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Maximum Allowable Deflection by Light Weight Deflectometer and Its Calibration and Verification

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16. Abstract

Implementing a quality assurance (QA) process for compaction of aggregates using light weight deflectometer (LWD) requires construction of a 100 ft. long, 24 ft. wide test section prior to other uses by Indiana Department of Transportation (INDOT). However, there are numerous cases where aggregates are used in construction of subgrade, subbase course, and base course in small areas, such as bridge approach, lane widening, patching, and shoulders, and construction of a test section is not possible. In addition, there are over 70 LWDs currently used in construction projects by INDOT. Maintaining a quality control process requires timely and appropriate calibration and verification of the LWD devices.

This study performed intensive laboratory experiments on the aggregate materials commonly used by INDOT to determine the target deflection values for compaction QA and evaluate the effects of the moisture content and the interface condition of the inner wall of the Proctor mold on LWD test results. Intensive experiments were also performed in the test pits with LWD to determine the effect of the thickness of aggregate layer and close the gap between the LWD measurements made in-situ and in the Proctor mold. Extensive field LWD testing was conducted during construction of many projects in 2016 and 2017 to validate the compaction of various types of projects. Statistical tests were performed on historical compaction QA data from LWD testing and LWD verification data. It was concluded that moisture content has a significant effect on LWD deflection or modulus. Unbound aggregates compacted near the optimum moisture are capable of providing a stable modulus value when becoming wet and experiencing dramatic gains in modulus with decreasing the moisture content. Maximum allowable deflections were developed for QA of aggregate compaction, particularly in small areas. It was also found that annual verification is necessary to ensure repeatability of LWD deflection measurements.

Recommendations were made for immediate and future implementations, such as improvement of the current field practice of compaction QA with LWD in terms of test frequency, test position, and moisture content test. Finally, it was recognized that urgent effort is needed to determine the optimum interval for LWD calibration and necessary training is needed to further improve compaction quality and consistency of compaction QA with LWD.

17. Key Words	18. Distribution Statement
small area compaction, aggregate, quality assurance, light weight deflectometer (LWD), proctor test, test pit, moisture content, deflection, modulus, target deflection, experimental-mechanistic (EM) procedure, maximum allowable deflection	No restrictions. This document is available through the National Technical Information Service, Springfield, VA 22161.

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EXECUTIVE SUMMARY

MAXIMUM ALLOWABLE DEFLECTION BY LIGHT WEIGHT DEFLECTOMETER AND ITS CALIBRATION AND VERIFICATION

Introduction

There are three primary factors that may substantially affect the stability and strength of subgrade, subbase course, or base course: type of material, construction, and environment. Construction, particularly compaction, may be the dominant factor because subgrade or subbase course is commonly constructed using local materials such as soil or granular soil, either natural or stabilized. One of the common practices to ensure compaction quality is the in-place density test, which determines whether compacted soil density meets requirements.

Currently, sand cone and nuclear gauge tests are widely used to find the in-place density of compacted soil; however, these tests have drawbacks. The sand cone test requires digging a hole and using calibrated sand—a time-consuming test for granular soil. The nuclear gauge test uses a probe that contains radioactive source material. In light of these disadvantages, there is a tendency for state departments of transportation (DOTs) to find alternative tests for field soil compaction quality control. The light weight deflectometer (LWD) test is one of the most promising alternative in-place tests and is increasingly used for field soil compaction control. The LWD test overcomes the disadvantages associated with the sand cone and nuclear gauge tests and is capable of providing the in-situ modulus of geomaterials—one of the key parameters used to characterize the properties of pavement structural layers.

To date, the LWD test has already been used by the Indiana Department of Transportation (INDOT) for compaction quality assurance (QA) of lime and cement modified soils, subgrade treatments with aggregates, and aggregate subbase or base. However, the implementation of LWD for compaction QA requires construction of a 100 ft. long, 24 ft. wide test section prior to other uses. There are numerous cases where aggregate No. 53 is used in construction of subgrade, subbase course, and base course in small areas, such as bridge approaches, lane widening, patching, and shoulders, and construction of a test section is not possible. Additionally, there are over 70 LWDs currently used in construction projects by INDOT. Maintaining a quality control process requires timely and appropriate calibration and verification of the LWD devices. This research study was therefore performed to address these issues, particularly to develop maximum allowable deflections for compaction QA in small areas.

Findings

The Proctor test for aggregates is performed in accordance with the AASHTO Designation: T 99 by INDOT (2017). Corrections may be necessary if the oversize material is above a certain percentage. However, the laboratory test results indicate that the differences between the original and corrected maximum densities and between the original and corrected optimum moisture contents for both materials were not significant for practical applications.

When performing LWD testing on aggregates in a Proctor mold, the interface condition between the aggregate material and the inner wall of the mold will affect the deflection measurements, depending on the aggregate size and moisture content. The deflections increased by about 11.8% to 18.8% for No. 43 aggregates and by 1.9% to 6.7% for No. 53 aggregates when the inner wall of the mold was lubricated.

Different from the well-known bell shaped moisture-density relationship, the moisture-deflection relationships for aggregates did not show an optimum moisture content at which the deflection would be at a turning point. The results of the laboratory experiments imply that a minimum deflection may not exist in terms of different moisture contents. When compacted at the optimum moisture content, the modulus of aggregates increased considerably as the moisture content decreased. When compacted at a random moisture content, the modulus of No. 53 aggregates remained relatively unchanged, but the modulus of No. 43 aggregates increased noticeably as the moisture content decreased. Coarser aggregates are more sensitive to the moisture content than finer aggregates with respect to deflection or modulus.

The results of LWD tests in the test pits indicate that No. 53 aggregates can contribute to the structural capacity, but No. 43 aggregates can only contribute to the structural capacity when its thickness is 8 in. or more. The deflection decreased as the thickness of aggregate layer increased. As the layer thickness increased to a certain level, the deflection became stable. The modulus back-calculated from the stable deflection value may represent the modulus of elastic half space made up of the aggregates. It is interesting to note that the elastic half space moduli of the two materials (51.7 MPa for No. 43 and 38.1 MPa for No. 53) are rather close to the moduli by the Proctor test (46.7 MPa for No. 43 and 35.3 MPa for No. 53).

The differences between the specified target deflections and the measured deflections are statistically significant for both 2015 and 2016 historical datasets. The current target deflections may be too large for the purpose of compaction QA. The measured deflections in 2015 were not statistically significantly different from the measured deflections in 2016. The quality of compaction in roadway construction remained consistent for 2015 and 2016.

It is necessary to adjust the target deflection or modulus by taking into consideration the field and construction conditions, particularly compaction effort, subgrade condition, lift thickness, and use of geotextile. However, caution should be exercised when selecting either deflection or modulus as the target parameter for field compaction QA using LWD, due to the potential effects of many factors.

It may become very challenging to compact geomaterials in small and confined areas to the same degree as those in large areas. Therefore, the target deflection values should be adjusted according to the characteristics of compaction in small areas. Field LWD tests revealed that the deflections for lightweight compactor were greater than those for large roller. The overall ratios between the deflections in small and large areas are 1.192, 1.239, and 1.227 for 2017, 2016, and historical projects, respectively. No rigorous scientific methods are currently available to determine a factor for adjusting the target deflection. To avoid unnecessary complexity, 1.219 (the average of the above three deflection ratios) is used as the adjustment factor for considering the characteristics of small area compactions.

Placing an unbound aggregate layer on chemically modified subgrades may produce an inverted two-layer system; thus, the deflections may increase as the aggregate layer thickness increases. Nevertheless, the field LWD test results did not fully agree with the variation trend of deflection for the inverted layer system. Many factors, such as layer thickness, subgrade strength, and degree of compaction, may affect the lateral confining stress in the unbound aggregate layer under the impulse load generated in the in-situ LWD test. These factors may also interact with each other, which make it more difficult to accurately determine the effect of inverted layer structure. The potential effect of inverted layer system was not considered when determining the maximum allowable deflections.

The structural response of an elastic layer system to external loading may vary dramatically with the boundary condition. The deflection at the outside edge may be up to 40% and 35% greater than the deflections in the middle and inside edge, respectively. Therefore, caution should be exercised when determining the position from which to perform LWD testing for compaction QA, particularly in small areas.

Extensive in-situ LWD testing indicates that for small area compaction, a minimum of 5 LWD tests are required to provide reliable compaction QA. A minimum of 8 to 10 LWD tests are necessary for large area compaction. The minimum sample size should increase as the compaction area increases, taking into account the requirement of at least 10 LWD tests for a test section of 100 ft. by 20 ft. for compaction of aggregates.

The majority of the projects have a COV of 20% to 35%. For small area compaction, a COV of 20% or less may indicate "Low" variation, a COV of 20% to 35% may indicate "Normal" variation, and a COV greater than 35% may indicate "Poor" variation.

Annual verification is necessary to ensure repeatability of LWD deflection measurements.

Implementation

The following recommendations are made for future implementation:

- When performing the laboratory Proctor test to determine the target deflection (or modulus), the inner wall of the mold should be properly lubricated.
- The use of LWD test for compaction QA does not change the procedures of field compaction in roadway construction. It is important to compact aggregate materials near the optimum moisture content level.
- For aggregate compaction, the LWD deflection varies significantly with the moisture content. It is recommended that the LWD test for compaction QA should be conducted within two hours after compaction. The in-situ moisture content test is necessary to implement QA for compaction with LWD.
- The maximum allowable deflections recommended by this study should be further fine-tuned, taking into account statewide field practice and experience in roadway construction.
- Back calculation of the aggregate modulus from the mold or in-situ deflection is subject to the effects of many factors. Also, changing to a modulus-based quality control or assurance would produce data that could not be compared with historical data. It is advisable for INDOT to continue to use deflection as the target parameter for QA of compaction.
- Different LWD devices may have different features, leading to different deflection or modulus measurements. Further effort is needed for INDOT to support more than one type of LWD devices.
- The structural response of an elastic layer system to external loading varies dramatically with the boundary condition. Caution should be exercised when determining the position for performing LWD testing for QA of compaction. In small compaction areas, it is advisable to perform LWD testing three feet away from the outside edge or in the middle of the lane or shoulder under uniform compaction.
- Calibration of LWD devices is costly and time-consuming; however, the use of LWD devices out of calibration can be even more costly. Urgent effort is needed to assess the possible positive and negative effects of the calibration interval and determine the optimum calibration interval.
- Discrepancies observed in field compaction and LWD testing by different contractors and inspectors suggest that necessary training is needed to further improve construction quality and ensure QA consistency.

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1. INTRODUCTION

1.1 Problem Statement

1.1.1 Current Issues

Pavement is typically a layered structural system consisting of surface course and base course (or subbase course) placed on subgrade. Subgrade is the foundation of pavement structure that not only provides a support to overlying surface and base (or subbase course) course, but also undergoes repeated frost action, effect of water, and application of vehicle wheel load. Subbase course is primarily utilized to increase the overall structural capacity of pavement, distribute the load over the subgrade, reduce the shrink and swell of subgrade, improve drainage, mitigate frost action, and provide an even surface to the surface course. Therefore, subgrade and subbase course play an important role in not only enhancing pavement structural capacity and mitigating the potential effect of water, or temperature, or both, but also improving pavement performance. To achieve this, both the subgrade and subbase course should present sufficient stability and strength throughout the design life cycle.

There are three primary factors that may substantially affect the stability and strength of a subgrade, subbase course, or base course, including type of material, construction, and surround environment. In many cases, construction, particularly compaction, may become dominant factor simply because subgrade or subbase course is commonly constructed using local materials such as soil or granular soil, either natural or stabilized. In general, compaction uses mechanical energy to force soil particles to move closer and the voids between large particles filled with small particles. As a result, the air voids are reduced and the dry density is increased, which eliminates soil deformation (particularly uneven deformation), restricts water movement, prevents buildup of water, and increases stability and load bearing ability. One of the common practices to ensure compaction quality is the in-place density test that determines the density of compacted soil to see if it meets the requirements.

Currently, sand cone test and nuclear gauge tests have been widely used to the in-place density of a compacted soil. The sand cone test requires digging a hole and use of calibrated sand and is time consuming, in particular, for granular soil. The nuclear gauge test uses a probe inserted into a hole punched into the compacted soil to determine the rate radiation through the soil and moisture content. It is faster and easier to perform than the sand cone test. However, the possible disturbance of the side wall of the punched hole may therefore affect the accuracy of test. The probe contains radioactive source material that raises concerns regarding increased regulatory restrictions. Although the radioactivity of the source material is sufficiently low to present a negligible risk to the operator, the nuclear gauges may only be used by certified operators. In addition, disposal of the radioactive material at the end of the gauge's life is very difficult. Therefore, there is a trend for state departments of transportation (DOTs) to look for alternative tests for field soil compaction quality control.

Light weight deflectometer (LWD) test is one of the most promising alternative in-place tests that is increasingly used for field soil compaction control (Siddiki, 2012). The LWD test measures the peak deflection resulting from the force pulse generated by dropping a weight from an appropriate height. The measured deflection can be used to determine the layer modulus, i.e., the modulus of the compacted soil using an appropriate back-calculation procedure. Several advantages have been identified with the use of LWD test for compaction control. Compared to the sand cone and nuclear gauge tests, the LWD not only overcomes their disadvantages, but also produces the layer modulus that may be used to estimate the resilient modulus used in pavement design. Although the elastic modulus of a compacted soil may be estimated from degree of compaction, i.e., the dry density of compacted soil, the relationship between the elastic modulus and degree of compaction vary with soil type and moisture content. Compared to other modulus tests such as the falling weight deflectometer (FWD) test, the LWD test is much easier to operate and provides an estimate of the same stiffness parameter.

To date, the LWD test has already been used by the Indiana Department of Transportation (INDOT) to determine the maximum allowable deflection criteria for compaction control of lime and cement modified soils, aggregates over lime modified soils, and aggregates over cement modified soils. However, the application of these criteria requires construction of a 100 ft. long, 24 ft. wide test section prior to other uses in accordance with ITM 513 (INDOT, 2017a). There are numerous cases where aggregate No. 53 is used in construction of subgrade, subbase course, and base course in small areas, in particular bridge approach, lane widening, patching, and shoulders, and construction of a test section is not possible. In addition, there are over 70 LWDs currently used on INDOT construction projects. Maintaining a quality control process requires timely and appropriate calibration and verification of the LWDs. This research study was therefore performed to address these issues, particularly to develop maximum allowable deflection criteria for ensuring the quality of compaction for aggregate courses in small areas.

1.1.2 Research Objective

The objectives of this research studies are twofold. The first objective is to determine the maximum allowable deflection criteria for subgrade treatments including Type IC (12 in. of the subgrade excavated and replaced with coarse aggregate No. 53) and Type II (6 in. of the subgrade excavated and replaced with coarse aggregate No. 53), and subbase and base of aggregates No. 53 and other new materials placed in small areas without the test section, respectively. The second objective is to examine the current calibration and verification procedures, if any, for LWD devices, and propose a practical procedure to ensure the reliability of LWD test. In addition, this research study also examined the differences between LWDs by different manufacturers, including Zorn Instruments and Dynatest.

1.2. Main Research Tasks and Deliverables

1.2.1 Main Tasks

To fulfill the objectives of this research study, the main tasks that have been completed are summarized as follows:

1. **Synthesis of state DOTs' current practice.** This task completed a critical review of LWD models, back-calculation methodologies, LWD configuration and deflection criteria for compaction control, field operation procedures, and LWD calibration and verification procedures.
2. **Historical data and analysis.** This task covered collection and statistical analysis of the historical compaction data for Type IC and Type II subgrade treatments, and subbase and base of No. 53 aggregate. The results were utilized not only to upgrade the current criteria for compaction quality assurance, but also to identify the potential effects of compaction effort in small areas.
3. **Establishment of a baseline for LWD-based compaction quality assurance.** This task was one of the key tasks for this research study and consisted of two subtasks. The first was the Proctor compaction experiments conducted to construct baseline data on LWD deflections and moduli under various compaction conditions. The second was the test pit evaluation performed to assess the effects of boundary condition and layer thickness using both finite element analysis (FEA) and elastic layer theories, which plays an important role in narrowing the gap between laboratory baseline data and field implementation.
4. **Field testing and result analysis.** Field tests, such as LWD test, water content test, and dry density test were performed at the selected project sites in both 2016 and 2017 construction season. Deflection, water content, and dry density measurements were made on both subgrade and subbase or base course whenever possible. Other in-place tests such as dynamic cone penetration (DCP) were also made if deemed necessary and if test devices were available. The effects of compaction effort were examined, in particularly in small areas. The effect of subgrade moisture was also examined in terms of surface deflections before and after use of geotextiles.
5. **Development of deflection criteria for field implementation.** This task was conducted to develop the deflection criteria, i.e., the allowable maximum deflections based on the laboratory experiment results, test pit results, and proven track records for the subgrade treatments and aggregate base/subbase courses commonly used by INDOT. The basic factors considered in the criteria include moisture content, layer thickness, and compaction effort.
6. **Evaluation of current verification and calibration procedures.** This task was conducted to evaluate the current verification and calibration procedures by state highway agencies (SHAs), if any, and proposed a harmonized procedure for LWD sensor verification and calibration to ensure the reliability of LWD test. An analytical guidance was proposed for establishing reliable LWD verification and calibration requirements and procedures. This task also examined the verification and calibration procedures currently utilized by INDOT for both falling-weight deflectometer and LWD.
7. **Final report.** This report will document the research procedures, data, results and findings, and recommendations for implementations.

1.2.2 Main Deliverables

To assist in the immediate and future implementations, this research study provide deliverables as follows:

1. The maximum allowable deflections for compaction of subgrade treatment of Type IC and Type II, and subbase and base of aggregates No. 53 and No. 43 in small construction areas including new construction, bridge approaches, lane widening, pavement patching, and shoulders.
2. Upgraded maximum allowable deflections for compaction quality assurance of large areas accessible to large compactors.
3. Harmonized procedures for LWD verification and calibration.
4. Technical report documenting research procedures, results, and findings.

2. EXPERIMENTAL ANALYSIS OF MATERIAL PROPERTIES

2.1 Laboratory Experiments of Material Modulus

LWD is designed to measure the compaction quality of a structural layer. However, the measured deflection and modulus are affected by the underlying structural materials. The measured modulus is actually the modulus of the entire structural system, rather than the modulus of the compacted top layer. Therefore, the effects of the materials below the compacted top layer should first be analyzed in order to accurately measure the modulus of the compacted material in the top layer with LWD. In a study published elsewhere (Schwartz, Afsharikia, & Khosravifar, 2017), the authors conducted LWD tests on both the compacted soil samples in Proctor molds and the compacted soils in test pits. The moduli measured with LWDs by different manufacturers were compared with the resilient moduli measured using the repeated triaxial compression test.

In the current study, LWD tests were also performed on the compacted materials in a Proctor mold. However, the scope of this study didn't include the laboratory triaxial compression tests. In Indiana, the No. 53 aggregate has been the major type of granular materials with specified gradations for subgrade treatment, granular sub-base for concrete pavement, and granular base for hot mix asphalt (HMA) pavement. Therefore, the main effort of the laboratory experiments was focused on the properties of the No. 53 aggregates related to the construction quality, including gradation, optimum moisture content, maximum dry density, deflection,

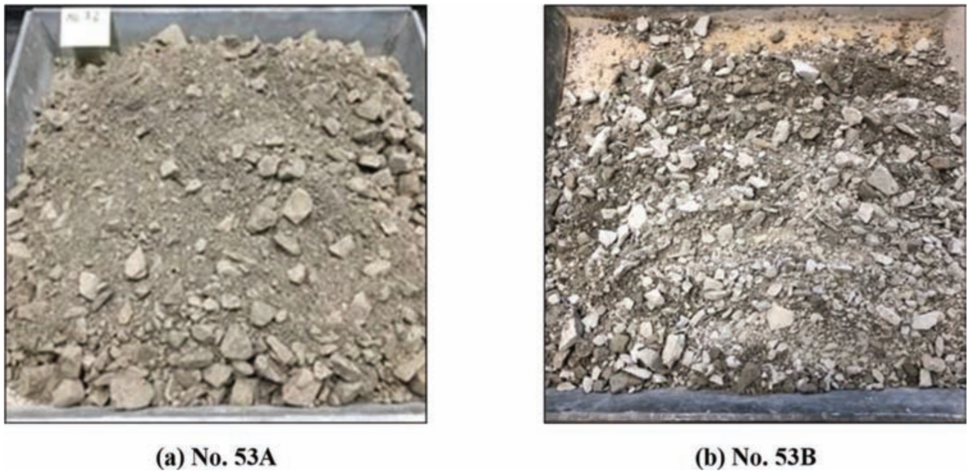


Figure 2.1 Two samples of No. 53 material.

TABLE 2.1
Gradations and Specifications for No. 53 Aggregates

	% Passing Sieve Size				
	No. 53A		No. 53B		
Sieve Size	Supplier	Test	Supplier	Test	INDOT Specification
1½″ (37.5mm)	100.0	100.0	100.0	100.0	100
1″ (25 mm)	90.9	90.7	91.3	94.0	80–100
¾″ (19 mm)	79.2	79.7	80.7	84.3	70–90
½″ (12.5 mm)	66.0	66.0	64.0	66.5	55–80
⅜″ (9.5 mm)	58.7	60.6		59.4	
#4 (4.75 mm)	47.1	51.6	39.6	39.4	35–60
#8 (2.36 mm)	33.1	35.4	29.5	29.4	25–50
#30 (0.6 mm)	14.5	14.0	16.0	17.7	12–30
#200 (0.075 mm)	8.9	8.1	10.0	10.0	5–10

and modulus. In addition, some other materials recently used for road construction by INDOT, such as No. 43 aggregate, steel slag, and recycled asphalt pavement (RAP), were also tested to provide first-hand data and baseline information.

2.1.1 Gradations of the Sample Materials

No. 53 aggregates samples, denoted as No. 53A and No. 53B, were obtained from two different suppliers for this study as shown in Figure 2.1. The information on the materials from the suppliers includes gradations, optimum moisture contents, and maximum dry densities. Table 2.1 presents the gradations provided by the suppliers of the two samples along with the INDOT gradation specifications (INDOT, 2017b). The given gradations of the material samples were within the INDOT specified ranges and, therefore, satisfied the requirements of the gradation specifications. Sieve analyses were conducted with the two material samples. The results of the sieve analyses as well as the gradations given by the suppliers are also presented in Table 2.1. It is shown that the actual gradations were very close to those provided by the suppliers. Therefore, both No. 53A and No. 53B aggregates meet the standard specifications.

2.1.2 Maximum Dry Density and Optimum Moisture Content

The optimum moisture content and the corresponding maximum dry density of a soil mixture are the most important values for achieving the desired compaction. These values provided by the suppliers of No. 53A and No. 53B are shown in Table 2.2. The given optimum moisture content and maximum dry density for No. 53A are 8.9% and 134.2 pcf, and those for No. 53B are 10.9% and 127.8 pcf. In order to analyze the relationships between the degree of compaction and LWD measurements, Proctor tests were performed to establish the moisture-density relationships for No. 53A and No. 53B materials. Notice that the AASHTO Designation T 99 Method D (2017) was chosen for the Proctor tests. This method is applicable to the materials with a maximum of 30% of the particles retained on the 19.0 mm (3/4 in.) sieve. As illustrated in Table 2.1, the No. 53A aggregate sample contained 20.3% of the particles greater than the sieve size of 19.0 mm (3/4 in.)

according to the lab sieve analysis. The No. 53B aggregate sample had 15.7% of the particles greater than the sieve size of 19.0 mm (3/4 in.).

With Method D of the AASHTO Designation T 99, all aggregate particles larger than the sieve size of 19.0 mm (3/4 in.) is defined as oversized material. Therefore, a correction may be necessary if the oversize material is above a certain percentage specified by the agency. If the agency does not specify such a percentage, it is recommend that a correction be made when more than 5 percent by weight of oversize particles is present (NDDOT, 2015). Since INDOT did not have the specified the percentage of oversize material, according to the recommended 5 percent criteria, corrections were necessary for both No. 53A and No. 53B materials. The correction method for Method D of the AASHTO Designation T 99 is specified as the AASHTO Designation T 224 (2010). Correction tests were conducted for the two No. 53 aggregate samples to adjust the densities to compensate for oversize coarse particles that were greater than the sieve size of 19.0 mm (3/4 in.). Presented in Figures 2.2 and 2.3 are the original and corrected Proctor curves for both the No. 53A and No. 53B aggregate samples, respectively.

It is shown that for No. 53A aggregates, the original maximum dry density and optimum moisture content were 125.8 pcf and 11.2%, while the corrected maximum dry density was 126.3 pcf and the corrected optimum moisture content was 11.4%. The corrected values were both slightly higher than their corresponding original values. For No. 53B aggregates, the original maximum dry density and optimum moisture content were 127.0 pcf and 11.1%, while the corrected maximum dry density was 127.9 pcf and the corrected optimum moisture content was 11.0%. The corrected values were very close to the original values. Therefore, the differences between the original and corrected maximum densities and between the original and corrected optimum moisture contents for both materials were not significant for practical applications. For comparison, the moisture contents and dry densities provided by the two suppliers are also presented in Figures 2.2 and 2.3, respectively. It is apparent that the laboratory results and the supplier provided values are quite different for No. 53A aggregates. For No. 53B aggregates, however, the optimum moisture content and the maximum dry density from the supplier are very close to those from the laboratory tests.

Table 2.3 summarizes all of the moisture and density values from the suppliers and from the laboratory tests for both No. 53A and No. 53B aggregates. Comparing No. 53A and No. 53B aggregates, the laboratory tests, original or corrected, yielded very similar values of optimum moisture contents and maximum dry densities of the two material samples. Also as presented in the sieve analysis results, the gradations of No. 53A and No. 53B aggregates were also similar. It was therefore justified to use either No. 53A or No. 53B to represent No. 53 material in the experiments and analysis. Therefore, only No. 53A was utilized to perform other experiments and analysis henceforth. The material would then be denoted as No. 53, rather than No. 53A, as presented in the remaining sections of this chapter.

