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Compaction Quality Assurance Specifications of Unbound Materials

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Abstract: Compaction quality control/assurance of unbound geomaterials is one of the crucial components in pavement and embankment construction to ensure their performance, stability, and sustainability. Conventional density-based methods such as nuclear density gauge to determine the compaction quality have been widely used due to the straightforward relationship between the readings and targeted material property. Recent modifications in construction standards and the introduction of the Mechanistic-Empirical Pavement Design Guide have inspired a growing interest in developing and implementing strength/stiffness-based compaction control quality assurance (QA) specifications. Numerous studies have been dedicated to investigating the efficiency and effectiveness of the stiffness-based compaction QA tools. This paper presents a comprehensive review of the recent compaction QA relevant literature and surveys. Findings of different approaches for studying QA devices, and the main results of the existing models, experiments, and engineering practices were summarized. Several in situ spot QA technologies, including the latest compaction QA technologies [e.g., the lightweight deflectometer (LWD)], were highlighted, and their efficiency and effectiveness were compared. The review also summarized the intercorrelations between different devices, the correlations between in situ QA device readings and mechanical properties of unbound material, findings of the numerical simulations, and case studies and current practices using different QA tools. The recommendations for future research needs and practical implementations were identified and discussed. **DOI: 10.1061/JPEODX.0000403.** © *2022 American Society of Civil Engineers.*

Author keywords: Compaction; Quality control assurance; Nuclear density gauge; Density-based; Modulus-based.

Introduction

Proper compaction is critical during construction of unbound geomaterials to ensure the performance and sustainability of pavements (Holtz 1990). Since the development of Proctor compaction curve in 1933 revolutionized the compaction process (Proctor 1933), density-based quality assurance (QA)/quality control (QC) specifications have been developed and extensively used by most of the transportation agencies around the world. Typically, the field density and moisture content (MC) have been determined by means of nuclear density gauges (NDG).

The resilient modulus of the unbound material layer is a required input for pavement design and structural analysis since it's recommended by the AASHTO pavement design guide in 1993 (Puppala 2008). This shift in pavement design led to the development of many compaction QA devices/technologies. Efforts have been dedicated by the transportation agencies and researchers to evaluate the effectiveness and efficiency of these devices from the practical and technical point of view. Many comprehensive review studies on the compaction QA methods have been conducted (e.g., Rathje et al. 2006; Fleming et al. 2007; Puppala 2008; Meehan et al. 2012; Berney et al. 2013; Tutumluer 2013; Nazzal 2014; Hamid 2015; Nazarian et al. 2015; Mehta and Ali 2016; Mata et al. 2018; McLain and Gransberg 2019; Mohajerani et al. 2020). However, some of them were completed several years ago and, consequently, they didn't cover many recent developments. They mostly focused on one specific technique [e.g., Mohajerani et al. (2020) for Clegg hammer (CH); Hamid (2015) for dynamic cone penetrometer (DCP); Fleming et al. (2007) for lightweight deflectometer (LWD)], the intercorrelations between different devices and numerical simulation finding (e.g., most of them) were neglected. Therefore, a significant literature review is required to stand on the latest developments and compile new information regarding compaction QA specifications.

Currently, a variety of in situ test devices are available to measure the modulus or strength of unbound materials. These devices include penetration devices (e.g., DCP and CH), static/impact loading devices [e.g., Briaud compaction device (BCD), GeoGauge, and LWD], geophysical-based devices [e.g., portable seismic property analyzer (PSPA)], and buried sensors [e.g., soil compaction supervisor (SCS)] (Puppala 2008; Tutumluer 2013; Nazzal 2014; MHTC 2019). Among these devices, LWD was recommended for a modulus-based construction specification for acceptance of compacted geo-materials (Nazarian et al. 2015). Many states have been exploring the use of the LWD as a replacement of the NDG and creating standards for acceptance of unbound material layers, including an ongoing project conducted by the authors (Riad et al. 2021). However, it has not been well documented.

Hence, this paper presents a comprehensive review of published materials (nationally and internationally) and ongoing research

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projects to gather the latest information for compaction QA of unbound materials layers. Density-based and strength/stiffness-based QA techniques were summarized with working principles, correlations with soils properties and other in situ devices, analysis methods and results. In addition, different from the existing review studies that focus on one specific technique, this study included several in situ spot QA technologies and compares their efficiency and effectiveness. The review reported the intercorrelations between different devices, the correlations between in situ QA device readings and mechanical properties of unbound material, findings of the numerical simulations, and case studies and current practices using different QA tools. Moreover, several case studies in which different compaction QA methods were used were provided and discussed to emphasize the degree of success and extend of use for each technique. The recommendations for future research needs and practical implementations were identified and discussed as well.

Density-Based QA

Density-based methods have been used by most state DOTs to assess the compaction quality of unbound materials, in which the field measurements (density and MC) with the target values are compared. The target MC and density values, typically, are determined in the laboratory by conducting a specific standard compaction test [i.e., ASTM D1557 (ASTM 2012); AASHTO T 180 (AASHTO 2020); AASHTO T99] on the same material used in the construction site. Several methods are available in practice to determine the field MC and/or dry density. Among these methods, the NDG has been extensively used by most of the state DOTs for the last four decades.

Nuclear Density Gauge

The NDG device was first introduced in 1962 to measure the MC of soils for agricultural purposes (Troxler 2000). Once a standard calibration method was developed, it gained popularity in the construction industry (Kim et al. 2010). The NDG device determines the wet density of the material to be tested by detecting the reflected rays of emitted gamma radiations; see ASTM D6938 (ASTM 2017). The gamma radiations interact with the photons contained in the material (i.e., unbound aggregates). Denser materials have higher electrons content, and therefore reflect fewer photons. The number of detected photons can be correlated to the wet density of the tested material through calibrated relationships. The NDG test is fast, accurate, and repeatable when adequately calibrated. The NDG also has the option of varying the depth of measurement when a direct transmission procedure is applied. Among all density tests, the NDG is one of the few that can provide both MC and density for the tested material. However, it is not designed to measure bulk density in soils containing large amounts of organic material (>5%) (Randrup and Lichter 2001; Labelle and Jaeger 2021). Moreover, extensive field testing in Delaware showed that the NDG test results are significantly scattered when compared to results from other in situ density test standards (e.g., sand cone, water balloon) (Meehan and Hertz 2011). Nonetheless, NDG has become the accepted industry standard for QC/QA only due to its practical superiorities over other tests.

Although the NDG is a standardized device and has been used for several decades, the device calibration methodology is still a considerable debate. The correlations between NDG count readings and materials density depend on several factors, such as the chemical composition of the tested material, the used test equipment, and the physical principle of measurement. Therefore, accurate calibration for the NDG can be challenging (Yin and Luo 2009; Chen et al. 2016). Researchers proposed several calibration curve fitting equations with different levels of complexity, starting from linear equations to curvilinear functions [A. Regimand and J. L. Molbert, "Method for recalibrating nuclear density gauges," US Patent No. 4,587,623 (1986); J. D. Pratt and R. L. Ely, "One-block calibration method for density gauges," US Patent No. 4,791,656 (1988); Morris and Williams 1990; Zha 2000; R. E. Troxler, W. L. Dep, and W. F. Troxler, "Apparatus and method for calibration of nuclear gauges," US Patent No. 6,369,381 (2002)]. However, none of them showed accurate predictions (Chen et al. 2016). Other researchers tried to calibrate it in the field. However, due to high variability and nonuniformity of field materials and the narrow range of densities, these trials were unsuccessful (Smith et al. 1969). Yin and Luo (2009) reinvestigated the accuracy of calibration procedures, including the standard calibration block method for NDG. They concluded that the currently used calibration method could not consistently provide the best result for various types of soils (sandstone, shale, gravel, and clay). Moreover, the chemical constitution of the tested material contributed to the device inconsistency. A new method for calibrating the NDG was then proposed to improve the accuracy of density estimations, which utilized four standard calibration blocks instead of three. The new method showed more accurate predictions for all studied cases (i.e., different types of geomaterials). However, due to many previously mentioned factors affecting the device readings in NDG, it is difficult to reach a calibration table that works for different kinds of materials (Chen et al. 2016).

Since the NDG works by counting the number of attenuated photons when gamma radiation passes through tested materials, the presence of water in tested materials (represented by the presence of hydrogen atoms in it) that have a different ratio of electron/atoms from other atoms may significantly affect the NDG density predictions. Chen et al. (2016) studied the effect of the presence of hydrogen atoms in the tested material on the NDG measurements. In this study, both photon and neutron nuclear sources were used to investigate the photon absorption of several mixes with different densities and MCs. The authors then analyzed the device's measurements in the light of known density and hydrogen contents. Correspondingly, they decoupled the effect of hydrogen content and density on the photon count number. Therefore, they proposed a decoupled model that can accurately predict the actual bulk density from photon count. The decoupled model provided a satisfactory assessment for all the test samples with different moistures and densities.

NDG measurements are usually considered as ground truth values in many studies concerned with implementing other QC/QA devices. Early studies on strength/stiffness devices focused on developing a relationship between the measured soil strength/stiffness and dry density (Ayers et al. 1989; Harison et al. 1989; Erchul and Meade 1990; Knutelski 2002; Salgado and Yoon 2003). They aimed to use these correlations to predict the field dry density and then compare it with the Proctor test results. However, conversion between soil stiffness/strength and density proves to be quite tricky without significant information about the soil of interest. Meehan et al. (2012) indicated that the NDG showed the most consistent increase in average measured values with increasing compaction energy compared to other modulus-based devices (i.e., soil stiffness gauge, LWD, and DCP). Moreover, NDG results showed significantly lower coefficient of variation (COV) compared to other devices, though modulus results correlated poorly to the NDG dry density.

The accuracy of NDG estimation has been investigated when used on new construction materials. The NDG overestimated the field MC due to misinterpretation of inheriting hydrogen atoms in crushed concrete and reclaimed asphalt pavement (RAP) as water molecules (Viyanant et al. 2004; Huber and Heyer 2019). Grubb et al. (2008) found that the NDG underestimated the

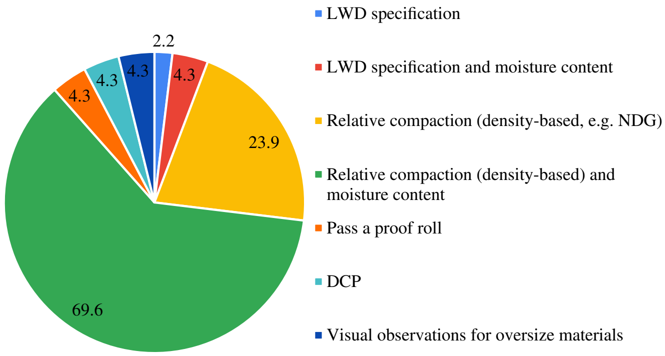

Fig. 1. Agencies' current specification for compaction control of unbound materials (percentage shown on related color; summation is greater than 100% because participants are allowed to choose more than one option).

field MC of crushed glass-dredged material (CG-DM) blends, contrary to theoretical expectations. Conversely, the NDG showed acceptable accuracy when used on subgrade treated with lime kiln dust and bottom ash materials (Gautreau et al. 2009; Jung et al. 2009). Besides, Fratta and Kim (2015) used the NDG to investigate the possibility of reducing the compaction lift thickness without sacrificing the mechanical performance of compacted material. Results showed that, when a quality management program is applied, a lift thickness of 0.30 m (12 in.) could be implemented for most soils (i.e., fine- and coarse-grained soils). In addition, Wersäll et al. (2017) used the NDG to identify the optimum compaction frequency of the vibratory roller to be 18 Hz compared to 31 Hz standard operation frequency. Lower frequency showed better compaction results and can increase the lifespan of the roller.

