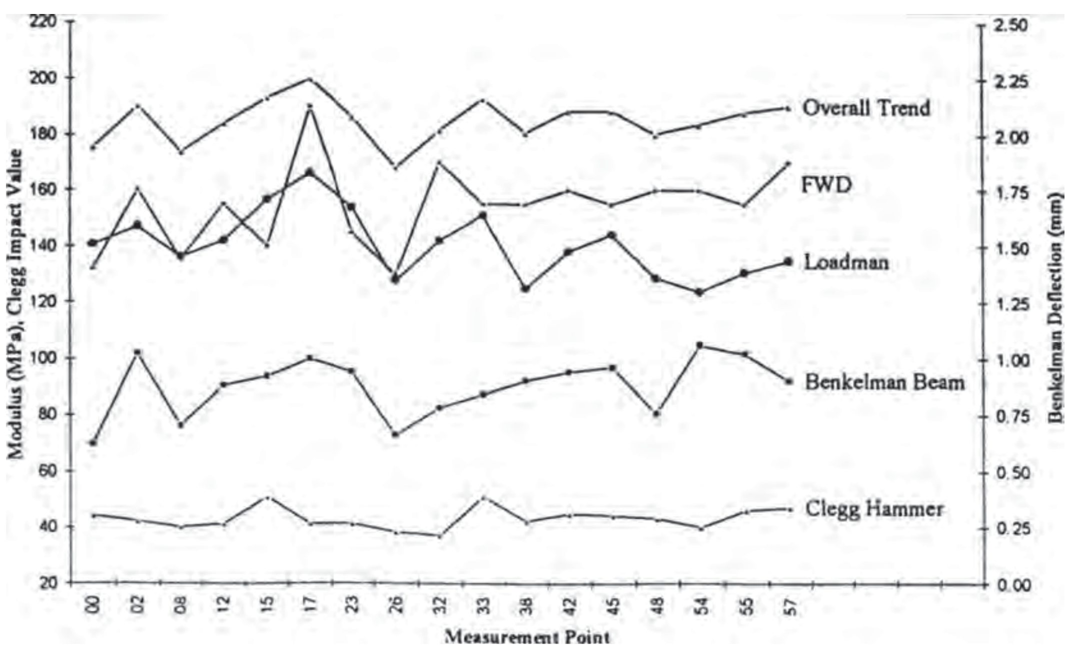


FIGURE 51 Comparison between Clegg hammer and other in situ tests (Whaley 1994).

FIGURE 52 Soil stiffness gauge or GeoGauge.

SOIL STIFFNESS GAUGE (GEOGAUGE)

The soil stiffness gauge, or GeoGauge (Figure 52), measures the in-place stiffness of compacted soil at the rate of about one test per 1.25 min. It weighs about 10 kg (22 lb) and measures 280 mm (11 in.) in diameter and 254 mm (10 in.) in height. The GeoGauge rests on the soil surface by means of a ring–shaped foot. Its annular ring contacts the soil with an outside diameter of 114 mm (4.5 in.), an inside diameter of 89 mm (3.5 in.), and a thickness of 13 mm (0.5 in.) (Lenke et al. 2003). The price reported in previous studies for the GeoGauge ranged between $5,000 and $5,500 (Mooney et al. 2008). The testing procedure of the GeoGauge involves setting it on the test location and giving the device a slight twist to ensure a minimum of 80% contact between the foot and the soil. The manufacturer recommends using a thin layer of sand when 80% contact cannot be achieved.

Principle of Operation

The principle of operation of the GeoGauge is to generate a very small dynamic force at frequencies of 100 to 196 Hz. In a laboratory study, Sawangsuriya et al. (2002) estimated the force generated by the GeoGauge to be 9 N. The GeoGauge operation includes generating very small displacements to the soil, which is less than 1.27×10^{-6} m (0.0005 in.), at 25 steady state frequencies between 100 and 196 Hz. The stiffness is determined at each frequency, and the average is displayed. The entire process takes about 1.5 min. The GeoGauge is powered by a set of six D-cell batteries. It is designed such that the deflection produced from equipment operating nearby will not affect its measurements because the frequency generated by traffic (at highway speed) is approximately 30 Hz, below the GeoGauge operating frequency (Sawangsuriya et al. 2001).

The force applied by the shaker and transferred to the ground is measured by differential displacement across the flexible plate by two velocity sensors (Figure 53). This can be expressed using Eq. 14. At frequencies of operation, the ground-input impedance will be dominantly stiffness controlled, such that soil stiffness can be obtained using Eq. 15.

$$F_{dr} = K_{\text{flex}}(X_2 - X_1) = K_{\text{flex}}(V_2 - V_1) \qquad (14)$$

where

F_{dr} = force applied by shaker,
K_{flex} = stiffness of the flexible plane,
X_1 = displacement at rigid plate,

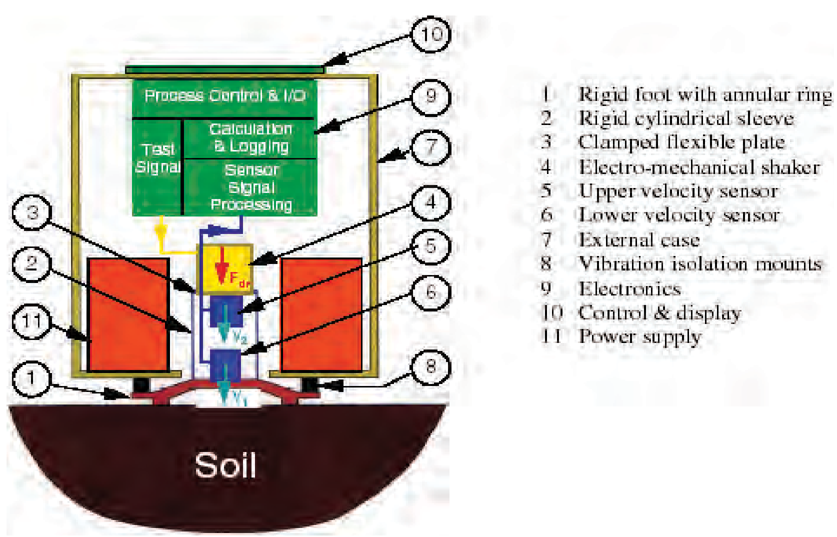


FIGURE 53 Schematic of the GeoGauge (Humboldt 1998).

X_2 = displacement at flexible plate,
V_1 = velocity at rigid plate, and
V_2 = velocity at flexible plate.

$$K_{\text{soil}} = K_{\text{flex}} \sum_1^n \left(\frac{\dfrac{(X_2 - X_1)}{X_1}}{n} \right) = K_{\text{flex}} \sum_1^n \left(\frac{\dfrac{(V_2 - V_1)}{V_1}}{n} \right) \qquad (15)$$

where

n = number of test frequencies, and
K_{soil} = stiffness of soil.

Using velocity measurements eliminates the need for a nonmoving reference for soil displacement and permits accurate measurement of small displacements. It is assumed that GeoGauge response is dominated by the stiffness of the underlying soil. The measured soil stiffness from the GeoGauge can be used to calculate the soil elastic modulus. The static stiffness, K, of a rigid annular ring on a linear elastic, homogeneous, and isotropic half space has the following functional form (Egorov 1965):

$$K = \frac{ER}{(1 - v^2)\omega(n)} \qquad (16)$$

where

E = modulus of elasticity,
V = Poisson's ratio of the elastic medium,
R = the outside radius of the annular ring, and
$\omega(n)$ = a function of the ratio of the inside diameter and the outside diameter of the annular ring.

For the ring geometry of the GeoGauge, the parameter $\omega(n)$ is equal to 0.565, thus

$$K = \frac{1.77ER}{(1 - v^2)} \qquad (17)$$

Based on Eq. 17, the GeoGauge stiffness could be converted to an elastic stiffness modulus using the equation proposed by CA Consulting Engineers, as follows (Eq. 18):

$$E_G = H_{SG} \frac{(1 - v^2)}{1.77R} \qquad (18)$$

where

E_G = the elastic stiffness modulus (MPa),
H_{SG} = the GeoGauge stiffness reading (MN/m), and
R = the radius of the GeoGauge foot [57.15 mm (2.25 in.)].

For a Poisson's ratio of 0.35, a factor of approximately 8.67 can be used to convert the GeoGauge stiffness (in MN/m) to a stiffness modulus (in MPa). It is recommended that the GeoGauge be used only for materials with stiffness to 23 MN/m because it may lose accuracy when measuring stiffness greater than that value (Chen et al. 2000).

The GeoGauge manufacturer also has suggested that the dry density of compacted soils can be determined from GeoGauge stiffness using Eq. 19, which was developed based on the work of Hryciw and Thomann (1993). In this equation, the calibration factor "C" should first be determined for tested geomaterial. This is done by measuring GeoGauge stiffness (K) along with the dry density of the geomaterial to be tested and solving Eq. 20. Several studies indicated that the GeoGauge performed poorly when used to determine dry density.

$$\rho_D = \frac{\rho_o}{1 + 1.2\left[\dfrac{Cm}{K} - 0.3\right]^{0.5}} \qquad (19)$$

$$C = \left(\frac{K}{m}\right)\left\{\left[\frac{\left(\dfrac{\rho_o}{\rho_D} - 1\right)}{1.2}\right]^2 + 0.3\right\} \qquad (20)$$

where

ρ_D = the dry density of the soil;
ρ_o = the ideal, void free density; and
m = moisture content.

Repeatability

Several previous studies have indicated that the GeoGauge had similar or better repeatability than other in situ test devices. Maher et al. (2002) reported that the GeoGauge had excellent repeatability when conducting consecutive measurements on different soil types. Abu-Farsakh et al. (2004) found that the GeoGauge's COV was between 0.2% and 11.38% for field test sections and ranged from 2.3% to 38.8% for tests conducted in laboratory test sections. Von Quintus et al. (2008) indicated that when testing seven types of soils at seven test sites, the COVs of GeoGauge measurements ranged from 7.1% to 20.1%. In another field study, Hossain and Apeagyei (2010) found that the GeoGauge had lower spatial variability than did the LWD and DCP, but the reported COV for GeoGauge moduli was 8% to 42%. As part of tests conducted in Phase I of the NCHRP 10-48 project, Nazarian (2012) reported that the COV of GeoGauge measurement was less than 10%. The precision of GeoGauge measurement on fine-grained soils was reported to be less than 2% and on coarse-grained soils and crushed aggregate less than 5%. The repeatability of the GeoGauge was also evaluated in a field study conducted at the Louisiana Transportation Research Center as part of a FHWA study [SPR-2(212)] for the validation of the seating procedure for the GeoGauge. Fifty-four GeoGauge measurements were taken at each test location, and the COV calculated for all measurements made. The COV for measurements made by all GeoGauges ranged from 6.1% to

9.5%. Finally, some studies have reported that the GeoGauge results are extremely inconsistent and highly dependent on the seating procedures and the operator (Bloomquist et al. 2003; Mooney et al. 2008).

Influence Depth

Several studies were performed to determine the zone of influence of the GeoGauge. Nazzal (2003) used two test boxes, one box containing compacted clay and another compacted Florolite (plaster of Paris), to determine zone of influence. The average zone of influence of the GeoGauge was found to be 190 to 203 mm (7.5 to 8 in.). Sawangsuriay et al. (2002) found a zone of influence of 127 to 254 mm (5 to 10 in.) for the GeoGauge using cubic boxes filled with medium sand, crushed lime rock, and a mixture of plastic beads with sand. Maher et al. (2002) reported a similar influence zone for the GeoGauge.

Advantages and Limitations

The GeoGauge test is simple with minimal training required to perform it. In addition, it is fast (75 s per test) and has a well-defined test specification (ASTM D6758). The GeoGauge device also has good portability, durability, data storage, and download capabilities. Previous studies have reported some of the GeoGauge's limitations. First, its reading is sensitive to the stiffness of the top 2 in. of the tested soil layer, as well as to the seating procedure (Bloomquist et al. 2003; Farrag et al. 2005). Furthermore, previous studies reported that often there was difficulty in achieving good contact between the GeoGauge ring and the tested soil (Simmons 2000; Ellis and Bloomquist 2003; Miller and Mallick 2003). Simmons (2000) also found that the use of leveling sand for surface preparation that is recommended by the manufacturer can significantly affect GeoGauge measurements.

Another limitation of the GeoGauge is the very small load that it applies, which does not represent the stress levels actually encountered in the field as a result of traffic. Therefore, the GeoGauge modulus must be corrected to account for design loads. In addition, GeoGauge measurement has been found to be very sensitive to changes in moisture content (Nazzal 2003). Finally, Miller and Mallick (2003) raised concerns about the GeoGauge malfunctioning owing to vibrations from passing vehicles, such as compaction equipment or trains.

Synthesis of Past Research Studies

Florida Study

Bloomquist et al. (2003) reported the results of a study that evaluated the effectiveness of GeoGauge as a tool for compaction control of pavement base and subgrade materials. The GeoGauge did not have definitive correlations with dry density, moisture content, or resilient modulus. Furthermore, the repeatability and precision of the GeoGauge was found to be largely dependent on the condition of the soil surface as well as the placement and operation procedure. The researchers attempted to enhance the design of the GeoGauge by developing a new handle to provide a uniform seating of GeoGauge on the soil. The results indicated that the repeatability of the GeoGauge was significantly improved when it was seated on the soil by twisting it with the newly developed handle. Finally, Bloomquist et al. (2003) found that the GeoGauge stiffness value tends to increase as the frequency increases. Therefore, they recommended that certain input frequency ranges be used during testing to reduce the variability in GeoGauge readings.

Hawaii Studies

Pu (2002) evaluated the relationship between GeoGauge stiffness with moisture content, dry unit weight, and CBR. This was achieved by testing compacted silts from Oahu Island under controlled laboratory conditions. Results showed no direct relationship between GeoGauge stiffness and dry unit weight, because a GeoGauge stiffness value can correspond to different values of dry unit weight depending on the moisture content. Pu (2002) derived a relationship between GeoGauge stiffness, dry unit weight, and moisture content. However, this relationship requires detailed information on the soil water characteristic curves. The author concluded that the GeoGauge could provide an alternative method for compaction control that used stiffness instead of dry unit weight. However, he indicated that the soil shrink/swell potential is not optimized if stiffness is used. Therefore this issue needs to be addressed before stiffness-based compaction control specification is implemented. Finally, Pu (2002) indicated that no direct relationship existed between GeoGauge measurements and the soaked CBR because the GeoGauge provided a measure of stiffness at a much smaller displacement than that encountered during CBR testing (2.5 to 5 mm).

Ooi et al. (2010) conducted GeoGauge and LWD tests on recycled concrete aggregate (RCA) and reclaimed asphalt pavement (RAP) lifts 6-in. thick compacted in bins 3 ft in diameter. The results showed that the LWD consistently provided higher moduli than did the GeoGauge. In addition, the LWD had better repeatability. Finally, the authors suggested that it is important to consider the zone of influence when interpreting LWD and GeoGauge moduli.

Louisiana Studies

Abu-Farsakh et al. (2004) conducted a comprehensive study to evaluate the use of the GeoGauge, DCP, and LWD to reliably measure the stiffness/strength characteristics of unbound materials for application in the quality control/

quality assurance (QC/QA) procedures during and after construction of pavement layers and embankments. The study included conducting GeoGauge, DCP, LWD, standard FWD, and static plate loading (PLT) tests on different base course materials and subgrade soils in several pavement sections at three project sites in Louisiana as well as at the Louisiana Department of Transportation and Development Accelerated Load Facility (ALF). In addition, tests were performed on sections constructed in two laboratory boxes measuring 1.5 × 0.91 × 0.76 m (5 × 3 × 2.5 ft). The CBR laboratory tests were also conducted on samples collected during the testing of different sections. Abu-Farsakh et al. (2004) found that the GeoGauge was the most user-friendly tool among the three devices evaluated in this study because it was durable, easy to operate, and provided rapid results. Based on the results of this study, the following correlations were found between GeoGauge and the two standard in situ tests (FWD and PLT):

$$M_{FWD} = -20.07 + 1.17(E_G)$$
$$\text{for } 40.8 \text{ MPa} < E_G < 194.4 \text{ MPa} \qquad (R^2 = 0.81) \tag{21}$$

$$E_{PLT(i)} = -75.58 + 1.52\,(E_G)$$
$$\text{for } 40.8 \text{ MPa} < E_G < 194.4 \text{ MPa} \qquad (R^2 = 0.87) \tag{22}$$

$$E_{PLT(R2)} = -65.37 + 1.50\,(E_G)$$
$$\text{for } 40.8 \text{ MPa} < E_G < 194.4 \text{ MPa} \qquad (R^2 = 0.90) \tag{23}$$

where

M_{FWD} = FWD back-calculated modulus (MPa),
$E_{PLT\,(i)}$ = initial moduli from the PLT (MPa),
$E_{PLT(R2)}$ = reloading moduli from the PLT (MPa), and
E_G = GeoGauge modulus (MPa).

Abu-Farsakh et al. (2004) also proposed the regression model shown in Eq. 24 to correlate the GeoGauge moduli with the CBR values obtained by testing soil samples collected from field test sections. All samples were prepared in accordance with ASTM D1883-99 without soaking them to mimic the field conditions.

$$\text{CBR} = 0.00392\,(E_G)^2 - 5.75$$
$$\text{for } 40.8 \text{ MPa} < E_G < 184.11 \text{ MPa} \qquad (R^2 = 0.84) \tag{24}$$

Zhang et al. (2004) evaluated the use of the GeoGauge for controlling trench backfill construction. Three trenches were excavated with dimensions of 1.3 × 5 × 1 m (4 × 15 × 3 ft). Each trench consisted of three layers, each with a thickness 300 mm (12 in.). Each trench was then divided into three equal sections compacted at different compaction efforts: light, moderate, and heavy. As expected, results indicated that both the dry density and GeoGauge modulus increased with increasing compactive effort. However, this depended on soil type as well as moisture content.

In another study, Mohammad et al. (2009) conducted resilient modulus (M_r) laboratory experiments on soil samples collected from sections tested in the study by Abu-Farsakh et al. (2004). Based on the results of the conducted tests, Mohammad et al. (2009) proposed two models (shown in Eqs. 25 and 26) to predict the laboratory-measured M_r from the GeoGauge modulus. Although the first model directly related the GeoGauge modulus to the M_r value, the second model predicted the M_r value based on the GeoGauge modulus as well as the moisture content of the tested soils.

$$M_r = 46.48 + 0.01 E_G^{1.54} \qquad (R^2 = 0.59) \tag{25}$$

$$M_r = -13.94 + 0.0397(E_G^{0.8}) + 601.08\left(\frac{1}{w^{0.78}}\right) \qquad (R^2 = 0.72) \tag{26}$$

where

M_r = resilient modulus (MPa),
E_G = modulus from GeoGauge test (MPa), and
w = moisture content (%).

Minnesota Studies

Siekmeier et al. (2000) compared the GeoGauge to other in situ test devices, such as the Loadman LWD and the standard FWD. The results of this study showed that the GeoGauge modulus was less than those measured by other in situ test devices. This was attributed to the lower stress imposed on the soil by the GeoGauge (0.02 to 0.03 MPa) compared with that imposed by the FWD and LWD (0.7 to 0.9 MPa). In addition, the resilient modulus values of the tested soils measured in the laboratory were found to be approximately twice those obtained by the GeoGauge. This suggested that the stress levels used during the laboratory testing may be much higher than those imposed by GeoGauge.

Petersen and Peterson (2006) reported the results of an intelligent compaction demonstration project in which the GeoGauge and the LWD were used to test the final lift of a 914-mm (3-ft) subcut consisting of a select granular borrow material. The material was compacted using a vibratory compaction roller outfitted with intelligent compaction (IC) technologies. The GeoGauge was conducted at 42 points along the project. Results showed a poor correlation between GeoGauge modulus and the IC roller measurements when the comparison was done on a point-by-point basis. This was attributed to the relatively shallow depth of influence of the GeoGauge and the soil's heterogeneity. However, a relatively good correlation was obtained between the GeoGauge and the LWD. In addition, the authors found that the GeoGauge was easy to use by a single individual, and it provided repeatable measurements when properly seated.

New Jersey Study

Maher et al. (2002) conducted field and laboratory investigations to evaluate the suitability of the GeoGauge for soil compaction control and dry density measurement. The field component included performing GeoGauge and nuclear density gauge tests at 400 points during placement and compaction of two embankments composed of Portland cement-stabilized dredge sediments. Approximately 50 points in the first embankment were used for the calibration. The laboratory investigation involved testing three types of subgrade soils and one subbase aggregate material compacted in a 55-gallon steel drum cut to a maximum height of 610 mm (24 in.). Field work results indicated that the GeoGauge could indeed be used to estimate the dry density of soils if proper calibration factors were determined for the tested soil. Laboratory results indicated that the GeoGauge had the potential to determine the resilient modulus of soils; however, calibration to the different applied stress conditions was needed for validation.

New Mexico Study

Lenke et al. (2003) evaluated the use of the GeoGauge for compaction control of pavement materials. Their results indicated that the GeoGauge was able to detect the increase in soil stiffness with compaction by roller passes (Figure 54). However, the authors found that their attempts to determine a field target value for the GeoGauge in the laboratory using modified Proctor molds were not successful because the GeoGauge annular foot size is comparable with that of the mold. Lenke et al. (2003) indicated that without being able to develop a laboratory-determined target value for stiffness in the field, compaction control specifications that use GeoGauge should include careful control of the moisture content of compacted soils. In addition, they suggested the use of test strips to determine the GeoGauge compaction control parameters for a given soil.

Texas Study

Chen et al. (1999) used the GeoGauge, traditional FWD, dirt seismic pavement analyzer (D-SPA), and Olson spectral analysis of surface waves (SASW) to measure the stiffness of base course materials at eight different locations. Results indicated that the modulus measured with the FWD was higher than that measured with the GeoGauge. The authors suggested a general relationship between GeoGauge stiffness and the FWD back-calculated modulus as follows:

$$M_{FWD} = 37.65 H_{SG} - 261.96 \qquad (27)$$

where

M_{FWD} = back-calculated FWD modulus (MPa), and
H_{SG} = GeoGauge stiffness reading (MN/m).

LIGHT WEIGHT DEFLECTOMETER

The light weight deflectometer (LWD) is a portable falling weight deflectometer that consists of a falling mass and a displacement-measuring sensor attached at the center

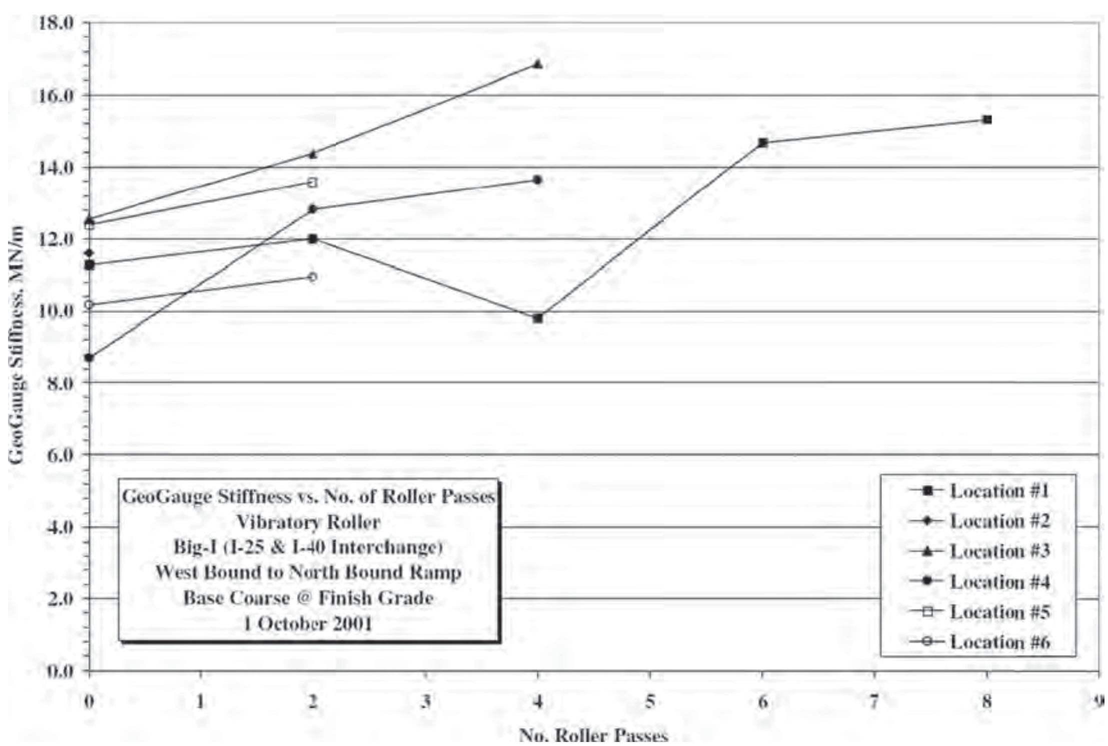


FIGURE 54 Variation of the GeoGauge with roller passes (Lenke et al. 2003).

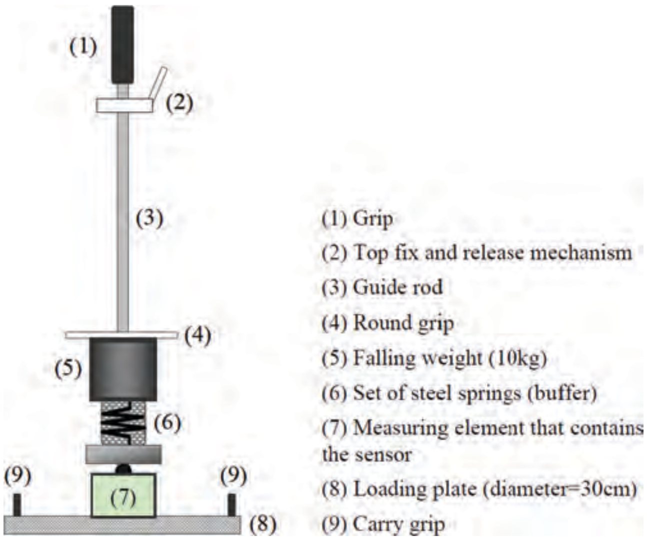


FIGURE 55 Schematic drawing of LWD showing various components of the equipment (Kim et al. 2010).

of a loading plate. Figure 55 provides a schematic representation of the LWD, including its various components. There are several types of LWD on the market that have been evaluated in previous studies, including the Dynatest, Keros, German dynamic plate (GDP), Prima 100, Transport Research Laboratory (prototype) Foundation Tester (TFT), and Zorn. These devices exhibit many similarities in their mechanics of operation, although differences in design and mode of operation lead to variations in the measured results. Table 12 provides a comparison of the different LWD devices. Figures 56 and 57 present photographs of two types of LWD.

Principles of Operation

The LWD test is performed by releasing the falling weight from a standard height onto the loading plate using the top fix-and-release mechanism. An impulse load is imposed on the compacted material through the plate. The resulting central deflection of the loading plate is obtained either by integrating the velocity measurements taken from a velocity transducer or by double-integrating the acceleration data taken from an accelerometer. The expression used to calculate LWD modulus is similar to the one used to calculate the surface modulus of a layered system having a homogeneous properties, assuming constant loading on an elastic half space (Boussinesq elastic half space). This expression is shown in Eq. 28. Currently, ASTM E2583-07 is the standard method for conducting LWD tests. Some state DOTs (Indiana and Minnesota) have developed standard test protocols for the LWD.

$$E_{LWD} = \frac{(1-v^2)\sigma \times R \times A}{\delta_c} \tag{28}$$

TABLE 12
COMPARISON BETWEEN DIFFERENT LWD DEVICES

Device	Plate Diameter (mm)	Plate Thickness (mm)	Falling Weight (kg)	Maximum Applied Force (kN)	Load Cell	Total Load Pulse (ms)	Type of Buffers	Deflection Transducer		
								Type	Location	Measuring Range (mm)
Zorn	100, 150, 200, 300	124, 45, 28, 20	10, 15	7.07	No	18 ± 2	Steel spring	Accelerometer	Plate	0.2–30 (±0.02)
Keros	150, 200, 300	20	10, 15, 20	15	Yes	15–30	Rubber (conical shape)	Velocity	Ground	0–2.2 (±0.002)
Dynatest 3031	100, 150, 200, 300	20	10, 15, 20	15	Yes	15–30	Rubber (flat)	Velocity	Ground	0–2.2 (±0.002)
Prima	100, 200, 300	20	10, 20	15	Yes	15–20	Rubber (conical shape)	Velocity	Ground	0–2.2 (±0.002)
Loadman	110, 132, 200, 300	Unknown	10	17.6	No	25–30	Rubber	Accelerometer	Plate	Unknown
ELE	300	Unknown	10	Unknown	No	Unknown	Unknown	Velocity	Plate	Unknown
TFT	200, 300	Unknown	10	8.5	Yes	15–25	Rubber	Velocity	Ground	Unknown
CSM	200, 300	Unknown	10	8.8	Yes	15–20	Urethane	Velocity	Plate	Unknown

Notes: Zorn Light Drop Weight Tester ZFG2000 by Gerhard Zorn, Germany; Keros Portable FWD and Dynatest 3031 by Dynatest, Denmark; Prima 100 Light Weight Deflectometer by Carl Bro Pavement Consultants, Denmark; Loadman by AL-Engineering, Oy, Finland; Light Drop Weight Tester by ELE; TRL Foundation Tester (TFT) is a working prototype at the Transport Research Laboratory, United Kingdom; Colorado School of Mines (CSM) LWD device.
After White et al. (2009b).

FIGURE 56 Dynatest 3031 LWD (Dynatest 2013).

FIGURE 57 Zorn LWD (Zorn Instruments 2013).

where

- σ = the applied stress,
- R = the loading plate radius,
- v = Poisson's ratio (usually set in the range of 0.3 to 0.45 depending on test material type),
- δ_c = central peak deflection, and
- A = plate rigidity factor: default is 2 for a flexible plate, $\pi/2$ for a rigid plate.

Factors Influencing the LWD Modulus

A number of factors may influence the measured LWD modulus, including falling mass, drop height, plate size, plate contact stress, type and location of deflection transducer, usage of load transducer, loading rate, and buffer stiffness (Fleming 2001; White et al. 2007a). Contrasting information is available in the literature on the effect of plate size on measured LWD modulus. Fleming et al. (2007) studied the effect of the plate size and drop weight on stiffness. Their results indicated that for the 15- or 20-kg drop mass, the modulus did not change significantly with different plate diameters. However, Chaddock and Brown (1995) demonstrated that using the TFT LWD with a 200-mm plate resulted in a modulus that was approximately 1.3 to 1.5 times greater than that with a 300-mm plate. Furthermore, based on field studies conducted using LWD, Deng-Fong et al. (2006) found that the LWD modulus measured using a 100-mm plate was about 1.5 times greater than that from a 300-mm plate. Lin et al. (2006) also concluded that the size of the loading plate was a significant factor affecting LWD modulus. They indicated that the use of an inappropriate loading plate could affect the measurements and the modulus calculation. Lin et al. (2006) also evaluated the effect of drop heights, concluding that different drop heights had very little effect on stiffness. To get consistent and comparable results, researchers have suggested using a LWD with the same mass, drop height, and plate size. Davich et al. (2006) recommend using a LWD with a mass of 10 kg, drop height of 500 mm, and plate diameter of 200 mm. They suggested that this combination resulted in the test volume extending to the bottom of a common lift.

Differences in type and location of sensors used in LWD devices can also lead to variations in measured LWD modulus. White et al. (2007a) compared the subgrade moduli measured using two LWD devices: the Zorn (Model ZFG 2000) and the Keros. The moduli measured with the Keros were found to be 1.75 to 2.2 times greater. The researchers attributed the differences in the measured modulus values to the different methods used to measure deflections in both devices. Although the Keros measures deflections on the ground with a geophone, the Zorn uses accelerometers that measure plate deflection, which is expected to measure larger deflections.

Some LWD devices (e.g., Zorn) assume a constant applied force based on calibration tests performed on a concrete surface, whereas others (e.g., Prima 100 and Keros) use a load cell to measure the actual applied load during the test. Previous studies have concluded that the assumption of constant applied force does not significantly affect the measured modulus when using the LWD to test relatively stiff compacted layers (Brandl et al. 2003; White et al. 2007a).

Some studies suggested that the spring stiffness of the buffer placed between the drop weight and the contact plate controls the loading rate and thus can affect the measured LWD modulus. Adam and Kopf (2002) found that the applied load pulse varied by about 30% with a change in rubber buffer temperature from 0°C to 30°C; it remained more constant, however, using a steel-spring buffer. This might explain why Germany has prohibited the use of rubber buffers (White et al. 2007a). Finally, Fleming (2000) found that a comparatively lower stiffness buffer provided more efficient results.

Repeatability

The reliability of LWD measurement is significantly influenced by its repeatability. Petersen et al. (2007) and Hossain and Apeagyei (2010) reported a relatively high COV, ranging from 22% to 77% for LWD-measured modulus when testing various types of unbound materials. Von Quintus et al. (2008) reported a COV between 13.9% and 77.3% for different types of LWD devices. Nazzal et al. (2007) evaluated the repeatability of the LWD by using the COV of five measurements taken at the same testing point. Figure 58 shows the COV with the corresponding average LWD elastic moduli measured in that study. The COV of the LWD measurements ranged from 2.1% to 28.1%. The general trend for the points in Figure 58 indicates that COV values decreased as LWD elastic moduli increased. Nazzal et al. (2007) indicated that during field testing it was difficult to conduct LWD tests on very weak subgrades because of uneven surfaces that caused tilting of the loading plate. Another reason for high COV values in weak subgrade soils was that those soils exhibited significant permanent deformation under the LWD test. Similar findings were reached by other researchers (e.g. Fleming 2000; George 2006; Fleming et al. 2007). Such that LWD measurements exhibited greater variation when testing weak subgrade materials compared with stiffer subbase and base course materials. Fleming et al. (2009) reported that the range of COV of LWD measurements varied between 25% and 60% for fine-grained subgrades owing to variation in moisture content. In addition, for granular capping (subbase) layers, the COV ranged from 10% to 40%, with higher values observed on very wet sites. For highly specified, well-graded, crushed aggregate base materials, the COV of LWD measurements typically was less than 15%.

Several factors affecting the variability in the moduli measured with the LWD have been reported in the literature. They include (1) the number of load drops, (2) the quality of the load and deflection curve, and (3) the level of contact between loading plate and tested layer surface (White et al. 2007a; Fleming et al. 2007; Ooi et al. 2010). Steinart et al. (2005) studied the influence of the number of load drops on the measured LWD modulus and found that the measurements from the first drop typically were smaller than those derived from subsequent ones, as shown in Figure 59. There-

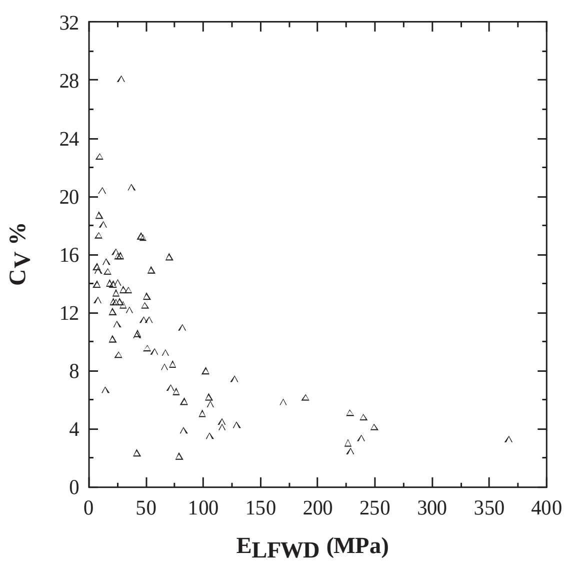


FIGURE 58 C_v variation with LWD modulus (Nazzal et al. 2007).

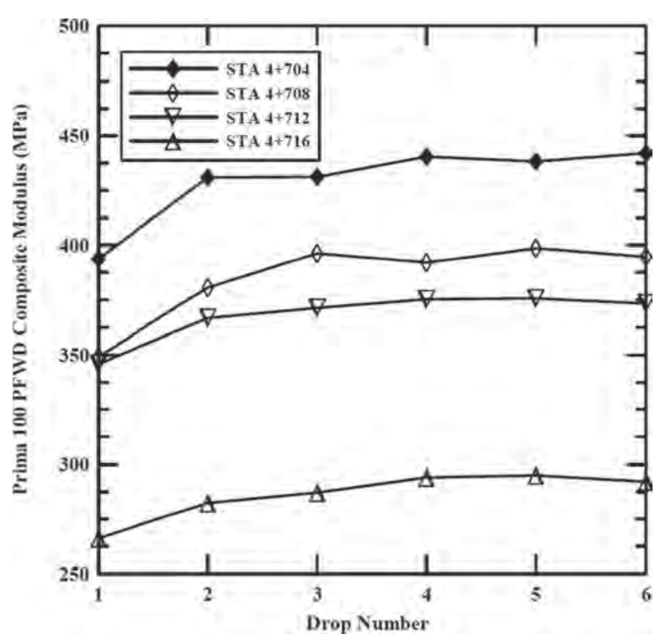


FIGURE 59 Effect of consecutive drops on composite modulus values (Steinart et al. 2005).