TABLE 2.2
Proctor Test Results for No. 53 Aggregates by Suppliers

	Aggregate Sample	
Proctor Test Value	**No. 53A**	**No. 53B**
Optimum Moisture Content	8.9%	10.9%
Maximal Dry Density	134.2 pcf	127.8 pcf

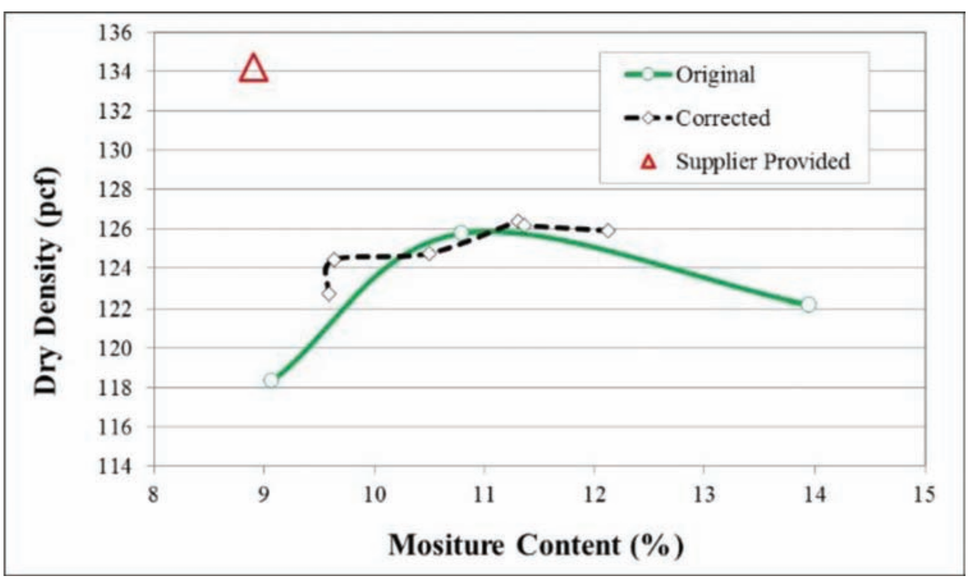


Figure 2.2 Original and corrected moisture-density curves for No. 53A.

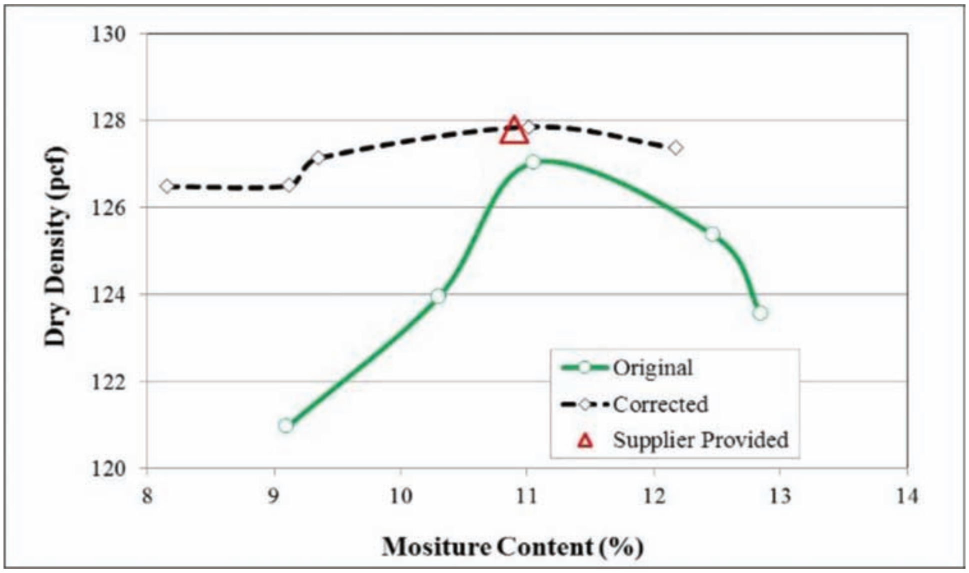


Figure 2.3 Original and corrected moisture-density curves for No. 53B.

TABLE 2.3
Supplier Provided and Laboratory Moisture-Density Values

	Optimum Moisture Content (%)			Maximum Dry Density		
Sample	**Supplier Provided**	**Original (T 99-D)**	**Corrected (T 224)**	**Supplier Provided**	**Original (T 99-D)**	**Corrected (T 224)**
No. 53A	8.9	11.2	11.4	134.2 pcf	125.8 pcf	126.3 pcf
No. 53B	10.9	11.1	11.0	127.8 pcf	127.0 pcf	127.9 pcf

2.1.3 Laboratory Testing of Material Densities and Deflections

In order to establish the relationship between optimum moisture content and LWD measurement, a series of laboratory tests were conducted. In addition to No. 53 aggregates, some other types of materials, including No. 43 material, steel slag aggregate, and reclaimed asphalt pavement (RAP), have also been utilized in pavement bases in Indiana. The samples of these materials were obtained and used in the laboratory experiments for modulus analysis. As it is well known, for a given compaction effort, a soil's dry density will increase to a peak point as the moisture content of the soil increases and then the dry density will decrease if the moisture content further increases beyond the peak point of the dry density. A moisture-density curve of a soil from the Proctor test is typically a bell shaped curve. The bell shape of the moisture-density curve is usually more apparent for clayey soils. Therefore, a clay sample was also included in the laboratory tests because of its typical plastic moisture-density relationship and its widespread existence in pavement subgrade in Indiana. Presented in Figure 2.4 are the photos of the material samples: No. 43, steel slag, RAP, and clay samples.

The samples of No. 43 aggregates, slag aggregate mixture, and the RAP material had similar gradations that meet the INDOT specified gradations. A sieve analysis was conducted with the No. 43 aggregate sample. The results of the sieve analysis along with the gradation given by the supplier are shown in Table 2.4. The sieve analysis proved that the actual gradation values were very close to those provided by the supplier and were also within the INDOT specified ranges. Similar to the study by Schwartz, et al. (2017), an aggregate sample was first compacted confirming to the AASHTO standard Proctor method, and then, the LWD measurements were made directly on the compacted sample in the Proctor mold.

In this study, it was desired to compact a sample to a denser state in order to effectively evaluate the relationships between the degree of compaction and the LWD deflection and modulus. Thus, the AASHTO Designation T 99 Method (2017) was modified so that the Proctor compactions were conducted with a 10-pound rammer rather than the 5.5-pound rammer. Figures 2.5 demonstrates the Proctor compaction and LWD measurement during the laboratory experiment. The deflections were measured six times on the material sample in the mold with the LWD. The first three deflection values were discarded and the average of the last three of the six deflection values were calculated as the measured deflection. The device was a Zorn LWD with a 5 kg drop weight and a 150 mm diameter base plate.

The main purpose of the LWD measurements over Proctor compacted materials was to reveal the change patterns of deflections and moisture contents in

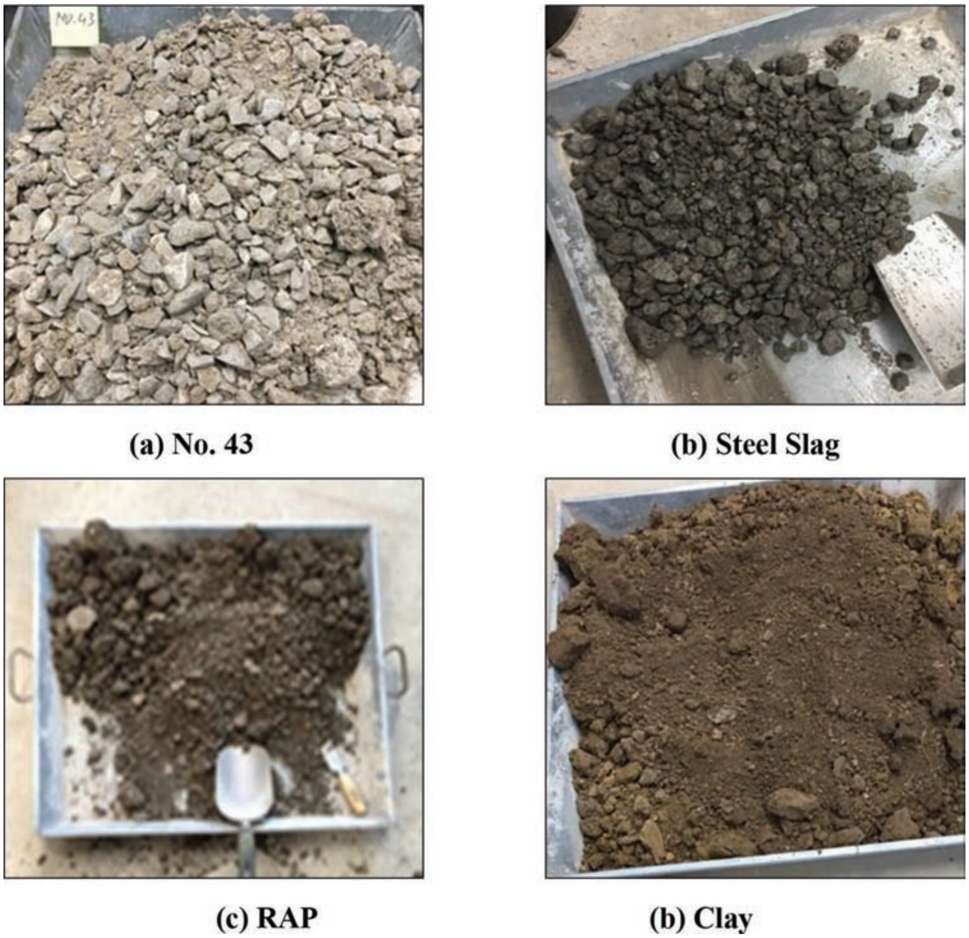


Figure 2.4 Photos of additional materials for laboratory tests.

TABLE 2.4
Sieve Analysis Results of No. 43 Aggregates

	% Passing Sieve Size		
Sieve Size	**Supplier Provided**	**Lab Test**	**INDOT Specification**
1½″ (37.5mm)	100	100	100
1″ (25 mm)	81.1	82.7	70–90
¾″ (19 mm)	64.5	65.1	50–70
½″ (12.5 mm)	45.4	44.5	35–50
⅜″ (9.5 mm)		35.9	
#4 (4.75 mm)	27.6	25.4	20–40
#8 (2.36 mm)	21.3	18.7	15–35
#30 (0.6 mm)	12.5	10.2	5–20
#200 (0.075 mm)	3.3	3.5	0–6

comparison with the moisture and density relationships. Plotted in Figure 2.6 are the moisture-density curve and the moisture-deflection curve for No. 53, No. 43, steel slag, RAP, and clay samples. It was expected that the changes of deflections and densities would have an inverse relationship so that as density increases the deflection decreases and vice versa.

As shown in Figure 2.6, however, the materials did not demonstrate the inverse correlations between density and deflection. Only RAP and clay materials showed slight deflection declines and reached a minimum deflection value as density increases within a limited range. In general, the materials exhibited a common pattern that as moisture content increases the deflection increases. It is indicated that the moisture content plays important but different roles in densities and deflections. Different from the well-known bell shaped moisture-density relationship, the moisture-deflection relationships did not commonly show an optimum moisture content at which the deflection

(a) Proctor Compaction

(b) LWD Test in Mold

Figure 2.5 Photos of laboratory LWD testing.

would be at a turning point. Therefore, the results of the laboratory experiments imply that a minimum deflection might not exist in terms of different moisture contents. This is because aggregate modulus increases as density increases, moisture content decreases, and aggregate interlocking increases. Compaction increases soil density and interlocking by reducing the voids in a soil with permanent deformation, while deflection is induced by an instant LWD impact with recoverable deformation.

An important implication of the moisture-density and moisture-deflection relationships is that a range of moisture contents must be specified when establishing the maximum allowed LWD deflection value in order to effectively control compaction quality. Therefore, during construction, compaction should be performed when the moisture content is at or close to the optimum moisture content and the LWD deflections should be made as soon as the compaction is completed or before the moisture content decreases beyond the specified range. That is, it should not be allowed to measure LWD deflections on compacted layer after the moisture content has dropped below the specified range of moisture contents.

2.1.4 Effect of Friction between Material and Proctor Mold

When a material in the mold is under the impact from LWD, the friction between the material and the inner wall of the mold could affect the deflections and material modulus considerably. The bonding condition is an important factor in computing modulus of the material. To analyze the effect of friction between the inner wall of the mold and the compacted materials in the mold, the laboratory testing was conducted by brushing a layer of lubrication oil on the inner wall of the mold. It was assumed that the friction between the inner wall and the material was reduced to a minimal level when lubrication oil was applied on the wall surface. Therefore, with lubrication, a zero-vertical friction between the material and the inner wall of the mold was assumed to calculate the material modulus. Two material samples, No. 43 and No, 53, were compacted in the Proctor molds with and without lubrication oil under different moisture contents, including the optimum moisture contents of the materials (5.5% for No. 43 and 11.2% for No. 53). The LWD measurements were made on the compacted materials in the molds.

The test results for the two materials are shown in Table 2.5. The deflections in the molds with and without lubrication indicate that the effect of lubrication on No. 43 was much greater than on No. 53. Based on the gradations of the two aggregate materials, No. 43 contained more large particles than No. 53. Thus, the laboratory tests suggest that lubrication affects more significantly on coarse materials than on fine materials. It was also observed that as moisture content decreased from the optimum moisture content, the lubrication effect on deflections increased. The data in Table 2.5 is also plotted in Figure 2.7 to further demonstrate the deflections of the two aggregate materials under different moisture contents with and without lubrication in the molds. The figure demonstrates the effects of friction and moisture content on LWD measured deflections.

With lubrication oil on the mold wall, the friction at the interface between the surface of mold inside wall and aggregates is assumed to be zero. The equation of modulus of the material can be derived based on the theory of elasticity (Timoshenko & Gere, 1961).

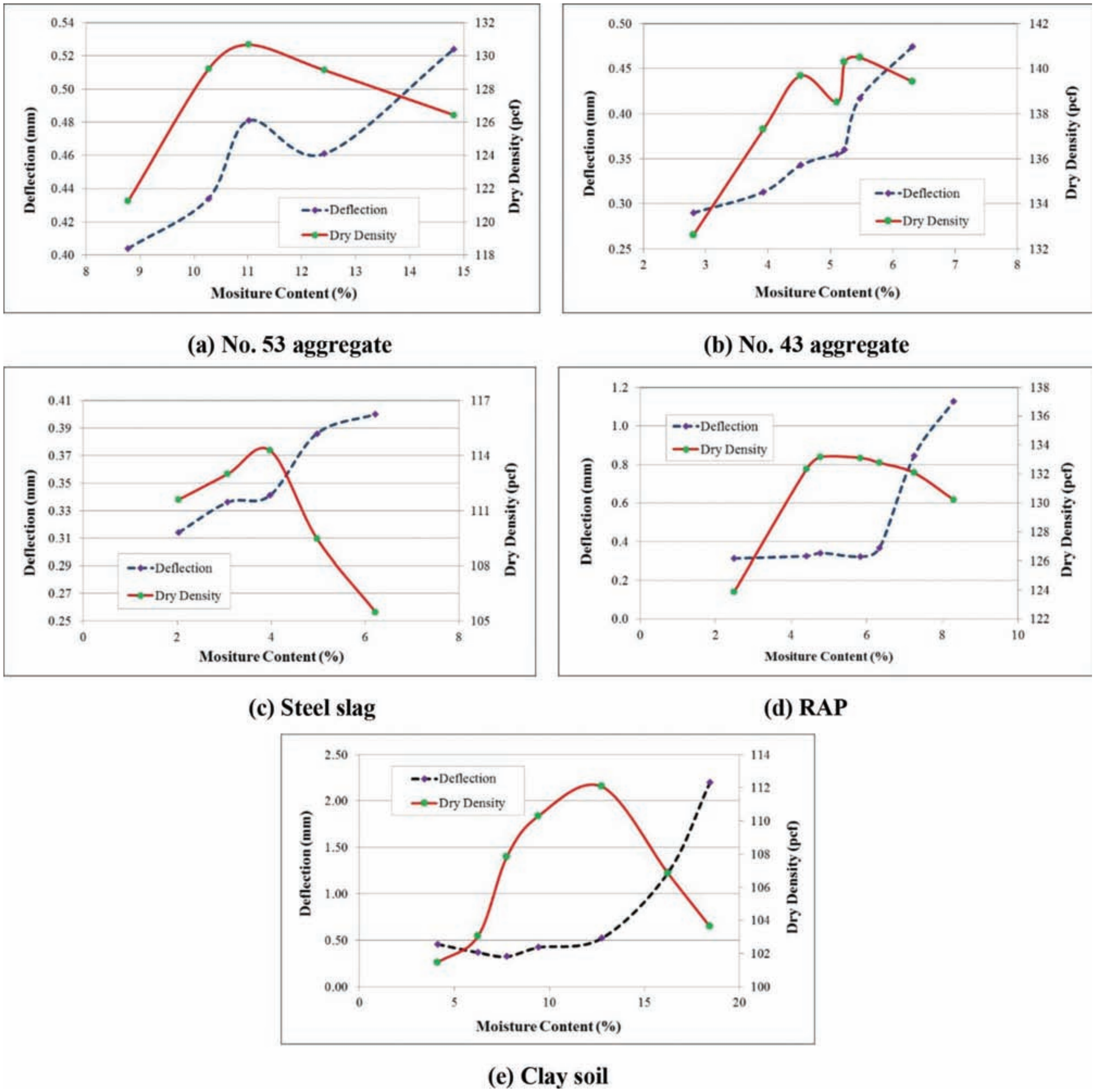


Figure 2.6 Moisture, dry density, and deflection relationships for different materials.

The relationship of the stresses on the specimen in the mold can be expressed as:

$$\sigma_r = \sigma_\theta = \frac{\mu}{1-\mu}\sigma_z \tag{2.1}$$

Where, σ_r = lateral stress; σ_θ = sheer stress; σ_z = vertical stress; and μ = Poisson's ratio.

Based on Hooke's law, the vertical strain (ε_z) is:

$$\varepsilon_z = \frac{\sigma_z - \mu(\sigma_r + \sigma_\theta)}{E} \tag{2.2}$$

where, ε_z = vertical strain; and E = material modulus.

For a specimen with a height (H), the total vertical deflection (d) of the specimen is:

$$d = \int_0^H \varepsilon_z dh = \frac{H}{E}[\sigma_z - \mu(\sigma_r + \sigma_\theta)] \tag{2.3}$$

Substituting σ_r and σ_θ in Eq. (2.3) with Eq. (2.1) yields the equation below:

$$d = \frac{H\sigma_z}{E}\left(1 - \frac{2\mu^2}{1-\mu}\right) \tag{2.4}$$

Since σ_z is the LWD measured pressure, q, Eq. (2.4) can be rewritten as:

TABLE 2.5
Deflections with and without Lubrication

Contact Condition	No.43 Deflection (mm)	No.43 Moisture Content (%)	No.43 Difference (%)	No.53 Deflection (mm)	No.53 Moisture Content (%)	No.53 Difference (%)
Non-lubrication	0.330	5.56	11.82	0.479	10.96	1.88
Lubrication	0.369	5.58		0.488	10.94	
Non-lubrication	0.290	2.80	14.83	0.419	9.13	4.43
Lubrication	0.333	2.92		0.438	9.04	
Non-lubrication	0.245	2.00	18.78	0.389	7.21	6.68
Lubrication	0.291	1.99		0.415	7.28	

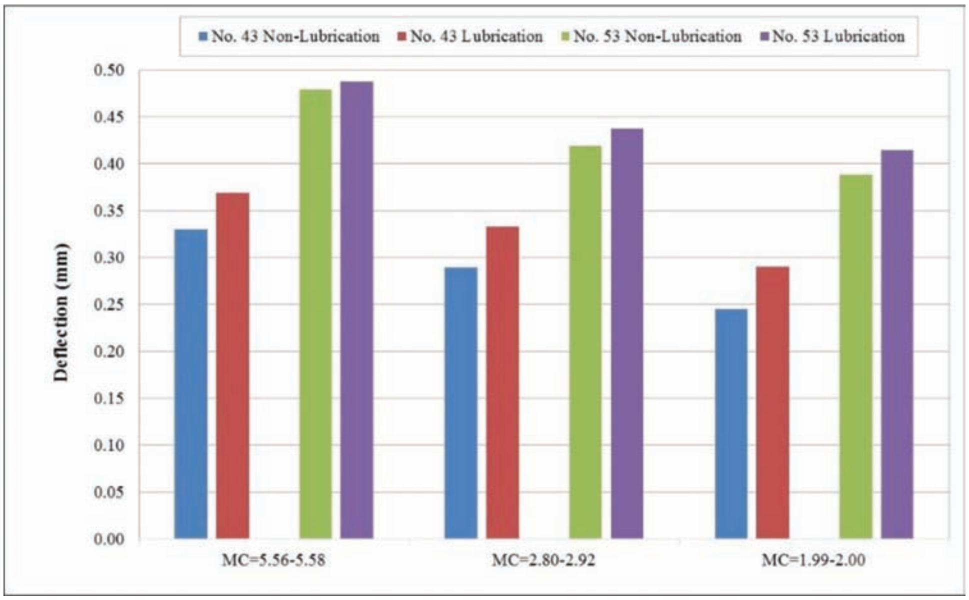


Figure 2.7 Deflections at different moisture contents.

$$d=\frac{Hq}{E}\left(1-\frac{2\mu^2}{1-\mu}\right) \quad (2.5)$$

Thus, the material modulus is compuedt as follows:

$$E=\frac{Hq}{d}\left(1-\frac{2\mu^2}{1-\mu}\right) \quad (2.6)$$

Because a range of Poisson's ratio $\mu = 0.1$ to 0.4 is recommended in the Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2015), $\mu = 0.3$ was selected to calcuate the modului from the LWD measured deflections. The modulus of a material can be computed with the deflection of the material that was compacted at the optimum moisture content using Eq. (2.6). The following moduli were obtained for the two material samples:

- The modulus for No. 53 at the optimum moisture content (11.2%): E = 35.32 MPa.
- The modulus for No. 43 at the optimum moisture content (5.5%): E = 46.66 MPa.

2.1.5 Effect of Moisture Content on Modulus

Moisture content is one of the important factors affecting the degree of compaction of geomaterials. In the traditional moisture-density controlled compaction process, a layer of pavement material is compacted at the optimum moisture content until the dry density of the material has reached the specified value. In order to examine the effect of moisture content on the modulus of geomaterial, laboratory tests were conducted to measure LWD deflections on the compacted materials at different moisture contents.

Two sets of the laboratory experiments were performed to reveal the change patterns of modulus at different moisture contents under compaction. The first set of the laboratory experiments was to compact the material specimen in Proctor mold at the optimum moisture content and to measure the deflection of the compacted specimen immediately after the compaction. After the first measurement with LWD, the specimen was placed in an oven at 230°F for 30 minutes and then

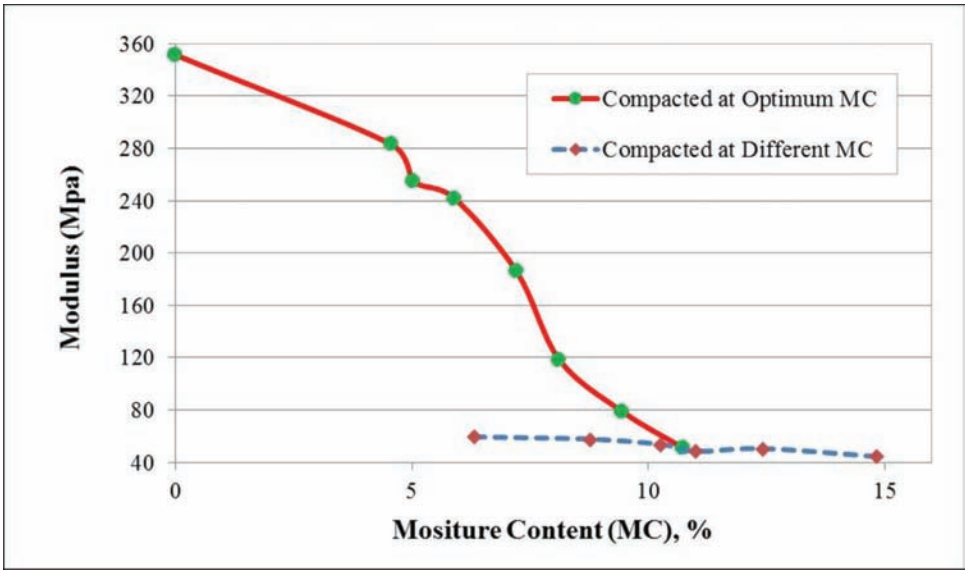


Figure 2.8 Variation of modulus with moisture content for No. 53 aggregates.

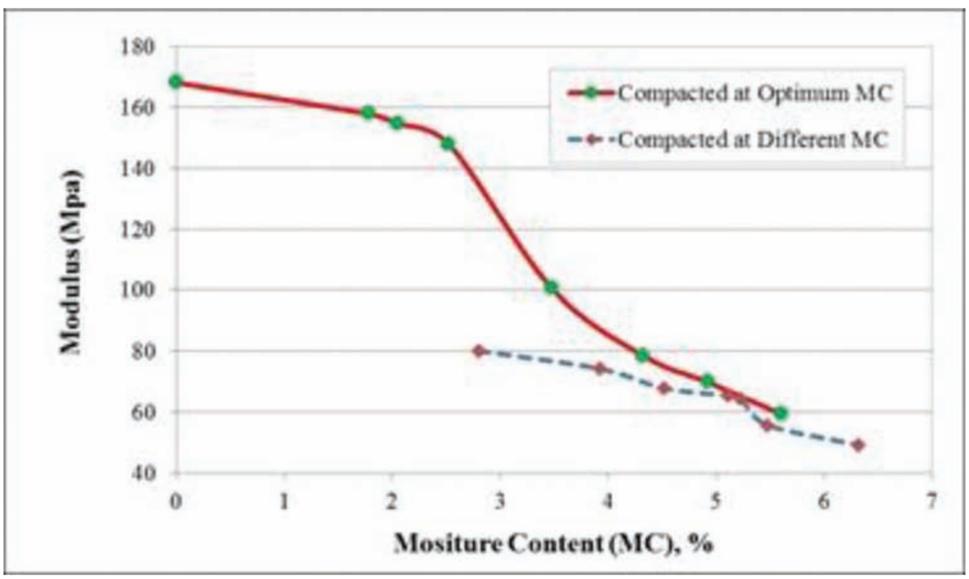


Figure 2.9 Variation of modulus with moisture content for No. 43 aggregates.

deflection was measured and the moisture content was determined. This process was repeated until the specimen was completely dried. The second set of the laboratory was to compact material at a moisture content in Proctor mold and to measure the deflection on the compacted specimen. This laboratory was conducted on six material specimens of different moisture contents to obtain the corresponding deflections. The measured deflections were all converted to moduli with Eq. (2.4).