According to previously published surveys, most of the state DOTs have been using the NDG for density-based compaction control of different kinds of unbound materials (Tutumluer 2013; Nazzal 2014; McLain 2015; Nazarian et al. 2015). A recent survey conducted by Riad et al. (2021) showed that 43 DOTs (94% of the responded DOTs) are using density-based specifications by means of NDG, as presented in Fig. 1. Among these 43 DOTs, 32 require the compacted layer to satisfy both relative compaction and MC requirements, while 11 state DOTs require it to meet relative compaction requirements only. Moreover, 40 DOTs indicated developing the target density value using laboratory standard/modified Proctor compaction tests (ASTM, AASHTO, or locally modified versions) for this purpose. Michigan DOT is using One-Point Michigan Cone to determine the maximum density on site. For oversize aggregates, five states are using the control strip method. Besides, three states identify the target density by means of a one-point field test. All the responded DOTs (37 DOTs) are using laboratory standard/modified Proctor compaction tests (ASTM, AASHTO, or locally modified versions) to develop the target MC (Riad et al. 2021).

The main downside of the NDG is containing radioactive materials. Consequently, strict procedures shall be employed during handling, storing, monitoring, calibrating, maintaining, and transporting, and using it (Puppala 2008; Vennapusa 2008; Nazzal 2014). Radiation safety training is required for all the authorized operators of NDG devices to get familiar with the safety procedures and regulations. The cost of owning an NDG is around \$15,000 (in 2020), and it can be as high as \$2,200 per year after adding the annual cost of calibration and routine safety procedures needed to maintain the gauges (Cho et al. 2011; Nazzal 2014). Moreover, the use of NDG may also include many prohibitive measurements (i.e., licensing and relicensing, record keeping, and storage) (Volovski et al. 2014). In addition, environmental contaminations may occur due to improper disposal of broken NDGs, and in some cases the cleanup cost several million dollars (Gamache 2007). According to a survey on 18 state DOTs, the frequency of NDG testing reduced by 40% from 2013 to 2014 due to these drawbacks (McLain 2015). McLain and Gransberg (2016), based on life cycle cost per test index, suggested that replacing the NDG with an alternative testing device for many long-term benefits including saving operations and maintenance funding for other purposes. In the surveys conducted by Tutumluer (2013) and Nazzal (2014), most of the responded agencies showed interest in utilizing non-nuclear density-based devices for compaction QC of the unbound aggregates owing to the aforementioned reasons.

Non-Nuclear Devices

During the past decade, many devices were proposed and evaluated for density measurement, focusing on using nonradioactive materials. The available devices can be categorized into two main groups based on the technology used to determine the soil density: (1) electrical-based devices and (2) volume replacement-based devices. Table 1 shows the devices included in each category along with the available ASTM standard. Berney et al. (2013) assessed the performance of several electrical-based devices [i.e., soil density gauge (SDG), electrical density gauge (EDG), and time domain reflectometry-based moisture density indicator (MDI)] and indicated that the SDG had the best overall performance, in terms of effectiveness, accuracy, and precision compared to the NDG. The SDG also performed better in granular soils compared with fine-grained soil (Berney et al. 2013). Among all the volume replacement-based density devices, the sand cone was the best volume replacement device and the steel shot replacement was the worst (Berney et al. 2013).

Several concerns have been raised regarding using the electrical-based devices (i.e., MDI, EDG, and SDG), such as time-consuming calibration and testing (e.g., MDI, EDG) (Sallam et al. 2004; Rathje et al. 2006; Runkles et al. 2006; Jackson 2007; Berney et al. 2013); more cumbersome operating process (e.g., MDI) (Rathje et al. 2006; Berney et al. 2013); low repeatability and accuracy compared to

Table 1. Available NDG density-based alternatives

Category	Included devices	ASTM test method
Electrical-based devices	Electrical density gauge	N/A
	Time-domain reflectometry-based moisture density indicator	N/A
	Soil density gauge	ASTM D7380/D7380M (ASTM 2015d)
Volume replacement-based density devices	Sand cone density	ASTM D1556 (ASTM 2015b)
	Steel shot replacement	N/A
	Water balloon	ASTM D2167 (ASTM 2015a)
Nuclear	Low nuclear density gauge	N/A

the NDG (e.g., MDI, EDG, and SDG) (Brown 2007; Mooney et al. 2008; Gamache et al. 2009; Ooi et al. 2010; Berney et al. 2011; Meehan and Hertz 2011; Nazzal 2014); limitations on the types of soil it can test (e.g., MDI, SDG) (Gamache et al. 2009; Berney et al. 2013); difficulty to build a satisfactory calibration model (e.g., SDG, EDG) (Rathje et al. 2006; McLain 2015); no testing standard available (e.g., EDG, MDI), and incapability to test frozen soils (e.g., MDI, EDG, SDG) (Nazzal 2014). Researchers also indicated limitations for the volume replacement-based devices (Sebesta et al. 2006; ASTM 2015b, a; Meehan and Hertz 2011; Dessouky et al. 2012; Berney et al. 2013). These limitations include: (1) they need to be accompanied by an MC device [e.g., sand cone density (SC), steel shot replacement (SS), water balloon (WB)]; and (2) the SS exhibits the greatest overall variability in accuracy and precision.

Recently, Mata et al. (2018) proposed the use of a low nuclear density gauge (LNDG) for field density measurements. The LNDG utilizes a low activity gamma-ray source, not within the reportable limits, to perform the density measurement in the same way as traditional NDG. The LNDG is accompanied by a separate moisture probe that utilizes an electromagnetic source for MC measurement using the same hole that is prepared for density measurement. However, the LNDG exhibited testing depth limitation [limited to 20 cm (8 in.)] and requires longer testing time (twice that of the NDG). Its smaller radioactive source showed sensitivity to naturally occurring radiation, and its service life is shorter than that of the NDG (Mata et al. 2018). In addition, the eGauge was recommended by the US Army Engineer Research and Development Center as a better replacement for the NDG for wet or dry density measurements (Berney and Mejias-Santiago 2017). Although many alternatives were proposed for the NDG, none of them was implemented into specifications or extensively used by transportation agencies for compaction QC/QA.

Moisture Content Measurement

In addition to the density measurement, the NDG can measure the MC of the soil. The NDG source releases many high-speed neutrons into the soil to be tested. The hydrogen atoms in water then interact with these neutrons and slow them down. The device can then compute the MC by detecting the number of slow neutrons; see ASTM D6938 (ASTM 2017). Some state agencies (for example, Missouri) require the determination of the moisture-density relations of soils using testing methods, including AASHTO T 99 in their construction specifications.

However, due to the same limitations mentioned earlier, most transportation agencies are putting tremendous research efforts into finding non-nuclear alternatives for field MC determination. Table 2 summarizes the most common non-nuclear devices and the corresponding advantages and disadvantages. According to a recent survey (Riad et al. 2021), the commonly used methods to assess the field MC in most DOTs are (1) the NDG, (2) speedy moisture

Table 2. Most common nonnuclear moisture content devices

Device	Advantages	Disadvantages	References
Microwave oven	Fast drying, simple testing concept, provides a fairly reliable MC estimation, and standard ASTM D4643 (ASTM 2008)	Causing outbound water in clay minerals, needing a power source, suitable only for materials finer than 4.75 mm (0.19 in.), not suitable for high MC silts and clays, and operator dependent results	Sallam et al. (2004), Petersen et al. (2007), and Berney et al. (2011)
Speedy moisture tester	Fast drying, simple testing concept, relatively accurate, and standard ASTM D4944 (ASTM 2004)	Not suitable for high plasticity clay and coarse-grained soils, and calcium carbide is a hazardous material	Petersen et al. (2007), Berney et al. (2011), and Sotelo et al. (2014)
DOT600 roadbed	Portable, easy to use, and quick test	Readings are affected by soil type and by soil salinity, not accurate, and no standard available	Berney et al. (2011), and Nazarian et al. (2015)
Open flame gas burner	Combines heating of the laboratory oven and the convenience of the microwave oven test, small and portable, suitable for high moisture soils (silt and clay), and standard ASTM D4959 (ASTM 2000)	Relatively unsafe due to using fire, the operator needs to stay near the device all the testing time, used balance needs a power source, and proper fuel supply must be available	Berney et al. (2011)
Dielectrics: trident moisture meter	Very accurate device, easy and quick test, and portable devices	Low precision, requiring soil-specific calibration, not suitable for highly plastic clays and large aggregates (>25 mm), complex and time-consuming calibration and operation, hard to derive probes into stiff soil, and no standard available	Rathje et al. (2006), Berney et al. (2011), and Nazzal (2014)
Time domain reflectometry	Provide accurate results, results are not operator dependent	Time-consuming in terms of calibration and testing, limited types of soil to be tested, more cumbersome operation than other methods, hard to derive probes into stiff soil not suitable for highly plastic clays, and no standard available	Brown (2007), Berney et al. (2011), and Nazzal (2014)
Moisture analyzer	Accurate test, easy to use, and results are not operator dependent	Not feasible for aggregates exceeding 1-in. in diameter, takes 30 min for one test, needing soil specific calibration, needing a power source on site, accuracy decreases as plasticity of soil increased, and no standard available	Berney et al. (2011), and Schwartz et al. (2017)

device, (3) hot plate method, (4) portable scale and field stove, (5) collect soils and test in the lab, and (6) Ohaus moisture analyzer. Nebraska DOT indicated testing many different probe-type moisture meters but did not succeed.

Problems Associated with Density-Based Methods

Density-based QC specifications are widely used due to their relative simplicity and practicality. However, there are some theoretical and practical challenges in using it. For instance, the target maximum dry density (MDD) is typically identified by performing standard or modified Proctor tests. However, the energy specified in these test methods, first developed several decades ago, does not accurately represent the field compaction energy levels (Davich et al. 2006). In addition, the eventual objective of compaction is not to reach higher densities, but to improve the mechanical properties of compacted materials. However, the mechanical properties such as strength and stiffness and density are not closely correlated (Schwartz et al. 2017; McLain and Gransberg 2019) as they should be.

Moreover, dry density should not be used as the only criterion for compaction/ construction QC since the same density can be achieved for at least two different MCs on either side of the compaction curve. There is a need to use compaction control specifications that can be correlated to the pavement structural performance, such as strength or stiffness-based specifications. In addition, the density-based devices showed poor behavior when used on new materials such as RAP (Viyanant et al. 2004; Smith and Diefenderfer 2008; Akmaz et al. 2020). This could be attributed to the fact that RAP absorbs gamma rays from the NDG radioactive source, which affects the MC results.

Strength and Stiffness-Based QA Methods

Resilient modulus (M_R) was first introduced by the *Guide for the Design of Pavement Structures* (AASHTO 1986) as an essential input for the pavement mechanical analysis. The M_R is indicative to the aggregate layer's ability to dissipate the stresses from wheel loads. For an accurate representation of compacted layer properties, a device should report the modulus value rather than stiffness (Briaud and Seo 2003). The interest in moving toward strength/ stiffness compaction control specifications has been growing due to the transition from empirical to mechanistic-empirical pavement design procedures (Tutumluer 2013; Nazzal 2014). According to a recent survey (Riad et al. 2021), 23% (9) of 40 responded state DOTs are using AASHTO 1993/1998 for pavement design, 42% of the DOTs (17) have been using the AASHTO mechanistic-empirical pavement design (MEPDG), and 30% (12) of DOTs showed interest in using the AASHTO MEPDG for future pavement design purposes (Fig. 2). Two state DOTs (i.e., Texas and Alaska) developed their own mechanistic pavement design methods. In terms of the means to determine pavement layers' resilient moduli, which are required inputs for the MEPDG, the most common methods were the laboratory resilient modulus tests according to AASHTO T307 (12 DOTs out of 39 responded) and presumptive values based on soil type (13 DOTs), as shown in Fig. 3. A total of 23 respondents indicated that moduli are back-calculated from field tests utilizing soil resistance value (R-value), CBR, falling weight deflectometer (FWD), or DCP. Two states indicated performing laboratory resilient modulus based on AASHTO T92 or T294. The survey also found that when using the MEPDG, 88% of the respondents consider the moduli of different layers (multilayer system) in the new pavement design, and 78% of them consider the moduli of different layers during pavement rehabilitation design.