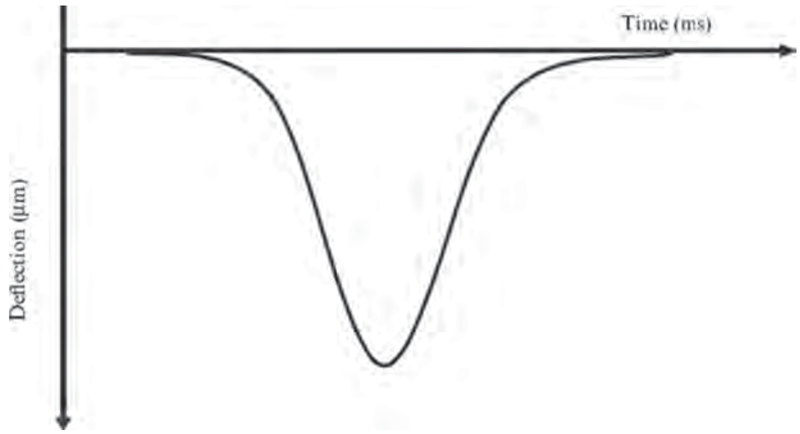


FIGURE 60 High-quality LWD reading (Fleming et al. 2007).

fore, they recommended that the first value be excluded in calculating the average LWD moduli value. Davich et al. (2006) suggested using three LWD seating drops followed by three drops at each test location to produce consistent LWD data. George (2006) recommended that two seating loads be applied at each station, followed by four or more load drops of 1,730 lb. In addition, George (2006) suggested that the LWD load-deflection history should be checked continuously for inconsistencies. Fleming et al. (2007) recommended that the deflection-time history of each drop be assessed to determine the quality of the LWD results. The authors demonstrated (Figure 60) that an ideal test should have no deflection at time zero, but then should increase to a peak, followed by a decrease. At the end of the pulse, no deflection should occur. The authors also illustrated a possible deflection-time (pulse) history other than the ideal one. For example, Figure 61 shows the deflection at the end of the pulse moving in the opposite direction instead of returning to zero. This typically happens when the instrument bounces off the ground upon impact. This type of deflection-time history is also possible if the tested material contains excess water. The effect of successive drops at the same spot on deformation-time histories is shown in Figure 62. Both the maximum and final deflections decrease as the number of repetitions increase. Figures 60 through 62 provide a guide for determining acceptable readings.

Previous studies have recommended that the LWD be conducted on a uniform surface to ensure optimal contact between the LWD loading plate and the tested material. For example, Lin et al. (2006) found that the repeatability of the LWD was very good only if there was an even contact surface. Higher variability was observed for uneven surfaces (e.g., coarse gravel). Remedies for uneven surfaces include using moist sand, removing as much as 102 mm (4 in.) of the compacted material before testing, and limiting testing to layers with a gradient of less than about 5%.

Influence Depth

Nazzal et al. (2007) evaluated the influence depth of the LWD by conducting laboratory tests inside two test boxes (1,824 × 912 × 912 mm). To clearly define the influence zone for the LWD, stiff soil was constructed on top of soft soil and vice versa. The results indicated that the influence depth for the LWD ranges between 270 and 280 mm (10.6 and

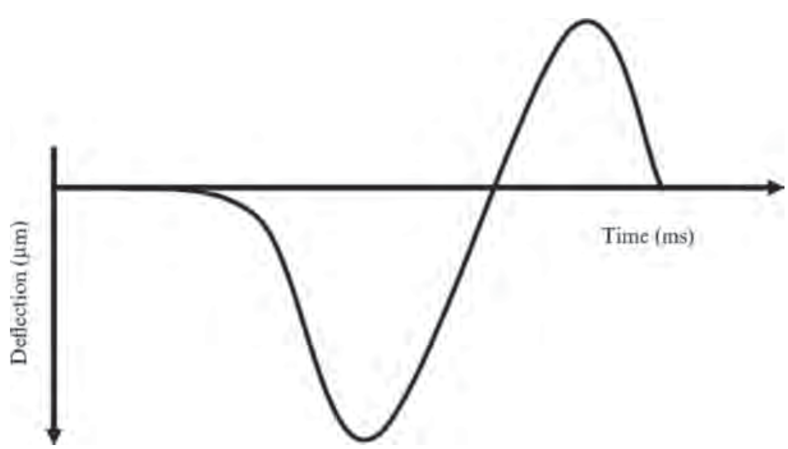


FIGURE 61 Example for high rebound LWD reading (Fleming et al. 2007).

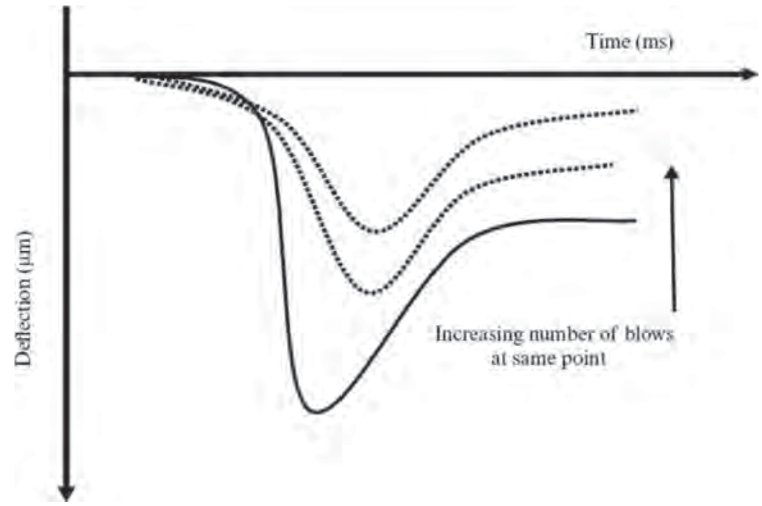


FIGURE 62 Effect of increasing number of blows on LWD reading (Fleming et al. 2007).

11 in.), which was about 1.5 times the diameter of the loading plate. Brandl et al. (2003) reached similar findings. However, Fleming et al. (2007) and Siekmeier et al. (2000) reported a lower influence zone equal to the diameter of the loading plate. Based on analysis of in situ strain data, Mooney et al. (2009) found that the LWD influence depth ranged from 0.9 to 1.1 of the loading plate diameter depending on soil type.

Advantages and Limitations

Several advantages of the LWD were reported in the literature. The setup and test times for LWD are relatively short (Sebesta et al. 2006; Siekmeier et al. 2009). In addition, the LWD measures the modulus value of tested pavement materials, which can be directly used as input in the pavement design. With additional sensors, the LWD can distinguish stiffness values between pavement layers. Siekmeier et al. (2009) indicated that the LWD could accurately test more material types, such as unbound materials with large aggregates, than could the standard density-based approach. In addition, LWD testing is safer because the field inspector is able to remain standing and visible during most of the testing process (Davich et al. 2006).

The LWD's main limitation is its high variability. Hossain and Apeagyei (2010) reported high variability in measured LWD modulus for the same material tested with different LWD devices. Other studies reported poor repeatability when testing weak cohesive materials or layers with uneven surfaces (Nazzal 2003). Petersen et al. (2007) indicated that the LWD tended to move during testing, which affected the reliability of the test results. They recommended using a two-buffer configuration when testing using the LWD to increase the dampening of the impact load and to limit the movement of the machine during testing. To provide uniform loading and reduce machine movement, the authors recommended that a smooth and level test area be selected so that a good contact exists between the loading plate and the test surface.

Another problem, reported during an interview with the Minnesota DOT, is the difficulty encountered when using the LWD in large projects as a result of its relatively heavy weight. Hossain and Apeagyei (2010) also found that the effect of the moisture content on the LWD-measured modulus was much more significant compared with the moduli measured with other in situ test devices.

Finally, there are some concerns about the effectiveness of the LWD in testing layered systems. This concern is mainly attributable to the possibility that the LWD's zone of influence may extend beyond the thickness of the tested layer. Sebesta et al. (2006) recommended using a three-sensor system that reduces the setup time and measures modulus values of the multiple pavement layers. Lin et al. (2006) indicated that with the three sensors the moduli for three layers can be computed based on the measured deflections and distances to the load. In this case, the modulus computed from deflections further away from the load represents the deeper layers. However, Steinart et al. (2005) suggested that until a program is developed to incorporate the deflections from all three sensors simultaneously into a back-calculation routine, the additional sensors will not be useful.

Synthesis of Past Research Studies

Kansas Study

Petersen et al. (2007) investigated the use of the LWD as a tool for compaction quality control of embankment soil. LWD and FWD tests were conducted on different types of soils in nine Kansas DOT embankment projects. Density and moisture measurements were taken at selected test locations, and resilient modulus tests were conducted on soil samples

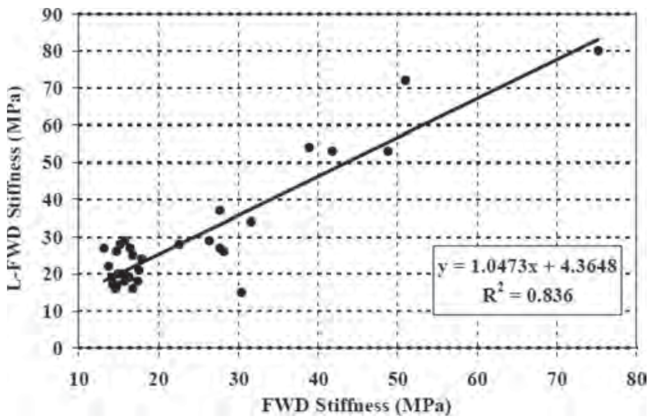


FIGURE 63 Relationship between LWD and FWD moduli.

obtained during field testing and prepared in the laboratory at varying density and moisture contents. Figure 63 presents the results of the LWD and FWD tests conducted in that study. It is clear that the LWD modulus was very close to that measured by the standard FWD. Petersen et al. (2007) indicated that they failed to develop a model that relates the LWD modulus to the laboratory-determined resilient modulus. They attributed that to the differences in the state of stress as well as the dry density and moisture content conditions of soil tested in the field as compared with that in laboratory. The authors also suggested that the moisture content of the soil in the field may vary within the testing area owing to desiccation of the surface layer that occurs between the end of compaction and the onset of stiffness testing. Finally, they found that the high degree of spatial variability obtained for the LWD moduli prevented the development of a quality control procedure based on a control test strip.

Louisiana Studies

Abu-Farsakh et al. (2004) conducted field and laboratory testing programs to evaluate the effectiveness of the LWD in measuring the stiffness properties of different types of geomaterial for application in the QC/QA procedures during and after the construction of pavement layers and embankments. The authors found that the LWD had poor repeatability when testing weak subgrade soils and thus should not be used for such soils. In addition, they indicated that for pavement layers less than 305 mm (12 in.) thick, the LWD measurement might not reflect the true modulus value of the tested layer but rather a composite modulus for the multiple layers below. In this case, the authors recommended that the LWD modulus be back-calculated. Based on the results of regression analysis conducted on the data obtained in this study, Abu-Farsakh et al. (2004) found the following correlation between LWD and FWD back-calculated resilient moduli, M_{FWD}, PLT initial and reloaded modulus, and CBR:

$$M_{FWD} = 0.97\,(E_{LFWD})$$
$$\text{for } 12.5 \text{ MPa} < E_{LFWD} < 865 \text{ MPa} \qquad (R^2 = 0.94) \quad (29)$$

$$E_{PLT(i)} = 22 + 0.7\,(E_{LFWD})$$
$$\text{for } 12.5 \text{ MPa} < E_{LFWD} < 865 \text{ MPa} \qquad (R^2 = 0.92) \quad (30)$$

$$E_{PLT(R2)} = 20.9 + 0.69\,(E_{LFWD})$$
$$\text{for } 12.5 \text{ MPa} < E_{LFWD} < 865 \text{ MPa} \qquad (R^2 = 0.94) \quad (31)$$

$$\text{CBR} = -14.0 + 0.66\,(E_{LFWD})$$
$$\text{for } 12.5 \text{ MPa} < E_{LFWD} < 174.5 \text{ MPa} \qquad (R^2 = 0.83) \quad (32)$$

In another study, Mohammad et al. (2009) reported the model in Eq. 33 directly predicts the laboratory measured M_r from the LWD modulus. To enhance the prediction, the moisture content was included as a variable in the M_r regression model (Eq. 34). It is noted that the moisture content was chosen based on stepwise selection analysis that included various physical properties of the tested unbound materials.

$$M_r = 27.75 \times E_{LFWD}^{0.18} \qquad (R^2 = 0.54) \qquad (33)$$

$$M_r = 11.23 + 12.64(E_{LFWD}^{0.2}) + 242.32\left(\frac{1}{w}\right) \qquad (R^2 = 0.7) \qquad (34)$$

where

M_r = resilient modulus (MPa),
E_{LFWD} = modulus from LWD test (MPa), and
w = moisture content (%).

Mississippi Study

George (2006) reported the results of a study to investigate the effectiveness of the LWD in testing subgrade soil. LWD, standard FWD, and nuclear density gauge measurements were obtained at 13 as-built subgrade sections reflecting typical subgrade soils in Mississippi. Resilient modulus and other routine laboratory tests were conducted on soil samples collected from the sections. The author concluded that the LWD is a viable device for characterizing subgrade soil provided that the imposed stress level is within the linear elastic range of the tested soil. He proposed Eq. 35 to relate the LWD modulus to the in-place density and moisture content of the tested soil. In addition, he developed a model (Eq. 36) to predict the laboratory-determined resilient modulus of soil compacted at 95% relative compaction from E_{LWD} and dry density, moisture content, and soil index properties. Because of concerns raised by the Mississippi DOT engineers about the availability of soil index properties, namely PI and P_{200}, George (2006) developed another model without the PI/P_{200} term (Eq. 37). Finally, based on the field test results, the author found the correlation shown in Eq. 38 between LWD and FWD back-calculated moduli.

$$E_{LWD} = 109{,}988\ \left(\mathrm{D}_{(f/o)}\right)^{5.544} \left(w_{(f)}\right)^{-0.594} \qquad (R^2 = 0.83) \qquad (35)$$

$$\frac{E_{LWD}}{M_{R95}} = -2.30 + 3.860 D_{\left(f/o\right)} - 0.316 M_{\left(f/o\right)} - 0.635 \frac{PI}{P_{200}} \quad \left(R^2 = 0.83\right) \tag{36}$$

$$\frac{E_{LWD}}{M_{R95}} = -3.907 + 5.435 D_{\left(f/95\right)} - 0.370 M_{\left(f/o\right)} \quad \left(R^2 = 0.70\right) \tag{37}$$

$$M_{FWD} = 1.09 E_{LWD}, \quad 2240 \text{ psi} < E_{LWD} < 30740 \text{ psi} \quad \left(R^2 = 0.64\right) \tag{38}$$

where

E_{LWD} = measured LWD elastic modulus (psi),
$D_{(f/o)}$ = ratio of field unit weight to unit weight at optimum moisture,
$w_{(f)}$ = field moisture (%),
$D_{(f/95)}$ = ratio of field unit weight to unit weight at 95% compaction, and
$M_{(f/o)}$ = ratio of field moisture to optimum moisture.

Maine Study

Steinart et al. (2005) studied the effectiveness of the LWD as a tool for compaction control of subgrade and base materials using field and laboratory tests. The laboratory component of the study included compacting five types of base and subbase aggregates in a $1.8 \times 1.8 \times 0.9$ m ($6 \times 6 \times 3$ ft) deep test container using 152 mm (6 in.) lifts. The compacted materials were tested using various in situ test devices: the LWD, CH, DCP, and NDG. The field component included testing two subgrade soils, sand, two base aggregates, and one reclaimed stabilized base product using the LWD and NDG. In general, the laboratory and field test results showed that the LWD modulus generally increased with increases in the percent compaction. However, whereas the laboratory tests showed poor correlation between LWD modulus and percent compaction, the field test sites where granular base materials were tested yielded a good correlation. In addition, the results of multivariable regression analyses conducted on the field test data to predict the LWD modulus as a function of percent compaction and moisture content yielded the model shown in Eq. 39. The obtained model had a relatively high R^2, which suggested that a strong correlation existed between the LWD modulus and percent compaction and moisture content. However, the authors indicated that the moisture content values for the tested field sites were all on the dry side of optimum, which may limit the significance of this conclusion.

$$E_{LWD} = -411.26 + 5.454(PC) - 2.757(RWC) \quad \left(R^2 = 0.823\right) \tag{39}$$

where

E_{LWD} = LWD composite modulus,
PC = percent compaction, and
RWC = relative water content.

Minnesota Studies

An earlier work done by Minnesota DOT was a study reported by Siekmeier et al. (2000) in which Loadman LWD, DCP, GeoGauge, and traditional FWD were used to test granular base materials at several construction sites in Minnesota. For each test location, five LWD tests were performed and the last three measurements were averaged. Laboratory resilient modulus tests also were conducted on cores obtained during field testing. Figure 64 presents the results by the different in situ devices. The LWD modulus had a trend similar to that of the FWD back-calculated modulus but a different magnitude. The authors attributed the observed differences to the variation in the stress conditions imposed by each of these two devices. As shown in Figure 64, there was little agreement between relative compaction and measured in situ moduli. The authors explained that it was not realistic to know the Proctor maximum density for every soil type found at a construction site. They suggested that compaction tests could be compared with in situ modulus tests only when the material is uniform with respect to a single maximum Proctor density (Siekmeier et al. 2000).

Hoffmann et al. (2003) indicated that prediction of the LWD modulus based on load and peak deflections could result in inaccurate modulus values; therefore, to improve prediction, they proposed a spectral-based procedure to analyze LWD data.

Davich et al. (2006) presented the results of a study conducted by the Minnesota DOT's Office of Materials to provide data needed for developing LWD compaction control specifications. LWD and moisture meter tests were conducted on

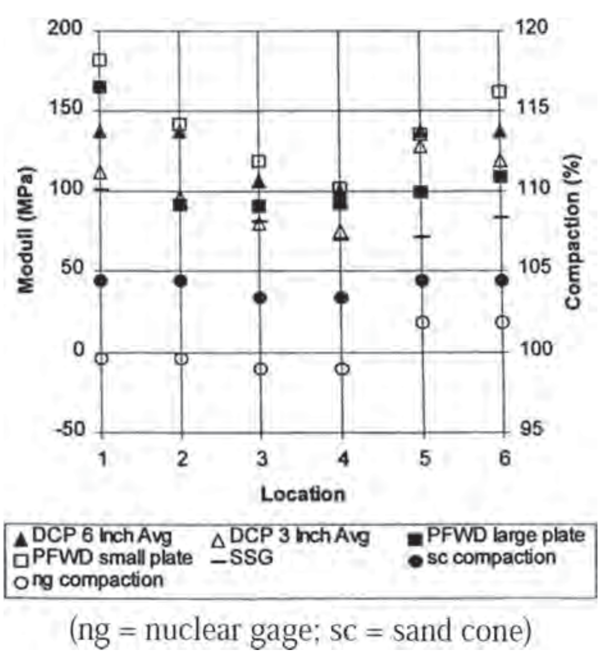


FIGURE 64 Moduli compared with location for granular base material (Siekmeier et al. 2000).

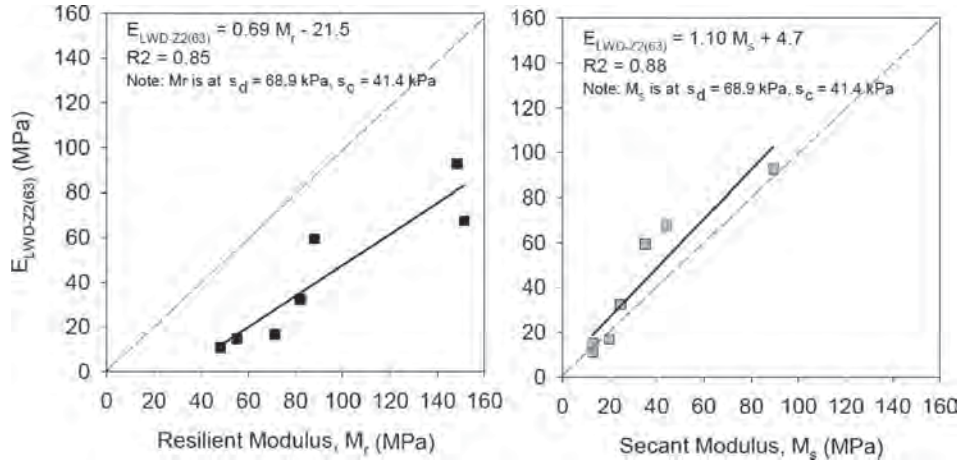


FIGURE 65 Relationship between the 200-mm Zorn E_{LWD} and laboratory M_r and M_s (White et al. 2007).

samples of three types of granular materials, which were compacted inside an open-topped, steel cylinder (half a 55-gallon steel drum) using a procedure similar to that of the standard Proctor test. The results of this study showed that the LWD provided a level of accuracy similar to that of DCP testing. However, the LWD has an advantage over the DCP because it directly measures quantities that characterize the pavement layers' mechanical response during traffic loading, such as force and displacement. Furthermore, it is nondestructive and requires less inspector effort than does DCP testing. The authors recommended that LWD plate size and falling mass drop height should be standardized to obtain consistent and reliable data.

As part of the Minnesota DOT's efforts to evaluate and implement intelligent compaction technology and other in situ tests into earthwork construction practice, White et al. (2007a) presented the results of a field study in which two types of LWD devices (Zorn and Keros) with different plate diameters were used to test subgrade soils and base course layers at construction sites in Minnesota. In addition, this research included conducting resilient modulus tests on Shelby tube samples obtained from the tested subgrade soils at the locations of LWD tests. The results of this study showed that the LWD modulus measured using the Keros device was on average 1.9 to 2.2 times greater than that measured with the Zorn. The authors attributed the differences in measured modulus values between the two devices to the Zorn measuring approximately 1.5 times greater deflection than did the Keros for the same plate diameter. The authors also compared Zorn and Keros LWD moduli with the M_r values and the secant modulus (M_s) based on the permanent strain and resilient strain data obtained from the resilient modulus test. As shown in Figures 65 and 66, strong linear correlations with high R^2 values were found between Zorn and Keros LWD moduli and each of the M_r and M_s.

White et al. (2007a) compiled ranges of LWD modulus values for various types of cohesive and granular soils under different compaction conditions, which were obtained from

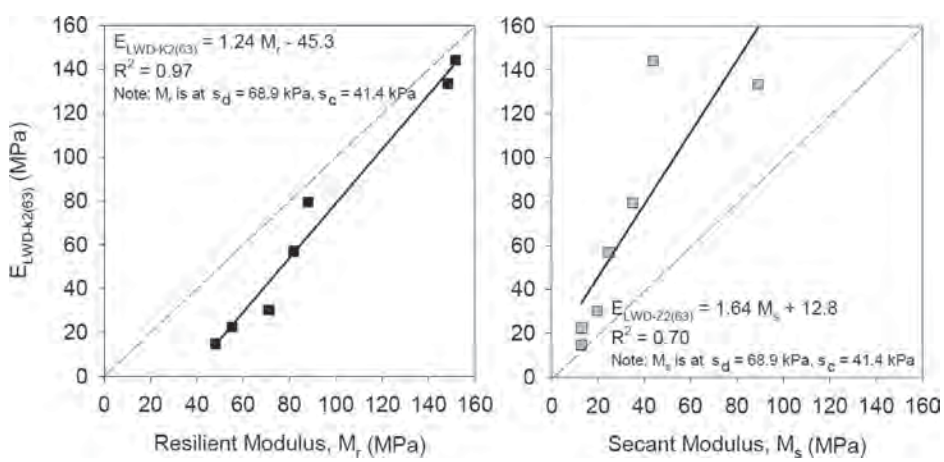


FIGURE 66 Relationship between the 200-mm Keros E_{LWD} and laboratory M_r and M_s (White et al. 2007).

TABLE 13
RANGE OF LWD MODULUS VALUES PUBLISHED IN LITERATURE

Soil Name	USCS	Loose Lift Thickness (mm)	Moisture Deviation (% Range)	Relative Compaction (% Range)	LWD Modulus (MPa) (COV)
Cohesive Soils					
Silt	ML	300	-2.5 to -3.0	94–98	47 (-)
		200	-1.5 to -4.0	96–102	127 (71)
Lean clay with sand	CL	250	+1.0 to +3.5	87–95	49 (58)
		250	-4.0 to + 0.5	86–93	59 (62)
Sandy lean clay	CL	250	–6.0 to -5.0	84–88	45 (46)
		250	-3.0 to -1.5	85–90	65 (58)
Cohesionless Soils					
Well-graded sand with silt	SW-SM	360	-5.0 to -3.5	96–99	24 (27)
		250	-6.0 to -4.5	96–100	28 (22)
Silty gravel with sand	GM	350	-0.5 to 0.0	88–90	33 (15)
Silty sand with gravel	SM	280	-6.0 to -5.5	95–100	33 (8)
Poorly graded gravel	GP	300	–	95–103	41 (17)
Silty sand	SM	360	-1.5 to -1.0	91–95	19 (24)
Clayey gravel with sand	GC	340	-2.0 to -1.5	86–92	37 (12)
Well-graded sand with silt	SW-SM	200	-0.5 to +2.0	99–102	8 (5)
		200	-5.0 to -4.5	99–101	33 (21)
		200	-2.5 to -1.5	97–102	27 (33)

Source: White et al. (2007a).
USCS, Unified Soil Classification System.

field testing programs conducted in previous studies. Table 13 presents the mean and COV of the LWD modulus corresponding to the range of moisture deviation from optimum and percent-relative compaction based on the standard Proctor test. It is clear that the LWD modulus values reported for cohesive soils have higher COV values (ranging from 46% to 71%) compared with those for granular soils (ranging from 5% to 27%). The authors indicated that cohesive soils showed more moisture sensitivity than did granular soils. This was apparent from the relatively high COV of LWD moduli for the sandy lean clay soil within a moisture deviation range of 1% and a relative compaction increase of 4%.

In a different study, White et al. (2009a) investigated the relationships between the LWD moduli and intelligent compaction rollers and proof rolling rutting measurements. Two roller-integrated compaction monitoring technologies, namely the compaction meter value (CMV) and the machine drive power (MDP), as well as three types of LWD devices (Dynatest, Zorn, and Keros), were evaluated in the study. Figure 67 presents the relationships between CMV and LWD measurements obtained on granular subgrade and base materials. It is clear that a strong correlation exists between the CMV and LWD deflection and modulus values. However, LWD measurements had better correlation with CMVs when tests were performed in a carefully excavated trench approximately 100 to 150 mm deep (US-10 project). Although the CMV is correlated with a linear regression relationship with LWD modulus values, it also is correlated with a nonlinear power relationship with LWD deflection values. Based on this, the authors suggested that 90% to 120% of the target values criteria used by the Minnesota DOT need to be reviewed for implementing LWD deflection values owing to the nonlinear nature of the relationship with CMV. White et al. (2009a) indicated that relationships between MDP with LWD measurements obtained from nongranular materials showed positive correlations, although with varying degrees of uncertainty (i.e., R^2 values varied from about 0.3 to 0.8). Nonetheless, the relationships between MDP improve when moisture content is included in the regression analysis. The authors found good correlations between the test rolling rut depth and LWD measurements, which are presented in Figure 68. They suggested that the scatter observed in the relationships was partly attributed to soil variability and the differences in influence depth between heavy test rollers (0.6 to 1.2 m) and LWD tests.

Virginia Study

Hossain and Apeagyei (2010) conducted a study to investigate the ability of the LWD to measure in situ modulus of base course materials and subgrade and assess their degree of compaction. The LWD, the GeoGauge, and the DCP were used to test the base course layer and the subgrade in seven pavement sections in five Virginia counties. The authors found that the modulus values measured by the three devices

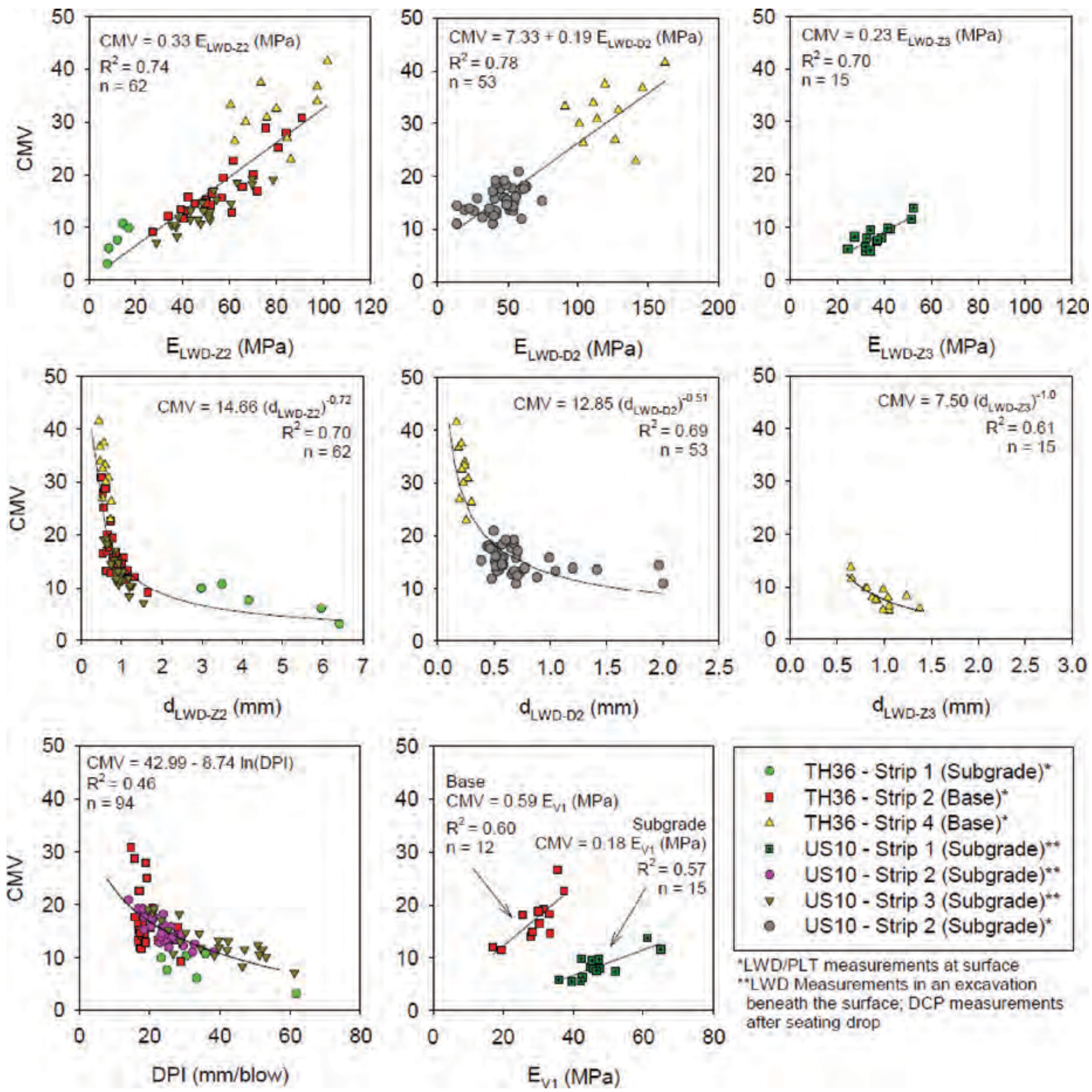


FIGURE 67 Correlation between CMV and LWD measurements obtained from field projects with granular materials (White et al. 2009a).

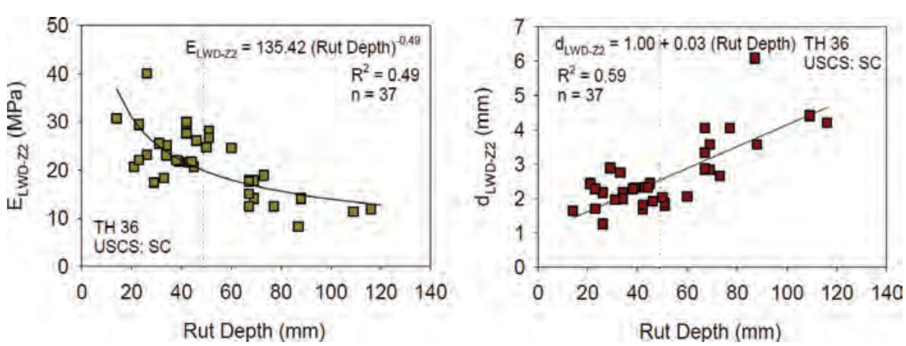


FIGURE 68 Relationship between rut depth and LWD measurements from TH-36 project (White et al. 2009a).

showed a high spatial variability. In addition, no good correlations were found between the LWD moduli and measurement of either the GeoGauge or the DCP. Results also showed that the effect of dry density on the measurements of the three devices was not significant. However, the moisture content showed a significant influence on the three in situ test device measurements, especially the LWD. Based on the obtained results, the authors suggested that the LWD not be used in the quality control of construction until further research could be conducted to determine the causes of the high spatial variability and the influence of moisture on the LWD modulus.

International Studies

Fleming et al. (1988) demonstrated a correlative ratio between the deformation moduli of the German dynamic plate (GDP), LWD, and the FWD of about 0.5. However, Fleming (1998, 2001) reported that his extensive field-stiffness measurements on construction sites showed a relatively consistent correlation of 0.6 between the stiffness moduli of the GDP and FWD. In another study, Fleming (2000) conducted field tests to correlate the moduli of three main types of LWD, namely the TFT, GDP, and Prima 100, with that of the FWD. The results showed that although the correlation coefficient between the FWD and Prima 100 moduli (Eq. 40) was close to one, it varied with the other LWD types, as shown in Eq. 41 and Eq. 42.

$$M_{FWD} = 1.031\, E_{\text{Prima100}} \tag{40}$$

$$M_{FWD} = 1.05 \text{ to } 2.22\, E_{GDP} \tag{41}$$

$$M_{FWD} = 0.76 \text{ to } 1.32\, E_{TFT} \tag{42}$$

Fleming et al. (2009) presented a review of LWD use in compaction control of pavement layers and subgrade soil in Europe. They found that the LWD has been increasingly used to test various types of materials before and during construction of major and minor roadways in the United Kingdom. In addition, it was included in the United Kingdom road foundation design and construction specifications. The authors indicated that the correlation coefficient between the LWD and FWD moduli was often reported as approximately one, but appeared to be variable and perhaps site dependent.

Kamiura et al. (2000) studied the relationship between the LWD and the plate load test measurements for subgrade materials, which contained volcanic soil, silty sand, and mechanically stabilized crushed stone. Based on the results of tests conducted in this study, the authors found the correlation in Eq. 43 and indicated that this correlation was affected by grain size of the tested material.

$$\text{Log}\left(\frac{k_{LWD}}{k_{30}}\right) = 0.0031 \log\left(k_{LWD}\right) + 1.12 \tag{43}$$

where

k_{LWD} = the ratio of stress on loading plate of the LWD to the measured deflection at this stress, and
k_{30} = the ratio of stress on the plate with a diameter of 300 mm for a PLT to the measured deflection at this stress.

Pidwerbesky (1997) evaluated the use of FWD and Loadman LWD to predict the performance of unbound granular base course layers and examine the relationship between these devices. A simulated loading and vehicle emulator (SLAVE) was used to load a pavement structure consisting of 90 mm (3.5 in.) of asphalt concrete layer and 200 mm (7.9 in.) of a crushed-rock base course layer on top of a silty clay subgrade with a CBR of 12% for approximately 1 year. After loading was completed, trenches were cut through the asphalt layer to test the base course and subgrade soil. The authors found that the Loadman LWD was not capable of differentiating the moduli of various layers within a multilayered pavement system, but it could provide an indication of the modulus of the tested layer. Based on regression analysis conducted on collected data, a relatively good correlation was obtained between Loadman LWD and FWD moduli, as shown in Figure 69.

DYNAMIC CONE PENETROMETER

The dynamic cone penetrometer (DCP) was initially developed in South Africa for in situ evaluation of pavement (Kleyn 1975). Since then, the DCP has been used the United States, the United Kingdom, Australia, and New Zealand for site characterization of pavement layers and subgrades. The standard DCP device consists of an upper fixed 575-mm travel rod with an 8-kg falling weight, a lower rod containing an anvil, and a replaceable cone with an apex angle of

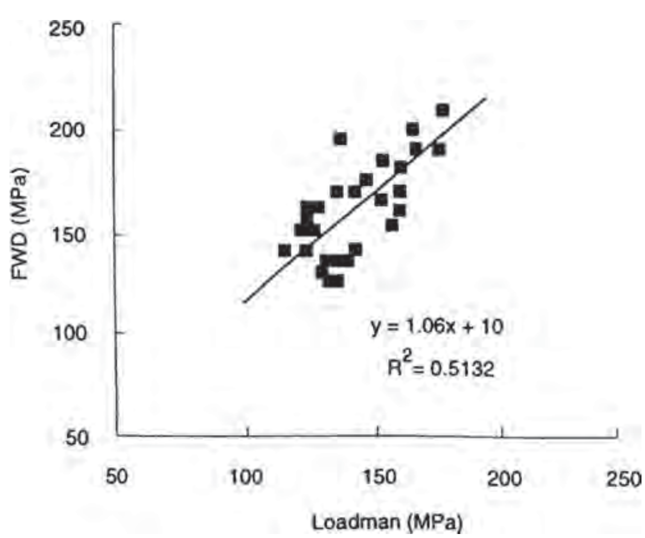


FIGURE 69 Correlation between Loadman LWD and FWD moduli (Pidwerbesky 1997).