Plotted in Figures 2.8 and 2.9 are the results of the laboratory experiments for both the No. 53 and No. 43 aggregates, respectively. The first observation from the two charts is that, for both No. 53 and No. 43 materials, after the material was compacted at the optimum moisture content the modulus increased considerably as the moisture content was reduced each time. This phenomenon may have some significant practical implications in compaction quality control with LWD devices. The practical meanings of compaction quality control with LWD would include: 1) Compaction must be performed at the optimum moisture content to achieve sufficient dry density; and 2) LWD deflection or modulus must be measured within a limited time window after compaction to obtain meaningful deflection values pertinent to the degree of compaction.

The second observation from the laboratory results is that No. 53 and No. 43 reflected differently to the changes of moisture contents in terms of moduli when the materials were compacted at different moisture contents. The modulus of No. 53 remained relatively

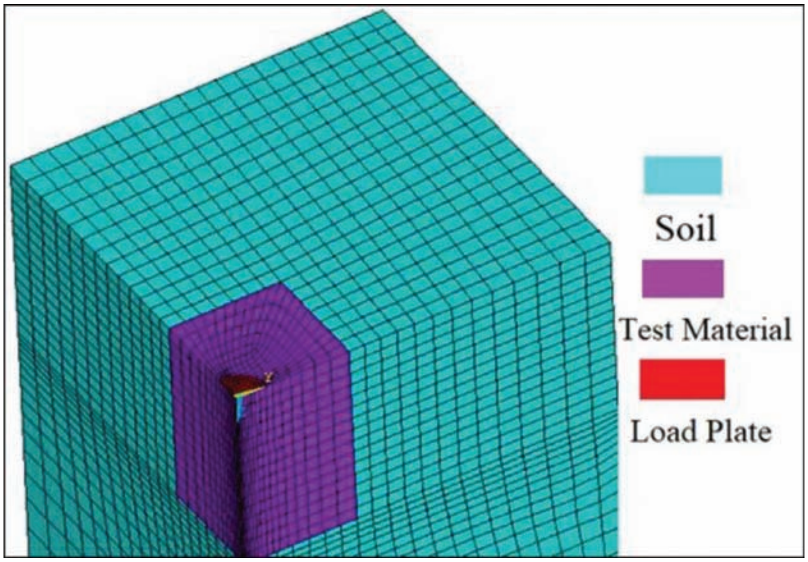


Figure 2.10 Illustration of FEA element mesh.

stable at different moisture contents. On the other hand, the modulus of No. 43 material increased noticeably as the moisture content decreased. The different patterns of the two materials indicate that the coarser material (No. 43) was more sensitive to the moisture content than the finer material (No. 53) with respect to LWD measured deflections or moduli.

2.2 Test Pits Experiments of Material Modulus

2.2.1 Determination of Test Pit Size

To calculate the material modulus from LWD deflection with Eq. (2.6), it is assumed that the material is homogeneous, linear elastic, and isotropic based on the theory of elasticity (Timoshenko & Gere, 1961). However, the pavement materials with aggregates, such as the No. 43 and No. 53 materials, are not actually uniform materials with elastic properties. The indoor Proctor-LWD experiments may not accurately reflect the actual pavement condition. Therefore, outdoor test pits were designed and constructed to simulate the real pavement structures. Notice that a heavier version of the Zorn LWD, i.e., ZFG 2000, was selected for the test pit experiments than the Zorn LWD used in the laboratory experiments. The ZFG 2000 LWD has a 10 kg drop weight and a 300 mm diameter base plate. The maximum impact force was 7.07 kN.

In order to determine appropriate dimensions of the test pits, a three-dimensional model was established for a finite element analysis (FEA). Since the finite element model is symmetric, the finite element can be analyzed on ¼ of the model as shown in Figure 2.10. To reflect the Zorn 200 LWD to be used for the test pit experiments, the load plate was assumed to be 10 mm thick with a diameter of 300 mm and a maximum pressure of 7.07 kN. The typical modulus of elasticity of steel, 210 GPa, was used as the modulus of the load plate. A relatively large initial soil size of 5m × 5m × 5m (green cuboid in Figure 2.10) was assumed in the finite element analysis to eliminate possible influence of boundary conditions. To simulate the testing condition in the test pits, the size of the material to be tested (purple cuboid in Figure 2.10) was specified as a height of 1 m and equal length and width of minimum 300 mm.

The material was assumed to be homogeneous and elastic. Two types of boundary conditions between the tested material and the surrounding soil were utilized in the analysis. One condition was that the vertical friction between the material and the soil was zero (frictionless) and the other one was that the material and the soil were fully bonded. The finite element analysis was conducted first with the minimum value of 0.3 m length and width. The calculated deflection at 0.3 m length and width is denoted as $d_{0.3}$. The analysis was then continued with an increased length and width. The deflection at a given length and width is denoted as d. The deflection ratios of $d/d_{0.3}$ at different withes are shown in Figure 2.11. The curves in the figure indicate that the calculated deflection ratios tended to be stable when the width of the material increased to 1.0 m and beyond under both frictionless and fully bonded conditions. Based on this, the size of the test pits was determined as 1m × 1m × 1m to provide sufficient testing space as well as satisfactory analysis accuracy.

2.2.2 Test Pit Experiments

Two 3 ft. × 3 ft. × 3 ft. test pits were constructed to test the No. 53 and No. 43 aggregate materials. The soil in the bottom of the pit was first compacted before placing aggregates. The process of the experiment included the following steps (Figure 2.12): mixing water with the material, placing a layer of the material and compacting the layer with a jumping jack, leveling the layer, and measuring deflection at the center of the pit

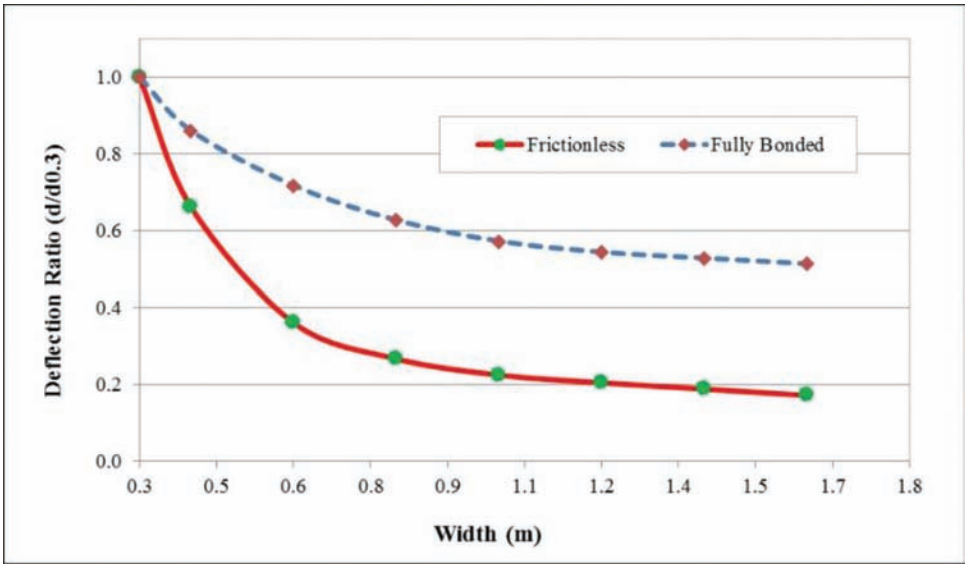


Figure 2.11 Variation of deflection with test pit width.

Figure 2.12 Test pit experiment process.

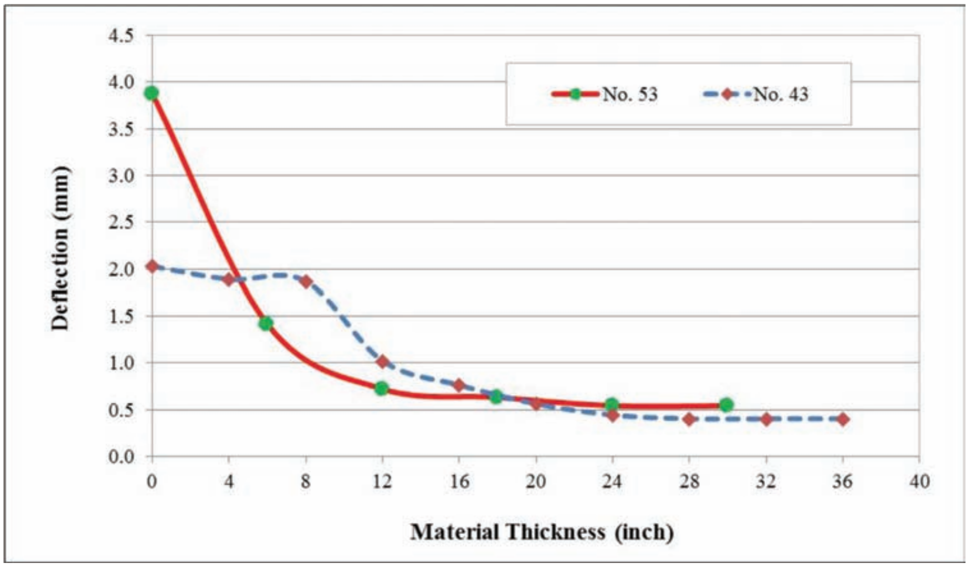


Figure 2.13 Variation of deflection with layer thickness in test pits.

with LWD. To assure sufficient compaction at each layer, the moisture content of the material must be at or close to the optimum value and the material must be compacted at least two times and deflection must be measured after each compaction. If the difference between the two deflections is less than 0.01 mm, the compaction is considered satisfactory. Otherwise, additional compaction will be performed until the difference between deflections of two adjacent compactions is below 0.01 mm.

The thickness of each layer was 6 in. for the No. 53 material and 4 in. for the No. 43 material. The compaction and measurement process at each test pit was repeated until the test pit was full. The test results are illustrated in Figure 2.13. The two curves in the figure demonstrates different effects of the materials on the deflections or moduli as more materials were added to the structures. For No. 53, the deflection decreases as the structure thickness increases. It is apparent that the No. 53 material improves the overall stiffness of the structure. However, for No. 43, the deflection remains stable when as much as 8 in. of the material are added to the structure. That is, the No. 43 material would not contribute to the structural capacity during construction when the thickness of the material is less than 8 in.

2.2.3 Analysis of Modulus

The modulus of the compacted material in a test pit can be treated as an elastic half space modulus. The elastic half space modulus of the material can be obtained through back-calculation using the LWD measured deflection. The soil in the test pit is compacted before the first layer of the material is placed. The modulus of the compacted soil is used as the subgrade modulus. After the first layer of the material is placed and sufficiently compacted, the new layer and the compacted soil will form a two-layer structure system. The modulus of the two-layer structure can be back calculated with the LWD measured deflection. This two-layer structure can be considered an equivalent subgrade if a new layer is added above the two-layer structure. Therefore, the modulus of the two-layer structure can be treated as an equivalent subgrade modulus for the new layer to be added. Subsequently, a new modulus can be obtained after the new layer is placed and compacted on the two-layer structure. Thus, the new layer and the two-layer structure below it will form a new structure system. This new structure system will then be treated as a new equivalent subgrade with a new equivalent subgrade modulus if another layer is added on top of the structure system. This process can be repeated as needed to add the desired thickness of the material.

Following the process described above, the moduli of the No. 43 and No. 53 materials were obtained. The material moduli from the laboratory and the test pits are listed in Table 2.6. As the thickness of the material in the test pit increased to a certain level, the equivalent subgrade modulus became stable. The stabilized modulus represents the material's elastic half space modulus. It is interesting to note that the elastic half space moduli of the two materials (51.69 MPa for No. 43 and 38.05 MPa for No. 53) are rather close to the laboratory moduli measured in Proctor molds (46.71 MPa for No. 43 and 35.32 MPa for No. 53).

In an elastic two-layer system, if the subgrade modulus is higher, the modulus of the added layer will be relatively lower for a given surface deflection. To examine the relationship between the modulus of the subgrade (or the equivalent subgrade) and the modulus of the added layer, the values of the equivalent subgrade modulus and the layer modulus are plotted in Figures 2.14 and 2.15 for the No. 43 and No. 53 materials, respectively. The two figures show the layer moduli of the materials corresponding to the given

TABLE 2.6
Laboratory and Test Pit Material Moduli

	Laboratory		Test Pits		
Material	**Deflection (mm)**	**Modulus (MPa)**	**Thickness (inch)**	**Deflection (mm)**	**Equivalent Subgrade Modulus (MPa)**
No.43	0.369	46.71	0	2.03	10.19
			4	1.89	10.94
			8	1.87	11.06
			12	1.02	20.27
			16	0.76	27.20
			20	0.56	36.92
			24	0.44	46.99
			28	0.4	51.69
			32	0.4	51.69
			36	0.4	51.69
No.53	0.488	35.32	0	3.88	5.34
			6	1.42	14.59
			12	0.72	28.58
			18	0.63	32.65
			24	0.54	38.05
			30	0.54	38.05

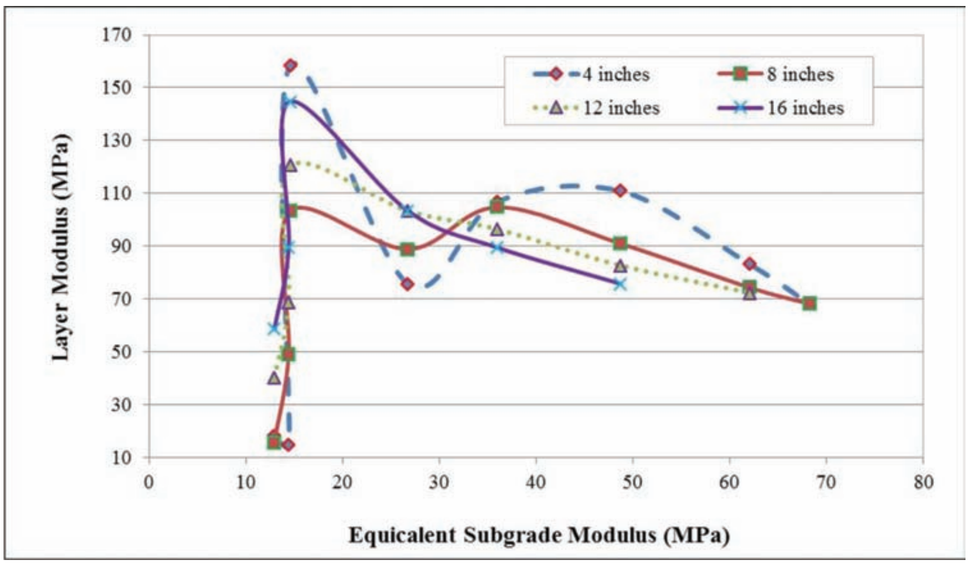


Figure 2.14 Subgrade and layer modulus (No. 43).

equivalent subgrade moduli with different thicknesses of layers above the equivalent subgrade.

Figure 2.14 demonstrates that for No. 43 the relationship between the layer modulus and the equivalent subgrade modulus is not an inverse relationship as it would be expected for an elastic half space system. This could imply that the two-layer system in an elastic half space may not be appropriate for the No. 43 material. On the other hand, as shown in Figure 2.15, the relationship between the equivalent subgrade modulus and layer modulus of the No. 53 material displays a clear inverse relationship. Therefore, the two-layer system in an elastic half space should be appropriate for the No. 53 material. As the equivalent modulus is above 40 MPa, the layer modulus tends to be stable for each new layer added to the structure system.

2.2.4 Applications of Modulus and Deflection in Compaction

In order to use LWD deflection to control the quality of compaction of pavement layers, it is desired to relate the layer modulus of the material to other material pertinent properties that are relevant to compaction. Since the No. 53 material displayed good fit of the two-layer elastic half space model, the following equations are developed with the No. 53 data and are applicable to the No. 53 material only. The layer modulus, E_{layer}, can be expressed in terms of the material modulus obtained in the Proctor mold, E_{mold} as follows:

$$E_{layer} = f_1 f_2 E_{mold} \tag{2.7}$$

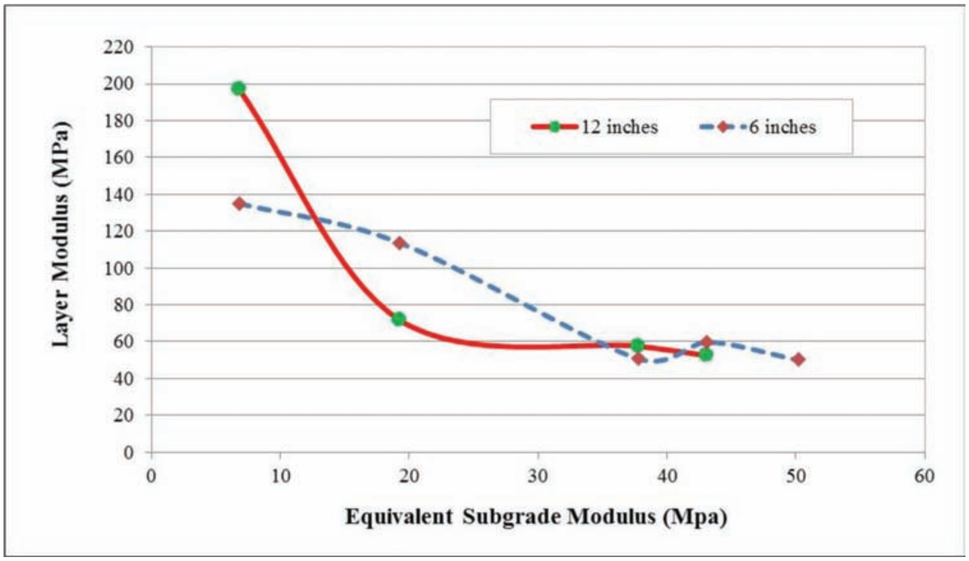


Figure 2.15 Subgrade and layer modulus (No. 53).

TABLE 2.7
Maximum Deflections Calculated under Different Conditions

h (inch)	E_0 (psi)	$m_o - 4\%$	$m_o - 3\%$	$m_o - 2\%$	$m_o - 1\%$	$\geq m_o$
6	725	0.848	0.922	1.021	1.149	1.298
	1450	0.642	0.696	0.769	0.863	0.970
	4000	0.418	0.452	0.510	0.580	0.661
	6000	0.349	0.380	0.420	0.480	0.565
	9000	0.265	0.291	0.330	0.380	0.452
	12700	0.212	0.234	0.265	0.316	0.387
	14000	0.198	0.219	0.263	0.300	0.370
12	725	0.513	0.563	0.630	0.720	0.830
	1450	0.393	0.433	0.489	0.566	0.661
	4000	0.283	0.318	0.372	0.451	0.555
	6000	0.261	0.298	0.352	0.437	0.564
	9000	0.214	0.248	0.299	0.379	0.501
	12700	0.184	0.215	0.265	0.342	0.462
	14000	0.176	0.207	0.256	0.333	0.452
18	725	0.294	0.343	0.410	0.500	0.601
	1450	0.247	0.287	0.347	0.441	0.567
	4000	0.200	0.236	0.295	0.384	0.514
	6000	0.189	0.227	0.284	0.380	0.520
	9000	0.167	0.204	0.260	0.350	0.488
	12700	0.154	0.188	0.243	0.331	0.467
	14000	0.150	0.184	0.238	0.327	0.463

where, f_1 = adjust factor for moisture content, and f_2 = adjust factor for structure effects.

The adjust factor for moisture content can be developed through regression method as:

$$f_{1-No.53} = -9955.3(m-m_o)^3 + 1255.9(m-m_o)^2 + 35.02(m-m_o) + 1 \quad (2.8)$$

where, m = moisture content, and m_o = optimum moisture content.

The adjust factor for structure effect can be developed through regression method as:

$$f_{2-No.53} = \begin{cases} = 39.008E_0^{-0.863} & when\ h = 6'' \\ = 19.463E_0^{-0.704} & when\ h = 12'' \end{cases} \quad (2.9)$$

where, m = moisture content, m_o = optimum moisture content, and E_o = the equivalent subgrade modulus. When $E_o \geq 40$ MPa, use E_o = 40 MPa.

With Equations 2.7, 2.8, and 2.9, E_{layer} can be calculated with laboratory modulus E_{mold}, moisture contents (m and m_o), layer thickness h, and the equivalent subgrade modulus E_o. With the calculated E_{layer}, the corresponding deflection can be computed with the elastic layered system method. The computed deflections of different layers can then be utilized as compaction quality control criteria. To demonstrate the calculation process, an example of the No. 53 material is presented as follows:

If the subgrade modulus is measured as $E_o = 725$ psi, the actual moisture content of the material is 10%, the optimum moisture content is 11%, the laboratory modulus $E_{mold} = 36$ MPa, and a layer of 6 in. is to be added. With the given values, the layer modulus is calculated as $E_{layer} = 513$ MPa. With the two-layer system theory, the deflection corresponding to this layer modulus is computed as 1.149 mm. Similarly, the deflection values for various moisture contents and subgrade conditions can be calculated as presented in Table 2.7 on the previous page.

3. ANALYSIS OF HISTORICAL LWD DATA

3.1 Subgrade Treatments and LWD Deflections

3.1.1 Typical Subgrade Treatments

In order to examine the specified LWD target deflections and the in-situ measured deflections, the historical LWD data recorded by all districts in the years of 2015 and 2016 were gathered, processed, and analyzed. The magnitudes of deflections under the given LWD impact loading are directly affected by the stiffness or strength of the subgrade. Therefore, the measured deflections were grouped in terms of the types of subgrade treatments. The subgrade treatments in Indiana are illustrated in Table 3.1 as defined in the INDOT Standard Specifications (INDOT, 2017b). It should be noted that the two treatments, i.e., Type IA and Type IIIA, have been deleted from the Standard Specifications since September 2014.

Historical Compaction QA Records. In the current practice of compaction quality control using LWD, the INDOT geotechnical engineers may specify a target deflection value, i.e., maximum allowable deflection in most cases, for subgrade treatment of each construction project based on the standard specifications, aggregate materials, type of project, and in-situ conditions. After appropriate compaction, the measured LWD deflection must be less than or equal to the specified target deflection to assure sufficient compaction. The typical items of the Indiana deflection datasets include district, contract type, contract number, subgrade treatment type, target deflection, and measured deflection. Table 3.2 shows the recorded values of target deflections and measured deflections of some construction projects. It is shown that the differences between the target and measured deflections vary considerably from project to project.

TABLE 3.1
Indiana Subgrade Treatments

Subgrade	Description
Type I	This treatment consists of 24 in. of compacted soil.
Type IA	Effective Sept. 2014, Type IA has been deleted from the Standard Specifications.
Type IB	This treatment consists of 14 in. chemical soil modification.
Type IC	This treatment consists of 12 in. of subgrade excavated and replaced with coarse aggregate No. 53.
Type II	This treatment consists of 6 in. of the subgrade excavated and replaced with coarse aggregate No. 53.
Type IIA	This treatment consists of 8 in. chemical soil modification.
Type III	This treatment consists of in-place compaction.
Type IIIA	Effective Sept. 2014, Type IIIA has been deleted from the Standard Specifications.
Type IV	This treatment consists of 12 in. of the subgrade excavated and replaced with coarse aggregate No. 53 on Type IB geogrid.
Type V	This treatment consists of 3 in. of subgrade excavated and replaced with 3 in. coarse aggregate No. 53.

TABLE 3.2
Target and Measured Deflections

District	Contract Type	Contract Number	Target Deflection	Measured Deflection	Treatment Type
Fort Wayne	B	28960	0.511	0.472	IC
Vincennes	B	29719	0.360	0.340	IIA
Greenfield	R	29895	0.600	0.483	II
Crawfordsville	RS	30902	0.300	0.299	IC
Greenfield	R	30935	0.467	0.402	IB
Greenfield	B	31302	0.500	0.410	IC
Crawfordsville	R	31532	0.523	0.500	II
Crawfordsville	R	31536	0.272	0.254	IB

In order to further illustrate magnitudes of the differences between the target and measured deflections, the deflection measurements from eight construction projects were calculated and the results are presented in Table 3.3 and Figure 3.1. As clearly shown in the figure, the difference ranges are from very small (0.33%) to very large (19.5%) for the eight selected projects. A large difference between the target and measured deflections may indicate the target deflection was specified too small or too large. If the target deflections are routinely too small, it would become impractical and costly to fully comply with the requirements for compaction. An unreasonably large target deflection, however, may suggest that the requirements are too lax to assure a sufficient compaction. In addition, lax compaction requirements may also result in concerns about uniform compaction. Because one of the main objectives of this study was to develop reasonable and consistent maximum allowable deflections for aggregate compaction in small areas, it is necessary to analyze the target and measured deflections of the completed construction projects to determine if their differences were significant.

TABLE 3.3
Differences between Target and Measured Deflections

Project	Target Deflection	Measured Deflection	Difference (%)
1	0.511	0.472	7.63
2	0.360	0.340	5.56
3	0.600	0.483	19.50
4	0.300	0.299	0.33
5	0.467	0.402	13.92
6	0.500	0.410	18.00
7	0.523	0.500	4.40
8	0.272	0.254	6.62

3.2 Statistical Analysis of Target and Measured Deflections

3.2.1 Analysis of 2015 Deflection Data

To determine if the recorded target and measured deflections differ significantly, the difference between the target and corresponding measured deflections was used as a variable in the statistical analysis performed with one sample t-test, one-sample median test, or one-sample Wilcoxon signed rank test (Samuels, Witmer, & Schaffner, 2012; Elliott & Woodward, 2010). There were 38 highway and bridge projects with LWD deflection data that could be used in the statistical analysis. The details of the 38 projects are presented in Table 3.4. It should be noted that 28 out of 38 projects were the subgrade treatment IC (12 in. of subgrade excavated and replaced with coarse aggregate No. 53) as this type of subgrade treatment has been the most common type of subgrade treatment in Indiana. There were also three Type IB (14 in. chemical soil modification) projects and seven Type II (6 in. of the subgrade excavated and replaced with coarse aggregate No. 53) projects. The main statistics of the deflection data are shown in Table 3.5 for the subgrade treatment projects in 2015. As can be seen in Table 3.5, Type IC projects had the largest differences between target and measured deflections and the largest standard deviations in terms of the deflection differences.