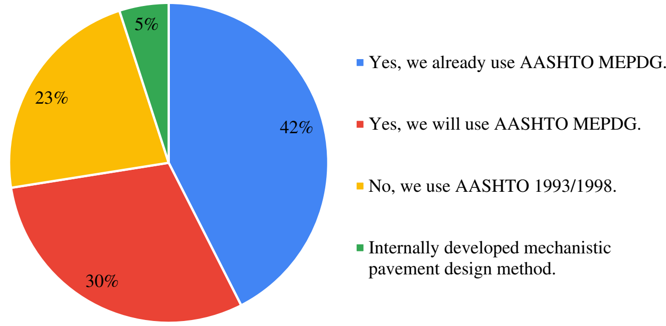

Fig. 2. Agencies' current used pavement design procedures.

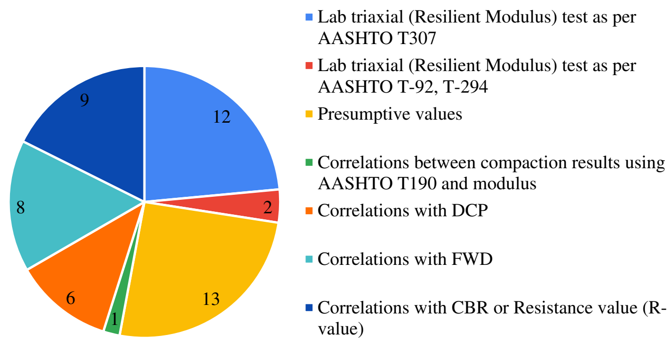

Fig. 3. Agencies' current procedures to determine the resilient moduli of subgrade/unbound materials.

Many states have conducted studies to investigate the use of stiffness- and strength-based compaction QA procedures for unbound aggregates that can be correlated to the pavement design (e.g., Choubane and McNamara 2000; Knutelski 2002; Ping et al. 2002; Salem 2004; Puppala 2008; Siekmeier et al. 2009; Chang and Gallivan 2011; Commuri et al. 2012; Tutumluer 2013; Nazzal 2014; Volovski et al. 2014; McLain 2015; Nazarian et al. 2015; Mehta and Ali 2016; Schwartz et al. 2017; Zhao et al. 2018; McLain and Gransberg 2019). These states included Georgia, Idaho, Indiana, Kansas, Maryland, Minnesota, Missouri, Mississippi, New York, New Jersey, North Carolina, North Dakota, Oklahoma, Pennsylvania, Texas, Virginia, and Wisconsin. These efforts led to the development of many devices for testing of unbound materials in the field (Puppala 2008; Tutumluer 2013; Nazzal 2014; MHTC 2019). These devices can be divided into four main categories: (1) penetration devices such as DCP and CH; (2) devices that estimate the soil stiffness by measuring the deflection resulting from applying static, vibratory, or impact load such as the Briaud compaction device, the soil stiffness gauge (SSG), and the lightweight deflectometer; (3) geophysical-based devices such as the portable seismic property analyzer; and (4) buried sensors that measure the improvement in compression waves' magnitude during compaction, such as the soil compaction supervisor.

Soil Compaction Supervisor

The SCS consists of a disposable sensor connected to a control unit powered by a battery. When testing, the sensor is embedded at the bottom of the soil layer to be compacted. The control unit then monitors the sensor's voltage and reports when a specific voltage is reached corresponding to a targeted stiffness. The SCS sensor

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could provide readings to approximately 762 mm depth (Farrag et al. 2005). The SCS is a simple device and can be operated with minimum training. Limited international research is available on using the SCS for compaction QC purposes, though Miller and Mallick (2003) indicated that the SCS performed well as a compaction QC/QA tool with strong correlations achieved between the SCS signals and the relative compaction. However, the SCS is not the best tool to be used with the lightweight aggregates due to the inability of this material to transmit compression waves effectively to the SCS. State DOTs had not evaluated the SCS yet (Rathje et al. 2006).

Portable Seismic Property Analyzer

The PSPA is a portable version of the conventional seismic pavement analyzer that used to be large and mounted on a trailer. It utilizes ultrasonic surface waves to determine the tested layer's modulus. The PSPA has been used extensively for evaluating the quality of construction of both rigid and flexible pavements (Mallick and Nazarian 2007; Li and Garg 2015). The PSPA is small, portable, and easy-to-handle (Mallick and Nazarian 2007), and the test is quick. It provides reliable seismic modulus for RAP layers (Mallick and Nazarian 2007), and is relatively simple to obtain the target field modulus (Nazzal 2014). However, for data acquisition and reduction, the device has to be connected to a laptop in the field. The data analysis process also requires a skilled operator. Moreover, the calibration procedure is very complicated and soil-specific. A complex resonant column-torsional shear laboratory testing needs to be performed to calibrate the device. The PSPA does not have a standard testing method, it was not suitable for compaction control for gravel soils (Rathje et al. 2006), and it is considerably more expensive than most of the in situ testing devices, including the NDG (Nazzal 2014). Very limited research is available for using the PSPA to evaluate the compaction QC/QA of unbound materials.

Briaud Compaction Device

The BCD consists of a readout unit on top of a rod. Attached to the bottom of that rod, there is a flexible plate with 150-mm diameter that has several strain gauges. It works by applying a small load (mostly human weight) to the material to be tested through the thin plate. The plate then bends due to applied loads, and the strain gauges (eight radial and axial gauges) measure the bending strains for computing the BCD low-strain modulus (Weidinger and Ge 2009). Briaud et al. (2009) compared results obtained from the BCD (E_{BCD}) with static plate tests (E_{PLT}) in the field and with maximum dry density, plate modulus, and resilient modulus in the lab. Their study confirmed the ability of the BCD to measure a soil modulus, and strong correlations were found between the E_{BCD} and E_{PLT} and between the resilient modulus (M_R) and the E_{BCD}. The BCD device is very light (a few kilograms), and the test is fast (approximately 5 s) and simple (Mehta and Ali 2016). The target field modulus can be determined in the laboratory (Nazzal 2014); its modulus has reasonable correlations with laboratory and field moduli (Mehta and Ali 2016) with relatively good repeatability (Briaud et al. 2009; Weidinger and Ge 2009).

However, it is a relatively new device and it has not been involved in many research studies to evaluate its performance compared to other devices (Nazzal 2014). Due to the limited range of capacity, it is not suitable for very soft or very stiff soils (Nazzal 2014). It works on geomaterials with moduli up to 150 MPa, eliminating most unbound base materials (Nazarian et al. 2015). Mehta and Ali's (2016) study concluded that the BCD was not a suitable device to replace the NDG. The BCD was not implemented into any compaction QA specifications.

Clegg Hammer

The CH was initially developed by Dr. Baden Clegg at the University of Western Australia in Nedlands in the 1960s to measure the hardness of a surface (Rathje et al. 2006). It was then used widely in Australia and New Zealand. It consists of a precision accelerometer attached at the end of a hammer, and they are entailed into a guide tube. The accelerometer measures the deceleration of the free-falling hammer upon contact with the soil surface and sends digital signals to the readout unit. The largest deceleration measured during a test is called the Clegg impact value (CIV), which is the main output of the device; see ASTM D5874 (ASTM 2016). Equations were proposed to determine the elastic modulus from the CIV readings (Clegg 1994). However, in practice, CIV is still used as the target field value during compaction construction. The CH has been successfully used to assess the stiffness of various types of natural and synthetic materials (Holt et al. 2014; Arulrajah et al. 2018; Mohajerani et al. 2020). Researchers developed many correlations between the CIV value and other mechanical parameters of soils [e.g., California bearing ratio (CBR)] (Clegg 1980; Mathur and Coghlan 1987; Al-Amoudi 2002; Al-Amoudi et al. 2002; Aiban and Aurifullah 2007; Fairbrother et al. 2010; Pattison et al. 2010), resilient modulus (M_R) (Mohajerani et al. 2016), unconfined compressive strength (Janoo et al. 1999), relative compaction (Farrag 2006). However, all the correlations are empirical and soil/site subjective and cannot reach a reasonable match when applied on other soils (i.e., not the soil used to develop the correlation).

In a survey conducted by Nazzal (2014), 15% (six states among 40 state DOTs involved) of the responded states used or evaluated the CH. These states included Colorado, Florida, Indiana, Maine, Texas, and Virginia. In addition to these states, the Gas Technology Institute studied and proposed modifications to the CH to improve its performance (Farrag 2006). The Main Roads Western Australia (2012) successfully used the CH for QC purposes by means of providing a relationship between CIV values and base course strength/stiffness. Medium repeatability for both the 10 and 20 kg hammers was indicated by Rathje et al. (2006). The CH's influence depth ranges between 200 to 300 mm (Farrag et al. 2005; White et al. 2007; Mooney et al. 2008). The CH is a quick test (less than 60 s), simple, not operator-dependent, and has a standard test procedure (Nazzal 2014). The results have good correlations with the California bearing ratio (Aiban and Aurifullah 2007; Fairbrother et al. 2010). Results also indicated that the CH reasonably reflects the degree of compaction, especially for granular materials (Erchul and Meade 1990). However, several limitations were reported in previous studies (Farrag et al. 2005; Steinert et al. 2005; Rathje et al. 2006; Mooney et al. 2008; Kim et al. 2010). These included (1) poor portability and mobility; (2) the target CIV obtained from testing on proctor mold found to be inaccurate; (3) for soft soils, the hammer penetrates the ground quickly and gets stuck; (4) it has considerable variability; (5) shallow bearing capacity failure sometimes occur; and (6) CIV values have weak correlations with the density of coarse-grained soils. Although several states evaluated the CH for QC/QA purposes, it was not implemented into any of the specifications.

Soil Stiffness Gauge

The SSG, or GeoGauge, measures the surface stiffness of soils by generating a tiny dynamic force at high frequencies (i.e., from 100 to 196 Hz). The device's rate of production is one test per 1.25 min, which can be considered a fast test. The influence depth ranges from 127 to 254 mm (Maher et al. 2002; Abu-Farsakh et al. 2004), and the generated force was estimated to be 9 N (Sawangsuriya et al. 2002). Many advantages were reported for the SSG. These advantages include: (1) it has good portability, durability, and data storage (e.g., Maher et al. 2002; Abu-Farsakh et al. 2004; Von Quintus 2009; Hossain and Apeagyei 2010; Nazarian et al. 2015); (2) it is user-friendly, durable, simple and requires minimal training to perform (Abu-Farsakh et al. 2004); (3) it is fast (1.25 min per test); and (4) it has a test specification per ASTM D6758 ASTM 2018a). However, some of the SSG limitations were reported. The main limitations were (1) it applies a minimal load, which does not represent the actual traffic load; (2) the device showed significantly inconsistent results that are operator dependent (Ellis et al. 2001; Senseney et al. 2008); (3) the readings are sensitive to the first 5 cm (2in.) of the tested soil and the seating procedure (Ellis et al. 2001; Farrag et al. 2005); (4) it is hard to achieve good contact with the soils surface (Ellis et al. 2001; Knutelski 2002); (5) there is no definitive correlation with other mechanical properties, such as resilient modulus; (6) the accuracy of results is highly dependent on the surface soils conditions (i.e., upper part of compacted soil) (Ellis et al. 2001; Pu 2002; McLain 2015); (7) it had lower reliability when applied on recycled aggregates (Ooi et al. 2010); and (8) the test readings are highly affected by the soils' moisture content (Chen et al. 1999; Siekmeier et al. 2000; Ellis et al. 2001; Maher et al. 2002; Pu 2002; Lenke et al. 2003; Abu-Farsakh et al. 2004; Zhang et al. 2004; Petersen and Peterson 2006; Mohammad et al. 2009; Ooi et al. 2010). Several state agencies (e.g., Florida, Hawaii, Louisiana, Minnesota, New Jersey, New Mexico, Missouri, and Texas) and researchers performed studies to investigate the effectiveness and accuracy of the SSG and correlations with other devices (Sawangsuriya 2001; Sawangsuriya et al. 2002; Alshibli et al. 2005; Quinta-Ferreira et al. 2012; Lee et al. 2014). A number of studies (Wu et al. 1998; Quinta-Ferreira et al. 2012; Choi et al. 2020) have demonstrated that SSG has great potential for use as an alternative QC device with good correlations with dry density. However, it has not been implemented for compaction QA specifications.