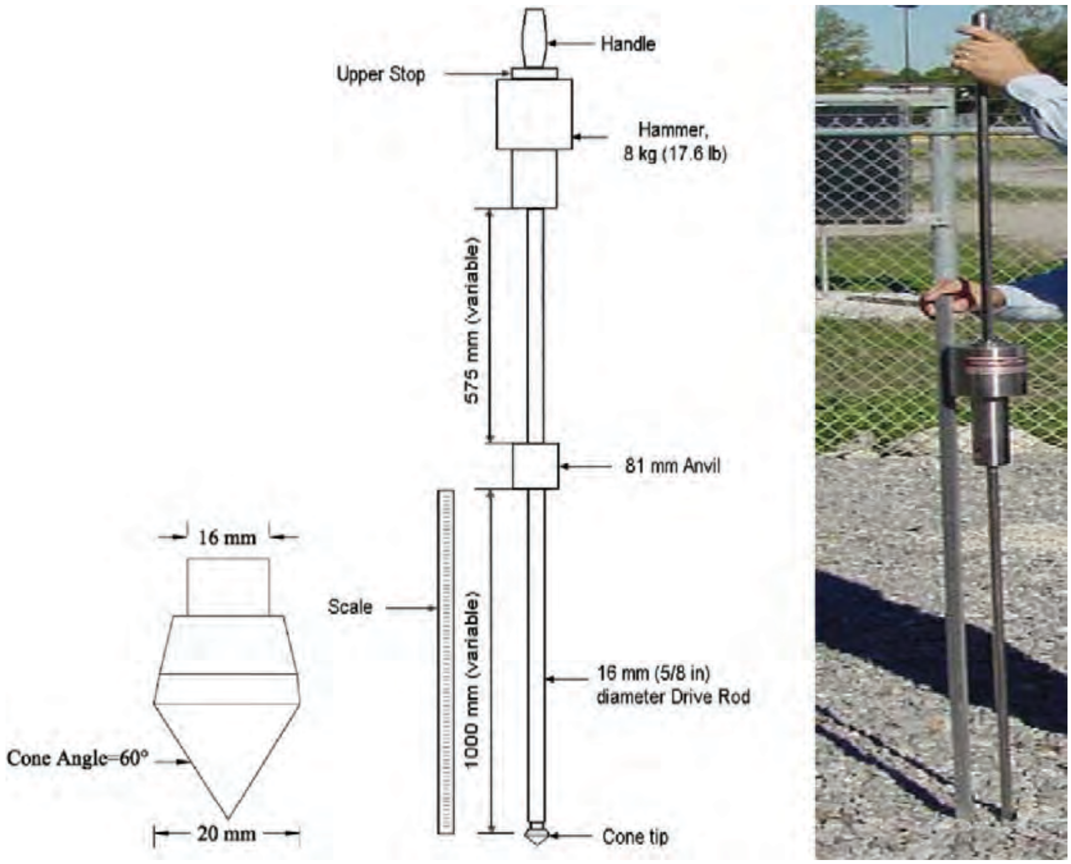


FIGURE 70 Dynamic cone penetrometer (DCP).

60° and a diameter of 20 mm, as shown in Figure 70. The DCP test is conducted according to ASTM D6951 or ASTM D7380, which involves dropping the weight from a 575-mm height and recording the number of blows versus depth. The penetration rate or PR, sometimes referred as the DCP ratio or penetration index (PI), is then calculated. The DCP ratio is defined by the slope of the curve relating to the number of blows to the depth of penetration (in mm/blow) at given linear depth segments. This device costs about $1,500.

Repeatability

Dai and Kremer (2006) reported DCP test results as a function of test location. At each location, they performed two DCP tests and found that, at some locations, the two test results were very close. Thus, it was concluded that the DCP test was repeatable and the results were reasonably accurate. A good repeatability was also reported by Petersen and Peterson (2006). However, Von Quintus et al. (2008) reported a COV of 2.9% to 27.4% for the DCP test results on 10 types of soil at seven different pavement sections. Similarly, Hossain (2010) found relatively higher values of COV (13% to 68%) for DCP measurements. Larsen et al. (2008) indicated that the high COV for DCP measurements was because the readings are influenced by slight variations in moisture and density. Siekmeier et al. (2009) also reported a significant influence of moisture content on the variability of the DCP measurements.

Influence Depth

The DCP can be used to evaluate compaction of underlying lifts to depths as great as 1.2 m (4 ft) (Mooney et al. 2008).

Advantages and Limitations

The DCP is simple, durable, economical, and requires minimum training and maintenance; it allows for easy access to sites and provides continuous measurements of the in situ strength of pavement sections and the underlying subgrade layers without the need for digging the existing pavement, as is required with other destructive tests (Chen et al. 2001). In addition, the DCP has a standard specification for testing, ASTM D6951, and requires no prior calibration. The DCP test is designed to estimate the structural capacity of pavement layers and embankments; it also has the ability to verify both the level and uniformity of compaction, which makes it an excellent tool for quality control of pavement construction. In addition, it can be used to determine the tested layers' thicknesses to a depth of 1.2 m (4 ft) (Chen et al. 2001). The DCP test has strong correlation with many strength and stiffness properties of various types of unbound materials, such as the CBR, shear strength, resilient modulus, and elastic modulus.

Despite its advantages, the DCP has limitations that have been reported in past studies. DCP testing should be limited to materials with a maximum particle size smaller than 51 mm (2 in.) because large aggregate particles may cause the device to tilt, affecting the accuracy of the test results (ASTM D6951). In addition, large particles may cause a significant increase in the DCP penetration rate that is not representative of the actual increase in density or strength (Rathje et al. 2006). DCP testing generally requires two people. Currently, the DCP does not have moisture measurement, GPS, or data storage capabilities. Farrag et al. (2005) indicated that the DCP needed to include a drop handle so that during testing the upper drop-height-stop does not become loose and slide down, decreasing the drop height. In addition, they recommended a confinement plate be used in granular materials to confine the top 2 to 3 in. for better lift measurement. Finally, some studies suggested that the DCP cannot be used in soft clay soils because it may actually advance under its own weight in such soils (Rathje et al. 2006).

Synthesis of Past Research Studies

Texas Study

Chen et al. (1999) conducted DCP and FWD tests on different subgrade and base materials in more than six districts in Texas. The subgrade resilient modulus was back-calculated from FWD data using EVERCALC. Based on the results, the correlation in Eq. 44 was developed between FWD back-calculated moduli and the DCP penetration rate. In a later study, Chen et al. (2007) developed a new correlation, presented in Eq. 45, based on tests conducted on subgrade and base soils in Texas.

$$M_{FWD} = 78.05 \times DPI^{-0.67} \tag{44}$$

$$M_{FWD} = 338\,(DPI)^{-0.39}$$
$$\left(\text{for } 10 \text{ mm/blow} < DPI < 60 \text{ mm/blow}\right) \tag{45}$$

where

M_{FWD} = FWD back-calculated moduli (MPa), and
DPI = DCP index or DCP penetration rate (mm/blow).

Jayawickrama et al. (2000) evaluated DCP use for compaction control of granular backfill materials for buried structures. Three types of granular backfill materials—concrete gravel, pea gravel, and 50-50 blend (50% concrete gravel and 50% sand)—were compacted using three different compactors, namely an impact rammer, vibratory plate, and air tamper. Test results indicated an increase in the DCP blows at greater depth, which was attributed to the effects of the confining pressure. However, Jayawickrama et al. (2000) suggested that the DCP was capable of differentiating between the compaction equipment and compaction energy levels that were applied to the backfill material.

Chen et al. (2001) conducted a study to evaluate the DCP effectiveness in assessing the modulus of compacted subgrade and base course materials. Sixty DCP and FWD tests were performed on two pavement test sections. M_r laboratory tests were performed on samples obtained during field testing. The authors also investigated the effect of mobile load simulator (MLS) loading on the modulus values by conducting field tests before and after loading. Figure 71 presents a comparison between the moduli values obtained based on FWD, DCP, and laboratory test results. The moduli obtained by using DCP test results yielded similar values to those obtained using FWD tests. In addition, the M_r laboratory-measured values were slightly higher than those determined from the DCP and FWD tests. The authors indicated that the average modulus values for the subgrade were approximately the same before and after loading. The moduli for the base layer were reduced after loading, but the reduction was statistically insignificant. Finally, the authors recommended that to achieve 95% confidence level and an error of estimate of less than 20%, a sample size of six DCP tests should be used for routine characterization of base and subgrade layers.

Rathje et al. (2006) reported that, in general, the DCP was able to distinguish between locations with smaller and larger dry unit weights for clayey, sandy soils and fine gravel but

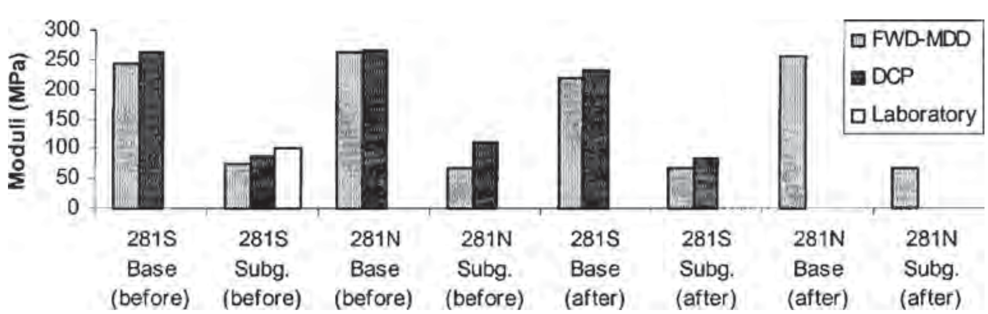


FIGURE 71 Comparison of moduli from different tests (Chen et al. 2001).

did not provide assessments of adequate compaction when compared with direct measurements of dry unit weight. Rathje et al. (2006) indicated that the DCP test could not be performed in coarse gravel aggregate.

Minnesota Study

The Minnesota DOT was one of the first state DOTs to use the DCP for the evaluation of unbound pavement layers. During the early 1990s, the DCP was used in various projects for locating high-strength layers in pavement structures and identifying weak spots in constructed embankments (Burnham 1993). Burnham (1993) analyzed 700 DCP test results on subgrade, subbase, and base materials to determine limiting DCP penetration rates that corresponded to conditions of "adequate compaction." In this study, the limiting values were determined to be 7 mm/blow (0.28 in./blow) for granular materials and 76.2 mm/blow (3 in./blow) for silty/clayey material. Although moisture content affected the DCP values, the Minnesota DOT specification at the time included limiting penetration rate values without any consideration of moisture content. No direct correlation between DCP penetration rate and dry unit weight was found in that study. Moreover, it was also determined that DCP penetration rates were not valid over the top few inches of a compacted lift owing to lack of confinement. Results indicated that the DCP test was able to distinguish between the locations with smaller and larger dry unit weights, but the Minnesota DOT criterion for adequate compaction did not agree with the direct measurements of dry unit weight or relative compaction.

Oman (2004) collected data from 21 construction projects around Minnesota. A total of 82 locations consisting of different types of unbound granular base materials were tested. Based on the analysis of collected data, a relationship between DCP penetration and gradation and moisture content was developed. The collected data also were used to develop trial DCP specifications. The enhanced DCP specifications greatly improved the capability of the DCP and reduced testing time. A simple spreadsheet was developed for the specification, which required gradation data, moisture content at the time of testing, and DCP penetration values. Owing to limited testing data, it was concluded that the proposed specification be further validated using additional field testing data.

Dai and Kremer (2006) attempted to verify and improve the trial Minnesota DOT DCP specifications developed by Oman (2004). Additional field tests were performed and pilot construction projects were implemented. A total of 11 construction projects were selected, and at each project, several locations were randomly selected for testing. Various devices were used at each location to obtain in situ stiffness, strength, density, and moisture content. Materials under consideration included typical granular base materials in Minnesota as well as reclaimed asphalt material. Data obtained from pilot projects confirmed the previously established relationship between the DCP penetration index, gradation, and moisture content. Moreover, the DCP and sand cone density data collected in four projects showed that the DCP specifications were consistent with the current sand cone density test specifications, which further validated the specifications. The authors indicated that one of the major advantages of the DCP was that it could be applied to materials on which the sand cone density test could not be performed.

Petersen and Peterson (2006) conducted DCP tests at 22 locations on the final lift of a 3-ft subcut consisting of a granular borrow compacted using a vibratory compaction roller equipped with intelligent compaction technologies. The authors indicated good correlation was obtained between intelligent compaction measurements (i.e., CMV) and the DCP penetration rate for depths between 203 mm (8 in.) and 406 mm (16 in.) when comparison was done on a point-by-point basis. This suggested that DCP measurement depth was close to the influence depth range of the roller sensors.

Davich et al. (2006) conducted a study as part of Local Road Research Board Investigation 829 to validate the DCP specification for compaction control of granular materials. DCP and speedy moisture tests were conducted on samples of three types of granular materials, which were compacted inside an open-topped steel cylinder (half of a 55-gallon steel drum) using a procedure similar to that of the standard Proctor test. The results of this study indicated that the DCP specification should not be limited to three DCP drops because additional drops might be needed to verify the compaction quality for the entire depth of the layer. The authors concluded that the seating requirement was found to be unnecessary for the subbase layer; however, the requirement was still useful for determining the suitability of an aggregate base surface for paving equipment loading. Regarding moisture content during DCP testing, it was concluded that moisture content should be capped at 10%. Three different ranges of moisture contents (less than 5%, between 5% and 7.5%, and between 7.5% and 10%) were recommended during DCP testing.

White et al. (2009a) compared DCP results to IC roller CMV and MDP measurements as well as test-rolling rut values. As shown in Figure 72, a fair correlation was obtained between the DCP values and CMVs for granular subgrade and base materials. The DCP penetration rate had a relatively weak correlation with the MDP. However, both correlations were improved when the moisture content was included in the regression analyses. The authors proposed a method to determine the bearing capacities under the heavy roller wheel using layered bearing capacity analytical solutions and DCP profiles. The ultimate bearing capacities determined using this method were empirically related to the measured rut depths at the surface during test rolling. This was used to determine the target DCP penetration rate

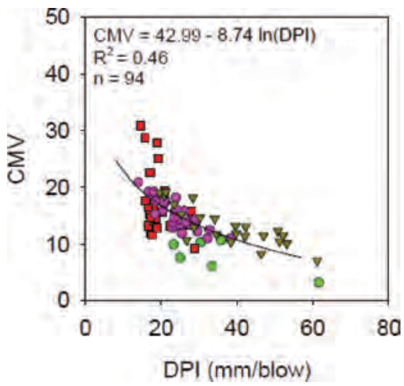


FIGURE 72 Correlations between CMV and in situ point measurements obtained from TH-36 and US-10 field projects with granular soils (White et al. 2009a).

of a subgrade soil to avoid rut failures under the test rolling, thus eliminating the need for this test.

Mississippi Study

George and Uddin (2000) conducted a study for the Mississippi DOT to relate the DCP penetration rate to the resilient modulus obtained from laboratory tests and FWD back-calculated moduli for various types of subgrade soils. Manual and automatic DCP tests, as well as FWD tests, were performed on fine-grained (A-6) and coarse-grained (A-3 and A-2-6) subgrade soils at 12 sites in Mississippi. Shelby tube samples were obtained from tested subgrade soils, and resilient modulus laboratory tests were conducted on those samples. The results indicated that measurements from the manual DCP and automatic DCP were statistically the same. Two prediction models provided in Eqs. 46 and 47 for fine-grained soil and coarse-grained soils, respectively, were developed. It was concluded that M_r prediction was not only dependent on DPI but also related to the soil's physical properties, such as dry density and moisture content.

$$M_r = 27.86(DPI)^{-0.144}\left[\gamma_{dr}^{7.82} + \left(\frac{LL}{w_c}\right)^{1.925}\right] \qquad (R^2 = 0.71) \tag{46}$$

$$M_r = 90.68\left(\frac{DPI}{\log c_u}\right)^{-0.305}\left(\gamma_{dr}^{-0.935} + w_c^{0.674}\right) \qquad (R^2 = 0.72) \tag{47}$$

where

M_r = resilient modulus (MPa),
DPI = DCP index (mm/blow),
γ_d = dry density,
w_c = moisture content,
LL = liquid limit,
PI = plasticity index, and
c_u = uniformity coefficient.

Colorado Study

Mooney et al. (2008) investigated the efficiency of in situ test devices, including the DCP, for quality assurance of Class 1 backfill in mechanically stabilized earth wall and bridge approach embankment. Extensive testing using the DCP was conducted at two construction sites. The results of conducted tests indicated that the DCP penetration index could replace the current density-based compaction method. However, the authors indicated that moisture content should be considered when developing DCP target values used for compaction acceptance. In addition, the DCP test results need to be corrected when the DCP penetrates through geosynthetic reinforcements placed in mechanically stabilized embankments.

Louisiana Studies

Abu-Farsakh et al. (2004) evaluated the viability of using the DCP as a tool for stiffness-based QC/QA procedures during and after the construction of pavement layers and embankments. Results showed that the DCP is an excellent and reliable device for evaluating stiffness/strength properties of various types of unbound materials. Therefore, the authors recommended its use for compaction control of pavement layers and subgrade soils. Based on the nonlinear regression analysis that was conducted on data collected in this study, strong correlations were found between the DCP penetration rate and the FWD and PLT moduli. Those correlations are provided in Eqs. 48 through 50.

$$\ln(M_{FWD}) = 2.04 + \frac{5.1873}{\ln(DPI)}$$
$$(3.27 < DPI < 66.67) \qquad (R^2 = 0.91) \tag{48}$$

$$E_{PLT(i)} = \frac{9770}{(DPI)^{1.6} - 36.9} - 0.75$$
$$(3.27 < DPI < 66.67) \qquad (R^2 = 0.67) \tag{49}$$

$$E_{PLT(R2)} = \frac{4374.5}{(DPI)^{1.4} - 14.9} - 2.16$$
$$(3.27 < DPI < 66.67) \qquad (R^2 = 0.72) \tag{50}$$

Regression analysis was also performed to correlate the laboratory CBR and the DCP penetration rate. The following nonlinear regression model was obtained:

$$\text{CBR} = 2559.44/(-7.35 + DPI^{1.84}) + 1.04$$
$$(6.31 < DPI < 66.67) \qquad (R^2 = 0.93) \tag{51}$$

Mohammad et al. (2009) conducted field and laboratory testing programs to develop models that would predict the resilient modulus of subgrade soils from results of the DCP. A total of four soil types (A-4, A-6, A-7-5, and A-7-6) were considered at different moisture-dry density levels.

A simple linear regression analysis was first conducted on the combined data set to develop a model that directly

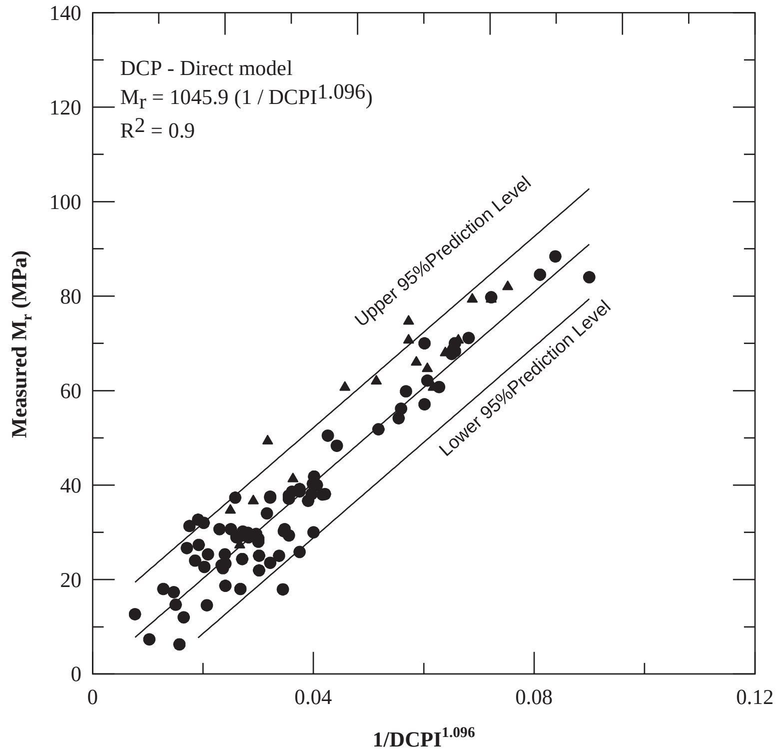


FIGURE 73 M_r—DCP direct model (Mohammad et al. 2009).

predicted the laboratory-measured M_r from the DCP penetration rate. The results of this analysis yielded the model shown in Eq. 52. Figure 73 illustrates the results of the regression analysis. A multiple nonlinear regression analysis also was conducted to develop a model that predicted laboratory-measured M_r from the DCP as well as the physical properties of the tested soils. The results of this analysis are presented in Eq. 53 and Figure 74.

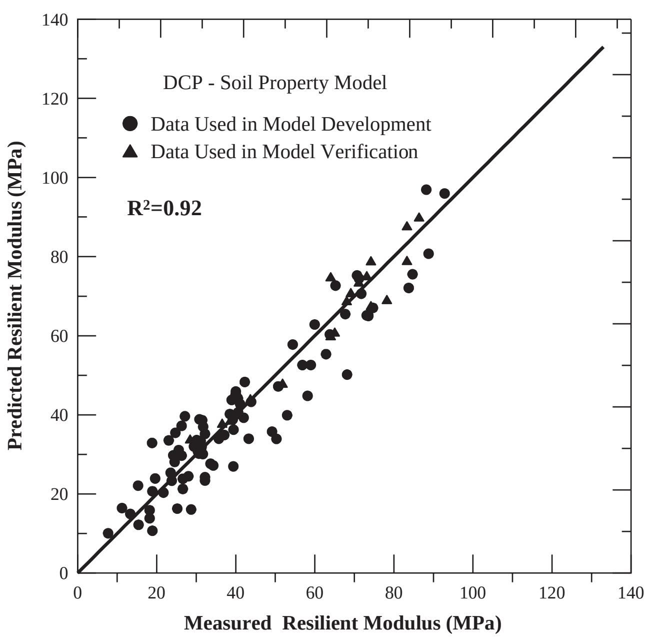


FIGURE 74 M_r—DCP soil property model (Mohammad et al. 2009).

It is noted that the developed model was able to provide good prediction for the data obtained from a study by George and Uddin (2000) that was not used in the development of the model.

$$M_r = \frac{1045.9}{(DPI)^{1.096}} \qquad (R^2 = 0.90) \tag{52}$$

$$M_r = 3.86 + 2020.2\left(\frac{1}{DPI^{1.46}}\right) + 619.4\left(\frac{1}{w^{1.27}}\right) \qquad (R^2 = 0.92) \tag{53}$$

where

M_r = resilient modulus (MPa),
DPI = DCP index (mm/blow), and
w = water content (%).

Florida Study

Parker et al. (1998) reported a study in which automated and manual DCP devices were compared. In this study, a series of DCP tests were conducted to evaluate the in situ strength of granular materials and subgrade soils in Florida. No considerable difference in the DPI was found when results from the manual and automated DCP were compared. Furthermore, the study indicated that confinement and depth affected the DCP strength measurement of granular materials, whereas the strength measurements of cohesive materials were minimally influenced by confinement.

Indiana Studies

Salgado and Yoon (2003) tested various subgrade soils at seven sites in Indiana using the DCP and the NDG. Four sites contained clayey sands, one contained a well-graded sand with clay, and two contained a poorly graded sand. Soil samples were obtained from the sites and tested in the laboratory. The results of this study indicated that despite having a considerable scatter in obtained data, a trend appeared to exist between the DCP penetration index and the soils' physical properties, such that the penetration index decreased as dry density increased and slightly increased as moisture content increased. The authors proposed the model shown in Eq. 54 to predict the dry density for clayey sand soil from the DCP penetration index. They recommended not using the DCP in testing soil with gravel because unrealistic DCP results could be obtained and the penetrometer shaft could be bent.

$$\gamma_d = \left(10^{1.5} \times DPI^{-0.14} \times \sqrt{\frac{\sigma'_v}{p_A}}\right)^{0.5} \times \gamma_W \qquad (54)$$

where

DPI = DCP index in mm/blow,
p_A = reference stress (100 kPa), and
σ'_v = effective stress.

Siddiki et al. (2008) conducted a study to develop a criterion for compaction quality control of a bottom ash embankment. DCP tests were conducted during the compaction of a test pad of coal ash. Based on the results, a criterion of 16 blows for every 300-mm-thick layer of bottom ash was developed, which was used for compaction control of the remaining 11-m-high embankment. Subsequent construction monitoring and postconstruction evaluation of the bottom ash embankment indicated that the developed criterion was very effective. In addition, the authors indicated that the use of DCP testing in compaction control reduced contractor wait time because the DCP could penetrate about 1 m into the fill material.

Wisconsin Study

Based on tests conducted on natural earthen materials, industrial by-products, chemically stabilized soils, and other materials at 13 construction sites in Wisconsin, Edil and Benson (2005) found that the DCP could be used to assess the compaction quality of subgrades by correlating a normalized DCP parameter with relative compaction for the subgrade soil in question.

Iowa Studies

In a study for the Iowa DOT, Bergeson (1999) investigated the use of the DCP for quality control and acceptance procedures during embankment construction. For cohesive soils, the field data indicated that the stability and shear resistance measured by the DCP increased with an increase in the compaction effort and were reduced as the moisture content increased. However, the DCP results did not correlate with moisture content or density measurements. For granular soils, the DCP was found to be an adequate tool for evaluating the in-place density when moisture control was applied to the embankment. Figure 75 shows the variation

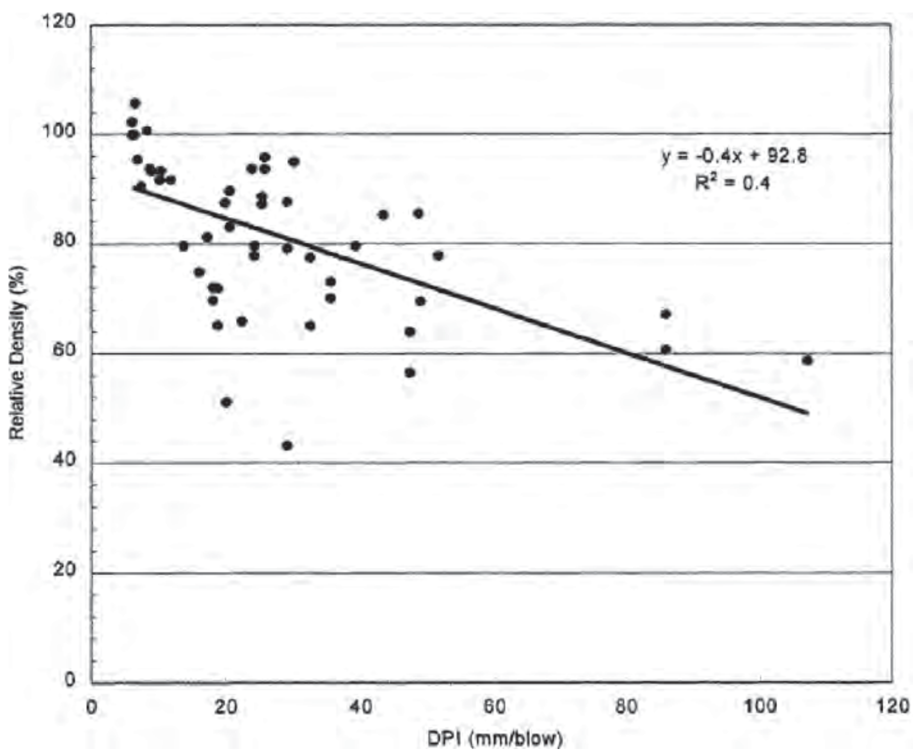


FIGURE 75 Relative density compared with DCP index for granular soil (Bergeson 1999).

of the DCP penetration rate with relative density for granular materials in that study. It was observed that a DCP penetration rate of 35 mm/blow corresponded to a relative density value of 80%, which was selected as the DCP limit for compaction control of granular materials. However, more study for validation of this limit value was suggested.

Gas Technology Institute Study

The Gas Technology Institute (GTI) conducted a research project in which several compaction devices, including the DCP, were evaluated (Farrag et al. 2005). Sand, silty clay, and aggregate base were tested using each device. The study found that the DCP provided only general postcompaction information, such as existence of weak layers or layer boundaries. The DCP penetration rate also did not correlate well with dry unit weight or other compaction parameters. In addition, the DCP provided unreliable data within the top 6 in. of most material tested because of lack of confinement.

U.S. Army Corps of Engineers Studies

The U.S. Army Corps of Engineers conducted a field DCP study for a wide range of granular and cohesive materials. The results of this study showed a strong correlation, shown in Eq. 55, between the CBR and the DCP penetration ratio (Webster et al. 1992). This equation has been adopted by many state DOTs and appears in the *Mechanistic–Empirical Pavement Design Guide* (MEPDG) (Webster et al. 1992; Livneh et al. 1995; Siekmeier et al. 2000; Chen et al. 2001).

$$\text{Log CBR} = 2.465 - 1.12\left(\log DPI\right)$$
$$\text{or} \quad \text{CBR} = 292/DPI^{1.12} \qquad (55)$$

Recently, Berney et al. (2013) reported the results of a study conducted at the U.S. Army Corps of Engineers Research and Development Center in Vicksburg, Mississippi, to examine the effectiveness of the DCP as an alternative to the NDG. As shown in Figure 76, a good correlation between the DCP test and NDG dry density measurements was obtained. However, DCP was not recommended because it did not provide a moisture content measurement.

International Studies

Based on a series of DCP tests conducted on various types of cohesive and granular soils in the United Kingdom, Huntley (1990) suggested a tentative classification system of soil, shown in Tables 14 and 15, based on DCP penetration resistance (blows per 100 mm). However, the author recommended the use of classification tables only with considerable caution until a better understanding of the mechanics of skin friction on the upper drive rods was established.

Different correlations were suggested between the DCP penetration rate and the CBR value. Kleyn (1975) conducted DCP tests on 2,000 samples of pavement materials in standard molds directly following CBR determination. Based on Kleyn's results, the correlation in Eq. 56 was proposed. In a field study, Smith and Pratt (1983) found the correlation presented in Eq. 57. Livneh and Ishai (1987, 1995) conducted a correlative study between DCP values and the in situ CBR values. During this study, both CBR and DCP tests were done

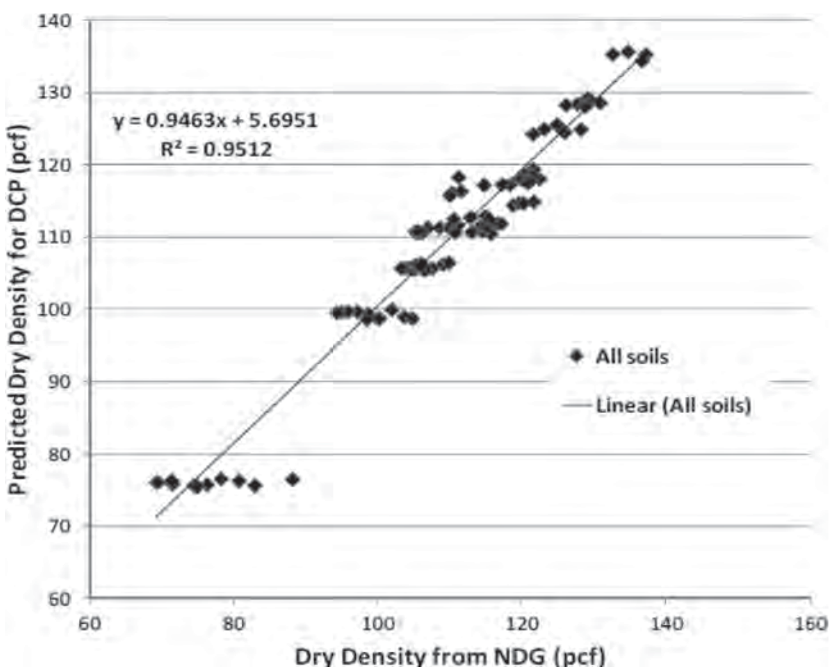


FIGURE 76 Correlation between DCP and dry density obtained using the NDG (Berney et al. 2013).

TABLE 14
SUGGESTED CLASSIFICATION FOR GRANULAR SOIL USING DCP

Classification	Range of n Values		
		Sand	Gravelly sand
Very loose	<1	<1	<3
Loose	1–2	2–3	3–7
Medium dense	3–7	4–10	8–20
Dense	8–11	11–17	21–33
Very dense	>11	>17	>33

Source: Huntley (1990).

in the laboratory on a wide range of undisturbed and compacted fine-grained soil samples, with and without saturation. Compacted granular soils were tested in flexible molds with variable, controlled lateral pressures. Field tests were conducted on the natural and compacted layers, representing a wide range of potential pavement and subgrade materials. The research resulted in the models shown in Eq. 58. Harrison (1989) also suggested Eqs. 59 and 60 to relate CBR to the DCP for different soils.

$$\text{Log CBR} = 2.62 - 1.27 \log DPI \qquad (56)$$

$$\text{Log CBR} = 2.56 - 1.15 \log DPI \qquad (57)$$

$$\text{Log CBR} = 2.2 - 0.71(\log DPI)^{1.5} \qquad (58)$$

$$\text{Log CBR} = 2.56 - 1.16 \log DPI$$
$$\text{for clayey-like soil of } DPI > 10\ (\text{mm/blow}) \qquad (59)$$

$$\text{Log CBR} = 2.70 - 1.12 \log DPI$$
$$\text{for granular soil of } DPI < 10\ (\text{mm/blow}) \qquad (60)$$

Some researchers have attempted to relate DCP results to the elastic modulus of various unbound materials. Table 16 summarizes the main correlations between the DCP and unbound materials modulus values that were reported in international studies.

TABLE 15
SUGGESTED CLASSIFICATION FOR COHESIVE SOIL USING DCP

Classification	Range of n Values
Very soft	<1
Soft	1–2
Firm	3–4
Stiff	5–8
Very stiff to hard	>8

Source: Huntley (1990).

SOIL COMPACTION SUPERVISOR

The soil compaction supervisor (SCS), formerly the soil compaction meter, consists of a disposable sensor that is connected by a cable to a battery-powered, handheld control unit, as shown in Figure 77. The SCS works by embedding the sensor at the bottom of the soil layer to be compacted. The sensor includes piezoelectric transducers that produce a voltage in response to the waves transmitted through the soil from the compaction equipment. The voltage is transferred to the SCS control unit through the connecting cable. The transmitted voltage increases with the increase in the soil stiffness and density owing to compaction. The main function of the SCS is to monitor the voltages from embedded sensors and report when the asymptotic value of stiffness has been reached. A green light on the display indicates that the soil did not reach maximum stiffness value, whereas a red light indicates that the voltage reached its asymptotic value and the compaction process should be stopped. The cost of the SCS device is $1,650.

Influence Depth

Previous studies by the Gas Research Institute indicated that SCS sensors could provide readings to approximately 762 mm (30 in.) of soil thickness above it (Cardenas 2000; Farrag et al. 2005).

Advantages and Limitations

The SCS device is portable, economical, and can be operated with minimum training. Red and green light signals provide a clear, instant indication of when compaction should be stopped or continued. This signaling system can reduce the risk of over-compaction and save time during compaction by indicating when rolling is no longer needed. However, some limitations of this device have been reported. Farrag et al. (2005) reported that although the red light had good correlation with 90% relative compaction in sand, the correlation was weak for clay. Although the SCS box has good durability, the sensors are less durable. The device also does not provide any test results applicable to design or quality

TABLE 16
SUMMARY OF DCP-MODULUS CORRELATIONS REPORTED IN INTERNATIONAL STUDIES

Correlation	Reference study
Log (*Es*) = 3.05 – 1.07 Log (*PR*)	De Beer (1990)
Log (*Es*) = 3.25 – 0.89 Log (*PR*)	
Log (*Es*) = 3.652 – 1.17 Log (*PR*)	Pen (1990)
Log (*EPLT*) = (-0.88405) Log (*PR*) + 2.90625	Konard and Lachance (2000)
E (in MPa) = 2,224 DCP – 0.99	Chai and Roslie (1998)
Log(*MFWD*) = 3.04785 1.06166*Log(*DPI*)	South Africa (8)

DCP is DCP in blows per 300 mm; *EPLT* is the modulus from plate load test (in MPa); *Es* is the elastic modulus (in MPa); and *PR* is the DCP penetration rate (in mm/blow).

control purposes. In addition, the SCS does not have moisture measurement, GPS, or good data storage capabilities. Currently, the SCS has no standard procedure. Rathje et al. (2006) also indicated that it has a weak theoretical basis. In addition, the authors noted that the SCS has not been evaluated by state DOTs. Thus, there is a lack of evidence of prior success in using it in compaction control of pavement layers and embankments.