Although the statistics in Table 3.5 might imply that the target deflections are too greater than the measured deflections, a definite conclusion could not be generalized without a proper statistical test. In order to use an appropriate statistical test method, the normality of the deflection data should be determined first. Two most frequently used normality test methods are Shapiro-Wilk (SW) test and Kolmogorov-Smirnov (KS) test.

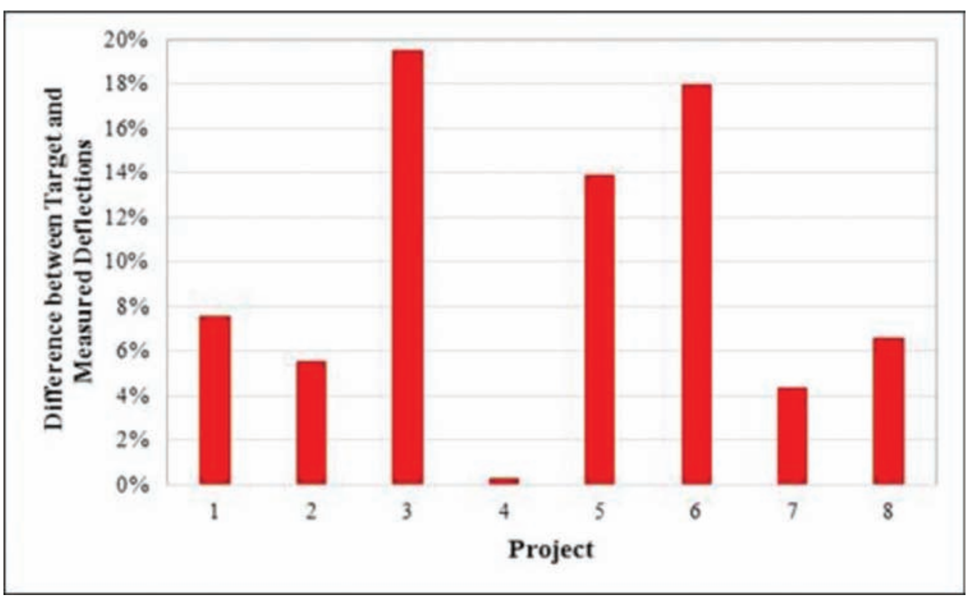


Figure 3.1 Differences between target and measured deflections.

TABLE 3.4
2015 Deflection Dataset

Observation	District	Contract Type	Contract Number	Target Deflection (mm)	Treatment Type	County	Comment	Measured Deflection (mm)	Deflection Difference (mm)
1	Vincennes	B	34632	1.268	IC	Gibson	No.53	0.450	0.818
2	Vincennes	B	34634	0.535	IC	Lawrence	No.53	0.487	0.048
3	Vincennes	B	34830	0.510	IC	Knox	No.53	0.432	0.078
4	Vincennes	R	37896	0.449	IC	Pery	No.53	0.398	0.051
5	Vincennes	RS	30688	0.660	IC	Warrick	No.53	0.650	0.010
6	Vincennes	RS	38469	2.138	IC	Daviess	–	0.760	1.378
7	Seymour	B	29528	0.631	II	Jackson	No.53	0.488	0.143
8	Seymour	B	31797	0.500	II	Franklin	No.53	0.405	0.095
9	Seymour	B	33830	0.400	IC	Switzerland	No.53	0.348	0.052
10	Seymour	B	34619	0.500	IC	Decatur	No.53	0.492	0.008
11	Seymour	B	34625	0.993	IC	Jackson	Dense Graded Subbase	0.524	0.469
12	Seymour	B	34627	0.500	IC	Washington	Dense Graded Subbase	0.465	0.035
13	Seymour	R	29990	0.444	IC	Bartholomew	No.53	0.380	0.064
14	Seymour	R	33838	0.500	IC	Bartholomew	No.53	0.334	0.166
15	LaPorte	B	28990	0.400	IC	Newton	No.53	0.377	0.023
16	LaPorte	B	34015	0.300	IC	LaPorte	No.53	0.342	-0.042
17	LaPorte	B	34723	0.500	IC	Starke	No.53	0.380	0.120
18	LaPorte	R	33952	0.371	IC	LaPorte	No.53	0.363	0.008
19	LaPorte	RS	34054	0.725	IC	LaPorte	No.53	0.429	0.296
20	LaPorte	T	33906	0.528	IC	Newton	No.53	0.522	0.006
21	Greenfield	B	29154	0.429	II	Howard	No.53	0.376	0.053
22	Greenfield	B	31368	0.600	IB	Delaware	No.53	0.376	0.224
23	Greenfield	B	34063	0.300	IB	Delaware	–	0.240	0.060
24	Greenfield	B	35649	0.500	IC	Grant	No.53	0.470	0.030
25	Greenfield	R	28946	0.500	IB	Rush	No.53	0.460	0.040
26	Greenfield	R	34179	0.500	IC	Marion	–	0.459	0.041
27	Fort Wayne	B	30379	0.475	II	Elkhart	No.53	0.401	0.074
28	Fort Wayne	B	33559	0.740	II	Hunting	No.53	0.457	0.283
29	Fort Wayne	B	34867	0.937	IC	Steuben	No.53	0.613	0.324
30	Fort Wayne	B	34880	0.751	II	Steuben	Dense Graded Subbase	1.000	-0.249
31	Fort Wayne	R	33561	2.350	IC	Dekalb	No.53	2.212	0.138
32	Crawfordsville	B	31511	0.544	II	Hendricks	No.53	0.509	0.035
33	Crawfordsville	B	31591	0.785	IC	Clay	No.53	0.781	0.004
34	Crawfordsville	B	33761	0.437	IC	Clinton	No.53	0.409	0.028
35	Crawfordsville	RS	31598	0.955	IC	Montgomery	No.53	0.796	0.159
36	Crawfordsville	RS	34144	1.160	IC	Clay	No.53	1.105	0.055
37	Crawfordsville	RS	35282	0.358	IC	Vigo	No.73	0.354	0.004
38	Crawfordsville	RS	37893	0.847	IC	Warren	–	0.836	0.011

The alternative hypothesis (H_a) is:

H_a: $\mu_d \neq 0$

TABLE 3.5
Statistics of Deflections in 2015

Treatment	Number of Projects	Mean			Standard Deviation		
		Target	Measured	Deflection Difference	Target	Measured	Deflection Difference
IB	3	0.467	0.359	0.108	0.153	0.111	0.101
IC	28	0.705	0.556	0.151	0.478	0.359	0.285
II	7	0.581	0.519	0.062	0.128	0.217	0.160

It is suggested in the SAS manual (Elliott, Alan, and Woodward, 2010), the SW test is preferred when the sample size is less than 2000. With the SW method, the deflection data in Table 3.4 was tested with a significance level of 95% ($\alpha = 0.05$). The null hypothesis for this test is that the differences between the target and measured deflections are normally distributed. The Pr < W value listed in the output is the p-value. If the p-value is less than 0.05, then the null hypothesis that the data are normally distributed is rejected. If the p-value is greater than 0.05, then the null hypothesis is not rejected. The SW normality test result is in Table 3.6. The test rejects the hypothesis of normality because the p-value (Pr < W) is less than $\alpha = 0.05$. This indicates that with 95% confidence the data does not fit the normal distribution.

Since the deflection data is not normally distributed, the one-sample Wilcoxon signed rank test, rather than the one-sample t-test, should be utilized to test if the deflection differences are significant. The one-sample Wilcoxon signed rank test for the deflection differences is to test if μ_d = (target deflection-measured deflection) is 0. The hypotheses for the test are as follows:

The null hypothesis (H_0) is:

$$H_0: \mu_d = 0$$

The statistics of the one-sample Wilcoxon signed rank test on the 2015 deflection dataset are shown in Table 3.7. With the values in this table, it can be concluded that at the significance level of 95% (α = 0.05), the null hypothesis (H_0) is rejected and the alternative hypothesis (H_a) is accepted because p-value ($Pr \geq |S|$) is less than $\alpha = 0.05$. That is, the differences between target deflections and measured deflections are significant in 2015. In other words, the target deflections were specified too large for the subgrade compaction projects in 2015. It is therefore necessary to evaluate the current target deflections and to develop new target deflections for improvement of subgrade compaction quality.

3.2.2 Analysis of 2016 Deflection Data

More construction projects were documented with LWD deflection data in 2016 than 2015. There were a total of 74 projects with LWD deflection data that could be used in the statistical analysis. The details of the 74 projects are presented in Table 3.8. Similar to the 2015 data, the majority (55 out of 74) of the projects were the subgrade treatment IC (12 in. of subgrade excavated and replaced with coarse aggregate No. 53). The dataset also included 8 Type IB (14 in. chemical soil modification) projects and 11 Type II (6 in. of the subgrade excavated and replaced with coarse aggregate No. 53) projects. The main statistics of the deflection data are summarized in Table 3.9 for the subgrade treatment projects in 2016. It is interesting to note that Type IC projects demonstrated the largest differences between target and measured deflections in 2015, but the smallest in 2016.

TABLE 3.6
Normality Test for Deflection Difference in 2015 Dataset

Test	Statistic		p-Value	
Shapiro-Wilk	W	0.611769	Pr < W	<0.0001

TABLE 3.7
Wilcoxon Signed Rank Test on 2015 Deflection Dataset

Test	Statistic		p-Value	
Signed Rank	S	323.5	Pr ≥ |S|	<0.0001

Similar to the analysis of the 2015 dataset, the normality of the deflection data in 2016 was tested using the Shapiro-Wilk (SW) method. With the SW method, the deflection data in Table 3.8 was tested with a significance level of 95% ($\alpha = 0.05$). The null hypothesis for this test is that the differences between the target and measured deflections are normally distributed. The Pr < W value listed in the output is the p-value. If the p-value is less than 0.05, then the null hypothesis that the data are normally distributed is rejected. If the p-value is greater than 0.05, then the null hypothesis is not rejected. The SW normality test result is in Table 3.10. The test rejects the hypothesis of normality because the p-value (Pr < W) is less than $\alpha = 0.05$. Therefore, with 95% confidence the data does not fit the normal distribution.

Since the deflection data is not normally distributed, the one-sample Wilcoxon signed rank test was performed to test if the deflection differences are significant. The one-sample Wilcoxon signed rank test for the deflection differences is to test if μ_d = (target deflection-measured deflection) is 0. The hypotheses for the test are as follows:

The null hypothesis (H_0) is:

$$H_0: \mu_d = 0$$

The alternative hypothesis (H_a) is:

$$H_a: \mu_d \neq 0$$

The statistics of the one-sample Wilcoxon signed rank test on the 2016 deflection dataset are shown in Table 3.11. With the values in this table, it can be concluded that at the significance level of 95% (α = 0.05), the null hypothesis (H_0) is rejected and the alternative hypothesis (H_a) is accepted because p-value is less than $\alpha = 0.05$. Therefore, the differences between target deflections and measured deflections are significant in 2016. Since the differences between target deflections and measured deflections were statistically significant in both years of 2015 and 2016, it can be concluded that the target deflections were too large for the subgrade compaction projects in these two years.

3.2.3 Comparison of 2015 and 2016 LWD Deflection Datasets

The differences between the target and measured deflections were tested separately for 2015 and 2016 as discussed in the previous sections. The differences were statistically significant according to the normality test

TABLE 3.8
2016 Deflection Dataset

Observation	District	Contract Type	Contract Number	Target Deflection (mm)	Treatment Type	Measured Deflection (mm)	Deflection Difference (mm)
1	Vincennes	R	34562	0.455	IC	0.391	0.064
2	Vincennes	R	34839	0.401	IC	0.386	0.015
3	Vincennes	B	34329	1.430	II	1.085	0.345
4	Vincennes	B	34831	0.400	IC	0.354	0.046
5	Vincennes	RS	37891	0.330	IC	0.327	0.003
6	Vincennes	B	34566	0.400	IC	0.296	0.104
7	Vincennes	R	32502	0.400	II	0.359	0.041
8	Crawfordsville	R	34587	0.475	IC	0.461	0.014
9	Crawfordsville	RS	37934	0.640	IC	0.640	0.000
10	Crawfordsville	RS	31537	0.799	IC	0.803	-0.004
11	Crawfordsville	RS	35518	0.953	IC	0.907	0.046
12	Crawfordsville	B	37404	0.400	II	0.348	0.052
13	Crawfordsville	R	37115	0.607	IB	0.461	0.146
14	Crawfordsville	R	31532	0.523	II	0.500	0.023
15	Crawfordsville	R	37937	0.553	IC	0.553	0.000
16	Crawfordsville	B	34985	0.445	IC	0.396	0.049
17	Crawfordsville	B	34584	0.445	IC	0.389	0.056
18	Crawfordsville	B	31559	0.467	IC	0.420	0.047
19	Crawfordsville	R	31536	0.272	IB	0.254	0.018
20	Crawfordsville	B	34418	0.477	IC	0.477	0.000
21	Crawfordsville	B	31569	0.464	IC	0.588	-0.124
22	Crawfordsville	R	34582	0.785	IC	0.785	0.000
23	Crawfordsville	B	37065	0.599	IC	0.593	0.006
24	Crawfordsville	B	32043	0.599	IC	0.538	0.061
25	Crawfordsville	B	32008	0.600	IC	0.426	0.174
26	Crawfordsville	RS	37934	0.721	IC	0.888	-0.167
27	Crawfordsville	R	31756	0.510	II	0.431	0.079
28	Crawfordsville	B	31654	0.500	IC	0.488	0.012
29	Crawfordsville	RS	31597	0.630	IC	0.574	0.056
30	Crawfordsville	B	35971	0.737	II	0.531	0.206
31	Crawfordsville	R	37115	0.547	IB	0.448	0.099
32	Crawfordsville	B	38161	0.500	II	0.319	0.181
33	Crawfordsville	R	34526	0.347	IC	0.343	0.004
34	Crawfordsville	RS	30902	0.300	IC	0.299	0.001
35	Fort Wayne	R	33557	0.388	IC	0.331	0.057
36	Fort Wayne	B	34319	0.300	IC	0.265	0.035
37	Fort Wayne	B	33507	0.418	IC	0.387	0.031
38	Fort Wayne	B	28960	0.511	IC	0.472	0.039
39	Fort Wayne	R	34915	0.401	IC	0.335	0.066
40	Fort Wayne	B	34878	0.878	IC	0.545	0.333
41	Fort Wayne	B	34312	0.787	IC	0.306	0.481
42	Fort Wayne	B	34315	0.503	IC	0.469	0.034
43	Fort Wayne	B	34923	0.300	IC	0.270	0.030
44	Fort Wayne	R	32029	0.922	IC	0.773	0.149
45	Fort Wayne	B	34321	0.438	II	0.317	0.121
46	Fort Wayne	RS	38286	1.500	II	1.373	0.127
47	Fort Wayne	B	35624	0.551	IC	0.596	-0.045
48	Fort Wayne	RS	34907	0.323	II	0.321	0.002
49	Fort Wayne	B	34315	0.503	IC	0.475	0.028
50	Fort Wayne	B	33442	0.400	IC	0.293	0.107
51	Fort Wayne	RS	34916	0.400	IC	0.379	0.021
52	Fort Wayne	B	34889	0.404	IC	0.396	0.008
53	Greenfield	RS	38324	0.450	IC	0.447	0.003
54	Greenfield	R	34780	0.500	IC	0.306	0.194
55	Greenfield	R	29895	0.600	II	0.483	0.117
56	Greenfield	R	34706	0.500	IB	0.421	0.079
57	Greenfield	B	31302	0.500	IC	0.410	0.090
58	Greenfield	R	34806	0.328	IC	0.297	0.031
59	Greenfield	R	35873	0.500	IC	0.375	0.125
60	Greenfield	B	34412	0.500	IC	0.477	0.023

TABLE 3.8
(Continued)

Observation	District	Contract Type	Contract Number	Target Deflection (mm)	Treatment Type	Measured Deflection (mm)	Deflection Difference (mm)
61	Greenfield	R	34939	0.500	IC	0.452	0.048
62	Greenfield	B	35663	0.400	IC	0.376	0.024
63	Greenfield	R	39396	0.500	IC	0.372	0.128
64	Greenfield	R	35655	0.500	IC	0.444	0.056
65	Greenfield	RS	38323	0.450	IC	0.436	0.014
66	Greenfield	R	36021	1.082	IB	0.918	0.164
67	Greenfield	R	36939	0.300	IB	0.241	0.059
68	Greenfield	R	36027	0.419	IC	0.404	0.015
69	Greenfield	R	36027	0.419	IC	0.381	0.038
70	Greenfield	SR	37053	0.500	IB	0.384	0.116
71	Greenfield	R	30935	0.467	IB	0.402	0.065
72	Greenfield	R	34938	0.600	IC	0.573	0.027
73	Greenfield	B	35028	0.500	IC	0.457	0.043
74	Greenfield	R	36027	0.410	IC	0.379	0.031

TABLE 3.9
Statistics of Deflections in 2016

		Mean			Standard Deviation		
Treatment	Number of Projects	Target	Measured	Deflection Difference	Target	Measured	Deflection Difference
IB	8	0.534	0.441	0.093	0.250	0.210	0.048
IC	55	0.508	0.459	0.050	0.152	0.150	0.092
II	11	0.641	0.534	0.170	0.367	0.350	0.071

TABLE 3.10
Normality Test for Deflection Difference in 2016 Dataset

Test	Statistic		p-Value	
Shapiro-Wilk	W	0.817799	Pr < W	<0.0001

TABLE 3.11
Wilcoxon Signed Rank Test on 2016 Deflection Dataset

Test	Statistic		p-Value	
Signed Rank	S	1087	Pr ≥ |S|	<0.0001

and the Wilcoxon signed rank test. The statistics of measured deflections in Tables 3.5 and 3.9 should reflect the actual compactions on the three types of subgrade treatments, IB, IC, and II in 2015 and 2016. The measured deflections recorded in the two years could be combined to represent the subgrade deflections after compactions if measured deflections of the two years were not statistically different.

As illustrated in the previous sections, the normality tests indicated that the deflections were not normally distributed. Because the measured deflections in 2015 and 2016 involved different projects, it was reasonable to assume that deflection values were from two

TABLE 3.12
Wilcoxon Rank Sum Test Results

Two-Sided Pr ≥ |S - Mean|, p Value	0.1354
99% Lower Confidence Limit	0.1327
99% Upper Confidence Limit	0.1382

independent samples. To determine if two independent samples with non-normal distributions are statistically equal, the Wilcoxon Rank Sum Test can be utilized to perform the statistical comparison of the two samples. The Wilcoxon Rank Sum Test for comparing the measured deflections in 2015 and 2016 is to test if the difference between the measured deflections (MD) in 2015 and 2016 is 0 at a confidence level of 95% (α = 0.05). The hypotheses for the test are as follows:

The null hypothesis (H_0) is:

$$H_0: \mu_d = MD_{2016} - MD_{2015} = 0$$

The alternative hypothesis (H_a) is:

$$H_a: \mu_d \neq 0$$

The statistics of Wilcoxon Rank Sum Test on the measured deflections in 2015 and 2016 are presented in Table 3.12. Because p-value = 0.1354 > α = 0.05, it can be concluded that at the significance level of 95%

TABLE 3.13
Statistics of Measured Deflections with Combined Datasets

Treatment	Number of Projects	Mean	Min	Max	Std. Dev.	95% LCLM	95% UCLM
IB	11	0.42	0.24	0.92	0.19	0.29	0.54
IC	83	0.50	0.27	2.21	0.25	0.44	0.55
II	18	0.54	0.32	1.37	0.30	0.39	0.69

(α = 0.05), the null hypothesis (H_0) is accepted. Thus, it can be conclude that the measured deflections in 2015 are not statistically different from the measured deflections in 2016. Since the measured deflections in the two years are statistically equal, the two-year data can be combined as shown in the Table 3.13. This table contains the following statistics: mean, maximum, minimum, standard deviation, lower confidence limits for means (LCLM), and upper confidence limit for means (UCLM). These statistics will be useful as a basis for determining appropriate target deflections. Among the three types of subgrade treatments, Type II treatments had the highest mean and standard deviation values of the measured deflections and Type IB had the lowest mean and standard deviation values of the measured deflections.

4. UPGRADES TO MAXIMUM ALLOWABLE DEFLECTIONS

4.1 Experimental-Mechanistic (EM) Procedure

4.1.1 Major Elements

The determination of upgrades to the current maximum allowable deflections (MADs) was made using an experimental-mechanistic (EM) procedure as shown in Figure 4.1. This EM procedure consists of three major elements, including laboratory experiment, test pit assessment, and field implementation that are connected with one another through analytical solutions. As presented in Chapter 2, the laboratory experiment was utilized to determine the target stiffness parameter at the optimum moisture content for compaction quality control based on LWD test results using the current Proctor test. The effect of moisture content was quantified to establish the relationship between the stiffness parameter and moisture content that takes into consideration the variability of moisture content in the field. The test pit assessment was performed to fill the gap between the mechanical behaviors of the aggregate compacted into the Proctor mold and compacted in the field, respectively. As a result, the results were utilized to determine the effects of boundary condition, support, and layer thickness, and adjust correct the target parameter.

It has been recognized that there are many factors, such as aggregate properties (type, size, and gradation), moisture content, lift thickness, subgrade condition, and compaction effort that may affect the compaction of unbound aggregate materials. For small area constructions such as bridge approach, lane widening, patching, and shoulder, the spaces are usually confined and not accessible to large rollers. Instead, small and lightweight compactors, including rammer, plate compactor, walk-behind roller, and trench compactor, are commonly utilized. It was also observed that in confined areas, water could collect and the subgrade soil tended to dry out slowly. Geotextile or geogrid was widely used between the subgrade and aggregate layer to reduce the effect of water and increase the stability of the subgrade. Therefore, the compaction characteristics of unbound aggregate materials may be different in small areas. It is necessary to adjust the target parameter by taking into consideration the field and construction conditions, particularly compaction effort, subgrade condition, lift thickness, and use of geotextile.

4.1.2 Target Parameter

During a standard LWD test (ASTM E2835-11, 2015), in reality, the surface deflection is directly measured under each drop and the modulus is estimated from the deflection measurement using a theoretical equation based on the LWD device and its configuration. Fundamentally, either deflection or modulus can be used as the target parameter for compaction quality control in the field. The pavement design methods currently utilized by state DOTs, such as the AASHTO (1993) design guide and the mechanistic-empirical design guide (AASHTO, 2015), commonly require the elastic or resilient modulus as one of the fundamental inputs for characterizing the mechanical properties of subgrade soil or aggregate layer. Therefore, it is natural to use modulus as the target parameter when LWD test is utilized for compaction quality control or assurance. As recommended by Schwartz, et al. (2017), the target modulus for compaction quality control of geomaterials using LWD drops can be determined on a compacted Proctor mold in the laboratory. Because the corresponding test is to be conducted as an add-on to either the standard or modified Proctor test (AASHTO T 99-17, 2017; AASHTO T 180-17, 2017), the procedure is familiar to both state DOTs and roadway construction industries and is suitable for implementation without a significant increase in field workload.

To the authors' knowledge, however, there are still several issues about the modulus-based construction quality control. First, the modulus of the soil in the mold is computed using Eq. 2.6 (see Chapter 2). Notice that this equation is derived from the theory of elasticity for a cylinder of elastic material with constrained lateral movement by assuming frictionless contact between the compacted sample and the inner wall

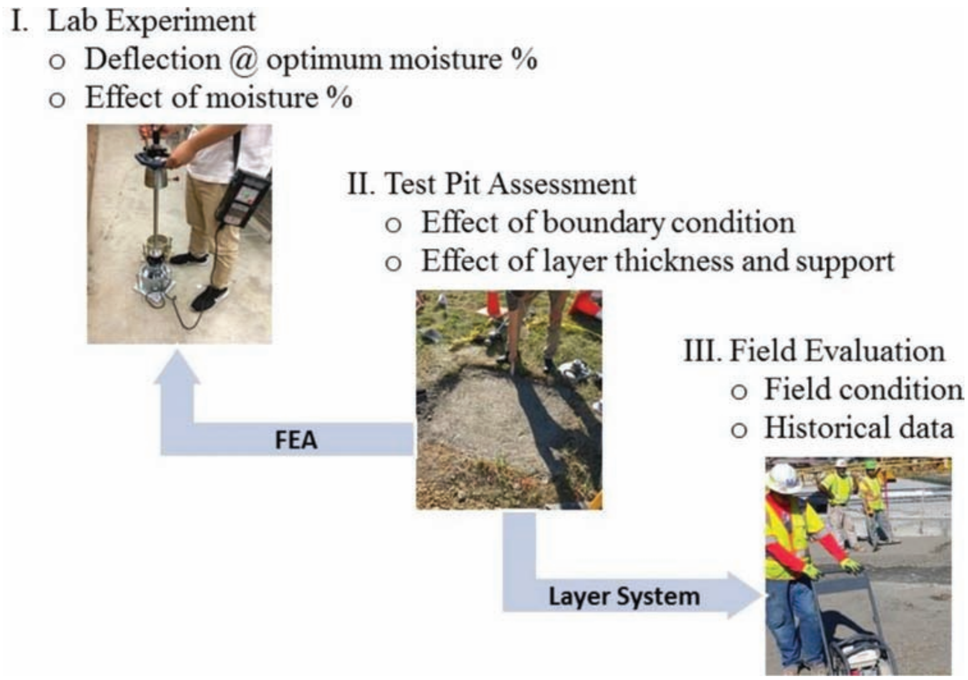


Figure 4.1 Graphical illustration of the EM procedure.

of the mold. It is demonstrated that in Chapter 2, friction does exist between the aggregate and the wall of the mold and increases as aggregate size increases and moisture content decreases. Errors may be involved in the calculation of the target modulus based on the LWD deflection on the compacted aggregate in a Proctor mold. Second, during field LWD testing, the modulus is estimated using the Boussinesq solution for an elastic half-space under an axisymmetric surface loading. Different LWD devices, however, may have different features in terms of sensing, applied load pulse and plate rigidity. These factors may induce additional degree of variance into the estimated modulus. Most importantly, INDOT has a proven track record in construction quality control or assurance based on LWD deflection. Changing to a modulus-based quality control or assurance would produce data that could not be compared with historical data. It is advisable for INDOT to continue to use deflection as the target parameter for construction quality control and assurance.