Dynamic Cone Penetrometer

The DCP (Fig. 4) was first developed in South Africa (Kleyn 1975). Since then, it has been used in the United States and internationally for site characterization of pavement layers and subgrades. The DCP is also considered as a nondestructive testing equipment,

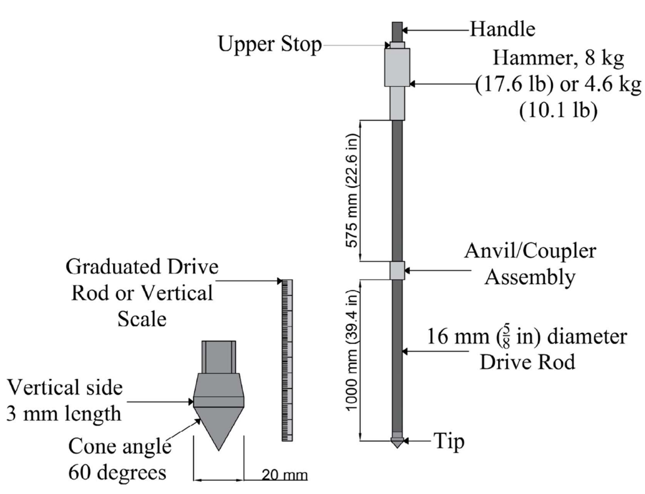

Fig. 4. Dynamic cone penetrometer.

and it was widely used in subgrade of the high-speed rail. The DCP test is performed by recording the number of blows versus depth when a weight is freely dropping from 575-mm height (ASTM D6951 or ASTM D7380/D7380M). The number of blows versus depth is called the penetration rate (PR). The influence depth of the DCP can be as great as 1.2 m (Mooney et al. 2008). Several studies showed that the DCP has good repeatability (Dai and Kremer 2006; Petersen and Peterson 2006). However, other studies indicated that the DCP measurements had a relatively high COV (Siekmeier et al. 2009; Von Quintus 2009).

Correlations with Other Soil Parameters and Other In Situ Testing Devices

Tremendous efforts have been spent to correlate the DCP PR to other mechanical properties for different geomaterials. These properties included resilient modulus (M_R), relative compaction (RC), California bearing ratio, unconfined compressive strength (UCS), and shear strength. Tables 3 and 4 summarize several PR-CBR and PR-M_R correlations for different materials in both laboratory and field. For example, Yang et al.'s (2016) proposed equations were developed based on laboratory testing but verified in the field, considering water content changes with high R^2. Many correlations between PR and other material parameters (e.g., UCS, relative density, dry density, moisture content, modulus of elasticity, shear modulus, shear strength, and angle of internal friction) can be found elsewhere (e.g., Salgado and Yoon 2003; Mohammadi et al. 2007; Puppala 2008; Ampadu and Fiadjoe 2015). Meshram et al. (2016) developed correlations to predict the in situ density of subgrade materials from PR and MC with a coefficient of determination of 0.95, which were then used to investigate the uniformity of subgrade materials using the DCP and field MC. Ampadu et al. (2017) investigated the effect of lateral confinement on the PR, and proposed preliminary correlations to derive the in situ relative compaction from in-mold (CBR mold) values. Note that most of the correlations were empirical and soil/site-specific. Therefore, careful examination shall be practiced when using them for other construction sites or soils.

Other researchers focused on developing correlations between PR and the measurements of other in situ devices. Siekmeier et al. (2000) utilized the DCP test, FWD, and SSG to assess the compaction quality of many subgrade and granular base materials in Minnesota. The PR was then converted into modulus using previously developed correlations. The moduli obtained from the three devices were compared to investigate each device's ability to measure the in situ stiffness accurately. The results showed a weaker correlation between the modulus values obtained from DCP and the measured values using FWD and SSG. Table 5 summarizes the developed correlations between PR and stiffness measured using other devices.

Edil and Sawangsuriya (2005) conducted SSG and DCP tests on 13 construction sites around the state of Wisconsin to investigate their use for soil property evaluation. The investigated materials consisted of natural earth, recycled and by-products materials, chemically stabilized, and other geomaterials. Correlations were developed between the PR and SSG stiffness (K_{SSG}) values and also with the density and moisture content. Then, to obtain a representative strength index, a weighted average of PR over depth of measurement is employed. The DCP penetration index was weighted over a depth of 152 mm that was equal to the influence depth of the SSG. A linear semilogarithmic relationship, with a reasonable R^2 value, was observed between the K_{SSG} and PR, as shown in Table 5.

Mejías-Santiago et al. (2015) conducted a comprehensive study on using the DCP to assess material strength and stiffness

Table 3. PR-CBR correlations

Correlation	R^2	Data points	Soil type	Testing	Reference
$\log(\mathrm{CBR}) = 2.62 - 1.27\log(\mathrm{PR})$	N/A	2,000	Unknown	Laboratory	Kleyn (1975)
$\mathrm{CBR} = 322.097/\mathrm{PR} - 1.738$	0.78	≈60	Clayey soils	In situ	Smith and Pratt (1983)
$\log(\mathrm{CBR}) = 2.555 - 1.145\log(\mathrm{PR})$	0.85	≈60	Clayey soils	In situ	Smith and Pratt (1983)
$\log(\mathrm{CBR}) = 2.81 - 1.32\log(\mathrm{PR})$	0.90	72	Various soils	In situ	Harison (1986)
$\log(\mathrm{CBR}) = 2.56 - 1.16\log(\mathrm{PR})$	N/A	72	Cohesive soils	Laboratory	Harison (1986)
$\log(\mathrm{CBR}) = 3.03 - 1.51\log(\mathrm{PR})$	N/A	72	Granular soils	Laboratory	Harison (1986)
$\mathrm{CBR} = 0.078969 \times FR + 0.211765 \times TP$	0.9146	>50	Wide range of soils	Laboratory	Webster et al. (1994)
$\mathrm{CBR} = 1/(0.002871 \times \mathrm{PR})$	0.9802	>50	CH soils	Laboratory	Webster et al. (1994)
$\mathrm{CBR} = 1/(0.017019 \times \mathrm{PR})^2$	0.9362	>50	CL soils	Laboratory	Webster et al. (1994)
$\log(\mathrm{CBR}) = 2.44 - 1.07\log(\mathrm{PR})$	N/A	75	Aggregate base course	Laboratory and in situ	Ese et al. (1994)
$\log(\mathrm{CBR}) = 2.46 - 1.12\log(\mathrm{PR})$	0.88	135	Granular and cohesive	Laboratory and in situ	Livneh et al. (1995)
$\mathrm{CBR} = 320/\mathrm{PR}^{0.943}$	0.76	77	Granular and fine grained	In situ	Truebe et al. (1995)
$\log(\mathrm{CBR}) = 2.20 - 0.71(\log(\mathrm{PR}))^{1.5}$	0.96	74	Granular and fine-grained	Laboratory	Hassan (1996)
$\log(\mathrm{CBR}) = 2.50 - 1.07\log(\mathrm{PR})$	0.69	>90	Granular and cohesive	Laboratory	Al-Refeai and Al-Suhaibani (1997)
$\log(\mathrm{CBR}) = 2.53 - 1.14\log(\mathrm{PR})$	N/A	N/A	Cohesive soils	Laboratory	Coonse (1999)
$\log(\mathrm{CBR}) = 1.40 - 0.55\log(\mathrm{PR})$	0.82	≈16	Aggregate base course	Laboratory and in situ	Gabr et al. (2000)
$\log(\mathrm{CBR}) = 2.256 - 0.954\log(\mathrm{PR})$	0.56	≈20	Granular and fine-grained	Laboratory	Abu-Farsakh et al. (2004)
$\mathrm{CBR} = 5.1/(\mathrm{PR}^{0.2} - 1.41)$	0.93	≈20	Granular and fine-grained	In situ	Abu-Farsakh et al. (2004)
$\mathrm{CBR} = 1161.1/\mathrm{PR}^{1.52}$	0.79	≈35	Granular and cohesive	Laboratory and in situ	Abu-Farsakh et al. (2004)
$\log(\mathrm{CBR}) = 2.256 - 0.954\log(\mathrm{PR})$	N/A	N/A	Granular and cohesive	Laboratory	Mohammadi et al. (2007)
$\mathrm{CBR} = 24.903/\mathrm{PR}^{1.331}$	N/A	29	Cohesive	Laboratory	Patel and Patel (2012)
$\log(\mathrm{CBR}) = 1.684 - 1.050\log(\mathrm{PR})$	0.85	30	Fine-grained soils	Laboratory	Thach Nguyen and Mohajerani (2015)
$\mathrm{CBR} = 48/\mathrm{PR}$	0.84	30	Fine-grained soils	Laboratory	Thach Nguyen and Mohajerani (2015)
$\mathrm{CBR} = 442.92/\mathrm{PR}^{1.119}$	0.87	≈28	Cohesive soils	In situ	Mejías-Santiago et al. (2015)
$\mathrm{CBR} = \begin{cases} 8745.70\mathrm{PR}^{-4.4982}; MC \le MC_{opt} \\ 53.26\mathrm{PR}^{-9615}; MC \ge MC_{opt} \end{cases}$	0.97 0.98	>20	Cohesive soils	Laboratory	Yang et al. (2016)

Note: FR = friction ratio; TP = tip pressure; and PR = penetration rate (mm/blow).

Table 4. PR-M_R correlations

Correlation	R^2	Data points	Soil type	DCP testing	Reference
$M_R(psi) = 7013.065 - 2040.783\ln\mathrm{PR}$	0.37	≈23	Granular and fine-grained	Laboratory	Hassan (1996)
$M_R = 14932 - 452.30 \times \mathrm{PR}$	0.66	11	Cohesive and cohesionless	Laboratory	Luo et al. (1998)
$M_R = 17273 - 86.878 \times \mathrm{PR}$	0.51	11		In situ	
$M_R(ksi) = 338 \times (\mathrm{PR})^{-0.39}$	0.42	140	Cohesive	In situ	Chen et al. (1999)
$M_R = 532.1 \times \mathrm{PR}^{0.492}$	0.40	>100	Fine-grained	In situ	George and Uddin (2000)
$M_R = 235.3 \times \mathrm{PR}^{0.475}$	0.40	>100	Course-grained		
$M_R = a_0(\mathrm{PR})^{a_1}[\gamma_{dr}^{a_2} + (MC)^{a_3}]$	0.71	180	Fine-grained	In situ	Rahim and George (2002)
$M_R = b_0\left(\frac{\mathrm{PR}}{\log c_u}\right)^{b_1}[\gamma_{dr}^{b_2} + (MC)^{b_3}]$	0.72	180	Coarse-grained soils		
$M_R = 114100 - 3279 \times \mathrm{PR}$	N/A	N/A	Cohesive and cohesionless	In situ	Salgado and Yoon (2003)
$M_R = 537.76 \times \mathrm{PR}^{-0.67}$	0.855	227	Granular base	In situ	Chen et al. (2005)
$M_R = 16.28 + 928.24/\mathrm{PR}$	0.82	24	Fine-grained	In situ	Herath et al. (2005)
$M_R = 1054.9/\mathrm{PR}^{1.096}$	0.90	≈100	Cohesive and cohesionless	In situ	Mohammad et al. (2007)
$M_R = \begin{cases} 1573.60\mathrm{PR}^{-1.26}; MC \le MC_{opt} \\ 573.78\mathrm{PR}^{-0.46}; MC \ge MC_{opt} \end{cases}$	0.99 0.88	≈18	Cohesive	Laboratory and in situ	Yang et al. (2016)

Note: Units are SI units otherwise indicated. LL = the liquid limit; and $a0$, $a1$, $a2$, $a3$, $b0$, $b1$, $b2$, and $b3$ are model parameters.

parameters. Based on a database containing 185 test points collected from 24 construction sites, they investigated the accuracy of available CBR-PR and FWD-PR correlations and developed new ones. Results indicated that most of the used correlations provided a similar trend, as shown in Fig. 5(a). However, the proposed correlation by Mejías-Santiago et al. (2015) provided a better match to the collected data. Fig. 5(b) shows the correlations between FWD back-calculated resilient modulus and the PR. Correlation developed by Mohammad et al. (2007) and George et al. (2009) provided the worst match to these results. These two correlations were developed for fine-grained soils. However, the data set included course-grained soils as well. Moreover, Mohammad et al. (2007) developed a correlation between PR and resilient modulus obtained from laboratory testing, not the back-calculated from FWD results.