Synthesis of Past Research Studies

This study's survey results indicated that no research studies have been performed by state DOTs to evaluate the SCS. Most of the studies that evaluated the device were conducted by the Gas Technology Institute.

Cardenas (2000) evaluated the efficacy of the SCS in controlling the compaction of soil layers. Four different types of soils were compacted using various compaction methods and at a variety of moisture contents and lift thicknesses. Nuclear density gauge measurements were obtained after the completion of each pass of the compaction equipment. Compaction was continued after the SCS red stop signal was displayed, and the density and moisture content were measured when two and four passes were subsequently completed. The results indicated that the average dry density measurements obtained after the red stop signal was displayed increased by less than 2% with additional compactive effort. Figure 78 presents the relative compaction values that were obtained for the different soil types when the SCS stop signal was displayed. As shown in the figure, all soil types, except low plasticity soil, were compacted to at least 95% relative compaction when the SCS displayed the red stop signal. Cardenas (2000) indicated that the low plasticity soil was overly wet when compacted and therefore did not represent optimal compaction conditions.

FIGURE 77 Soil compaction supervisor sensor and control unit (MBW 2003).

In another study, Farrag et al. (2005) investigated the use of the SCS to monitor the compaction of several bell holes and keyholes that were filled and compacted with sand, silty-clay, and stone-base materials. Figures 79 through 81 compare the results of the SCS device with the obtained relative compaction for sand, silty-clay, and stone-based materials, respectively. The results showed that most of the output signals in sand and stone-base soils corresponded to 90% compaction or higher. However, for the silty-clay soil the SCS red stop signal was obtained at relative compaction values that were less than 90%. Figures 79 through 81 also show that the SCS device failed to produce signals when soil height was more than 762 mm (30 in.) above the sensor.

Juran and Rousset (1999) conducted a field study in which the SCS was used to assess the compaction quality of five test trenches compacted with sandy backfill material. The results of this study indicated that the SCS generally displayed the red stop signal, at relative compaction values that were less than the required 95% value.

PORTABLE SEISMIC PROPERTY ANALYZER

The PSPA is a portable version of the large, trailer-mounted seismic pavement analyzer (SPA), which was first developed at the University of Texas at El Paso to test both flexible and rigid pavements for early signs of distress and provide general quality control during pavement construction. As

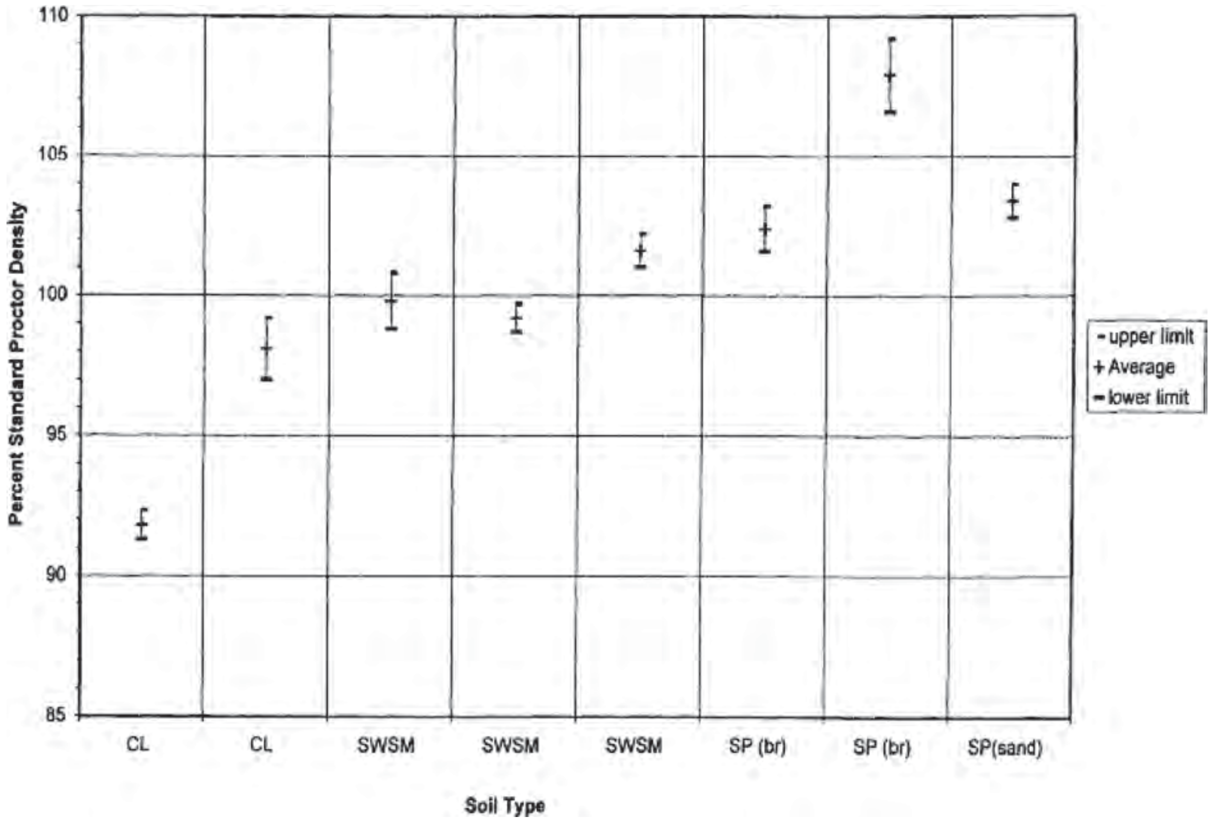


FIGURE 78 Relative compaction at SCS stop signal (Cardenas 2000).

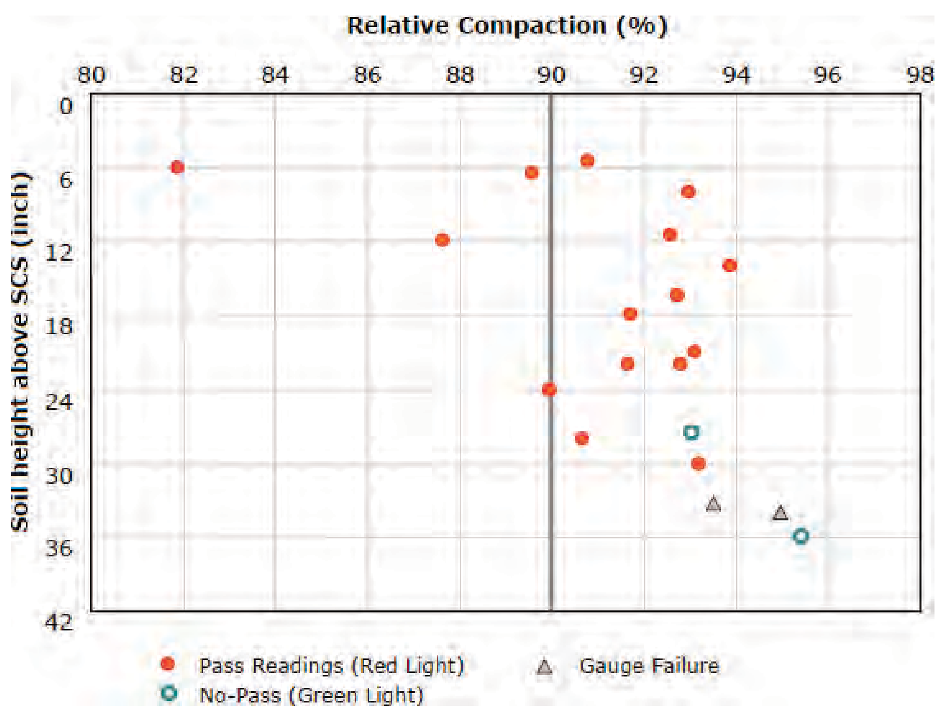


FIGURE 79 Comparison between SCS output and relative compaction at various depths in sand (Farrag et al. 2005).

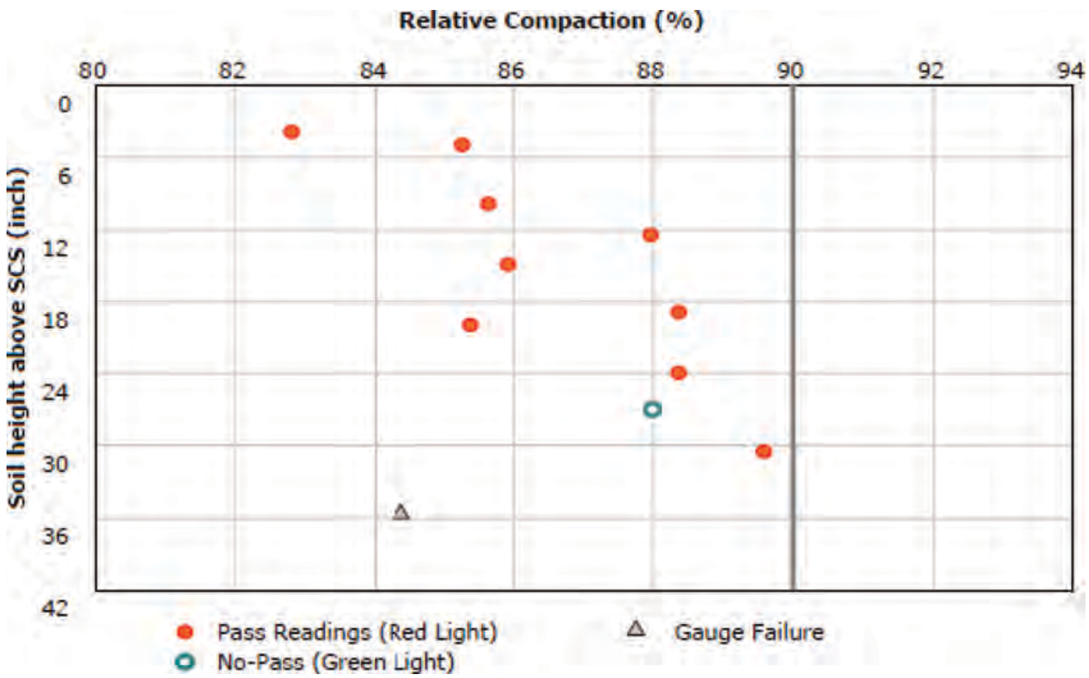


FIGURE 80 Comparison between SCS output and relative compaction in silty-clay (Farrag et al. 2005).

shown in Figure 82, the PSPA consists of two receivers (or accelerometers) and a wave source packaged into a hand-portable system. The device is operated by a laptop, which is connected to the hand-carried transducer unit through a cable that carries power to the receivers and wave source and returns the detected signal to the data acquisition board inside it. The PSPA costs from $20,000 to $30,000.

Principle of Operation

The PSPA principle of operation is based on generating and detecting surface waves in the tested layer and using the ultrasonic surface wave (USW) method to determine its modulus.

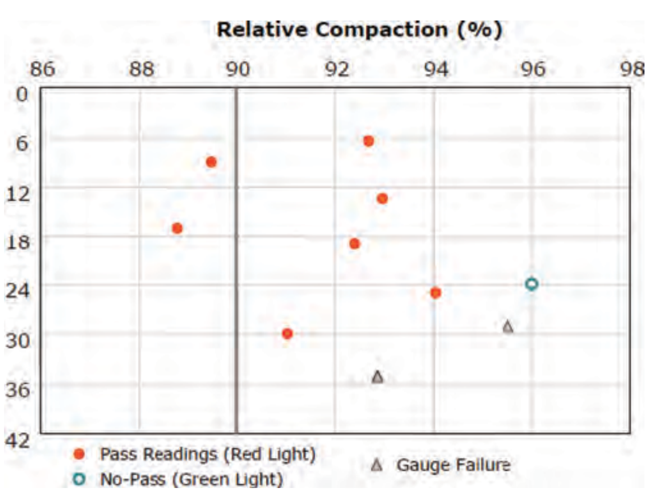


FIGURE 81 Comparison between SCS output and relative compaction in stone-base materials (Farrag et al. 2005).

The USW method is an offshoot of the spectral analysis of surface waves (SASW) method applied to high-frequency seismic tests. Both methods are based on the measurement of the dispersive nature of the Rayleigh-type surface waves propagating in a layer to determine the shear wave velocity. The main difference between the two methods is that in the USW, the modulus of the tested layer is directly determined without an inversion algorithm. PSPA testing involves activating the source through the laptop to generate high-frequency surface waves that propagate horizontally and are detected and measured by the two receivers. The two receiver outputs are used to compute the Rayleigh wave velocity (VR) at different frequencies, which represent the variation of VR with depth. The VR can be used to compute the Young's modulus using Eq. 61 (Jersey and Edwards 2009). The PSPA software computes an average modulus over the depth measured. It is worth noting that the depth of the material tested by the PSPA can be controlled by adjusting the receivers' spacing.

$$E = 2\rho\left(1.13 - 0.16v\right)VR^2 \qquad (61)$$

where

ρ = total mass density, and
V = Poisson's ratio of soil.

Repeatability

There are limited data on the repeatability of this device. Von Quintus et al. (2008) reported a COV of PSPA modulus ranging from 6.0% to 18.5% when testing different pavement materials. Jersey and Edwards (2009) found that COV values

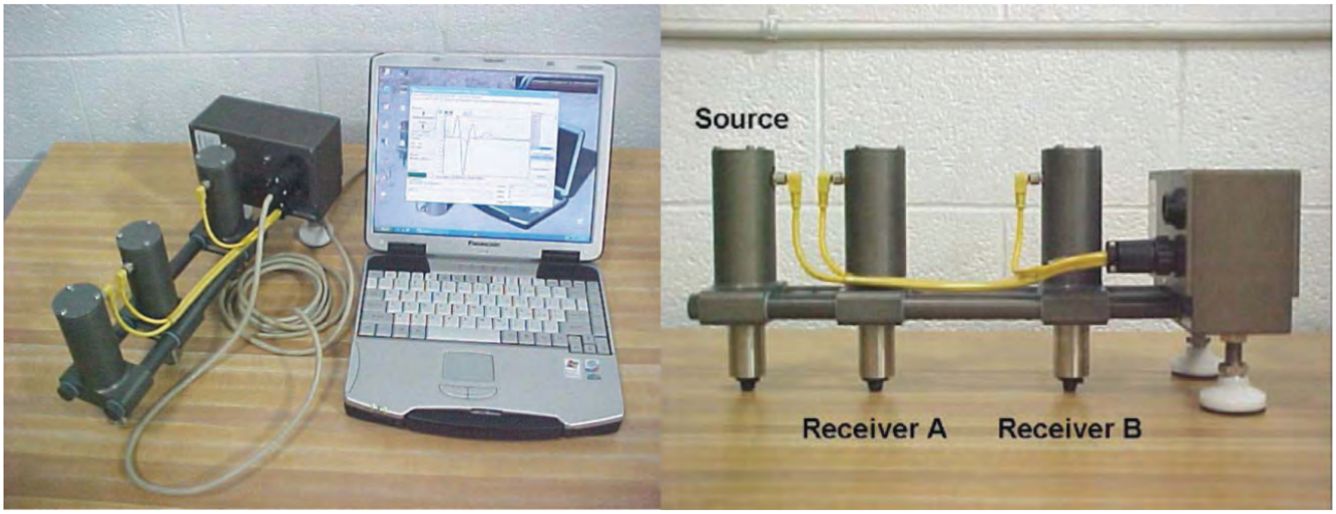


FIGURE 82 PSPA components and data acquisition system (Ellen et al. 2006).

for PSPA measurements ranged between 10% and 21% for sands and 7% and 36% for fine-grained soils. They indicated that PSPA measurements were repeatable for the same location but varied among test locations, leading to higher COV values.

Advantages and Limitations

The PSPA is a small, portable, and easy-to-handle device. Testing with the device takes about 15 s to complete. The PSPA allows for monitoring the stiffness changes in the subgrade and aggregate base at different stages of compaction. In addition, results of lab and field seismic tests are anticipated to be similar for the same materials. This allows for obtaining a lab target modulus value that can be used in the field. However, the PSPA device requires a laptop computer in the field for data acquisition and reduction. In addition, a skilled operator is needed to conduct and analyze the data. The PSPA also requires soil-specific calibration, which involves conducting complex resonant column-torsional shear laboratory testing. Furthermore, the PSPA-measured modulus does not represent the stress level encountered in the field and may have to be adjusted to account for the design loading frequency and strain. The PSPA also does not have a standard test method. Finally, the PSPA is considerably more expensive than the NDG and other in situ test devices.

Synthesis of Previous Studies

Texas Studies

Rathje et al. (2006) evaluated the performance of the PSPA as a tool for compaction control of earth embankments and mechanically stabilized earth wall backfill. In the study, the PSPA was used to test five different soils ranging from high plasticity clay to gravel. The results indicated that for clayey soil the PSPA modulus was influenced by the moisture content more than the dry density. The PSPA modulus, in general, increased with increasing dry density of sandy soil. However, there was significant scatter in these data. Finally, for the gravel soils, the authors reported that it was difficult to use these devices for compaction control because there was significant scatter in the PSPA modulus.

Nazarian et al. (2006) conducted an implementation project for the Texas DOT. Based on the results of this study, procedures to measure the seismic moduli of pavement layers in the lab and the field were developed. Protocols for using the PSPA to assess the compaction quality of different pavement layers also were presented. The authors found that the seismic modulus correlated well with the resilient modulus design value. Thus, the use of the PSPA allowed for validating the modulus design input value.

U.S. Army Corps of Engineers Study

Jersey and Edwards (2009) presented the results of a study in which the PSPA, LWD, and GeoGauge tests were conducted on 11 soil test beds that were constructed at the U.S. Army Corps of Engineers Research and Development Center. The results of the different tests conducted in this study are presented in Figure 83. For poorly graded sands (items 1 and 2) the PSPA moduli had similar trends to the GeoGauge and LWD but had approximately twice the magnitude. For fine-grained soils (items 3, 4, and 5), however, there was no observed trend for PSPA moduli. The authors indicated that the PSPA and the other evaluated devices were simple to use and generally obtained repeatable results. However, additional information about the true nature of the modulus measured by these tools was needed to implement their use in compaction control.

Joh et al. (2006) tested 21 subgrade soils and 11 base course materials using an accelerated SASW system (similar to the PSPA), the DCP, the PL, and FWD tests. The authors

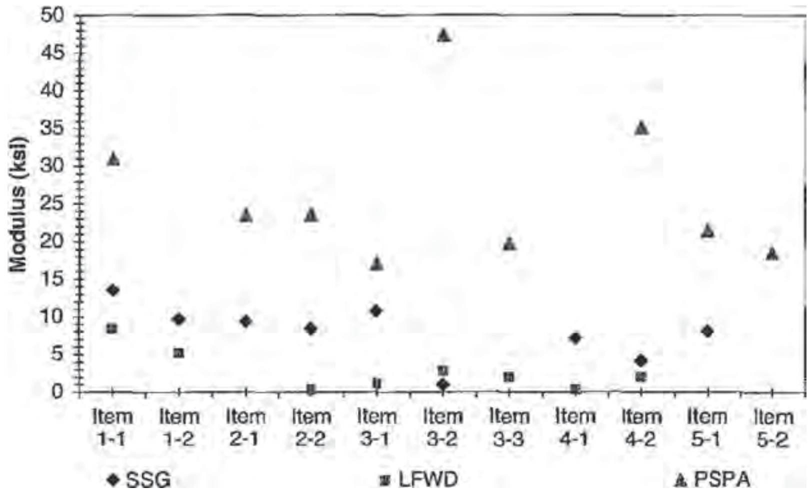


FIGURE 83 Modulus measured by GeoGauge, LWD, and PSPA (Jersey and Edwards 2009).

found that there was a favorable correlation between the DPI and the shear wave velocity measured using the SASW system for both subgrades and base materials. As shown in Figure 84, this correlation became stronger when shear wave velocity increased. However, the authors could not find any strong correlation between SASW modulus and the coefficient of subgrade reaction obtained from the PLT. Finally, the FWD modulus had good correlation with the SASW modulus.

BRIAUD COMPACTION DEVICE

The BCD consists of a 150-mm-diameter flexible thin plate attached at the bottom of a rod. The plate is instrumented with eight radial and axial strain gauges, as shown in Figure 85.

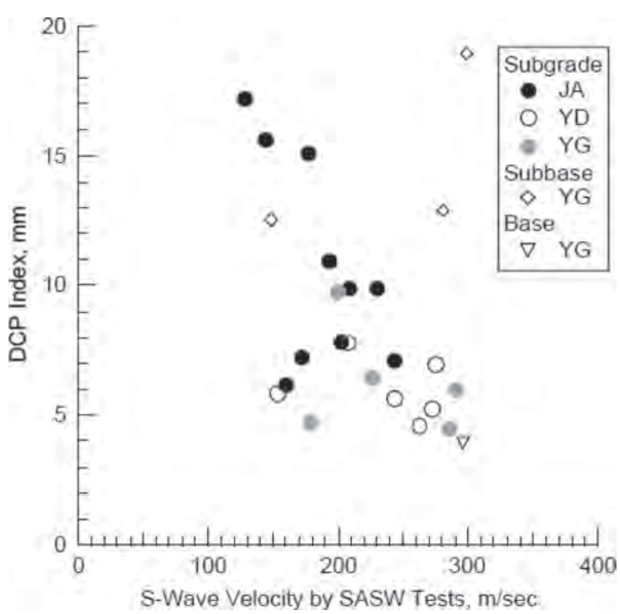


FIGURE 84 DCP index compared with shear wave velocity (Joh et al. 2006).

A load cell is located above the plate to measure the force applied by the person leaning on the BCD. The BCD works by applying a small repeatable load to a thin plate in contact with the compacted material to be tested. Once loaded, the plate bends and the bending strains are instantaneously measured by the strain gauges mounted on the plate. Proprietary software within the device uses correlations from the field and laboratory to compute the BCD low-strain modulus based on the measured strains. The strain level associated with the BCD measured modulus is on the order of 0.1% (Weidinger and Ge 2009).

The BCD has two modes of operation that account for the boundary effects of the Proctor mold that would not occur in the field (Li 2004). A modulus compaction curve in the lab first has to be developed to establish a target modulus from that curve. Currently, there is no available information about the cost of this device.

Influence Depth

Briaud and Rhee (2009) found that the depth of influence varied with modulus of materials such that it decreased from 121 to 311 mm (3.4 to 12.24 in.) as the modulus increased from 3 to 300 MPa. The authors reported that for materials with a modulus between 5 and 100 MPa, the depth of influence was at least 150 mm (5.9 in.). No other studies have validated these results.

Repeatability

Briaud and Rhee (2009) evaluated the repeatability of the BCD by testing the same rubber block eight times. The COV of the strain output was found to be 0.5%. Weidinger and Ge (2009) reported a COV of 4% for BCD modulus when testing silt soil samples compacted in the split Proctor mold.

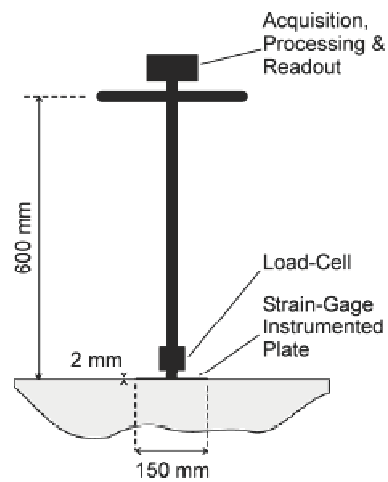


FIGURE 85 Photographs and a conceptual sketch of the BCD (Briaud and Rhee 2009).

Advantages and Limitations

The BCD has several advantages. It is easy to use and can be carried and operated by one person because it weighs only 9.6 kg (4.35 lb). In addition, the BCD is much faster than other in situ test devices, with an actual testing time of approximately 5 s. It also can be used in the lab to determine the target modulus that can be utilized for compaction control of unbound materials in the field.

The BCD is a relatively new device, so it has not been extensively evaluated by previous studies, especially those sponsored by state DOTs. One of the main limitations of this device is that it cannot be used for very soft or very stiff soils. For soft soils, the BCD plate simply penetrates into the soil without bending. For stiff soils, the bending of the plate is not adequate for precise measurement of the strains. The device is considered effective in soils with moduli ranging from 5 to 150 MPa (725.2 to 2,175.6). Finally, its relatively shallow influence depth might affect its efficacy as a tool for compaction control (Weidinger and Ge 2009).

Synthesis of Past Research Studies

NCHRP Highway IDEA Project 118

As part of the NCHRP Highway IDEA program, Briaud and Rhee (2009) reported the results of a project that aimed at improving the design of the previous prototype BCD for compaction control of various unbound materials. The project included conducting PL and BCD tests on 10 test sections with different types of subgrade base materials. The results of those tests are presented in Figure 86. The BCD modulus had an excellent linear correlation with that obtained using the PLT. In addition, to comparing the BCD modulus with the resilient modulus, the authors conducted BCD tests on silty clay samples before performing resilient modulus laboratory tests

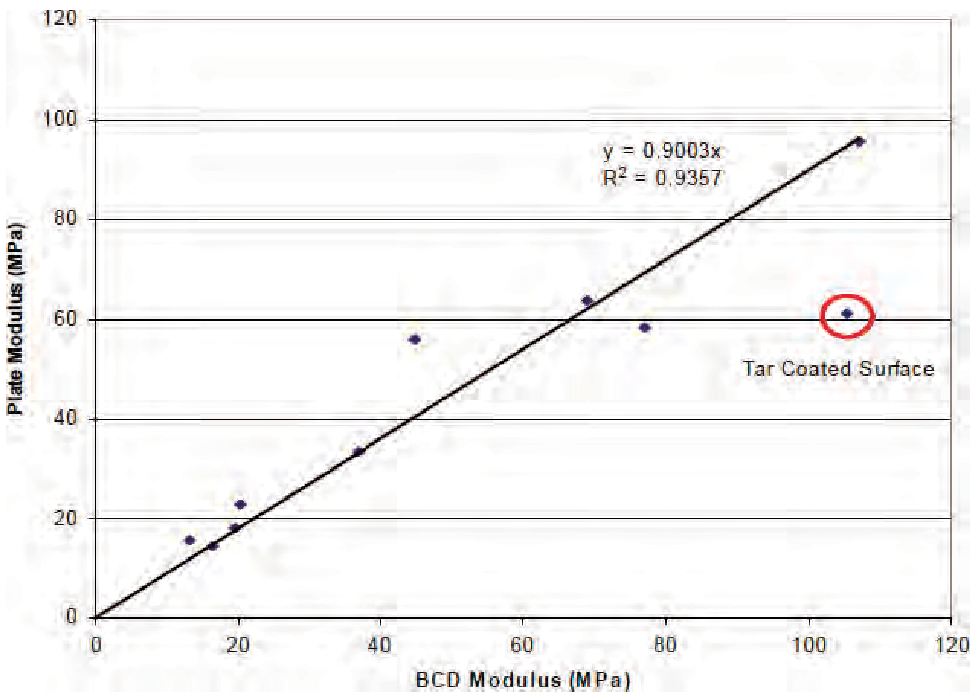


FIGURE 86 Correlation between plate loading test and the BCD test (Briaud and Rhee 2009).

on them. A total of five samples at five different moisture contents were tested. The authors concluded that there was an excellent correlation between the BCD modulus and the resilient modulus.

The authors proposed a procedure for using BCD for compaction control of soil layers. In this procedure, standard or modified Proctor tests are performed and the optimum moisture content and the maximum dry density are determined from the compaction curves. In those tests, the BCD is conducted on top of the Proctor mold sample. This is done to obtain the BCD modulus versus the moisture content curve, which is used to define the maximum BCD modulus and the corresponding optimum moisture content. The authors suggested that the target field modulus value be 75% of the maximum modulus value obtained in the Proctor tests. This target value is verified by conducting BCD tests on the compacted soil in the field. In addition, the moisture content should be verified independently through field testing.

Weidinger and Ge (2009) evaluated BCD laboratory procedures for compacted silty soil. That study also compared the BCD modulus to the dynamic and shear moduli determined from ultrasonic pulse velocity tests on the same compacted silt samples. The authors found that the BCD modulus correlated well with the ultrasonic pulse velocity results. In addition, they suggested that conducting BCD tests on the Proctor compacted soil was simple and quick, which allowed for developing two important soil trends: the dry density versus moisture content curve and the BCD modulus versus moisture content curve. This could be used to establish field compaction specifications based on both dry density and modulus, which ultimately would result in uniformly dense and strong compacted soil layers. However, the authors noted that because of the limitation of the BCD's influence depth, it would be difficult to effectively assess the soil modulus beyond several inches below the surface.

INTELLIGENT COMPACTION

All of the aforementioned in situ test devices can assess the mechanical properties of only a very small portion of the compacted materials around the testing location (Kim et al. 2010). Consequently, there may be weak compaction areas unidentified by the limited spot tests. This may result in nonuniform and inadequate compaction, leading to unsatisfactory long-term performance of the compacted layer. To address this issue, research has been performed to assess the quality of compaction along the entire volume of the compacted material using new compaction technologies, such as continuous compaction control (CCC) and intelligent compaction techniques. The development and evaluation of CCC technologies were initiated in Europe during the late 1970s for use on vibratory rollers compacting granular material (Forssblad 1980; Thurner and Sandström 1980). However, since then CCC technologies have been expanded to different materials and are currently available for different configurations and roller types.

The CCC technologies involve using rollers equipped with a real-time kinematic system (RTK), GPS, roller-integrated measurement system, and an onboard, real-time display of all compaction measurements. If the roller has an automatic

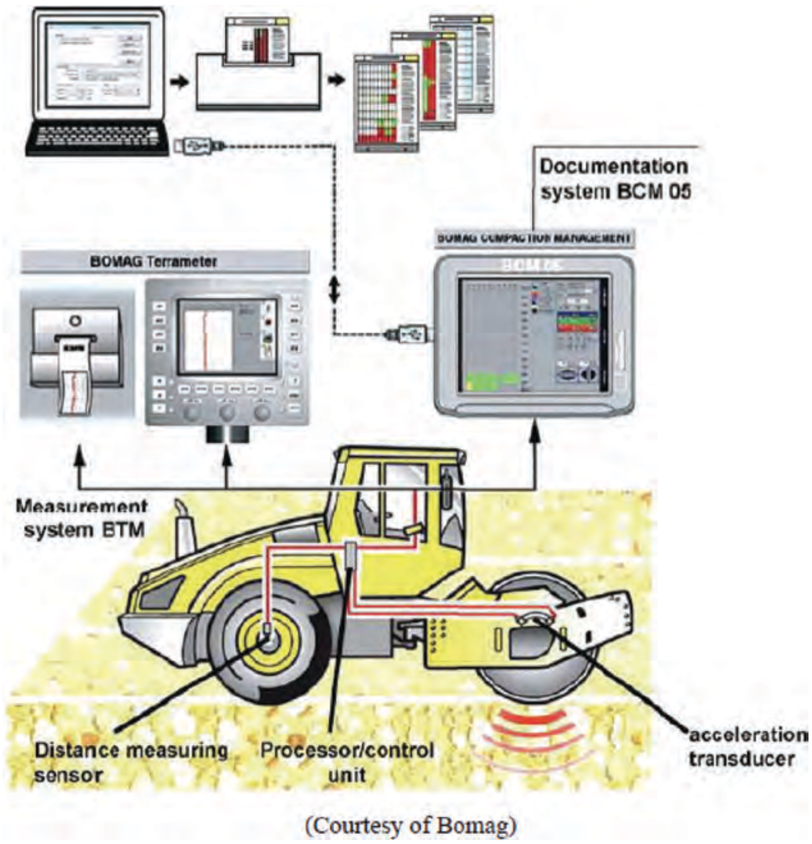


FIGURE 87 An example of IC roller, the Bomag VarioControl System (Chang et al. 2011).

feedback control for its vibration amplitude and/or frequency, the system is referred to as "intelligent" compaction (IC). During compaction, IC rollers maintain a continuous record of measurements, including the number of roller passes, roller GPS location, IC measurement value (ICMV), and roller vibration amplitudes and/or frequencies. Real-time, onboard, color-coded displays of those measurements provide a spatial record of compaction quality and are used to optimize the compaction by adjusting the roller settings manually or automatically. Figure 87 presents an example of an IC roller.

There currently are seven types of IC single-drum rollers in the United States that are used to compact various types of unbound materials (see www.intelligentcompaction.com). Those rollers use different ICMVs to evaluate the level of compaction. A summary of the ICMV measurements is provided in Table 17. ICMVs are computed based on either the measurements of accelerometers mounted on the roller drum or machine drive power (MDP) measurements. Two different approaches are used to compute ICMVs based on accelerometer measurements. The first involves computing the ratio of selected frequency harmonics for a set time interval (e.g., CMV and C_{CV}). The second includes computing stiffness (e.g., k_s) or elastic modulus (e.g., E_{vib}) of compacted material based on a drum-ground interaction model and some assumptions (Chang et al. 2011). ICMVs are influenced by factors such as machine settings (frequency, amplitude). Therefore, it is suggested that all measurements at calibration areas and production areas during quality assurance be obtained at a constant amplitude setting to avoid complication in data analysis and interpretation.

Influence Depth

The influence depth of the IC roller varies with type of ICMV measurement used. For accelerometer-based measurement systems, the influence depths of measurements were reported to range between 0.8 and 1.5 m (2.62 to 4.92 ft) under a 12-ton vibratory roller (International Society for Soil Mechanics and Geotechnical Engineering 2005; NCHRP 21-09 2009; White et al. 2009). On the other hand, for MDP-based measurements the depth of influence ranged between 0.3 and 0.6 m (1 to 2 ft) depending on the variability of the underlying layer (White et al. 2009a). The ICMV influence depth is affected by roller size, vibration frequency, speed of roller, and the force level that it can generate (Chang et al. 2011). However, Mooney et al. (2011) found that the vibrational amplitude has a minimal effect on the ICMV depths.

Advantages and Limitations

There are several benefits of IC technologies that have been identified in the literature. IC technologies provide more

TABLE 17
SUMMARY OF ICMV MEASUREMENTS

IC Measurements	Units	IC Systems	Model Definition
Compaction meter value (CMV)	None	Caterpillar, Dynapac, Volvo	$\text{CMV} = C\dfrac{A_{2\Omega}}{A_{\Omega}}$
Machine drive power (MDP)	None	Caterpillar	$\text{MDP} = P_g - W\text{v}\left(\sin\alpha + \dfrac{A'}{\text{g}}\right) - (mv + b)$
Compaction control value (CCV)	None	Sakai	$\text{CCV} = \left[\dfrac{A_{0.5\Omega} + A_{1.5\Omega} + A_{2\Omega} + A_{2.5\Omega} + A_{3\Omega}}{A_{0.5\Omega} + A_{\Omega}}\right] \times 100$
Stiffness (K_b)	MN/m	Ammann/ Case	$K_b = \omega^2\left[m_d + \dfrac{m_o e_o \cos(\phi)}{z_d}\right]$
Vibration modulus (E_{vib})	MN/m^2	Bomag	$\dfrac{\Delta F}{\Delta z_1} = \dfrac{E_{vib}\cdot 2\cdot a\cdot \pi}{2\cdot(1-v^2)\cdot\left(2.14+0.5\ln\left(\dfrac{\pi\cdot(2\cdot a)^2\cdot E_{vib}}{(1-v^2)\cdot 16\cdot(m_b+m_e+m_r)\cdot g\cdot(d/2)}\right)\right)}$

Source: Chang et al. (2011).

efficient and uniform construction process control and QA practice as the rollers map out the stiffness characteristics and quality of compaction for the whole compacted area. Thus, IC provides an effective approach for identifying weak areas in subgrade and base layers that require additional compaction before the placement of surface layers. In addition, over-compaction that can occur during conventional compaction can be prevented by using IC because it reduces the number of roller passes and proof-rolling tests. The compaction information collected using IC rollers provides better assessment of the achieved compaction levels because of the significantly larger depth of influence of ICMVs compared with those obtained using in situ tests such as the NDG, LWD, and GeoGauge, as demonstrated in Figure 88. IC technologies also can be especially beneficial for maintaining consistent rolling patterns under lower visibility conditions, such as night paving operations (Chang et al. 2011).