4.2 Adjustment of Target Deflection Values

4.2.1 Degree of Compaction Achievable in Small Areas

Use of lightweight compactors and effect of the subgrade condition may become important for small area compaction. Presented in Figure 4.2 are two photos taken during compaction. Plate compactors are commonly used for compaction in bridge approach construction. However, walk-behind rollers are usually ideal for compaction in pavement patching. It is well known that a greater compaction effort not only reduces the optimum moisture content, but also increases the maximum dry density. The use of a lightweight compactor for small area compaction may present a difficult problem in achieving the desired degree of compaction. In small areas, subgrades in poor, particularly wet or saturated conditions may become too weak to be a working platform and even result in a waterbed effect inhibiting compaction of the overlying materials (Schaefer et al., 2017). Unbound aggregate materials also tend to suck water from the saturated subgrade and pump during compaction (Burak, 2005). Therefore, it may become very challenging to compact the geomaterials in small and confined areas to the same degree of compaction in large areas.

Field tests were conducted to compare the compaction of unbound aggregates in both small and large areas. Plotted in Figure 4.3 are the average deflections of No. 53 aggregate materials upon completion in both small and large areas. The gray and black bars, respectively, indicate small and large area compactions. It is shown that there exist evident differences between the compaction degrees of No. 53 aggregate materials in small and large areas, respectively. While LWD deflections as low as 0.46 mm obtained for bridge approach compaction on US 41, the LWD deflection measurements in small area compaction were generally larger than those in large area compaction. Particularly when the moisture content in the subgrade soil is high, such as the bridge approach on I-69 and patching on SR 18, the LWD deflection measurements may be very large. Therefore, it may become practically impossible to compact the geomaterials in small areas to the same degree of compaction in large areas. The target deflection values should be adjusted according to the characteristics of compaction in small areas.

Figure 4.2 Photos of geomaterial compaction in small areas.

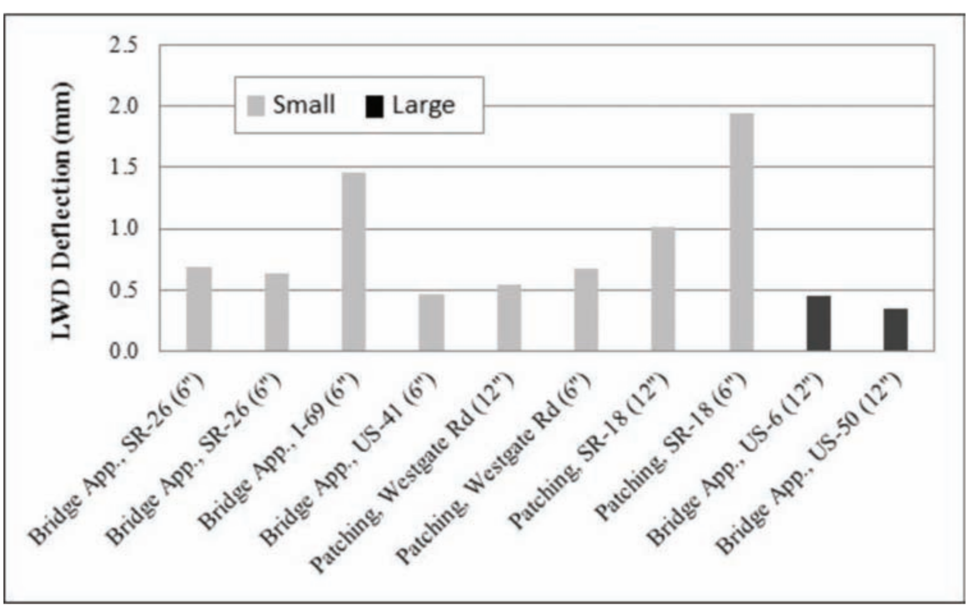


Figure 4.3 Comparison of compaction degrees in small and large areas.

4.2.2 Adjusted Target Deflections to Small Area Compaction

Table 4.1 presents a summary of the LWD deflections measured on No. 53 aggregate materials in 2016 and 2017, respectively. The All column covers all thickness values, including 6 in. and 12 in. The deflection ratio is the ratio between the average deflections for lightweight and large compactors, respectively. For 6 in. thick aggregate in 2016, for example, the average deflection is 0.680 mm for lightweight compactor and 0.550 mm for large roller. The deflection ratio is 0.680/0.550 = 1.236. Evidently, the deflections for lightweight compactor are greater than those for large roller, regardless of aggregate layer thickness. The authors also examined a total of 20 historical projects, the average deflection is 0.560 mm for bridge approach projects, 0.420 mm for road reconstruction projects, and 0.500 mm for resurfacing projects. The deflection ratio is 1.120 or 1.333 between bridge approach and reconstruction or resurfacing projects.

Figure 4.4 shows the overall deflection ratios calculated for 2016, 2017, and historical projects, without considering the effect of aggregate layer thickness. The overall deflection ratio varies between 1.192 for 2017 projects and 1.239 for 2016 tested projects. The variation is less than 4.0%. No rigorous methods are currently available to adjust the target deflection for small area compaction. To avoid unnecessary complexity, it is recommended to use the average of the three overall deflections ratio values in Figure 4.4, i.e., 1.220, as the adjustment factor for considering the characteristics of small area compaction. As an illustration, multiplying the target deflections given in Table 2.7 by 1.200 yields the adjusted target deflections for the compaction of No. 53 aggregate materials in small areas as shown in Table 4.2.

4.3 Determination of Maximum Allowable Deflections

4.3.1 Effect of Inverted Layer System

Notice that in Table 4.2, the adjusted target deflections increase as the thickness of No. 53 aggregate increases when the subgrade modulus equals 12700 psi

TABLE 4.1
Statistics Summary of LWD Deflection Measurements

		Lightweight Compactor			Large (Roller) Compactor		
Aggregate Layer Thickness (in.)		**6**	**12**	**All**	**6**	**12**	**All**
2016	No. of Projects	6	3	10	3	4	14
	No. of Test Points	51	22	76	82	52	249
	Average Deflection (mm)	0.680	0.522	0.634	0.550	0.465	0.467
	Deflection Ratio	1.236	1.124	1.357	–	–	–
2017	No. of Projects	2	N.A.	2	1	2	8
	No. of Test Points	23	N.A.	23	3	16	49
	Average Deflection (mm)	0.597	N.A.	0.597	0.405	0.499	0.537
	Deflection Ratio	1.318	N.A.	1.067	–	–	–

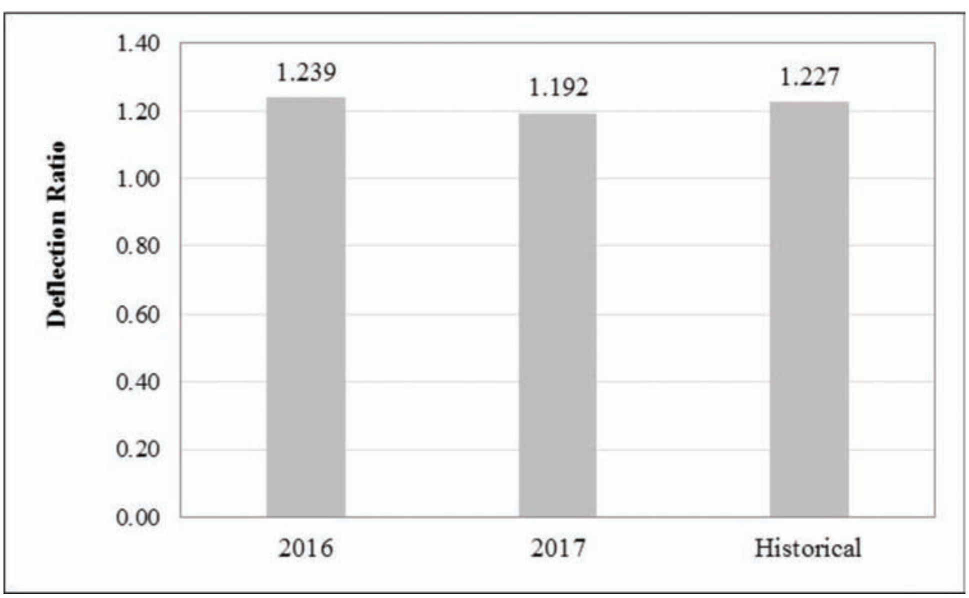


Figure 4.4 Overall deflection ratios for 2016, 2017, and historical projects.

TABLE 4.2
Adjusted Target Deflections for Compaction of No. 53 Aggregate in Small Areas

Aggregate Layer Thickness (in.)	Subgrade Modulus (psi)	Adjusted Target Deflection (mm)							
		$w_0 - 4\%$	$w_0 - 3\%$	$w_0 - 2\%$	$w_0 - 1\%$	w_0*	$w_0 + 1\%$	$w_0 + 2\%$	$w_0 + 3\%$
6	725	1.035	1.125	1.246	1.402	1.584	1.615	1.631	1.663
	1450	0.783	0.849	0.938	1.053	1.183	1.207	1.219	1.243
	4000	0.510	0.551	0.622	0.708	0.806	0.823	0.831	0.847
	6000	0.426	0.464	0.512	0.586	0.689	0.703	0.710	0.724
	9000	0.323	0.355	0.403	0.464	0.551	0.562	0.568	0.579
	12700	0.259	0.285	0.323	0.386	0.472	0.482	0.486	0.496
	14100	0.242	0.267	0.321	0.366	0.451	0.460	0.465	0.474
12	725	0.626	0.687	0.769	0.878	1.013	1.033	1.043	1.063
	1450	0.479	0.528	0.597	0.691	0.806	0.823	0.831	0.847
	4000	0.345	0.388	0.454	0.550	0.677	0.691	0.697	0.711
	6000	0.318	0.364	0.429	0.533	0.688	0.702	0.709	0.722
	9000	0.261	0.303	0.365	0.462	0.611	0.623	0.630	0.642
	12700	0.224	0.262	0.323	0.417	0.564	0.575	0.581	0.592
	14100	0.215	0.253	0.312	0.406	0.551	0.562	0.568	0.579
18	725	0.359	0.418	0.500	0.610	0.733	0.748	0.755	0.770
	1450	0.301	0.350	0.423	0.538	0.692	0.706	0.712	0.726
	4000	0.244	0.288	0.360	0.468	0.627	0.640	0.646	0.658
	6000	0.231	0.277	0.346	0.464	0.634	0.647	0.653	0.666
	9000	0.204	0.249	0.317	0.427	0.595	0.607	0.613	0.625
	12700	0.188	0.229	0.296	0.404	0.570	0.581	0.587	0.598
	14100	0.183	0.224	0.290	0.399	0.565	0.576	0.582	0.593

*w_0 = optimum moisture content.

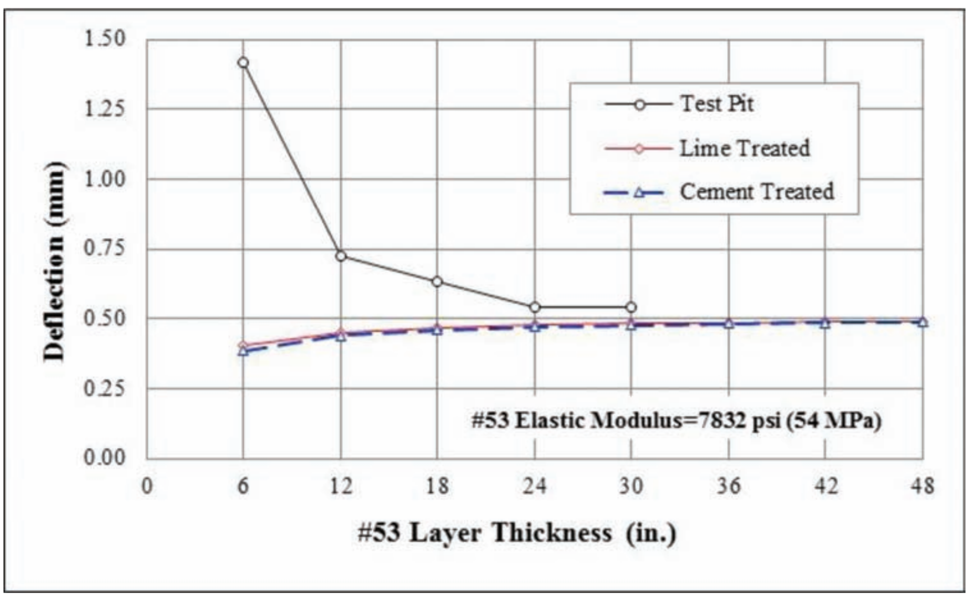


Figure 4.5 Variation of deflection with layer thickness (inverted two-layer system).

or 14100 psi. This is due to the mechanistic behavior of inverted pavement structure. It is a common practice to place unbound aggregate materials on chemically modified subgrades in pavement construction. The INDOT specifications require an allowable maximum deflection of 0.30 mm and 0.27 mm for lime and cement treated subgrades, respectively (INDOT, 2017b). The former results in a subgrade modulus of 12700 psi and the latter results in a subgrade modulus of 14100 psi. It is well known that the resilient moduli of unbound aggregate materials are determined by repeated load triaxial compression tests and range between 30,000 psi

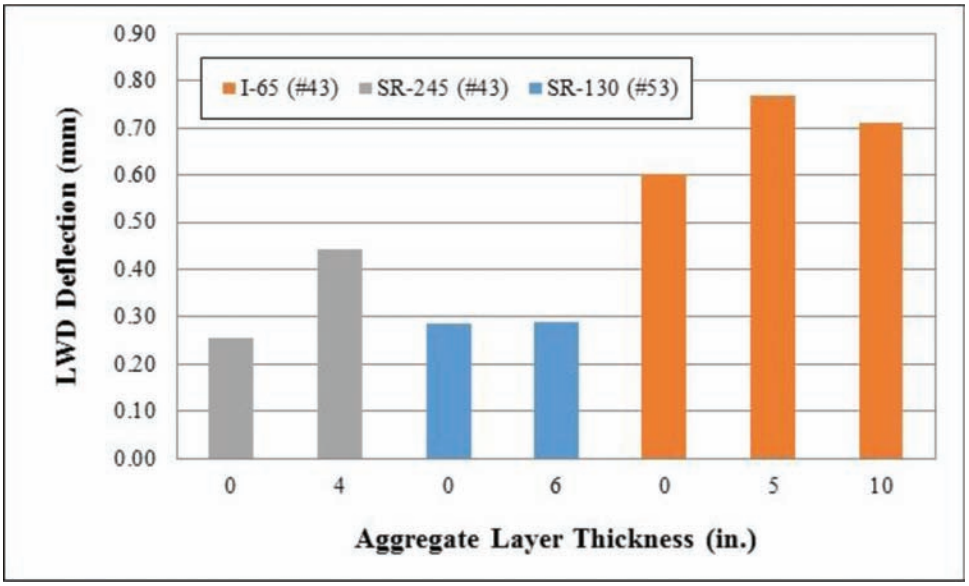


Figure 4.6 Deflections for unbound aggregate placed on cemented treated subgrade.

and 40,000 psi (ARA Inc., 2005). However, unbound aggregate materials are not fully elastic. Their mechanistic behavior under LWD loading cannot be the same as that under repeated triaxial loading. In the resilient modulus test, the ratio of confining pressure to cyclic stress ratio is commonly less than 1/3 (AASHTO T 307-99, 2017).

In the in-situ LWD test, the lateral stress may be very low because the impulse load generated by dropping a weight can be transmitted to the deeper layer over a small area and the particle interlocking in the unbound aggregate materials may not be fully produced. Therefore, relatively large deflection and low lateral stress may be produced and the in-situ modulus determined by the LWD test may be much less than that by the resilient modulus test (Z. M. Tan, personal communication, June 3, 2018). The above indicates that placing an unbound aggregate layer on a chemically modified subgrade may produce an inverted two-layer structure. Figure 4.5 shows the deflection variation with the layer thickness for an inverted two-layer system. The modulus for the No. 53 aggregates is the dynamic modulus determined by the LWD test performed in the test pit. The deflections increase as the aggregate layer thickness increases for both the lime and cement treated subgrades. However, the deflections remain unchanged when the aggregate layer thickness exceeds 36 in, which indicates that effect of the subgrade may be neglected when the aggregate layer thickness exceeds 36 in. This finding can also be made from the LWD deflection measurements made in the test pit (see Figure 4.5). It should be highlighted that placing a layer of No. 53 aggregate yields a conventional two-layer system.

4.3.2 Recommended Maximum Allowable Deflections

Nevertheless, the field LWD test results do not fully agree with the variation trend of deflection for the inverted layer system presented in Figure 4.5. As shown in Figure 4.6 are the deflections measured on the unbound aggregate materials placed on cemented treated subgrades for three construction projects. The 0 in. thickness indicates that the deflection was measured on the cement treated subgrade before placing the aggregate materials. It is shown that for all the three projects, the deflections increased after placing the aggregate materials. As specified earlier, the maximum allowable deflection is 0.27 mm for cement treated subgrades, which represents a subgrade modulus of 14100 psi or greater. Therefore, placing a 4 in. or 6 in. unbound aggregate layer on the cement treated subgrade yielded an inverted two-layer system. Moreover, the deflection on the 10 in. thick aggregate layer is less than that on the 5 in. thick aggregate layer on I-65. This, again, can be explained by the effect of lateral stress induced during LWD testing.

There are many possible factors, such as layer thickness, subgrade strength, and degree of compaction, may affect the lateral confining stress in the unbound aggregate layer under the impulse load generated in the in-situ LWD test. For instance, the lateral confining stress increases as the thickness of aggregate layer and the degree of compaction increase. Under the same compaction effort, however, a thinner thickness commonly yields a higher degree of compaction. Therefore, the above factors may interact with each other and it may become very difficult to accurately determine the in-situ effect of the mechanistic behavior of an inverted layer structure system. It is recommended that currently, the effect of inverted layer system should not be considered for determining the maximum allowable deflections. Furthermore, the MADs for a 6 in. aggregate layer can be applied to a 12 in. or 18-in aggregate layer. Presented in Tables 4.3 and 4.4 are the MADs derived from the values in Table 4.2 without considering the effect of inverted layer system. Notice that the

MDAs for the moisture contents of w_0+1%, w_0+2%, and w_0+3% were determined by multiplying the corresponding MDAs at the optimum moisture content (w_0) by a factor of 1.02, 1.03, and 1.05, respectively. These three factors were derived from the laboratory tests performed on the aggregate samples compacted at a moisture content of w_0+1%, w_0+2%, and w_0+3%, respectively.

TABLE 4.3
Recommended MADs for No. 53 Aggregates on Chemically Modified Soil Subgrade

		No. 53 Moisture Content (w_0: optimum moisture content)							
No. 53 Thickness	**Chemically Modified Subgrade***	$w_0 - 4\%$	$w_0 - 3\%$	$w_0 - 2\%$	$w_0 - 1\%$	w_0	$w_0 + 1\%$	$w_0 + 2\%$	$w_0 + 3\%$
(1) Large Compactor Accessible Areas									
6″	Lime Treated	0.212	0.234	0.266	0.316	0.387	0.395	0.399	0.406
	Cement Treated	0.198	0.219	0.263	0.300	0.370	0.377	0.381	0.389
12″	Lime Treated	0.184	0.215	0.265	0.316	0.387	0.395	0.399	0.406
	Cement Treated	0.176	0.207	0.256	0.300	0.370	0.377	0.381	0.389
18″	Lime Treated	0.154	0.188	0.243	0.316	0.387	0.395	0.399	0.406
	Cement Treated	0.150	0.184	0.238	0.300	0.370	0.377	0.381	0.389
(2) Small Areas Using Small Compactors									
6″	Lime Treated	0.259	0.285	0.325	0.386	0.472	0.482	0.486	0.496
	Cement Treated	0.242	0.267	0.321	0.366	0.451	0.460	0.465	0.474
12″	Lime Treated	0.224	0.262	0.323	0.386	0.472	0.482	0.486	0.496
	Cement Treated	0.215	0.253	0.312	0.366	0.451	0.460	0.465	0.474
18″	Lime Treated	0.188	0.229	0.296	0.386	0.472	0.482	0.486	0.496
	Cement Treated	0.183	0.224	0.290	0.366	0.451	0.460	0.465	0.474

*The chemical soil modification chemical should refer to as Item 301.09 of INDOT Standard Specifications, 2018.

TABLE 4.4
Recommended MADs for No. 53 Aggregates on Compacted Soil Subgrade

No. 53 Thickness	Subgrade			No. 53 Moisture Content (w_0: optimum moisture content)							
	Soil Type		DCP	$w_0 - 4\%$	$w_0 - 3\%$	$w_0 - 2\%$	$w_0 - 1\%$	w_0	$w_0 + 1\%$	$w_0 + 2\%$	$w_0 + 3\%$
(a) Large Compactor Accessible Areas											
6″	Clay		6	0.368	0.399	0.443	0.504	0.583	0.595	0.600	0.612
			7	0.340	0.370	0.411	0.473	0.554	0.565	0.571	0.582
			8	0.307	0.335	0.373	0.432	0.510	0.520	0.525	0.536
	Silt		9	0.284	0.311	0.348	0.404	0.480	0.490	0.494	0.504
			11	0.245	0.269	0.303	0.357	0.430	0.439	0.443	0.452
	Sandy		12	0.233	0.256	0.289	0.342	0.414	0.422	0.426	0.434
			15	0.199	0.220	0.251	0.301	0.372	0.379	0.383	0.391
	Backfill	#30	6	0.368	0.399	0.443	0.504	0.583	0.595	0.600	0.612
		#4	7	0.340	0.370	0.411	0.473	0.554	0.565	0.571	0.582
		½″	11	0.245	0.269	0.303	0.357	0.430	0.439	0.443	0.452
		1″	16	0.191	0.212	0.243	0.292	0.362	0.369	0.373	0.380
12″	Clay		6	0.267	0.303	0.358	0.443	0.558	0.569	0.575	0.586
			7	0.255	0.291	0.347	0.431	0.557	0.568	0.574	0.585
			8	0.237	0.272	0.325	0.408	0.532	0.543	0.548	0.559
	Silt		9	0.224	0.259	0.311	0.392	0.515	0.525	0.530	0.541
			11	0.203	0.235	0.286	0.357	0.430	0.439	0.443	0.452
	Sandy		12	0.196	0.228	0.278	0.342	0.414	0.422	0.426	0.434
			15	0.177	0.208	0.251	0.301	0.372	0.379	0.383	0.391
	Backfill	#30	6	0.267	0.303	0.358	0.443	0.558	0.569	0.575	0.586
		#4	7	0.255	0.291	0.347	0.431	0.557	0.568	0.574	0.585
		½″	11	0.203	0.235	0.286	0.357	0.430	0.439	0.443	0.452
		1″	16	0.173	0.203	0.243	0.292	0.362	0.369	0.373	0.380

TABLE 4.4
(Continued)

No. 53 Thickness	Subgrade Soil Type		DCP	No. 53 Moisture Content (w_0: optimum moisture content) $w_0 - 4\%$	$w_0 - 3\%$	$w_0 - 2\%$	$w_0 - 1\%$	w_0	$w_0 + 1\%$	$w_0 + 2\%$	$w_0 + 3\%$
18″	Clay		6	0.192	0.230	0.288	0.380	0.515	0.525	0.530	0.541
			7	0.187	0.224	0.282	0.377	0.514	0.524	0.529	0.540
			8	0.179	0.216	0.272	0.364	0.503	0.513	0.518	0.528
	Silt		9	0.172	0.210	0.265	0.356	0.493	0.503	0.508	0.518
			11	0.163	0.198	0.254	0.357	0.430	0.439	0.443	0.452
	Sandy		12	0.160	0.195	0.250	0.342	0.414	0.422	0.426	0.434
			15	0.150	0.185	0.251	0.301	0.372	0.379	0.383	0.391
	Backfill	#30	6	0.192	0.230	0.288	0.380	0.515	0.525	0.530	0.541
		#4	7	0.187	0.224	0.282	0.377	0.514	0.524	0.529	0.540
		½″	11	0.203	0.235	0.286	0.357	0.430	0.439	0.443	0.452
		1″	16	0.148	0.183	0.243	0.292	0.362	0.369	0.373	0.380
(b) Small Areas Using Small Compactors											
6″	Clay		6	0.449	0.487	0.540	0.615	0.711	0.726	0.732	0.747
			7	0.415	0.451	0.501	0.577	0.676	0.689	0.697	0.710
			8	0.375	0.409	0.455	0.527	0.622	0.634	0.641	0.654
	Silt		9	0.346	0.379	0.425	0.493	0.586	0.598	0.603	0.615
			11	0.299	0.328	0.370	**0.436**	**0.525**	**0.536**	**0.540**	**0.551**
	Sandy		12	0.284	0.312	0.353	**0.417**	**0.505**	**0.515**	**0.520**	**0.529**
			15	0.243	0.268	**0.306**	**0.367**	**0.454**	**0.462**	**0.467**	**0.477**
	Backfill	#30	6	0.449	0.487	0.540	0.615	0.711	0.726	0.732	0.747
		#4	7	0.415	0.451	0.501	0.577	0.676	0.689	0.697	0.710
		½″	11	0.299	0.328	0.370	**0.436**	**0.525**	**0.536**	**0.540**	**0.551**
		1″	16	0.233	0.259	**0.296**	**0.356**	**0.442**	**0.450**	**0.455**	**0.464**
12″	Clay		6	0.326	0.370	0.437	0.540	0.681	0.694	0.702	0.715
			7	0.311	0.355	0.423	0.526	0.680	0.693	0.700	0.714
			8	0.289	0.332	0.397	0.498	0.649	0.662	0.669	0.682
	Silt		9	0.273	0.316	0.379	0.478	0.628	0.641	0.647	0.660
			11	0.248	0.287	0.349	**0.436**	**0.525**	**0.536**	**0.540**	**0.551**
	Sandy		12	0.239	0.278	0.339	**0.417**	**0.505**	**0.515**	**0.520**	**0.529**
			15	0.216	0.254	**0.306**	**0.367**	**0.454**	**0.462**	**0.467**	**0.477**
	Backfill	#30	6	0.326	0.370	0.437	0.540	0.681	0.694	0.702	0.715
		#4	7	0.311	0.355	0.423	0.526	0.680	0.693	0.700	0.714
		½″	11	0.248	0.287	0.349	**0.436**	**0.525**	**0.536**	**0.540**	**0.551**
		1″	16	0.211	0.248	**0.296**	**0.356**	**0.442**	**0.450**	**0.455**	**0.464**
18″	Clay		6	0.234	0.281	0.351	0.464	0.628	0.641	0.647	0.660
			7	0.228	0.273	0.344	0.460	0.627	0.640	0.646	0.658
			8	0.218	0.264	0.332	0.444	0.614	0.626	0.632	0.644
	Silt		9	0.210	0.256	0.323	0.434	0.601	0.613	0.620	0.632
			11	0.199	0.242	0.310	**0.436**	**0.525**	**0.536**	**0.540**	**0.551**
	Sandy		12	0.195	0.237	0.304	**0.417**	**0.505**	**0.515**	**0.520**	**0.529**
			15	0.183	0.226	**0.306**	**0.367**	**0.454**	**0.462**	**0.467**	**0.477**
	Backfill	#30	6	0.234	0.281	0.351	0.464	0.628	0.641	0.647	0.660
		#4	7	0.228	0.273	0.344	0.460	0.627	0.640	0.646	0.658
		½″	11	0.248	0.287	0.349	**0.436**	**0.525**	**0.536**	**0.540**	**0.551**
		1″	16	0.181	0.223	**0.296**	**0.356**	**0.442**	**0.450**	**0.455**	**0.464**

5. PRACTICES RECOMMENDED FOR FIELD COMPACTION, IMPLEMENTATION, CALIBRATION, AND VERIFICATION

5.1 Practical Solutions to Small Area Compaction

5.1.1 Compactor Operating Parameters

Geomaterial compaction decreases voids, compressibility and permeability, and, eventually, increases density, strength, stability and resistance to the effects of moisture and frost heaving. Several technical documents are currently available to be used as a user's guide to compaction theories, testing, control of quality, and on-the-job procedures (Caterpillar Paving Products, 2013; Christopher, Schwartz, & Boudreau, 2006). Nevertheless, those technical documents focus primarily on soil compaction with large rollers. Compacting unbound aggregates may become very difficult because lightweight compactors commonly used for small area compaction vary in weight from several hundred to several thousand pounds. As shown in Figure 5.1 are the LWD deflection measurements made on No. 5 structural backfills. The deflection achieved by the large roller is approximately 40% less than that by the vibratory plate. It is evident that care should be exercised to achieve the desired compaction in small areas.