DCP for Compaction QA

The available studies correlated PR with other material mechanical properties (e.g., CBR, M_r, E) showed that the DCP is an adequate tool for assessing the quality of compaction for pavement applications. It offers a viable alternative to other more complex and time-consuming procedures for characterizing subgrade soil

Table 5. Correlations between PR and stiffness parameters measured using other in situ devices

Other devices	Correlation	R^2	Data points	Soil type	Reference
FWD	$E_{FWD} = 338 \times (\mathrm{PR})^{-0.39}$ for 10 mm/blow < PR < 60 mm/blow	0.42	140	Subgrade and base materials	Chen et al. (1999)
PLT	$E_{PLT} = -0.34(N_{\mathrm{DCP}})^2 + 13{\cdot}97\ N_{\mathrm{DCP}} - 13.67$ $2 < N_{\mathrm{DCP}} < 15$; N_{DCP} is no. of blows per 10 cm	0.85	≈46	Backfill and subgrade soils	Zhang et al. (2004)
FWD	$M_{FWD} = 21.64(N_{DCP})^{0.699}$, $2 < N_{DCP} < 15$	0.89	≈46		
FWD	$E_{\mathrm{FWD}} = 537.76 \times (\mathrm{PR})^{-0.6645}$	0.855	227	Granular base	Chen et al. (2005)
SSG	$K_{SSG} = 17.9 - 7.54\log(\mathrm{PR})$	0.60	≈80	Natural earthen materials	Edil and Sawangsuriya (2005)
	$K_{SSG} = 19.30 - 8.30\log(\mathrm{PR})$	0.55	≈30	Granular materials	
	$K_{SSG} = 17.10 - 7.10\log(\mathrm{PR})$	0.64	≈80	Fine-grained soils	
	$K_{SSG} = 26.40 - 11.10\log(\mathrm{PR})$	0.47	≈40	Fly-ash stabilized soils	
	$K_{SSG} = 25.6 - 12.0\log(\mathrm{PR})$	0.72	≈200	Wide range of materials	
PLT	$E_{PLT(i)} = 9770/(36.9 + \mathrm{PR}^{1.6}) - 0.75$	0.67	≈50	Cohesive and cohesionless soils	Abu-Farsakh et al. (2005)
PLT	$E_{PLT(R)} = 4374.5/(14.9\mathrm{PR}^{1.4}) - 2.16$	0.78	≈50		
FWD	$\ln(M_{FWD}) = 2.35 + 5.21/\ln(\mathrm{PR})$	0.91	≈50		
FWD	$M_{FWD} = 668.27/\mathrm{PR}^{-0.556}$	0.65	≈30	Cohesive and cohesionless soils	Mejías-Santiago et al. (2015)
IC	$CMV = 61.304–2.4021\mathrm{PR}$	0.219	>90	Various soils	Foroutan et al. (2020)

Note: M_{FWD} = resilient modulus back-calculated from FWD; K_{SSG} = soil stiffness (MN/m); $E_{PLT(i)}$ = initial modulus; $E_{PLT(R)}$ = reloading modulus; IC = intelligent compaction; and CMV = compaction measurement value from IC test.

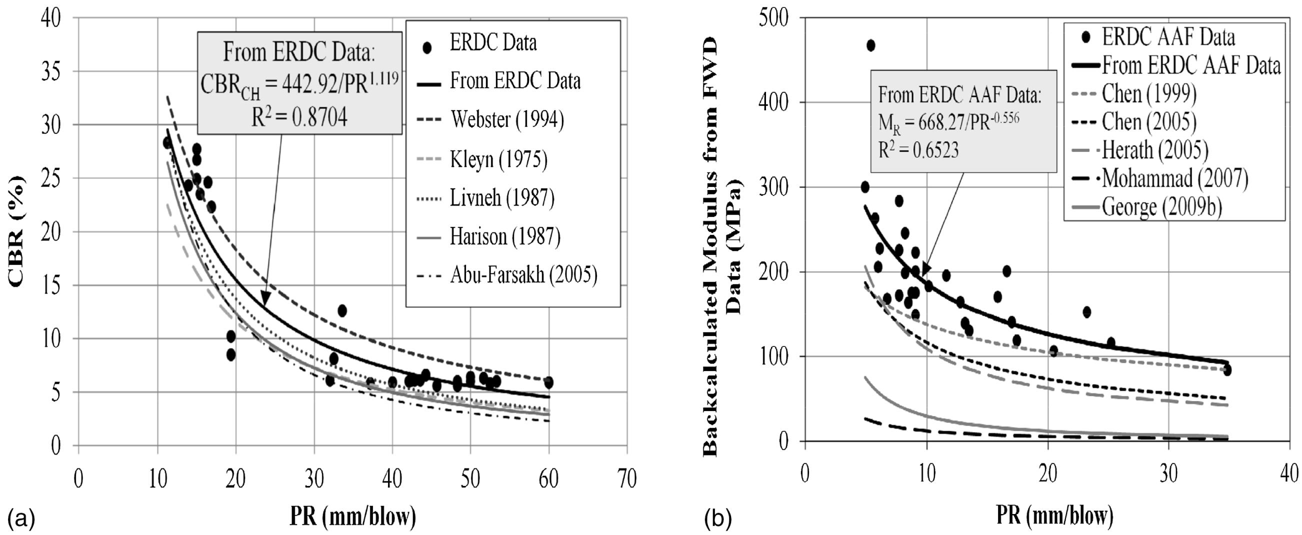

Fig. 5. Correlations between PR and other soil parameters: (a) CBR-PR correlations for high plasticity clay; and (b) FWD modulus and PR for course and fine-grained soils. (Reprinted from Mejías-Santiago et al. 2015, © ASCE.)

(George and Uddin 2000). However, studies concluded that the DCP should not be utilized as a replacement for the in situ density testing such as NDG (Burnham 1997; Rathje et al. 2006; Berney and Kyzar 2012; Cho et al. 2012). Instead, they recommended the DCP to be used as a supplementary tool for compaction QC/QA, as another device should be accompanied by the DCP for field MC measurements that is often a critical requirement for QC/QA. Another concern was that the DCP could not distinguish the density increase with different compaction levels for a given soil type (Berney and Kyzar 2012).

Despite these concerns, a survey conducted by (Nazzal 2014) indicated that the DCP has been used by around 50% (20 states out of 41) of the state DOTs for compaction QC/QA. Twelve of these state DOTs (60%) indicated that the DCP had very good repeatability and accuracy. Several US agencies (e.g., Missouri, Texas, Minnesota, Mississippi, Louisiana, Florida, Indiana, Wisconsin, Iowa, and the USACE) performed studies to investigate the effectiveness and accuracy of the DCP (Burnham 1997; Abu-Farsakh et al. 2004; Tingle and Jersey 2007; Prezzi et al. 2010; Nazzal 2014; Siekmeier 2018). Few of them (i.e., Minnesota, Missouri, Indiana, and Illinois) implemented the DCP to assess the compaction QC/QA and developed their own specifications/standard.

Minnesota has been a leading state to research the DCP since 1991. MnDOT developed specifications for using the DCP for compaction QC by means of defining a limiting PR value for each soil type (Burnham 1997). Originally, PR was the only specified parameter to assess the quality of compaction by correlating with CBR values for base aggregate. Later, the effect of other parameters, such as MC and gradation, are being considered by the modified specifications (Nazzal 2014). For the material to be accepted, MnDOT requires that the measured seating penetration (SEAT) [reading after seating (2 blows) − initial reading] values must be less than or equal to the maximum allowable values. The maximum allowable values can be selected for each material from specified tables based on the gradation number (GN) and field MC.

MnDOT also requires that the DCP penetration during testing be less than the targeted layer thickness. Other applications for the DCP testing specified by MnDOT included (1) backfill compaction of pavement edge drain trench, (2) QC of granular base layer compaction (Abu-Farsakh et al. 2004), and (3) a DCP performance-based specification for the geogrid reinforced aggregate base (Siekmeier 2018).

In 2013, Indiana DOT (INDOT) implemented the DCP to be used for various types of natural and chemically modified soils. The DCP acceptable PR is based on the tested material type, and tables are provided for the allowable lift thickness for every material along with the required PR in Indiana standard specifications (INDOT 2020). The Illinois DOT uses the DCP to justify the need for subgrade stabilization before and during construction (Nazzal 2014). They consider the subgrade material accepted if the immediate bearing value (IBV) is greater than 6% to 8%. They use an equation to correlate between the PR and IBV values, $\mathrm{IBV} = 10^{(0.84-1.26\log(\mathrm{PR}))}$. When the IBV is less than 6%, Illinois DOT requires material stabilization before construction. Missouri DOT (MoDOT) implemented the DCP to be used to assess the compaction quality of Type 7 aggregates within 24 h after construction, which consist of crushed stone, sand, gravel, or reclaimed asphalt or concrete (MoDOT 2018). MoDOT uses Type 7 aggregates under roadways and shoulders. MoDOT considers the compaction accepted if the penetration rate was less than 0.40 inches per blow, and the DCP test shall meet the ASTM D6951 requirements.

Prezzi et al. (2010) successfully used the DCP for compaction quality control of fly-and bottom ash mixtures embankment in Indiana. To establish QC criteria, PR was correlated to the field dry density and moisture content. Then the target PR values were identified, satisfying the requirement of relative compaction over 95%.

Advantages and Limitations

The DCP test has many reported advantages (Nazzal 2014). These advantages can be summarized as follows: (1) it is simple and requires minimal maintenance (durable); (2) the DCP device is relatively cheap; (3) it is easy to operate without prior calibration; (4) it has standard specifications for testing per ASTM D6951 (ASTM 2018b); and (5) it provides continuous measurement of the in situ strength (Chen et al. 2001). Moreover, studies and practices have confirmed that its results have a strong correlation with strength and stiffness properties (Parker et al. 1998; Chen et al. 1999, 2001, 2005; Salgado and Yoon 2003; Abu-Farsakh et al. 2004; Oman 2004; Rathje et al. 2006; Dai and Kremer 2006; Davich et al. 2006; Petersen and Peterson 2006; Siddiki et al. 2008; White et al. 2009; Mehta and Ali 2016). However, some limitations were also reported for the DCP. These limitations included the following: (1) it cannot be used for boulders and rocks that has particle size greater than 51 mm (ASTM D6951; Salgado and Yoon 2003; Rathje et al. 2006); (2) it requires two persons to perform the test; (3) it cannot be used in soft clay soils due to low penetration effort (Rathje et al. 2006); (4) the test results significantly affected by the MC and confinement pressure (Abu-Farsakh et al. 2004; Mehta and Ali 2016); (5) controversy conclusions were reached regarding the device readings and dry density correlations (Farrag et al. 2005; Berney et al. 2013); and (6) its results for the top 152 mm (6 in.) are not reliable due to low confinement (Farrag et al. 2005).