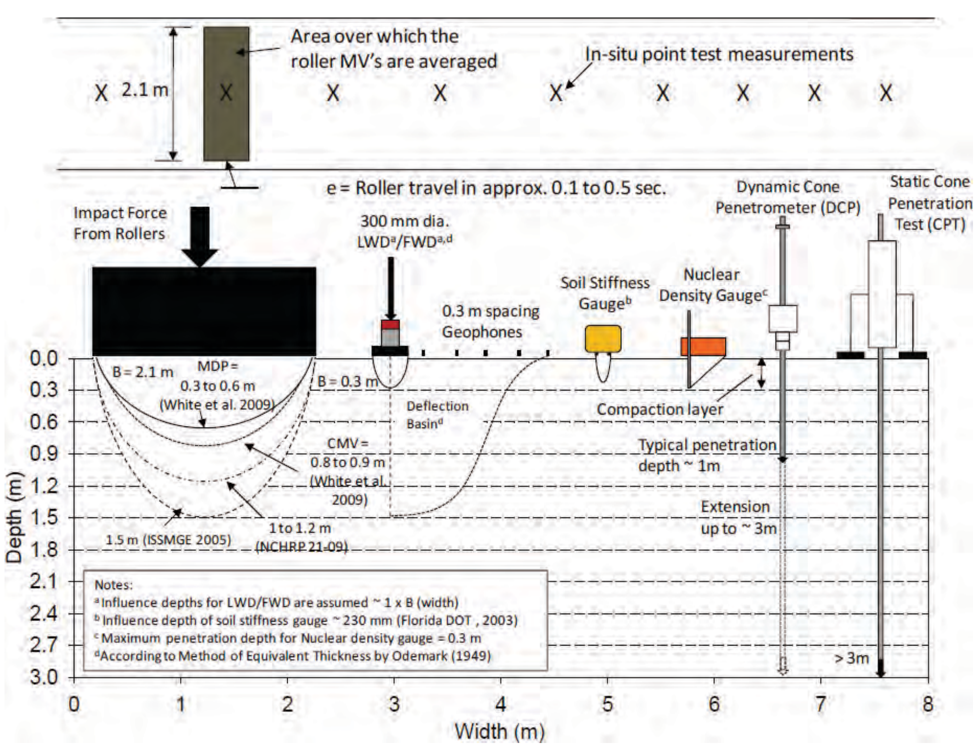


FIGURE 88 Illustration of differences in measurement influence depths obtained by using different testing devices (White et al. 2007a).

However, there are several disadvantages of the IC system. IC rollers are more expensive than ordinary ones. IC system measurements are sensitive to moisture; however, currently they are not capable of recording the moisture content of compacted material. There is also no consistent relationship between the ICMVs of different IC systems owing to their different computation algorithms and definitions. This inconsistency in the ICMV definition along with the lack of comprehensive correlations between IC outputs and conventional tests are the main obstacles for industry standardization and the development of IC acceptance specifications.

Finally, although IC technologies have been implemented in Europe and Japan for many years, they have been introduced to the United States only recently (White et al. 2009a). Therefore, there is still a lack of experience, knowledge, and availability of IC equipment in the United States. These limitations may explain the difficulties in implementing IC technologies by state DOTs and paving contractors.

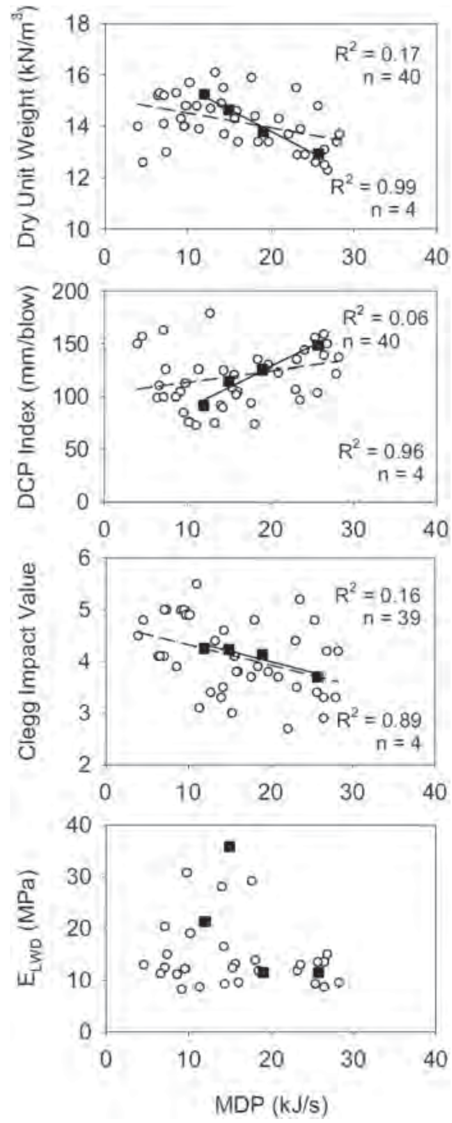


FIGURE 89 MDP correlation with in situ compaction measurements using spatially nearest data pairs (circles) and averaged measurements for given roller pass (squares), kickapoo silt, strip 1 (White et al. 2007a).

Synthesis of Past Research Studies

Oklahoma Study

Mooney et al. (2003) presented the results of a study that involved monitoring the roller drum vibration during compaction of well-graded sand test beds. The DCP test was also conducted and used to assess the mechanical properties of the compacted material. The study's results indicated that the time-domain drum and frame acceleration amplitudes were mildly sensitive to increases in underlying material stiffness properties such that normalized drum acceleration values slightly increased when the DCP penetration index more than doubled. Harmonic content, reported as total harmonic distortion, had more sensitivity to changes in underlying material properties. In addition, it had good correlation with the DCP penetration index, especially when the sublift material was densified and stiffened.

Iowa Studies

White et al. (2007a, 2009b) conducted several studies to evaluate the use of CCC technologies in Iowa. White et al. (2006) conducted a study that used CCC rollers at three project sites. For the first two projects, the rollers had internal sensors to monitor the power consumption used to move them. An onboard computer and display screen and a GPS system were used to compact test sections consisting of different types of cohesive soils. In the third project, rollers equipped with CMV and MDP technologies were used to compact clayey and sandy subgrade soils. NDG, GeoGauge, LWD, CH, and DCP tests were conducted during compaction stages in all three projects. The results of this study indicate that the compaction monitoring technology identified "wet" and "soft" spots incorporated into a test section. In addition, for cohesive soil, good correlations were generally obtained between the average MDP measurements and those of different in situ tests. Figure 89 presents the simple linear correlations obtained in one of the projects. Those correlations were improved by incorporating moisture content as a regression variable. The results of this study also showed that the CMV measurements were weakly correlated with machine power for sand. Finally, White et al. (2007a) concluded that a single in situ test point did not provide confidence in representing the average soil engineering property values over a given area. Therefore, multiple tests should be performed to determine soil properties with any degree of confidence.

White and Thompson (2007b) evaluated the relationship between the CMV as well as the MDP and various in situ

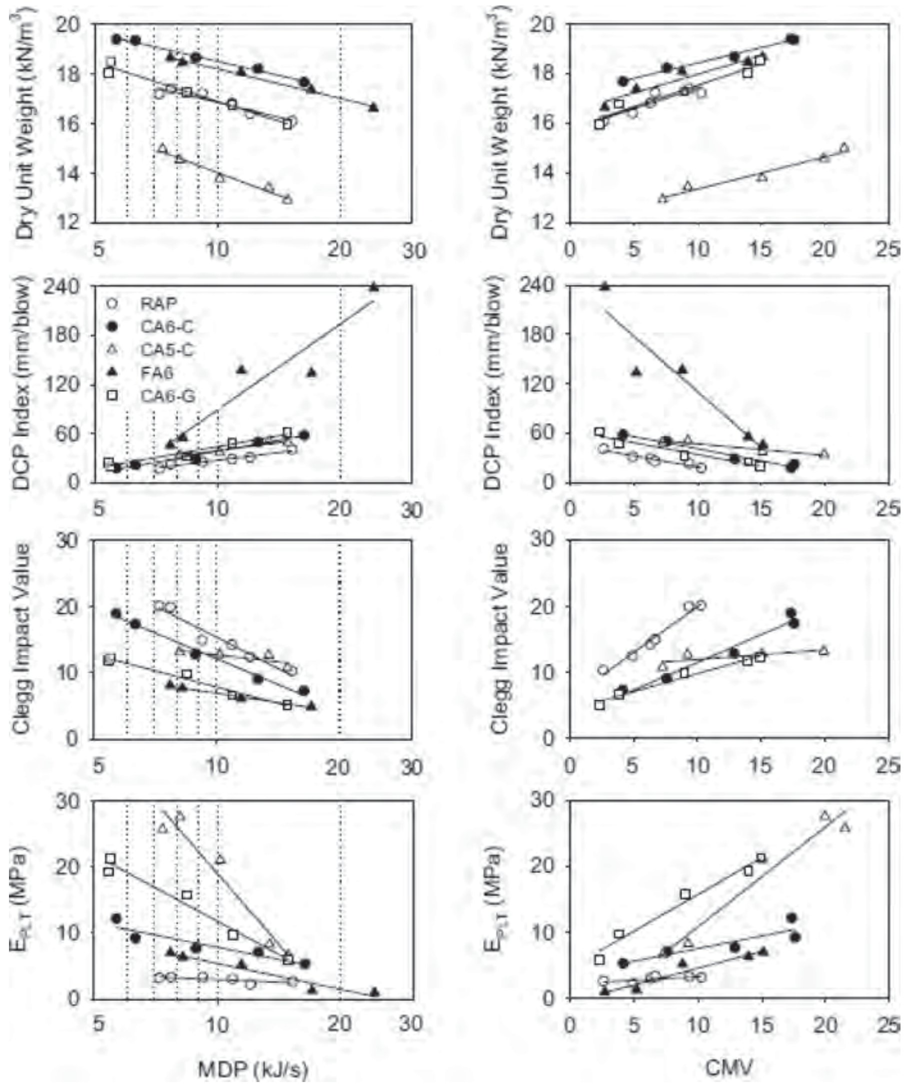


FIGURE 90 Relationships between average in situ and roller integrated compaction measurements (White and Thompson 2007b).

test device measurements by constructing and testing five strips that consisted of different granular materials. Figure 90 shows the relationships between the average in situ test measurements and the CMV and MDP. Although the correlations between CMV and in situ test measurements were linear, the correlations between MDP and these measurements showed a logarithmic relationship.

Minnesota Studies

Petersen and Peterson (2006) documented an IC demonstration project for the Minnesota DOT and the associated field and laboratory testing. The IC vibratory roller equipped with CMV technology was used to compact 914-mm (3-ft) subcut consisting of a select granular borrow material. Results showed that poor correlations between the CMV measurements and the GeoGauge and LWD moduli were obtained when the comparison was done on a point-by-point basis. The authors attributed this to the relative differences in influence depth between CMV measurements and those of the GeoGauge and LWD. However, relatively good correlations were obtained between CMV and DCP measurements for greater depths (200 to 400 mm).

White et al. (2007a, 2007b, 2009a) conducted several studies for the Minnesota DOT to evaluate and implement the use of IC in compaction control of pavement layers and subgrade soils. White et al. (2007a) conducted three field studies at two project sites to investigate the relationships between CCV, MDP, and k_B stiffness measurements obtained using three IC rollers and the dry unit weight, soil strength, and modulus parameters determined from NDG, DCP, and LWD, respectively. The authors found strong correlations between k_B and in situ test results for test sections with a relatively wide range of material stiffness and comparatively poor correlations for sections with more uniform conditions.

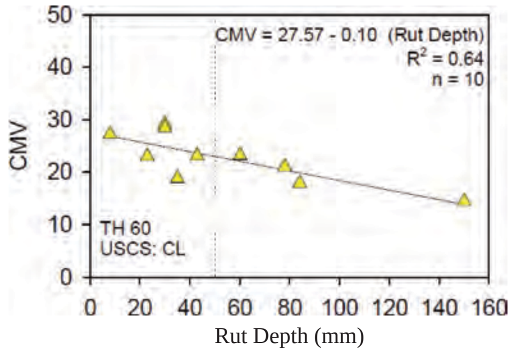


FIGURE 91 CMV compared with rut depth (White et al. 2009a).

Furthermore, at the project scale average, the dry unit weight, and the DPI values had good correlations with CCV with R^2 values of 0.52 and 0.79, respectively. However, poor correlations were found between the LWD moduli and the CCV values. This was attributed to differences in the influence depths of LWD and the IC roller measurements. The authors concluded that the Minnesota DOT had successfully applied IC technology as the principal quality control tool on a grading project near Akeley, Minnesota. The entire project passed the test-rolling acceptance criteria.

In another study, White et al. (2009a) documented the results of IC implementation projects. In this study, proof-rolling rut measurements were compared with various IC roller and in situ test measurements at four different sites in Minnesota. The results of measurements obtained in a project that included granular subgrade materials showed that the LWD modulus and DCP results correlated well with the CMVs when the LWD was performed in a carefully excavated trench 100 to 150 mm (4 to 6 in.) deep, and the DCP first blow was regarded as a seating blow. White et al. (2009a) indicated that the significantly different stress paths for loading under roller and LWD loading were found to be one of the causes of scatter in the relationships between CMV and LWD modulus measurements. As shown in Figure 91, the CMV had a strong correlation with test-rolling rut values. However, poor correlation was reported between dry unit weight and CMVs. CMV data obtained from repeated passes demonstrated that the measurements were repeatable, but they were not reproducible when the amplitude was changed. Therefore, measurements obtained at different amplitudes should be treated separately. In another study, White et al. (2009b) found that the CMV was reproducible with variation in nominal speeds between 3.2 and 4.8 km/h. The authors recommended that CMVs be evaluated in conjunction with roller resonant meter values because roller "jumping" affected CMVs.

Results of multiple calibration test strips and production areas from one project consisting of clayey subgrade soil showed correlations with varying degrees of uncertainty (i.e., R^2 values varied from about 0.3 to 0.8) between the MDP and measurements obtained using the LWD. White et al. (2009a) indicated that MDP values were repeatable with changes in amplitude (from a = 0.85 mm to 1.87 mm) and at a nominal speed of 3.2 km/h. However, MDP values were highly variable when operated at a 6.4 km/h nominal speed. It is worth noting that in another study, White et al. (2009b) indicated that MDP values were affected by the change in amplitude.

NCHRP Project 21-09

NCHRP Project 21-09, "Intelligent Soil Compaction Systems," evaluated several rollers equipped with different types of IC technologies (summary of CCC and IC rollers investigated in NCHRP Project 21-09) and compared their measurements to different in situ and laboratory test results (see Table 18). Simple linear correlations between ICMVs (i.e., MDP, CMV, E_{vib}, k_s, CCV) and NDG, LWD, and PLT modulus were possible for a layer underlain by a relatively homogenous and stiff layer. However, different factors were found to affect those correlations:

- Heterogeneity in underlying layer support conditions
- High moisture content variation
- Narrow range of measurements
- Machine operation setting variation (e.g., amplitude, frequency, speed) and roller "jumping"
- Nonuniform drum/soil contact conditions
- Uncertainty in spatial pairing of point measurements and ICMV
- Intrinsic measurement errors associated with the ICMV and in situ point measurements.

TABLE 18
SUMMARY OF CCC AND IC ROLLERS INVESTIGATED IN NCHRP PROJECT 21-09

Roller Manufacturer	Intelligent Compaction Features		
	Roller-Integrated Measurement	Automatic Feedback Control of:	GPS-Based Documentation
Ammann/Case	Stiffness k_s	Eccentric force, amplitude, and frequency	Yes
Bomag	Stiffness E_{vb}	Vertical eccentric for amplitude	Yes
Caterpillar	MDP, CMV_C	None	Yes
Dynapac US	CMV_D	Eccentric force amplitude	Yes
Volvo	CMV_V	None	No
Sakai America	CCV	None	Yes

Of all the factors cited, heterogeneity in conditions of underlying layers was identified as the major factor affecting the correlations. A multiple regression analysis approach was proposed that included ICMV measurements and in situ test results of underlying layers to improve correlations.

Mooney et al. (2010) concluded that modulus measurements generally captured the variation in ICMVs better than did traditional dry unit weight measurements. In addition, DCP tests were found effective in detecting deeper, "weak" areas that were commonly identified by ICMVs and not by other in situ tests. Finally, it was concluded that relatively constant machine operation settings were critical for calibration strips (e.g., constant amplitude, frequency, and speed), and correlations generally were better for low-amplitude settings (e.g., 0.7 to 1.1 mm).

SUMMARY

The previous sections provide comprehensive details of various in situ methods that can assess the stiffness or strength of unbound materials and have been used as tools for controlling the quality of their compaction. The CH, DCP, GeoGauge, and LWD have been evaluated by many DOTs. The DCP and LWD are currently the most widely used test devices in field projects for compaction quality control and assurance of base layers, subgrade soils, and embankments. All devices except the PSPA might have difficulties in establishing target field value in the laboratory owing to boundary effects on their measurement accuracy. Therefore, several DOTs have attempted to establish those values based on pilot projects or by constructing control strips along a project. Some devices also have limitations on the type of unbound material they can test. In addition, those devices apply different load magnitudes during the test, resulting in different measurement results. Although the results of in situ stiffness/strength devices were found to be affected by moisture content, none of these devices have the ability to measure it. The devices possess different influence depth values. Thus, careful consideration should be given when analyzing their results and using them for compaction control. Several correlations were developed between the in situ test devices' measurements and design input parameters, such as the resilient modulus and CBR. However, those correlations are to be used with caution because they can be applied to only certain types of unbound materials and were developed for very specific configurations of these devices. In general, no strong correlation was found between in situ stiffness/strength measurements and in-place density because their relationship continuously changes depending on the moisture content. Almost all of the research and implementation conducted by the FHWA and state DOTs focusing on the use of CCC and IC have reported considerable success. However, those agencies are still hesitant to widely use these techniques in the field primarily because of the lack of available industry standardization as well as standards and acceptance specifications. Implementation projects in different states can help to address such issues and incorporate CCC and IC into DOT practice.

CHAPTER FIVE

STIFFNESS-BASED SPECIFICATIONS FOR COMPACTION CONTROL OF UNBOUND MATERIALS

INTRODUCTION

Although current common quality control specifications that use dry density and/or moisture content criteria are relatively straightforward and practical, they do not reflect the engineering properties of unbound materials necessary to ensure a high-quality product. In addition, the design of pavements and embankments is based on stiffness and/or strength parameters. Thus, there is a missing link between the design process and construction quality control practices of unbound materials. To address this problem, several studies have been conducted by state DOTs to develop quality control procedures for construction based on a criterion that closely correlates to the stiffness/strength parameters used in the design process. This chapter presents a review of stiffness/strength-based specifications that have been developed in the United States and Europe. In addition, it summarizes the experience of state DOTs that have implemented these specifications in field projects. The information in this chapter is derived from published documents identified in the literature review as well as interviews with selected state DOTs that were conducted in this study.

REVIEW OF CURRENT STIFFNESS/STRENGTH-BASED COMPACTION CONTROL SPECIFICATIONS

Currently, only a few state DOTs have developed compaction control specifications for unbound materials that are based on in situ stiffness/strength measurements. Table 19 provides the links for these specifications. Although some DOTs have developed such specifications primarily for subgrade soils or select types of base course materials, other states have developed comprehensive specifications that cover a wide range of unbound materials. A summary of the stiffness- and strength-based compaction control specifications that have been developed in the United States and Europe is presented in the following sections.

Minnesota DOT Specifications

The Minnesota DOT has been one of the leading state DOTs in developing and implementing stiffness-based specifications for compaction control of unbound materials. Currently, the Minnesota DOT has specifications for the using the DCP and the LWD in assessing the quality of compacted layers of unbound materials during construction. The following sections discuss these specifications.

DCP Specification

The DCP has been used by the Minnesota DOT as a compaction quality control device for pavement edge drain trenches since 1993. In 1999, the agency implemented a new specification using the DCP penetration index (DPI) for compaction acceptance of three types of base course aggregate materials, specifically classes 5, 6, and 7. In this specification, the DPI value is recorded in millimeters (or inches) per blow and is the only parameter used to determine the compaction effort needed to reach the target CBR value for base aggregates. This specification was later modified to expand its applications to other granular materials by taking into account the effect of gradation and moisture content.

The current Minnesota DOT DCP specification is used for compaction control of three types of unbound granular materials: base aggregates, granular subgrade materials, and edge drain trench filter aggregates. Although procedures for the granular materials are similar, a slightly modified procedure is specified for edge drain trench filter aggregate. For assessing compaction quality of base aggregates and granular subgrade materials, Minnesota DOT currently requires determining the gradation and in situ moisture content of the compacted material as well as performing DCP tests on it. Readings for the first five DCP drops need to be recorded. The first two drops are treated as seating drops and are used to compute the SEAT value using Eq. 62. The penetration depth after the fifth drop is used to compute the DPI value using Eq. 63. The measured SEAT and DPI values are then compared with the maximum allowable SEAT and DPI values. These should be selected from Table 20 or by using Eq. 64, based on the tested material's gradation properties [represented by the grading number (GN)] and recorded moisture content. The compacted material passes the test only if the measured SEAT and DPI values are found to be less than or equal to the maximum allowable values. However, Minnesota DOT also requires that the penetration of the five DCP drops be smaller than the tested layer thickness.

$$\text{SEAT} = \text{Reading after seating}\,(2\text{ blows}) - \text{Initial Reading} \tag{62}$$

$$\text{DPI} = \frac{[\text{Reading after 5 blows} - \text{Reading after 2 blows}]}{3} \tag{63}$$

TABLE 19
LINKS FOR STIFFNESS- AND STRENGTH-BASED COMPACTION CONTROL SPECIFICATIONS

State DOT	Specification Links
Minnesota	DCP specification: http://www.dot.state.mn.us/materials/gbmodpi.html LWD specification: http://www.dot.state.mn.us/materials/gblwd.html
Indiana	DCP specification: http://www.state.in.us/indot/files/Fieldtesting.pdf LWD specification: http://www.in.gov/indot/div/mt/itm/pubs/508_testing.pdf
Missouri	http://www.modot.org/business/standards_and_specs/Sec0304.pdf
Illinois	http://www.dot.il.gov/bridges/pdf/S-33%20Class%20Reference%20Guide.pdf

$$\text{Maximum Allowable DPI}\,(\text{mm/blow}) = 4.76 \times \text{GN} + 1.68\text{MC} - 14.4 \qquad (64)$$

where

MC = the moisture content at the time of testing, and
GN = grading number obtained using the following equation:

$$\text{GN} = \frac{25\text{ mm} + 19\text{ mm} + 9.5\text{ mm} + 4.75\text{ mm} + 2.00\text{ mm} + 425\,\mu\text{m} + 75\,\mu\text{m}}{100} \qquad (65)$$

The SEAT value requirement initially was included in the Minnesota DOT specification to ensure that the aggregate base layer had sufficient surface strength to allow construction equipment, such as a paver, to operate on its surface without significant rutting. However, this requirement was maintained for other materials to make certain that thin, loose, or irregular surface material does not unduly affect measurements. Minnesota DOT's specification uses the grading number (GN) to represent the gradation properties of the tested granular base and subgrade materials. The GN is computed using Eq. 65, which requires performing a sieve analysis with seven of the most common sieves: 25, 19, 9.5, 4.75, and 2.00 mm and 425 and 75 µm. The GN increases with the increase of fine sand particles in the material. A material with an extremely fine gradation will yield a GN close to 7, whereas an extremely coarse material yields a GN close to zero. Materials with larger amounts of gravel and coarse sand

TABLE 20
MODIFIED DCP PENETRATION REQUIREMENTS

GN	In Situ Moisture (% by dry weight)	Maximum Allowable Seating (mm)	Maximum Allowable DPI (mm/blow)
3.1–3.5	<4.0	40	10
	4.1–6.0	40	10
	6.1–8.0	40	13
	8.1–10.0	40	16
3.6–4.0	<4.0	40	10
	4.1–6.0	40	12
	6.1–8.0	45	16
	8.1–10.0	55	19
4.1–4.5	<4.0	45	11
	4.1–6.0	55	15
	6.1–8.0	65	18
	8.1–10.0	70	21
4.6–5.0	<4.0	65	14
	4.1–6.0	75	17
	6.1–8.0	80	20
	8.1–10.0	90	24
5.1–5.5	<6.0	90	19
	6.1–8.0	100	23
	8.1–10.0	110	26
	10.1–12.0	115	29
5.6–6.0	<6.0	110	22
	6.1–8.0	120	25
	8.1–10.0	125	28
	10.1–12.0	135	32

Source: Minnesota DOT (2012).

TABLE 21
REQUIRED CONTROL STRIP DIMENSIONS

Specification	Material/Location	Length	Width	Thickness	Number of Lifts
2211, 2215	Base	≥100 m (300 ft)	Layer	Layer	1
2105, 2106	Roadbed embankment soil (excavation and borrow)		Excavated embankment width	Planned layer thickness, but not exceeding a maximum thickness of 4 ft (1.2 m)	1 every 304.8 mm (1 ft)
	Miscellaneous, trench, culvert or other tapered construction Embankment soil granular bridge approach treatments and other embankment adj. to structures (excavation and borrow)	≥100 m (300 ft)			1 every 608.6 mm (2 ft)

Source: Minnesota DOT (2012).

have small GN values. Such materials have larger strength and modulus values than do those with large amounts of fine sand. Therefore, strength and modulus values calculated from the DCP are expected to increase as the GN decreases. The DCP specification was modified to include GN as an input to ensure that a material with a larger GN remains acceptable even if it has lower modulus/strength values. This was done to remain consistent with Minnesota DOT's preexisting construction standards.

The Minnesota DOT provided a description of the standard DCP device that should be used in this specification. However, there are a few configuration options available for the DCP, including change of hammer mass, type of tip, and recording method. The standard hammer mass is 8 kg, but a 4.6-kg alternative can be used. For pavement applications, the 8-kg mass is used because of the highly compacted soil. The DCP tip can be a replaceable point or a disposable cone. The replaceable point stays on the DCP for an extended period—until damaged or worn beyond a defined tolerance—and is then replaced. The disposable cone remains in the soil after every test, making it easier to remove the DCP; however, a new disposable cone must be placed on the DCP before the next test. Manual or automated methods are available to gather penetration measurements. The reference ruler can be attached or unattached to the DCP.

LWD Specifications

Minnesota DOT has also developed specifications for using the LWD in compaction QC/QA of unbound materials. This specification provides two methods for developing acceptance criteria for compacted unbound materials: the control strip method and the comparison test method. In the first method, a control strip meeting the requirements provided in Table 21 is constructed within the roadway core to determine the LWD target deflection value (LWD-TV). This is done by conducting LWD tests during the construction of the control strip at the rates shown in Table 22. The average deflection values of all LWD tests are plotted against the roller pass count to create a compaction curve. The LWD-TV for each lift is the minimum deflection of the compaction curve. Once the LWD-TV is determined, the LWD test is to be performed on the compacted layer at the rates specified in Table 23. The compacted layer is accepted only if the average deflection value of all performed LWD tests is less than 1.1 times the LWD-TV.

For the comparison test method, six sets of tests comparing the LWD deflections with either the DPI (for granular and base) or the Minnesota DOT-specified density method (for nongranular) must be conducted. Each set should be spaced 304.8 mm (1 ft) longitudinally from the next, and spaced at

TABLE 22
LWD TESTING RATES DURING CONTROL STRIP CONSTRUCTION

Specification	Material	Form Number	Minimum Required Agency (Field Test Rate)
2211, 2215	Base	G and B-601	4 tests/roller pass/lift
2105, 2106	Roadbed embankment soil (excavation and borrow)		
2105, 2106	Miscellaneous, trench, culvert or other tapered construction Granular bridge approach treatments and other embankment adj. to structures (excavation and borrow)	G and B-602	2 tests/roller pass/lift

Source: Minnesota DOT (2012).

TABLE 23
LWD TESTING RATES (CONTROL STRIP METHOD)

Specification	Material	Form Number	Minimum Required Agency (Field Test Rate)
2211	Base	GandB-604	1 LWD test/500 yd^3 (CV) 1 LWD test/400 m^3 (CV)
2215	Base		1 LWD test/3,000 yd^2 1 LWD test/2,000 m^2
2105, 2106	Roadbed embankment soil (excavation and borrow)		1 LWD test/2,000 yd^3 (CV) 1 LWD test/1,500 m^3
	Miscellaneous, trench, culvert or other tapered construction embankment soil (excavation and borrow)		1 per 2-ft thickness/250 ft 1 per 600-mm thickness/75 m
	Granular bridge approach treatments and other embankment adj. to structures (see Note 1)		1 per 2-ft thickness 1 per 600-mm thickness

any width throughout the first 3,058.2 m^3 (4,000 yd^3). The LWD-TV can be determined by using the maximum deflection measurement where DPI or density values are passing. This should continue until there are six passing comparison tests in locations that are close to failure. The LWD-TV should then be validated by conducting 10 additional LWD tests and performing three additional sets of comparison tests. The compacted layer is accepted only if the deflection value of the LWD test is less than the determined LWD-TV.

Based on the results of several research projects, Minnesota DOT developed LWD-TVs for different types of unbound materials. Table 24 provides a summary of LWD-TVs that were recommended for granular materials. These should be selected based on their grading number and moisture content. For compacted, fine-grained soil, Minnesota DOT uses the plastic limit and field moisture content to determine the LWD-TVs, as shown in Table 25. It is worth noting that Minnesota DOT requires that the moisture content of embankment materials should be maintained from 65% to 95% of the target moisture content.

Because the properties of LWD devices might vary, Minnesota DOT requires that the LWD used in testing have a plate diameter of 200 mm (7.9 in.) and a falling mass of 10 kg (22.1 lb), with a height of fall of about 500 mm (19.7 in.). Minnesota DOT specifies that the force applied by the LWD should be of 6.28 kN (1,539.9 lbf), which results in a stress of 0.2 MPa (29 psi) for the 200-mm (7.9-in.) diameter plate. Those requirements allow for obtaining consistent and reliable

TABLE 24
LWD TARGET VALUES FOR GRANULAR MATERIAL

Grading Number (GN)	Moisture Content (%)	Target LWD Modulus		Target LWD Deflection Zorn (mm)
		Keros/Dynatest (MPa)	Zorn (MPa)	
3.1–3.5	5–7	120	80	0.38
	100	100	67	0.45
	75	75	50	0.6
3.6–4.0	5–7	120	80	0.38
	80	80	53	0.56
	63	63	42	0.71
4.1–4.5	5–7	92	62	0.49
	71	71	47	0.64
	57	57	38	0.79
4.6–5.0	5–7	80	53	0.56
	63	63	42	0.71
	52	52	35	0.86
5.1–5.5	5–7	71	47	0.64
	57	57	38	0.79
	48	48	32	0.94
5.6–6.0	5–7	63	42	0.71
	50	50	33	0.9
	43	43	29	1.05

Source: Siekmeier et al. (2009).

TABLE 25
TARGET LWD DEFLECTION VALUES FOR FINE-GRAINED SOILS

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

Source: Siekmeier et al. (2009).

LWD data. They also ensure that the LWD influence depth extends to the bottom of a common lift thickness.

Minnesota DOT also provides guidelines for conducting LWD tests. First, tests should be conducted immediately after compaction. Second, before the LWD test is performed, the surface of the tested layer should be leveled by removing any loose or rutted surface material to a depth of 50 to 100 mm (1.97 to 3.94 in.). Third, three LWD seating drops should be performed before the data are collected at any test location. This ensures that any permanent deformation of the surface material will not affect the LWD measurements.

Once the LWD has been seated, three additional LWD drops should be performed at the same test location. The average of the maximum deflections or modulus values resulting from these drops is used as the mean value at the test location. As the modulus or deflection values change slightly during the three measurement drops, Minnesota DOT requires that this change does not exceed 10% of original modulus or deflection value. Otherwise, the material has to be additionally compacted before more LWD tests are conducted. Minnesota DOT also recommends that the LWD maximum deflections recorded during the three measurement drops range between 0.3 and 3.0 mm (0.012 and 0.12 in.). In addition, LWD tests are prohibited when the water table is less than 600 mm (23.6 in.) or when the embankment thickness is less than 300 mm (when no site preparation is needed) and 460 mm (when site preparation is needed). Finally, LWD devices are not to be used when the temperature falls below 5°C (41°F) to ensure that the device's components, particularly the rubber buffers, work as intended. There is no practical upper limit on the temperature.

Minnesota Experience

In a follow-up interview, representatives of the Minnesota DOT stated that the DCP is used as the default device for compaction control of granular materials and is currently used in about 50% of projects involving such materials. The LWD has been used to a much lesser extent by some districts mainly for granular materials but also for projects involving nongranular materials. The selection of compaction control device depends on the district's experience with the different devices as well as the particular application. The DCP is not preferred in projects with large aggregate particles because of the high DCP refusal rate. At the same time, the LWD has posed problems in some districts because it is heavy, less portable, and needs annual calibration. The Minnesota DOT staff stated that the department had good experience with DCP and is planning to extend its use to other applications. However, the DOT had mixed success with the LWD.

Indiana DOT Specifications

The Indiana DOT has recently developed and implemented comprehensive specifications for compaction control of various unbound materials based on the results of DCP and LWD tests. In those specifications, the Indiana DOT uses the DCP for clay, silty, or sandy soils, and granular soils with aggregate sizes smaller than 19 mm (¾ in.), and structural backfill sizes 1 in., ½ in., and Nos. 4 and 30. The LWD test is implemented to assess granular soils with aggregate sizes greater than 19 mm (¾ in.), coarse aggregate sizes Nos. 43, 53, and 73, and structural backfill sizes 2 and 1.5 in. Finally, the Indiana DOT allows using either the DCP or LWD for chemically modified soils.

DCP Specifications

The DCP acceptance criterion is based on the type of soil being tested. For clayey, silty, and sandy soils, the DCP acceptance criteria is determined using Figure 92 based on the maximum dry density and optimum moisture content of the soil of interest. These values are estimated in the field using the Indiana DOT's modified version of the one-point Proctor test procedure (AASHTO T272). Indiana DOT specifies that the strength is measured after completion of the compaction for each 152.4 mm (6 in.) of the clayey soils, and for each 304.8 (12 in.) of silty and sandy soils. Therefore, the acceptance criterion for the DCP, determined from Figure 92, represents the minimum required number of DCP blows for a penetration of 152.4 mm (6 in.) for clay soils and a penetration of 304.8 (12 in.) for silty and sandy soils.

For granular materials—defined by the Indiana DOT as noncohesive soils with 35% or less passing sieve No. 200—the DCP test should be performed after the completion of compaction for each 457.2 mm (18 in.) lift of material. The DCP should first be penetrated into the material a depth of 152.4 mm (6 in.). The number of DCP blows is measured for the penetration between 152.4 and 457.2 mm (6 and 18 in.) into the granular material. This should be compared with the minimum required DCP blow count determined using Eq. 66 or Table 26, based on the optimum moisture content of the tested granular material. It is worth noting that the optimum moisture content is used as an index of the amount of fine particles contained in the tested granular material.

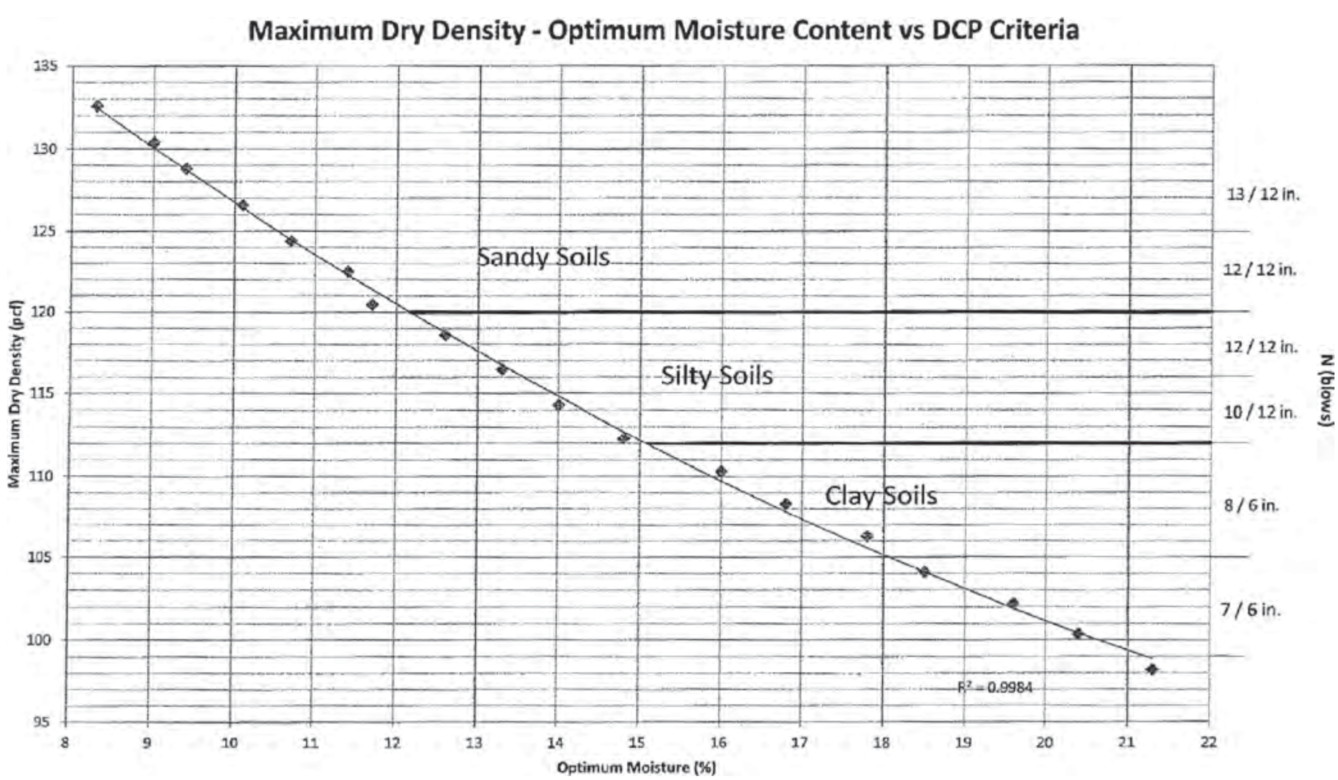


FIGURE 92 DCP criteria for different types of soils (Indiana DOT 2013).