To achieve the desired degree of compaction of unbound aggregates by a lightweight compactor, it is essential to determine the operating parameters of the compactor, such as operation mode, vibration amplitude and frequency, and rolling speed, at the start of compaction. There are many factors, including to the condition of underlying subgrade, layer thickness, moisture content, and desired compaction. An optimum selection of the compactor operating parameters can only be achieved by experience and experimentation.

Operation Mode. When a lightweight compactor is utilized for compaction in small areas such as bridge approach, lane widening, patching, and shoulder, the compactor should be operated in vibratory mode. Vibrating plates or vibrating rollers may be best suited to these situations. Applying a final static compaction can yield more uniform surface.

Amplitude and Frequency. Vibration amplitude and frequency should be selected according to the factors such as the characteristics of aggregates (type, gradation, moisture content, and thickness) and underlying subgrade condition. Generally, the vibration amplitude should increase as the aggregate layer becomes more compacted. A higher vibration frequency should be used for aggregates with smaller particles.

Rolling Speed. Using a faster rolling compaction speed does not necessarily drive productivity higher, in particular for vibratory compactors. The rolling compaction speed should be adjusted according to vibration frequency, rolling amplitude, and layer thickness. In general, rolling speed should be slow first, and then gradually increase. There exists an optimum working speed to achieve the desired compaction economically and efficiently.

5.1.2 On-The-Job Procedures

Moisture Content. Due to the relatively low force imparted by a lightweight compactor, it becomes very important to compact unbound aggregates at or near the optimum moisture content. At the optimum moisture content, geomaterials can be compacted to the maximum dry density more economically. In addition,the geomaterials compacted at the optimum moisture content tend to provide the necessary stabilities in both density

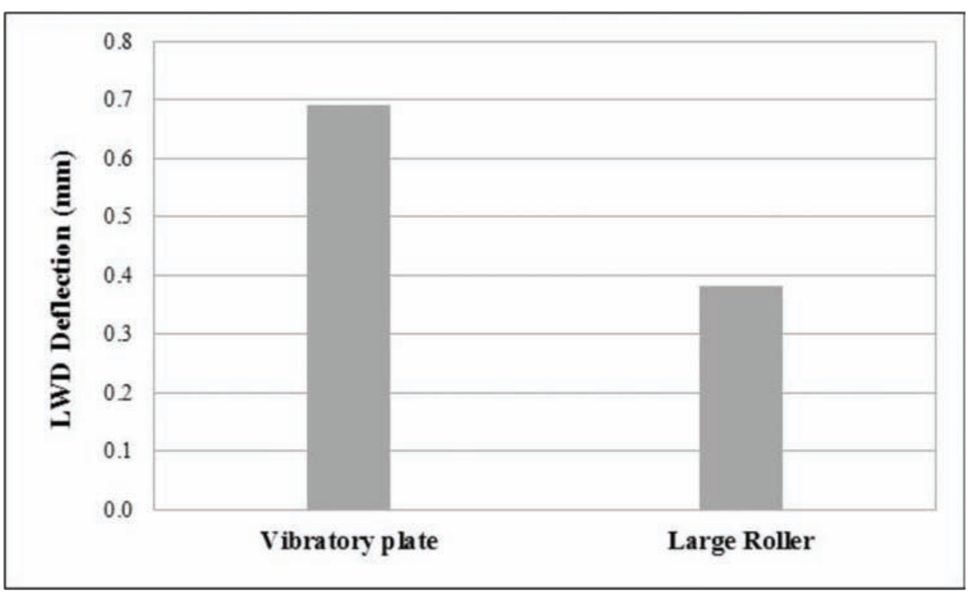


Figure 5.1 Deflections on No. 5 backfills compacted by large and small compactors.

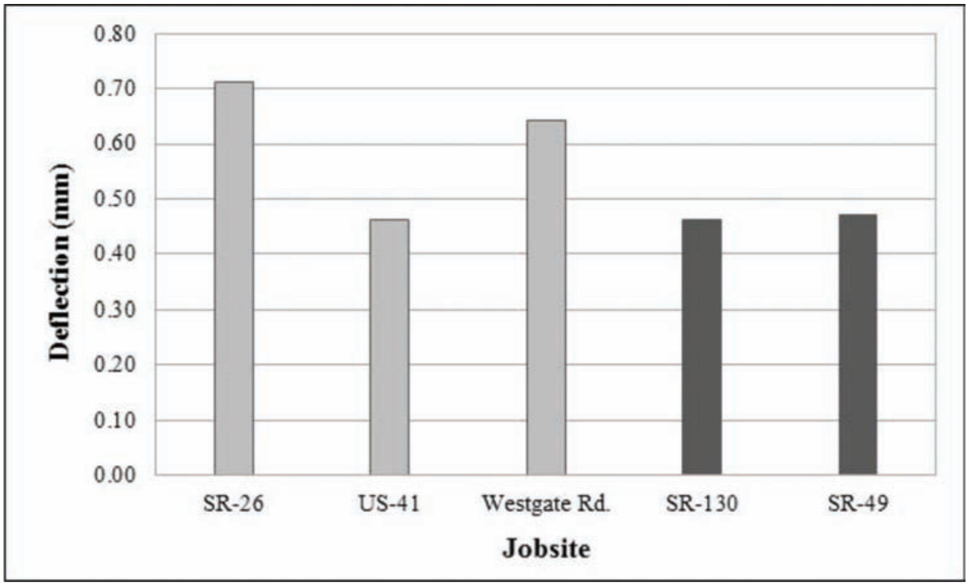


Figure 5.2 Deflections on backfills compacted by large and small rollers.

and stiffness when the materials become wet or saturated, which plays an important role in maintaining long-term pavement structural capacity and performance. This can be simply verified by the laboratory test results presented in Chapter 2.

Lift Thickness. Lift thickness should be decided by taking into account the characteristics of aggregate materials, underlying subgrade condition, achievable compaction effort, and desired compaction. Typical lift thickness is commonly 6 in. compacted. Figure 5.2 shows the LWD deflections on compacted 6 in. No. 53 aggregates at five jobsites. In small areas (light color), the deflections varied between 0.46 mm and 0.71 mm. In large areas (black color), the deflections were less than 0.50 mm. This implies that it may be difficult to achieve the desired results for small area compaction in some situations. Therefore, the lift thickness for small areas may be revisited to 3~4 in. compacted when the subgrade experiences moisture problems.

Coverage and Number of Passes. The compaction of geomaterials should be performed immediately after spreading and continues until the movement of geomaterials stops or the compaction wheel marks disappear. Compaction should start from outside and progress toward the middle of the area, and from the low side to the high side where a slope or grade exists. A minimum overlap of one-third the width of the compactor is recommended between two adjacent compaction passes. Geomaterials become more compacted with increasing the number of compaction passes. After a certain number of passes, additional passes may be useless or detrimental, particularly when the subgrade is wet. Field tests were conducted in this study to verify the effect of the number of compaction passes. Figure 5.3 shows the deflections measured after various passes of compaction in both large and small areas. Evidently, it becomes useless to make further effort after 6 and 8 passes for 6 in. and 12 in. unbound aggregates.

5.1.3 Preparation of Underlying Subgrade

A properly prepared subgrade is essential to achieve the desired degree and uniformity of compaction for the overlying aggregate layer. Silty clay is very common to many areas in Indiana. Silty clay has demonstrated great variations in engineering properties and moisture susceptibility. In some situations such as bridge approach, pavement patching, and lane widening, therefore, silty clay subgrades tend to experience moisture related issues and may not provide sufficient supporting capability for compacting overlying aggregates. Figure 5.4(a) shows the LWD test results on the No. 53 aggregates placed on the subgrades with high moisture contents. The deflections are greater than 0.60 mm for all projects. For the three projects with a deflection greater than 1.00 mm, the subgrade moisture contents were very high. As a result, compaction effort produced the so-called pumping action of subgrade soil as shown in Figure 5.4(b). To address the issues associated with pumping subgrades, two methods, including geosynthetic reinforcement and chemical modification, are commonly utilized.

Placing geosynthetic materials at the subgrade elevation will provide reinforcement of the unbound aggregates, separation of soil and aggregate, and filtration of fines, and improve the long-term performance of the subgrade support condition. However, using geosynthetic reinforcement does not necessarily decrease the deflection under the LWD loading. Presented in Figure 5.5 are the deflections measured on

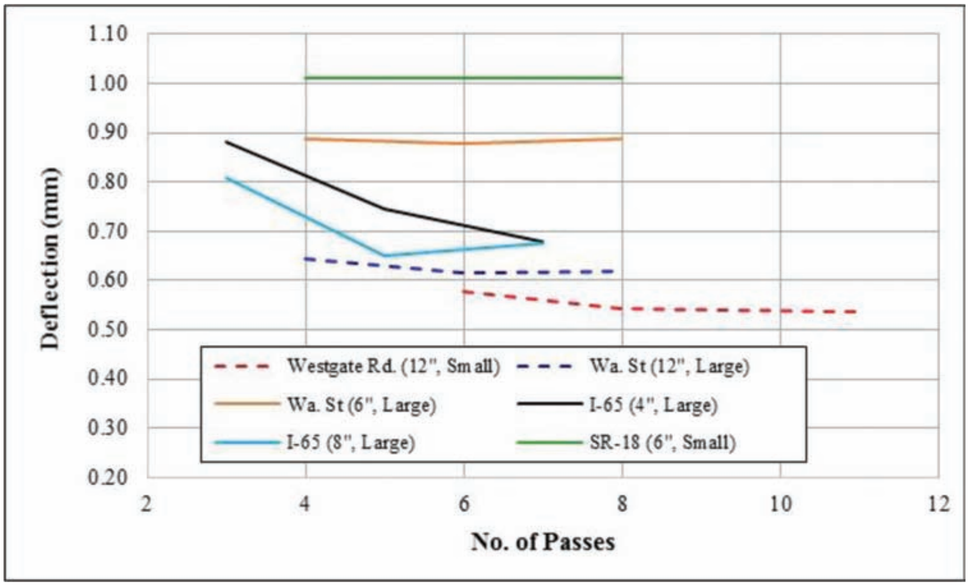


Figure 5.3 Variations of deflections with No. of passes.

geomaterials with and without geosynthetic reinforcement. As shown in Figure 5.5(a), the deflection with geotextiles is greater than that without geotextiles. At the three bridge approach jobsites with geogrids placed between subgrade and 6 in. aggregate layer, the deflections varied between 0.70 mm and 1.50 mm and are greater than 0.50 mm, the commonly expected deflection for 6 in. compacted aggregates. When the chemical modification of subgrade soil is considered the feasible treatment, INDOT commonly uses either 8 in. or 14 in. chemical soil modification. It is advisable to maintain consistency in chemical and thickness to provide uniform support at the entire jobsite.

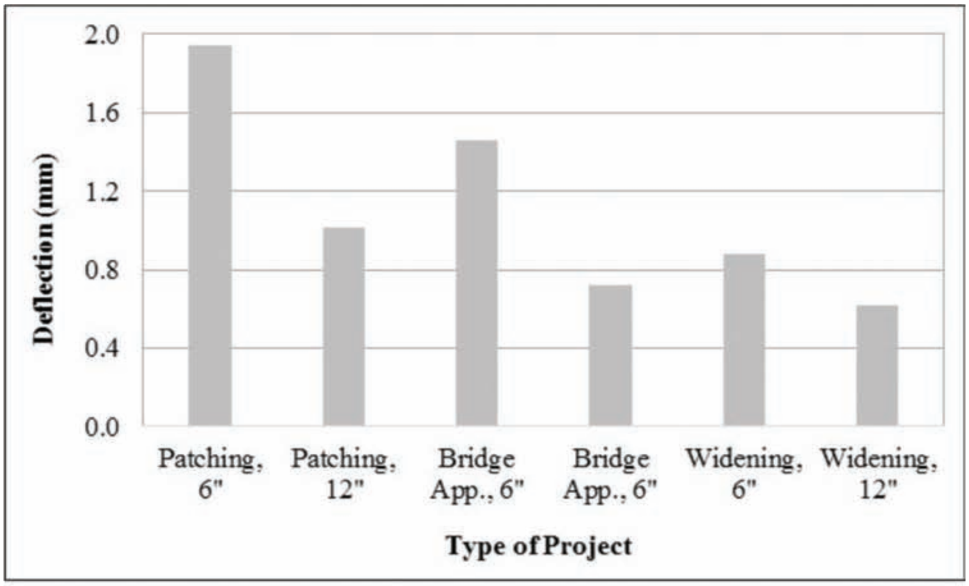
5.2 Field QA Implementation

5.2.1 LWD Testing

Test Time. As demonstrated by the laboratory test results presented in Chapter 2, the deflection of geomaterials varies significantly with the moisture content. In-situ LWD tests were also conducted at three jobsites, including one bridge approach on SR 25, one rehabilitation (SR 18), and one reconstruction on SR 245, at different times after compaction, and the deflection measurements are plotted in Figure 5.6. The deflections at the SR 25 and SR 18 jobsites, respectively, decreased by 33% and 7% due to an approximate 3-hour time difference between the two tests. The deflection decreased by 42% at SR 245 jobsite approximately 36 hours after compaction. Evidently, it is of significance to perform LWD testing in a timely manner. Because the variation of in-situ moisture content depends not only on layer thickness, but also on climatic factors such as air temperature, relative humidity, wind speed, and solar radiation, it is very difficult to precisely determine the time for in-situ LWD testing. It is recommended that elsewhere (Schwartz et al., 2017), the LWD test for compaction quality control should be conducted within two hours after compaction.

Test Position. The structural response of an elastic layer system to loading varies dramatically with the boundary condition. As demonstrated in Chapter 2, the effect of boundary condition on the result of LWD test may be neglected if the horizontal dimension of the layer system is approximately 3 times the diameter of the loading plate or larger. In-situ LWD tests were conducted to compare the deflections at different positions at different jobsites. As shown in Figure 5.7(a) are the deflections measured at three typical positions, including the right (outside) edge, middle, and left (inside) edge in the compaction area. In reality, the left edge is actually the middle of the roadway at the 2-lane jobsites. Notice that small compactor was utilized at the jobsite of 1-lane bridge approach on SR 26; however, large rollers were used for compaction at other jobsites. At the two bridge approach sites on SR 26 (6 in., No. 53 aggregate) and US 50 (12 in., No. 53 aggregate), the deflections at the right (outside) edge, respectively, are 28% and 15% greater than the deflections in the middle of the area. At the 2-lane jobsites on SR 7 (18 in., No. 53 aggregate), US 6 (17 in., No. 53 aggregate), a dSR-130 (6 in., No. 53 aggregate), the deflections at the outside edge are 8% to 31% greater than those in the middle of the lane.

Figure 5.7(b) shows the deflections measured at the outside edge, middle, and inside edge of the compacted shoulder on I-469. The 10-foot shoulder consisted of a 5 in. No. 53 aggregates placed on the structural base. Large roller was utilized for compaction. The LWD tests were conducted after a total of six passes of compaction. It is shown that the deflections at the outside



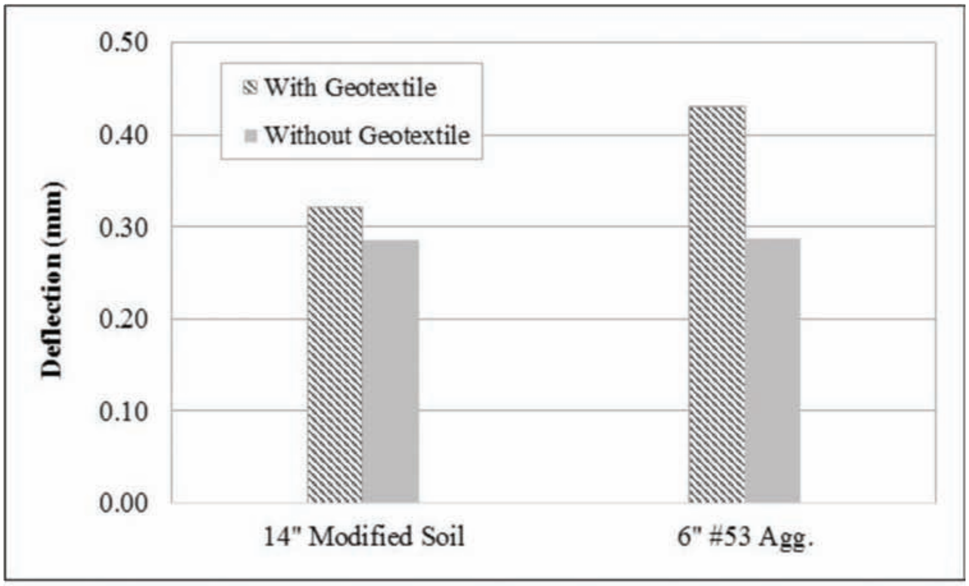
(a) LWD deflection

(b) Subgrade pumping

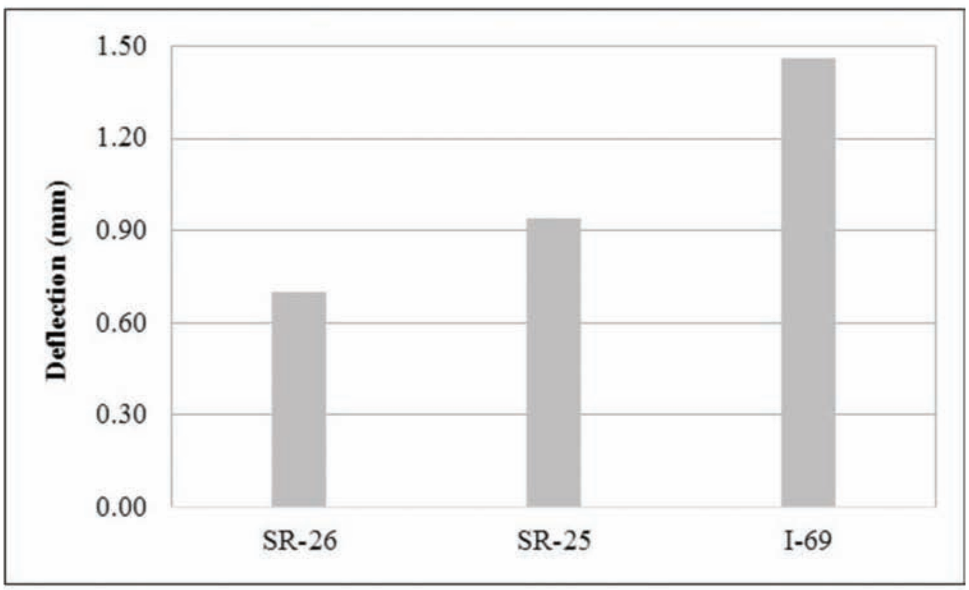
Figure 5.4 Effect of subgrade moisture condition.

edge are generally greater than the deflections at the middle and inside edge of the shoulder. The average deflections are 0.243 mm, 0.233 mm, and 0.327 mm at the inside edge, middle, and outside edge of the shoulder, respectively. Overall, the deflection at the outside edge is 40% and 35% greater than the deflections in the middle and inside edge, respectively. This is due to the effect of free edge boundary condition. Therefore, caution should be exercised to determine the position when performing LWD testing for compaction quality control, in particular in small compaction areas. If uniform compaction is achieved, it is advisable to perform LWD testing 3 feet away from the outside edge or in the middle of the lane or shoulder.

Sample Size. In statistics, the error to estimate a population parameter using a sample statistic decreases with increasing sample size (or test frequency). To evaluate the minimum sample size required for compaction quality control of aggregate materials, extensive LWD testing was conducted in-situ at different jobsites statewide in both 2016 and 2017, Figures 5.8 and 5.9,



(a) Geotextiles



(b) Geogrids

Figure 5.5 Effect of geosynthetic reinforcement on LWD deflection.

respectively, present the variations of sample deflection means and standard deviations with sample size at a total of 24 jobsites, including eight bridge approaches sites, six shoulder and lane widening sites, four pavement patching sites, and six 4R project sites. Unless otherwise stated, the aggregate materials are typically No. 53 limestone aggregates. Other materials include No. 43 limestone, No. 43 slag, and No. 53 limestone aggregates. The layer thickness of aggregates varied between 6 in. and 18 in. in-situ. Large rollers were utilized for compaction at the shoulder, lane widening, and 4R project jobsites, and small compactors were used at the patching jobsites. At the bridge approach jobsites, either small or roller compactors were utilized, depending on if the jobsites were accessible to large compactors.

Figure 5.8 shows the greatest variations arise when the sample size is 3 or less. Afterwards, the sample means become stable, in particular when the sample size is 5 or more. Abnormal patterns occurred at the three jobsites, including the bike lane in Figure 5.8(b) and 10 in. No. 53 limestone and 5 in. No. 43 slag aggregates in Figure 5.8(d). The bike lane was adjacent to the bottom of a hill where different types of organic soils were observed. At the jobsite of 10 in. No. 53

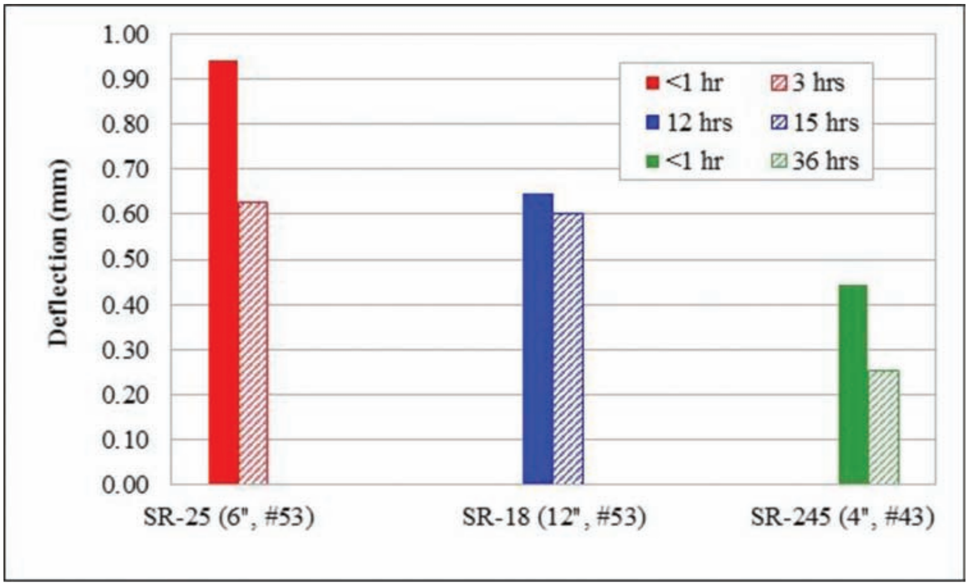


Figure 5.6 In-situ deflections measured at different times after compaction.

aggregates, 4 in. of additional aggregates were added randomly to adjust the grade. The 5 in. No. 43 slag materials are coarse-graded slag aggregates and are basically unable to provide any structural capacity. Similar trends can be observed in the variation of standard deviation with sample size in Figure 5.9. The above observations imply that in small areas, a minimum of 5 LWD tests are required to provide reliable compaction quality control. A minimum of 8 to 10 LWD tests are necessary for large area compaction. The minimum sample size should increase as the compaction area increases. Notice that a test section for compaction of aggregates should be an area of 100 ft. by 20 ft. with at least 10 LWD tests (INDOT, 2015).

5.2.2 Compaction Variation Assessment

To ensure compaction quality, the variation of compaction should also be properly considered. The factors affecting the variation of compaction are the same as those affecting the degree of compaction described earlier. Therefore, estimates on the variation of compaction may become very complex and subject to significant variation. Plotted in Figure 5.10 are the coefficients of variation (COVs) for the in-situ deflection measurements from different construction projects. Unless otherwise stated, the aggregate materials are No. 53 aggregates. Significant variations occurred regardless of layer thickness, material type, compactor size, and project type. The COVs varied from 11% to 53% for bridge approach projects, from 23% to 54% for shoulder and lane widening projects, and from 26% to 35% for patching projects. For large area compaction, the COVs varied from 8% to 38%. Evidently, Compaction has experienced greater variation in small areas than in large areas.

To further assess the nature of the variation of compaction in small areas, the cumulative frequencies were computed with respect to COVs under two scenarios: all projects and small areas, as shown in Figure 5.11. Two observations can be made through careful inspection of the two curves. It is demonstrated that the two curves follow a similar trend. There are two inflection points, i.e., COV = 20% and COV = 35%, on each curve. Approximately, the projects with a COV of 20% or less account for 25% of the total projects, and the projects with a COV of 35% or higher account for 10% of the total projects under both scenarios. The majority of the projects have a COV of 20% to 35%. The above can be extended to imply that for small area compaction, a COV of 20% or less indicates "Low" variation, a COV of 20% to 35% indicates "Normal" variation, and a COV greater than 35% indicates "Poor" variation.