Lightweight Deflectometer

Device Description

The LWD (Fig. 6) is a dynamic plate loading test developed to determine the modulus (E_{LWD}) of soils and unbound fill material. The LWD works by allowing a specific weight to freely fall from a fixed

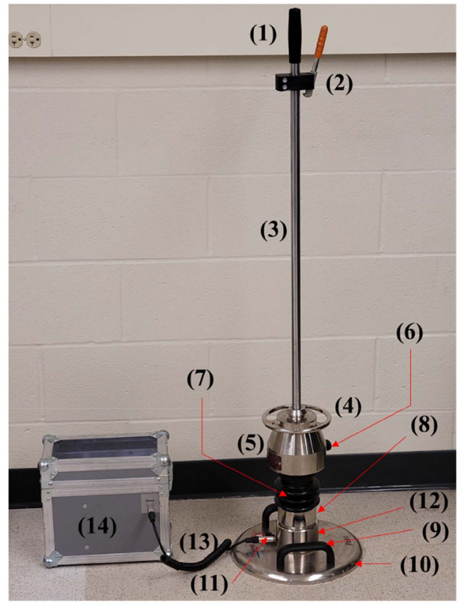

Fig. 6. Configurations of the LWD (Zorn ZFG 2000 light drop weight tester): (1) grip; (2) top fix and release mechanism; (3) guide rod; (4) round grip; (5) 10 or 15 kg falling weight; (6) lock pin; (7) set of steel springs; (8) anti-tipping fixture; (9) plate carry grip; (10) loading plate (300, 150, or 50 mm); (11) socket for connection to electronic device; (12) adapter plate; (13) connection wire; and (14) electronic data acquisition system.

height onto the loading plate according to ASTM E2835 (ASTM 2020). The plate then imposes a dynamic (impulse) load inro the material to be tested. The soil's deflection is calculated either by integrating the velocity measured using a velocity transducer or double integrating the acceleration measured using an accelerometer. The deflection and applied load can then be used to calculate the surface modulus using Boussinesq's equation (Boussinesq 1871), assuming the test media to be a linearly elastic, isotropic, and homogeneous semi-infinite continuum.

Tremendous efforts were dedicated to evaluating the effectiveness and accuracy of the LWD to be used for field compaction control purposes. These studies included but were not limited to the following areas: (1) correlating field LWD modulus with in-place density and M_R (George 2006; Petersen et al. 2007); (2) comparing results with other devices such as standard FWD, SSG, CH, DCP, and NDG (Steinert et al. 2005; Hossain and Apeagyei 2010; McLain and Gransberg 2016, 2019); (3) procedures to improve LWD data interpretation (Hoffmann et al. 2003); (4) comparing results measured from different types of LWD devices (White et al. 2007; Schwartz et al. 2017); (5) investigating influencing factors and determining target LWD deflection/modulus (Volovski et al. 2014; Khosravifar et al. 2013; Khosravifar 2015; Schwartz et al. 2017); (6) evaluating the effectiveness of the LWD in assessing the stiffness properties of various geomaterials (Abu-Farsakh et al. 2004); and (7) validating the compaction of various types of projects (Zhao et al. 2018). Efforts have also been spent evaluating the feasibility of E_{LWD} for compaction QA by many state DOTs (e.g., Kansas, Louisiana, Mississippi, Maine, Minnesota, Virginia, Indiana, Maryland, and Missouri) for various soil types.

Data Analysis and Influencing Factors

Researchers indicated that the measured E_{LWD} can be affected by many factors (Fleming et al. 2007; White et al. 2007; Vennapusa and White 2009; Fleming et al. 2009; Ooi et al. 2010). They included (1) the number of load drops, (2) drop weight and height, (3) the quality of load-deflection curves, (4) loading rate and stiffness of buffer, (5) geophone/loading plate attachment (fixed/loose), (6) size and rigidity of loading plate, (7) type and location of deflection sensor, (8) the contact stress and quality of contact between loading plate and soil surface, and (9) use of extra geophone. It is suggested that the influence depth of the LWD range between 0.90 to 1.50 times the loading plate diameter depending on soil type (Fleming et al. 2007; White et al. 2007; Vennapusa and White 2009; Ooi et al. 2010). Previous studies [e.g., Lin et al. (2006) and ASTM E2835 (ASTM 2020)] have recommended to perform the LWD test on a uniform surface to ensure good contact between the loading plate and soil surface. However, for in situ testing, the judgment of poor contact is generally subjective and difficult (Fleming et al. 2009).

Abu-Farsakh et al. (2004) performed comprehensive laboratory and field testing using the LWD on different types of geomaterials to evaluate the effectiveness of utilizing the LWD to assess the stiffness properties of these materials. They investigated the effect of time after compaction on E_{LWD} for cemented soils (2% and 4% cement content). However, the trend for change in E_{LWD} with time was not clear due to high standard deviations. In contrast, Afsharikia and Schwartz (2019) found that significant modulus gain due to drying, particularly at the first few hours, based on field study. The LWD moduli measured in the field reasonably increased with the number of passes (Fig. 7). Due to the evaporation, the soil modulus increased significantly during the first few days (3 to 4 days) and then reached a stable value (Fig. 8). Results showed that the COV ranged from 2.1% to 28.1%; however, the COV values decreased with increasing the stiffness modulus. This trend was attributed to the difficulty in performing LWD tests on weak materials. Abu-Farsakh et al. (2004) also indicated that the measured E_{LWD} could be affected by underlying layers for pavement layers less than 305 mm (12 in.) thick. In this case, they recommended that the E_{LWD} be back-calculated using multilayered system analysis.

In another study on five types of compacted geomaterials, Steinert et al. (2005) found that the E_{LWD} is affected by both percent compaction (PC) and MC. They developed a model to correlate between E_{LWD}, PC, and MC with a reasonable coefficient of determination (R^2). Other studies compared results from different portable devices (e.g., LWD, DCP, and SSG) (Apeagyei and Hossain 2010; Hossain and Apeagyei 2010). Results showed that density had a limited effect on the soil stiffness, but the MC had a significant effect on the devices' readings, particularly the LWD. Laboratory results showed that high E_{LWD} might occur at very low and very high saturation levels due to the presence of suction and excess pore water pressure, respectively. According to these studies, it is not recommended to use the LWD for construction QC before further research to evaluate the reasons for high spacial variability and the effect of MC. A similar conclusion was reached by Schwartz et al. (2017).

Accordingly, efforts have been spent on applying different theories, approaches, and tools for LWD data collection and analysis. To address the concern on LWD's capability to accurately characterize multilayered soil systems, Senseney and Mooney (2010) utilized the LWD with radial sensors on one and two-layered systems test beds. It was able to accurately estimate the layered moduli in case of stiff-over-soft systems. A close match was shown between E_{LWD} and the elastic modulus values by means of triaxial testing at a similar stress state. However, the measured depth of influence was found to be 1.80 times the loading plate diameter compared to 1.0 to 1.50 for conventional LWDs. The authors attributed this difference to the existence of radial sensors that gave the possibility to measure strains affected to deeper materials.

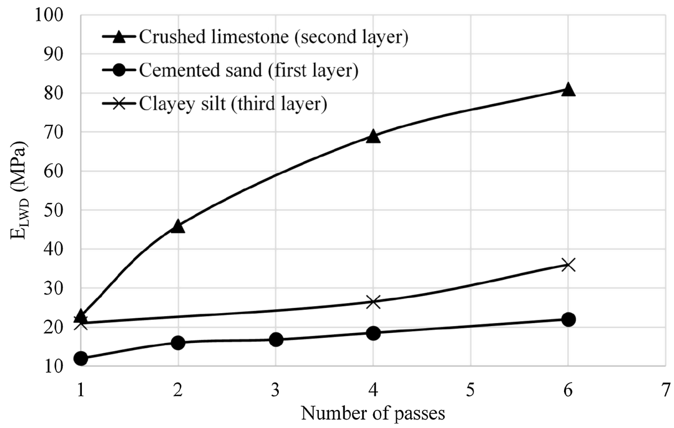

Fig. 7. E_{LWD} with the number of passes for several soils. (Reprinted from Abu-Farsakh et al. 2004.)

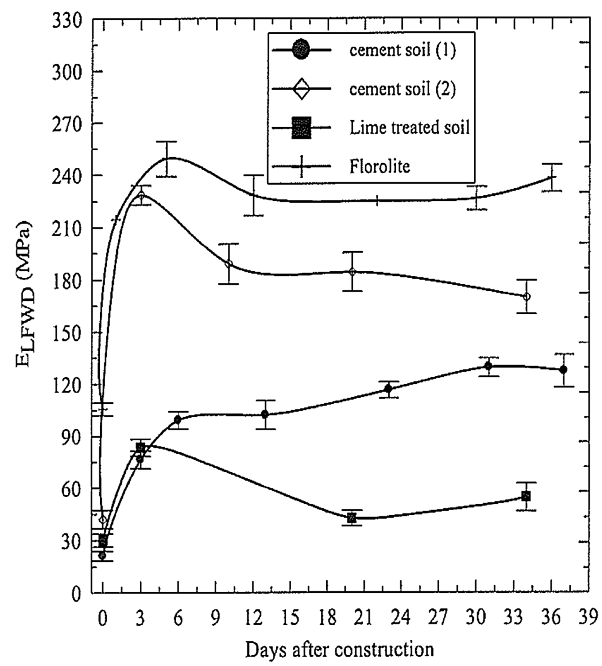

Fig. 8. Field E_{LWD} with the number of days after construction for different soils. (Reprinted from Abu-Farsakh et al. 2004.)

Hoffmann et al. (2003) indicated that inaccurate LWD moduli may be estimated when using the applied load and peak deflection technique. Therefore, they proposed a spectral-based procedure to better interpret the test results. Mooney and Miller (2009) used homogeneous, isotropic, linear elastic half-space assumption to estimate soil modulus (E_{LWD}). Results showed that the measured in situ stresses matched well with the stresses predicted. However, measured in situ strains did not match the strains predicted, which indicated that for the purpose of extracting constitutive soil properties Boussinesq's equations are inappropriate.

The approach provided by Marradi et al. (2011, 2014) focused on determining the energy loss parameter that can be calculated from the area under the load-deflection time history. A strong correlation between the energy loss and degree of compaction for various unbound materials was found, with R^2 higher than 80%.

Table 6. E_{LWD}—conventional laboratory measurements correlations

Test	Correlation	R^2	Data points	Soil type	Testing	Reference
M_R	$E_{LWD} = 0.989M_R - 44.1$	0.78	N/A	Granular	In situ	Van Gurp et al. (2000)
CBR	$\log(\text{CBR}) = 1.40 \log E_{LWD} - 1.6$	0.36	≈17	Granular and find-grained	Lab	Abu-Farsakh et al. (2004)
CBR	$\text{CBR} = 0.66 \log E_{LWD} - 14.0$	0.83	≈20		In situ	
M_R	$E_{LWD-Z} = 1.50M_{R_L} - 42.70$	0.76	≈7	Cohesive soils	In situ	White et al. (2007)
M_R	$E_{LWD-Z} = 0.67M_{R_H} - 35.1$	0.80	≈7			
M_R	$E_{LWD-K} = 2.6M_{R_L} - 80.50$	0.84	≈7			
M_R	$E_{LWD-K} = 1.1M_{R_H} - 58.0$	0.77	≈7			
TX	$E_{LWD-Z} = 2.15M_{s_L} + 13.2$	0.83	≈7			
TX	$E_{LWD-Z} = 0.90M_{s_H} + 5.20$	0.85	≈7			
TX	$E_{LWD-K} = 3.14M_s + 26.6$	0.63	≈7			
TX	$E_{LWD-K} = 1.36M_s + 13.1$	0.69	≈7			
CBR	$\text{CBR} = 0.2867E_{LWD} - 2.7543$	0.93	≈50	Lateritic soils	In situ	Rao et al. (2008)
DCP	$E_{LWD} = 155.52(\text{PR})^{-0.6193}$	0.81	≈50			
M_R	$M_R = 27.75E_{LWD}^{0.18}$	0.54	≈12	Cohesive	In situ	Mohammad et al. (2009)
M_R	$M_R = 11.23 + 12.64E_{LWD}^{0.20} + 242.32\left(\frac{1}{MC}\right)$	0.70	≈12			
M_R	$M_R = 22\delta_B^{-0.96}$	N/A	N/A	Granular	Lab	Ebrahimi and Edil (2013)

Note: E_{LWD-Z} is measured using standard (300 mm) Zorn LWD; M_{R_L} and M_{R_H} = low and high MR based on AASHTO T-307; E_{LWD-K} is measured using standard (300 mm) Keros LWD; M_{s_L} and M_{s_H} = secant moduli measured for low and high MR soils, respectively; w_c = moisture content; TX = triaxial test; and δ_B = deflection of base material from LWD (mm).