TABLE 26
(NDCP)req|0~12 in. FOR TYPICAL OPTIMUM MOISTURE CONTENT VALUES

Optimum Moisture Content (wc_{opt})	NDCP | 0~12 in.
10	18
11	16
12	14
13	13
14	11

Source: Indiana DOT (2013).

$$(\text{NDCP})\text{req}|0\sim 12\text{ in.} = 59\exp(-0.12wc_{opt}) \qquad (66)$$

where

wc_{opt} = the optimum moisture content, and
(NDCP)req|0~12 in. = the minimum required blow count for 0- to 12-in. penetration, which implies an RC of 95% with high probability. The minimum required blow count should be rounded up to the nearest integer.

The Indiana DOT specifies that the moisture content of compacted clayey, silty, sandy, and granular soils should be maintained within −3% and +1% of the optimum moisture content. In addition, one moisture content test is required for each day that density or strength measurements are taken. The sample for moisture content is required to be representative of the entire depth of the compaction lift being tested.

LWD Specification

For controlling the compaction of aggregate materials and chemically modified soils, the Indiana DOT specifies performing LWD tests at 609.6 mm (2 ft) from each edge of the construction area and at the midpoint of the site width. To accept the compacted layer, the average value of the maximum deflection obtained in the three LWD tests is to be equal to or less than the maximum allowable deflection determined from a test section. In addition, the Indiana DOT requires that the moisture content of the aggregate be within –6% of the optimum moisture content. One moisture content test of the compacted aggregates is required per day. In addition, the Indiana DOT specifies that compaction control testing be done for each 800 tons of compacted aggregate and for each 1,400 yd^3 of chemically modified soil.

The agency provides two options for determining the maximum allowable LWD deflection using a test section. In both options, the test section should have an area approximately 30 m (100 ft) long by the width of the layer to be constructed. In addition, the test section is to be constructed with the available equipment of the contractor to determine the roller type, pattern, and number of passes for the maximum allowable deflection. In the first option, only LWD tests are conducted on the test section. In this case, the roller is operated in the vibratory mode, and 10 random LWD tests should be conducted at the approximate locations, as shown in Figure 93, after the completion of four and five roller passes on the test section. If the difference between the average values of the maximum deflections of the LWD tests conducted after the fourth and fifth pass is equal to or less than 0.01 mm, the compaction will be considered to have peaked and the average of the 10 LWD values after the fifth pass will be used as the maximum allowable LWD deflection. However, if the difference between the average deflection values of LWD tests is greater than 0.01 mm, an additional roller pass in the vibratory mode is applied and 10 LWD tests performed at the same locations. This procedure is continued until the difference of the average maximum deflection values of the 10 LWD tests between consecutive roller passes is equal to or less than 0.01 mm. The maximum allowable deflection will be the lowest average maximum deflection value of the last conducted LWD tests. The Indiana DOT also provides another option for determining the maximum allowable LWD deflection value. With this option, LWD and nuclear density gauge tests are concurrently conducted on the test section. The maximum allowable deflection will be the average deflection value of the 10 LWD test values conducted once the compacted test section has met Indiana DOT density requirements. In a follow-up interview, the Indiana DOT staff stated that using control strips in their current specification for LWD had made the contractor more comfortable when using this new device.

The Indiana DOT requires that the LWD used in compaction control tests have a metal loading plate with a diameter

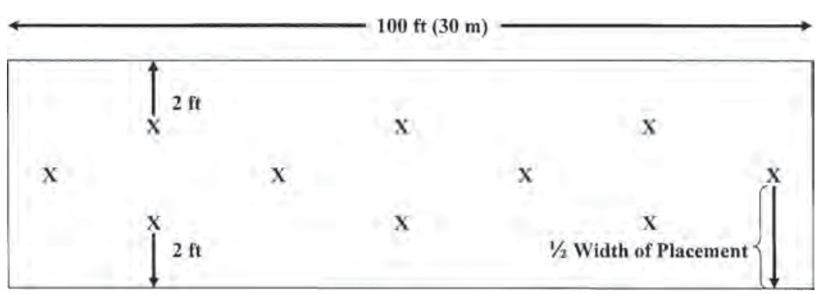


FIGURE 93 Test location for LWD tests (Indiana DOT 2013).

of 300 mm (11.8 in.), an accelerometer attached to the center of the loading plate for measuring the maximum vertical deflection, and falling mass of 10 kg (22 lb). In addition, the agency specifies that the maximum force applied by the LWD should be 7.07 kN (1,589.4 lbf).

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

Indiana Experience

Indiana DOT staff stated in a follow-up interview that the DCP and LWD specifications are currently being used for compaction control of unbound materials in at least 80% of their construction projects. Indiana DOT has found that stiffness- and strength-based compaction control specifications allow for the linking of construction and design processes. This has helped to provide more accurate design input values. For example, for lime-treated soils the modulus design input value was increased by 25% based on the results of a DCP test conducted during and after construction of lime-treated soil layers. The thickness of lime-treated layers was also reduced from 16 to 14 in., which resulted in huge savings. So far, Indiana DOT staff stated, the DOT has had success with the DCP and LWD for most projects and will continue using them. Indiana DOT will be selling 90% of its nuclear density devices this year. According to the Indiana DOT website, the projected annual savings for using the DCP, rather than nuclear testing equipment, as a tool for soil compaction quality control is $480,000 annually.

Indiana DOT noted that the main limitation in the implementation LWD specifications was the difficulty of using the LWD in confined areas and small projects for which control strips cannot be constructed. In addition, the DCP posed problems for use with sand. However, the agency is working on solving these issues.

Missouri DOT DCP Specification

The Missouri DOT has recently implemented a compaction control specification for Type 7 aggregate base material under both roadways and shoulders that is based on DCP test results. In this specification, Missouri DOT requires that Type 7 aggregate base material be compacted to achieve an average DPI value through the base lift thickness less than or equal to 0.4 in. (10 mm) per blow. DCP tests should be conducted within 24 h of placement and final compaction, and the device used have an 8-kg (17.6-lb) hammer and meet the requirements of ASTM D6951. DPI values are determined using Eq. 63, which was proposed by the Minnesota DOT. According to Missouri's specification, the moisture content of the Type 7 base material is not to be less than 5% during compaction.

In a follow-up interview, the Missouri DOT staff stated that the main reason for developing this specification was that the Proctor test cannot be conducted on the Type 7 base because it contains large amounts of aggregates with sizes greater than 19 mm (¾ in.). In addition, Missouri DOT staff indicated that the 0.4 in./blow DCP limiting criterion corresponds to a CBR value of 10.

Illinois DOT DCP Specifications

The Illinois DOT uses the DCP to assess the stability of subgrade soil before and during construction activities. The DCP test is conducted to ensure that the subgrade provides adequate support for the placement and compaction of pavement layers and will not develop excessive rutting and shoving during and after construction. The Illinois DOT acceptance criterion for subgrade is based on the immediate bearing value (IBV), which is computed from the DPI using Eq. 67. Illinois DOT requires a minimum subgrade IBV of 6% to 8% for construction activities. For values less than 6%, subgrade treatment before construction is required. The IBV values from DCP testing are also used as subgrade inputs in most Illinois DOT flexible pavement design procedures for local roads.

$$IBV = 10^{[0.84-1.26\mathrm{XLOG(PR)}]} \tag{67}$$

Iowa DOT Quality Management-Earthwork Specifications

Owing to concerns about the quality compaction of roadway embankments that caused premature high roughness pavements in the state of Iowa, a study was initiated by the Iowa DOT to address such problems and improve construction practices for roadway embankments. The study consisted of a four-phase research project conducted from 1997 to 2007. Based on the findings of the study's first phase, it was concluded that current Iowa DOT specifications failed to consistently produce quality embankments. The compaction quality was monitored using standard Proctor testing, which was found to be inadequate because it tends to overestimate the degree of compaction. Subsequent phases developed and evaluated a new specification for quality management-earthwork (QM-E).

TABLE 27
REQUIREMENTS FOR MEAN DCP INDEX INDICATING STABILITY

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

Source: White et al. (2007a).

The QM-E specification includes five key evaluation criteria for compaction control of soil embankment layers: moisture content, density, lift thickness, stability, and uniformity. The tests to assess the moisture content, density, and lift thickness are required for every 500 m^3 of fill. The stability and uniformity criteria of the compacted materials are evaluated using DCP tests, which are conducted to a depth of 1 m (39.4 in.) for every 1,000 m^3 (1,308 yd^3) of compacted fill. To ensure stability, Iowa DOT specifies that the four-point moving average of the mean DPI of the compacted soil does not exceed the maximum value provided in Table 27 for the type and grade of borrow material. In addition, for uniformity of compaction, the mean change in the DPI should be less than the maximum values shown in Table 28.

As part of the QM-E provisional specification, control strips are to be constructed to establish proper rolling patterns and lift thickness. Four random locations within each test area are tested for thickness, moisture content, density, mean DPI, and mean change in DPI of the compacted lift. Density, mean DPI, and mean change in DPI are recorded as a function of roller passes. The number of roller passes and the lift thickness are adjusted until the minimum test criteria are met.

Europe's LWD Specification

In 2008, the European Union developed a standard for using the LWD in evaluating compacted layers of unbound materials (CEN ICS 93.020: Measuring Method for Dynamic Compactness & Bearing Capacity with Small-Plate Light Falling Weight Deflectometer). This standard specifies performing LWD tests in accordance with the technical requirements and specifications shown in Table 29. The test process involves using the LWD with a loading plate diameter of 163 mm, a falling weight of 10 kg (22 lb), and a drop height of 720 mm (28.3 in.), generating a load of 7 kN (1,589.4 lbf) for testing unbound materials. The LWD is used to obtain two main parameters: the dynamic modulus and the dynamic compactness rate. Although the dynamic modulus is used to assess the bearing capacity of the tested unbound material, the dynamic compactness rate is used to evaluate the quality of compaction. The testing process to obtain those two parameters involves performing six sequences consisting of three LWD drops (for a total of 18 drops) on the loose, noncompacted material at the site. From the second measuring sequence, the average deformation of the three LWD drops is used to determine the initial dynamic modulus. From the sixth measuring sequence, the final modulus is obtained. In both cases, the dynamic modulus is computed based on the elastic Boussinesq method using the following equation:

$$E_d = \frac{c\left(1 - v^2\right) p_{dyn} \times r}{s_{1a}} \tag{68}$$

where

E_d = LWD dynamic modulus;
c = Boussinesq plate multiplier (considering $\pi/2$ rigid plate);
s_{1a} = average vertical travel of the center of the plate, $s_{1a} = \frac{s_{11} + s_{12} + s_{13}}{3}$;
v = Poisson ratio;
r = radius of the loading disc (mm);
p_{dyn} = theoretical pressure applied to, computed using as F_{dyn}/A;
A = loading plate area (mm^2);
F_{dyn} = applied load, $F_{dyn} = \sqrt{2 \cdot m \cdot g \cdot h \cdot K}$;

TABLE 28
REQUIREMENTS FOR MEAN CHANGE IN DCP INDEX INDICATING UNIFORMITY

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

Source: White et al. (2007a).

TABLE 29
INFORMATIVE REQUIREMENTS OF THE LWD TEST

• Mass of the falling weight (including handle)	10.5 ± 0.5 kg
• Total mass of guide rod	
(including the spring consisting of spring elements,	
transportation protection of the falling weight,	
triggering structure and tilting protection)	Maximum 5 ± 0.5 kg
• Dynamic loading	0.35 MPa
• Loading time	18 ± 2 ms
Design requirements of the loading plate:	
• Diameter of the loading plate	163 ± 2 mm
• Thickness of the loading plate	Minimum 20 mm
• Total mass of the loading plate complete masse	
(including measuring cell for the sensor and handles)	15 ± 1.0 kg
Fixed technical data of acceleration gauge applied for deformation measurement:	
• Measurement range of in-built acceleration gauge	0–50 g
In case of applying other strain gauge and the acceleration gauge:	
Measurement time	18 ± 2 ms minimum signals/18 ms
• Processed measurement signal	Minimum 0.01 mm
• Reading accuracy of deformation	Maximum ± 1.5 s per day
• Quartz clock accuracy	
• Reading accuracy of deformation	Minimum 0.01 mm

Source: CEN ICS 93.020.

m = mass of falling weight;
g = acceleration of gravity (9.81 m/s^2);
h = drop height (m); and
K = spring constant (N/m).

According to the CEN ICS 93.020 standard, the work imparted on the material during the six LWD sequences (the 18 LWD drops) is equivalent to that applied in the modified Proctor test (see Table 29). Thus, a compaction curve can be generated based on the average deflection value obtained in each of the six LWD sequences. The relative compactness rate (T_{rE}) at the field moisture content is determined from this compaction curve using the following equation:

$$T_{rE}\% = 100 - \Phi_0 * D_m \qquad (69)$$

where

Φ_0 = a linear coefficient of the calculated from the Proctor test results (in general, it is taken to be 0.365 ± 0.025); and
D_m = deformation index, it is calculated from the sum of the elements of the data line formed from the difference of the subsequent deflections up to the drop.

This relative compactness rate (determined based on the soil's moisture content at the site) must be adjusted to the optimal moisture content using Eq. 70. This adjustment is needed to make the relative compactness rate in the field equivalent to the relative compaction ratio determined based on the maximum dry density obtained in a modified Proctor test. The adjusted value is referred to as the dynamic compactness rate, T_{rd}.

$$T_{rd} = T_{rE} \cdot T_{rw} \qquad (70)$$

where

T_{rE} = site relative compaction at a given water content; and
T_{rw} = the moisture correction coefficient to adjust for differences between the measured moisture content and optimum moisture content. This is determined based on the results of a modified Proctor test conducted on samples of the material used in the field using the following equation:

$$T_{rw} = \frac{\rho_{di}}{\rho_{d\max}} \qquad (71)$$

where

$\rho_{d\max}$ = maximum dry density value obtained in the modified Proctor test; and
ρ_{di} = dry density value on compaction curve of the modified Proctor tests corresponding to the in situ moisture content.

United Kingdom Specifications

The United Kingdom specifications (Highway Agency 2009) define four classes of foundation material (base course materials) based on the long-term, in-service foundation surface modulus value (a composite value with contributions from all underlying layers). Table 30 presents the four foundation classes and their corresponding surface moduli. Those modulus values are then used in design. A minimum subgrade modulus of 30 MPa (2.5 CBR) also is specified. If a subgrade modulus is less than 30 MPa, it should be stabilized or treated before being included in the permanent pavement works. For construction quality control, a target mean and minimum sur-

TABLE 30
TOP OF FOUNDATION SURFACE MODULUS REQUIREMENTS

		Surface Modulus (MPa)			
		Class 1	Class 2	Class 3	Class 4
Long-term in-service surface modulus		≥50	≥100	≥200	≥400
Mean foundation surface modulus	Unbound mixture types	40	80	*	*
	Fast-setting mixture types	50	100	300	600
	Slow-setting mixture types	40	80	150	300
Minimum foundation surface modulus	Unbound mixture types	25	50	*	*
	Fast-setting mixture types	25	50	150	300
	Slow-setting mixture types	25	50	75	150

*Unbound materials are unlikely to achieve the requirements for Class 3 and 4.
Source: Highway Agency (2009).

face modulus value is specified for each type of pavement layer, as shown in Table 30. The moving mean of five consecutive in situ foundation surface modulus measurements should be equal to or greater than the target mean foundation surface modulus in the table. In addition, all individual in situ foundation surface moduli should exceed the target minimum foundation surface before construction of the overlying pavement layers. According to UK specifications, the in situ foundation surface modulus should be measured using the standard FWD. The LWD can be used if a correlation between LWD and FWD measurements is developed by conducting both tests at 25 points within the demonstration site.

SYNTHESIS OF PAST AND ONGOING STUDIES IN OTHER STATES

New England Transportation Consortium Study

In a study funded by the New England Transportation Consortium, Steinart et al. (2005) proposed a procedure that used the LWD to assess the compaction of granular base course layers. In this procedure, a target modulus at the optimum moisture content is selected for a base course material from Table 31, based on the required relative compaction value. LWD tests should be conducted on the compacted base course materials to ensure that a target modulus value is achieved. In addition, moisture content is obtained to determine the correction factor based on Table 32. The correction factor needs to be added to the LWD moduli measured in the field to achieve an equivalent LWD modulus at the optimum moisture content.

In this procedure, the authors required using a LWD with the 850-mm (33.5-in.) drop height, 20-kg (44-lb) drop weight, and 300-mm (12-in.) diameter loading plate. In addition, they recommended conducting six LWD drops at each test location, with the first drop discarded and the measurements of the remaining five drops averaged to determine the modulus value at that test location. Three locations should be tested within a 3-m (10-ft) diameter area and the average used to determine a representative value for that particular station.

Louisiana Study

Based on a field study in Louisiana, Abu-Farsakh et al. (2005) recommended using a limiting DCP penetration ratio of 5.5 mm/blow for compacted base course layers. This was the average DCP penetration ratio measured for all base course test sections that had satisfactory FWD and PLT stiffness moduli and acceptable compaction levels. The DPI value of 5.5 mm/blow corresponds to a CBR value of 87%, according to Eq. 72. This value was found to meet the minimum CBR value of 80% specified by the U.S. Army Corps of Engineers for stone bases in airfield flexible pavements.

TABLE 31
TENTATIVE EQUIVALENCES BETWEEN PERCENT COMPACTION AND COMPOSITE MODULUS AT OPTIMUM WATER CONTENT FOR BASE AND SUBBASE COURSE AGGREGATE

Relative Compaction Based on AASHTO T180 (%)	Equivalent LWD Composite Modulus (MPa) at Optimum Water Content
90	92
95	115
98	130
100	139

Source: Steinart et al. (2005).

TABLE 32
FACTOR TO CORRECT COMPOSITE MODULUS MEASURED AT FIELD WATER CONTENT TO EQUIVALENT VALUE AT OPTIMUM WATER CONTENT

Water Content Relative to Optimum		Correction Factor to Be Added to Composite Modulus (MPa) Measured at Field Moisture Content
Dry of OMC	-4%	-31
	-3%	-23
	-2%	-15
	-1%	-8
At OMC		0
Wet of OMC	+1%	8
	+2%	15
	+3%	23
	+4%	31

Source: Steinart et al. (2005).

NCHRP 10-84 Study

Currently, the ongoing NCHRP project 10-84 (2011) is anticipated to develop a modulus-based construction specification for compaction of unbound materials. Based on the results of Phase I of this project, the researchers indicated that the modulus-based compaction control specification that will be developed in this study ideally may follow the flowchart shown in Figure 94. As this figure demonstrates, several interrelated steps should be included in such specification. In the first step, a mechanistic-empirical design procedure such as the *Mechanistic–Empirical Pavement Design Guide* (MEPDG) should be selected and construction specifications tied to the procedure. The second step consists of selecting the most suitable material to ensure a durable layer. In the third step, the selected material for each layer should be tested in the laboratory at the field compaction and moisture conditions to obtain a representative design modulus. The fourth step involves establishing a target modulus value that will be used in the field based on laboratory tests performed in conjunction with determining the representative design modulus. Alternatively, this value is set based on field test strips. In the fifth step, field moduli are measured during construction with an appropriate device to ensure that the target modulus value is achieved. The final step consists of developing statistical control charts to ensure that the modulus and its variability along the project are in control.

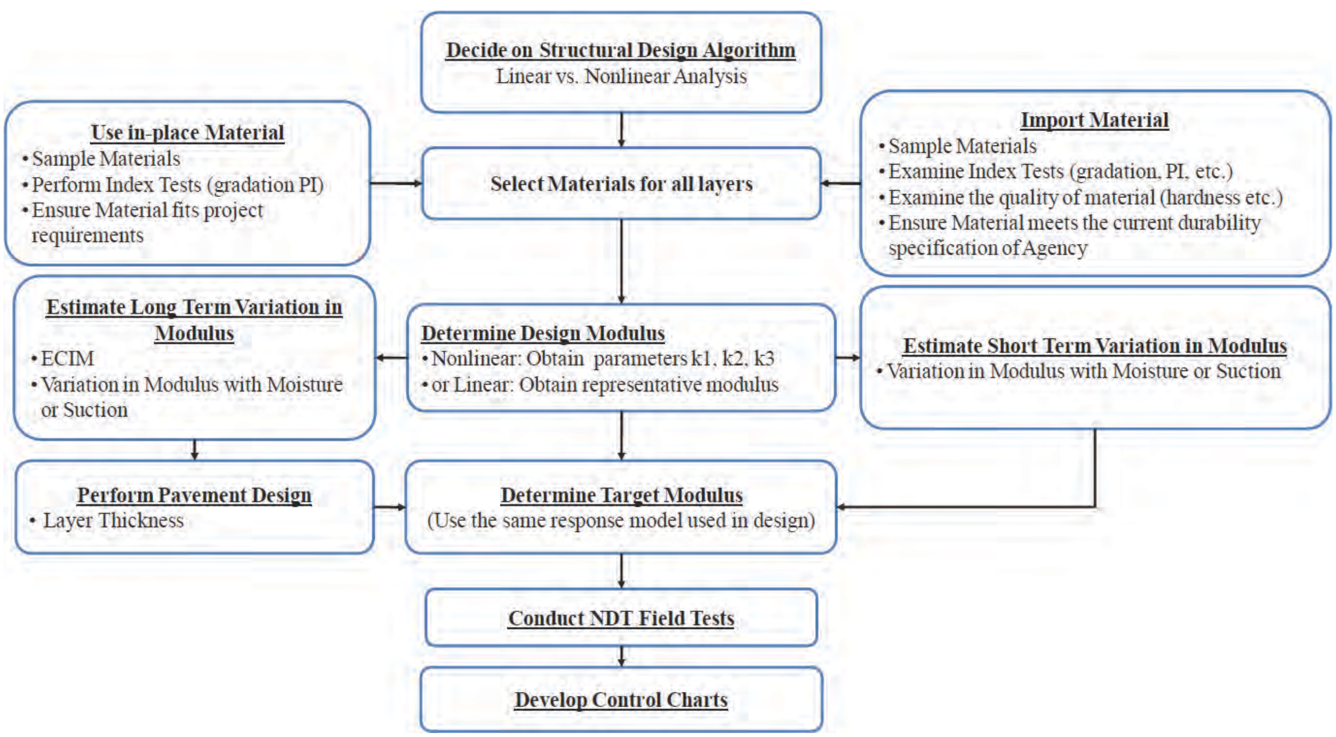


FIGURE 94 An ideal flowchart for modulus-based specifications (NCHRP 10-84, 2011).

REVIEW OF CURRENT INTELLIGENT COMPACTION SPECIFICATIONS

During the past two decades, several European countries have developed specifications for intelligent compaction. Recently, a few state DOTs, including those of Indiana, Minnesota, and Texas, have developed specifications to facilitate the implementation of IC into earthwork construction practices. In addition, the FHWA in July 2011 released generic specifications for compaction of unbound materials using IC technologies. These generic specifications are to be modified by individual agencies to meet specific requirements (see www.intelligent compaction.com). More recently, two approaches for compaction control of unbound materials that included using IC technologies were proposed as part of TRB's second Strategic Highway Research Program (SHRP 2) Renewal Project R07 (Scott et al. 2013). Tables 33 and 34 provide a summary of key elements of the current IC specifications in the United States and other countries, respectively. Two main approaches were followed in these specifications. In the first approach the ICMV measurements obtained in the final pass of the IC roller are used to map the weak areas in the compacted layers. Acceptance of these layers depends on satisfying a minimum density or stiffness target value in these weak areas based on the results of spot in situ tests, such as the NDG, PLT, or LWD. Another approach that has been pursued in the specifications developed by state DOTs and the FHWA involves evaluating the ICMV measurements between successive passes until the target ICMV is achieved in a minimum percentage of the compacted area (typically 80% to 90%). The target ICMV is determined based on calibration tests on control strips selected at the construction site. A review of the most recent IC specifications in the Indiana, Minnesota, and Texas DOTs is provided in the following sections.

Indiana IC Specification

The Indiana DOT recently developed a specification for IC construction. This specification can be used only when the construction area evaluated is equal to or greater than 5,000 ft^2 (500 m^2). In this specification, at least 90% of the construction area is to be mapped with an approved IC roller, and a minimum of 70% of the mapped construction area should have or exceed the target ICMV. Deficient areas that do not meet the ICMV target and are larger than 1,500 ft^2 should be reworked and retested. The reworked areas will be accepted if the ICMVs meet the minimum target ICMV.

The Indiana DOT specifies using the IC roller on selected test sections to establish the target ICMV that corresponds to DCP test results. Test sections are to be approximately 100 ft long and 20 ft wide. Moisture tests at two random locations and DCP tests at four random locations in each test section should be performed. The moisture content should be within −3% to +2% of the optimum moisture content for silt-clay soils and controlled within −6% of the optimum moisture content for granular materials.

Minnesota IC Specification

A pilot specification was implemented by the Minnesota DOT in 2006 during the construction of TH 64 in Akeley, Minnesota, and was later updated in 2010 and 2012. In this specification, all segments of projects in which IC rollers are used should be compacted so that at least 90% of the IC measurements are equal to or exceed 90% of the target ICMV before the next lift is placed. If localized areas have IC measurements less than 80% of the target ICMV, those areas should be recompacted. In addition, the target ICMV should be reevaluated if a significant portion of the project is more than 30% of it. The Minnesota DOT requires that moisture content of compacted material be between 65% and 100% of its optimum moisture content value.

The Minnesota DOT also specifies determining the target ICMV using control strips at least 100 m (300 ft) by 10 m (32 ft) at their base, with a thickness equal to that of the layer to be constructed. One control strip is required for each different type/source of grading material used on the construction site. As shown in Table 35, smooth drum or padfoot vibratory IC rollers weighing at least 25,000 lb can be used.

Texas IC Specification

In 2008, the Texas DOT developed a special specification for quality compaction of subgrade soils and base course layers using IC rollers. A list of Texas DOT-approved IC rollers was released in 2009 (see ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/mpl/ic_rollers.pdf). In this specification, a control strip of at least 500 ft in length and a width equal to that of the material course is compacted using the same IC roller procedures intended for the remainder of the project. The moisture content during compaction must be no less than 1% below the optimum moisture content. Control strip results are used to determine the IC compaction parameters and the level of compaction necessary to achieve the maximum target dry density. It is worth noting that as part of this specification, seismic modulus should be determined at the same locations on respective measurement passes as those used for density measurements. The acceptance of compacted sections is based on density and moisture content measurements, which should be obtained within 24 h of compaction completion. A section is deemed acceptable if a maximum of one of the five density measurements taken falls no more than 3 pcf below the target density value.

SUMMARY

This chapter reviews practices and experiences regarding the implementation of stiffness/strength-based specifications for compaction control of unbound materials in Europe and the United States. Although several states have evaluated the use of in situ device(s) to assess stiffness/strength for compaction control of unbound materials, only the DOTs of Indiana and Minnesota have developed and implemented comprehensive

TABLE 33
SUMMARY OF IC SPECIFICATIONS IN THE UNITED STATES

Agency	Equipment	Field Size	Location Specs	Documentation	Compaction Specs	Speed	Frequency
FHWA (2012)	Vibratory self-propelled single-drum roller	225 ft (75 m) long and 24 ft (8 m) wide		Results from the moisture, strength, and maximum dry density, and optimum moisture content tests; IC roller data (manufacturer, model, type, positioning); analysis of IC roller data for coverage area and uniformity; limit of construction area	At least 90% of the individual construction area shall meet the optimal number of roller passes and 70% of the target ICMV determined from the test sections. Rework and reevaluate if areas do not meet the IC criteria.	Constant speed and frequency throughout a section	
Indiana (2012)	Self-propelled vibratory roller with drum accelerometers and smooth or padfooted drums	Approximately 100 ft long and 20 ft wide	One calibration/ control strip per type of unbound material	Results from the moisture, strength, and maximum dry density, and optimum moisture content tests; IC roller data (manufacturer, model, type, positioning); analysis of IC roller data for coverage area and uniformity; limit of construction area	At least 90% of the construction area shall be mapped with the IC roller. The percentage of the mapped area that equals or exceeds the target ICMV shall be at least 70%. The reworked deficient areas will be accepted if the ICMVs meet a minimum of 100% of the target ICMV.	Moisture and density: one test for each 1,400 yd^3 of each lift.	
Minnesota DOT (2007)	Smooth drum or padfoot vibratory roller (25,000 lb)	300 ft by 32 ft (minimum at base). Maximum 4 ft thick	One calibration/ control strip per type or source of grading material	Compaction, stiffness, moisture, QC activities, and corrective actions (weekly report)	90% of the stiffness measurements must be at 90% of the compaction target value.	Same during calibration and production compaction	
Texas (2008)	Self-propelled IC rollers equipped with a measurement and documentation system	At least 500 ft in length, and width must be equal to the full width of the material course	Uniform layer, free of loose or segregated materials	Roller speed, frequency, amplitude, roller measurement values (RMV); dry density, moisture content and seismic modulus of soil	Accept if one of the five most recent density values is below the target density and the failing test is no more than 3 pcf below the target density. Rework, recompact, and refinish material that fails to meet the criteria.		

After Chang et al. (2010).

TABLE 34
SUMMARY OF INTERNATIONAL IC SPECIFICATIONS

Agency	Equipment	Field Size	Location Specs	Documentation	Compaction Specs	Speed	Frequency
ISSMGE	Roller chosen by experience	100 m by the width of the site	Homogenous, even surface. Track overlap ≤ 10% drum width.	Rolling pattern, sequence of compaction and measuring passes; amplitude, speed, dynamic measuring values, frequency, jump operation, and corresponding locations	Correlation coefficient ≥ 0.7. Minimum value ≥ 95% of Ev1, and mean should be ≥ 105% (or ≥ 100% during jump mode). Dynamic measuring values should be lower than the specified minimum for ≤ 10% of the track. Measured minimum should be ≥ 80% of the specified minimum. Standard deviation (of the mean) must be ≤ 20% in one pass.	Constant 2–6 km/h (± 0.2 km/h)	Constant (± 2 Hz)
Earthworks (Austria)	Vibrating roller compactors with rubber wheels and smooth drums suggested	100 m long by the width of the site	No inhomogeneitie s close to surface (materials or water content). Track overlap ≤ 10% drum width.	Compaction run plan, sequence of compaction and measurement runs, velocity, amplitude, frequency, speed, dynamic measuring values, jump operation, and corresponding locations	Correlation coefficient ≥ 0.7. Minimum value ≥ 95% of Ev1, and median should be ≥ 105% (or ≥ 100% during jump mode). Dynamic measuring values should be lower than the specified minimum for ≤ 10% of the track. Measured minimum should be ≥ 80% of the set minimum. Measured maximum in a run cannot exceed the set maximum (150% of the determined minimum). Standard deviation (of the median) must be ≤ 20% in one pass.	Constant 2–6 km/h (± 0.2 km/h)	Constant (± 2 Hz)
Research Society for Road and Traffic (Germany)	Self-propelled rollers with rubber tire drive are preferred; towed vibratory rollers with towing vehicle are suitable	Each calibration area must cover at least three partial fields ~20 m long	Level and free of puddles. Similar soil type, water content, layer thickness, and bearing capacity of support layers. Track overlap ≤ 10% machine width.	Dynamic measuring value; frequency; speed; jump operation; amplitude; distance; time of measurement; roller type; soil type; water content; layer thickness; date, time, file name, or registration number; weather conditions; position of test tracks and rolling direction; absolute height or application position; local conditions and embankments in marginal areas; machine parameters; and perceived deviations	The correlation coefficient resulting from a regression analysis must be ≥ 0.7. Individual area units (the width of the roller drum) must have a dynamic measuring value within 10% of adjacent area to be suitable for calibration.	Constant	
Vägverket (Sweden)	Vibratory or oscillating single-drum roller; minimum linear load 15–30 kN; roller-mounted compaction meter optional	Thickness of largest layer 0.2–0.6 m	Layer shall be homogenous and nonfrozen. Protective layers < 0.5 m may be compacted with subbase.		Bearing capacity or degree of compaction requirements may be met. Mean of compaction values for two inspection points ≥ 89% for sub-base under road base and for protective layers over 0.5 m thick; mean should be ≥ 90% for road bases. Required mean for two bearing capacity ratios varies depending on layer type.	Constant 2.5–4.0 km/h	

After Chang et al. (2010).

TABLE 35
IN SITU TEST DEVICES EVALUATED BY STATE DOTs

State	Non-Nuclear Density Devices	Devices for In Situ Stiffness/Strength Measurements
Alaska		
Alabama		
Arkansas		
Arizona		
California		
Colorado		CH, DCP, LWD
Connecticut		
Delaware	Delaware	DCP, GeoGauge, LWD
Florida	EDG, MDI	CH, DCP, GeoGauge, LWD, PSPA
Georgia		DCP
Hawaii	EDG	DCP, GeoGauge, LWD
Iowa		DCP, LWD
Idaho	EDG, SDG	GeoGauge
Illinois	MDI	DCP, GeoGauge
Indiana	MDI	CH, DCP, LWD
Kansas		GeoGauge
Kentucky		
Louisiana	EDG	DCP, GeoGauge, LWD
Massachusetts		
Maryland		DCP, GeoGauge, LWD
Maine		DCP, LWD
Michigan		
Minnesota	EDG, MDI, SDG	CH, DCP, GeoGauge, LWD
Missouri	EDG	DCP
Mississippi		DCP, GeoGauge, LWD
Montana		DCP, GeoGauge
North Carolina		DCP, GeoGauge, LWD
North Dakota		DCP, LWD
Nebraska	EDG, MDI, SDG	DCP, GeoGauge, LWD
New Hampshire	EDG	GeoGauge
New Jersey	EDG, MDI	
New Mexico	MDI	CH, DCP, GeoGauge
Nevada		
New York	EDG, SDG	DCP, GeoGauge, LWD
Ohio		
Oklahoma		DCP, LWD
Oregon		
Pennsylvania		GeoGauge
Rhode Island		
South Carolina		GeoGauge
South Dakota		GeoGauge
Tennessee		
Texas	MDI, SDG	CH, DCP, GeoGauge, LWD, PSPA
Utah		DCP
Virginia		DCP, GeoGauge
Vermont	EDG, MDI	
Washington		
Wisconsin		DCP, GeoGauge
West Virginia		GeoGauge
Wyoming		GeoGauge

stiffness- and strength-based compaction control specifications for various types of unbound materials. Both DOTs use the DCP and LWD in their specifications. Other states, such as Missouri, have used the DCP in compaction control but only for a specific type of unbound material. Staff of the Indiana and Minnesota DOTs reported that they have had good experiences with the DCP in compaction control of unbound materials. Although the Minnesota DOT had a mixed experience with the LWD, the staff of the Indiana DOT noted its success with the device in compaction control of unbound materials. The FHWA, along with three state DOTs (Indiana, Minnesota, and Texas), also has developed IC specifications. These specifications include selecting target ICMVs based on acceptable stiffness or density spot-testing measurements obtained on control strips compacted using IC rollers. Acceptance is based on achieving the target ICMV for a minimum percentage of compacted area (range, 80% to 90%). Another type of IC specification that has been adopted by some European countries, including Sweden, consists of using ICMV measurements to identify weak areas at a project site. These areas should be assessed using in situ point tests (e.g., density, plate load, and/or LWD) for acceptance.

CHAPTER SIX

CONCLUSIONS

INTRODUCTION

For this report, all available knowledge and information from a variety of sources on various non-nuclear devices and methods that have been used for compaction control of unbound materials were compiled and summarized. Included were non-nuclear devices that measure density, as well as those that evaluate in situ stiffness- and strength-related properties of unbound materials. Information on the devices' accuracy, repeatability, ease of use, test time, cost, Global Positioning System (GPS) compatibility, calibration, compatibility with various unbound materials, and depth of influence was documented and discussed. In addition, the main advantages, disadvantages, and limitations of the devices were identified. All correlations between the measurements of the considered devices and density, as well as the input parameters for designing transportation and geotechnical structures, were provided. This report reviewed stiffness- and strength-based specifications that have been developed and implemented in the United States and in Europe for compaction control of unbound materials. The main findings and conclusions are provided in the sections that follow. In addition, gaps in knowledge and current practices, along with recommendations for future research to address these gaps, are highlighted.

MAIN FINDINGS

Density-Based Compaction Control Specification

The majority of state departments of transportation (DOTs) use field density and moisture content measurements obtained by the nuclear density gauge (NDG) for compaction control of various types of unbound materials. Furthermore, almost all DOTs use impact compaction methods, such as AASHTO T99 and AASHTO T180, to determine the target density value to be achieved in the field. However, those methods are limited for unbound materials that have 30% or less by mass of their particles with sizes greater than 19 mm (¾ in.). A review of states' construction manuals indicated that there are differences in the relative compaction values required by each DOT for unbound materials in embankments, subgrade soils, and base course layers.

Non-Nuclear Devices for Measuring Density and Moisture Content

The majority of DOTs expressed interest in having non-nuclear density devices that could replace the NDG. As shown in Table 35, several of them have evaluated such devices. Survey results showed that among the DOTs that evaluated non-nuclear density devices, overall satisfaction was so low that none of them recommended the use of such devices. As shown in Table 36, which presents a comparison between the non-nuclear density devices and the NDG based on the results of previous reported studies and the survey conducted in this synthesis, all currently available non-nuclear density devices are more difficult to operate and require longer testing time than does the NDG. In addition, the electrical density gauge (EDG) and moisture density indicator (MDI) were reported to have some limitations when used for testing high-plasticity clay and stiff soils.