5.3 LWD Test Devices and Errors

5.3.1 LWD Test Devices

Several types of LWD test devices are commercially available from manufacturers such as Zorn Instruments, Dynatest, Humboldt Mfg. Co., and Olson Instruments. Studies have been published to make comparison of different LWD test devices that are widely used by the corresponding states and the detailed results can be found elsewhere (Nazzal, 2014; Siekmeier et al., 2009; Vennapusa, 2008). All LWD devices measure the deflection due to a force pulse generated by a falling mass dropped onto a damping system that transmits the load pulse to a plate resting on the material under test. Summarized in Table 5.1 are the main characteristics of LWD field test devices that may make one

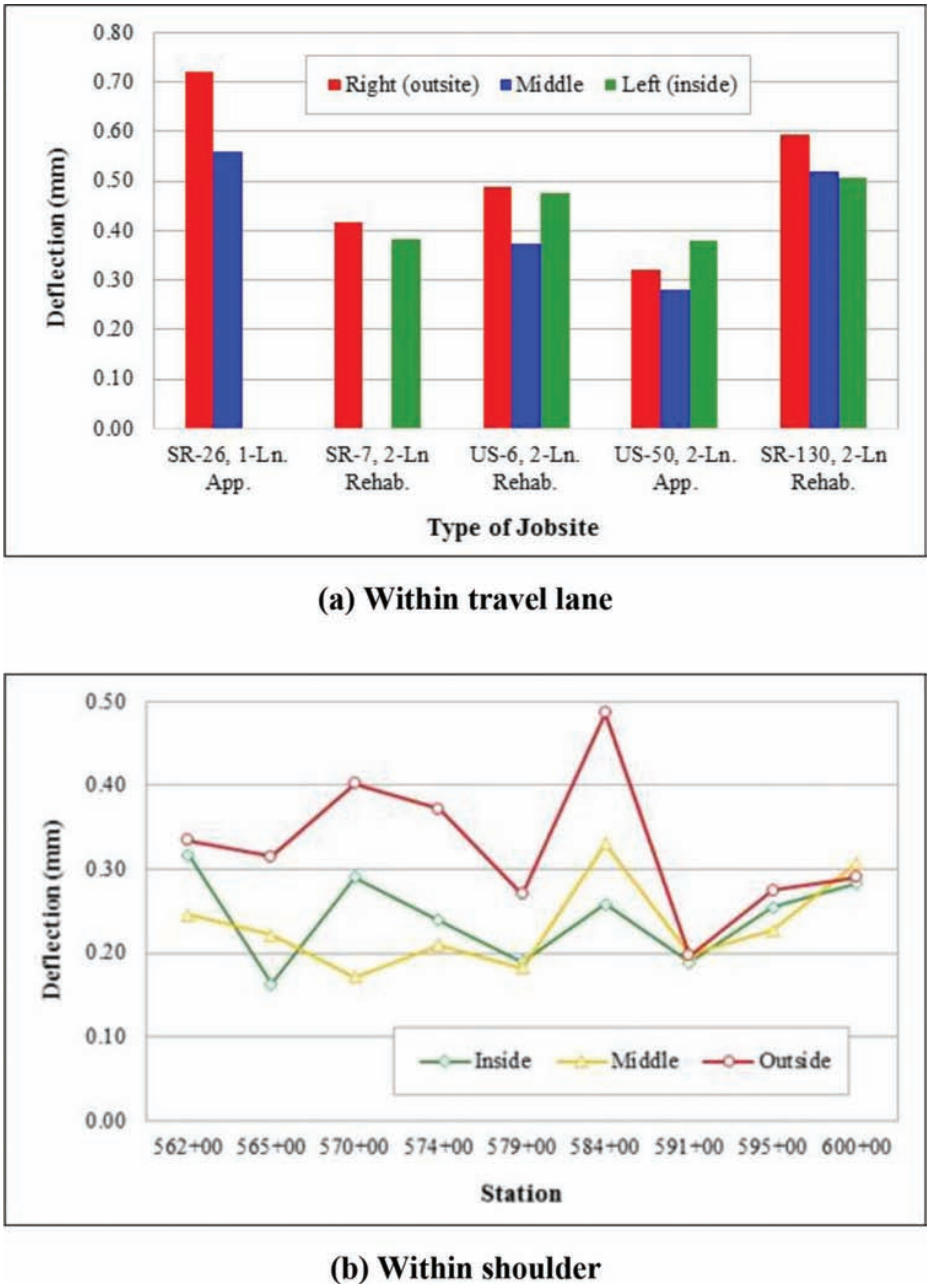


Figure 5.7 In-situ deflections measured at different positions within travel lane.

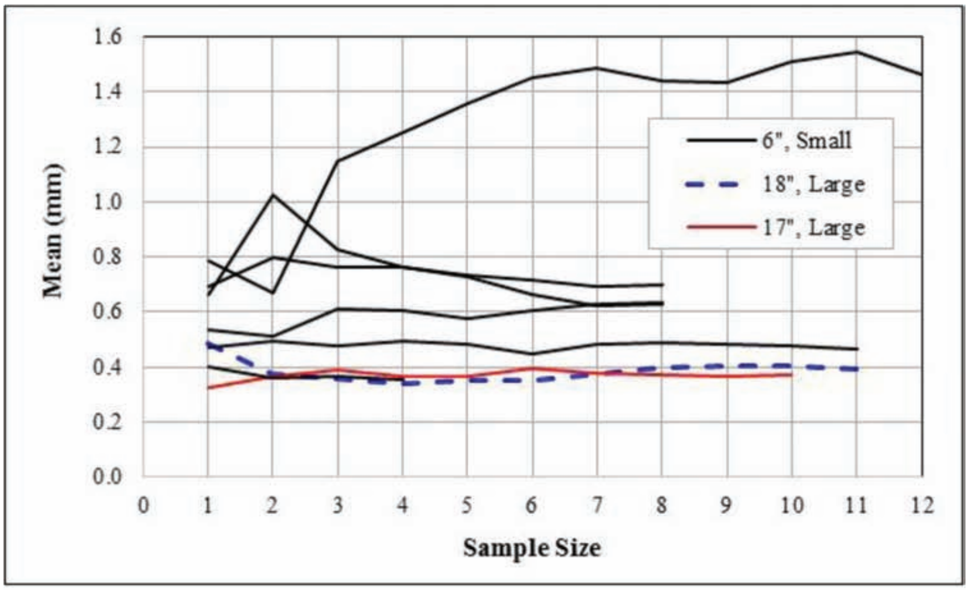
device different from the others. Three major differences can be identified between these LWD devices as follows:

- *Deflection Measuring*. The Zorn, Humboldt, and Olson LWDs measure the deflection at the top of the load plate using an accelerometer, and the Dynatest LWD measures the surface deflection of the ground using a geophone. When the contact between the load plate and the compacted materials are perfectly uniform, the deflection of the load plate should be the same as the deflection of the surface of the compacted materials. However, it is summarized that the LWD devices with accelerometers that measure deflections on the plate are expected to measure larger deflections (Nazzal, 2014), and therefore, lower moduli. Plotted in Figure 5.12 are the moduli measured with Zorn ZFG 2000 and Dynatest LWD 3031 in both the laboratory and test pit. The moduli produced by LWD 3031 are 20%~35% greater than those by ZFG 2000.
- *Load Measuring*. The Dynatest and Olson LWD devices have an in-line load cell, but the Zorn and Humboldt LWD devices don't have one. However, the impact force is assumed to be constant for both Zorn and Humboldt LWD devices. An in-line load cell allows simultaneous measuring of the actual, site-specific impact force tran-

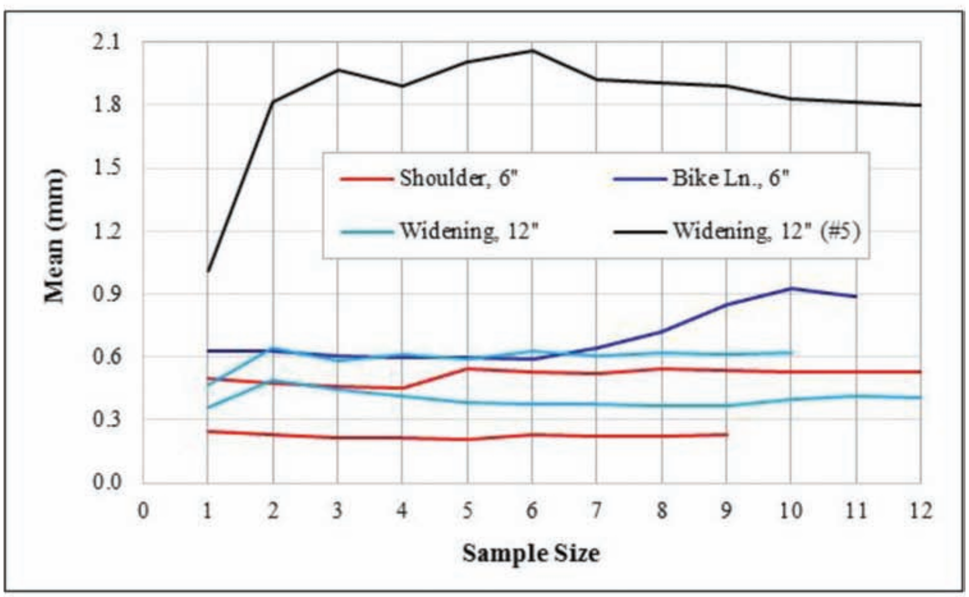
smitted to the load plate. Therefore, LWD devices with an in-line load cell are capable of providing insight into the time history and peak value of the impact force and more accurate estimates of in-situ moduli.

- *Buffering System.* Dynatest LWD uses a rubber buffer and the other three LWD devices use either steel spring or disk spring to control the shape and length of the transient load pulse. A steel spring system commonly has a linear relationship between force and deflection, but a rubber buffer system may or may not have a linear relationship between force and deflection. In addition, the stiffness of rubber varies with the rate of deflection. As shown in Table 5.1, the Dynatest LWD has longer load pulse time than the other LWDs with a steel spring assembly. However, it was reported that mixed results have been obtained on the effect of changing the shape and time of load pulse (Siekmeier et al., 2009).

As a result of the above differences, different LWD devices may yield different test results. In general, the LWD tests should be performed in accordance with ASTM E2835-11 (2015) when using an LWD device from Zorn, Humboldt, or Olson LWDs, and in accordance with ASTM E2583-07 (2015) when using

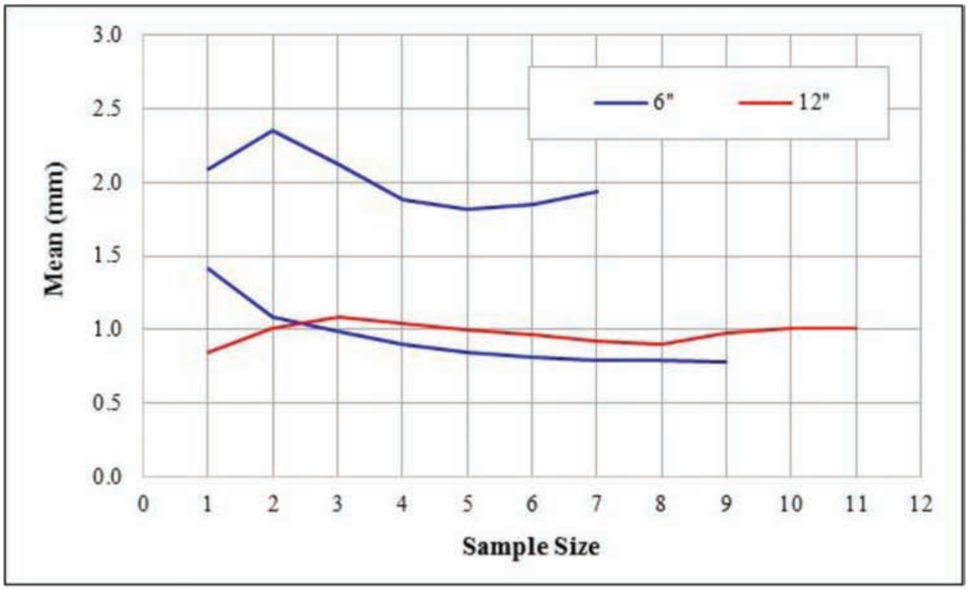


(a) Bridge approach

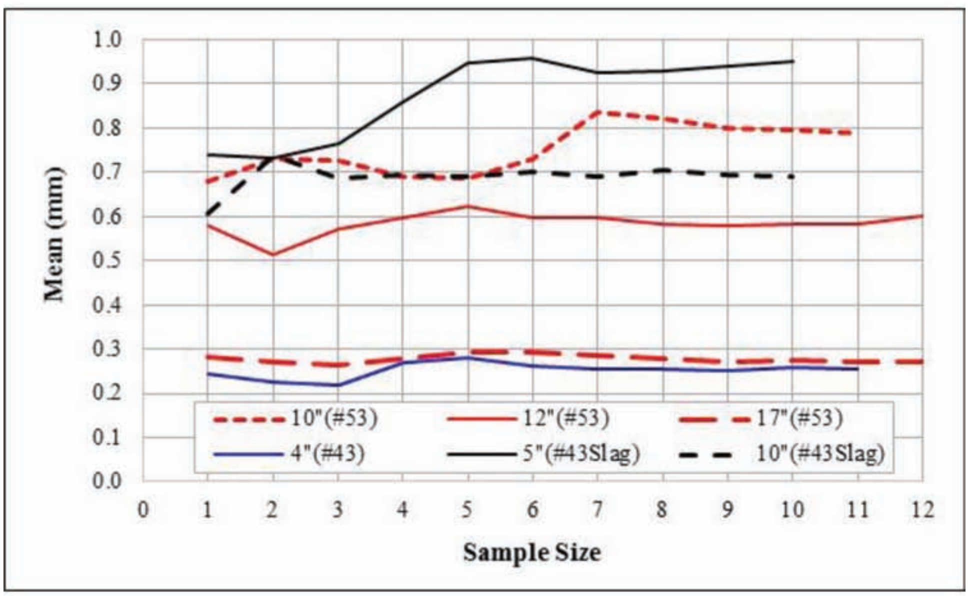


(b) Shoulder or lane widening (large roller)

Figure 5.8 Variation of deflection sample mean with sample size. (Figure continued next page)



(c) Patching (small roller)



(d) 3R projects (large roller)

Figure 5.8 (Continued)

a Dynatest LWD. In order to ensure compaction quality consistency statewide and over time, INDOT currently supports only the LWD test devices manufactured by Zorn Instruments, such as ZFG 2000 (2005) and ZFG 3.0 (2011), for compaction quality assurance.

5.3.2 Estimation of Test Errors

The LWD testing on subgrade is governed by the Boussinesq's solutions for an elastic, homogeneous half-space. When the load is applied over a single circular area with a loading plate, the ground surface deflection at the center of the circular loaded area is:

$$E = \frac{2pa\left(1-\mu^2\right)}{w} \times f \tag{5.1}$$

$$E = \frac{2F\left(1-\mu^2\right)}{w\pi a} \times f \tag{5.2}$$

where, E = elastic modulus of the subgrade; w = deflection at the center of loading plate; p = average pressure under the loading plate; a = radius of the loading plate, μ = Poisson ratio of the subgrade; F = force applied onto the plate; and f = pressure distribution factor, i.e., 1.0 under a flexible plate, and $\pi/4$ under a rigid plate.

In the case of Zorn LWD testing, the applied force, F, is defined as follows:

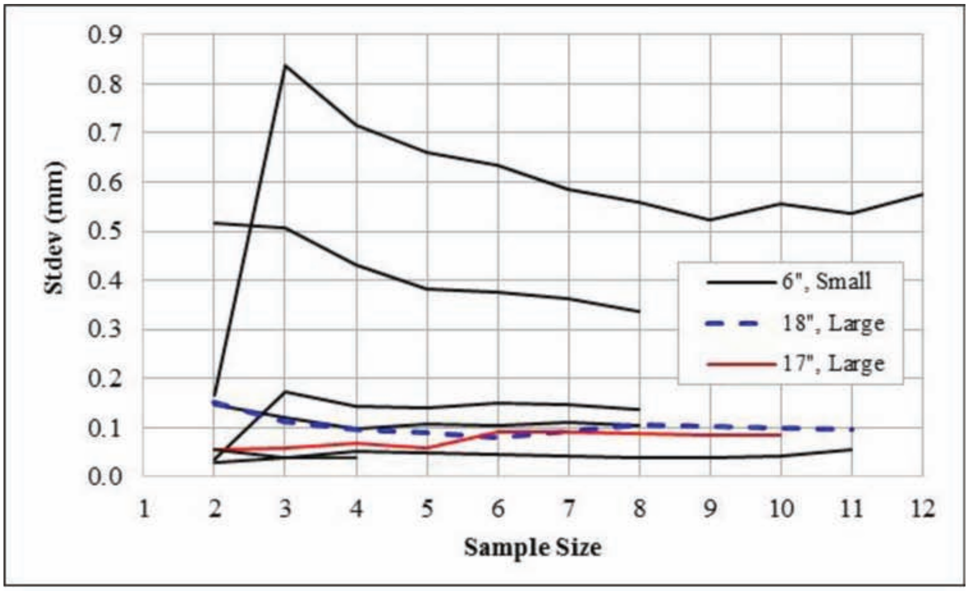
$$F = \sqrt{2 \times m \times h \times g \times k} \tag{5.3}$$

where, F = as defined earlier, N; m = mass of falling weight, kg; h = drop height, m; g = acceleration of gravity, i.e., 9.81 m/s^2; and k = spring constant, N/m.

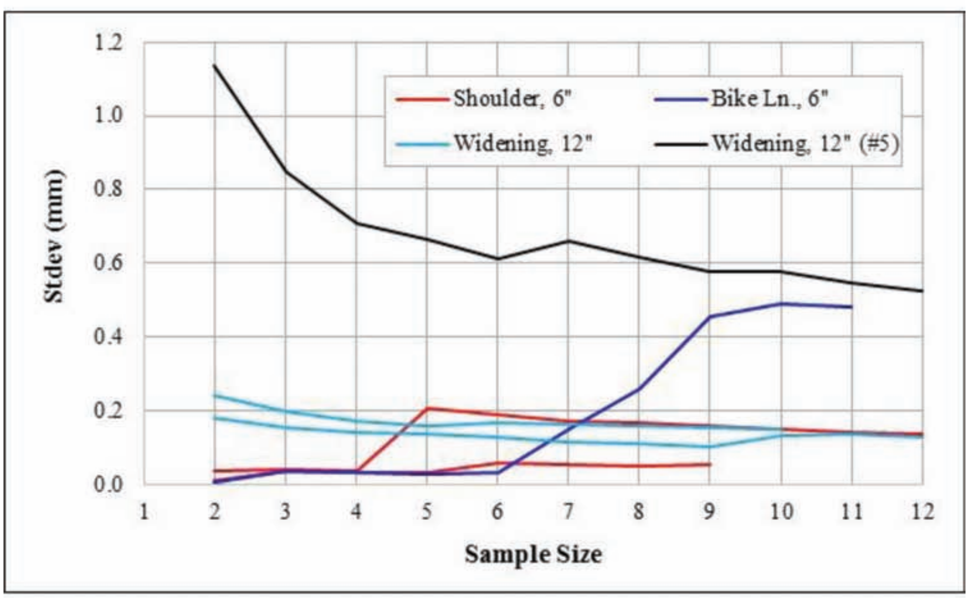
Substituting m = 10 kg, h = 0.724 m, and g = 9.81 m/s^2 into Eq. 5.2 yields $F = \sqrt{142k}$. Therefore, Eq. 5.2 can be rewritten as follows, by substituting $\sqrt{142k}$, for F, 0.50 for μ, and $\pi/4$ for f,

$$E = 38\frac{\sqrt{k}}{w} \tag{5.4}$$

where, all variables are as defined earlier.

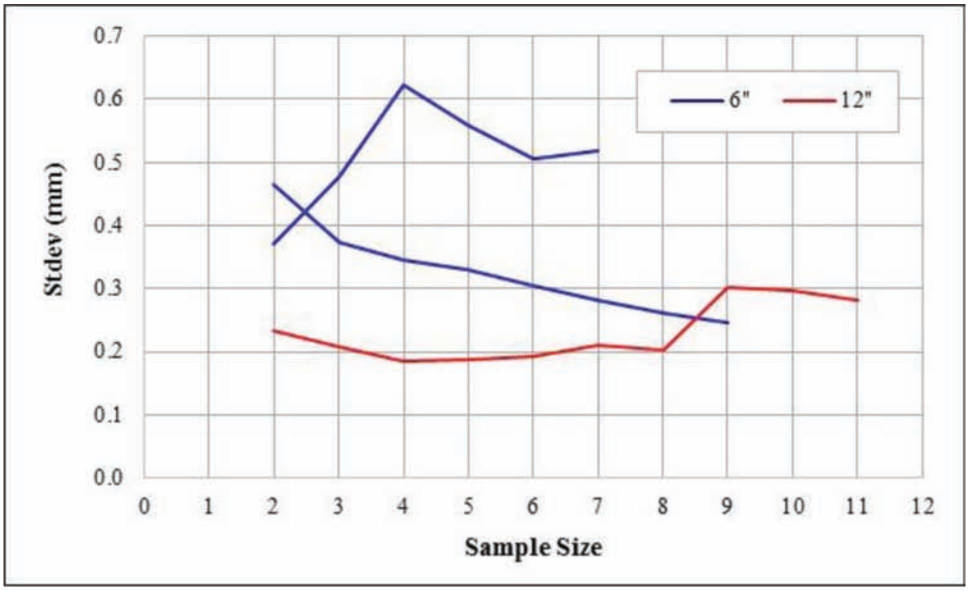


(a) Bridge approach

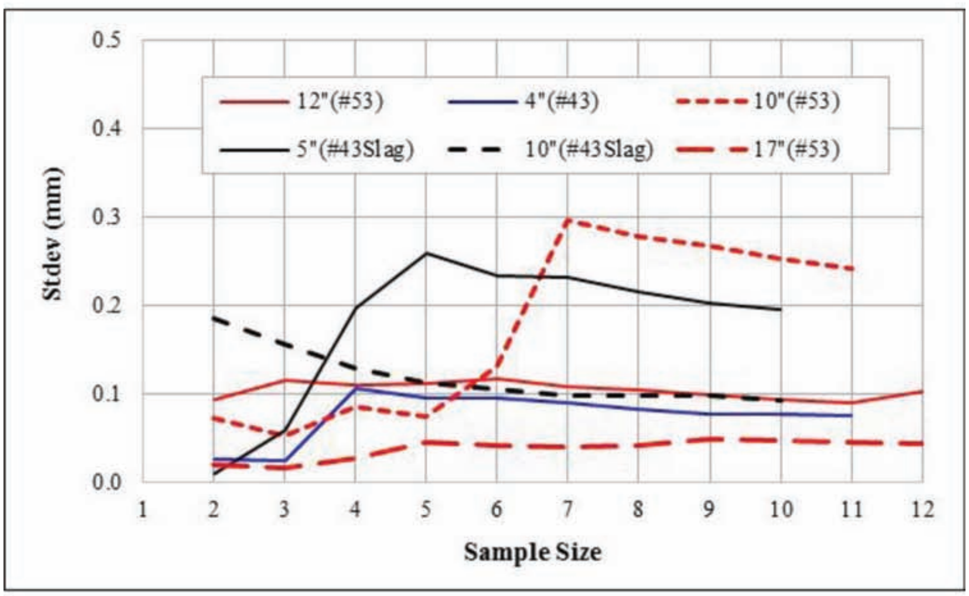


(b) Shoulder or lane widening (large roller)

Figure 5.9 Variation of deflection sample standard deviation with sample size. (Figure continued next page)



(c) Patching (small roller)



(d) 3R projects (large roller)

Figure 5.9 (Continued)

Applying Taylor series expansion to Eq. (5.4), and then, using the first-order approximation (Li, 1990), the standard deviation of E is given as follows:

$$\sigma_E^2 = \left(\frac{\partial E}{\partial k}\right)^2 \sigma_k^2 + \left(\frac{\partial E}{\partial w}\right)^2 \sigma_w^2 \tag{5.5}$$

where, σ_*= standard deviation of variable $*$; and the first partial derivatives are defined below:

$$\frac{\partial E}{\partial k} = \frac{16}{w\sqrt{k}} = \frac{E}{2k} \text{ ; and } \frac{\partial E}{\partial w} = \frac{-38\sqrt{k}}{w^2} = \frac{-E}{w}$$

where, all variables are as defined in Eq. (5.4).

The coefficient of variation for E can be estimated as follows:

$$COV_E = \left(\frac{COV_k^2}{4} + COV_w^2\right)^{1/2} \tag{5.6}$$

where, COV_*= coefficient of variation for variable $*$.

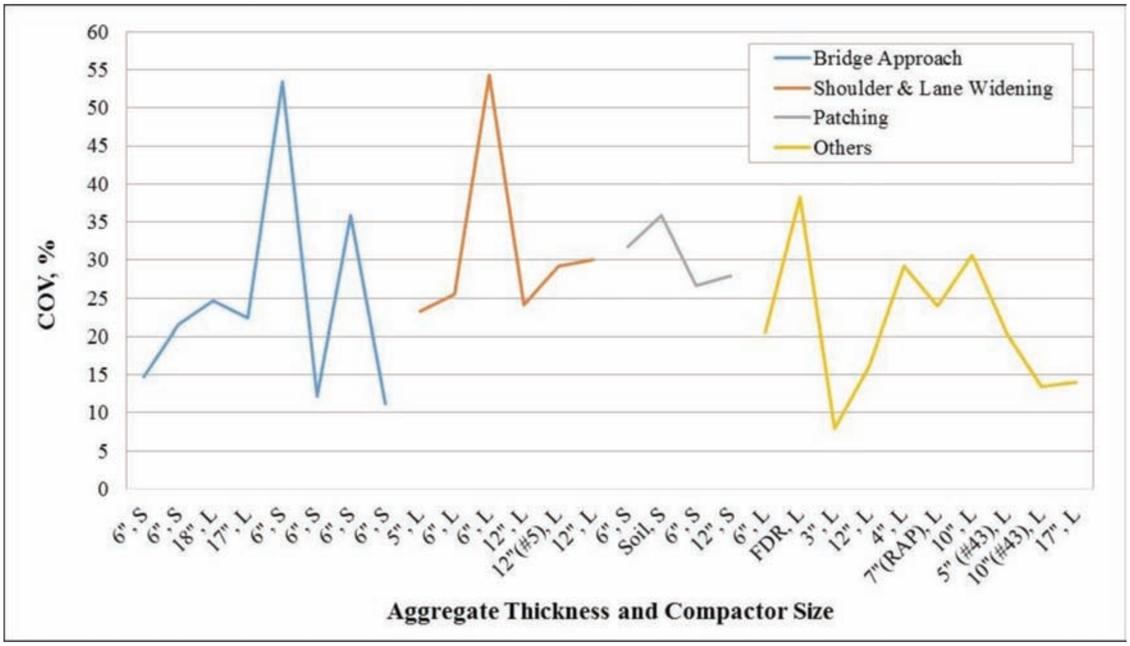


Figure 5.10 Coefficients of variation for deflection measurements (S = small compactor; L = large roller).