The results confirmed that energy loss could be used to enhance an LWD-based procedure for evaluating the compaction level achieved on site. However, to evaluate the energy loss for each drop, a load cell has to be mounted in the used LWD. In addition, the LWD software for on-site evaluation of the compaction level needs to be modified to include this method.

Correlations with Other Soil Parameters and Other In Situ Testing Devices

To evaluate the LWD effectiveness and efficiency as a compaction QA tool, the correlations between E_{LWD} and other soil properties from conventional soil tests (i.e., CBR, resilient modulus, proctor compaction) have been made, as summarized in Table 6. George (2006) developed a model to correlate between the FWD modulus and other properties of the tested soil, such as dry density, MC, and soil index properties based on measurements obtained from 13 as-built subgrade sections. It is concluded that if the soil is in the elastic range, the portable FWD (i.e., LWD) can be utilized to characterize the stiffness of subgrade soils. The COV decreased with the increase of the modulus, which was consistent with Abu-Farsakh et al. (2004).

The correlations between E_{LWD} and other in situ test moduli such as FWD, plate load test, DCP, and SSG have also been investigated, as summarized in Table 7. For example, a strong correlation was found between E_{LWD}, FWD back-calculated resilient moduli (M_{FWD}), and plate load test (PLT) stiffness moduli, E_{PLT} (Abu-Farsakh et al. 2004). In addition, Siekmeier et al. (2000) utilized several devices such as LWD, DCP, SSG, and traditional FWD to assess the stiffness of granular base materials at several construction sites. Results indicated that the E_{LWD} had a trend similar to that of the E_{FWD} but a different magnitude due to differences in the applied stress ranges.

Comparisons between Different LWD Types

Different LWD types may present differences in responses during measurements (Fleming 2000; White et al. 2007). Vennapusa and White (2009) performed a comprehensive comparison between three different LWD devices (Zorn, Keros, and Dynatest). The effect of device configurations on the test results, such as plate diameter, contact stress, buffer stiffness, and deflection measurement technique were investigated. Correlations were also developed between the LWD modulus and static plate load test measurements (E_{PLT}).

Table 7. Correlations between LWD moduli and stiffness parameters measured used other in situ devices

Other devices	Correlation	R^2	No. of points	Testing	Soil type	Reference
PLT	$E_{PLT(i)} = 0.71E_{LWD} + 18.63$	0.87	≈45	Laboratory and in situ	Granular and fine-grained soils	Abu-Farsakh et al. (2004)
PLT	$E_{PLT(R2)} = 0.65E_{LWD} + 13.8$	0.87	≈45	Laboratory and in situ	Granular and fine-grained soils	Abu-Farsakh et al. (2004)
FWD	$M_{FWD} = 0.97E_{LWD}$	0.94	≈30	In situ	Granular and fine-grained soils	Abu-Farsakh et al. (2004)
IC	$E_{LWD} = 55.6 - 19.92 \log(MDP)$	0.76	≈9	In situ	Granular	White et al. (2007)
IC	$E_{LWD} = 4.7 + 0.88(CMV)$	0.53	≈9			
DCP	$E_{LWD} = 110.4(\text{PR})^{-0.27}$	0.67	≈22			
DCP	$E_{LWD} = 3.93(CIV) + 2.53$	0.80	≈22			
MPMT[a]	$E_{LWD} = 4.22 + 3.36E_i + 0.040E_i^2$	0.84	≈48	In situ	Poorly graded sand	Shaban and Cosentino (2016)
MPMT[a]	$E_{LWD} = 7.07 + 0.66E_r + 0.001E_r^2$	0.79	≈48			

[a]All units are SI units otherwise indicated.

Note: MPMT = miniaturized pressure meter test where $E_{PLT(i)}$ and $E_{PLT(R2)}$ are the PLT initial and reloading moduli, respectively; M_{FWD} = resilient modulus back-calculated from FWD; MDP = machine drive power (kJ/s); CMV = compaction meter value (unitless); and E_i and E_r = initial and reloading moduli from MPMT stress-strain results.

The study indicated that most LWDs are similar in terms of operation and testing methodology. However, variabilities exist in the calculated E_{LWD} values due to the different stresses applied and deflection measurement techniques. The LWDs that use accelerometers to measure the deflection of the loading plate (e.g., Zorn LWD) reported larger deflections than the LWDs that measure deflections of the ground surface (e.g., Dynatest LWD). As a result, the moduli estimated by Dynatest and Keros LWD were on average 1.7 and 1.75 to 2.16 times greater than the moduli estimated from Zorn LWD, respectively, when 300 mm plate diameter used. White et al.'s (2007) study presented a similar statement. Despite the difference in buffer stiffness, both Keros and Dynatest LWDs showed very close results. The variability observed with Zorn E_{LWD} was generally lower than Keros and Dynatest E_{LWD} values (Vennapusa and White 2009).

Stamp and Mooney (2013) summarized the concerned characteristics when using different types of LWDs. They were (1) sensor type (accelerometer versus geophone); (2) sensor location (measuring plate deflection versus measuring ground deflection); (3) plate rigidity; and (4) applied load pulse. The results revealed that the sensor location was the predominant reason for different readings. The plate deflection was higher than the ground deflection by 65% to 310% on soils and 20% on asphalt. The other characteristics led to a relatively less significant difference (<10%). The authors concluded that these differences will always be there due to different LWD configurations.

Numerical Simulations

Few efforts were dedicated to using numerical simulations to study the performance of LWD and influencing factors, particularly those involve unsaturated soil mechanics. Tang and Yang (2013) and Tang et al. (2013a, b) applied dynamic finite-element modeling to predict the soil deflection using the recorded time histories of the LWD load as the input. Tirado et al. (2015) studied the soil response to different makes of the LWD through FEM. The dynamic nature of the LWD load was considered, and a nonlinear geomaterial model was incorporated. Two ASTM types of LWD [ASTM E2835 and E2583 (ASTM 2015c, 2020)] were considered in this study. Results showed that the LWDs were manufactured by different vendors and not interchangeable. Thus, any specification should be clear about the type and make of the LWD to be used and consider the target field values accordingly. This study showed that the depth of influence of the LWD was between 2 and 4 times the plate diameter, which is deeper than the values reported by previous studies (1 to 1.5 plate diameter).

To address the effect of contact stress and plate rigidity, Nazarian et al. (2015) developed a dynamic nonlinear FEM to simulate the LWD testing on top of a pavement system. According to their study, for the Zorn LWD, stress concentration was noticed at the edge of the plate, and maximum surface deflection occurs at the center of the plate. Moreover, the deflections under the Zorn LWD are 1.4 to 1.7 times greater than Dynatest LWD. Consequently, the resulting moduli were lower for Zorn LWD device ($E_{LWD\mathrm{Dynatest}} = 1.65\, E_{LWD\mathrm{Zorn}}$). These results were in agreement with the findings from Vennapusa and White (2009).

Umashankar et al. (2016) performed a comprehensive field and FEM analysis to evaluate the effect of the bottom pavement layer on E_{LWD}. Results indicated that the top layer absorbed the LWD load due to its higher stiffness. They recommended using the LWD for future QC/QA testing in India due to its accuracy, low coefficient of variation, ease of use, and fast and straightforward results.

Tirado et al. (2017) utilized dynamic and static FEM to evaluate the accuracy of using the elastic half-space theory (Boussinesq 1871) to obtain E_{LWD}. They noticed that the dynamic modulus was on average 0.58 times the static modulus. Moreover, the depth of influence under the dynamic loads was deeper than that for static loads. They proposed a correction factor for the stiffness modulus to account for the relative rigidity between the soil and loading plate. This relationship can help estimate a more representative target E_{LWD} considering nonlinear parameters obtained from laboratory resilient modulus. Similarly, nonlinear FEMs were used to develop transfer functions between the surface moduli from LWD and IC (Fathi et al. 2019).

Discrete element models (DEMs) have been used as well for analysis. Tan et al. (2013, 2014) investigated several discrete element models to simulate the physical behavior of the soil and concluded that a coupled model between standard liquid bridge model (simulates moisture content) and effective friction angle (considers the effect of fines content) gave the best match. The combined model was then used to simulate the standard LWD test, and the model accurately captured the systematic decrease in stiffness modulus with moisture and fines content increase (Fig. 9), which were also confirmed by Tamrakar and Nazarian's (2018) laboratory testing.

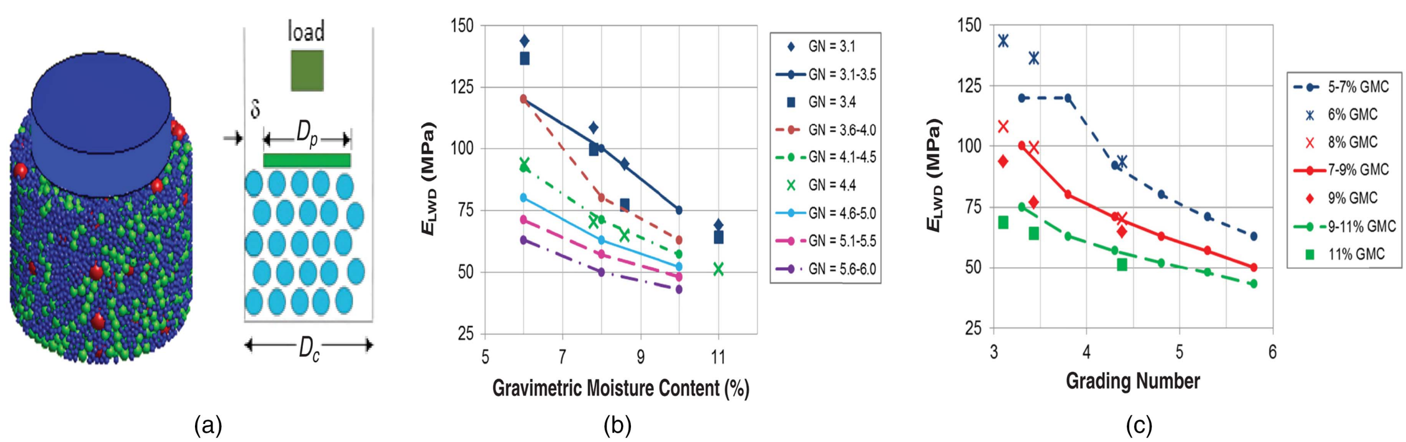

Fig. 9. Results of DEM for LWD test: (a) simulated LWD test on soil mixture (left) and schematic plot for LWD (right); (b) LWD modulus change with gravimetric MC for different grading numbers (note: GN higher for more granular soil); and (c) LWD modulus change with grading number for different MCs. (Reprinted from Tan et al. 2014.)

LWD for Compaction QA

The interest in implementing the LWD for compaction QC/QA of unbound materials dramatically increased worldwide in the last decade. In the Middle East, tremendous research has been conducted related to using the LWD for compaction QC/QA purposes (e.g., Ahmed and Khalid 2011; El-Badawy and Kamel 2011; Elhakim et al. 2014; Zabielska-Adamska and Sulewska 2015; Gonawala et al. 2019). Germany has over 3,000 LWDs that are used by road builders for QC/QA (Kessler 2009). The LWD has been specified as the portable plate test in the UK highway agency guidance since 2006, and the change in pavement foundation design to a performance-based approach has broadened the use of LWD for the field assessment of stiffness modulus (Fleming et al. 2009). In 2008, the European Union implemented the LWD into their standards for evaluating the compaction of unbound materials in CEN ICS 93.020 (Nazzal 2014). The CEN ICS 93.020 requires specific configurations for the used LWD that generated a load of 7 kN, including 163 mm plate diameters, 10 kg falling weight, and 720 mm drop height. It requires the LWD test be performed in six consecutive sequences of three drops (18 drops in total). The test should be performed on loose, noncompacted, materials at the site to obtain two main parameters: E_{LWD} and the dynamic compactness rate. Similar specifications were recommended by Aryal et al. (2005) in a study funded by the New England Transportation Consortium.