There are several non-nuclear devices that have been developed to measure the moisture content; however, limited studies have been conducted to evaluate most of these devices. The speedy moisture tester and field microwave are the most common non-nuclear devices used to measure in situ moisture content of unbound materials. According to the survey conducted in this study, 13 state DOTs recommend their use but four do not. The main limitation of both devices is that they cannot be used for all types of unbound materials. According to previous studies documented in this report, the speedy moisture tester cannot be used for highly plastic clayey soil or coarse-grained granular soil. Furthermore, the field microwave is suitable only for materials consisting of particles smaller than 4.75 mm (0.19 in.).

In Situ Test Devices for Stiffness/Strength Measurements

As shown in Table 37, the dynamic cone penetrometer (DCP), GeoGauge, and light weight deflectometer (LWD) are the devices most thoroughly evaluated by DOTs among all in situ test devices. The DCP and LWD have been implemented by some DOTs in the field for compaction control of unbound materials. Previous studies found the GeoGauge measurement to be very sensitive to the seating procedure and to the stiffness of the top 2 in. of the tested soil layer, which significantly affected its reliability.

Table 38 presents a comparison between the different in situ test devices that have been used for compaction control of unbound materials, based on the information collected in this synthesis. All of the devices are quick and easy to use. Although the devices' measurements were found to be influenced by the moisture content, none of these devices

TABLE 36
COMPARISON BETWEEN DIFFERENT DEVICES FOR IN-PLACE DENSITY MEASUREMENT

Feature	NDG	MDI	EDG	SDG
Test Method	Nuclear	Electrical	Electrical	Electrical
ASTM standard	D2922, D3017	D6780	D7698	None
Measurement	γd, w	γd, w	γd, w	γd, w
Moisture readings	Yes	Yes	Yes	Yes
Calibration of device	Requires calibration	Laboratory testing in Proctor mold	Field calibration using direct measurement of γd, w	Field calibration using direct measurement of γd, w
Portability	Good	Medium	Medium	Good
Durability	Good	Good	Good	Good
Operator skill	Extensive, licensed technician	Moderate	Moderate	Extensive
Ease of use—training	Medium—requires training	Difficult	Difficult	Difficult
Initial cost	About $6,950	$6,000	$9,300	$10,000
Data storage	Yes	Yes	Yes	Yes
Repeatability	Good	Good	Mixed Results	*
Accuracy	Good	Mixed Results	Mixed Results	*
GPS	Yes	No	Yes	Yes
Main limitations	- Contains radioactive materials that can be hazardous - Requires intense regulations - High costs to own and maintain	- Complex and time consuming - Cannot test highly plastic clay	- Complex and time consuming - NDG is required for calibration - Cannot test highly plastic clay	- Extensive operator training

*Not enough data were reported.

has the ability to measure it. The Briaud compaction device (BCD), DCP, LWD, and soil compaction supervisor (SCS) may not be suitable for very soft, fine-grained soils. According to ASTM D6951, the DCP is also limited to use with materials that have a maximum particle size smaller than 50 mm (2 in.). The influence depth differs among the various in situ devices. Some devices, such as the BCD, have shallow depths that may not allow them to assess the properties of the entire lift; this creates a problem for their usage in compaction control procedures. On the other hand, careful consideration should be given when analyzing the results for relatively thin lifts because the zone of influence of some devices might exceed the lift thickness, thus providing a composite value of two layers, rather than only the tested layer. In addition, devices apply different load magnitudes during the test, so the measurement will be different, and various devices apply different load magnitudes during testing, which affects measurement. Thus, measurement must be corrected to account for design loads. Most devices were reported to have limitations with regard to the type of unbound material they can test such as that there is not one single in situ test device that can assess all types of unbound materials. Finally, all of the devices are comparable in price with the NDG, except the portable seismic property analyzer (PSPA), which is much more expensive than the others.

As shown in Table 36, several correlations between the in situ test device measurements and those obtained by other standard in situ tests as well as design input parameters [e.g., M_r, California bearing ratio (CBR)] were reported in the literature. However, those correlations are empirical and thus can be used only in conditions similar to those encountered during their development. In general, no strong correlation was found between in situ stiffness/strength measurements and in-place density because this relationship continuously changes with moisture content.

All devices except the PSPA might have difficulties in establishing target field value in the laboratory owing to boundary effects on their measurement accuracy. Therefore, several DOTs attempted to establish those values based on pilot projects or by constructing control strips along a project. According to an interview with Indiana DOT staff, the use of control strips to develop target values of new in situ devices for compaction control facilitates the implementation of these devices in field projects because it provides the opportunity to identify their limitations. It was also found to help familiarize contractors with device procedures and measurements.

Stiffness- and Strength-Based Compaction Control Specifications

The majority of DOTs are interested in implementing stiffness- and strength-based specifications for compaction control of unbound materials, but few DOTs have developed such specifications. This was mainly attributed to the lack of trained personnel and funds, the need for new testing equipment, and

TABLE 37
COMPARISON BETWEEN DIFFERENT IN SITU SPOT DEVICES MEASURING STIFFNESS/STRENGTH

Features	BCD	CH	DCP	GeoGauge	LWD	PSPA	SCS
ASTM standard	None	D5874	D6951	ASTM 6758	E2583	None	None
Measurement	Modulus	CIV	DPI	Modulus	Modulus	Modulus	Compaction indicator
Moisture measurement	No	Yes	No	No	No	No	No
Calibration of device	UC test and BCD test on rubber blocks	Laboratory testing in Proctor mold	None	Calibration plate	Required	Laboratory stiffness testing	Preset system
Portability	Good	Medium	Good	Good	Medium	Medium	Good
Durability	N/A	Poor	Good	Good	Good	Good	Box—good Sensors—fair
Ease of use/training	Easy—minimal	Easy—minimal	Easy—minimal	Easy—minimal	Moderate	Moderate	Easy—minimal
Initial cost	Not available	$2,500	$1,000	$5,000–$5,500	$8,000–$15,000	$30,000	About $1,650
Data storage	Yes	Yes	No	Yes	Yes	Yes	Yes
Influence depth (inch)	About 6	10–12	48	5–8	11 (1–1.5 D)[a]		30 (maximum)
Repeatability	Good[b]	Medium	Good	Fair	Fair	Good[b]	Good[b]
GPS	No	Yes	No	Yes	No	No	No
Main strengths	- Simple, very quick - Can be used in the lab to determine the target modulus - Operated by one person	- Simple, quick - Strong correlations with CBR - GPS	- Simple, quick for shallow depth - Economical - Assess up to 4-ft-thick layers - Strong correlation with CBR and M_r - Used in many DOTs	- Simple, quick and nonintrusive - Good portability and durability	- Quick - Measure a wide range of modulus values - Not influenced by aggregate size	- Can be compared with lab measurement - Measure properties of multiple layers separately - Not influenced by aggregate size	- Economical - Requires minimum training
Main limitations	- Not evaluated by DOTs yet - Cannot be used for very stiff or soft soil - Shallow influence depth	- Boundary effects during calibration - Different CIV for CH models	- May require two persons - Maximum allowed particle size is 2 in. - Deeper testing can take as long as 15 min/location	- Extremely sensitive to seating conditions - Inconsistencies in testing data - Unfavorable findings by several DOTs	- High variability in weak soft soils - May require two persons	- Can be time consuming and can require complex data processing - No ASTM procedure - May be affected by the surrounding geometry - Expensive	- Not evaluated by DOTs yet - Fair durability of sensors - Does not provide any test results applicable to design or quality control purposes

[a] D is the diameter of loading plate.
[b] Based on limited data collected.
N/A = No available information.

TABLE 38
CORRELATION REPORTED IN LITERATURE FOR IN SITU DEVICES

Parameter Device	Correlations	Reference	Soil Type
	Clegg Hammer		
CBR	CBR = 0.07 (CIV)^2	Clegg (1980)	Wide range of soils
	CBR = 0.8610 (CIV)$^{1.1360}$ (R^2 = 0.757)	Al-Amoudi et al. (2002)	GM soil
	CBR = 1.3577 (CIV)$^{1.0105}$ (R^2 = 0.845)		SM soil
	CBR = 1.3489 (CIV)$^{1.0115}$ (R^2 = 0.846)		GM and SM combined soils
	CBR = 0.513 (CIV)$^{1.417}$ (R^2 = 0.94)	Aiban and Aurifullah (2006)	Steel slag and limestone aggregate base materials
	CBR = 0.564 (CIV)$^{1.144}$	Fairbrother et al. (2010)	Subgrade soils
	Dynamic Cone Penetrometer (DCP)		
M_r	M_r (psi) = 7,013.065 – 2,040.783 ln(DPI)	Hassan (1996)	Fine-grained soil
	$M_r = 27.86(DPI)^{-0.144}\left[\gamma_{dr}^{7.82} + \left(\frac{LL}{w_c}\right)^{1.925}\right]$ (R^2 =0.71)	George and Uddin (2000)	Fine-grained soil
	$M_r = 90.68\left(\frac{DPI}{\log c_u}\right)^{-0.305}\left(\gamma_{dr}^{-0.935} + w_{cr}^{0.674}\right)$ (R^2 = 0.72)		Coarse-grained soils
	$M_r = \frac{1045.9}{(\mathrm{DPI})^{1.096}}$	Mohammad et al. (2009)	A-4, A-6, A-7-5, and A-7-6
	$M_r = 3.86 + 2020.2\left(\frac{1}{\mathrm{DPI}^{1.46}}\right) + 619.4\left(\frac{1}{w^{1.27}}\right)$		
CBR	CBR = 2,559.44 / (-7.35 + DPI$^{1.84}$) + 1.04 for 6.31 < DPI < 66.67 mm/blow (R^2 = 0.93)	Abu-Farsakh et al. (2004)	Subgrade and base course materials
	Log CBR = 2.465 – 1.12 log (DPI) or CBR = 292/(DPI)$^{1.12}$	Webster et al. (1992)	Granular and cohesive materials
	Log CBR = 2.62 – 1.27 log (DPI)	Smith and Pratt (1983)	
	Log CBR = 2.56 – 1.15 log DPI	Livneh and Ishai (1987, 1991)	Fine-grained soil
	Log CBR = 2.2 – 0.71 (log DPI)$^{1.5}$		Granular soils
	Log CBR = 2.56 – 1.16 log DPI for DPI > 10	Harrison (1989)	Clayey soil
	Log CBR = 2.70 – 1.12 log DPI for of DPI < 10 (mm/blow)		Granular soil
E	Log (*E*) = 3.05 – 1.07 log (DPI)	De Beer (1990)	Subgrade soil
	Log (*E*) = 3.25 - 0.89 log (DPI)		
	Log (*E*) = 3.652 - 1.17 log (DPI)	Pen (1990)	Subgrade soil
	E = 2,224 (DPI)$^{-0.99}$	Chai and Roslie (1998)	Subgrade soil
PLT	Log (E_{PLT}) = (-0.88405) Log (DPI) + 2.90625	Konard and Lachance (2000)	Unbound aggregates and natural granular soils
	$E_{PLT\,(i)} = \frac{9770}{(\mathrm{DPI})^{1.6} - 36.9} - 0.75$ (3.27 < DPI < 66.67) (R^2 = 0.67)	Abu-Farsakh et al. (2004)	Subgrade and base course materials
	$E_{PLT\,(R2)} = \frac{4374.5}{(\mathrm{DPI})^{1.4} - 14.9} - 2.16$ (3.27 < DPI < 66.67) (R^2 = 0.72)		
FWD	M_{FWD} = 78.05 × (DPI)$^{-0.67}$	Chen et al. (2007)	Base soils
	M_{FWD} = 338 (DPI)$^{-0.39}$ for 10 mm/blow < DPI < 60 mm/blow		and subgrade soils
	$\ln(E_{FWD}) = 2.04 + \frac{5.1873}{\ln(\mathrm{DPI})}$ (3.27 < DPI < 66.67) (R^2 = 0.91)	Abu-Farsakh et al. (2004)	Subgrade and base course materials

(*continued on next page*)

TABLE 38
(*continued*)

Parameter Device	Correlations	Reference	Soil Type
	GeoGauge		
M_r	$M_r = 46.48 + 0.01 E_G^{1.54}$ ($R^2 = 0.59$) $M_r = -13.94 + 0.0397\left(E_G^{0.8}\right) + 601.08\left(\frac{1}{w^{0.78}}\right)$ ($R^2 = 0.72$)	Mohammad et al. (2009)	Subgrade and base course materials
CBR	CBR = 0.00392 $(E_G)^2$ -5.75 ($R^2 = 0.84$) for 40.8 MPa < E_G < 184.11 MPa	Abu-Farsakh et al. (2002)	Subgrade and base course materials
PLT	$E_{PLT\,(i)} = -75.58 + 1.52\,(E_G)$ ($R^2 = 0.87$) for 40.8 MPa < E_G < 194.4 MPa $E_{PLT\,(R2)} = -65.37 + 1.50\,(E_G)$ ($R^2 = 0.90$) for 40.8 MPa < E_G < 194.4 MPa	Abu-Farsakh et al. (2002)	Subgrade and base course materials
FWD	$M_{FWD} = 37.65\,H_{SG} - 261.96$	Chen et al. (2000)	Base course materials
	$M_{FWD} = -20.07 + 1.17\,(E_G)$ ($R^2 = 0.81$) for 40.8 MPa < E_G < 194.4 MPa	Abu-Farsakh et al. (2002)	Subgrade and base course materials
	Light Weight Deflectometer (LWD)		
M_r	$M_r = 27.75 \times E_{LWD}^{\;0.18}$ ($R^2 = 0.54$) $M_r = 11.23 + 12.64\left(E_{LWD}^{0.2}\right) + 242.32\left(\frac{1}{w}\right)$ ($R^2 = 0.7$)	Mohammad et al. (2009)	Subgrade and base course materials
	$\frac{E_{FWD}}{M_{R95}} = -3.907 + 5.435\,D_{(f/95)} - 0.370\,M_{(f/o)}$ ($R^2 = 0.70$) $\frac{E_{FWD}}{M_{R95}} = -2.30 + 3.860\,D_{(f/95)} - 0.316\,M_{(f/o)} - 0.635\frac{PI}{P_{200}}$ ($R^2 = 0.83$)	George (2006)	Subgrade soil
CBR	CBR = -14.0 + 0.66 (E_{LWD}) for 12.5 MPa < E_{LWD} < 174.5 MPa ($R^2 = 0.83$)	Abu-Farsakh et al. (2004)	Subgrade and base course materials
FWD	$M_{FWD} = 0.97\,(E_{LFWD})$ for 12.5 MPa < E_{LFWD} < 865 MPa ($R^2 = 0.94$)	Abu-Farsakh et al. (2004)	Subgrade and base course materials
	$E_{FWD} = 1.09 E_{LWD}$, 2,240 psi < E_{PFWD} < 30,740 psi ($R^2 = 0.64$)	George (2006)	Subgrade soil
	$M_{FWD} = 1.031\,E_{LWD(\text{Prima }100)}$	Fleming et al. (2000)	Granular layer over silty clay
	$M_{FWD} = 1.05$ to $2.22\,E_{GDP}$ $M_{FWD} = 0.76$ to $1.32\,E_{TFT}$	Fleming et al. (2000)	Granular layer over silty clay
	$\text{Log}\left(\frac{k_{LWD}}{k_{30}}\right) = 0.0031\log(k_{LWD}) + 1.12$	Kamiura et al. (2000)	Subgrade soil
PLT	$E_{PLT\,(i)} = 22 + 0.7\,(E_{LWD})$ for 12.5 MPa < E_{LWD} < 865 MPa ($R^2 = 0.92$) $E_{PLT\,(R2)} = 20.9 + 0.69\,(E_{LWD})$ for 12.5 MPa < E_{LFWD} < 865 MPa ($R^2 = 0.94$)	Abu-Farsakh et al. (2004)	Subgrade and base course materials

CBR = California bearing ratio; CIV = Clegg impact value; DPI = DCP penetration index; E = elastic modulus; E_G = GeoGauge elastic stiffness modulus; E_{GDP} = stiffness modulus from German dynamic plate; E_{LWD} = stiffness modulus of LWD; E_{PLT} = modulus from plate load test; E_{TFT} = deformation modulus from Transport Research Laboratory Foundation Tester; H_{SG} = GeoGauge stiffness reading (MN/m); M_{FWD} = FWD back-calculated modulus; M_R = resilient modulus.

contractors' unfamiliarity with stiffness- and strength-based specifications. Only the Indiana and Minnesota DOTs have widely implemented stiffness- and strength-based specifications, and both use the DCP and LWD in those specifications. Both DOTs also reported that they had positive experiences with using the DCP as a tool for compaction control of unbound materials. Other states, such as Missouri, have used the DCP in compaction control but only for a specific type of granular base material. The Indiana and Minnesota DOTs use the DCP and LWD.

Continuous Compaction Control and Intelligent Compaction

Some studies documented in this synthesis have reported good correlations between intelligent compaction measurement values

(ICMVs) and spot in situ tests (particularly DCP and LWD) when project scale averages were used rather than point-to-point comparisons. However, all correlations were project specific and not universal because they were affected by different factors, including the heterogeneity in conditions of underlying layers, moisture content variation, and differences in influence depth between intelligent compaction (IC) rollers and other in situ test measurements. Continuous compaction control (CCC) or IC measurements areas during quality control/quality assurance (QC/QA) were found to be affected by the roller vibration amplitudes.

As documented in this report, most research and implementation projects that were conducted by the FHWA and state DOTs focusing on the use of CCC and IC reported considerable success with and numerous benefits of these technologies. However, currently only three state DOTs (Indiana, Minnesota, and Texas) have IC specifications. These specifications include the selection of a target ICMV based on acceptable stiffness or density spot-testing measurements obtained on control strips compacted using IC rollers. Acceptance is based on achieving the target ICMV for at least 80% to 90% of the compacted area.

SUGGESTIONS FOR FUTURE RESEARCH

The following suggestions are meant to address the gaps in knowledge and practices that were identified in this synthesis:

- Future research is needed to develop more suitable laboratory compaction tests for unbound granular materials to better replicate field conditions in the laboratory.
- There is a need to fully understand the effects of using stiffness- and strength-based compaction control specifications on a pavement structure's longevity. This can be done by comparing the performance of similar pavement structures where conventional and stiffness- and strength-based compaction control specifications have been used. Future studies might investigate the relationship between the in situ stiffness measurements of unbound pavement materials and subgrade soils and ultimate pavement performance.
- The swell potential of fine-grained soils may not be optimized if stiffness/strength properties are used for their compaction control. Thus, future work could identify a criterion to address this issue in stiffness- and strength-based compaction control specifications.
- Limited research has been conducted on the cost-effectiveness of using non-nuclear devices for compaction control of unbound materials. Thus, future research might include life-cycle cost studies to evaluate the economic benefits of using such devices.
- A database for target values of in situ stiffness/strength measurements needs to be established for different soil types and moisture contents to facilitate the use of these devices in compaction control specifications. This database should be verified for local materials in each state before it is used in quality control. It is recommended that DOTs start with the DCP or LWD because the use of these devices has been successfully implemented in some states.
- A database of the relationships between in situ test devices used in compaction control and the resilient modulus design input value for different types of unbound materials should be developed. This will ensure that construction and design processes are fully integrated.
- Because different models and types of the same in situ devices provide different measurements, there is a need to promote standardized protocols for in situ test devices used in compaction control specifications. Pooled fund studies, such as the one recently developed for LWD (*Standardizing Lightweight Deflectometer Measurements for QA and Modulus Determination in Unbound Bases and Subgrades*), will help in developing such protocols. In addition, state DOTs are to be encouraged to develop certification and training programs, such as those developed in Indiana and Minnesota.
- Future research might investigate the development of statistical specifications for compaction control of unbound materials, which can account for the spatial variability of earthwork projects. The advent of the use of new devices that can rapidly assess the in situ stiffness/strength of unbound material can help to facilitate the implementation of such specifications.
- There is still a lack of experience and knowledge of how CCC and IC technologies can be used to improve conventional earthwork operations. Thus, more pilot and implementation projects of these technologies can help to optimize their usage and facilitate their implementation in DOT construction practices and specifications. It is essential to monitor the cost and long-term performance of future projects in which CCC and IC technologies are used, to fully understand their benefits and further support their effectiveness.

ACRONYMS

AC	Asphalt concrete
BCD	Briaud compaction device
CBR	California bearing ratio
CCC	Continuous compaction control
CH	Clegg hammer
CMV	Compaction meter value
COV	Coefficient of variation
DCP	Dynamic cone penetrometer
DOT	Department of transportation
DPI	Dynamic penetration index
EDG	Electrical density gauge
FWD	Falling weight deflectometer
GC	Clayey gravel
GDP	German dynamic plate
GM	Silty gravel
GN	Grading number
GP	Poorly graded gravel
GPS	Global Positioning System
GW	Well-graded gravel
GWT	Groundwater table
HMA	Hot-mix asphalt
IBV	Immediate bearing value
IC	Intelligent compaction
ICMV	Intelligent compaction measurement value
LL	Liquid limit
LWD	Light weight deflectometer
MDD	Maximum dry density
MDI	Moisture density indicator
MDP	Machine drive power
MEPDG	*Mechanistic–Empirical Pavement Design Guide*
ML	Silt with low plasticity
MLS	Mobile load simulator
M_r	Resilient modulus
NDG	Nuclear density gauge
OMC	Optimum moisture content
P_{200}	Percentage of material (by weight) passing No. 200 sieve
PI	Plasticity index, penetration index
PLT	Plate load test
QA	Quality assurance
QC	Quality control
RAP	Reclaimed asphalt pavement
RCA	Recycled concrete aggregate
RICM	Roller integrated compaction measurement
RMV	Roller measurement value
RTK	Real-time kinematic system
SASW	Spectral analysis of surface waves
SCS	Soil compaction supervisor
SDG	Soil density gauge
SLAVE	Simulated loading and vehicle emulator
TDR	Time domain reflectometry
TPF	Transportation pooled fund
TRIS	Transportation Research Information System
USW	Ultrasonic surface wave

CONVERSION FACTORS

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

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

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APPENDIX A

Questionnaire

NCHRP Synthesis 20-05/Topic 44-10: Non-Nuclear Methods for Compaction Control of Unbound Materials

Dear Survey Participant:

The Transportation Research Board (TRB) is preparing a synthesis on *Non-Nuclear Methods for Compaction Control of Unbound Materials*. This is being done for NCHRP, under the sponsorship of the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration.

The main objective of this synthesis is to compile all information on available non-nuclear devices that have been used for compaction control of unbound materials. This will allow State DOTs to know the advantages and limitation of the different alternatives for the nuclear density gauge.

This survey is being sent to U.S. state departments of transportation. Your cooperation in completing the questionnaire will ensure the success of this effort. **If you are not the appropriate person at your agency to complete this questionnaire, please forward it to the correct person.**

Please compete and submit this survey by **February 15, 2013**. We estimate that it should take approximately 30 minutes to complete. If you have any questions, please contact our principal investigator Munir D. Nazzal at nazzal@ohio.edu or 740-593-1080. Any supporting materials can be sent directly to Munir D. Nazzal.

Please Note that multiple people from your agency can collaboratively respond to this survey. Instructions on how to do this are provided below.

SURVEY INSTRUCTIONS:

1. To view and print the entire questionnaire, Click on the following link and print using "control p": http://surveygizmolibrary.s3.amazonaws.com/library/64484/NCHRP44101.pdf
2. To save your partial answers and complete the questionnaire later, click on the "Save and Continue Later" link at the top of the page (center). A link to the incomplete questionnaire will be emailed to you from *SurveyGizmo.* To return to the questionnaire later, open the email from *SurveyGizmo* and click on the link. We suggest using the “Save and Continue Later” feature if there will be more than 15 minutes of inactivity while the survey is opened, as some firewalls may terminate due to inactivity.
3. To pass a partially completed questionnaire to a colleague, click on the on the "Save and Continue Later" link at the top of the page (center). A link to the incomplete questionnaire will be emailed to you from *SurveyGizmo.*" Open the email from *SurveyGizmo* and forward it to a colleague.
4. To view and print your answers before submitting the survey, click forward to the page following the last question. Print using “control p.”

5. To submit the survey, click on "Submit" on the last page.

Thank you very much for your time and expertise.

Please enter the date (MM/DD/YYYY).

Please enter your contact information.

First Name *

Last Name *

Title *

Agency *

City

State *

Email Address *

Phone Number *

Fax Number

CURRENT PRACTICES FOR COMPACTION CONTROL OF UNBOUND MATERIALS

1. Please select the types of unbound materials that your agency has dealt with (check all that apply):

	Compacted Subgrade Soil	Base	Embankment
Organic soil (OL and OH)	☐	☐	☐
Low plasticity clay (CL)	☐	☐	☐
High plasticity clay (CH)	☐	☐	☐
Low plasticity silt (ML)	☐	☐	☐
High plasticity silt (MH)	☐	☐	☐
Sands	☐	☐	☐

Gravel	☐	☐	☐
Limestone	☐	☐	☐
Sandstone	☐	☐	☐
Recycled HMA	☐	☐	☐
Recycled PCC	☐	☐	☐

Other (please specify)

2. Your agency's current specifications for compaction control of unbound materials (e.g. soils, base materials, etc…) are based on (check all that apply):

☐ Relative compaction (density based) only

☐ Relative compaction (density based) and moisture content

☐ Stiffness/strength related measurements

☐ Other(please specify)

If density based criterion is used, please provide the means for developing the target density value (e.g. lab Proctor test)

3. What is your agency's acceptance criterion for unbound base layers? (check all that apply)

☐ Minimum average relative compaction values higher than

☐ Individual relative compaction values higher than

☐ Moisture content within limits of

☐ Other

Please provide the number of test points per length of highway or volume of soil that your agency specifies

4. What is your agency's acceptance criterion for compacted subgrade soils? (check all that apply)

- ☐ Minimum average relative compaction values higher than
- ☐ Individual relative compaction values higher than
- ☐ Moisture content within limits of
- ☐ Other

Please provide the number of test points per length of highway that your agency specifies

5. What is your agency's acceptance criterion for compacted soil layers in embankments? (check all that apply)

- ☐ Minimum average relative compaction values higher than
- ☐ Individual relative compaction values higher than
- ☐ Moisture content within limits of
- ☐ Other

Please provide the number of test points per length of highway or volume of soil that your agency specifies

6. Please describe the extent of usage of intelligent compaction in projects in your state (check all that apply):

- ☐ Implemented in field projects

- Evaluated in research studies only
- Demonstrated its usage
- Plan to use in the future
- Not used nor evaluated

CURRENT PRACTICES FOR COMPACTION CONTROL OF UNBOUND MATERIALS

7. Does your agency have different QC/QA specifications (e.g. number of test points is reduced) when intelligent compaction is used?

- Yes
- No

If yes, please list those changes and provide a link for the modified specifications

or upload the modified specifications

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NON-NUCLEAR METHODS FOR DENSITY MEASUREMENT

8. Please describe your agency's level of interest in using non-nuclear density devices for compaction control of unbound materials:

- Interested and have already implemented it
- Interested and will implement it
- Interested but have not implemented it
- Not Interested
- Other (please specify)

9. What are the main obstacles that will stop/impede the implementation of non-nuclear density

devices for compaction control in your state? (check all that apply)

- ☐ Need for new testing equipment
- ☐ Lack of funds
- ☐ Lack of trained personnel
- ☐ Familiarity of contractors with such devices
- ☐ Other (please specify)

---

10. What is the extent of usage of non-nuclear density devices for compaction control of unbound materials in your state? (check all that apply)

- ☐ Research
- ☐ In-house evaluation
- ☐ Field test section or demonstration project
- ☐ Developmental or experimental specification
- ☐ Production specification
- ☐ Not yet utilized

If your agency has a production specification for a non-nuclear density device, how often is it used (e.g. every project, large projects, never used, etc...)

---

11. Which of the following devices has your agency used or evaluated for measuring density? (check all that apply) *

- ☐ Moisture Density Indicator (MDI)
- ☐ Electrical Density Gauge (EDG)
- ☐ Soil Density Gauge (SDG) also known as Soil Quality Indicator (SQI)
- ☐ None
- ☐ Other (please specify)

NON-NUCLEAR METHODS FOR DENSITY MEASUREMENT

12. Describe the extent of evaluation your agency has done for the following devices (check all that apply):

	Not Evaluated	Demonstrated its use	Evaluated through in house research	Evaluated through university/consultant research
Moisture Density Indicator (MDI)	☐	☐	☐	☐
Electrical Density Gauge (EDG)	☐	☐	☐	☐
Soil Density Gauge (SDG) also known as Soil Quality Indicator (SQI)	☐	☐	☐	☐

Please upload the conducted research studies (maximum file size is 10 MB)

Choose File No file selected Upload

or provide links for the conducted research studies

13. Which of the following devices has your agency developed or currently developing specifications for use in density measurement of unbound materials? (check all that apply)

☐ Moisture Density Indicator (MDI)

☐ Electrical Density Gauge (EDG)

☐ Soil Density Gauge (SDG) also known as Soil Quality Indicator (SQI)

☐ None

☐ Other (please specify)

Please upload the developed specifications (maximum file size is 10 MB)

Choose File No file selected Upload

or provide links for the developed specifications

14. Which of the following devices has your agency implemented its use in field projects for measuring density and compaction control?(check all that apply)

- Moisture Density Indicator (MDI)
- Electrical Density Gauge (EDG)
- Soil Density Gauge (SDG) also known as Soil Quality Indicator (SQI)
- None
- Other (please specify)

15. Based on your agency's experience, please evaluate the following devices as compared to the nuclear density gauge:

	Ease of Use	Level of Calibration Needed	Testing Time	L
MDI	-- Please Select --	-- Please Select --	-- Please Select --	-- F
EDG	-- Please Select --	-- Please Select --	-- Please Select --	-- F
SDG or SQI	-- Please Select --	-- Please Select --	-- Please Select --	-- F

16. Based on your agency's experience, please evaluate the following devices as compared to the nuclear density gauge:

	Cost	Durability	Safety	C
MDI	-- Please Select --	-- Please Select --	-- Please Select --	-- F
EDG	-- Please Select --	-- Please Select --	-- Please Select --	-- F
SDG or SQI	-- Please Select --	-- Please Select --	-- Please Select --	-- F

17. Based on your agency's experience, please evaluate the following devices as compared to the nuclear density gauge:

	Accuracy	Repeatability	Recommendation for use
MDI	-- Please Select --	-- Please Select --	-- Please Select --
EDG	-- Please Select --	-- Please Select --	-- Please Select --
SDG or SQI	-- Please Select --	-- Please Select --	-- Please Select --

18. Based on your agency's experience, please indicate the compatibility of the following devices with various unbound materials (check all that apply):

	Fine grained soil	Sand	Unbound base material
MDI	☐	☐	☐
EDG	☐	☐	☐
SDG or SQI	☐	☐	☐

METHODS FOR MEASUREMENT OF IN-SITU STIFFNESS/STRENGTH

19. Which of the following devices has your agency used or evaluated for compaction control of unbound materials? (check all that apply) *

☐ Clegg Hammer

☐ GeoGauge

☐ Dynamic Cone Penetrometer (DCP)

☐ Light Weight Deflectometer (LWD), please specify type (e.g. Zorn, Prima 100, etc..)

☐ Portable Seismic Property Analyzer (PSPA)

☐ Soil Compaction Supervisor (SCS)

☐ Briaud Compaction Device (BCD)

☐ Other (please specify)

☐ None

METHODS FOR MEASUREMENT OF IN-SITU STIFFNESS/STRENGTH

20. Describe the extent of evaluation your agency has done for following devices (check all that apply):

	Not Evaluated	Demonstrated its use	Evaluated through in house research	Evaluated through university/consultant research
Clegg Hammer	☐	☐	☐	☐
Dynamic Cone Penetrometer (DCP)	☐	☐	☐	☐
Light Weight Deflectometer (LWD)	☐	☐	☐	☐
GeoGauge	☐	☐	☐	☐
Portable Seismic Property Analyzer (PSPA)	☐	☐	☐	☐
Soil Compaction Supervisor (SCS)	☐	☐	☐	☐
Briaud Compaction Device (BCD)	☐	☐	☐	☐

Please upload the conducted research studies (maximum file size is 10 MB)

Choose File No file selected Upload

or provide links for the conducted research studies

21. Which of the following devices has your agency developed or currently developing specifications for use in compaction control of unbound materials? (check all that apply)

☐ Clegg Hammer

☐ GeoGauge

☐ Dynamic Cone Penetrometer (DCP)

- Light Weight Deflectometer (LWD), please specify type (e.g. Zorn, Prima 100)
- Portable Seismic Property Analyzer (PSPA)
- Soil Compaction Supervisor (SCS)
- Briaud Compaction Device (BCD)
- Other (please specify)
- None

Please upload the developed specifications (maximum file size is 10 MB)

Choose File No file selected Upload

or provide links for the developed specifications

22. Which of the following devices has your agency implemented its use in field projects for compaction control of unbound materials? (check all that apply):

- Clegg Hammer
- GeoGauge
- Dynamic Cone Penetrometer (DCP)
- Light Weight Deflectometer (LWD), please specify type (e.g. Zorn, Prima 100)
- Portable Seismic Property Analyzer (PSPA)
- Soil Compaction Supervisor (SCS)
- Briaud Compaction Device (BCD)
- Other (please specify)
- None

23. Based on your agency's experience, please evaluate the following devices:

	Ease of Use	Level of Calibration Needed	Testing Time
Clegg Hammer	-- Please Select --	-- Please Select --	-- Please Select --
GeoGauge	-- Please Select --	-- Please Select --	-- Please Select --
DCP	-- Please Select --	-- Please Select --	-- Please Select --
LWD	-- Please Select --	-- Please Select --	-- Please Select --
PSPA	-- Please Select --	-- Please Select --	-- Please Select --
SCS	-- Please Select --	-- Please Select --	-- Please Select --
BCD	-- Please Select --	-- Please Select --	-- Please Select --

24. Based on your agency's experience, please evaluate the following devices:

	Cost	Durability	Safety
Clegg Hammer	-- Please Select --	-- Please Select --	-- Please Select --
GeoGauge	-- Please Select --	-- Please Select --	-- Please Select --
DCP	-- Please Select --	-- Please Select --	-- Please Select --
LWD	-- Please Select --	-- Please Select --	-- Please Select --
PSPA	-- Please Select --	-- Please Select --	-- Please Select --
SCS	-- Please Select --	-- Please Select --	-- Please Select --
BCD	-- Please Select --	-- Please Select --	-- Please Select --

25. Based on your agency's experience, please evaluate the following devices:

	Accuracy	Repeatability	Recommendation for use
Clegg Hammer	-- Please Select --	-- Please Select --	-- Please Select --
GeoGauge	-- Please Select --	-- Please Select --	-- Please Select --

DCP	-- Please Select --	-- Please Select --	-- Please Select --
LWD	-- Please Select --	-- Please Select --	-- Please Select --
PSPA	-- Please Select --	-- Please Select --	-- Please Select --
SCS	-- Please Select --	-- Please Select --	-- Please Select --
BCD	-- Please Select --	-- Please Select --	-- Please Select --

26. Based on your agency's experience, please indicate the compatibility of the following devices with various unbound materials (check all that apply):

	Fine grained Soil	Sand	Unbound base material
Clegg Hammer	☐	☐	☐
GeoGauge	☐	☐	☐
DCP	☐	☐	☐
LWD	☐	☐	☐
PSPA	☐	☐	☐
SCS	☐	☐	☐
BCD	☐	☐	☐

STIFFNESS BASED SPECIFICATION FOR COMPACTION CONTROL

27. Please describe your agency's level of interest in implementing stiffness/strength based specification for compaction control of unbound materials:

Interested and have already implemented it

Interested and will implement it

Interested but have not implemented it

Not Interested

Other (please specify)

28. What are the main obstacles that will stop/impede the implementation of stiffness/strength

based specification for compaction control of unbound materials in your state? (check all that apply):

- ☐ Need for new testing equipment
- ☐ Lack of funds
- ☐ Lack of trained personnel
- ☐ Familiarity of contractors with such devices
- ☐ Other (please specify)

---

29. What is the level of implementation of stiffness/strength based specifications for compaction control of unbound materials in your state? (check all that apply)

- ☐ Research
- ☐ In-house evaluation
- ☐ Field test section or demonstration project
- ☐ Developmental or experimental specification
- ☐ Production specification

If your agency has a production specification based on stiffness related parameters, how often is it used (e.g. every project, large projects, never used, etc...)