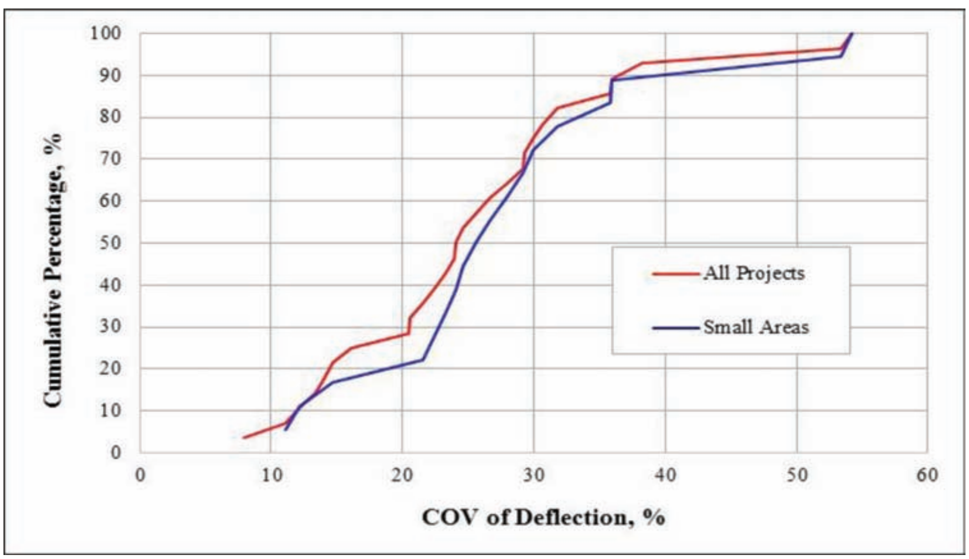


Figure 5.11 Cumulative frequency distribution with respect to COV of deflection.

TABLE 5.1
Main Characteristics of LWD Devices by Different Manufacturers

	Load Measuring			Damping	Deflection Measuring		
LWD Device*	**Max. Force**	**Load Cell**	**Load Pulse**	**System**	**Transducer**	**Location**	**Range**
Zorn 2000	7.07 kN	No	18±2 ms	Steel Spring	Accelerometer	Plate	0.2–30 mm
Dynatest 3031	15 kN	Yes	15–30 ms	Rubber	Geophone	Ground	0–2.2 mm
Humboldt HMP LFG	7.07 kN	No	17±1.5 ms	Disk Spring	Accelerometer	Plate	0.1–2.0 mm
Olson LWD-1	6.9 kN	Yes	15–20 ms	Steel Spring	Geophone	Plate	0.1–2.5 mm

*Differences may exist between the investigated model and its predecessor or successor.

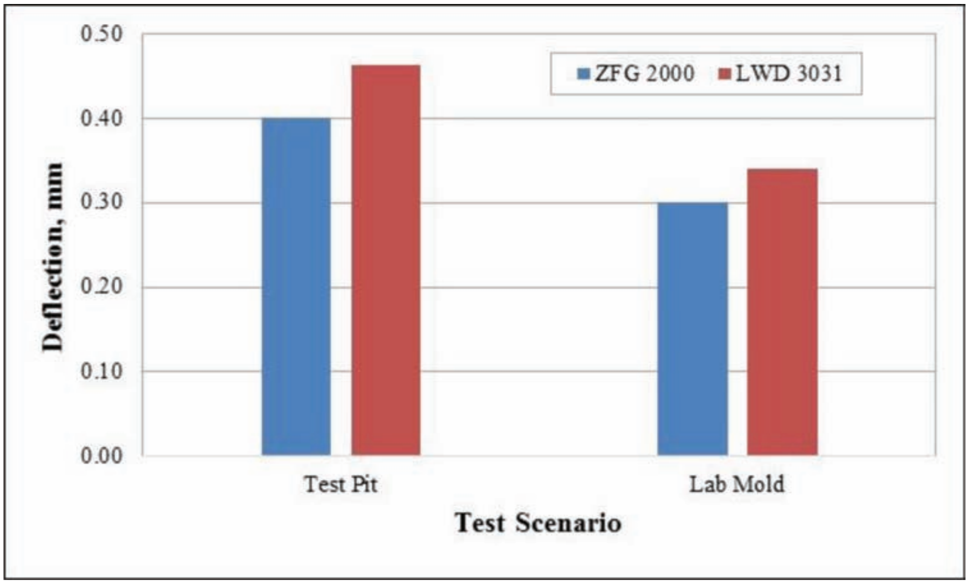


Figure 5.12 Comparison between Zorn and Dynatest LWD test results.

As indicated by Eq. 5.6, the accuracy of the back-calculated modulus, E, is govern by the accuracies of the spring constant, k, and the measured deflection, w. It is stated that the deviation of the impact force from the specified value, i.e., 7.07 kN, shall be no greater than 1% (ASTM E2835-11, 2015; FGSV, 2003). Therefore, the maximum COV_k for the spring constant is no greater than 2% that can be derived from Eq. 5.3 by assuming that both m and h are constant. As shown earlier (see Figure 5.10), in-situ LWD deflections are subject to significant variation due to the combined effect of different errors. Notice that the procedures addressing the variation of in-situ compaction (or material properties) are discussed earlier. Basically, LWD testing is a variation to falling weight deflectometer (FWD) testing. The errors affecting the accuracy of FWD deflections, such as seating errors, random errors, and systematic errors, can also affect the accuracy of LWD deflections. The procedures addressing the errors involved in FWD testing can be found elsewhere (Irwin, Orr, & Atkins, 2011). Presented hereafter is a discussion of the procedures for calibration of LWD devices to reduce systematic errors.

5.4 Calibration and Verification of LWD Devices

5.4.1 Procedures Recommended by Manufacturers

To ensure the validity and reliability of in-situ LWD test using Zorn LWD devices, ASTM E2835-11 defines the test approach, necessary precision of the result, instrumentation system, and calibration (Siekmeier et al., 2009). The detailed procedures and requirements for calibration can also be found in TP BF-StB Part B 8.3, the German technical code for soil and rock in road construction (Li, 1990). Summarized in Table 5.2 are the key aspects of the calibration procedures for both the force-generating device and the deflection sensor. In addition, it is also recommended that during the calibration interval (FGSV, 2003), the users should inspect the accuracy of the deflection measuring device, once every three months when the devices are regularly used.

It is stated that the calibration must be repeated at least once per year (FGSV, 2003). On the one hand, the calibration of LWD devices is costly and time-consuming. On the other hand, the use of LWD devices that are out of calibration can be even more costly due to inaccurate estimates of compaction or in-situ properties. Clearly, this leads LWD owners to make a decision on calibration in a dilemma situation. To avoid the possible consequences due to inaccurate test results, LWD owners should take into consideration the recommendations by manufacturers. The question is if the recommended calibration interval can be adjusted to ensure both proper calibration and cost-effectiveness. Many factors affect calibration intervals that may vary from manufacturer to manufacturer and from device to device. However, a large amount of LWD test data has been generated from compaction QA activities over the years. Risk analysis is needed to assess the possible positive and negative effects of the calibration interval.

5.4.2 Verification Procedures by INDOT Districts

In reality, annual verification policies have been widely implemented by INDOT districts. In general, annual verification testing is conducted to establish the repeatability of deflection measurements under well-defined conditions. Calibration will be considered when the verification criteria are not met.

TABLE 5.2
Key Aspects of Calibration Procedures Recommended by Manufacturers

<table>
<tr><th>Force-Generating Device</th><th>Deflection Sensor</th></tr>
<tr><td>• Calibration frequency
Once per year by a test institute
• Calibration equipment
◦ A load cell rated from 20 to 50 kN
◦ Test amplifier, and
◦ Devices measuring and storing the entire force course
• Calibrate the force-generating device prior to calibrating deflection sensor
• Calibration procedures
◦ Ensure the load cell is uniformly supported by a rigid base of 32 in long, 32 in wide, and 20 in thick
◦ Use an amplifier with a low-pass filter of at least the fourth order (critical frequency 200 Hz at 3 dB damping)
◦ Perform no less than ten drops from the same height, and
◦ Record the individual peak load, F_i, that meets the following requirements:
✓ Mean = 7.07 kN ± 1%
✓ Individual F_i = Mean ± 2%</td><td>• Calibration frequency
Once per year by a test institute
• Calibration equipment
◦ Inductive travel sensor
◦ Measuring bridge
◦ Test amplifier
◦ Devices measuring and storing the entire settling sequence, and
◦ Three different rubber pads that, respectively, allow the loading plate to settle 0.1 mm~0.7 mm, 0.7 mm~ 1.3 mm, and 1.3 mm~2.0 mm, under an impact force of 7.07 kN
• Calibration procedures
◦ Drop the falling mass 10 times for each of the above deflection ranges
◦ Record the peak force for each drop
◦ Check conformity of deflections measured by the LWD sensor and the control travel sensor in terms of mean and individual measurement:
✓ $|Mean_{LWD} - Mean_{Control}| \leq 0.02$ mm
✓ $|Max_{LWD} - Min_{LWD}| \leq 0.04$ mm</td></tr>
</table>

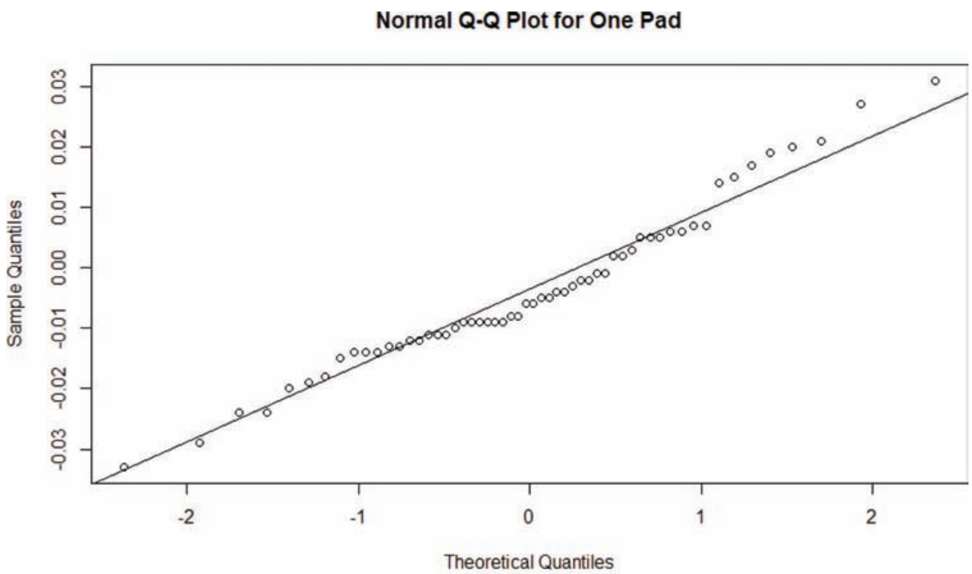
The annual verification testing is typically conducted for each LWD device using the following procedure:

- Inspect the falling height.
- Perform testing with four pad combinations, including 1 pad (Pad 1), 2 pads (Pad 2 on Pad 1), 3 pads (Pad 3 on Pad 2 on Pad 1), and 4 pads (Pad 4 on Pad 3 on Pad 2 on Pad 1), on rigid foundation.
- Precondition with 12 drops on Pad 1.
- Perform 12 drops, i.e., 4 sets of 3 drops, for each pad combination, and record the deflection under each drop.
- Check the repeatability of deflection measurement under each pad combination using the following criteria:
  - $S_{max} - S_{min} \leq 0.004$ mm
  - $S_{mean} - S_{min} \leq 0.02$ mm
  - $S_{max} - S_{mean} \leq 0.02$ mm

The INDOT Crawfordsville District conducted verification tests on a total of 56 Zorn LWD devices in both 2014 and 2015 (Campanell, 2014–2015). It was found that two years of in-house verification testing demonstrate that the LWD devices are basically precise under repeatability conditions. It was also recognized that the verification data does not provide sufficient information to further draw any conclusions about the accuracy of the deflection measurements. This study reexamined the verification data mentioned above with a more rigorous approach of statistical test on the verification data as follows:

Normality Check. In general, Shapiro-Wilk normality test has been widely utilized to check the normality of sample data. However, the p-value by the test is highly dependent on the sample size, which means that for a large sample, the small deviation from the normal distribution can make the test p-value significant and may lead to misleading conclusions. Instead, the visual inspection method, i.e., Q-Q plot, was used as a supplement approach to detect sample normality of the verification data in both 2014 and 2015. As shown in Figure 5.13 are the Q-Q plots generated for the difference of 2014 and 2015 verification data under each pad combination. Based on these plots, it is reasonable to assume that normality is followed, and the paired sample t-test can be used.

Paired Samples t-test. The paired samples t-test was performed by assuming (a) Null hypothesis: the difference between 2014 and 2015 LWD average in means is equal to zero; and (b) Alternative Hypothesis: the difference between 2014 and 2015 LWD Average in means is not equal to zero. Presented in Table 5.3 is the summary of the paired samples t-test results. The p-values show that for the combinations of one pad, two pad, and bearing pad, the 2014 LWD average is significantly different from the 2015 LWD average, while for the substrate of three pads, the difference of LWD average between 2014 and 2015 is not significant. The above implies that annual verification is necessary to ensure repeatability.



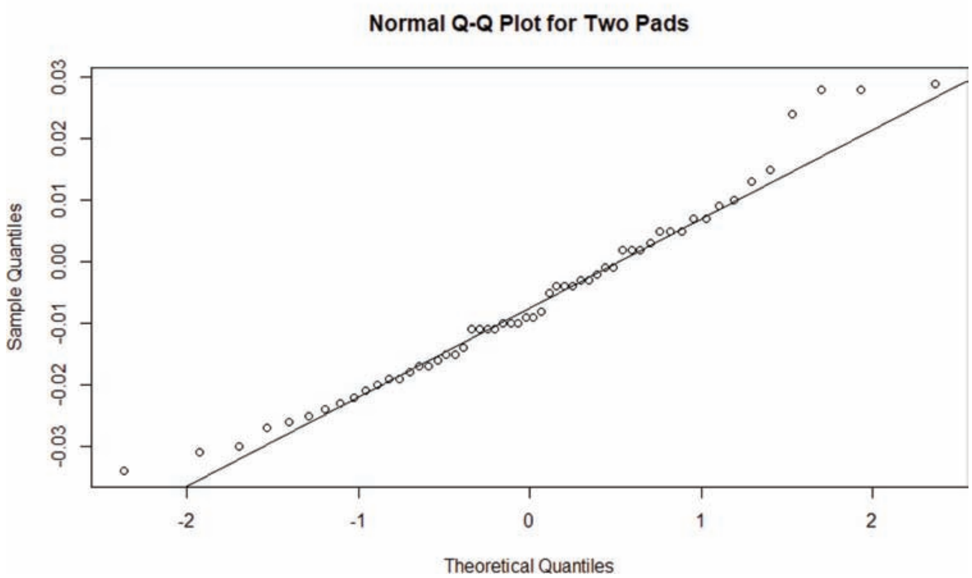
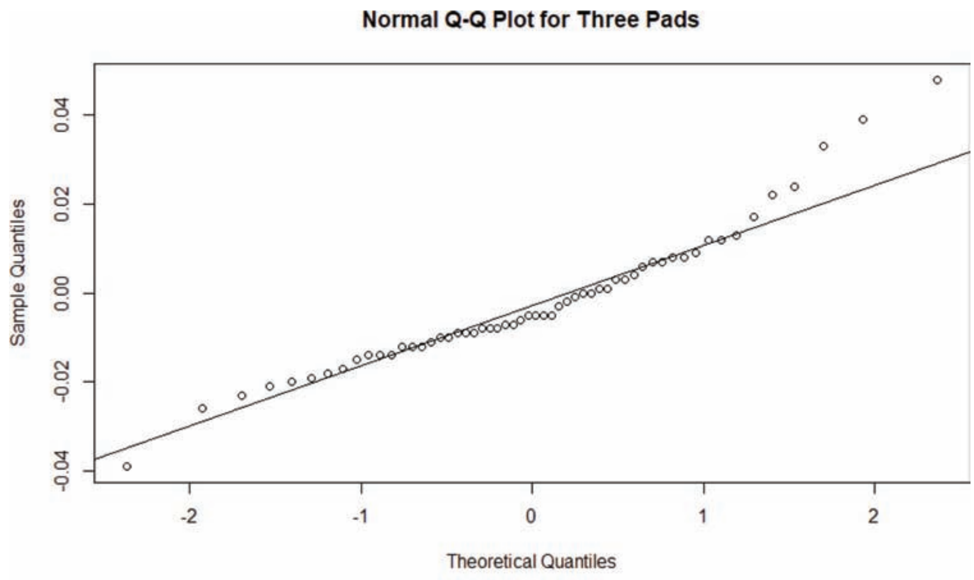


Figure 5.13 Q-Q plots for 2014 and 2015 LWD verification data. (Figure continued next page)



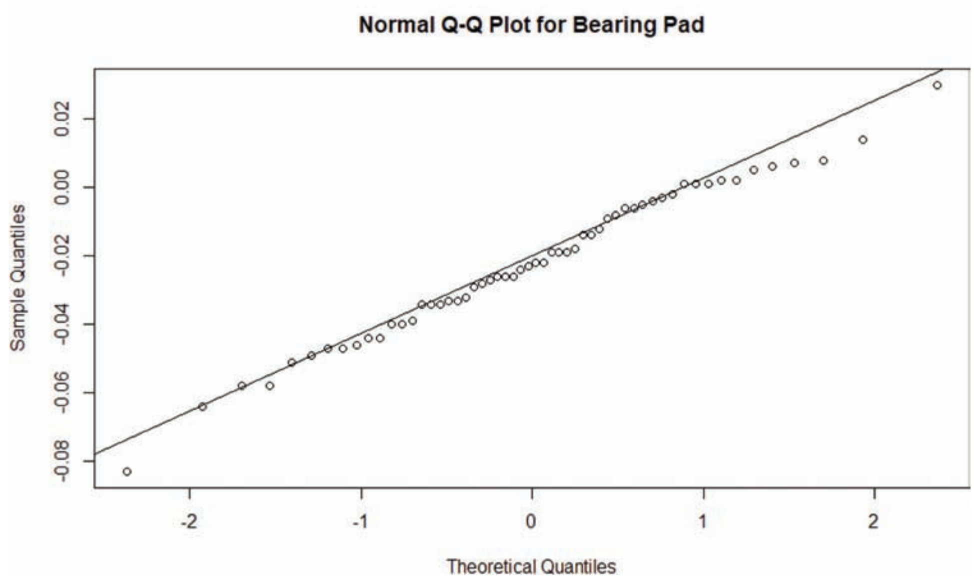


Figure 5.13 (Continued)

TABLE 5.3
Summary of Paired Samples t-test on Verification Data

Pad Combination	Mean of Difference	t-statistic	Degree of Freedom	p-value
One Pad	−0.0038	−2.1313	55	0.0376
Two Pads	−0.0065	−3.2479	55	0.0020
Three Pads	−0.0023	−1.0792	55	0.2852
Bearing Pad	−0.0222	−7.5269	55	<0.0001

6. FINDINGS AND RECOMMENDATIONS

6.1 Major Findings

The Proctor test for aggregates is performed in accordance with the AASHTO Designation: T 99 by INDOT (2017b). Corrections may be necessary if the oversize material is above a certain percentage. However, the laboratory test results indicate that the differences between the original and corrected maximum densities and between the original and corrected optimum moisture contents for both materials were not significant for practical applications.

When performing LWD testing on aggregates in a Proctor mold, the interface condition between the aggregate material and the inner wall of the mold will affect the deflection measurements, depending on the aggregate size and moisture content. The deflections increased by about 11.8% to 18.8% for No. 43 aggregates and by 1.9% to 6.7% for No. 53 aggregates when the inner wall of the mold was lubricated.

Different from the well-known bell shaped moisture-density relationship, the moisture-deflection relationships for aggregates did not show an optimum moisture content at which the deflection would be at a turning point. The results of the laboratory experiments imply that a minimum deflection may not exist in terms of different moisture contents. When compacted at the optimum moisture content, the modulus of aggregates increased considerably as the moisture content decreased. When compacted at a random moisture content, the modulus of No. 53 aggregates remained relatively unchanged, but the modulus of No. 43 aggregates increased noticeably as the moisture content decreased. Coarser aggregates are more sensitive to the moisture content than finer aggregates with respect to deflection or modulus.

The results of LWD tests in the test pits indicate that No. 53 aggregates can contribute to the structural capacity, but No. 43 aggregates can only contribute to the structural capacity when its thickness is 8 in. or more. The deflection decreased as the thickness of aggregate layer increased. As the layer thickness increased to a certain level, the deflection became stable. The modulus back calculated from the stable deflection value may represent the modulus of elastic half space made up of the aggregates. It is interesting to note that the elastic half space moduli of the two materials (51.7 MPa for No. 43 and 38.1 MPa for No. 53) are rather close to the moduli by the Proctor test (46.7 MPa for No. 43 and 35.3 MPa for No. 53).

The differences between the specified target deflections and the measured deflections are statistically significant for both 2015 and 2016 historical datasets. The current target deflections may be too large for the purpose of compaction QA. The measured deflections in 2015 were not statistically significantly different from the measured deflections in 2016. The quality of compaction in roadway construction remained consistent for 2015 and 2016.

It is necessary to adjust the target deflection or modulus by taking into consideration the field and construction conditions, particularly compaction effort, subgrade condition, lift thickness, and use of geotextile. However, caution should be exercised when selecting either deflection or modulus as the target parameter for field compaction QA using LWD due to the potential effects of many factors.

It may become very challenging to compact the geomaterials in small and confined areas to the same degree of compaction in large areas. Therefore, the target deflection values should be adjusted according to the characteristics of compaction in small areas. Field LWD tests revealed that the deflections for lightweight compactor were greater than those for large roller. The overall ratios between the deflections in small and large areas are 1.192, 1.239, and 1.227 for 2017, 2016, and historical projects, respectively. No rigorous scientific methods are currently available to determine a factor for adjusting the target deflection. To avoid unnecessary complexity, 1.219, i.e., the average of the above three deflection ratios, is used as the adjustment factor for considering the characteristics of small area compactions.

Placing an unbound aggregate layer on chemically modified subgrades may produce an inverted two-layer system, and thus, the deflections may increase as the aggregate layer thickness increases. Nevertheless, the field LWD test results did not fully agree with the variation trend of deflection for the inverted layer system. Many factors, such as layer thickness, subgrade strength, and degree of compaction, may affect the lateral confining stress in the unbound aggregate layer under the impulse load generated in the in-situ LWD test. These factors may also interact with each other, which make it more difficult to accurately determine the effect of inverted layer structure. The potential effect of inverted layer system was not considered when determining the maximum allowable deflections.

The structural response of an elastic layer system to external loading may vary dramatically with the boundary condition. The deflection at the outside edge may be up to 40% and 35% greater than the deflections in the middle and inside edge, respectively. Therefore, caution should be exercised when determining the position to perform LWD testing for compaction QA, in particular in small areas.

Extensive in-situ LWD testing indicate that for small area compaction, a minimum of 5 LWD tests are required to provide reliable compaction QA. A minimum of 8 to 10 LWD tests are necessary for large area compaction. The minimum sample size should increase as the compaction area increases, taking into account the requirement of at least 10 LWD tests for a test section of 100 ft. by 20 ft. for compaction of aggregates.

The majority of the projects have a COV of 20% to 35%. For small area compaction, a COV of 20% or less may indicate “Low” variation, a COV of 20% to 35% may indicate “Normal” variation, and a COV greater than 35% may indicate “Poor” variation.

Annual verification is necessary to ensure repeatability of LWD deflection measurements.

6.2 Major Recommendations

When performing laboratory Proctor test to determine the target deflection (or modulus), the inner wall of the mold should be properly lubricated.

The use of LWD test for compaction QA does not change the procedures of field compaction in roadway construction. It is of significance to compact aggregate materials near the optimum moisture content.

For aggregate compaction, the LWD deflection varies significantly with the moisture content. It is recommended that the LWD test for compaction QA should be conducted within two hours after compaction. In-situ moisture content test is necessary to implement QA for compaction with LWD.

The maximum allowable deflections recommended by this study should be further fine-tuned taking into account the field practice and experience in roadway construction statewide.

Back calculation of the aggregate modulus from the mold or in-situ deflection is subject to the effects of many factors. Also, changing to a modulus-based quality control or assurance would produce data that could not be compared with historical data. It is advisable for INDOT to continue to use deflection as the target parameter for QA of compaction.

Different LWD devices may have different features, leading to different deflection or modulus measurements. Further effort is needed for INDOT to support more than one type of LWD devices.

The structural response of an elastic layer system to external loading varies dramatically with the boundary condition. Caution should be exercised when determining the position for performing LWD testing for QA of compaction. In small compaction areas, it is advisable to perform LWD testing 3 feet away from the outside edge or in the middle of the lane or shoulder under uniform compaction.

Calibration of LWD devices is costly and time-consuming. However, use of LWD devices out of calibration can be even more costly. Urgent effort is needed to assess the possible positive and negative effects of the calibration interval and determine the optimum calibration interval.

Discrepancies observed in field compaction and LWD testing by different contractors and inspectors suggest that necessary training is needed to further improve construction quality and ensure QA consistency.

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The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,600 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

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