According to a recent survey in the US (Riad et al. 2021), among 46 participating state DOTs, 29 are unfamiliar with the LWD and never used it. A total of 24 DOTs have evaluated (in-house or through university/consultant) or demonstrated the use of LWD. Most of the DOTs found the LWD good to very good (on average 4.7 out of 5) in terms of testing time and ease of use. However, they evaluated the device repeatability and accuracy to be acceptable (on average 3.8 out of 5). Three DOTs (i.e., Nebraska, Indiana, and Minnesota) implemented and used the LWD for compaction QC/QA of unbound materials. The MnDOT has been one of the leading state DOTs in this field. According to MnDOT's specifications, the test shall be conducted as per ASTM E2583 (ASTM 2015c)and E2835, using deflection as the target field value by means of control strip or comparison test methods. For the control strip method, a control strip that satisfies all the compaction requirements should be constructed in the center of the roadway. Then, this strip shall be tested, and the average deflection is the target deflection value (LWD-TV). MnDOT mandates the LWD tests be conducted immediately after compaction to ensure minimal MC changes. MnDOT requires that the change of the deflection values for a test (i.e., the measurement drops) not to exceed 10% of the mean value. MnDOT has used the LWD to test all types of materials (Riad et al. 2021), including geogrid-reinforced aggregate base (Siekmeier 2018). Similarly, the Indiana DOT has developed specifications for utilizing the LWD for assessing the compaction control. They use the LWD to assess the stiffness of granular soils and structural backfill (Siddiki et al. 2014). The INDOT recommends performing LWD tests at the center of constructed lanes [at least 609.6 mm (2 ft) from each edge]. The maximum allowable deflection should be determined by means of a test section (i.e., control strip). One MC test of the compacted aggregates is required per day and must be within −6% of the optimum MC.

Nebraska DOT (NDOT) implemented the LWD as the only compaction QA tool for unbound materials per NDR T 2835 (Nebraska DOT 2014). The maximum allowable deflection should be determined by means of a control strip. Recently, NDOT developed group indices that can be used to identify the target deflection value based on the soil group. NDOT has been extremely satisfied with the LWD results and practicality (Riad et al. 2021). Table 8 summarizes the details of the testing requirements in DOTs' LWD specifications.

Current practices showed that two target field values are typically used for LWD projects (1) maximum allowable deflection (Indiana DOT; Minnesota DOT; European Union Standard CEN ICS 93.020; Aryal et al. 2005; Volovski et al. 2014; Zhao et al. 2018); and (2) target modulus for the unbound layer (Vennapusa 2008; Meehan et al. 2012; Nazarian et al. 2015; Schwartz et al. 2017). Commonly, there are three methods to determine the target field value (recommended in the implemented QC/QA specifications): (1) field control strip or test section; (2) comparison test method; and (3) LWD test on Proctor mold. For example, the three DOTs previously mentioned all determine the target field value (deflection) by means of control strip. In NCHRP Project 10-84, a numerical algorithm was employed to estimate the target field modulus based on inputs of parameters related to the structural design (Nazarian et al. 2015). These parameters include thickness, unit weight, nonlinear resilient modulus, and Poisson's ratio of each layer. Results indicated that Poisson's ratio significantly affected the target field modulus, and it should be standardized in the specification. Schwartz et al. (2017) developed two modulus-based QA

Table 8. Summary for the DOTs specified LWD testing requirement

	Subgrade material		Embankment material	
State DOT	Acceptance criteria	Number of test points	Acceptance criteria	Number of test points
Indiana	INDOT uses LWD for testing aggregates. Individual deflection value less than the target value. The target value is determined by means of field control strip.	One to four per 2,000 cubic yards.	INDOT uses LWD for testing aggregates. Individual deflection value less than the target value. The target value is determined by means of field control strip.	One to four per 2,000 cubic yards.
Minnesota	Individual field deflection less than the target value and Moisture content within limits of 65%–102% optimum moisture.	One test per 10,000 cubic yards.	Individual field deflection less than the target value (predetermined based on the type of material) and moisture content within limits of 65%–102% optimum moisture.	One test per 10,000 cubic yards.
Nebraska	Maximum average field deflection less than or equal to the target deflection value determined by means of control strip and the moisture content is required to be −3% to 2% of the optimum moisture from the proctor.	One test for every 1,500 cubic yards of material.	Maximum average field deflection less than or equal to the target deflection value determined by means of control strip and the moisture content is required to be −3% to 2% of the optimum moisture from the proctor.	One test for every 1,500 cubic yards of material.

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specifications that can be implemented by state DOTs. They recommended performing the LWD test on the Proctor compaction mold to determine the field target modulus. They also developed bilinear correlations to estimate the target field modulus as the corresponding field water content and plate pressure. The compaction quality can then be assessed by comparing the field modulus against the estimated target modulus (E_{field}/E_{target}). The LWD on mold moduli showed strong correlations with laboratory M_R, and field moduli. The target modulus criteria (E_{field}/E_{target}) showed a good match with the percent compaction from NDG measurements, which confirms the applicability of this LWD testing methodology for field QA evaluation.

Advantages and Limitations

Studies previously mentioned have commented that LWD is a viable device for characterizing unbound materials. In addition, it has been successfully used for broader applications related to the transportation industry compared to all other compaction QC devices, including testing the entire pavement sections (with asphalt or concrete layer, and with new materials such as RAP, geosynthetic products, and foamed materials) (Akbariyeh et al. 2016; Chou et al. 2017; AlShareedah and Nassiri 2018, 2019; AlShareedah et al. 2020). The advantages of using the LWD have been identified (Davich et al. 2006; Sebesta et al. 2006; Siekmeier et al. 2009; Riad et al. 2021), including (1) the setup and testing time for LWD is relatively short; (2) the measured modulus can be used to enhance the pavement design; (3) the LWD could accurately test more material types than the standard density-based approach; (4) it is nondestructive and requires minimal effort compared to the DCP; (5) it provides quick results with minimal calculations; (6) it has no regulatory burden; and (7) the test involves minimal human safety hazards. Previous studies reported some limitations for the LWD (Lin et al. 2006; Sebesta et al. 2006; Petersen et al. 2007; Hossain and Apeagyei 2010; Riad et al. 2021), such as (1) high variability in results from different LWD devices; (2) poor repeatability when used with soft cohesive materials or layered uneven surface; (3) it is relatively heavy which may add difficulty to use in large projects; (4) sometimes results are unrepresentative for the compaction quality, for instance a bone dry poorly compacted soil may give very high modulus values; and (5) education and explanations to contractors are critically required. In addition, the effect of many significant factors (i.e., underlying layers, MC, stress dependency, and soil suction) on the LWD results have not been adequately addressed. This may explain the LWD's low accuracy and repeatability indicated by most DOTs (Riad et al. 2021).

Conclusions and Recommendations

A comprehensive literature review of density-based and strength/stiffness-based QA techniques and specifications was conducted in this study. The review highlighted testing procedures, types of testing devices, correlations between testing parameters with other laboratory and in situ mechanical properties, factors influencing the results and analysis, status of practices and implementations, and advantages and limitations. Based on the literature review and analysis, the following conclusions have been made, from which recommendations for future study and implementation were provided.

The current density-based QC specifications are relatively simple and practical. The NDG is currently used by most of the state DOTs to assess the compaction quality for various types of unbound materials. Among all density-based devices, it is one of the few devices that can provide both MC and density measurements. However, NDG testing becomes less desirable because of safety, regulatory, and cost concerns. During the past decade, various nonradioactive devices for the field determination of density/MC have been proposed and evaluated. Their pros and cons were identified in past research studies, and more studies are needed for further evaluation. In addition, density-based QC specifications do not provide the engineering properties that can be used to ensure optimal performance of the tested material. Though efforts have been dedicated to correlating NDG measurements and the measured soil strength/stiffness, they were not successful. General research and engineering practices to develop stand-alone modulus-based specifications can be noticed and should be encouraged. This will result in better quality control, and therefore, a better-constructed product. Moreover, it will provide engineering properties that can be linked to pavement design and, in turn, long-term pavement performance.

With the growing interests in modulus-based compaction QA of unbound materials, several in situ test devices (e.g., BCD, CH, DCP, SCS, LWD, and PSPA) have been utilized to assess the mechanical properties of geomaterials. Among these devices, the DCP, SSG, and LWD are the most studied and used devices among state DOTs in the United States. Internationally, the most commonly adopted techniques for compaction QC of unbound aggregate layers are the LWD, SSG, and surface seismic. However, SSG has not been implemented into any of the QC/QA specifications, nationally and internationally, and DOTs that evaluated it reported poor to fair accuracy and repeatability (Nazzal 2014). Several state DOTs (e.g., Illinois, Indiana, Missouri, Minnesota, and Nebraska) have implemented modulus-based specifications using LWD and DCP. DCP had very good repeatability and accuracy. However, the device cannot be used for oversized aggregates (>51 mm) or soft clays. The LWD is a versatile and portable stiffness measuring tool. It has been increasingly used on a variety of unbound materials, including during construction and in-service around the world, and successfully used for broader applications related to the transportation industry compared to all other compaction QC devices. Studies recommended it for a modulus-based construction specification to accept compacted geomaterials but did indicate high degree of spatial variability and significant effect of MC on LWD modulus, which is one of the common challenges associated with modulus-based devices.

To facilitate further implementation of modulus-based QA specifications, it is strongly recommended to identify an accurate, rapid, and cost-effective field MC device included in the modulus-based QA specifications. MC is one of the main factors influencing soil modulus and should be performed concurrently with field modulus measurement for compaction QA.

In addition, with the growing interests and increasingly use of the LWD as replacement of the NDG, systematically investigation of important influencing factors on measured LWD modulus such as moisture, stress states, spatial variability, and development of reliable correlations with the NDG are needed to gain a better understanding of this relatively new technique. Other parameters such as types and makes of the LWD (e.g., target field values, the plate size and falling mass drop height) and pavement layer properties (e.g., Poisson's ratio, multilayer or singly-layer system) should be considered as well to obtain consistent and reliable data for the purpose of developing LWD specifications. More research is needed to improve the repeatability and reproducibility of LWDs, including optimization of data analysis and local calibration protocols. A comparative study would be helpful to identify the most robust, accurate, and practical data analysis procedure to be implemented into LWD software for in situ compaction evaluation. Proper training for field inspectors, contractors, and technicians is also needed for practical implementation by state DOTs and engineers.

Data Availability Statement

All data, models, and code that support the findings of this study are available from the corresponding author upon reasonable request.

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Notation

The following symbols are used in this paper:

a_0, a_1, a_2, a_3, b_0, b_1, b_2, and b_3 = model parameters;
E_i and E_r = initial and reloading moduli from miniaturized pressure meter test stress strain results;
E_{FWD} = falling weight deflectometer modulus;
E_{LWD} = lightweight deflectometer modulus;
E_{LWD-Z} = lightweight deflectometer modulus from ZORN device;
E_{LWD-K} = lightweight deflectometer modulus from Keros device;
$E_{PLT(i)}$ = initial modulus from plate load test;
$E_{PLT(R)}$ = reloading modulus from plate load test;
E_{PLT} = plate load test modulus;
K_{SSG} = soil stiffness estimated from the SSG;
MC_{OPT} = optimum moisture content;
M_{FWD} = resilient modulus back-calculated from FWD test;
M_R = resilient modulus;
M_{R_H} and M_{R_L} = low and high resilient modulus based on AASHTO T-307;
M_{S_H} and M_{S_L} = Secant moduli measured for low and high resilient modulus soils;
N_{DCP} = number of DCP blows per 10 cm;
γ_{dr} = dry density; and
δ_B = deflection measured of base material from LWD test (mm).

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