---

30. For which of the following devices and unbound materials did your agency develop a target modulus/strength value for compaction control? (check all that apply)

	Compacted Subgrade Soils	Embankment	Base
Clegg Hammer	☐	☐	☐
GeoGauge	☐	☐	☐
Light Weight Deflectometer (LWD) (e.g. Zorn, Prima 100)	☐	☐	☐
Dynamic Cone Penetrometer (DCP)	☐	☐	☐
Portable Seismic Property Analyzer (PSPA)	☐	☐	☐
Soil Compaction Supervisor (SCS)	☐	☐	☐

Briaud Compaction Device (BCD)	☐	☐	☐

Please provide links for the studies in which those target values were developed or briefly discuss the developed target values

FIELD METHODS FOR MOISTURE CONTENT MEASUREMENT

31. Which of the following devices has your agency evaluated for measuring the in-situ moisture content of unbound materials? (check all that apply):

	Not Evaluated	Demonstrated its use	Evaluated through in house research	Evaluated through university/consultant research	Currently use
DOT600	☐	☐	☐	☐	☐
Moisture Analyzer	☐	☐	☐	☐	☐
Speedy Moisture	☐	☐	☐	☐	☐
Field Microwave	☐	☐	☐	☐	☐
Moisture Density Indicator (MDI)	☐	☐	☐	☐	☐
Electrical Density Gauge (EDG)	☐	☐	☐	☐	☐
Soil Density Gauge (SDG) also known as Soil Quality Indicator (SQI)	☐	☐	☐	☐	☐
Time Domain Reflectometry (TDR) based Devices	☐	☐	☐	☐	☐

If your agency has evaluated a device for measuring the in-situ moisture content other than those listed above, please specify:

32. Based on your agency's experience, please evaluate the following devices:

	Accuracy	Repeatability	Ease of use
DOT600	-- Please Select --	-- Please Select --	-- Please Select --
Moisture Analyzer	-- Please Select --	-- Please Select --	-- Please Select --
Speedy Moisture	-- Please Select --	-- Please Select --	-- Please Select --
Field Microwave	-- Please Select --	-- Please Select --	-- Please Select --

Survey Review and Print

Thank You!

Thank you for taking our survey. Your response is very important to us. If you have any questions or comments, please feel free to contact Munir D. Nazzal at:

- E-mail: nazzal@ohio.edu
- Phone: 740-593-1080
- Mailing Address: 138 Stocker Center, Ohio University, Athens, OH 45701

APPENDIX B

Survey Respondent Information

NCHRP PROJECT 20-5
TOPIC 44-10

State	Title	Agency/Organization
AL	Asst. State Materials and Tests Engineer	Alabama DOT
AR	Materials Engineer	Arkansas State Highway and Transportation Department
AZ	Quality Assurance Engineer	Arizona DOT
CA	Chief, Office of Construction Engineering	California DOT (Caltrans)
CO	Soil Engineer	Colorado DOT
CT	Transportation Principal Engineer	Connecticut DOT
DE	Statewide Q/A Supervisor	Delaware DOT—Materials & Research
FL	State Geotechnical Materials Engineer	Florida DOT
HI	Geotechnical Unit Head	Hawaii DOT—Highways Division
ID	Structural Materials Engineer	Idaho Transportation Department
IL	Central Office Geotechnical Engineer	Illinois DOT
IN	Geotechnical Const. & Tech Support Engineer	Indiana DOT
KY	Geologist Supervisor	Kentucky Transportation Cabinet
LA	Field Quality Assurance Administrator	Louisiana DOTD Materials & Testing Section
MD	Soils & Aggregate Division chief	Maryland State Highway
ME	Quality Assurance Engineer	Maine DOT
MI	Grading and Drainage Engineer	Michigan DOT
MN	Grading and Base Engineer	Minnesota DOT
MO	Construction and Materials Engineer	Missouri DOT
MT	Testing Engineer	Montana DOT
NC	Assistant State Geotechnical Engineer	North Carolina DOT
ND	Transportation Engineer III—Geotechnical	North Dakota DOT
NE	Geotechnical Engineer	Nebraska Department of Roads
NH	Chief of Materials Technology	New Hampshire DOT
NJ	Manager, Bureau of Materials	New Jersey DOT
NM	State Pavement Engineer	New Mexico DOT
NV	Staff II, Corporate RSO	Nevada DOT
NY	Director, Geotechnical Engineering Bureau	New York State DOT
OH	State Construction Geotechnical Engineer	Ohio DOT
OK	Assistant Materials Engineer	Oklahoma DOT
OR	Pavement Design Engineer	Oregon DOT
RI	Associate Chief Engineer	Rhode Island DOT
SC	State Pavement Design Engineer	South Carolina DOT
SD	Soils Engineer	South Dakota DOT
TN	State Bituminous Engineer	Tennessee DOT
UT	Quality Assurance/Aggregate Engineer	Utah DOT
VA	State Materials Engineer	Virginia DOT
VT	Soils and Foundations Engineer	Vermont Agency of Transportation
WA	Bituminous/Chemical/Electrical Materials Engineer	Washington State DOT
WI	Foundation and Pavement Engineering Supervisor	Wisconsin DOT

Ontario	Head, Soils and Aggregates Section	Ministry of Transportation of Ontario

APPENDIX C

Survey Responses

NCHRP PROJECT 20-5

Topic 44-10

1. Please select the types of unbound materials that your agency has dealt with (check all that apply):

Choice	Compacted Subgrade		Base		Embankment		Total Count
	Percent	Count	Percent	Count	Percent	Count	
Organic soil (OL and OH)	41.5%	17	2.4%	1	36.6%	15	22
Low plasticity clay (CL)	87.8%	36	9.8%	4	78.0%	32	38
High plasticity clay (CH)	68.3%	28	7.3%	3	63.4%	26	33
Low plasticity silt (ML)	90.2%	37	4.9%	2	80.5%	33	40
High plasticity silt (MH)	58.5%	24	0.0%	0	56.1%	23	28
Sands	97.6%	40	41.5%	17	87.8%	36	40
Gravel	80.5%	33	80.5%	33	70.7%	29	38
Limestone	34.1%	14	70.7%	29	39.0%	16	31
Sandstone	34.1%	14	31.7%	13	39.0%	16	21
Recycled HMA	34.1%	14	61.0%	25	39.0%	16	32
Recycled PCC	26.8%	11	70.7%	29	41.5%	17	34

2. Your agency's current specifications for compaction control of unbound materials (e.g., soils, base materials, etc.…) are based on (check all that applies):

Choice	Count	Percent
Relative compaction (density based) only	12	29.3%
Relative compaction (density based) and moisture content	34	82.9%
Stiffness/strength related measurements	5	12.2%
Other (please specify):	6	14.6%

3. What is your agency's acceptance criterion for unbound base layers (check all that apply)?

Choice	Base		Compacted Subgrade		Embankment	
	Count	Percent	Count	Percent	Count	Percent
Minimum average relative compaction values higher than	8	19.5%	9	22.0%	9	22.0%
Individual relative compaction values higher than	34	82.9%	31	75.6%	34	82.9%
Moisture content within limits of	20	48.8%	21	51.2%	27	65.9%
Other	10	24.4%	11	26.8%	7	17.1%

4. What is your agency's acceptance criterion for compacted subgrade soils (check all that apply)?

Value	Count	Percent
Minimum average relative compaction values higher than	9	22.0%
Individual relative compaction values higher than	31	75.6%
Moisture content within limits of	21	51.2%
Other	11	26.8%

5. What is your agency's acceptance criterion for compacted soil layers in embankments (check all that apply)?

Value	Count	Percent
Minimum average relative compaction values higher than	9	22.0%
Individual relative compaction values higher than	34	82.9%
Moisture content within limits of	27	65.9%
Other	7	17.1%

6. Please describe the extent of usage of intelligent compaction in projects in your state (check all that apply):

Value	Count	Percent
Implemented in field projects	3	7.3%
Evaluated in research studies only	10	24.4%
Demonstrated its usage	11	26.8%
Plan to use in the future	13	31.7%
Not used nor evaluated	17	41.5%

7. Does your agency have different QC/QA specifications (e.g., number of test points is reduced) when intelligent compaction is used?

Value	Count	Percent
Yes	1	8.3%
No	11	91.7%

8. Please describe your agency's level of interest in using non-nuclear density devices for compaction control of unbound materials:

Value	Count	Percent
Interested and have already implemented it	3	7.3%
Interested and will implement it	2	4.9%
Interested but have not implemented it	18	43.9%
Not Interested	4	9.8%
Other (please specify)	14	34.2%

9. What are the main obstacles that will stop/impede the implementation of non-nuclear density devices for compaction control in your state (check all that apply)?

Value	Count	Percent
Need for new testing equipment	21	51.2%
Lack of funds	11	26.8%
Lack of trained personnel	15	36.6%
Familiarity of contractors with such devices	20	48.8%
Other (please specify):	23	56.1%

10. What is the extent of usage of non-nuclear density devices for compaction control of unbound materials in your state (check all that apply)?

Value	Count	Percent
Research	11	26.8%
In-house evaluation	13	31.7%
Field test section or demonstration project	11	26.8%
Developmental or experimental specification	0	0.0%
Production specification	0	0.0%
Not yet utilized	29	70.7%

11. Which of the following devices has your agency used or evaluated for measuring density (check all that apply)?

Value	Count	Percent
MDI	6	14.6%
EDG	12	29.3%
SDG or SQI	5	12.2%
None	29	70.7%
Other (please specify):	2	4.9%

12. Describe the extent of evaluation your agency has done for the following devices (check all that apply):

Choice	Not Evaluated		Demonstrated Its Use		Evaluated Through In-house Research		Evaluated Through University/Consultant Research		Total Responses
	Count	(%)	Count	(%)	Count	(%)	Count	(%)	
MDI	76.2%	16	4.8%	1	14.3%	3	9.5%	2	21
EDG	40.9%	9	22.7%	5	36.4%	8	13.6%	3	22
SDG or SQI	66.7%	14	9.5%	2	19.0%	4	9.5%	2	21

13. Which of the following devices has your agency developed or is currently developing specifications for use in density measurement of unbound materials (check all that apply)?

Value	Count	Percent
Moisture Density Indicator (MDI)	0	0.0%
Electrical Density Gauge (EDG)	0	0.0%
Soil Density Gauge (SDG) also known as Soil Quality Indicator (SQI)	0	0.0%
Time Domain Reflectometry (TDR) based Devices	0	0.0%
None	19	79.2%
Other (please specify):	5	20.8%

14. Which of the following devices has your agency implemented its use in field projects for measuring density and compaction control (check all that apply)?

Value	Count	Percent
Moisture Density Indicator (MDI)	0	0.0%
Electrical Density Gauge (EDG)	0	0.0%
Soil Density Gauge (SDG) also known as Soil Quality Indicator (SQI)	0	0.0%
Time Domain Reflectometry (TDR) based Devices	0	0.0%
None	5	50.0%
Other (please specify):	5	50.0%

15. Ease of Use—EDG

Value	Count	Percent
Easy	0	0.0%
Moderately easy	2	14.3%
Slightly complex	3	21.4%
Complex	3	21.4%
I don’t know	4	42.9%

16. Ease of Use—MDI

Value	Count	Percent
Slightly complex	2	20.0%
Complex	2	20.0%
I don’t know	0	60.0%

16. Ease of Use—SDG or SQI

Value	Count	Percent
Easy	1	8.3%
Moderately easy	1	8.3%
Slightly complex	1	8.3%
Complex	2	16.7%
I don’t know	7	58.3%

16. Level of Calibration Needed—EDG

Value	Count	Percent
Time-consuming	7	58.3%
I don’t know	5	41.7%

16. Level of Calibration Needed—MDI

Value	Count	Percent
Time-consuming	3	33.3%
I don’t know	3	66.7%

16. Level of Calibration Needed—SDG or SQI

Value	Count	Percent
Time-consuming	2	18.2%
Simple and quick	1	9.1%
I don’t know	2	72.7%

16. Testing Time—EDG

Value	Count	Percent
Short	0	0.0%
Moderately short	4	33.3%
Slightly long	3	25.0%
Long	1	8.3%
I don’t know	4	33.3%

16. Testing Time—MDI

Value	Count	Percent
Short	1	11.1%
Moderately short	1	11.1%
Slightly long	1	11.1%
Long	1	11.1%
I don’t know	2	55.6%

16. Testing Time—SDG or SQI

Value	Count	Percent
Short	1	8.3%
Moderately short	4	33.3%
Slightly long	1	8.3%

16. Level of Expertise Required—EDG

Value	Count	Percent
High	4	30.8%
Intermediate	5	38.5%
I don’t know	3	30.8%

16. Level of Expertise Required—MDI

Value	Count	Percent
High	2	22.2%
Intermediate	2	22.2%
I don’t know	2	55.6%

16. Level of Expertise Required—SDG or SQI

Value	Count	Percent
High	2	20.0%
Intermediate	3	30.0%

17. Cost—EDG

Value	Count	Percent
More expensive	0	0.0%
About the same	2	13.3%
Less expensive	3	20.0%
I don’t know	10	66.7%

17. Cost—MDI

Value	Count	Percent
More expensive	0	0.0%
About the same	0	0.0%
Less expensive	1	9.1%
I don’t know	10	90.9%

17. Cost—SDG or SQI

Value	Count	Percent
More expensive	0	0.0%
About the same	1	8.3%
Less expensive	1	8.3%
I don’t know	10	83.3%

17. Cost—TDR-based devices

Value	Count	Percent
More expensive	0	0.0%
About the same	0	0.0%
Less expensive	0	0.0%
I don't know	0	0.0%

17. Durability—EDG

Value	Count	Percent
Very good	0	0.0%
Good	1	7.1%
Fair	3	21.4%
Poor	0	0.0%
I don't know	10	71.4%

17. Durability—MDI

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	1	10.0%
Poor	1	10.0%
I don't know	8	80.0%

17. Durability—SDG or SQI

Value	Count	Percent
Very good	0	0.0%
Good	2	18.2%
Fair	1	9.1%
Poor	0	0.0%
I don't know	8	72.7%

17. Safety—EDG

Value	Count	Percent
Safe	6	42.9%
Moderately safe	1	7.1%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don't know	7	50.0%

17. Safety—MDI

Value	Count	Percent
Safe	3	30.0%
Moderately safe	0	0.0%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don't know	7	70.0%

17. Safety—SDG or SQI

Value	Count	Percent
Safe	4	36.4%
Moderately safe	0	0.0%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don't know	7	63.6%

17. GPS compatibility-EDG

Value	Count	Percent
Yes	1	7.1%
No	0	0.0%
I don't know	13	92.9%

17. GPS compatibility-MDI

Value	Count	Percent
Yes	2	20.0%
No	0	0.0%
I don't know	8	80.0%

17. GPS compatibility—SDG or SQI

Value	Count	Percent
Yes	2	20.0%
No	0	0.0%
I don’t know	8	80.0%

18. Accuracy—EDG

Value	Count	Percent
Good	1	6.7%
Fair	4	26.7%
Poor	4	26.7%
I don’t know	3	40.0%

18. Accuracy—MDI

Value	Count	Percent
Fair	1	9.1%
Poor	3	27.3%
I don’t know	2	63.6%

18. Accuracy—SDG or SQI

Value	Count	Percent
Fair	2	16.7%
Poor	3	25.0%
I don’t know	7	58.3%

18. Repeatability—EDG

Value	Count	Percent
Good	2	14.3%
Fair	4	28.6%
Poor	3	21.4%
I don’t know	5	35.7%

18. Repeatability—MDI

Value	Count	Percent
Good	1	10.0%
Fair	2	20.0%
Poor	1	10.0%
I don’t know	6	60.0%

18. Repeatability—SDG or SQI

Value	Count	Percent
Very good	0	0.0%
Good	1	9.1%
Fair	1	9.1%
Poor	3	27.3%
I don’t know	6	54.6%

18. Recommendation for use—EDG

Value	Count	Percent
Yes	0	0.0%
No	9	64.3%
I don’t know	5	35.7%

18. Recommendation for use—MDI

Value	Count	Percent
Yes	0	0.0%
No	4	40.0%
I don’t know	6	60.0%

18. Recommendation for use—SDG or SQI

Value	Count	Percent
Yes	0	0.0%
No	5	45.5%
I don’t know	6	54.6%

19. Based on your agency's experience, please indicate the compatibility of the following devices with various unbound materials (check all that apply):

	Fine Grained Soil		Sand		Unbound Base Material		Total Responses
	Percent	Count	Percent	Count	Percent	Count	
MDI	100.0%	2	100.0%	2	0.0%	0	2
EDG	71.4%	5	71.4%	5	85.7%	6	7
SDG or SQI	100.0%	1	100.0%	1	100.0%	1	1

20. Which of the following devices has your agency used or evaluated for compaction control of unbound materials (check all that apply)?

Value	Count	Percent
Clegg Hammer	6	14.6%
GeoGauge	19	46.3%
DCP	20	48.8%
LWD	13	31.7%
PSPA	1	2.4%
SCS	0	0.0%
BCD	1	2.4%
Other (please specify):	6	14.6%
None	15	36.6%

21. Describe the extent of evaluation your agency has done for following devices (check all that apply):

	Not Evaluated		Demonstrated Its Use		Evaluated Through In-House Research		Evaluated Through University/Consultant Research		Responses
	%	No.	%	No.	%	No.	%	No.	No.
Clegg Hammer	70.6%	12	11.8%	2	17.6%	3	17.6%	3	17
Dynamic Cone Penetrometer (DCP)	20.8%	5	45.8%	11	50.0%	12	33.3%	8	24
Light Weight Deflectometer (LWD)	45.5%	10	22.7%	5	40.9%	9	13.6%	3	22
GeoGauge	19.0%	4	23.8%	5	52.4%	11	33.3%	7	21
Portable Seismic Property Analyzer (PSPA)	76.9%	10	15.4%	2	23.1%	3	7.7%	1	13
Soil Compaction Supervisor (SCS)	100.0%	14	0.0%	0	0.0%	0	0.0%	0	14
Briaud Compaction Device (BCD)	92.3%	12	7.7%	1	7.7%	1	0.0%	0	13

22. Which of the following devices has your agency developed or is currently developing specifications for use in compaction control of unbound materials (check all that apply)?

Value	Count	Percent
Clegg Hammer	0	0.0%
GeoGauge	0	0.0%
Dynamic Cone Penetrometer (DCP)	5	20.8%
Light Weight Deflectometer (LWD), please specify type (e.g., Zorn, Prima 100)	3	12.5%
Portable Seismic Property Analyzer (PSPA)	0	0.0%
Soil Compaction Supervisor (SCS)	0	0.0%
Briaud Compaction Device (BCD)	0	0.0%
Other (please specify)	2	8.3%
None	16	66.7%

23. Which of the following devices has your agency implemented its use in field projects for compaction control of unbound materials (check all that apply)?

Value	Count	Percent
Clegg Hammer	0	0.0%
GeoGauge	0	0.0%
Dynamic Cone Penetrometer (DCP)	2	16.7%
Light Weight Deflectometer (LWD), please specify type (e.g., Zorn, Prima 100)	2	8.3%
Portable Seismic Property Analyzer (PSPA)	0	0.0%
Soil Compaction Supervisor (SCS)	0	0.0%
Briaud Compaction Device (BCD)	0	0.0%
Other (please specify)	2	16.7%
None	7	58.3%

24. Ease of Use—BCD

Value	Count	Percent
Easy	1	14.3%
Moderately easy	0	0.0%
Slightly complex	0	0.0%
Complex	0	0.0%
I don’t know	6	85.7%

24. Ease of Use—Clegg Hammer

Value	Count	Percent
Easy	4	40.0%
Moderately easy	1	10.0%
I don’t know	1	50.0%

24. Ease of Use—DCP

Value	Count	Percent
Easy	7	38.9%
Moderately easy	9	50.0%
Slightly complex	1	5.6%
I don’t know	3	5.6%

24. Ease of Use—GeoGauge

Value	Count	Percent
Easy	6	37.5%
Moderately easy	6	37.5%
Complex	1	6.3%
I don’t know	6	18.8%

24. Ease of Use—LWD

Value	Count	Percent
Easy	2	14.3%
Moderately easy	5	35.7%
Slightly complex	4	28.6%
I don’t know	1	21.4%

24. Ease of Use—PSPA

Value	Count	Percent
Easy	0	0.0%
Moderately easy	0	0.0%
Slightly complex	1	16.7%
Complex	0	0.0%
I don’t know	5	83.3%

24. Ease of Use—SCS

Value	Count	Percent
Easy	0	0.0%
Moderately easy	0	0.0%
Slightly complex	0	0.0%
Complex	0	0.0%
I don’t know	7	100.0%

24. Level of Calibration Needed—BCD

Value	Count	Percent
Difficult	1	20.0%
Time-consuming	0	0.0%
Simple and quick	0	0.0%
I don't know	4	80.0%

24. Level of Calibration Needed—Clegg Hammer

Value	Count	Percent
Difficult	1	12.5%
Time-consuming	1	12.5%
Simple and quick	2	25.0%
I don't know	2	50.0%

24. Level of Calibration Needed—DCP

Value	Count	Percent
Difficult	1	5.9%
Time-consuming	2	11.8%
Simple and quick	12	70.6%
I don't know	5	11.8%

24. Level of Calibration Needed—GeoGauge

Value	Count	Percent
Difficult	3	21.4%
Time-consuming	1	7.1%
Simple and quick	6	42.9%
I don't know	9	28.6%

24. Level of Calibration Needed—LWD

Value	Count	Percent
Difficult	1	9.1%
Time-consuming	3	27.3%
Simple and quick	3	27.3%
I don't know	6	36.4%

24. Level of Calibration Needed—PSPA

Value	Count	Percent
Difficult	1	20.0%
Time-consuming	0	0.0%
Simple and quick	0	0.0%
I don't know	4	80.0%

24. Level of Calibration Needed—SCS

Value	Count	Percent
Difficult	0	0.0%
Time-consuming	0	0.0%
Simple and quick	0	0.0%
I don't know	5	100.0%

24. Testing Time—BCD

Value	Count	Percent
Short	1	20.0%
Moderately short	0	0.0%
Slightly long	0	0.0%
Long	0	0.0%
I don't know	4	80.0%

24. Testing Time—Clegg Hammer

Value	Count	Percent
Short	3	33.3%
Moderately short	2	22.2%
I don't know	1	44.4%

24. Testing Time—DCP

Value	Count	Percent
Short	5	27.8%
Moderately short	8	44.4%
Slightly long	4	22.2%
I don't know	3	5.6%

24. Testing Time—GeoGauge

Value	Count	Percent
Short	8	53.3%
Moderately short	4	26.7%
I don't know	7	20.0%

24. Testing Time—LWD

Value	Count	Percent
Short	3	23.1%
Moderately short	7	53.9%
Slightly long	1	7.7%
I don't know	2	15.4%

24. Testing Time—PSPA

Value	Count	Percent
Short	0	0.0%
Moderately short	1	20.0%
Slightly long	0	0.0%
Long	0	0.0%
I don't know	4	80.0%

24. Testing Time—SCS

Value	Count	Percent
Short	0	0.0%
Moderately short	0	0.0%
Slightly long	0	0.0%
Long	0	0.0%
I don't know	5	100.0%

24. Level of Expertise Required—BCD

Value	Count	Percent
High	0	0.0%
Intermediate	1	20.0%
Low	0	0.0%
I don't know	4	80.0%

24. Level of Expertise Required—Clegg Hammer

Value	Count	Percent
Intermediate	2	22.2%
Low	3	33.3%
I don't know	4	44.4%

24. Level of Expertise Required—DCP

Value	Count	Percent
Intermediate	7	38.9%
Low	10	55.6%
I don't know	2	5.6%

24. Level of Expertise Required—GeoGauge

Value	Count	Percent
High	3	20.0%
Intermediate	5	33.3%
Low	3	20.0%
I don't know	4	26.7%

24. Level of Expertise Required—LWD

Value	Count	Percent
Intermediate	8	66.7%
Low	1	8.3%
I don't know	3	25.0%

24. Level of Expertise Required—PSPA

Value	Count	Percent
High	1	16.7%
Intermediate	0	0.0%
Low	1	16.7%
I don't know	4	66.7%

24. Level of Expertise Required—SCS

Value	Count	Percent
High	0	0.0%
Intermediate	0	0.0%
Low	0	0.0%
I don't know	5	100.0%

25. Cost—BCD

Value	Count	Percent
Expensive	0	0.0%
Moderately expensive	0	0.0%
Not expensive	0	0.0%
I don't know	6	100.0%

25. Cost—Clegg Hammer

Value	Count	Percent
Moderately expensive	3	33.3%
Not expensive	2	22.2%
I don't know	4	44.4%

25. Cost—DCP

Value	Count	Percent
Moderately expensive	4	22.2%
Not expensive	12	66.7%
I don't know	2	11.1%

25. Cost—GeoGauge

Value	Count	Percent
Expensive	1	6.7%
Moderately expensive	7	46.7%
Not expensive	1	6.7%
I don't know	6	40.0%

25. Cost—LWD

Value	Count	Percent
Expensive	5	38.5%
Moderately expensive	5	38.5%
I don't know	3	23.1%

25. Cost—PSPA

Value	Count	Percent
Expensive	1	16.7%
Moderately expensive	0	0.0%
Not expensive	0	0.0%
I don't know	5	83.3%

25. Cost—SCS

Value	Count	Percent
Expensive	0	0.0%
Moderately expensive	0	0.0%
Not expensive	0	0.0%
I don't know	6	100.0%

25. Durability—BCD

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	1	20.0%
Poor	0	0.0%
I don't know	4	80.0%

25. Durability—Clegg Hammer

Value	Count	Percent
Good	3	33.3%
Fair	1	11.1%
I don't know	5	55.6%

25. Durability—DCP

Value	Count	Percent
Very good	6	33.3%
Good	8	44.4%
Fair	3	16.7%
I don't know	1	5.6%

25. Durability—GeoGauge

Value	Count	Percent
Very good	1	6.3%
Good	5	31.3%
Fair	3	18.8%
I don't know	7	43.8%

25. Durability—LWD

Value	Count	Percent
Very good	2	15.4%
Good	5	38.5%
Fair	2	15.4%
Poor	1	7.7%
I don't know	3	23.1%

25. Durability—PSPA

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	1	16.7%
I don't know	5	83.3%

25. Durability—SCS

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	0	0.0%
I don't know	5	100.0%

25. Safety—BCD

Value	Count	Percent
Safe	1	20.0%
Moderately safe	0	0.0%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don't know	4	80.0%

25. Safety—Clegg Hammer

Value	Count	Percent
Safe	3	37.5%
Moderately safe	0	0.0%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don't know	5	62.5%

25. Safety—DCP

Value	Count	Percent
Safe	6	35.3%
Moderately safe	9	52.9%
Slightly hazardous	1	5.9%
Unsafe	0	0.0%
I don’t know	1	5.9%

25. Safety—GeoGauge

Value	Count	Percent
Safe	11	68.8%
Moderately safe	1	6.3%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don’t know	4	25.0%

25. Safety—LWD

Value	Count	Percent
Safe	4	33.3%
Moderately safe	4	33.3%
Slightly hazardous	1	8.3%
Unsafe	0	0.0%
I don’t know	3	25.0%

25. Safety—PSPA

Value	Count	Percent
Safe	1	20.0%
Moderately safe	0	0.0%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don’t know	4	80.0%

25. Safety—SCS

Value	Count	Percent
Safe	0	0.0%
Moderately safe	0	0.0%
Slightly hazardous	0	0.0%
Unsafe	0	0.0%
I don’t know	5	100.0%

25. GPS compatibility—BCD

Value	Count	Percent
Yes	0	0.0%
No	0	0.0%
I don’t know	5	100.0%

25. GPS compatibility—Clegg Hammer

Value	Count	Percent
Yes	0	0.0%
No	2	22.2%
I don’t know	7	77.8%

25. GPS compatibility—DCP

Value	Count	Percent
Yes	1	5.9%
No	9	52.9%
I don’t know	7	41.2%

25. GPS compatibility—GeoGauge

Value	Count	Percent
Yes	0	0.0%
No	4	26.7%
I don’t know	11	73.3%

25. GPS compatibility—LWD

Value	Count	Percent
Yes	4	33.3%
No	2	16.7%
I don't know	6	50.0%

25. GPS compatibility—PSPA

Value	Count	Percent
Yes	0	0.0%
No	0	0.0%
I don't know	5	100.0%

25. GPS compatibility—SCS

Value	Count	Percent
Yes	0	0.0%
No	0	0.0%
I don't know	5	100.0%

26. Accuracy—BCD

Value	Count	Percent
Very good	0	0.0%
Good	1	16.7%
Fair	0	0.0%
Poor	0	0.0%
I don't know	5	83.3%

26. Accuracy—Clegg Hammer

Value	Count	Percent
Good	2	25.0%
Poor	1	12.5%
I don't know	5	62.5%

26. Accuracy—DCP

Value	Count	Percent
Very good	4	23.5%
Good	8	47.1%
Fair	3	17.7%
Poor	1	5.9%
I don't know	1	5.9%

26. Accuracy—GeoGauge

Value	Count	Percent
Good	2	12.5%
Fair	3	18.8%
Poor	7	43.8%
I don't know	4	25.0%

26. Accuracy—LWD

Value	Count	Percent
Very good	2	16.7%
Good	4	33.3%
I don't know	6	50.0%

26. Accuracy—PSPA

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	1	16.7%
Poor	0	0.0%
I don't know	5	83.3%

26. Accuracy—SCS

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	0	0.0%
I don't know	5	100.0%

26. Repeatability—BCD

Value	Count	Percent
Very good	0	0.0%
Good	1	20.0%
Fair	0	0.0%
Poor	0	0.0%
I don't know	4	80.0%

26. Repeatability—Clegg Hammer

Value	Count	Percent
Good	2	28.6%
Fair	1	14.3%
Poor	0	0.0%
I don't know	4	57.1%

26. Repeatability—DCP

Value	Count	Percent
Very good	1	5.9%
Good	9	52.9%
Fair	5	29.4%
Poor	1	5.9%
I don't know	1	5.9%

26. Repeatability—GeoGauge

Value	Count	Percent
Very good	1	6.7%
Good	2	13.3%
Fair	2	13.3%
Poor	7	46.7%
I don't know	3	20.0%

26. Repeatability—LWD

Value	Count	Percent
Very good	1	8.3%
Good	6	50.0%
Fair	1	8.3%
I don't know	4	33.3%

26. Repeatability—PSPA

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	1	20.0%
Poor	0	0.0%
I don't know	4	80.0%

26. Repeatability—SCS

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	0	0.0%
I don't know	5	100.0%

26. Recommendation for use—BCD

Value	Count	Percent
Yes	1	20.0%
No	0	0.0%
I don’t know	4	80.0%

26. Recommendation for use—Clegg Hammer

Value	Count	Percent
Yes	2	25.0%
No	1	12.5%
I don’t know	3	62.5%

26. Recommendation for use—DCP

Value	Count	Percent
Yes	11	64.7%
No	2	11.8%
I don’t know	4	23.5%

26. Recommendation for use—GeoGauge

Value	Count	Percent
Yes	0	0.0%
No	8	57.1%
I don’t know	11	42.9%

26. Recommendation for use—LWD

Value	Count	Percent
Yes	9	69.2%
No	0	0.0%
I don’t know	4	30.8%

26. Recommendation for use—PSPA

Value	Count	Percent
Yes	0	0.0%
No	1	20.0%
I don’t know	4	80.0%

26. Recommendation for use—SCS

Value	Count	Percent
Yes	0	0.0%
No	0	0.0%
I don’t know	5	100.0%

27. Based on your agency’s experience, please indicate the compatibility of the following devices with various unbound materials (check all that apply):

	Fine-grained Soil		Sand		Unbound Base Material		Total Responses
	%	No.	%	No.	%	No.	No.
Clegg Hammer	50.0%	3	66.7%	4	33.3%	2	4
GeoGauge	36.8%	7	42.1%	8	21.1%	4	9
DCP	65.0%	13	65.0%	13	50.0%	10	16
LWD	53.8%	7	76.9%	10	69.2%	9	10
PSPA		1	100.0 %	1	100.0%	1	1
SCS		0	0.0%	0	0.0%	0	
BCD		1	100.0 %	1	100.0%	1	1

28. Please describe your agency’s level of interest in implementing stiffness/strength based specification for compaction control of unbound materials:

Value	Count	Percent
Interested and have already implemented it	2	4.9%
Interested and will implement it	6	14.6%
Interested but have not implemented it	19	46.3%
Not Interested	9	22.0%
Other (please specify):	5	12.2%
		100.0%

29. What are the main obstacles that will stop/impede the implementation of stiffness/strength based specification for compaction control of unbound materials in your state (check all that apply)?

Value	Count	Percent
Need for new testing equipment	22	55.0%
Lack of funds	10	25.0%
Lack of trained personnel	19	47.5%
Familiarity of contractors with such devices	18	45.0%
Other (please specify):	21	52.5%

30. What is the level of implementation of stiffness/strength based specifications for compaction control of unbound materials in your state (check all that apply)?

Value	Count	Percent
Research	19	67.9%
In-house evaluation	15	53.6%
Field test section or demonstration project	9	32.1%
Developmental or experimental specification	4	14.3%
Production specification	3	10.7%

31. For which of the following devices and unbound materials did your agency develop a target modulus/strength value for compaction control (check all that apply)?

	Compacted Subgrade Soils		Embankment		Base		Responses
	%	No.	%	No.	%	No.	No.
Clegg Hammer	0.0%	0	0.0%	0	0.0%	0	0
GeoGauge	100.0%	1	100.0%	1	100.0%	1	1
Light Weight Deflectometer (LWD) (e.g., Zorn, Prima 100)	75.0%	3	75.0%	3	75.0%	3	4
Dynamic Cone Penetrometer (DCP)	80.0%	4	100.0%	5	60.0%	3	5
Portable Seismic Property Analyzer (PSPA)	0.0%	0	0.0%	0	0.0%	0	0
Soil Compaction Supervisor (SCS)	0.0%	0	0.0%	0	0.0%	0	
Briaud Compaction Device (BCD)	0.0%	0	0.0%	0	0.0%	0	

32. Which of the following devices has your agency evaluated for measuring the in situ moisture content of unbound materials (check all that apply)?

	Not Evaluated		Demonstrated Its Use		Evaluated Through In-house Research		Evaluated Through University/Consultant Research		Currently Use		Responses
	%	No.	%	No.	%	No.	%	No.	%	No.	No.
DOT600	100%	28	0.0%	0	0.0%	0	0.0%	0	0.0%	0	28
Moisture Analyzer	100%	27	0.0%	0	0.0%	0	0.0%	0	0.0%	0	27
Speedy Moisture	29.0%	9	29.0%	9	25.8%	8	0.0%	0	32.%	10	31
Field Microwave	53.3%	16	6.7%	2	13.3%	4	0.0%	0	40.0%	12	30
MDI	86.2%	25	0.0%	0	3.4%	1	6.9%	2	3.4%	1	29
EDG	75.0%	21	7.1%	2	17.9%	5	7.1%	2	0.0%	0	28
SDG	89.3%	25	0.0%	0	7.1%	2	7.1%	2	0.0%	0	28

33. Accuracy-DOT600

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	0	0.0%
I don’t know	14	100.0%

33. Accuracy—Field Microwave

Value	Count	Percent
Very good	2	8.3%
Good	13	54.2%
Fair	1	4.2%
Poor	1	4.2%
I don't know	7	29.2%

33. Accuracy—Moisture Analyzer

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	0	0.0%
I don't know	14	100.0%

33. Accuracy—Speedy Moisture

Value	Count	Percent
Very good	3	10.7%
Good	10	35.7%
Fair	5	17.9%
Poor	2	7.1%
I don't know	8	28.6%

33. Repeatability—DOT600

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	0	0.0%
I don't know	12	100.0%

33. Repeatability—Field Microwave

Value	Count	Percent
Very good	2	8.7%
Good	13	56.5%
Fair	1	4.4%
Poor	1	4.4%
I don't know	6	26.1%

33. Repeatability—Moisture Analyzer

Value	Count	Percent
Very good	0	0.0%
Good	0	0.0%
Fair	0	0.0%
Poor	0	0.0%
I don't know	11	100.0%

33. Repeatability—Speedy Moisture

Value	Count	Percent
Very good	3	11.5%
Good	10	38.5%
Fair	6	23.1%
Poor	2	7.7%
I don't know	5	19.2%

33. Ease of use—DOT600

Value	Count	Percent
Easy	0	0.0%
Moderately easy	0	0.0%
Slightly complex	0	0.0%
Complex	0	0.0%
I don't know	12	100.0%

33. Ease of use—Field Microwave

Value	Count	Percent
Easy	6	26.1%
Moderately easy	9	39.1%
Slightly complex	1	4.4%
I don't know	7	30.4%

33. Ease of use—Moisture Analyzer

Value	Count	Percent
Easy	0	0.0%
Moderately easy	0	0.0%
Slightly complex	0	0.0%
Complex	0	0.0%
I don't know	12	100.0%

33. Ease of use—Speedy Moisture

Value	Count	Percent
Easy	7	26.9%
Moderately easy	9	34.6%
Slightly complex	4	15.4%
Complex	2	7.7%
I don't know	4	15.4%

33. Recommendation for use—DOT600

Value	Count	Percent
Yes	0	0.0%
No	1	8.3%
I don't know	11	91.7%

33. Recommendation for use—Field Microwave

Value	Count	Percent
Yes	13	56.5%
No	4	17.4%
I don't know	6	26.1%

33. Recommendation for use—Moisture Analyzer

Value	Count	Percent
Yes	0	0.0%
No	1	8.3%
I don't know	11	91.7%

33. Recommendation for use—Speedy Moisture

Value	Count	Percent
Yes	13	50.0%
No	9	34.6%
I don't know	4	15.4%

Abbreviations used without definitions in TRB publications:

A4A	Airlines for America
AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation