

**Utilization of the Light Weight Deflectometer for  
Flexible Pavement Construction and Performance**

by

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## **Abstract**

At the National Center for Technology (NCAT) Pavement Test Track in Auburn, Alabama, accelerated pavement testing occurs in three year cycles on 46 unique 200 foot pavement sections that are subjected to 10 million equivalent single axle loads (ESALs). Typically, at this facility, the focus is testing on the asphalt layer itself, however it was noticed in previous research cycles that premature failure of the sections may have been caused by the unbound foundational layers (i.e., subgrade, aggregate base) of the pavement structures.

The current standard practice for quality control (QC) of these foundational layers during construction is monitoring in situ density and moisture content with a Nuclear Density Gauge (NDG). While the moisture and density values may meet the specified criteria, they are not parameters that directly affect pavement design. Another device, the Light Weight Deflectometer (LWD), measures the deflection and modulus of the material and more fundamentally characterizes the engineering properties of the in situ materials and has potential to improve construction QC by establishing and testing for target modulus values of the unbound materials.

To develop an understanding of how an LWD could help improve construction controls, a study relating to these unbound layers was created by introducing the use of a commercially-available LWD at the Test Track in the 2024 research cycle. The LWD is a portable device similar to a small scale Falling Weight Deflectometer (FWD). The desire to use the LWD for testing on unbound foundational layers has been on the rise due to the relationship modulus values have related to structural capacity during pavement construction as opposed to density and moisture values from typical QC devices such as the NDG. Testing was performed with the LWD alongside the NDG during the 2024 Test Track reconstruction on six fully rebuilt sections at twelve locations

throughout each section. Four out of those sections were used for an in depth study to relate NDG values to LWD results and create standard target values for the material tested through different equations relating to field testing. The main finding from this research was by using the optimum moisture content related to laboratory Proctor testing and LWD data collected in reconstruction, a minimum modulus value and maximum deflection value were set for the base material used at the Test Track. For the Test Track Base materials a QC limit of maximum deflection of 0.020 inches (0.504 mm) and a minimum modulus value of 6,908 psi (47.64 MPa) was determined. This also established a method that could be used for other unbound materials based on the optimum moisture content.

Furthermore, since construction is typically done every three years at the Test Track, there was interest in using the LWD in other ways apart from construction monitoring. Additional research was performed on three existing asphalt sections that were built in the 2021 test cycle. These three sections were part of an ongoing additive group experiment section and were experiencing varying levels of cracking distress (high, medium, and none). This allowed for a relationship between cracking distress level and average modulus values of the asphalt layer produced by the LWD to be established. Areas with heavy distress levels had an average modulus of 18,319 psi, areas with medium distress levels had an average of 27,270 psi, and areas with light to none had an average of 35,942 psi. These values were found from an average of all three test dates with no temperature corrections. These average values can help to understand the quality of distress a section is experiencing. Also, this additional LWD testing was performed alongside the FWD to create comparisons between the two devices. It was determined that the FWD produced significantly higher deflection and modulus values than the LWD, however the trends of the two results were similar despite the magnitude difference. Two additional LWDs were also tested

alongside the main LWD used at the Test Track to understand differences between different LWD devices.

Overall, from the performed research, a primary result was determining preliminary modulus and deflection values for QC for the Test Track Base material. A testing method was also determined for thin asphalt pavement sections to determine the structural integrity of a pavement section based on modulus values produced by the LWD. The overall purpose of this thesis was to determine how the LWD could be used in two different, practical and beneficial ways at the NCAT Test Track for QC and other evaluations.

## **Artificial Intelligence (AI) Use Disclosure Statement**

In preparation of this thesis, no Artificial Intelligence (AI) tools were used.

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## List of Abbreviations

AC	Asphalt Concrete
APT	Accelerated Pavement Testing
ASG	Asphalt Strain Gauge
ASTM	American Society for Testing and Materials
DOT	Department of Transportation
EPC	Earth Pressure Cell
EVD	Modulus
ESAL	Equivalent Single Axle Load
FDR	Full Depth Reclamation
FWD	Falling Weight Deflectometer
GB	Granular Base
GTR	Ground Tire Rubber
LWD	Light Weight Deflectometer
NCAT	National Center for Asphalt Technology
NDG	Nuclear Density Gauge
OMC	Optimum Moisture Content
PSI	Pounds per Square Inch

PCF       Pounds per Cubic Feet

QA       Quality Assurance

QC       Quality Control

RL       Random Line

## **Chapter 1: Introduction**

### **1.1: Background**

The National Center for Asphalt Technology (NCAT) Pavement Test Track is 1.7-mile full-scale research and testing facility that has 46 different 200-foot test sections that experience trafficking for 10 million ESALs during a three-year test cycle. The main focus of the research conducted at the NCAT Test Track is on the performance of varying types of asphalt materials, however recent studies from the Test Track have noted that the unbound materials (i.e., subgrade and aggregate base) that make up the foundational layers may have affected surface performance. This may be due to subgrade and aggregate base layers typically having larger variations in their as-built properties, unlike the asphalt concrete layer.

When test sections are reconstructed at the Test Track, the normal practice is similar to typical road-building where a nuclear density gauge (NDG) is used to perform quality control and ensure the properties of the foundational layers meet the specification. The NDG measures the moisture and density of the material, and these values correlate to the laboratory Proctor compaction test results (AASHTO T180). Despite tested values of the subgrade and base passing the set NDG criteria during construction, backcalculated moduli obtained through Falling Weight Deflectometer (FWD) testing indicated a wide range of values from recent Test Track experiments. These variations were suspected to have played a role in performance of the asphalt layers. Due to this finding, there has been an interest in performing more structural evaluations of the foundational layers during construction.

Another research effort at the Test Track is using the FWD to evaluate structural conditions of the pavement post construction. However, testing with the FWD is time consuming for the 12

fixed locations per section it routinely tests. A device that could be utilized in a more efficient and easier manner than the FWD as a rapid structural assessment tool as well as a tool for QC during construction would be extremely beneficial for Test Track research. A portable device for spot specific testing would also increase performance data throughout sections. A light weight deflectometer (LWD) was decided to be used to test viability for use both during construction and post construction at the Test Track.

For all research efforts, a commercially available LWD was used to conduct experiments during the 2024 Test Track cycle during reconstruction in the summer of 2024 and during trafficking from 2024 through 2026. The LWD used was the Zorn ZFG 3.1 shown in Figure 1.1.



***Figure 1.1: Zorn ZFG 3.1 on Aggregate Base at the NCAT Test Track***

The LWD is a nondestructive testing device that measures the deflection of the surface under impact loading which allowed for the modulus to be measured in real time for the unbound foundational layers. During reconstruction testing at the Test Track, six sections were tested with

the LWD on multiple days throughout the construction time frame. These six sections were being completely rebuilt; they are known as “structural” sections meaning that they have all previous materials removed down to the subgrade and replaced while including instrumentation during the construction process. This allowed for testing on the subgrade and base materials. Post construction, the LWD was used for additional testing on the asphalt surfaces of three sections that were constructed in the 2021 test cycle as part of the Additive Group Experiment.

Using the LWD at the NCAT Test Track as a versatile testing device was the key motivation for this research. The viability of the device in the construction phases as well as utilization as an accelerated pavement testing (APT) structural condition assessment tool was a key overall goal of this research. The following are specific objectives of this study.

## **1.2: Objectives**

The main objectives for this research were to:

1. Establish a way to implement the LWD into future reconstruction and testing efforts at the NCAT Pavement Test Track.
2. Create an evaluation framework for using the LWD as a QC device alongside the NDG during Test Track reconstruction.
3. Develop a method to use the LWD as an APT structural condition assessment tool during trafficking at the Test Track.

## **1.3: Scope of Work**

This research started in May 2024, at the start of the 2024 NCAT Pavement Test Track reconstruction effort. LWD testing was done from May 2024 until September 2024 on unbound

materials. During reconstruction, nuclear density gauge, weather station data, and overall property data were also obtained. Initial laboratory testing was done in the same time frame as the start of construction. Traffic on the Test Track started in October of 2024 which was when the data under traffic collection was started. This included falling weight deflectometer data, surface cracking data, temperature information, and pressure data. Additional LWD and FWD data were collected in June 2025, October 2025, and February 2026 on a select number of sections to assess LWD data for structural condition evaluation. Trafficking on the Test Track for this cycle will continue until 2027.

#### **1.4: Organization of Thesis**

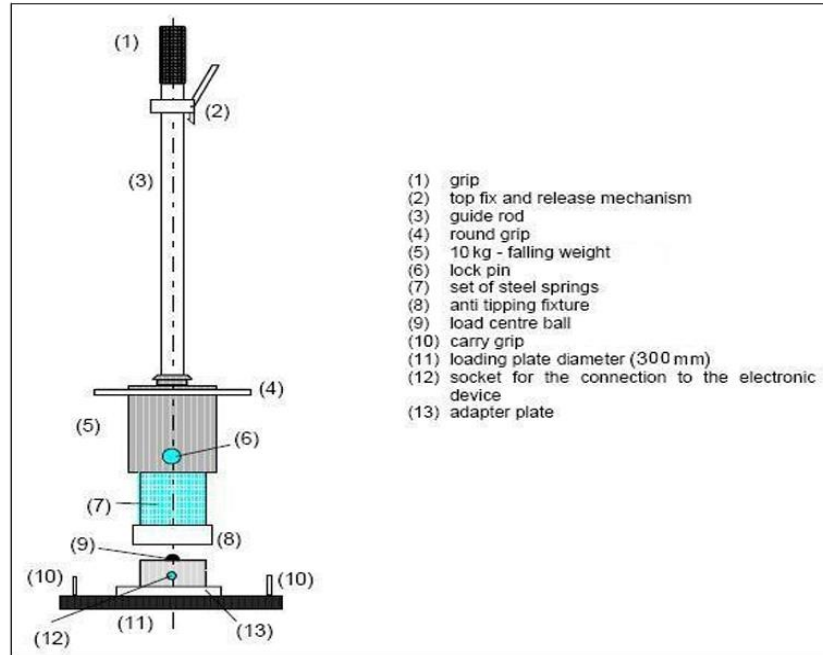
The organization of this thesis starts with the introduction of work in Chapter 1. This chapter describes the motivation for this research, the main objectives, and the scope of work. Chapter 2 is comprised of a Literature Review focused on the Light Weight Deflectometer. All the facilities, equipment used, and testing done are discussed in Chapter 3. The following two chapters summarize the two main data collection efforts. Chapter 4 covers the field testing done during construction and the results from those efforts. Chapter 5 is a summary of the data that was gathered under traffic at the Test Track and the resulting analysis. Both Chapters 4 and 5 include detailed discussion on the results from testing. Conclusions and recommendations are discussed in Chapter 6. References and appendices are provided at the end of the thesis.

## **Chapter 2: Literature Review**

This chapter presents background information on previous research done on LWD equipment and testing. This includes an overview of the LWD, current practices, the relationships between the LWD and other testing devices, and the benefits of using the LWD.

### **2.1 The Light Weight Deflectometer**

Generally speaking, the LWD is comprised of a loading plate, a guide rod, a falling weight, an electronic data collection device, and its connection wire, as shown in Figure 2.1 (Wang et al., 2024). This device is also referred to as a Portable Falling Weight Deflectometer (PFWD). This is due to it being a small scale version of the FWD. A typical configuration of the LWD includes of a 10 kg falling weight, maximum falling height of 500 mm, a plate diameter between 150 mm and 300 mm, and a sampling frequency of 5 kHz (Kongkitkul et al., 2014). The drop height for the LWD can be determined by the user but many have a mechanism whereby a consistent drop height can be used when testing.



**Figure 2.1: Schematic of the LWD (Wang et al., 2024)**

The use of the LWD has been on the rise in recent years due to the desired shift from solely density and moisture quality control to incorporating additional structural and stiffness-based quality control of pavement foundational layers. Since the only QC specifications often measured during construction are on moisture and density, contractors only seek to meet these QC parameters in the cheapest way, perhaps not the best way for future performance (Ordaz et al., 2025). One of the main reasons for this shift is that both the density and moisture do not directly relate to the pavement design input properties and expected performance (Hossain and Apeagyei, 2010). The focus of the newer mechanistic-empirical design approach focuses on key inputs of elastic and resilient moduli that are not found directly from the moisture and density (Hossain and Apeagyei 2010). Density is an important characteristic of the unbound layers, but the stiffness characteristics of those same unbound layers in the field is also important (Kumar et al., 2017). The LWD has the ability to test and monitor the pavement layers stiffnesses individually during installation and

compaction (Xu et al., 2011). This is a key component of using the LWD during construction. The use of the LWD to determine stiffness and modulus values are due to the stiffness of the subgrade being a critical input in the long-term stability and performance of the infrastructure (Wang et al., 2024). There have been efforts to correlate testing that is already in place to the up-and-coming LWD. Specifically, the Nuclear Density Gauge (NDG) and the Falling Weight Deflectometer (FWD).

The workings of the LWD are from the falling weight creating a shockwave, similar to the FWD, through the soil (Steinert et al., 2005). Deflection is then determined by either velocity transducers or accelerometers (Steinert et al., 2005), depending on the particular design of the LWD. The in situ modulus is determined by using the Boussinesq half-space equation from the deflection recorded on the LWD. This calculation occurs almost instantaneously within the handheld device. For data collected from the field, the following Boussinesq half-space equation is used for modulus calculations used in the Zorn instrument LWD devices (Ordaz et al., 2025):

$$E_{field} = \frac{2\left(\frac{L}{\delta}\right)(1-\nu^2)}{Ad} \quad (\text{Equation 1})$$

Where,

$E_{field}$  = modulus from the field, MPa

L = maximum load, kN

$\delta$  = maximum measured deflection from LWD, mm

A = stress distribution factor ( $\pi$  is assumed for uniform distribution)

$\nu$  = Poisson's ratio (assumed to be 0.5)

d = plate radius of LWD, cm

There are a few brands of LWDs that are prominent in the market today. These include brands such as Zorn Instruments, Dynatest, Olson Instruments, and Prima. All models, despite the brand, follow similar device configurations. Some testing has been done to compare the different versions. Some Dynatest LWDs models resemble the FWD with geophone attachments, different plate size attachments, and drop weights, whereas Zorn products tend to be on the simpler side, but also includes different plate sizes and drop weights (Ordaz & Doyle, 2025). Ordaz and Doyle tested a Dynatest and Zorn LWD alongside an FWD. The results between the two LWDs provided statistically different impulse stiffness modulus (ISM) results, which is the evaluated parameter in the research. It was noted that the Zorn LWD produced higher ISM results than Dynatest. Stamp and Mooney also compared two LWDs, a Zorn and a Prima LWD. The conclusion from this study was that the difference in configuration, one having an accelerometer and one having a geophone, of the two LWDs was the main cause in differing results (Stamp & Mooney, 2013).

## **2.2 Standardization Efforts for the Use of the LWD**

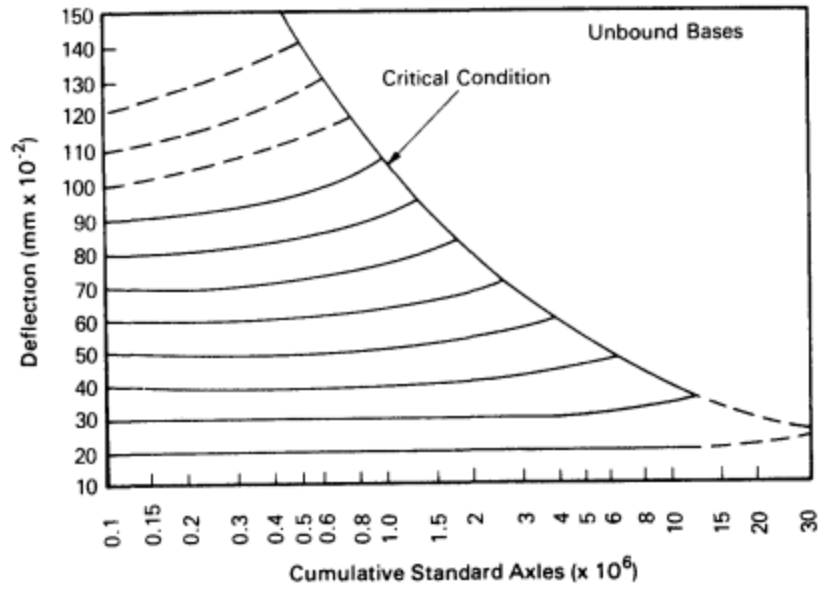
Currently there are two ASTM standards for the LWD:

- ASTM E2583-07: *Standard Test Method for Measuring with a Light Weight Deflectometer*
- ASTM E2835-21: *Standard Test Method for Measuring Deflections Using a Portable Impulse Plate Load Test Device*

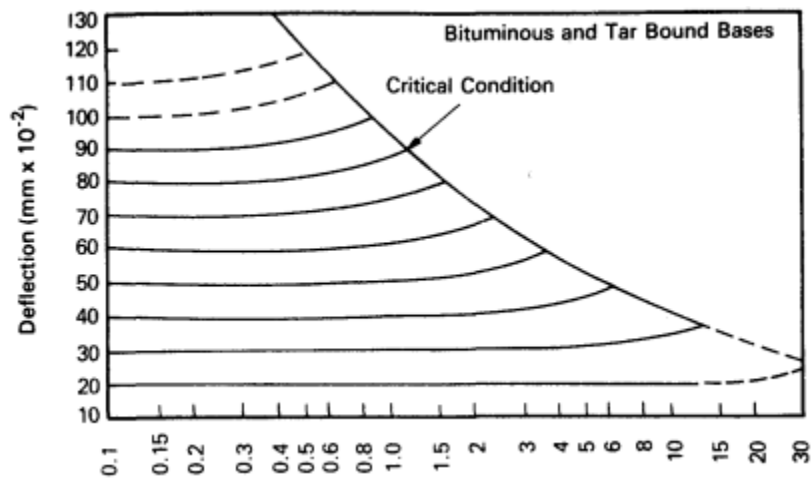
The main difference between these two standards is that E2583-07 has been reapproved most recently in 2025 and it also includes testing on both unbound and bound pavement surfaces. In E2835-21, there is only reference to unbound pavement layers. This standard was last revised in

2021. Despite these differences, both standards have similar procedures and methods of operations. In both standards a key mention from the significance and use section reflects on how the data from the LWD testing can be used for QC and QA efforts relating to the compacted unbound foundation layers, structural evaluation of the load-carrying capacity, as well as requirements for thickness on highway and airfield pavements.

From these specifications, the LWD could be used as a QC/QA device as stated in ASTM E2583-07, the stiffness of either bound or unbound materials can be determined by the deflection measurements and proper back or forward calculation methods. In the significance and use section in both standards reflect on the use of the recorded deflection as either a direct correlation to the pavement performance or used in the determination of in-situ pavement foundational layer material characteristics. In ASTM E2835-21, it is stated that unbound testing should be done directly after compaction, and the pavement foundation layers should not be frozen. From these standards, the use of deflection data for use as a QC/QA device can be beneficial. The implementation of these standards and using the deflection data from the LWD could also be used in a similar fashion to the FWD data analysis presented in the AASHTO Guide for Design of Pavement Structures (1993). This document has a section for using deflection results from nondestructive testing (NDT). Deflection data from NDTs are typically used for evaluating the pavement's structural capacity (AASHTO 1993). Shown in Figure 2.2 are two graphs that relate the deflection and cumulative standard axles with a critical zone of both unbound bases and bituminous and tar bound bases. This idea could be incorporated into LWD deflection data on unbound materials.



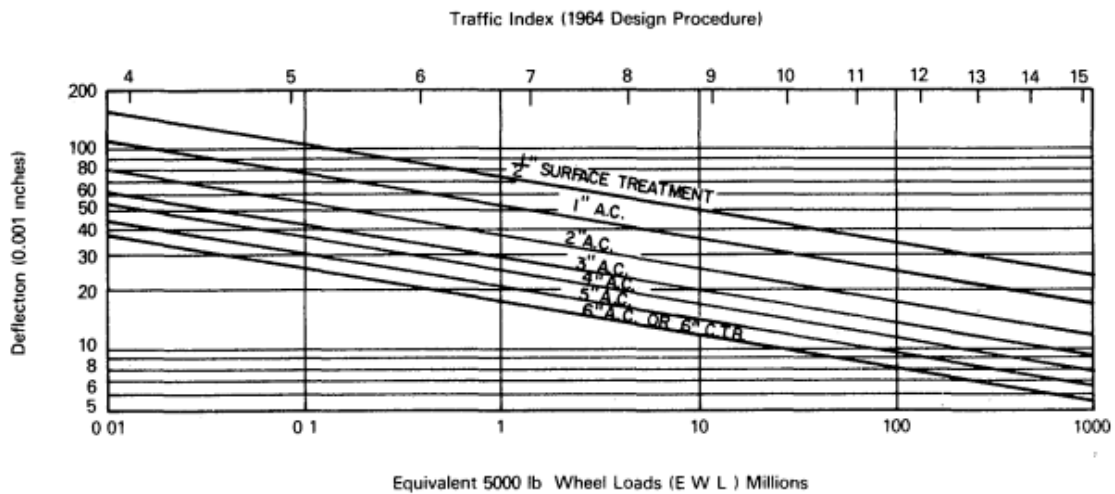
(a)



(b)

**Figure 2.2: Relationship Between Deflection and Life (AASHTO 1993)**

Also in the AASHTO 1993 document is Figure 2.3, which shows deflection, equivalent wheel loads, and traffic index relating to an overlay surface treatment. This graph is for asphalt surfaces.



**Figure 2.3: Relationship Between Deflection and Life for Asphalt Pavements [AASHTO 1993]**

This figure from AASHTO 1993 could influence the use of the LWD deflection data in ways that set a critical deflection value that indicates significant need for treatment of the asphalt surface.

Current practice does not include a strong presence of the LWD in most US states. However, there is an increase in desire for use due to the cost effectiveness, lack of radiation in comparison to devices such as the NDG, and ease of use. The main state DOTs that are researching the implementation of the LWD specifically for QC of unbound pavement layers are Minnesota, Illinois, Indiana, Nebraska, Missouri, Maryland, and Georgia. Not all DOTs are performing the testing with the same QC parameters, but the desire to use the LWD as a QC device overall is consistent between the states.

The Minnesota DOT (MnDOT) was interested in utilizing the light weight deflectometer and the dynamic cone penetrometer (DCP) test instead of the destructive sand cone test (Berkland et al., 2008). MnDOT was able to implement this approach by performing research using the LWD and DCP on two projects alongside the sand cone test. Validation from this research is what led to

the specifications for drop and test numbers for both devices. Report 2007-11 “Pavement Design Using Unsaturated Soil Technology” (Gupta et al., 2007.) was also used to help create the MNDOT specifications. The main document for the initial implementation was Report 2006-20 “Validation of DCP and LWD Moisture Specifications for Granular Materials”. This assured the quality of testing using the DCP and LWD on the unbound foundational layers during pavement construction. From this report (Davich et al., 2006), the following recommendations specifically for the LWD were made for testing:

- Consistent combinations of the falling mass, drop height, and plate size are needed. The recommended combination is a mass of 10 kg, drop height of 50 cm, and plate diameter of 20 cm.
- Seating drops are necessary in testing. Three seating drops should be performed before the three actual testing drops.

These guidelines are meant to produce the most consistent results by using the LWD for MnDOT projects. MnDOT has a few methods for QC using the LWD target values. The first is to test in a constructed calibration area or to use the predetermined target values (MnDOT, 2026). These predetermined values can be seen in Table 2.1.

**Table 2.1: MnDOT Target LWD Values (MnDOT, 2026)**

<b><i>Specification</i></b>	<b><i>Material Type</i></b>	<b><i>Minimum Elastic Modulus (MPa)</i></b>
2106	Granular	40
2106	Clay and Clay Loam	20
2211 or 2215	Base or Reclamation	50

Similarly to MnDOT, INDOT has also created some standards for LWD testing and acceptance in the field. However, INDOT’s standards (*INDOT 215-R-715, 2020*) are based on the allowable deflection of the material rather than the modulus value. Table 2.2 shows the maximum allowable deflection for chemically modified soils. The decision to base LWD QC on deflection or modulus is based on engineering judgement from the DOT.

**Table 2.2: Allowable Deflection for Chemically Modified Soils [INDOT 215-R-715, 2020]**

<i>Material Type</i>	<i>Allowable Average Deflection (mm)</i>	<i>Maximum Deflection at a Single Test (mm)</i>
Cement Modified Soils	0.27	0.31
Lime Modified Soils	0.30	0.35

There has been an increase in standards for using the LWD in Indiana, specifically for the Zorn model of the LWD. Another study performed in Indiana correlated the optimum moisture content and densities determined by laboratory Proctor tests and test pit research to the maximum allowable deflection from the LWD to establish QC requirements (Yao et al., 2023). Shown in Table 2.3 is the recommended maximum deflections for different layer thicknesses, different moisture contents, and different modulus values.

**Table 2.3: Recommended Maximum Deflection Values (Yao et al., 2023)**

Layer thickness (cm (in.))	$E_0$ (psi)	$m=m_o-4\%$	$m=m_o-3\%$	$m=m_o-2\%$	$m=m_o-1\%$	$m \geq m_o$
15.2 (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
	12,700	0.212	0.234	0.265	0.316	0.387
	14,000	0.198	0.219	0.263	0.300	0.370
30.5 (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
	12,700	0.184	0.215	0.265	0.342	0.462
	14,000	0.176	0.207	0.256	0.333	0.452
45.7 (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
	12,700	0.154	0.188	0.243	0.331	0.467
	14,000	0.150	0.184	0.238	0.327	0.463

A main recommendation derived from Yao et al. (2023) was that QC testing with the LWD should only be performed immediately after compaction when the unbound foundational layer is at the optimum moisture content. It was also noted that the QC limits with the LWD should be set forth by using maximum deflection values as they relate to moisture content.

The current practice for the Nebraska DOT is to perform LWD testing in between roller passes done by the contractor for compaction. Testing in the field for NDOT field personnel is done for both moisture and LWD deflection on embankments, subgrade, foundation course,

granular fill, and select granular backfill (NDOT, 2017). Similarly to INDOT, NDOT uses the deflection values for testing rather than the modulus.

In Missouri, with MoDOT, there has been research conducted to implement the LWD for construction acceptance for the unbound material layers. Something interesting in the reports from MoDOT, are that they are looking to steer completely away from the NDG by using a different moisture content testing device in the field alongside the LWD. This allows for the removal of the NDG and the radiation it inherent to its use. Some relevant conclusions from this research were the design of an isolation unit to effectively use an Ohaus MB 120 to collect moisture content data and that the LWD had good performance compared to the NDG evaluation method, with better performance on fine-grained soils rather than coarse-grained material (Zhang et al., 2024).

Similar to the research being implemented in Missouri, Maryland DOT has been standardizing the modulus measurements from the LWD alongside non-nuclear moisture testing devices for quality assurance of compaction (Schwartz et al., 2017). It is noticeable that there has been a shift from using the NDG to non-nuclear devices due to safety concerns, costs, and training. In this research study, three LWDs, two non-nuclear moisture content measurement devices, and an NDG were used. Their findings were that the Ohaus MB45 was able to provide accurate correlation to the NDG after a correction factor of 1.11 was applied. The research also deduced that the need for a set target modulus is imperative for QA, which proved to be difficult and there was no confidence in the estimation for the field LWD modulus values. However, after field testing two test methods/specifications were drafted for LWD testing in the field and determining the target modulus in the laboratory. The research done with MDOT allows for a shift from density-based methods to a modulus-based QA method (Schwartz et al., 2017).

Georgia DOT has used the LWD for testing for full depth reclamation (FDR) performance testing (Kwon et al., 2020) and design guidelines for pavement embankment construction (Kim et al., 2019). For FDR pavement construction, the recommendation is to use the LWD or FWD to monitor the stiffness characteristics over the construction period (Kwon et al., 2020). From the testing done by GDOT for FDR construction, the LWD is recommended for QC/QA especially for the first three days after treatment on FDR projects. The LWD was able to distinguish the changes in stiffness over the curing period of the FDR projects. For pavement embankments, the LWD was used to determine effects that geosynthetics had on the stiffness of the foundational pavement layers. LWD testing was done pre-traffic and post-traffic to see the difference in stiffness of the subgrade materials (Kim et al., 2019).

The main use of the LWD has been on unbound foundational layers during pavement construction however, the LWD has been used on thin asphalt pavement sections and related to existing structural testing devices, this is highlighted more significantly in the next section.

### **2.3 Correlations with the LWD**

The main correlations that have been studied relating to the LWD are relationships with the NDG for unbound foundation layers and with the FWD for both unbound and bound pavement layers. There have also been relationships derived between other unbound layer testing equipment that can be used during construction, inputs for pavement design software, and soil properties.

The NDG tests the density and moisture of the material being tested. From these values a degree of compaction can be determined by correlating the results to the laboratory Proctor test of the material. It has been seen in LWD data that was compared to the degree of compaction that when the LWD modulus increased, the degree of compaction also increased. This showed that

when material such as subgrade is well-compacted, less deflection will occur due to the increased rigidity of the material (Wang et al., 2024). The increased compaction efforts can tie together an increase in the dry density from the degree of compaction, bearing capacity from the California bearing ratio, and surface stiffness from the LWD (Kongkitkul et al., 2014). In the previously mentioned MoDOT Research Report #24-010 (Zhang et al., 2024), comparisons were made between assessments from LWD and NDG tests performed on 3 construction sites. Three LWDs were tested alongside the NDG, the results were that two of the LWDs produced 2/3 of the same acceptance as the NDG, whereas the third LWD only 1/2 of the results were in agreement.

Another distinct study done was able to establish base relationships between soil properties and the LWD. This study reflects on how important the shear strength parameters of soil are and created a relationship between the LWD deflection and modulus values to friction angle and cohesion values of tested soils (Nabizadeh et al., 2019). The authors indicated that knowing the shear strength parameters of the soil are important due to the bearing capacity and shear failure due to the use of heavy vehicle loadings on pavements.

There has been interest in using the modulus determined from LWD testing on unbound materials to correlate to the resilient modulus parameter that is used in pavement design software, specifically AASHTOWare Pavement ME Design Software (Chowdhury & Kassem, 2025). In the software there are three levels of specificity tied to the type of input used in design. Level 1 is extremely specific, Level 2 is more general region specific inputs, and Level 3 has the least amount of detail. Being able to derive a correlation between the LWD modulus and resilient modulus would allow for higher level input to be used in the design software. Chowdhury and Kassem (2025) concluded that modulus results from the LWD had a good correlation to predict resilient modulus with an  $R^2$  value of 0.77 allowing for Level 2 inputs to be used. Another correlation

between the deflection and resilient modulus has been made with Equation 2. This equation relates the deflection recorded by the LWD to the resilient modulus (Umashankar et al., 2016):

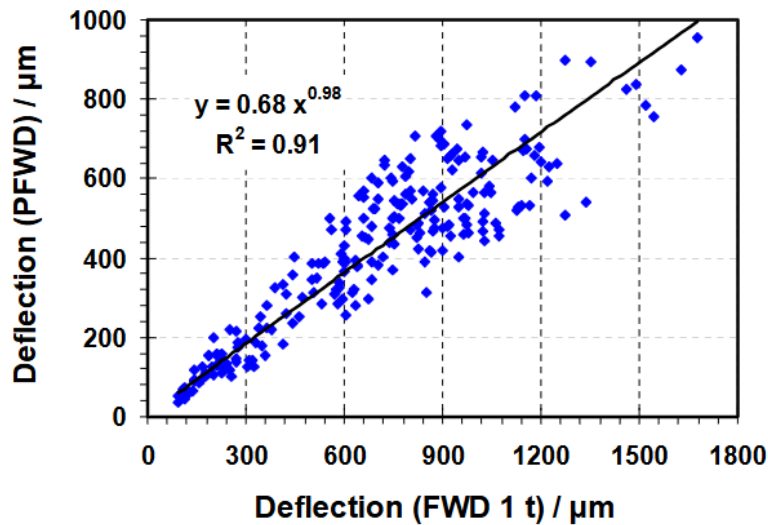
$$M_R = 22\delta_B^{-0.96} \quad \text{(Equation 2)}$$

Where,

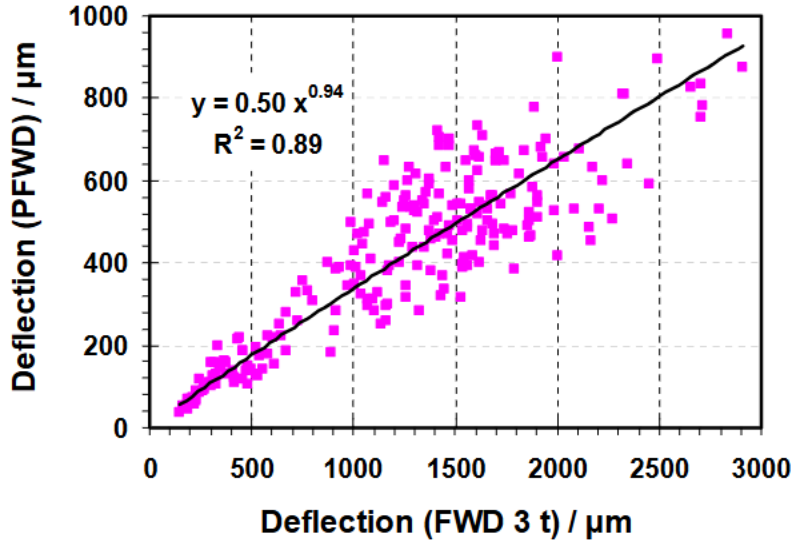
$M_R$  = resilient modulus of base material, MPa

$\delta_B$  = deflection of base material from LWD, mm

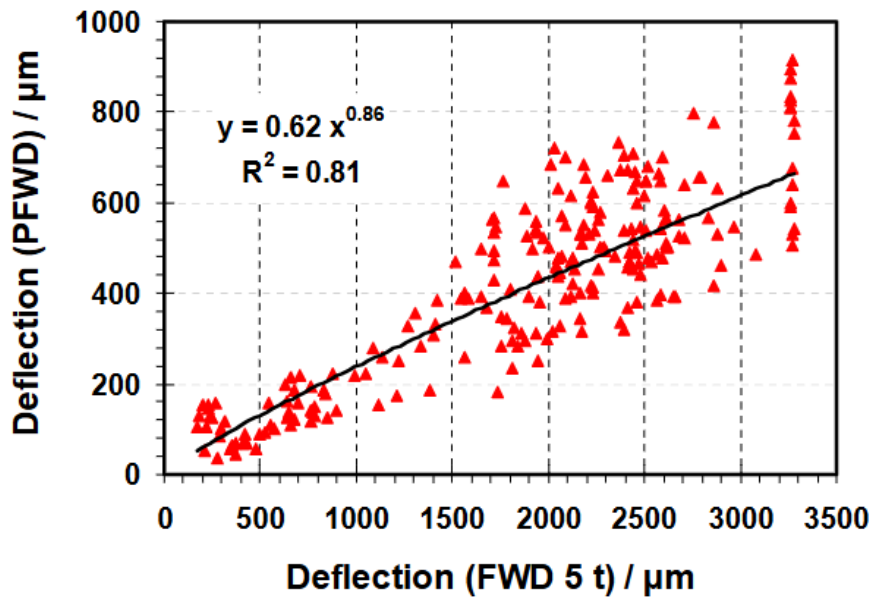
A study performed on the subgrade of a highway in China tested the relationship between the FWD and LWD on unbound pavement layers. Shown in Figure 2.4 is a correlation made between a 1, 3, and 5 ton FWD and an LWD done by Xu et al. (2012). In this study the deflection produced by the LWD and FWDs were compared, rather than the modulus values from each testing device.



(a) 1 Ton FWD



(b) 3 Ton FWD



(c) 5 Ton FWD

Figure 2.4: Correlation Between LWD and FWD Deflections (Xu et al., 2011)

What was seen in the research done by Xu et al. (2012) was that as the FWD weight increased, the correlation with the LWD decreased. However, there was a strong correlation between all three FWDs, with the strongest being an  $R^2$  value of 0.91 between the LWD and the 1

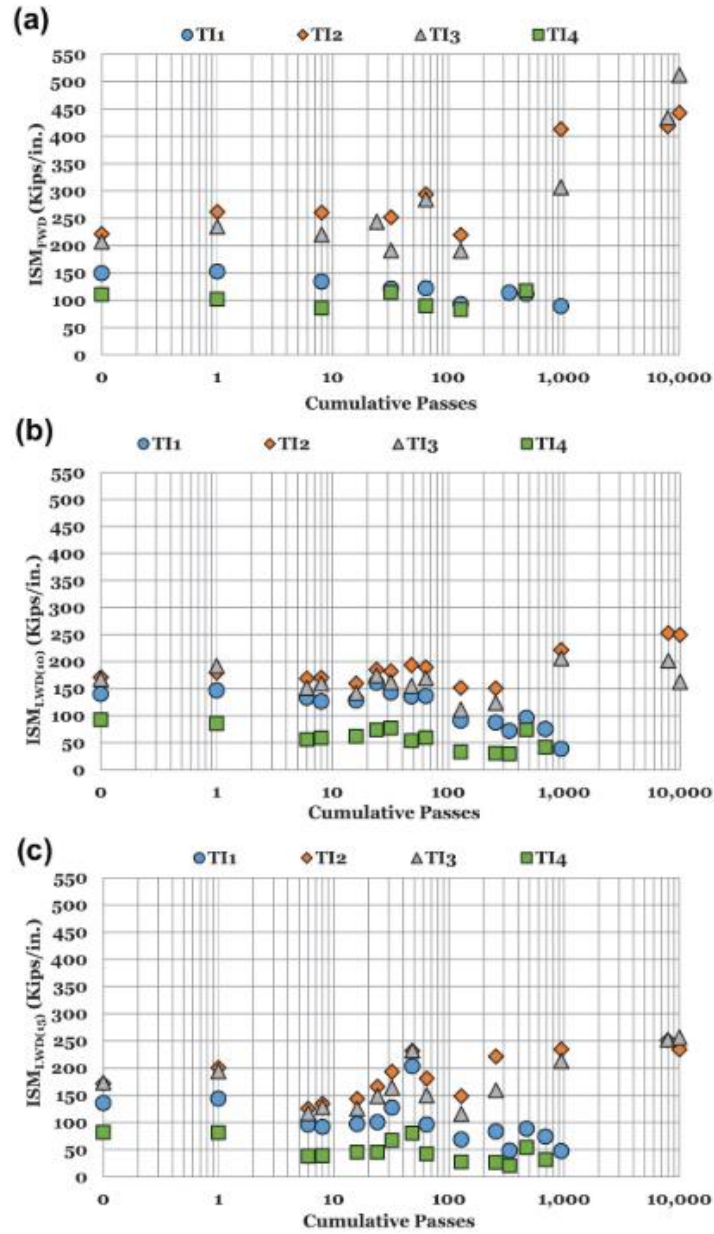
ton FWD (Xu et al., 2011). The mass of the LWD that was used was only 10 kg which is significantly less than the FWDs. This is a key comparison that has been made by many researchers. Since the shockwave that is produced by the LWD is less powerful than a shockwave produced by an FWD it cannot penetrate as deep into the layers below the test layer (Wang et al., 2024). This fact also makes the correlation of the instantaneously-produced modulus value of the LWD to multilayer backcalculated moduli from the FWD more difficult. This has led to thinking that the LWD is better for using as a control device for compaction during construction of the subgrade layers rather than using the LWD like an FWD to detect the resilient modulus after the last layer of the pavement is completed (Wang et al., 2024).

The FWD is primarily used for testing the deflection of the entire pavement which is done routinely on all the pavement sections at the Test Track. One of the preliminary studies when the LWD was first emerging in popularity was done in 2001 by Livneh and Goldberg (2001) where the LWD was tested against the FWD for quality assessment during the construction of a roadway. The two devices were tested on the subgrade, granular subbase, granular base course, and the asphalt layer. The main findings from this study were that the LWD modulus was 0.3-0.4 times smaller than the magnitude of the FWD modulus, therefore there needs to be emphasis on using proper safety factors when utilizing the equipment (Livneh and Goldberg 2001).

Other research has concluded that there is a poor relationship between the LWD and FWD, where the values can differ from between 20% to 30% (Steinert et al., 2005). It can be noted that the depth of influence for the LWD and FWD are significantly different. A trend that was seen between the LWD and FWD results on bound pavement was despite the modulus differences; their results followed the same trends overall (Steinert et al., 2005). The results of comparing the FWD and LWD seem to differ in multiple studies that have been done, some indicating a higher

correlation and others showing a low correlation. Factors such as the material or pavement that is being tested, the version of LWD being used, and the overall goal of using the LWD can all play a role in creating variation of correlations between the LWD and the FWD (Ordaz et al., 2024). In some studies, the LWD is recommended for testing on the asphalt layer, however in other studies it is not recommended. One study that did not recommend using the LWD for testing the stiffness of the asphalt after construction was done by Commuri et al. (2012) with the Oklahoma Transportation Center. In this study, they were looking into the LWD as a more efficient and cost effective method for pavement testing instead of the FWD. They noted that the LWD modulus values were not correlating to their dynamic modulus or density of the asphalt mat. This is what led to their conclusion on not recommending using the LWD after construction.

An experiment was conducted by Ordaz et al. (2024) to evaluate the use of a LWD on low volume roads that were deteriorating due to accelerated loading. This testing was done on the asphalt pavement layer, rather than the unbound foundational layers. In this research two LWDs and an FWD was utilized in performing evaluation of the pavement based on gathered data. A 15 kg LWD and a 10 kg LWD were used in this research alongside a typical FWD. By using these devices, it allowed for a correlation between each LWD device to the FWD. A variable identified as the impulse stiffness modulus (ISM) was used to define the pavement structural condition at test locations. This parameter is a ratio of the force applied by the test device divided by the center deflection (Ordaz et al., 2024). Seen in Figure 2.5 is the modulus reported by each device used against the number of cumulative passes of a heavily loaded vehicle.



**Figure 2.5: Impulse Stiffness Modulus (ISM) from (a) FWD, (b) 10 kg LWD, and (c) 15 kg LWD versus cumulative pass number (Ordaz et al., 2024)**

Each LWD produced similar results, whereas the FWD produced higher results. Despite the difference in magnitudes, the trend for all three devices was similar. Figure 2.6 illustrates the comparisons that were made from comparing the LWDs to the FWD (Ordaz et al., 2024). From these plots, the highest correlation was between the 15 kg LWD and the FWD.

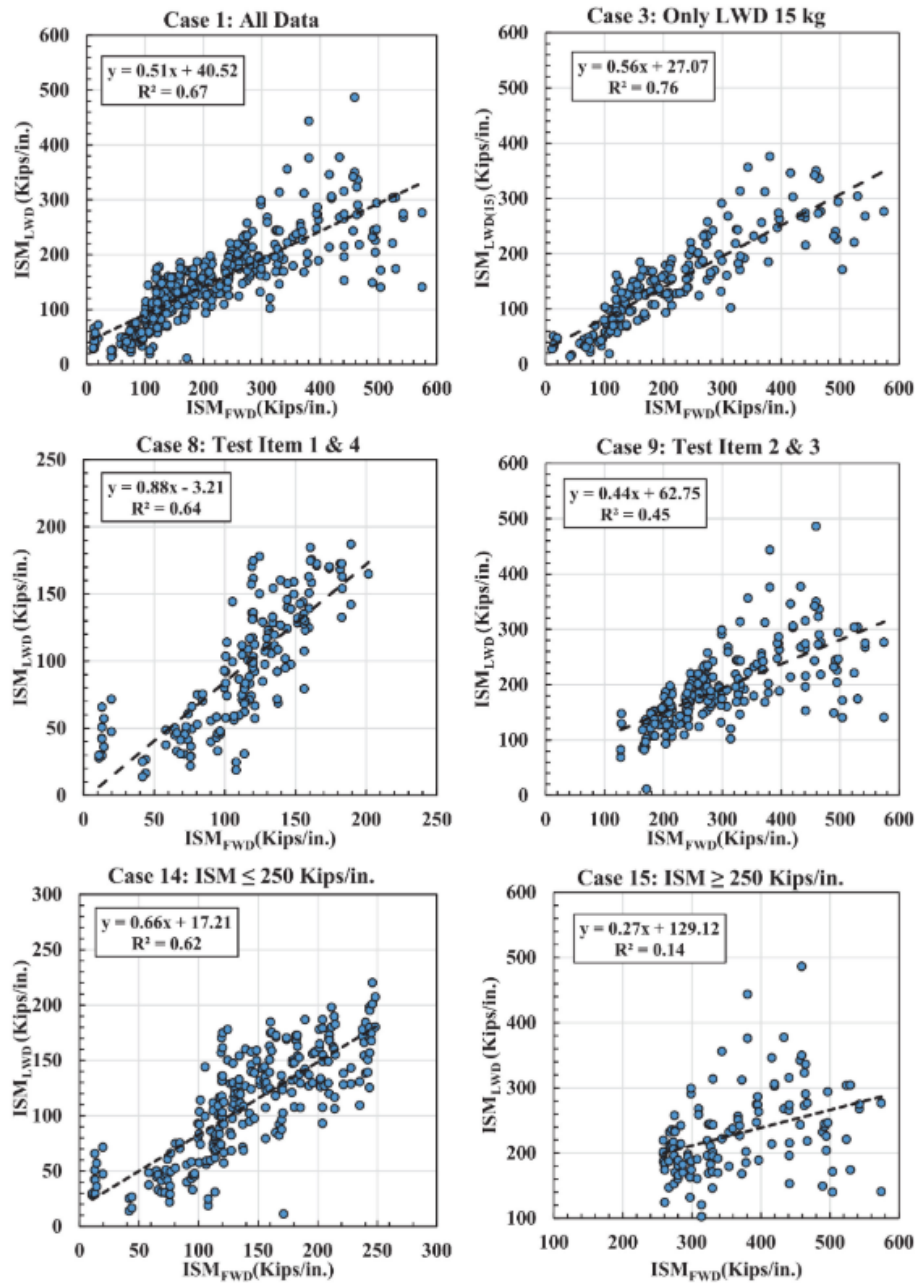


Figure 2.6: Specific correlations between LWDs tested and the FWD (Ordaz et al., 2024)

Another study done by Ordaz (Ordaz & Doyle, 2025) was conducted to validate the relationship between the LWD and FWD on thin pavement layers for testing structural evaluation.

Ten different set up configurations of the LWD with two different LWDs were compared to an FWD. Results indicated that by using a LWD with a 5.91 inch plate diameter, 22 inch drop height, and largest drop weight that is available for a given LWD there will be a good correlation to the FWD on thin asphalt pavement layers (Ordaz & Doyle, 2025).

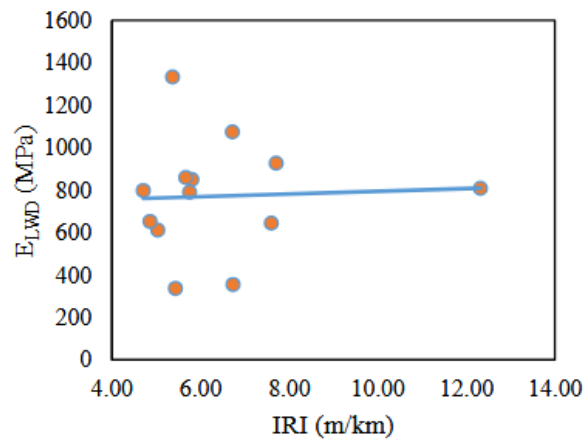
Similar to the use of LWD as a structural assessment tool on highway pavements, it has been looked into as a quick way to evaluate airfield pavements by the US Army Corps of Engineers (Garcia & Robinson, 2025). This research also tied in the use of a FWD to compare it to the LWD. This was to also validate the use of the LWD as a structural assessment tool due to its practicality and portability. Similar to the study performed by Ordaz (Ordaz & Doyle, 2025), the ISM was used as a defining parameter. Key findings from this research were that the LWD is primarily influenced by only the surface layer whereas due to the greater load magnitude the FWD has a wider scope of influence on the structure. The conclusion that both the LWD and FWD could both adequately assess the deterioration of the airfield pavements was also made (Garcia & Robinson, 2025).

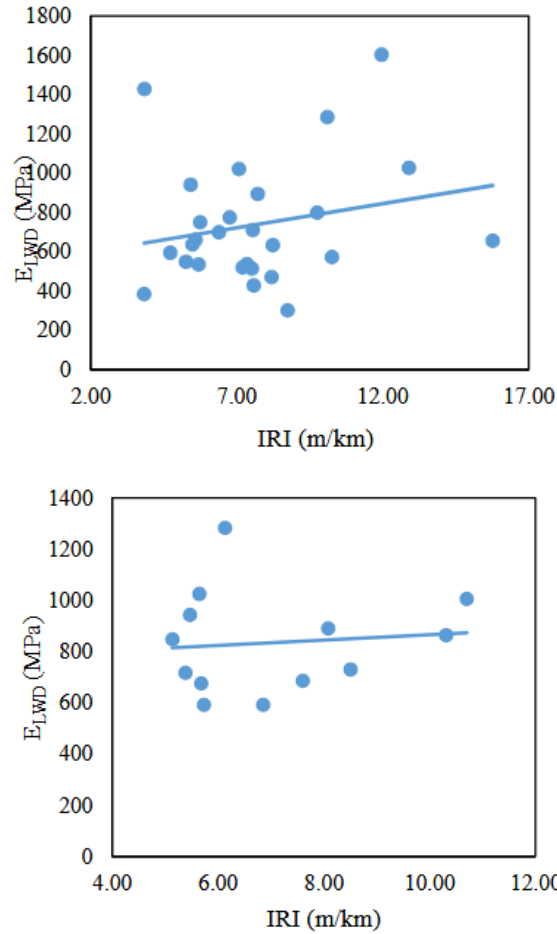
More testing with the LWD has also been done in relation to quality assessment of pavements. The LWD is used in this way to evaluate the modulus of the pavement section as a whole in a way that is faster and covers more area than a typical FWD. A study done in Indonesia describes how the typical way of measuring the quality of the road pavements is by utilizing the International Roughness Index (IRI) (Isnaini Kurniawati Djaha & Prayuda, 2019). In this study an LWD was used on the same sections of roadway that IRI was tested to see if a relationship between IRI and the pavement modulus could be found. Overall, with IRI it was found that there was no significant correlation or relationship between the modulus from the LWD and the IRI of the road (Isnaini Kurniawati Djaha & Prayuda, 2019). The data from this research is shown in Table 2.4.

**Table 2.4: FWD and IRI Comparisons (Isnaini Kurniawati Djaha & Prayuda, 2019)**

<i>Segment</i>		<i>Mean</i>	<i>Std. Deviation</i>	<i>n</i>	<i>COV</i>	<i>Coeff. Correlation</i>	<i>PCI Average</i>	<i>Visually</i>
24.22k	IRI(m/km)	4.438	2.106	13	25.423	0.047	99.92	Bump
	ELWD (MPa)	772.538	268.500					
24.13k	IRI(m/km)	7.022	1.889	13	37.598	0.1	91.85	Bump and Cracks
	ELWD (MPa)	834.462	198.230					
25.13k	IRI(m/km)	7.660	2.764	27	187.575	0.217	95.78	Plastic Deformation
	ELWD (MPa)	737.667	312.317					

Presented in the table, it shows that there is not a significant coefficient of correlation. Due to this observation, the results were not used in establishing a relationship between the two parameters. The data found on the three test segments was also plotted for a visual representation, this is shown in Figure 2.7. In the plots, the lack of a strong relationship between the two parameters is clearly evident.





**Figure 2.7: Plotted IRI and LWD Data (Isnaini Kurniawati Djaha & Prayuda, 2019)**

### 2.4 Benefits of the LWD

Some of the main benefits of the LWD are the speed of testing, portability, being a non-destructive test, and the lower cost. Some significant factors of using the LWD for field testing that have been noted are its portability and efficiency, data collection in real time, the accuracy and reliability, and the non-destructive testing (Aimil, 2023). Another benefit of using the LWD is the versatility of the device in different testing scenarios. The LWD can be used during new construction, pavement rehabilitation, during road maintenance, and for airport pavements to name a few (Aimil, 2023).

A main benefit that gives motivation for much of this research is the speed at which this test can be performed. When compared to testing such as Benkelman Beam and the sand cone method, the LWD produces speedier test results allowing for contractors and those in charge of construction to make decisions faster with rapid feedback (Xu et al., 2011). The NDG has a long list of disadvantages such as high initial cost, certification requirements, health risks, and complicated end of life disposal procedures (Steinert et al., 2005). These disadvantages of the NDG make the LWD seem like a better and more practical option for testing during construction.

An important benefit is that the LWD is a nondestructive test unlike some other geotechnical testing procedures that are done on pavement foundational layers. Shown in Table 2.5 is a comparison between the sand cone, dynamic cone penetration (DCP) and the LWD made by Roksana et al. (2018) This table lists some key advantages of using the LWD. In the table, the one disadvantage of the LWD is that a correlation between the deflection and modulus are needed between other parameters.

**Table 2.5: Comparing Sand Cone, DCP, and LWD (Roksana et al., 2019)**

<i>Method</i>	<i>Advantage</i>	<i>Disadvantage</i>
Sand Cone	<ul style="list-style-type: none"> <li>• Easy to operate and cheap</li> <li>• Reliable</li> </ul>	<ul style="list-style-type: none"> <li>• Time consuming</li> <li>• Destructive</li> </ul>
DCP	<ul style="list-style-type: none"> <li>• Minimal surface disturbance</li> <li>• Simple, reliable</li> </ul>	<ul style="list-style-type: none"> <li>• Not suitable for cohesive soil</li> <li>• DCP can break under repetitive drops</li> </ul>
LWD	<ul style="list-style-type: none"> <li>• Fast and easy to operate</li> <li>• Non-destructive</li> <li>• One person can conduct the test</li> </ul>	<ul style="list-style-type: none"> <li>• Need a correlation between deflection modulus and other parameters</li> </ul>

Another benefit that was mentioned by the Alaska Department of Transportation & Public Facilities (DOT & PF) when comparing the LWD to the NDG is the distinct cost savings where the LWD can eliminate most of the \$250,000 the DOT & PF spends due to using the NDG. This

is due to the LWD not requiring specialized contractors, licenses, safety training, storage costs, and hazardous material certifications (Simon and LaBelle, 2023). Another benefit the DOT & PF stated was the ease of use of the LWD in remote locations, especially in places such as Alaska. This benefit can be experienced in most remote locations, not just Alaska.

## **2.5 Summary**

From this literature review, there is a relevance of using the LWD during construction as a modulus or deflection-based QA/QC device. There have been an increasing number of studies done on the implementation of the LWD alongside non-nuclear moisture content measurement devices at a state DOT level in hopes of shifting away from the NDG. This is a similar hope for the testing done at NCAT. There has also been multiple correlations made between the LWD and other typical NDT tests to shed light on the benefits and practicality of the LWD for testing on unbound pavement foundational layers. There has been less testing utilizing the LWD as a structural assessment tool on the pavement surface post construction. All the works that were reviewed in this synthesis were taken into consideration when performing and analyzing the research done at the NCAT Test Track.

## **Chapter 3: Facilities and Testing Methodology**

### **3.1: NCAT Research Facilities**

The National Center for Asphalt Technology is comprised of two main testing facilities: the NCAT Pavement Test Track and the NCAT Main Laboratory. The main portion of this research was conducted at the NCAT Test Track, with a small portion of testing for background information completed at the NCAT main laboratory.

#### **3.1.1: NCAT Pavement Test Track**

As previously described in Chapter 1, the NCAT Pavement Test Track is a 1.7-mile closed loop test track located in Opelika, Alabama. The Test Track consists of 46 200-foot test sections with each consisting of unique pavement designs. An aerial view of the Test Track is shown in Figure 3.1. The Test Track is subjected to two-year cycles of accelerated trafficking consisting of 5 trucks weighing an average of 156,995 pounds, spread across 8 axles, running simultaneously to simulate 10 million equivalent single axle loads (ESALs) for the entire two-year test cycle. Typical nomenclature at the Test Track uses “N” to denote sections on the north tangent of the Track and “S” for the south tangent. LWD and NDG testing was completed during reconstruction of the 2024 Test Track for this investigation.



*Figure 3.1: NCAT Test Track*

During 2024 reconstruction, the six sections that were tested were N4, S1, S3, S4, S8, and S11. During trafficking, testing was done on sections N1, N2, and N7 which are part of the NCAT additive group (AG) experiment that was originally constructed in 2021. All nine sections are known as structural sections meaning they are instrumented with asphalt strain gauges (ASGs), earth pressure cells (EPCs), and temperature probe arrays. During construction, the NDG and LWD were tested on the subgrade and aggregate base materials. After construction, FWD and LWD testing was done on previously constructed test sections exhibiting a range of cracking distress.

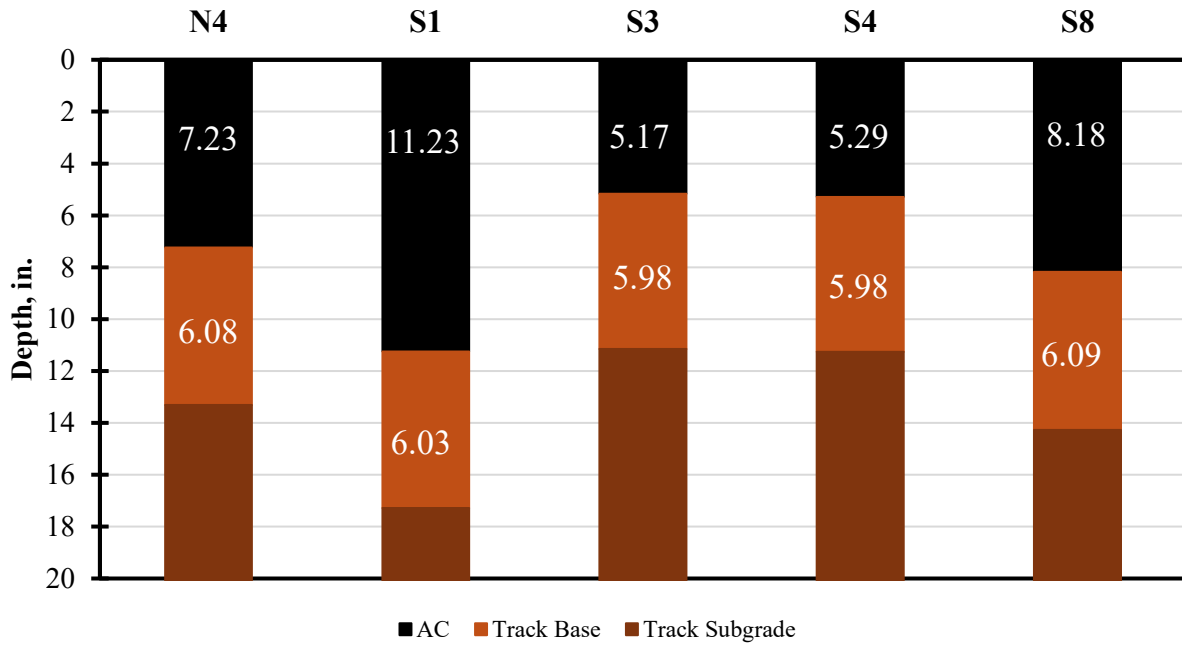
### **3.1.2: NCAT Main Laboratory**

The main NCAT Laboratory is where the smaller scale material testing was done. This testing included testing unbound materials tests for the Test Track subgrade, aggregate base, and section S11 Mississippi untreated and treated subgrade and base materials. The results from these tests are shown in the following section. The tests that were performed on the aggregate base and subgrade

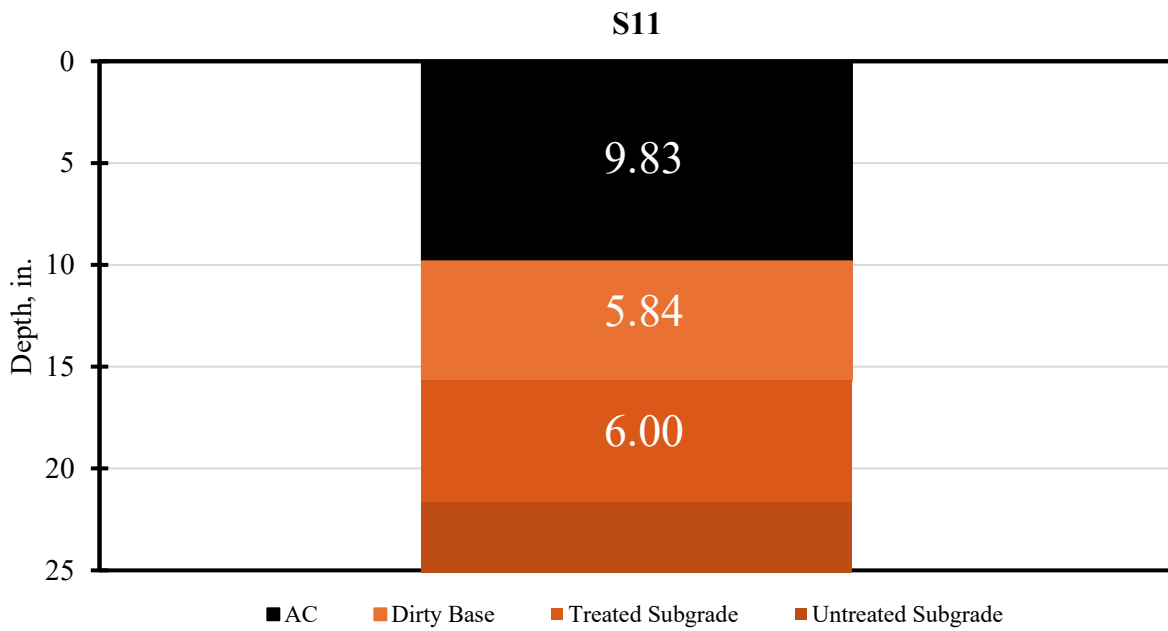
materials were Modified Proctor tests run in accordance with AASHTO T180 “Standard Method of Test for Moisture-Density Relations of Soils Using a 4.54 kg (10-lb) Rammer and a 457-mm (18-in.) Drop. The Plasticity Index was determined following AASHTO T89 “Standard Method of Test for Determining the Liquid Limit of Soils” and AASHTO T90 “Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils”.

### **3.2: Materials**

During Test Track reconstruction, the six sections that were used in this research were comprised of three primary layers like that of a typical flexible pavement structure (i.e., subgrade, aggregate base, and asphalt concrete). Five out of six of these test sections, N4, S1, S3, S4, and S8, all used a local chert subgrade, as well as a local aggregate base. Section S11 had an experimental enzyme treated subgrade and aggregate base. Both the subgrade and aggregate base for S11 were outsourced from Mississippi. Cross sections of all newly reconstructed sections are shown in Figure 3.1. In the sections that are comprised of the typical Test Track unbound materials, despite different AC surface layer thicknesses, the base layer and subgrade as built thicknesses were consistent. As previously mentioned, S11 did not follow this design. The “dirty base” referenced in Figure 3.2 for S11 refers to the Mississippi base material that was mixed with the experimental product. This was also the case for the treatment for the labeled “treated subgrade”.



(a)



(b)

Figure 3.2: Cross Sections of (a) N4, S1, S3, S4, S8 and (b) S11 Reconstructed Structural Sections

Laboratory testing had been performed on the Test Track Base Material and historical data was analyzed for other unbound materials. For the Test Track Base, a Modified Proctor test was run in accordance with AASHTO T180. Method D was run with a 6 inch mold with 5 lifts at 56 blows per lift. The results are shown in Table 3.1.

**Table 3.1: Proctor Values for Test Track Base**

<b>Parameter</b>	<b><i>Before Correction Factor</i></b>	<b><i>After Correction Factor</i></b>
<b>OMC, %</b>	5.55	4.86
<b>Max Dry Density, pcf</b>	139.5	143.4

For section S11 due to the use of outsourced material from Mississippi, standard Proctor tests were performed on the subgrade, base, and the subgrade and base mixtures that contained cement and the proprietary enzyme. Table 3.2 presents S11 results. During construction, the cement content was increased to 2% for both the subgrade and base.

**Table 3.2: S11 Proctor Values**

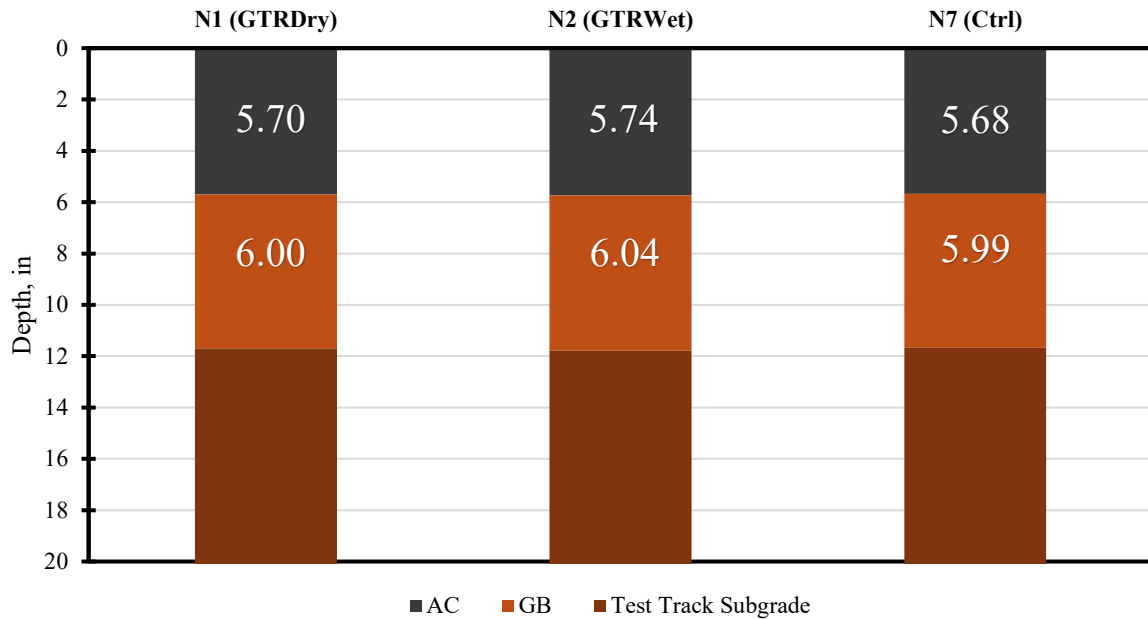
<b>Material</b>	<b><i>OMC %</i></b>	<b><i>Max Dry Density, pcf</i></b>
<b>Subgrade</b>	17.3	110.6
<b>Clay + 1% Cement + Enzyme</b>	16.8	107.0
<b>Base</b>	13.9	106.4
<b>25% Clay + 75% Base + 1% Cement + Enzyme</b>	11.8	120.9

These two tests were the only new testing performed for the 2024 reconstruction effort. Table 3.3 shows the historical data that is typically referred to during construction that was summarized from NDG field testing. Blank cells in Table 3.3 represent data that was not available.

**Table 3.3: Historical Test Track Unbound Material Data**

<b>Material</b>	<b>Average Density, pcf</b>	<b>Moisture Content, %</b>
<b>Existing Track Subgrade</b>	118.0	9.0
<b>Last 8" of Subgrade</b>	123.8	9.0
<b>Granite Base</b>	140.2	4.0
<b>5" Limerock Base (FL)</b>	112.0	13.0
<b>2021 Loachapoka Base</b>	136.1	
<b>2021 Limestone Base</b>	147.2	
<b>2024 MS Clay (Top 36")</b>		17.9
<b>2024 MS Clay (Below 36")</b>		17.9

During trafficking, the sections that were used for testing were N1, N2, and N7. These sections were asphalt pavement sections constructed in 2021 as part of a so-called Additive Group (AG) experiment. Similar to the sections evaluated during construction, these are typical flexible pavement structures, made up of the subgrade, aggregate base, and asphalt layers. The main difference is the additive included in the asphalt mixture. Section N1 is a Ground Tire Rubber (GTR) dry mix, N2 is a GTR wet mix, and N7 is the control mix. More information on the AG experiment can be found in *“Laboratory and Field Characterization of Additive-Modified Asphalt Concrete Mixtures”* [Kmetz 2023]. Figure 3.3 shows the as-built cross profiles of these three sections. The main purpose of using them in this investigation is that they provided three different materials exhibiting three different amounts of cracking. The type of asphalt additive was purely incidental to this investigation.



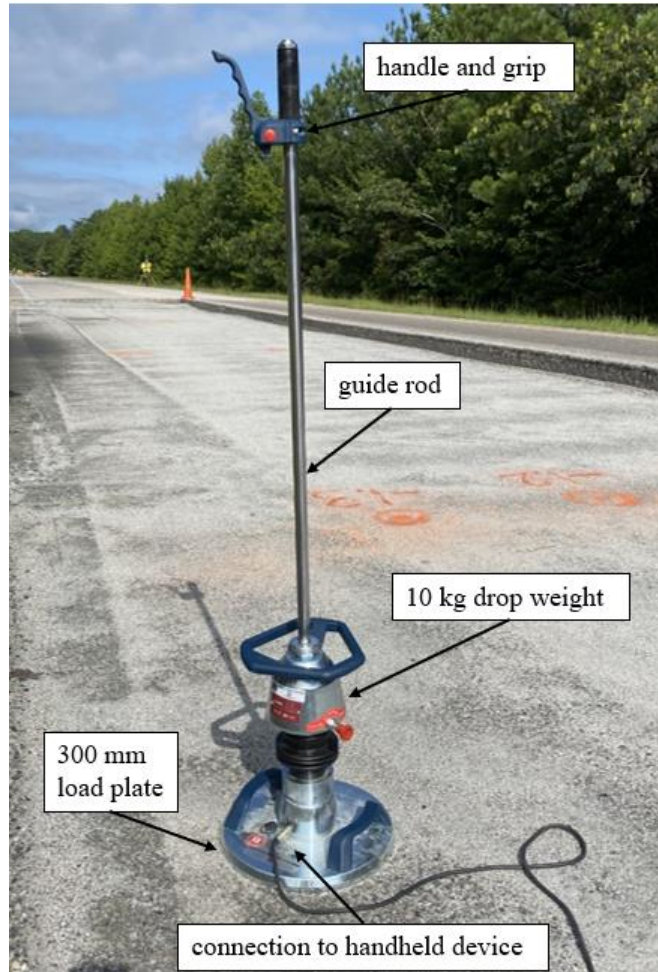
*Figure 3.3: Cross Sections of AG Sections*

### 3.3: Equipment Overview

The primary experimental testing device for this investigation was the Light Weight Deflectometer. Other supporting equipment that was used included the Nuclear Density Gauge (NDG) and the Falling Weight Deflectometer (FWD). Each is described below.

#### 3.3.1: Light Weight Deflectometer

The Zorn ZFG 3.1 Light Weight Deflectometer was used in this research and is pictured in Figure 3.4. The testing protocol followed ASTM E2835-21 “Standard Test Method for Measuring Deflections Using a Portable Impulse Plate Load Test Device”. The four main components of this device are the guide rod, the falling weight, the load plate, and the handheld device. A more detailed schematic of the device was shown in Figure 2.1 in Chapter 2.



**Figure 3.4: Schematic of LWD at NCAT Test Track**

For testing, the standard 300 mm load plate, 10 kg drop weight, a 17 millisecond impact loading duration and 27.5 inch drop height were used for all testing done during construction and during trafficking. Using consistent settings allowed for more correlations between data sets without confounding the results with variable plate sizing or drop heights. Each LWD test consisted of six drops. The first three drops were referred to as the seating drops followed by three recorded drops. The last three recorded drops were averaged to determine the modulus and deflection at the test location.

### 3.3.2: Nuclear Density Gauge

The NDG used at the Test Track was the Troxler 3430 as shown in Figure 3.5. The testing protocol followed ASTM D6938 “Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)”.



*Figure 3.5: NDG at NCAT Test Track*

During construction the NDG is used in direct mode for testing the subgrade soil and the granular base. Typically, for testing with the NDG at the track, the stack is struck into the ground and removed so testing can be done at 6 inches into the unbound material. The readings that were obtained from the NDG were the moisture content and the wet and dry density values of the material. The current practice is to use the moisture content values and density to measure the compaction of the material compared to the Proctor test results.

### 3.3.3: Pavement Instrumentation

Asphalt strain gauges (ASG) and earth pressure cells (EPC) were placed within the pavement test sections N4, S1, S3, S4, S8, and S11 during 2024 reconstruction. Sections N1, N2, and N7 were previously instrumented and constructed during the 2021 Test Track reconstruction. For the aforementioned sections, there were two EPCs embedded in each section, except for in section S11 due to the use of base stabilization products. The EPCs are located one at the top of the subgrade and one at the top of the base. The strain gauge array is located at the bottom of the asphalt layer. The overall gauge array is shown in Figure 3.6 on top of the aggregate base before placing the asphalt layer while Figure 3.7 shows a close up of an EPC prior to covering with base material in S1.



*Figure 3.6: Section Gauge Array on Top of Aggregate Base*



***Figure 3.7: EPC Placement Before Paving***

During construction, LWD testing was performed on the EPC located in the subgrade and aggregate base. The data received from the EPC was recorded, and showed the consistency of the LWD testing drops which is presented in Chapter 4.

### **3.3.4: Falling Weight Deflectometer**

The FWD used at the Test Track for two out of three of the test dates for this investigation was the Dynatest 8002 hydraulic FWD. The other FWD that was used for the last test date on the asphalt surface was the Dynatest 8012 FFWD (FastFWD). Both FWDs are shown in Figure 3.8. ASTM D4694 “Standard Test Method for Deflections with a Falling Weight-Type Impulse Load Device” was followed.



(a)



(b)

**Figure 3.8: (a) Dynatest 8002 FWD and Dynatest 8012 FFWD at NCAT Test Track**

Both FWDs were equipped with 9 sensors for testing. Table 3.4 identifies the sensor offset locations with sensor 1 being the load center. This is a key difference from the LWD that only has one sensor at the load center.

**Table 3.4: FWD Sensor Offset Locations**

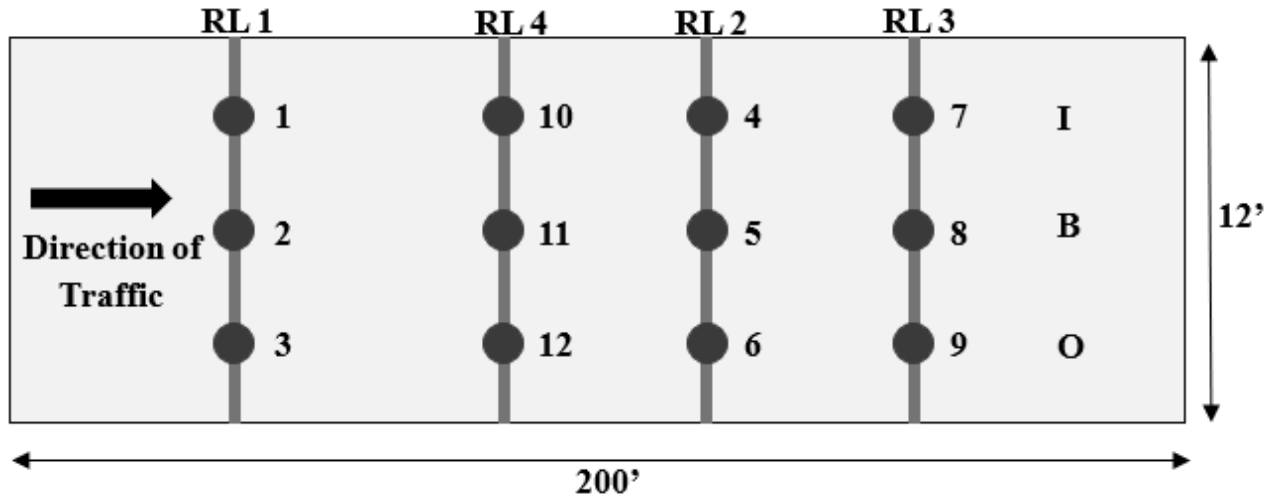
<b><i>Sensor Number</i></b>	<b><i>Offset Distance from Center of Loading, in.</i></b>
<b><i>1</i></b>	0
<b><i>2</i></b>	8
<b><i>3</i></b>	12
<b><i>4</i></b>	18
<b><i>5</i></b>	24
<b><i>6</i></b>	36
<b><i>7</i></b>	48
<b><i>8</i></b>	60
<b><i>9</i></b>	72

Sensor 1, directly under the load plate, was used for LWD correlations in the following chapters. The modulus of the pavement was determined from backcalculation in EVERCALC 5.0 using the deflection recorded from these sensors. The load plate used for the FWD had a radius of 5.91 inches. Targeted loadings of 6, 9, and 12 kips were performed with three replicates at each load level.

### **3.4: Testing Methodology**

Testing with the LWD was performed on the unbound materials in sections N4, S1, S3, S4, S8, and S11. There were 12 main testing locations per section. The locations of testing for the LWD during construction followed the preestablished testing pattern for the FWD postconstruction. The testing pattern consisted of testing done on the four predetermined random line (RL) locations on

the inside wheelpath (I), between the wheelpath (B), and the outside wheelpath (O). A schematic of this testing routine is shown in Figure 3.9.



**Figure 3.9: LWD Testing Schematic During Construction**

The RL locations were predetermined by Test Track engineers. RL 4 was located in the middle of the gauge array. The naming convention for data collection was the section – test location number (e.g., N1-1). Additional test locations were added to the base layer once EPC pressure plates were installed in the sections. This was labeled as test location 13. In section S11, two additional testing locations were installed due to the extra pressure plate installed in this section. These were labeled test locations 13 and 14. Results from LWD testing on the EPCs is presented in Chapter 4.

To perform the testing, a team of at least three researchers was used. This was found to be the most efficient number to perform fast and effective data collection. Figure 3.10 shows the team testing on unbound material during construction and on the asphalt pavement surface.



**(a)**

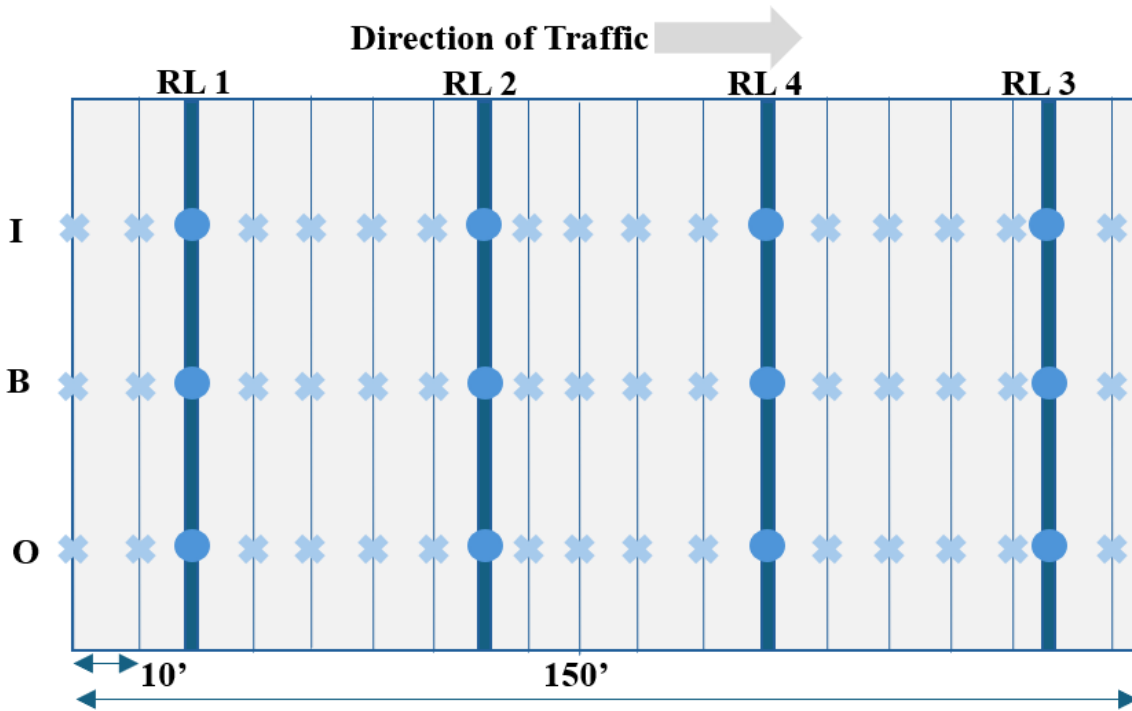


**(b)**

***Figure 3.10: Research Team Performing LWD Testing on (a) Unbound Material and (b) Asphalt Surface***

One person performed the dropping of the load, one person was in charge of moving the load plate, and the final team member controlled the handheld device and recorded data. An optional fourth team member could record data instead of one person controlling the device and recording data. Once the data was collected for the day, all data was uploaded to an Excel file for further analysis. LWD testing of the base was done alongside the testing of the NDG with Test Track personnel operating the NDG. NDG testing was only done on the RL lines in the inside and outside wheelpaths. Once the pressure plates were installed around the RL 4 locations, NDG testing around that area was avoided to protect the installed sensors.

Post construction, the FWD testing locations are the same as that of the LWD on the unbound material. This is historically how the FWD test is run. However, the LWD testing was increased to run at those 12 historical locations as well as 10 foot increments starting at the beginning of the section. This testing was done on N1, N2, and N7. In the following schematic shown in Figure 3.11, FWD and LWD testing is denoted with a circle on an RL, whereas solely LWD testing is denoted with an “X” in each offset.



*Figure 3.11: Schematic of LWD Test Locations During Traffic*

The LWD testing was performed following the FWD in each offset. This allowed for correlations between the LWD and FWD to be made at the same location without the interference of temperature differences. The LWD was run throughout the entire section in 10 foot increments to compare the modulus to known distresses throughout the sections. Temperature was recorded throughout testing using the temperature probes embedded in the pavement layers to understand the temperature effect on the data.

Results and discussion of the relationship between LWD and NDG data during construction are presented in Chapter 4. Similarly, results and discussion of the relationship between LWD and FWD data on the final pavement surfaces are shown in Chapter 5.

### **3.5 Summary**

The main testing done for this research was done at the NCAT Pavement Test Track where the LWD, NDG, and FWD testing was conducted. The NCAT Main Lab was used for material testing of the aggregate base and subgrade materials. Historical Test Track data was included as a reference for testing. The LWD was the main testing device used for testing on both unbound materials and asphalt pavement. Chapter 4 details the results from the LWD and NDG testing during construction. Chapter 5 presents the results from testing the LWD and FWD on the asphalt pavement layer.

## **Chapter 4: Field Testing During Construction**

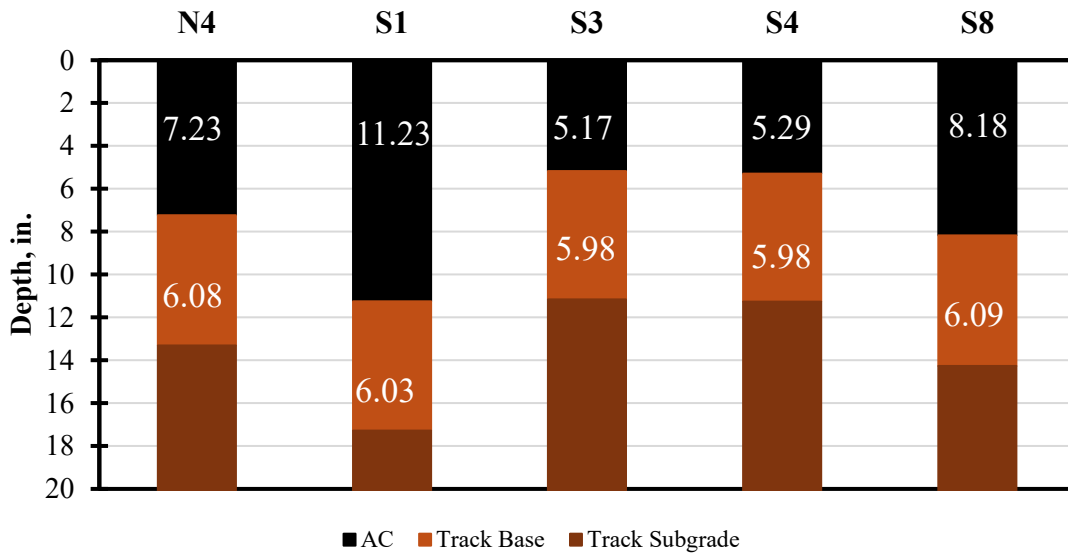
As previously described, during the 2024 reconstruction of the Test Track, the LWD was utilized on six sections that were fully rebuilt. These six sections were N4, S1, S3, S4, S8, and S11. These six sections are known as structural sections, meaning they are fully reconstructed down to the subgrade as well as instrumented with EPCs and ASGs. Due to the in depth reconstruction, QC is needed to be performed regularly on the lifts of subgrade and base as they were placed and compacted. During testing, section S1 was not tested as frequently with the LWD as the other sections due to constraints at the Test Track. Therefore, the following data analysis in this chapter does not include significant S1 data. Similarly, section S11 is not compared to the other four sections, N4, S3, S4, and S8, due to the difference in the type of unbound material used for both the subgrade and aggregate base during construction.

At the Test Track the NDG is currently the main QC device for the base and subgrade construction. The Test Track follows ALDOT specifications and requirements of at least 95% compaction and within  $\pm 2\%$  of the optimum moisture content (OMC) determined through Proctor testing. The NDG was tested simultaneously on the aggregate base material in sections N4, S3, S4, S8, and S11 due to its primary role as the QC device during reconstruction at the Test Track. This allowed for comparisons to be developed between the results from the LWD and the NDG. This chapter presents the results from the LWD on the subgrade and base materials during construction, comparisons between the LWD and NDG, and data from testing on the embedded EPCs.

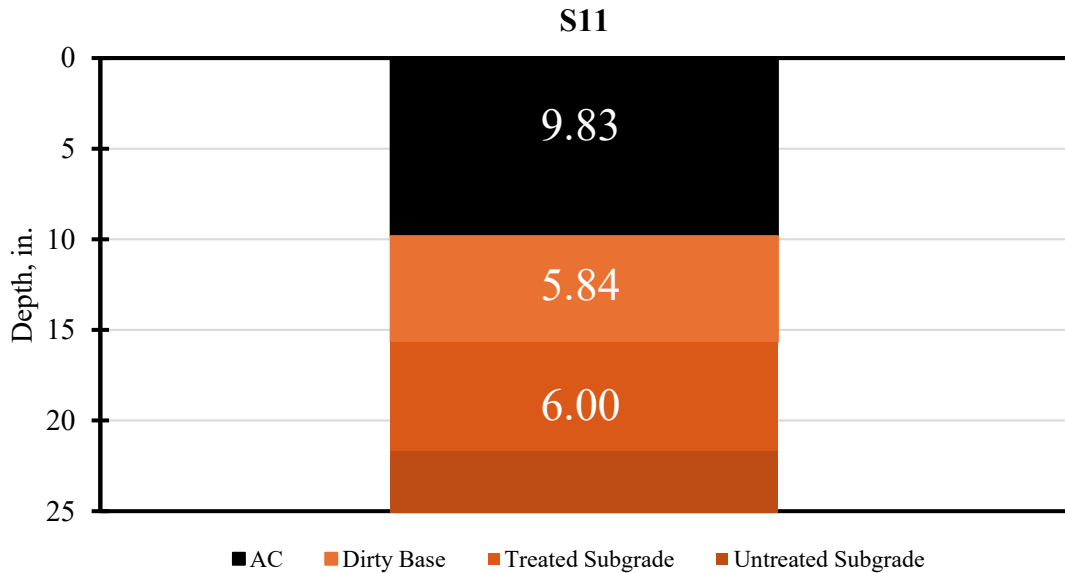
### 4.1: In-Place Properties

Each of the six tested sections had unique structural designs. The individual cross-sectional diagrams of each section and their materials were previously shown in Figure 3.1 in Chapter 3.

For convenience, they are shown again in Figure 4.1.



(a)



(b)

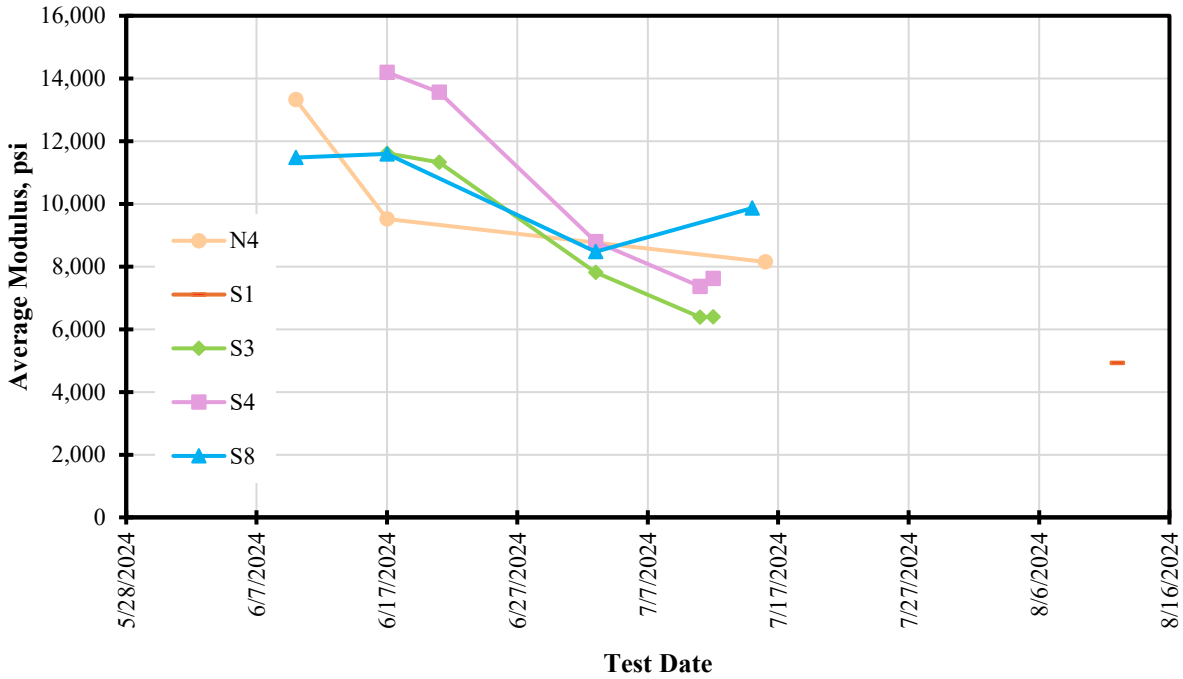
**Figure 4.1: Cross Sections of 2024 Structural Sections**

Sections N4, S1, S3, S4, and S8 were comprised of the same local chert subgrade and local aggregate base. Despite the ultimate surface layer thickness, the base layer had an average thickness of 6.00 inches for all six sections. The subgrade and base of S11 were comprised of sourced subgrade and base materials from Mississippi and treated with proprietary enzyme stabilization treatments to simulate conditions and properties more similar to how their product is used in practice. Testing was performed on both the treated and untreated base and subgrades of S11. Section S11 also received a tack coat between the base and the asphalt concrete layer. LWD testing was performed on the tack coated base. The soil properties and characteristics were previously discussed in Chapter 3.

#### **4.2: LWD Data**

During construction, a total of 244 LWD data points were collected on the subgrade, 372 LWD data points on the base, and 129 LWD data points on all phases of S11 treatments. Testing on the subgrade with the LWD started in June of 2024, when the sections were excavated down to the subgrade and preparing to be rebuilt for the 2024 test cycle. The subgrade was tested around four times per section over the next month. The exception to this was section S1, which was only tested once in August. This was due to constraints relating to this particular section during reconstruction. Shown in Figure 4.2 is the subgrade between June 10<sup>th</sup>, 2024, and August 12<sup>th</sup>, 2024. The last data point for each data set shows the last day it was tested before the next foundational layer was added, this is the same for the base layer shown in Figure 4.2. From this plot it is seen that there was a significant decrease in the subgrade modulus between the middle of June and the beginning of July. The subgrade modulus was beginning to increase in some sections, such as S8, towards

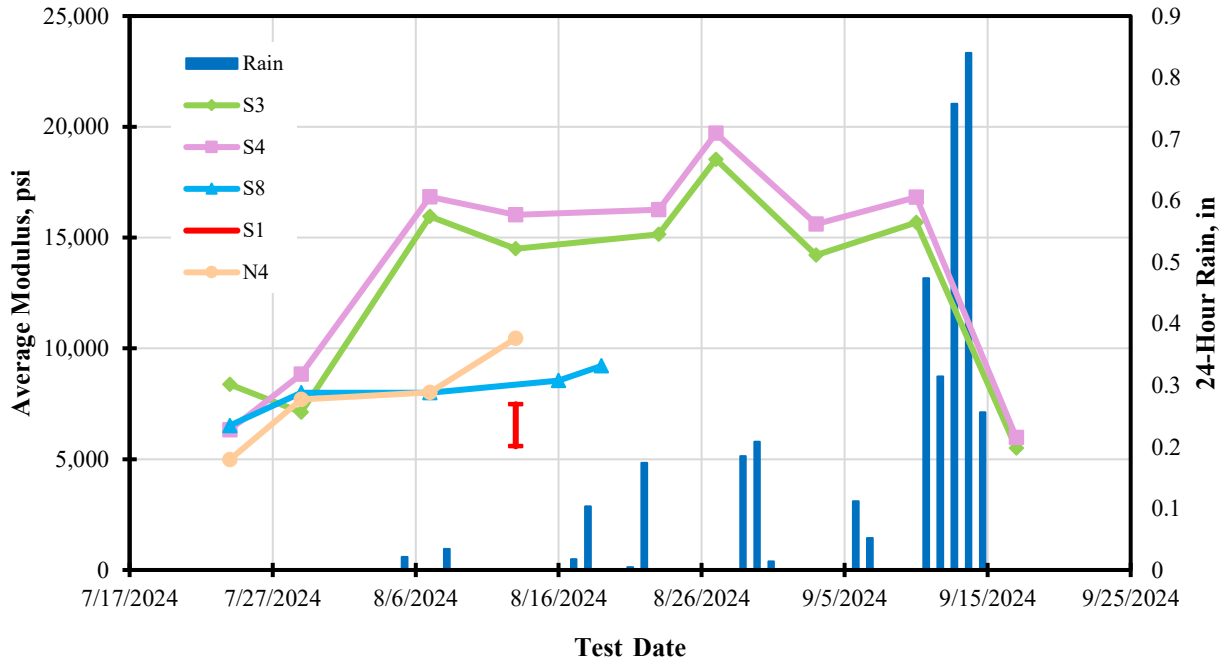
the start of construction of the next foundational layer, however sections S3, S4, and N4 did not experience a large increase and experienced the lowest modulus values.



**Figure 4.2: Test Track Subgrade Modulus Over Time**

Testing with the LWD on the Track Base started in July of 2024 and concluded in the middle of September of 2024. During this time frame, sections S3 and S4 had the most testing performed due to the longer construction timeframe. The amount of testing per section correlated to the pace of construction as well as when other layers such as CCPR or HMA were placed. It was noticed when analyzing the data that the modulus of the Track Base was affected by the weather over the period of construction. During the summer in Alabama, it is typical to experience afternoon rainfall, which can affect construction efforts. Figure 4.3 shows the Track Base LWD modulus plotted over the time period of the construction of base layer. Also included in Figure 4.3 is the available weather station data from the Test Track. The weather station data provided the

amount of rain that occurred on each day within the testing period. In Figure 4.3, there is only one test date for section S1 where it was tested twice on the same day. The second test increased when the section was given time to dry throughout the day.

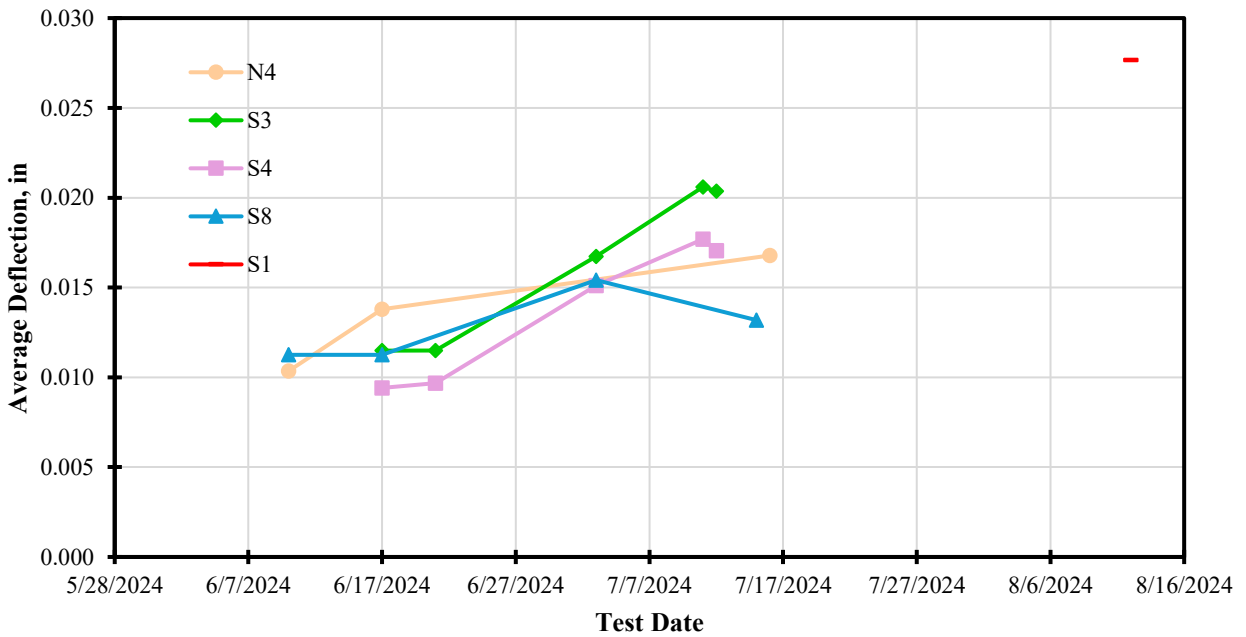


**Figure 4.3: Test Track Base Modulus Over Time**

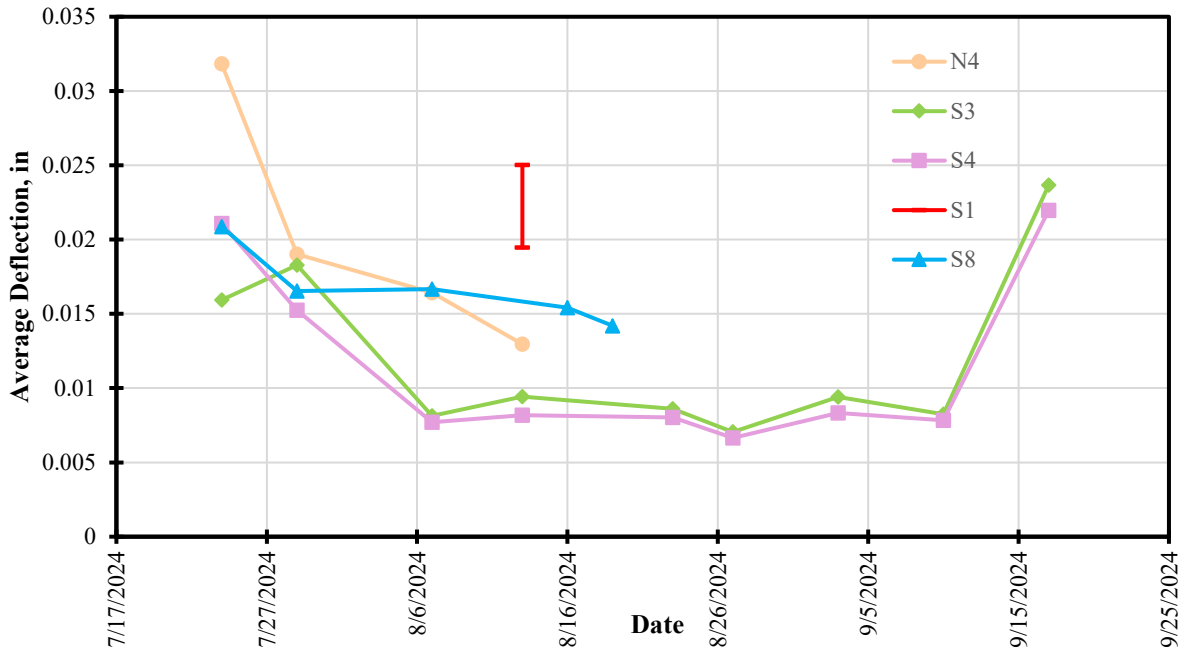
Figure 4.3 prominently illustrates the effect the rain had on the modulus in each section. There were small rain events sporadically throughout the test period, with the most significant rain event having occurred in the middle of September, near the end of testing on the base. The typical trend relating to the rain was that the testing done after rain events would result in a drop in modulus from previous measurements if the previous test date had been done under no or limited rain conditions. It can be seen that when there was no rain in between test dates, the base had time to dry out, allowing the modulus to increase and stiffen. This is an important understanding due to the moisture contents in the foundational layers and the impact it can have on the variability of section performance. Understandably, the larger rain event caused the most significant drop in the

modulus of sections S3 and S4, as they were the only sections unpaved allowing the base to be subjected to the weather. Sections S3 and S4 performed very similarly throughout the entire testing period. This was likely due to the fact they are situated adjacent to each other and the base constructed continuously as if they were one long section.

The deflection of the unbound materials was also recorded at the same time as the modulus with the LWD. Figures 4.4 and 4.5 show the deflection values from the subgrade and base layers, respectively, over the same dates as the modulus values. The same comment about having one test day for section S1 for the Track Base is relevant to Figure 4.5. The trend in each of these graphs is inverse of the modulus graphs previously shown in Figures 4.2 and 4.3. This relationship was expected to do the nature of determining the modulus from the deflection value. When there is a larger deflection, it shows that the material does not exhibit a high strength or stiffness which corresponds to the modulus values being lower.



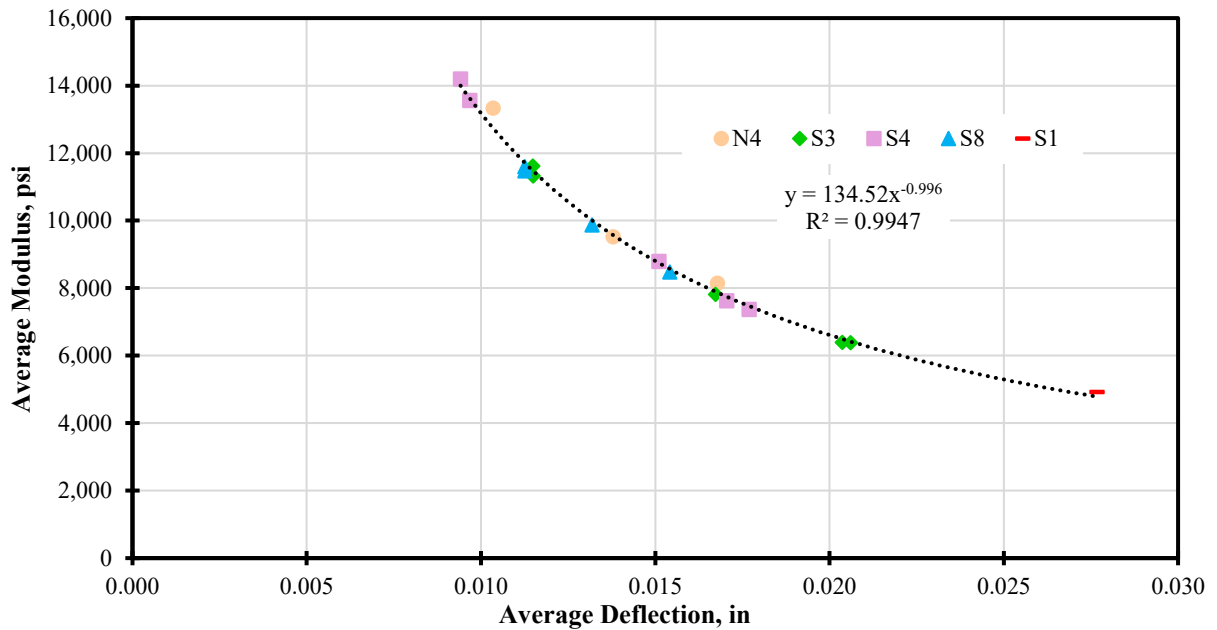
**Figure 4.4: Test Track Subgrade Deflection Over Time**



**Figure 4.5: Test Track Base Deflection Over Time**

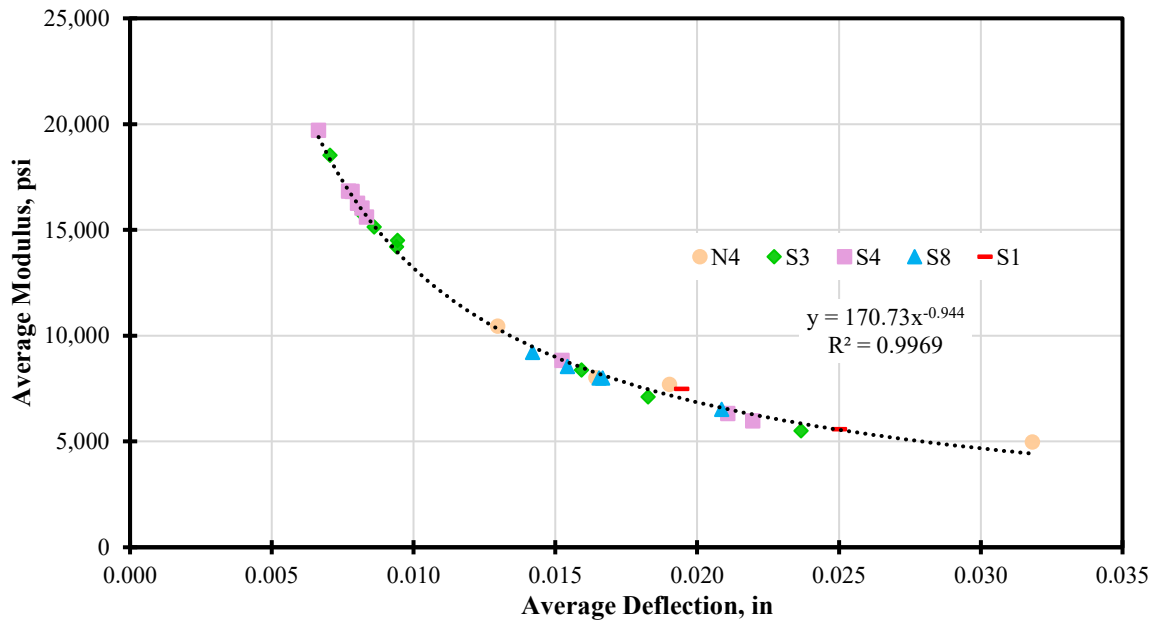
Shown in Figures 4.6 and 4.7 are relationships between the deflection and modulus of the subgrade and base, respectively. The subgrade comparison has a highly significant  $R^2$  value of 0.9947 with a power trend line.

The equation, previously mentioned in Chapter 2, that the Zorn LWD uses to get from the recorded deflection to the resulting modulus values indicates that when the deflection is higher, the modulus will be lower, just by nature of the equation. When the material is not as stiff, in this case due to moisture from environmental conditions, the deflection will increase, and the modulus will therefore decrease simultaneously. This relationship with moisture and both deflection and modulus is the foundation for using the LWD as a QC device during construction at the Test Track. Both modulus and deflection could be viable options for QC with further studies on the subgrade and base of the Test Track. Further relationships between the deflection, modulus, and moisture are discussed in the next sections.



**Figure 4.6: Average Subgrade Deflection vs Average Subgrade Modulus**

Figure 4.7 shows the relationship between the base modulus and the deflection values. A strong correlation is shown between them with a  $R^2$  value of 0.9969 for a power function trendline. Again, this was to be expected since deflection is used to compute modulus within the LWD device.



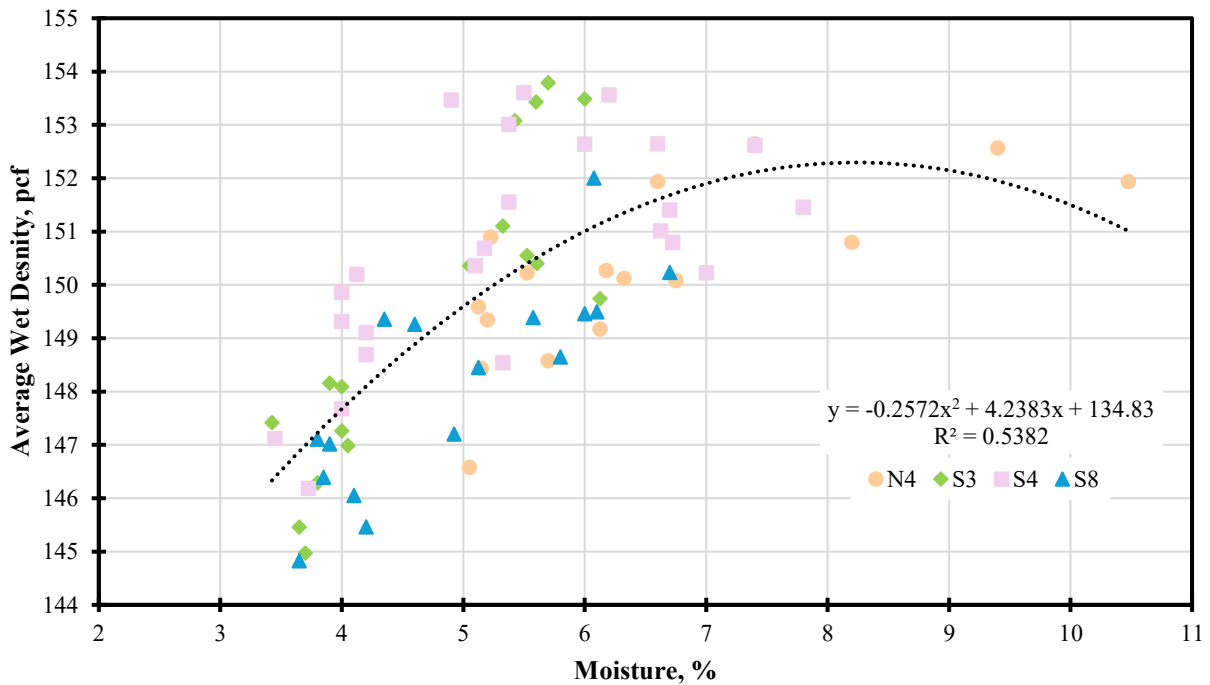
**Figure 4.7: Average Base Deflection vs Average Base Modulus**

This relationship is important in the desire to use the LWD as a QC device for the foundational layers of the pavement that utilizes characteristics that can ultimately be used in pavement design. Specifically, using the modulus of the subgrade and base can be important when designing the pavement, so by setting guidelines based on the deflection of the material it allows a good correlation between the modulus values that were used in design.

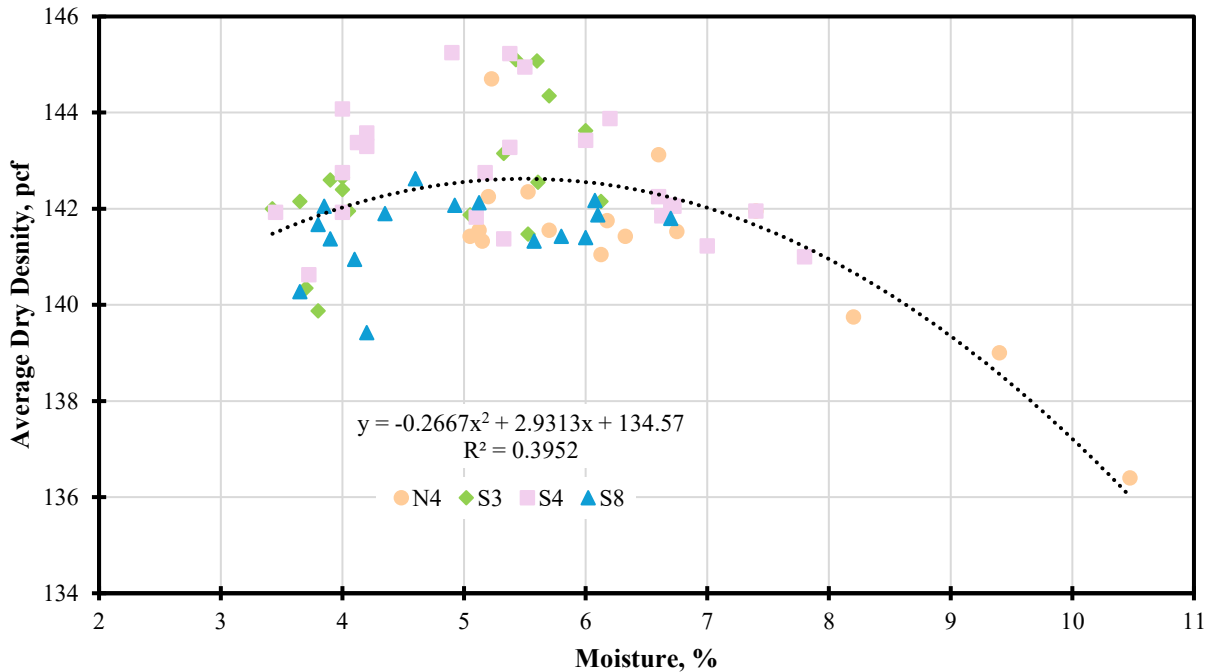
### 4.3: Nuclear Density Gauge Data

The following NDG data was obtained from the NCAT Test Track. The NDG run by Test Track technicians during construction as the main QC standard. Due to the QC standard, the NDG was performed on every lift of fill for the subgrade and base whereas the LWD was only performed on the final lifts of the Base to the LWD being solely used for research purposes at the time of construction. The NDG data from the final lifts is what has been compared to the LWD data. A comparison between the moisture and the density values were plotted to acknowledge the

relationship within the NDG measurements. Figure 4.8 shows the moisture and the wet density, and Figure 4.9 shows dry density. These graphs show a relationship similar to that produced by Proctor testing. The highest modulus values relate to the optimum moisture content (OMC). Previously mentioned in Chapter 3, the OMC for the base is around 4.86%. From both graphs, the highest modulus values occur around 5.5% to 6%. These values are within the  $\pm 2\%$  range given for QC requirements for moisture in the field. In Figure 4.8, the  $R^2$  value (0.5382) from the polynomial fitted trendline falls within the moderate relationship between the wet density and the moisture. The dry density had a less impactful effect as shown by the  $R^2$  value (0.3952), also from a polynomial trendline. This signifies that there may be a stronger effect from the moisture on the wet density values rather than dry density from NDG testing.



**Figure 4.8: Average Moisture vs Average Wet Density of Base Material**



**Figure 4.9: Average Moisture vs Average Dry Density of Base Material**

When testing with the NDG, the Proctor value for maximum dry density is input into the device and is used to help determine the relative compaction of the material being tested. This allowed for the range of tested NDG values to be within about a 10 pcf range for both dry and wet density values.

#### 4.4: LWD and NDG Correlations

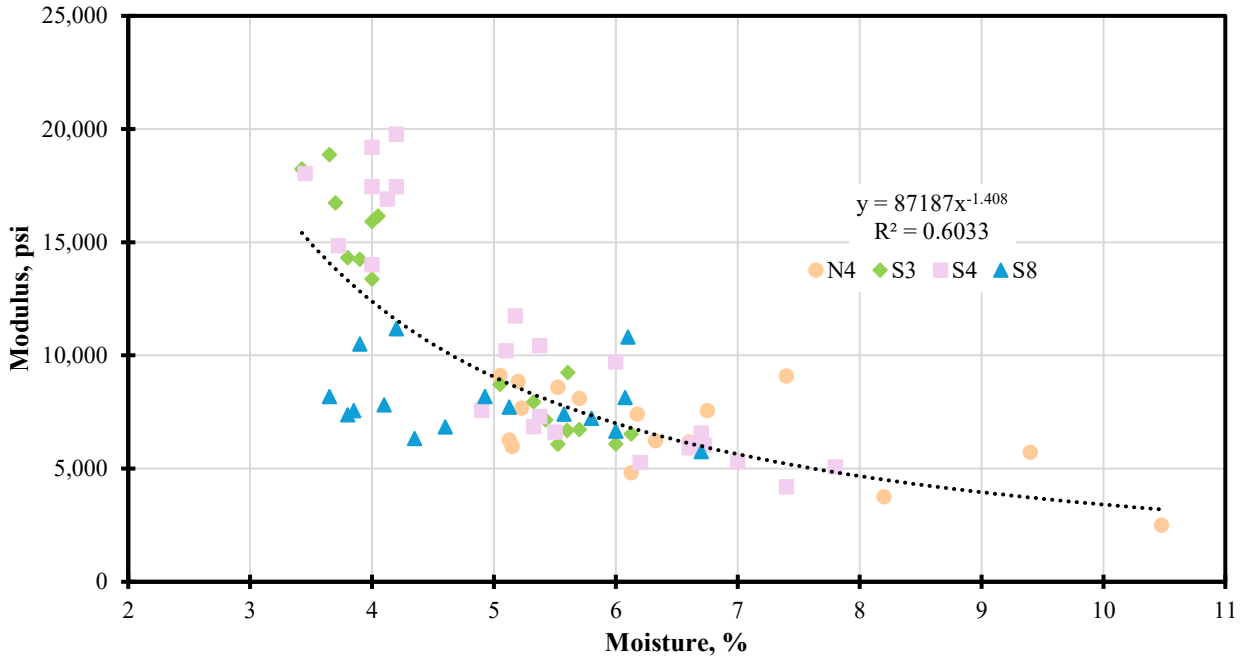
Testing of the subgrade with the NDG was not performed in sequence with the LWD; therefore, subgrade data is excluded from further correlation analysis in this section. Similarly to the previous sections in this chapter, there are no data to include about section S1 due to limitations for testing and section S11 was not compared to the other four sections due to the difference in treatments and composition. However, sections N4, S3, S4, and S8 provided sufficient data for the base layer from both the LWD and the NDG. There was a total of 73 data points that were used that had both

LWD and NDG data. This allowed for the strongest analysis to be evaluated between the two devices. The comparisons that were made between the two devices were:

- NDG Moisture versus LWD Modulus,
- NDG Wet Density versus LWD Modulus,
- NDG Dry Density versus LWD Modulus,
- NDG Moisture versus LWD Deflection,
- NDG Wet Density versus LWD Deflection, and
- NDG Dry Density versus LWD Deflection.

All data collected was analyzed within these categories and further analysis was then done based on specific locations within each section. It should also be noted that the NDG was only tested on the I and O wheelpath locations on the four RL locations due to historical testing patterns. The LWD was also tested at these locations, therefore the comparisons with the NDG from N4, S3, S4, and S8 exclude between the wheelpaths.

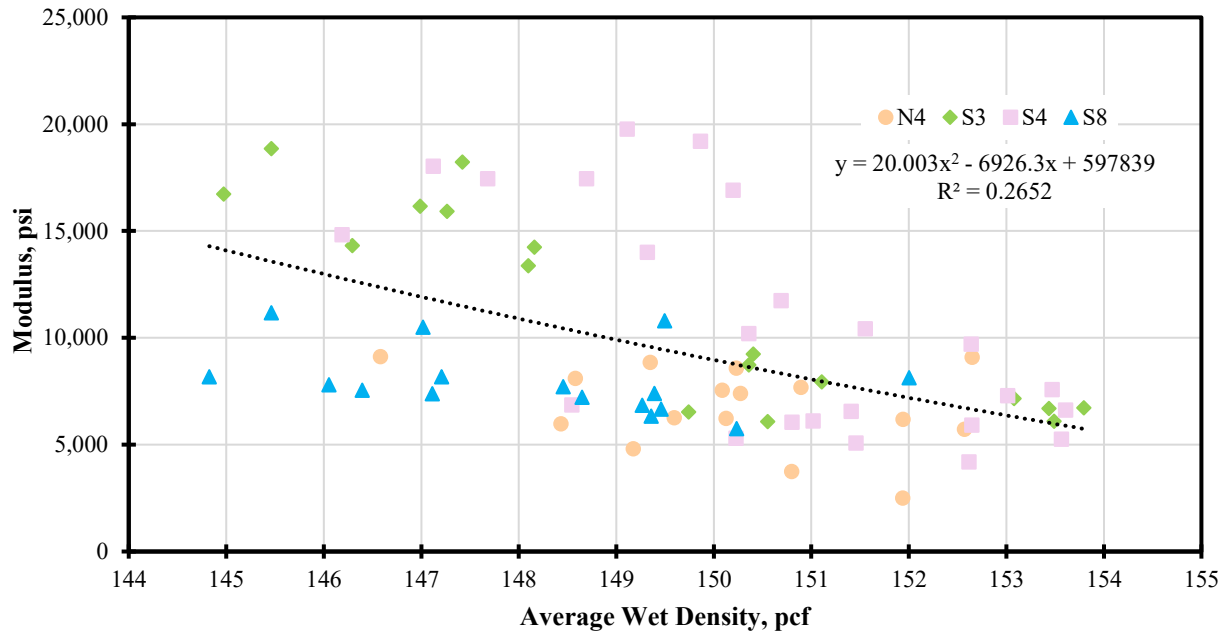
Figure 4.10 depicts the moisture content versus the modulus for the four sections. A power function trendline was fitted against the data with a resulting  $R^2$  value of 0.6033. This  $R^2$  value exhibits a moderately strong relationship between the two variables. Which ultimately demonstrates that there is a correlation between the amount of moisture in a material and the corresponding modulus. Since all sections in this analysis are comprised of the same material, it is understandable that each section follows the similar trends and patterns. These results show that when the moisture is increased in this material, the modulus will consequentially decrease. This is expected due to the relationship between increased moisture past the optimum moisture content reducing the strength of soil.



**Figure 4.10: Moisture vs Modulus**

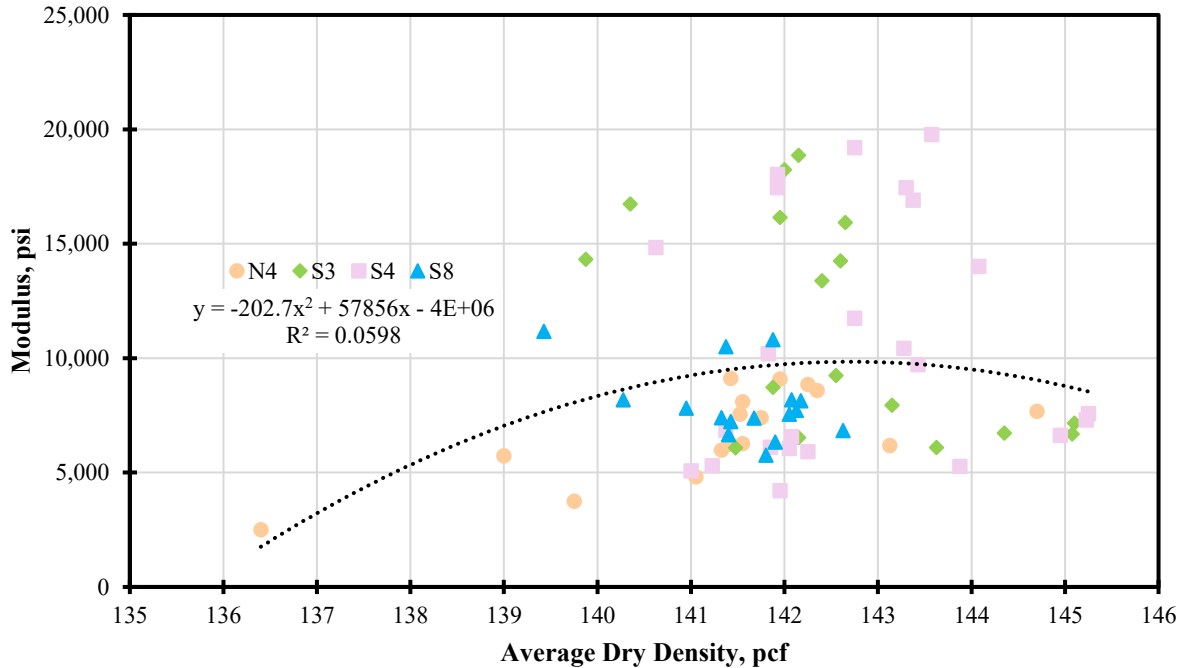
As seen in Figure 4.10, section N4 had the lowest modulus values, while also having the highest moisture content. Sections S3, S4, and S8, all on the south tangent, had more similar ranges for moisture and modulus values. The highest modulus tended to occur around a 4% moisture content, which corresponds with the OMC found during the Proctor test.

Figure 4.11 shows the wet density plotted against the modulus. A polynomial trendline was fit to these data. This produced an  $R^2$  value of 0.2652. This is notably less than that of the previously mentioned  $R^2$  value for moisture versus modulus. This graph shows the variability in the relationship between the wet density and the modulus of this material. No strong correlation or relationship can be established between these two measured properties.



**Figure 4.11: Wet Density vs Modulus**

In Figure 4.11, similar to the moisture readings, section N4 had the higher wet density. This is understandable due to the greater amount of moisture in the section compared to the other three sections. Individually, none of the sections in this graph seem to follow a cohesive trend with their data. Similar to the wet density, the dry density was compared to the modulus and had even less of a correlation with an  $R^2$  value of 0.0598 from a second order polynomial best fit trendline. This is shown in Figure 4.12. The relationships between the modulus and both wet and dry density are interesting since with the NDG, density is half of the QC/QA parameters. As mentioned before in the literature review in Chapter 2, it is typical for construction of foundational layers to be done in ways that may not be the most beneficial to the actual strength of the material, but to pass the minimum requirements for moisture and density testing. Without having a strong relationship with the modulus values, which indicate the stiffness and are a significant material property, the density QC may be the reason for underlying failures of pavement structures.

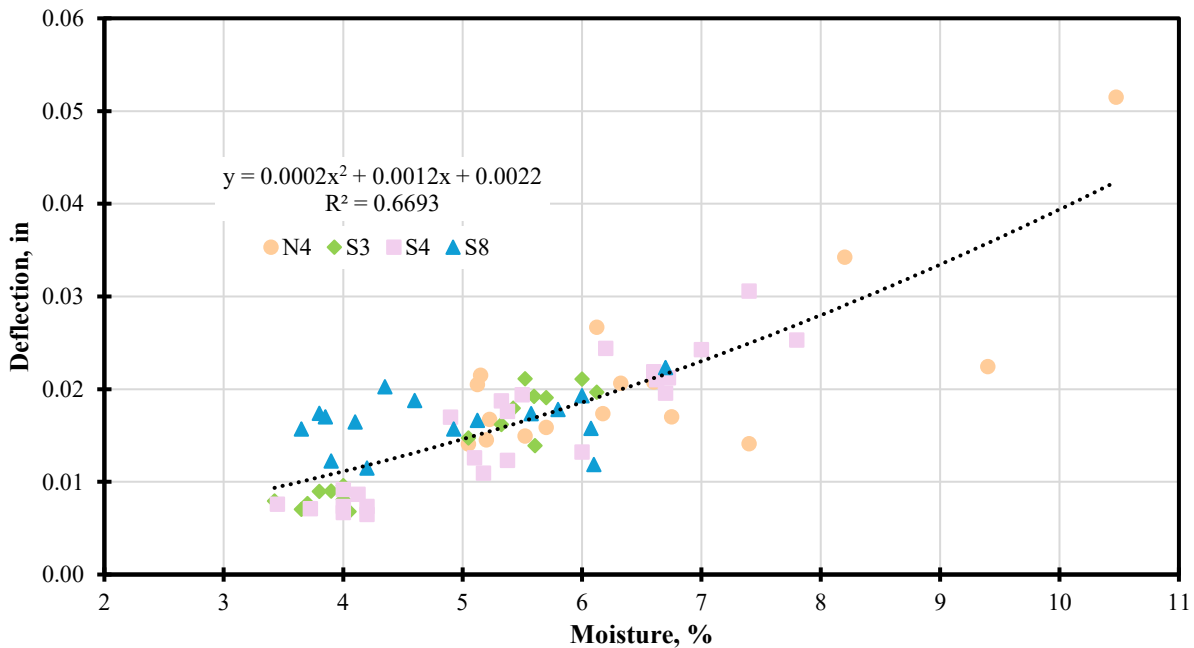


**Figure 4.12: Dry Density vs Modulus**

In Figure 4.12, the data seems to be more clustered around the dry density range 141 pcf to 143 pcf. This relates to the values obtained by the Proctor before and after correction. Before correction the maximum dry density was 139.5 pcf and after correction it was 143.4 pcf. Similar to the previous graph, each section does not seem to follow a specific trend. Another interesting point that is presented in this graph is that the density values are all within a 10 pcf range, but the range of the modulus is about 15,000 psi. This can be seen prominently in section S3 data that has only two data points that are at around 142 pcf, but have vastly different modulus values, one being almost 6,000 psi and the other being almost 19,000 psi. This is about a 13,000 psi difference, which is a significant difference in the stiffness of the base material, but only going by the density value, the materials could be considered the same compaction.

The next three figures are the deflection produced by the LWD plotted against the moisture, wet density, and dry density as well. In all of the following figures, similarly to previous discussion,

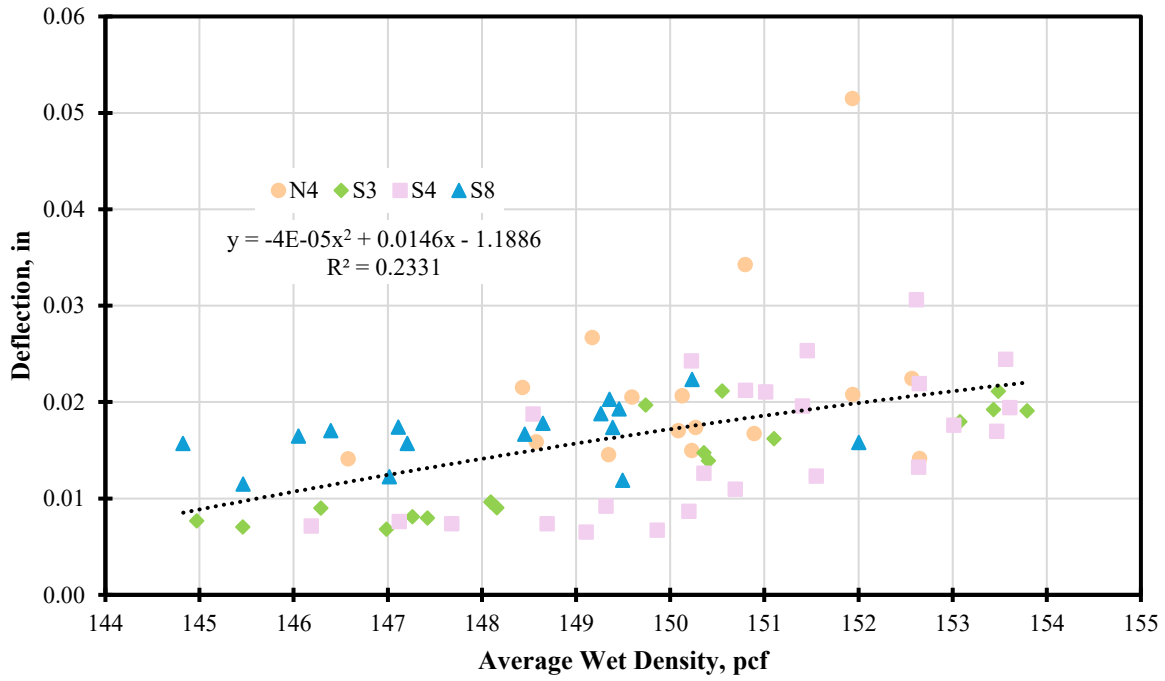
the deflection follows the opposite trend that the modulus followed. This relationship, as previously mentioned, is to be expected. In Figure 4.13, the deflection versus the moisture has a slightly greater  $R^2$  value than that of the modulus versus moisture with a value of 0.6693. The best fit trendline for the deflection against the moisture was a second order polynomial opposed to the power function for the modulus against the moisture. Figure 4.13 shows how the increase in moisture in the material will create a higher deflection. Knowing this is beneficial in understanding the changes in stiffness due to environmental conditions such as rain during construction. This relationship can also help create an adequate range for using the LWD as a QC validation tool.



**Figure 4.13: Moisture vs Deflection**

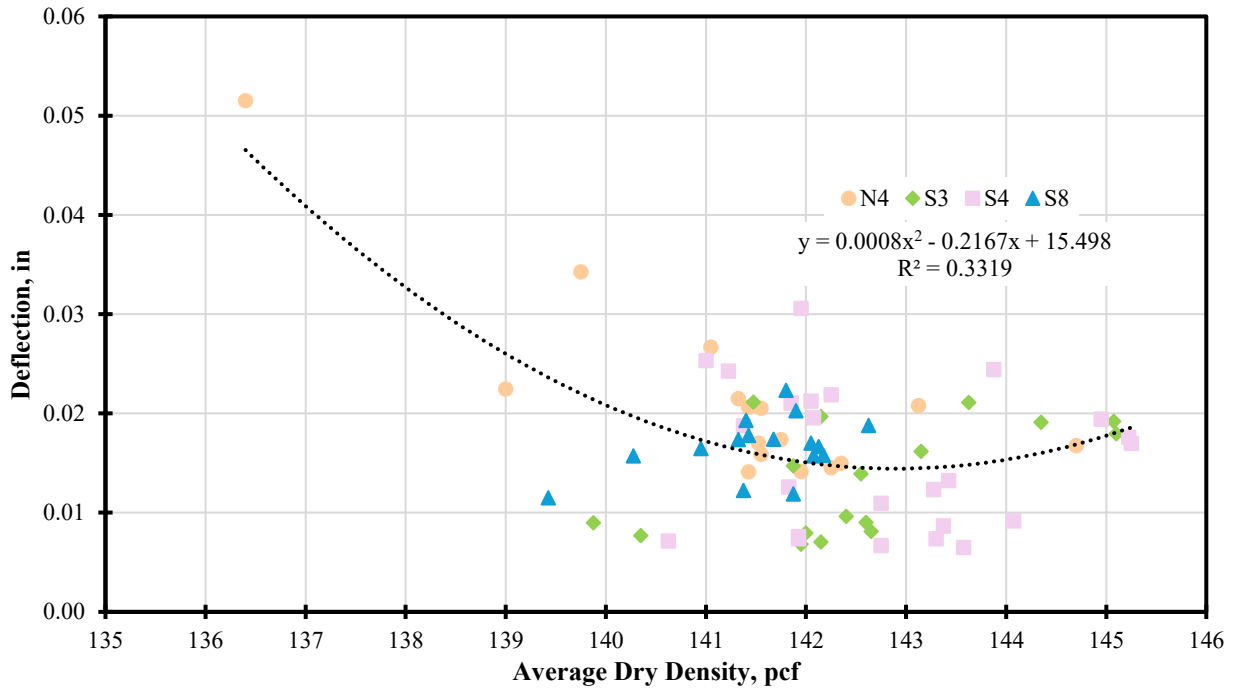
Figure 4.14 shows the LWD deflection versus the wet density. Similarly to the modulus versus wet density, there is not a strong relationship between the two with a  $R^2$  value of 0.2331. Both trendlines for modulus and deflection compared to the wet and dry density were second order

polynomial functions. The modulus relationship had a slightly higher  $R^2$  value than that of the deflection. However, neither relationship had a strong correlation to the wet density.



**Figure 4.14: Wet Density vs Deflection**

Reiterating what was discussed when comparing the modulus to the wet density, there is no distinguishable trend to establish between the wet density and the deflection values. However, when comparing the deflection to the measured dry density, there is a slightly higher correlation. This is shown in Figure 4.15. When a second order polynomial trendline was used, the  $R^2$  value resulted in 0.3319, which is higher than the deflection compared to the wet density or the modulus compared to either density values.



**Figure 4.15: Dry Density vs Deflection**

#### 4.5 LWD Deflection and Modulus Standardization for the Test Track

Using the relationships described above, an initial idea of determining a range of deflection values from the relationship between the deflection and moisture content of the unbound material. It has been established that the moisture content of the material is important in the strength of the soil. Therefore, knowing the optimum moisture content from a Proctor test can help to establish a range for deflection values. Using the previously mentioned relationship between the deflection values and the moisture content of the Track Base, the power function trendline is shown in Equation 3.

$$\delta = 0.0002M^2 + 0.0012M + 0.0022 \quad (\text{Equation 3})$$

Where,

$\delta$  = deflection, in.

M = Moisture content, %

This trendline produced an  $R^2$  value of 0.6693, indicating a good correlation between the two. This increases confidence in the relationship between the two and using this relationship for a QC range. Using the OMC found from the Proctor analysis, 4.86 %, and the typical  $\pm 2$  range, in Equation 4, a range of deflection values were found. From this range, the preestablished fundamental relationship of deflection and modulus was also used. This relationship is presented in Equation 4 which is from Figure 4.6 and the second order polynomial trendline.

$$E = 170.73\delta^{-0.944} \quad \text{(Equation 4)}$$

Where,

E = Modulus, psi

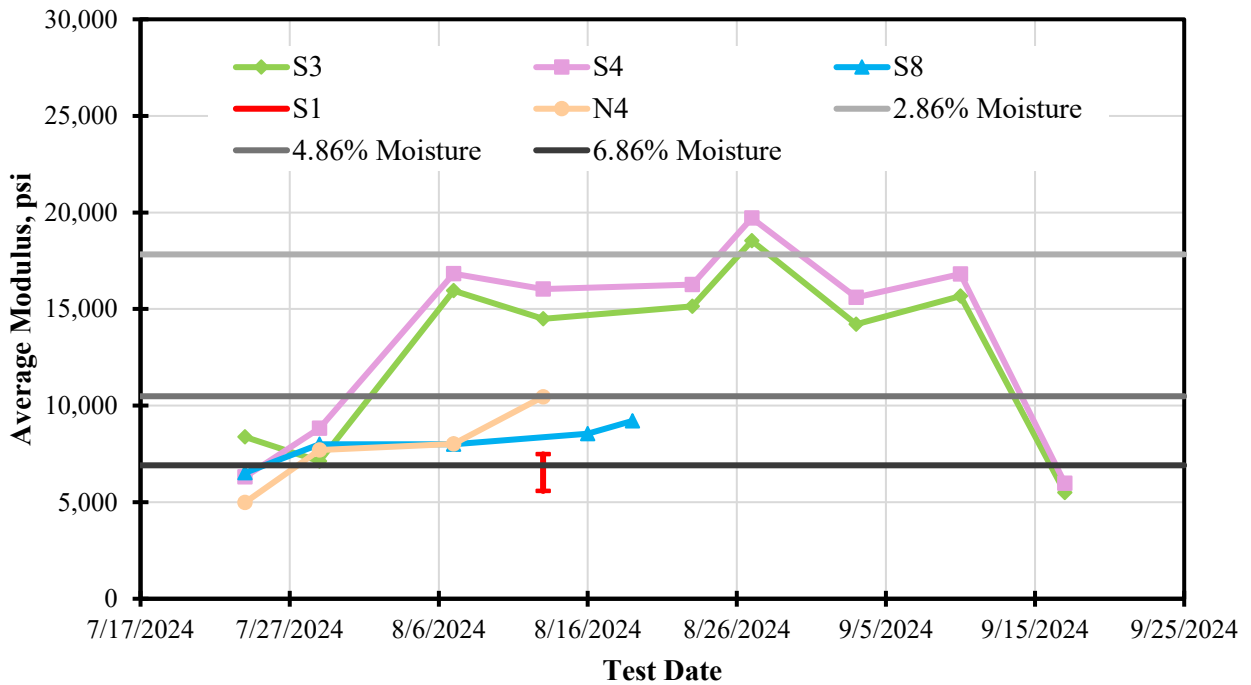
$\delta$ =deflection, in

From these relationships, Table 4.1 presents the ranges derived from the OMC. The deflection values in this table are maximum limits, and the modulus values are minimum values. The lowest allowable modulus value would be 6,908 psi; this would be useful after significant rainfall events that are prominent during construction season. The target modulus should be 10,484 psi due to the exact OMC. Reflecting on the data that was collected during reconstruction, there were seven instances where the modulus averages would not pass the lowest minimum value. This occurred in sections N4, S4, and S8 on 7/24/2024. In section S1 on 8/13/2024 and section S11 on 8/20/2024. The most prominent was in sections S3 and S4 on 9/17/2024. This last date is important to note since it was the last day before the sections were paved. If the only value that was used was the OMC modulus of 10,484 psi, sections N4, S1, and S8 would not have passed any QC throughout the construction period. These sections should be monitored throughout the trafficking period.

**Table 4.1: LWD Deflection and Modulus Limits Based on OMC**

<i>Moisture, %</i>	<i>Deflection, in</i>	<i>Modulus, psi</i>
2.86	0.007	17,829
4.86	0.013	10,484
6.86	0.020	6,908

Figure 4.16 presents the modulus values determined from testing with the modulus limits relating to each moisture content. This allows for a visualization of the dates that would not have passed the limits during reconstruction.



**Figure 4.16: LWD Modulus Values from Reconstruction with Modulus Limits**

A second method utilizing a multivariable equation to predict modulus values from deflection and moisture was also created. Incorporating both parameters in the same equation to determine a modulus value allowed for each parameter to have a different contribution to the overall value. Equation 5 presents the multivariable equation to determine the modulus. The

resulting  $R^2$  value from the use of this equation on collected data was 0.9412. This is likely due to the strong relationship between deflection and modulus that is the main effective parameter in this equation. The p-value for deflection was found to be  $6.2 \times 10^{-10}$  whereas the p-value for the moisture was 0.5689. This shows the higher significance and use of the deflection-modulus relationship in this equation.

$$E = 15307147\delta^2 - 1150807\delta + 23137 + 2.16 \times 10^9 M^{-11.06} \quad (\text{Equation 5})$$

Where,

E = Modulus, psi

$\delta$ =deflection, in

M = Moisture content, %

The main difference between this equation and the previous method is that both parameters are used at once to calculate the modulus. The same deflection values were used in this method as in the first method to identify differences in predicted modulus. By using these deflection values in Equation 5, the modulus values had some variation from the previous method. The values are shown in Table 4.2.

**Table 4.2: LWD Modulus Values from Multivariable Equation**

<i>Moisture, %</i>	<i>Deflection, in</i>	<i>Modulus, psi</i>
2.86	0.007	35,198
4.86	0.013	10,818
6.86	0.020	6,245

The most notable difference between the two sets of values found is for the lower Moisture content limit. There is almost a 20,000 psi difference between the two. However, for the OMC and

the upper limit, there is very little difference. Similarly to the first method, comparing the data collected during reconstruction there would be dates that would not pass QC. All testing days for N4 would only pass the lower limit minimum. The last two test dates for sections S3 and S4 would not have passed any of the minimum limits, this was due to the high moisture content from a significant rain event. Section S11 and S8 would pass for all dates using both the lower limit and OMC minimums. As previously mentioned, it will be important to monitor these sections during trafficking to see if the lower modulus of the base before paving has an impact on the performance of the pavement section.

A third method that was tested was to utilize the moisture and density in a model to predict the deflection. This method would help to establish quick limits during testing if the NDG is being used. If the NDG is used the immediate testing values would be able to be plugged into the equation to give a value of what the deflection and modulus of the material should be around. The model is presented in Equation 6:

$$\delta = 0.00187M^{0.877527} - 7.7 * 10^{-5}D^2 + 0.02006D - 1.2938 \quad \text{(Equation 6)}$$

Where,

$\delta$ =deflection, in

M = Moisture content, %

D = dry density, pcf

From this model the resulting  $R^2$  value is 0.5159, which shows a mid-range level of correlation. However, it allows for the use of both NDG parameters. From the historical Proctor data, the OMC is 4.86% and the maximum dry density is 143.4 pcf. Using those two values in this

model the resulting deflection would be 0.00597 in. This deflection correlates to a modulus value of 21,468 psi. Comparing these values to the previous two methods, they are significantly higher for the OMC. This method may not be the most accurate due to the lower  $R^2$  relationship.

These presented values would be used as minimum values, the higher the strength the better it would be for the integrity of the pavement structure. The most relevant value to use would be the modulus corresponding to the OMC. However, since the OMC has a  $\pm 2\%$  range for acceptance, the lower limit could be used if the NDG is also being used as a QC device for construction. It would not be recommended to use the OMC lower limit (2.86) as the minimum modulus value due to the variation between the two methods and the significantly higher requirement that would make it impractical to meet at every testing location. These are all preliminary ranges of values that reflect the data collected during the 2024 construction phase. This relationship and found range is beneficial as the Track Base material is consistent throughout construction for each cycle. This allows for a good starting point for analyzing the base during the next construction phase. Analyzing all the Test Track base deflection data, including the data without a NDG counterpart, it can be noticed that the days with the larger deflection results are after rain events. This is due to the increased moisture in the material. Despite the lack of NDG data for comparison on some of these dates, it is evident the moisture would be significantly above the given range for QC approval. This shows the importance of knowing the moisture alongside the deflection. In the field an important step before utilizing this data would be to create a test section of the base and perform validation testing and adjust the range if necessary. As mentioned in Chapter 2, using a test section before using the LWD for QC is typical. Similar to how the Track Base range was created, an analysis should also be performed for the subgrade.

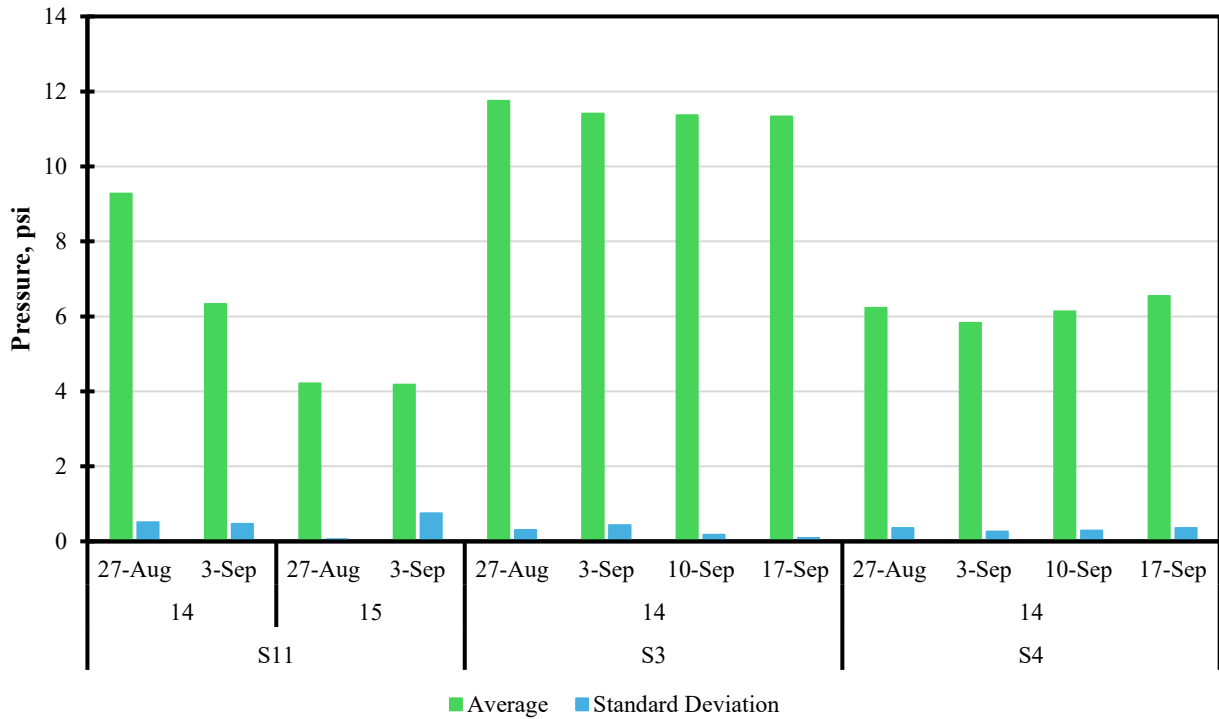
#### 4.6: EPC Data

Another set of data was created by performing the LWD on the EPCs that were instrumented into the top of the subgrade layer. The base lifts were constructed as normal on top of the layer that included the EPC. Testing with the LWD was then performed on the layers on top of the EPCs. For sections S3 and S4 that layer was the aggregate base and for section S11 it was base layer that had been covered with the tack coat. Testing was done typically when full section LWD testing was being performed. The location of the EPC in the sections is close to the location of RL 4 in the outside wheelpath. In sections S3 and S4, a pressure plate is installed at the top of the subgrade layer and at the top of the base layer. However, the EPC at the top of the base layer is not installed until the day of paving. Therefore, only one pressure plate was tested in these two sections. For these sections the pressure plate has an identification number of 14. This corresponds with other Test Track research. In section S11, due to the different treatments of the subgrade and base material the section has a total of three EPCs. For this testing, an EPC was located between the untreated and the treated subgrade and between the treated subgrade and the base. For this section, the pressure plates are labeled 15 for the EPC in the untreated base and 14 for the EPC in the treated base. S11 results show the data from two EPCs. Since typical testing with the LWD consists of three seatings drops and three reading drops, all six drops were recorded. The resulting data from testing on the EPCs was a voltage reading, this was then converted to a pressure value by utilizing the calibration factors from each specific EPC. Table 4.3 presents the calibration factors for each pressure plate per section. The calibration factor was multiplied by the voltage.

*Table 4.3: Calibration Factors for EPCs*

<i>Section</i>	<i>Pressure Plate ID</i>	<i>Calibration Factor, psi/Volt</i>
S3	14	7.270
S4	14	7.302
S11	14	7.294
S11	15	7.315

Figure 4.17 shows the pressure from the LWD when it was tested on each of the sections. It can be seen that in section S11 the difference between the two EPC results due to the depth they were placed in the base.

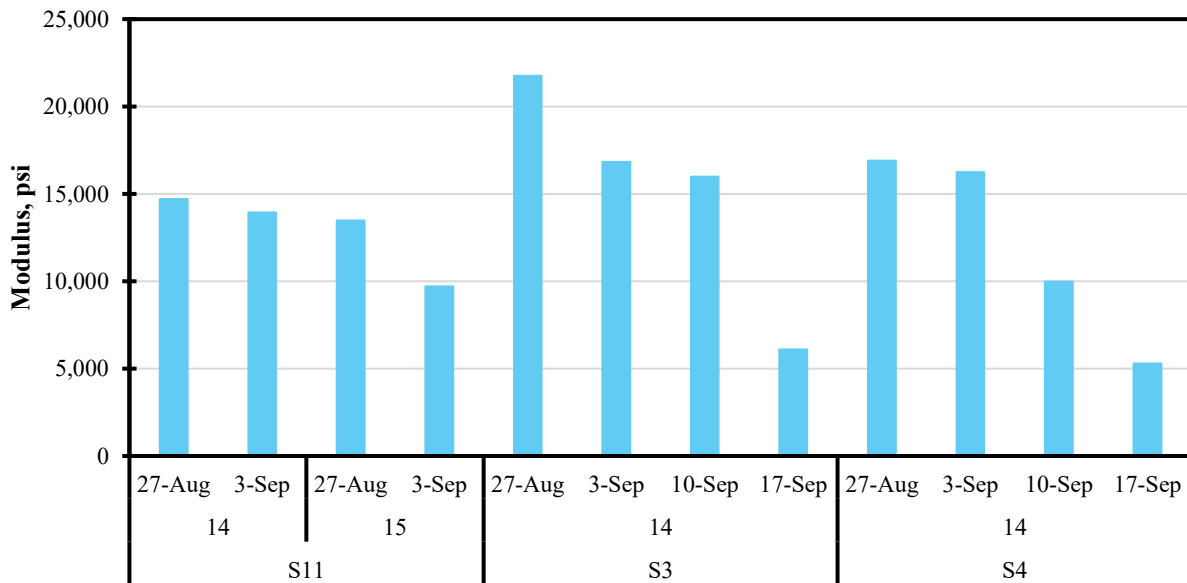


**Figure 4.17: Average EPC Voltage Values**

What is strongly indicated in Figure 4.17 is the consistency of the LWD testing regardless of the date or any other conditions. This is largely seen in sections S3 and S4. It should be noted that section S11 contains a stabilization enzyme with a tack coat on top. As previously mentioned, the EPC recorded all six of the LWD drops per date, therefore the standard deviation by date was included in Figure 4.17 alongside the average values.

The low standard deviations in the pressure values help show consistency of the drops that the LWD is producing for each test. EPC 14 in section S11 had the highest standard deviation on both test days, however with the different Shown in Figure 4.17 is the modulus values recorded by

the LWD on the pressure plate locations. EPC 15 in section S11 on September 3<sup>rd</sup> had the highest overall standard deviation. It is important to reiterate that section S11 was not pure Test Track Base like sections S3 and S4. Section S11 was Mississippi base with stabilization and tack coat. Unlike the pressure readings, the modulus is not consistent over time, it was affected by other conditions such as moisture. Similar to the EPC readings, the modulus values were separated by the pressure plate they were tested on. The modulus values are presented in Figure 4.18.



**Figure 4.18: LWD Modulus Values on EPCs**

Despite the modulus values being affected by other factors, the consistency that the LWD is performing its tests did not seem to have any change due to those same factors. This can be seen in the most extreme when comparing the LWD modulus change from the 10<sup>th</sup> of September to the 17<sup>th</sup>, there is a significant drop in modulus values, however, in the readings from the EPCs there is not a noticeable difference in the values. This is interesting because it shows that the pressure

produced by the LWD for each section is consistent in the individual sections but does not relate to the changing modulus values that are dependent on other conditions such as weather.

#### **4.7: Summary**

The data that was collected during the 2024 construction of the Test Track was largely focused on the base materials of the six structural sections utilizing the NDG, LWD, and embedded EPCs. Relationships can be seen between some parameters recorded by the NDG and LWD such as the moisture content, modulus, and deflection. The density recorded by the NDG lacked any relationships with the LWD data. The main relationship that was derived from this testing was between the LWD deflection and modulus and the moisture content of the base material. The moisture and deflection had a more sensitive relationship than the moisture with modulus. This motivated a prediction for deflection to be made to ultimately determine minimum values for LWD modulus QC standards. The strong correlation between the known OMC value from Proctor testing and a calculated range of deflection values provides a starting point for the use of the LWD as QC device during construction. This is a beneficial tool for testing the base materials after large rain events in particular. The calculated minimum values from both methods provided insight into what the properties of the base material should be during construction. The ultimate method that should be used was using Equations 3 and 4 separately to determine the values. From this method the preliminary minimum for the OMC of 4.86 would be 10,484 psi and the minimum for the higher moisture content limit of 6.86 would be 6,908 psi for the Test Track Base material. The lower minimum would be satisfactory to use in relationship with historical standards correlating with the NDG. This method can also be used to determine the minimums for other materials used at the Test Track. The embedded EPCs allowed for validation of consistency during LWD testing. The

next chapter discusses how the LWD was tested as a structural condition testing device on the asphalt pavement surface layer.

## Chapter 5: Data Gathered Under Traffic

At the NCAT Test Track, the sections are subjected to 10 million ESALs during the two year test cycle. Over the course of the testing cycle, various types of testing are performed, and data are collected relating to each section. In addition to the six structural sections that were tested while being rebuilt at the start of the 2024 Test Track cycle, there are ten other structural sections that are instrumented with EPCs and ASGs on the Track. These sections have varying years of when they were constructed, which also means a variation in amount of ESALs and distresses the sections have been subjected to since they began trafficking. The amount of ESALs, distresses, and any other changes within each section are monitored and recorded over the course of each test cycle.

These sections are monitored and evaluated by Test Track staff using the FWD, crack mapping and panoramic photography, and collection of strain data from the ASGs. During the 2024 test cycle, LWD testing was slowly implemented into the data collection rotation as well. Testing was done simultaneously with the FWD to establish relationships between the devices on specific dates. Routine testing by multiple methods allows for multiple data sets and parameters to better understand the health and structural integrity of each section while evaluating the use of the LWD as a pavement condition monitoring device. Testing frequently over the test cycles also allows for a trend to be seen over the increasing amount of ESALs and time since construction.

As mentioned in Chapter 3, the deflection data from the FWD was utilized to compute the modulus of the tested material using backcalculation. Since the LWD also uses deflection to calculate modulus, although at a smaller scale, there was an interest in using it as another pavement condition testing method. Before true implementation of the LWD in the routine testing cycle for

the sections, an understanding and relationship was needed to be developed between both the LWD and the sections being tested. Therefore, one objective of this research was to establish a relationship between the LWD and the FWD measurements of the asphalt surface, post construction. Since the LWD is a portable and faster testing method than the FWD, it was able to be run in 60-75 testing locations per tested section in around an hour. The FWD tested only 12 locations per section in the same amount of time. The simultaneous testing with the LWD and FWD was done in three sections that were constructed in the 2021 test cycle, sections N1, N2, and N7. These three structural sections are a part of the AG experiment. N1 was a dry rubber mix, N2 was a wet rubber mix, and N7 was the control mix. Sections N1 and N2 had notable cracking within the sections, mainly in the wheelpaths. Whereas N7, the control group, had no indications of any cracking distress. After testing with the LWD, the modulus values were overlaid onto the crack mapping that is done by panoramic pictures and AutoCAD. This allowed a relationship to be developed between the modulus throughout the sections and the visible cracking and distresses. During testing, the FWD was run first, with the LWD following behind it. This allowed for the temperature difference to be kept to a minimum between the start and finish of both the LWD and FWD testing. Both devices are shown in Figure 5.1.



*Figure 5.1: Test Track FWD and LWD Simultaneous Testing*

### **5.1: LWD Data**

Testing on the asphalt surface was performed on N1, N2, and N7. The first test date was June 2<sup>nd</sup>, 2025, the second test date was October 20<sup>th</sup>, 2025, and the third test date was on March 2<sup>nd</sup>, 2026. The testing was performed in the same manner for all days. For the June testing, 69 total tests were done in N1, 57 tests were done in N2, and 75 tests were done in N7. In October and March, the same amount of testing was done in each section, with outlying data removed. The amount of testing per section was correlated to the actual as built length of each of the sections. This variation was due to difference in the length of the transition areas between each section. Transition areas were included in testing for additional data. The transition areas are the parts of the section that are not included in the main 150 foot testing area of each section. The LWD was tested in 10 foot

increments in each of the three offset locations (I, B, and O) and as well as on the predetermined FWD testing locations on the RL locations. Schematics representing the testing patterns of both the FWD and LWD were depicted previously in Chapter 3. On all testing days, the three sections were not compared to each other due to the increase in temperature throughout the testing period leading to making comparisons based on modulus unreasonable. The pavement temperature was recorded by the temperature probes that are installed in each section at time of construction. Table 5.1 shows the temperature variations of the pavement between the sections during testing in June.

**Table 5.1: Temperature Changes During Testing on 06/02/2025**

	<i>Start Temperature, °F</i>		
<i>Offset</i>	<i>N1</i>	<i>N2</i>	<i>N7</i>
<i>O</i>	79.45	84.93	90.67
<i>B</i>	80.98	86.32	92.72
<i>I</i>	82.47	87.66	95.38

The temperature difference between the start of testing N1 and the start of testing N7 was 15.93 °F. Similarly, Table 5.2 has the temperature differences in the sections from the testing in October.

**Table 5.2: Temperature Changes During Testing on 10/20/2025**

	<i>Start Temperature, °F</i>		
<i>Offset</i>	<i>N1</i>	<i>N2</i>	<i>N7</i>
<i>O</i>	66.24	71.19	76.27
<i>B</i>	67.36	73.52	77.57
<i>I</i>	68.41	72.28	79.46

The temperatures for October testing per section were significantly lower than those of the June testing. The overall temperature difference from the start of N1 to the start of N7 was 13.22 °F. This difference is less than the temperature difference from the June testing, however it was

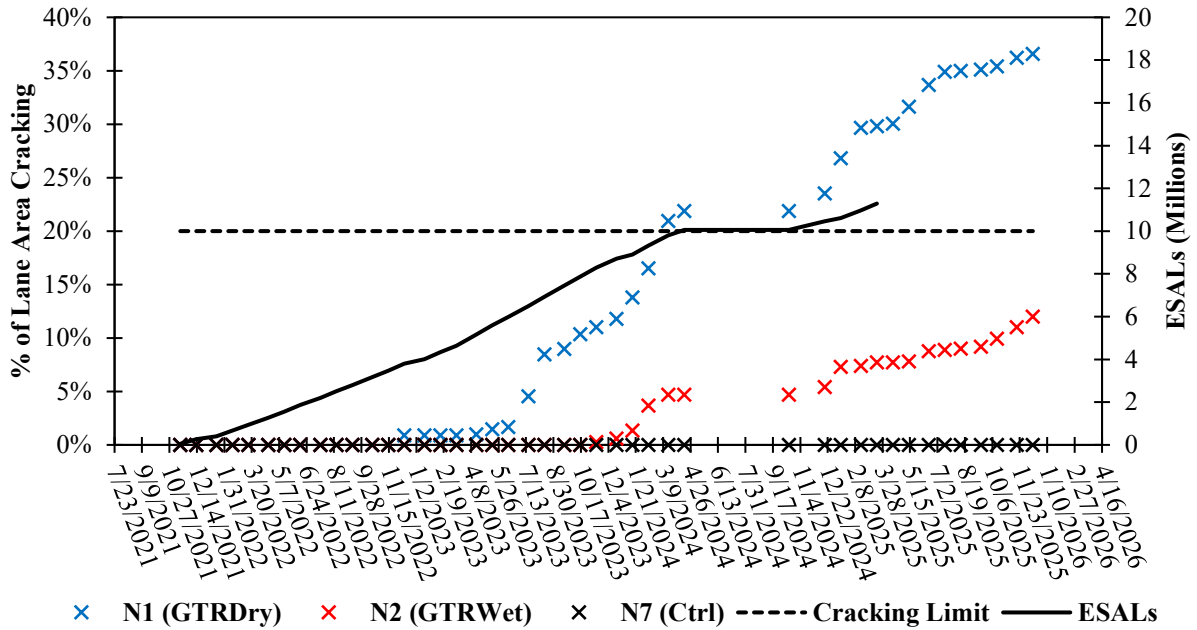
still vast enough to refrain from section comparisons. Something beneficial from the October testing temperatures were that they were closer to 68 °F, which is the typical FWD temperature correction value. Similarly, in March the lowest temperatures for each section were tested. The temperatures are shown in Table 5.3. The overall temperature difference from the start of N1 to the end of N7 was 14.34 °F.

**Table 5.3: Temperature Changes During Testing on 3/2/2026**

<i>Offset</i>	<i>Start Temperature, °F</i>		
	<i>N1</i>	<i>N2</i>	<i>N7</i>
<b><i>O</i></b>	59.42	63.55	69.91
<b><i>B</i></b>	60.87	64.52	71.88
<b><i>I</i></b>	62.18	65.47	73.76

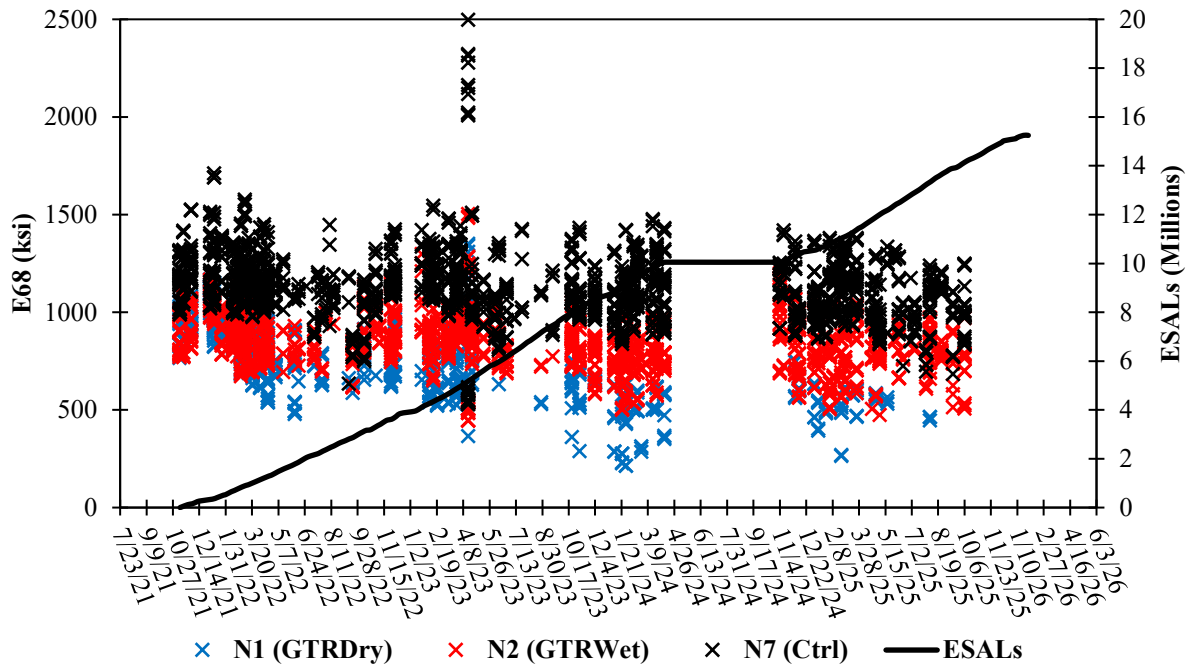
It can also be noted that even though section N7 was tested in the highest temperature on both testing dates, it still recorded the highest modulus values. This is interesting because higher temperatures typically cause the modulus of an asphalt concrete pavement to decrease due to its viscoelastic nature. The individual sections from each test date were compared to each other to consider changes over time and increased ESALs. The modulus values of each section were compared for each testing date to understand the effect temperature plays on the modulus values for each section. All raw LWD data that was collected during trafficking is presented in the appendix.

Section N1 has presented the most visible pavement distress since the end of the 2021 test cycle with 16.5% cracking in the wheel paths [West et al 2024]. This trend has continued into the 2024 test cycle, which can be seen in Figure 6.2. N1 has exceeded the 20% cracking threshold, however it has been kept in place to watch the crack progression in the 2024 test cycle. Before the third test date section N1 had almost doubled the cracking limit in the percent of lane area cracking.



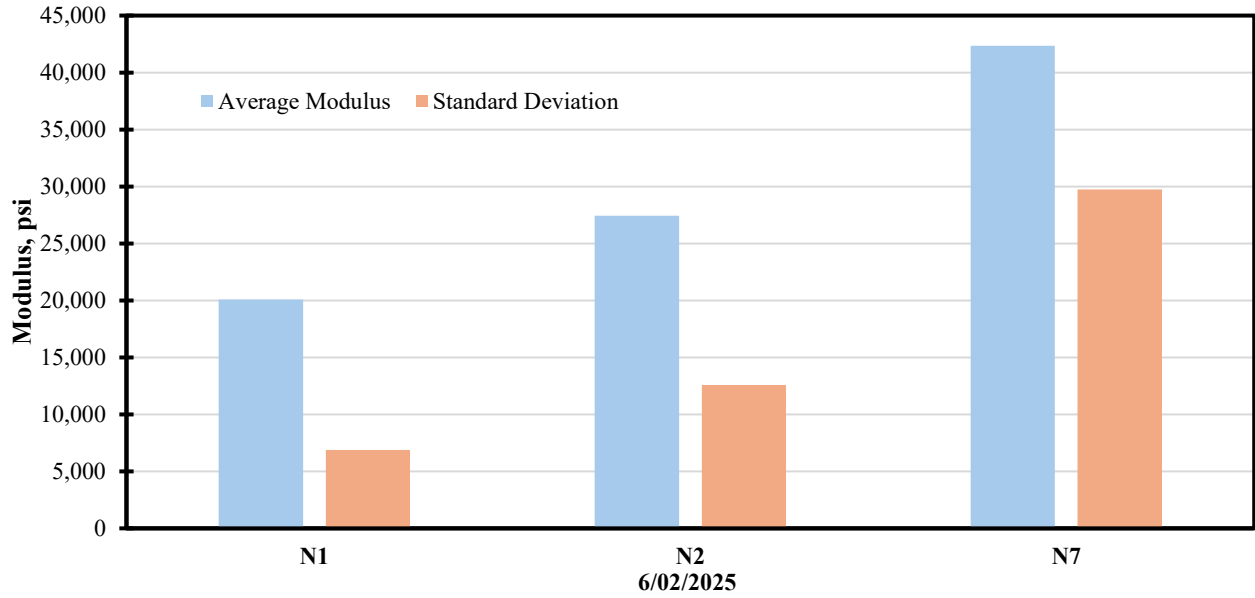
**Figure 5.2: N1, N2, and N7 Percent Cracking**

Similar to percent cracking, section N7 had the highest modulus values from backcalculation using FWD deflection data, N2 was in the middle, and N1 had the lowest modulus values. The modulus values over time and ESALs from the 2021 test cycle into the 2024 test cycle are shown in Figure 5.3. The modulus values for section N1 have decreased more than those of the other two sections over the given time period and ESAL loading. The N1 modulus values also started lower than the other two sections. The modulus values in this graph are backcalculated using EVERCALC 5.0 and are also temperature corrected to 68°F.

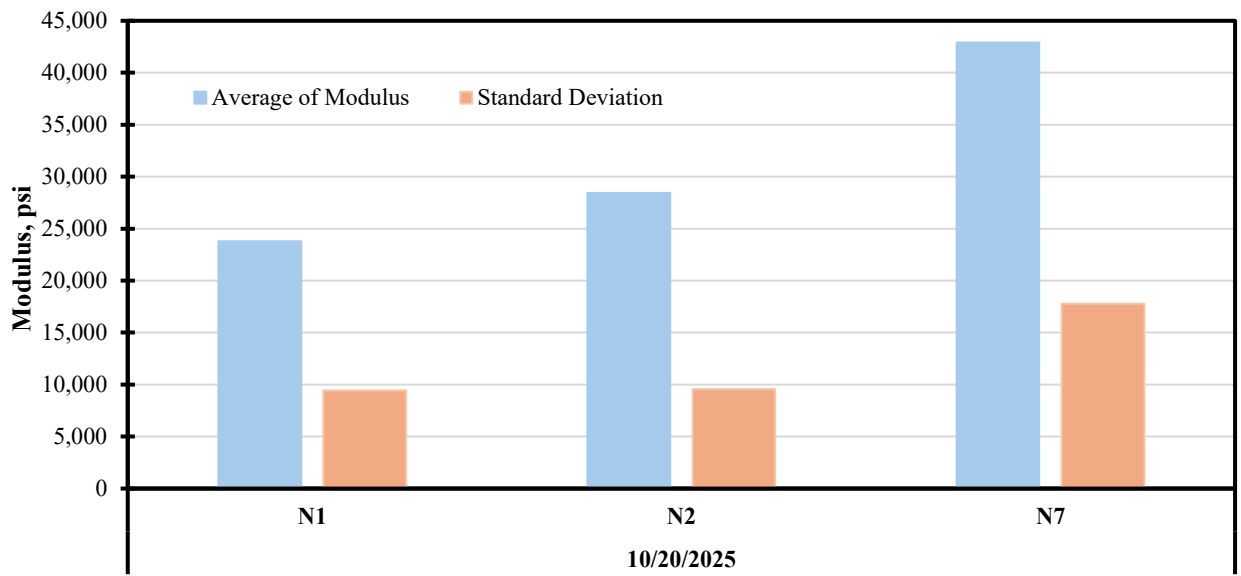


**Figure 5.3: N1, N2, and N7 Modulus Value**

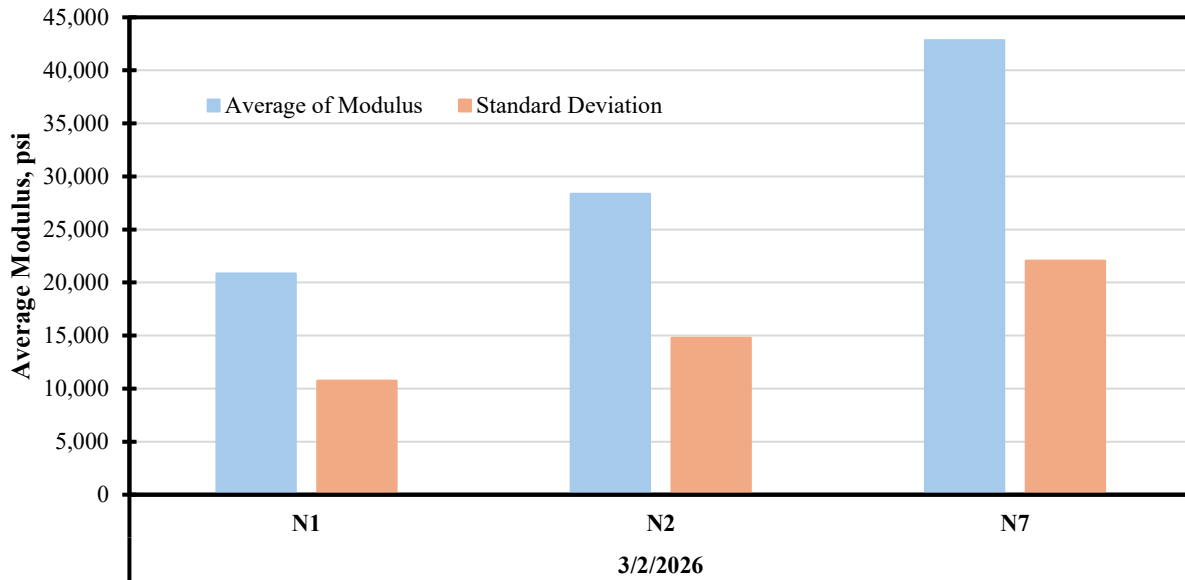
Shown in Figure 5.4 is the average modulus values in each section. Due to the temperature variation that was previously discussed, each section was not ultimately compared to each other, however ANOVA tests were comparing the averages between the sections to validate the decision to not compare each section. The resulting P-value ( $1.778 \times 10^{-09}$ ) showed there was significant differences between the average modulus values.



(a)



(b)



(c)

**Figure 5.4: Average Section LWD Modulus from (a) 6/02/2025, (b) 10/20/2025, and (c) 3/2/2026**

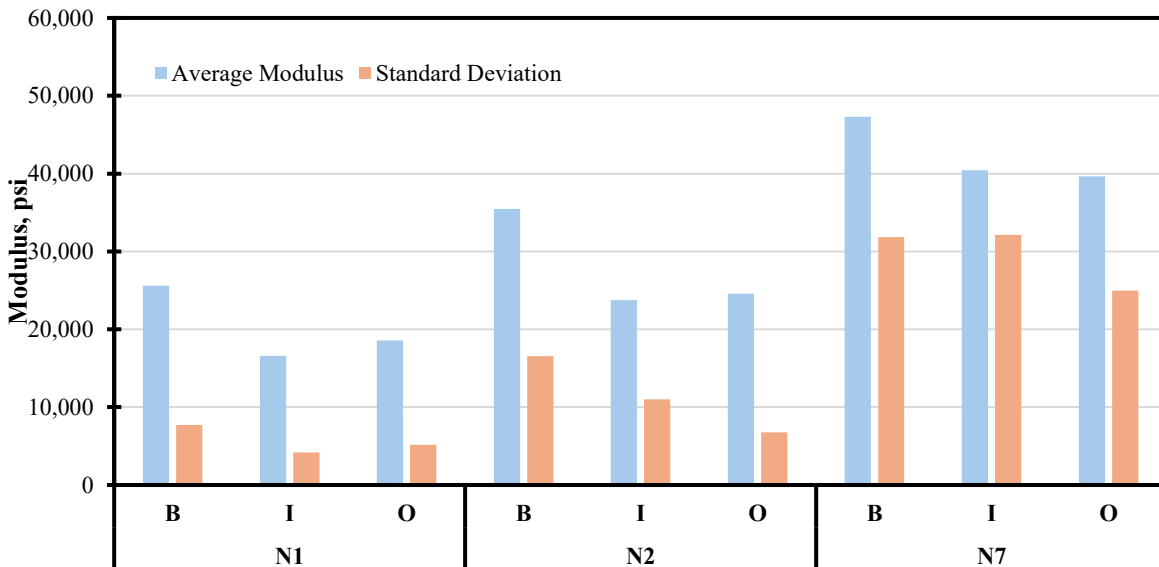
From Figure 5.4, on all testing days, section N7 had the highest average modulus value as well as the highest standard deviation, whereas section N1 had the lowest average modulus. The standard deviation for N1 was the lowest for all test days. This is an interesting result due to N1 having the most cracking distress present whereas N7 has no cracking at all. The lack of cracking in N7 may have played a role in the increased variation due to the inability to test in certain locations as well as the lack of deflection being produced by the low load level of the LWD on undamaged pavement. N7 also had longer transition areas so more testing was done within this section. Despite the temperature differences between the three testing days, the trend was consistent with N1 having the lowest average modulus and N7 having the highest average modulus. The lower temperature on the second and third test dates played a role in increasing the stiffness of the asphalt resulting in higher average modulus values for all sections. Table 5.4 shows

a tabulated version of the breakdown of the average modulus values by offset in each section for both testing dates.

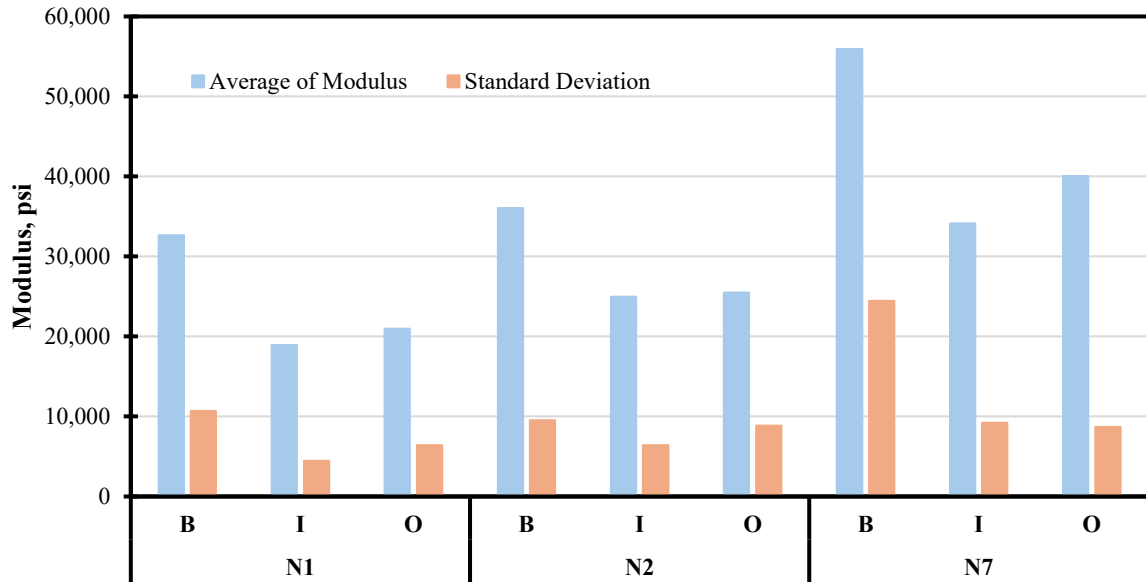
*Table 5.4: Average LWD Modulus by Offset*

Average LWD Modulus, psi									
	6/2/2025			10/20/2025			3/2/2026		
	I	B	O	I	B	O	I	B	O
N1	16,581	25,581	18,575	18,950	32,667	20,987	18,411	27,501	17,294
N2	23,748	35,469	24,562	24,969	36,064	25,489	22,953	37,474	24,639
N7	40,419	47,297	39,668	34,149	55,920	40,061	35,397	55,882	39,492

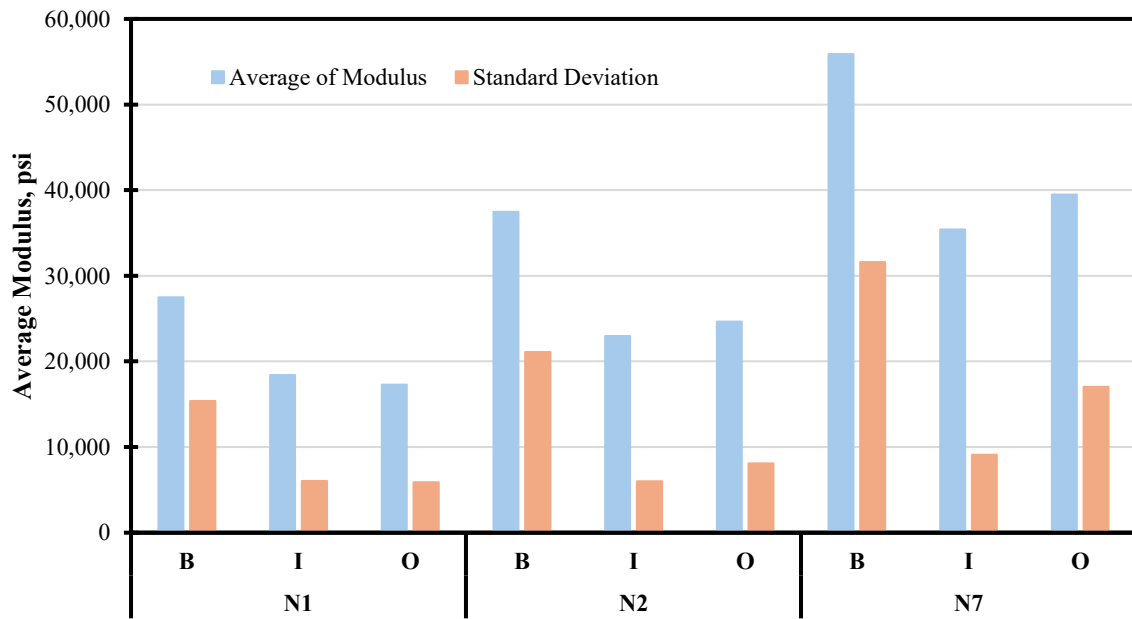
Testing in October and March produced the most similar results due to the similar testing temperatures. However, despite testing in March being the lowest, most of the highest recorded values were in October. In Figure 5.5, the average modulus values are broken down by the offset locations within each section in a graphical representation. The B offset for all sections on all tests days is consistently the highest. This is likely due to the lack of truck passes between the inside and outside wheelpaths. This results in less distresses in the B offset.



(a)



(b)



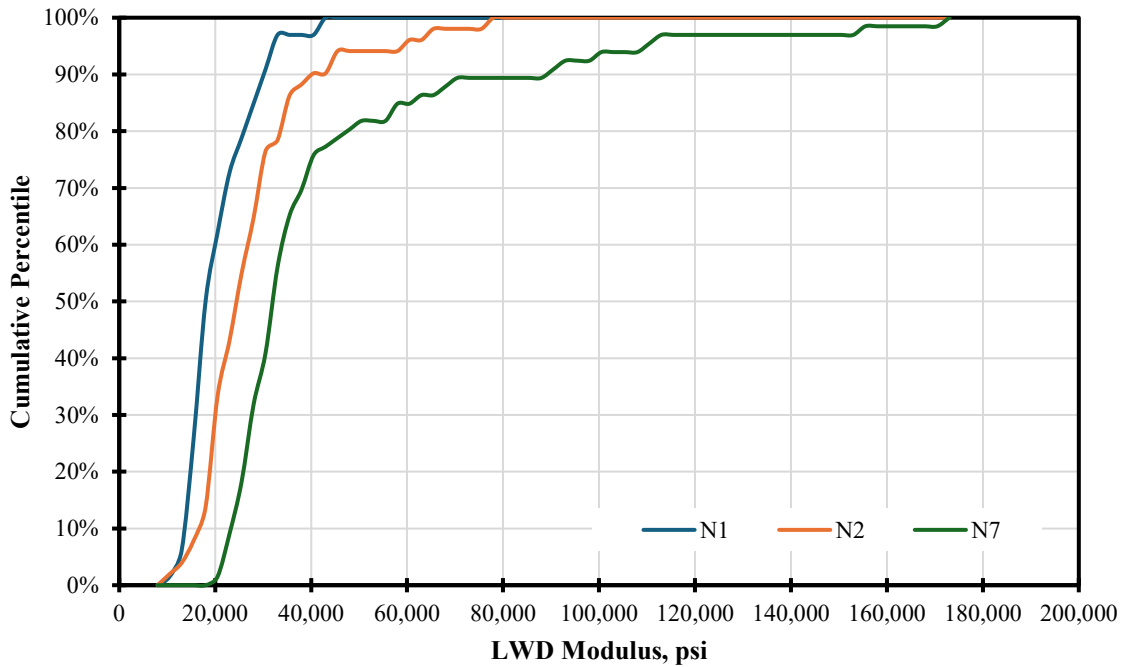
(c)

Figure 5.5: Average LWD Modulus by Offset from (a) 6/02/2025, (b) 10/20/2025, (c) 3/2/2026

From the June testing, all sections show the highest modulus values to be between the wheelpath, this makes sense due to the lack of loading in that area. ANOVA tests were run within each section to determine if the average modulus values from each offset location could be considered statistically different. The resulting P-values were  $7.1 \times 10^{-6}$ , 0.0105, and 0.6578, for sections N1, N2, and N7, respectively. From these results sections N1 and N2 cannot be considered the same for all three offset locations, but section N7 is statistically the same throughout. It was also noticed that the I and O wheelpaths had similar average modulus values in each of the sections. Therefore, since the B wheelpath has the highest modulus value, it was then removed from the ANOVA tests to test if the I and O wheelpaths, that are subjected to the most loading could be considered statistically similar. The resulting P-values for N1, N2, and N7 were 0.1581, 0.7867, and 0.9308, respectively. These results conclude that the I and O wheelpath modulus values are statistically the same in each of the three sections, respectively. This result is a good indicator that the LWD is able to produce results that correspond with what is actually happening to the section. In this case it is that the I and O wheelpaths are the most loaded due to the nature of the trucks driving on them causing them to have the most damage. In sections N1 and N2, most of the fatigue cracking throughout the sections occur in the I and O wheelpaths, which is where the LWD modulus values were lowest. By using the LWD throughout the entire section in all three wheelpaths, it allowed for an analysis of how the applied ESALs, and their resulting damage, are affecting the health of the sections.

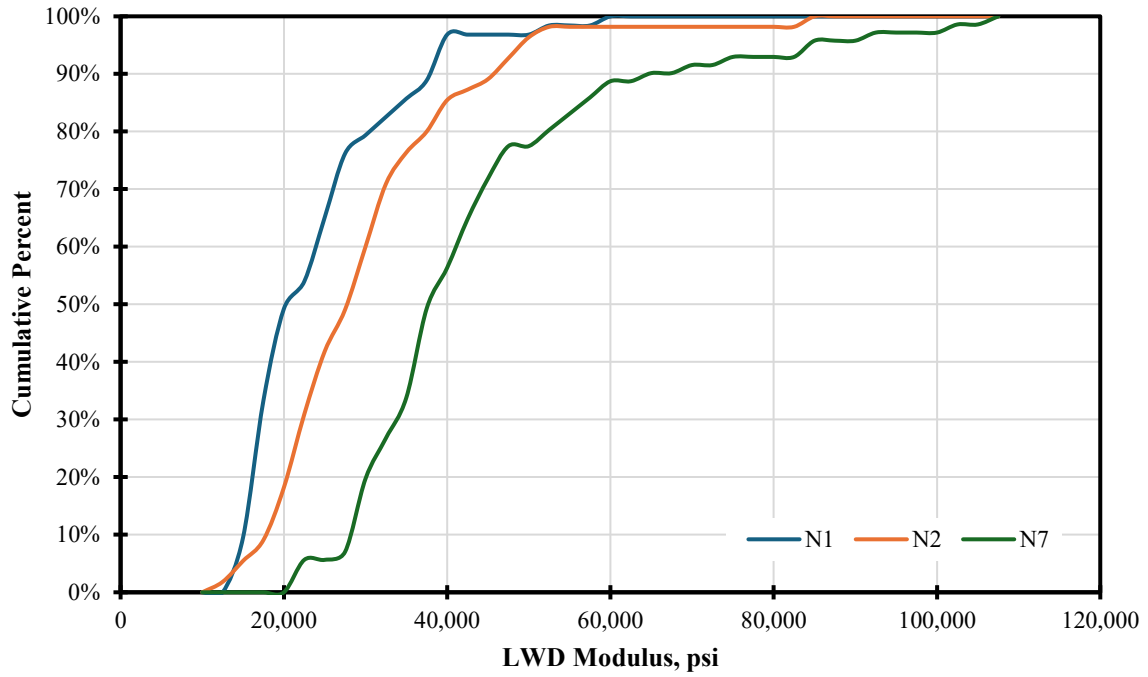
Similarly, from the testing done in October, the B wheelpath still recorded the highest average modulus values in all three sections. All ANOVA tests that were run for June testing were also run on the data from October testing to determine any differences between the sections.

The cumulative distribution of the LWD modulus of each section is shown in Figure 5.6. In this graph the minimum value for the modulus was 8,000 psi and the maximum value was 173,000 psi. The minimum and maximum values chosen to reflect the overall minimum and maximum modulus values out of all three sections from the collected data.



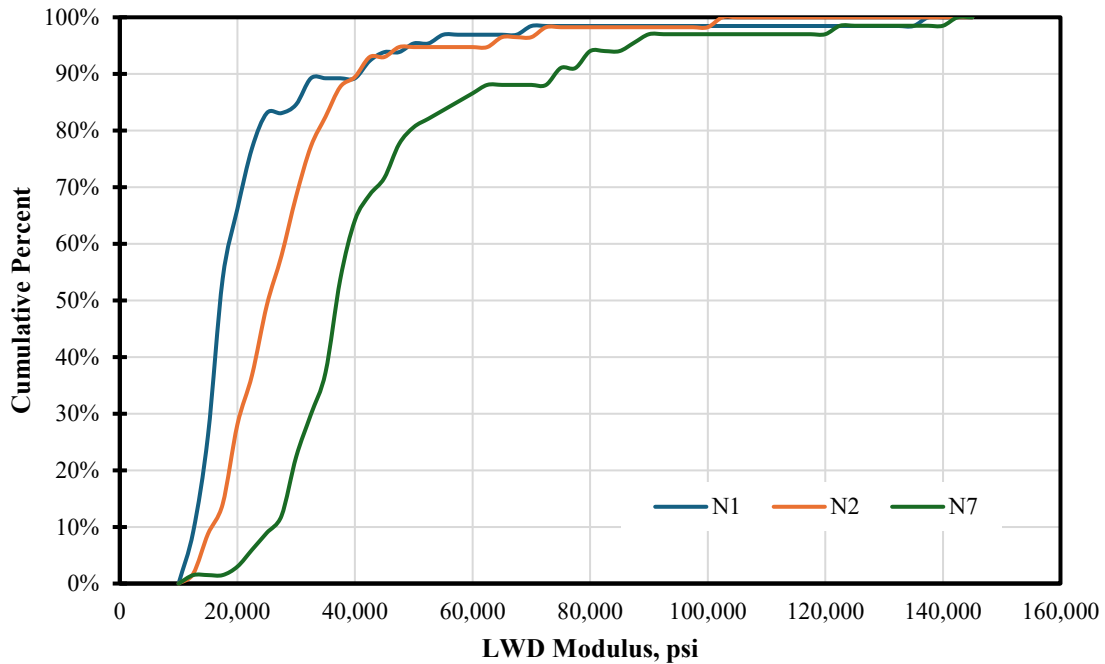
**Figure 5.6: Cumulative Distribution of LWD Modulus on 6/02/2025**

This graph is a good visual representation of the variation contributing to the different standard deviations in each of the three sections. Section N1 had the lowest variation and seen in this graph it has a steep slope indicating similar values from the highest to lowest values. Section N7 has the opposite trend. Where it increases rapidly up to a point and then gradually increases until it reaches 100%, this indicates larger differences in the modulus values throughout the section, which is reflected by having the highest standard deviation out of the three sections. Figure 5.7 is the cumulative distribution of the LWD modulus from the October testing date.



**Figure 5.7: Cumulative Distribution of LWD Modulus on 10/20/2025**

Similarly to the testing done in June, the N7 data in October had the most variation within the section causing a more gradual increase in in the cumulative percentage data. The data collected in March followed the same trends as the data collected in October. The data from March is shown in Figure 5.8. Unlike in June, in October and March the N1 data becomes more gradual.



**Figure 5.8: Cumulative Distribution of LWD Modulus on 3/2/2026**

The three cumulative distribution charts from each test date follow the same trends that are seen throughout all the research done during trafficking. The trend shows N1 having the lowest LWD modulus values over time. The three sections had similar trends for each test date, however there were differences in LWD modulus values over time. This is likely due to the heavy, consistent loading that occurs at the Test Track. Table 5.5 shows the average modulus differences between test dates by section. In this research temperature effects were not factored in. However, with more analysis a temperature correction similar to what is done with FWD data for temperature correction to 68 °F could likely be done.

**Table 5.5: Difference in Modulus by Section**

<i>Section</i>	<i>Average Difference in Modulus, psi</i>		
	<i>N1</i>	<i>N2</i>	<i>N7</i>
<i>June to October</i>	-3,815	4,637	-1,973
<i>October to March</i>	2,652	-7,325	2,172

A decrease in modulus makes sense when the pavement material is undergoing heavy trafficking. The only negative differences in testing were from June to October in sections N1 and N7 and from October to March in section N2. The temperature condition that the pavement was tested in may also play a large role in the lack of a decrease in modulus due to the viscoelastic nature of the asphalt. As previously mentioned, no temperature correction was performed on the LWD data during this research. Relating the modulus values to the distress levels in the section can also help understand a decrease in modulus values. The next section presents crack mapping for each section overlain with the modulus values. An increase in cracking can be seen in the maps for each test date.

## **5.2: LWD Crack Mapping**

Another point of interest was to use the LWD as an APT condition assessment tool for the sections at the Test Track. The initial process for this was to establish a relationship between the modulus values recorded by the LWD throughout each of the sections and overlay that onto the crack maps of the section. The crack maps for each section were created by the technicians at the Test Track by utilizing panoramic photos and videos of the section and using AutoCAD to draw in the cracks and other distresses present to emphasize their locations. By doing this, it allowed for a visual representation of the difference in modulus values throughout the section. The benefit of using the LWD for this instead of the FWD is that the LWD was able to cover the entire section, not just the

RL location testing spots. The testing for this relationship was done at the same time the testing for creating a relationship with the FWD was completed. Crack mapping is routinely done at the Test Track by using a high definition camera attached to a research vehicle. Cracks are denoted with blue markings in each section. The studied 150 foot area is shown as a red rectangle. The portions of the sections before and after the red rectangle are the transition areas, the start and end of this area are also labeled on each figure. Figure 5.9 shows the first set crack maps of each section with the LWD modulus values from testing overlain from June testing. The LWD values are marked with a colored circle representing which percentile the value falls into as explained in the following paragraph.

For this analysis each test date had its own set of cumulative percentiles created which then correlated to a set color that was labeled on the crack maps. The cumulative percentile ranges were 0-20%, 20-40%, 40-60%, 60-80%, and 80-100%. Percentiles were created for each separate date due to different conditions, such as temperature and increased distress over time. Using the percentiles exclusively for each specific test date allowed for a visual comparison between the distress levels in each section with similar temperatures from specific testing dates.

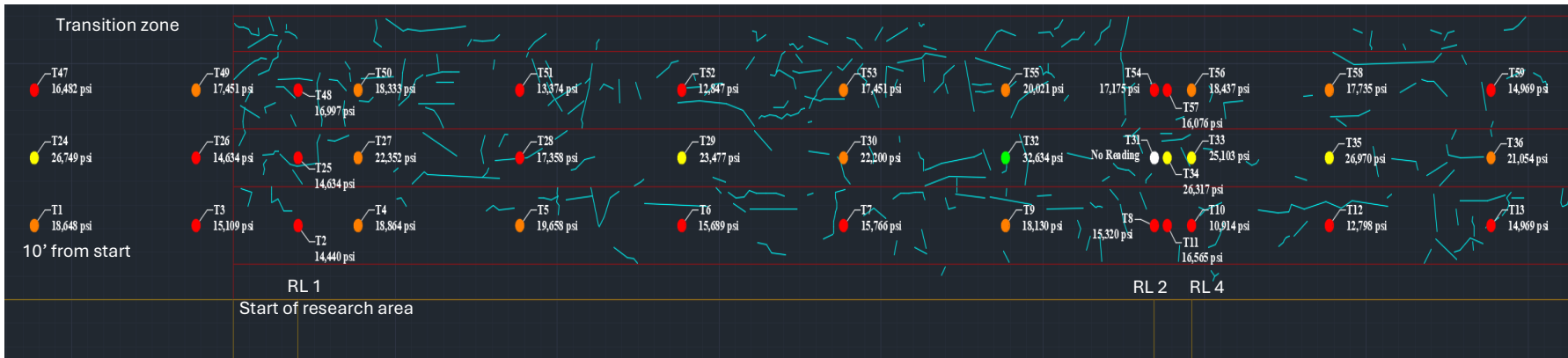
The first set of crack maps represent the data from June 2025 testing, the second set represents the testing done in October 2025, and the third the last test date in March 2026. The crack maps represent continuous sections, however for ease of visualization they were broken into two figures per section. The top figure is the start of the section and the second is the continuation. This occurs for all 9 crack maps. There are also labels for the transition zones at the beginning and end of the section, these areas are not considered in most research done at the Test Track.

Table 5.6 provides information on the percentiles that the colors on the June map represent. Red is 0-20%, orange is 20-40%, yellow is 40-60%, light green is 60-80%, dark green in 80-100%, and white is areas where a reading was not able to be obtained.

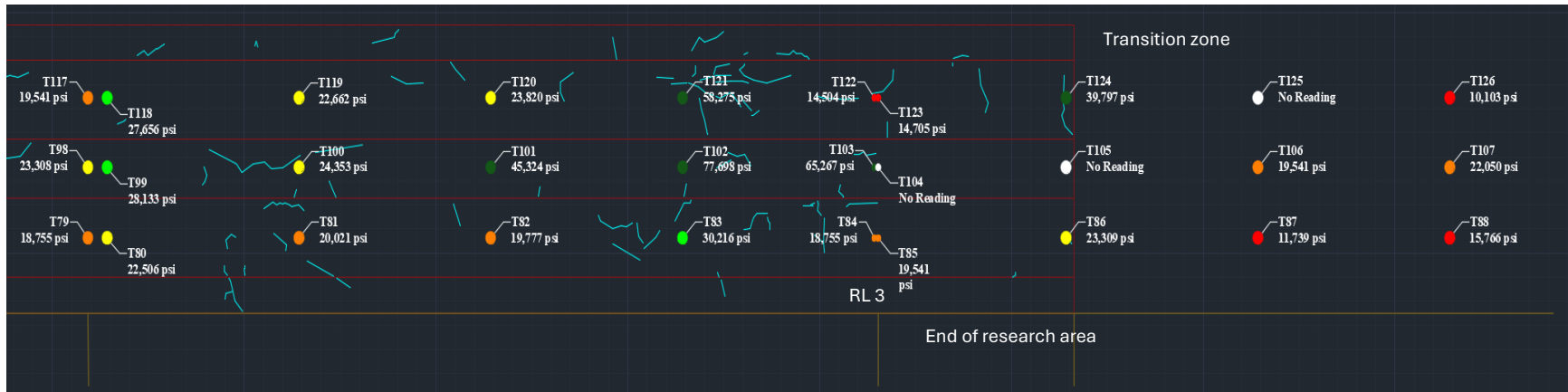
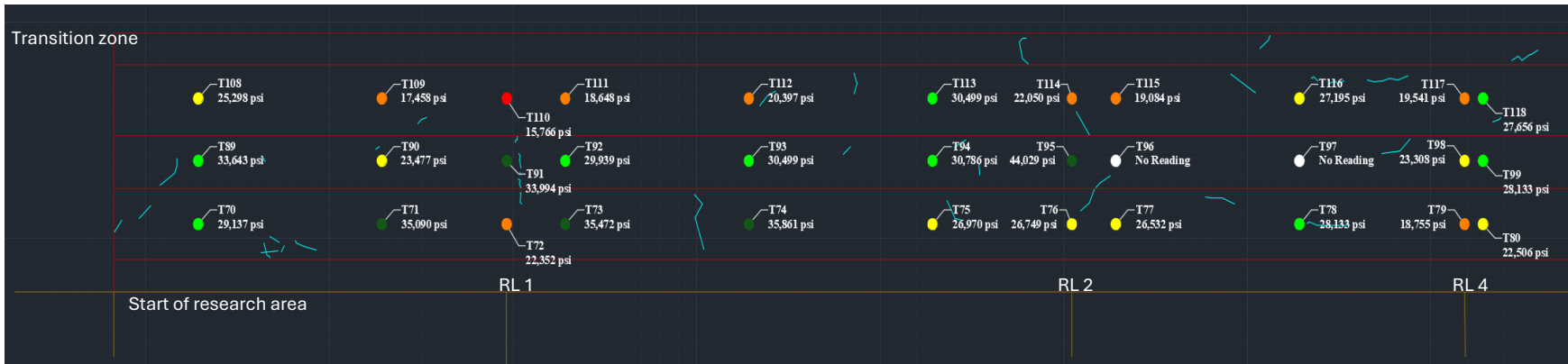
**Table 5.6: June Crack Map Legend**

<b>Color</b>	<b>Percentile</b>	<b>Modulus Range, psi</b>
Red	0-20%	0-17,451
Orange	20-40%	17,451-22,505
Yellow	40-60%	22,505-27,655
Light Green	60-80%	27,655-33,994
Dark Green	80-100%	33,994-171,751
White	N/A	No Reading

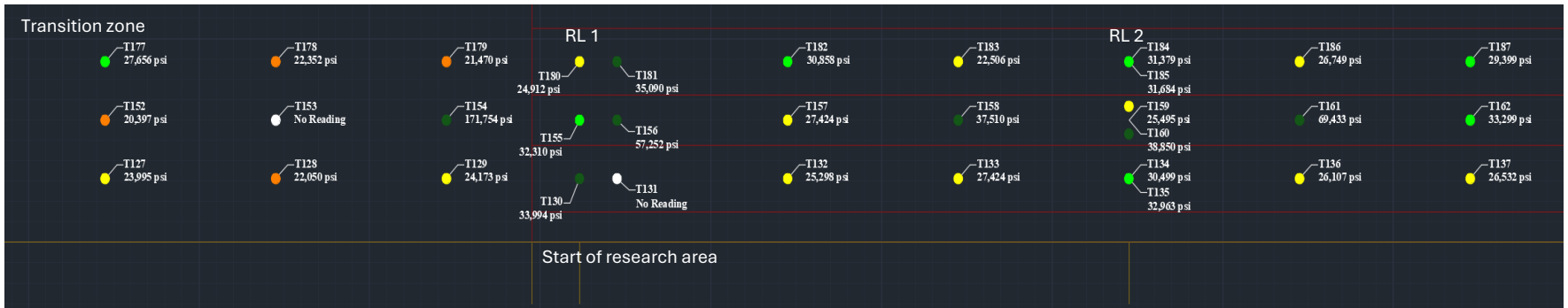
The locations in white that indicate a reading was not able to be produced occurred in all three sections. This might have occurred due to the lack of significant deflection at the location causing the LWD equations to not work.



(a)



(b)



(c)

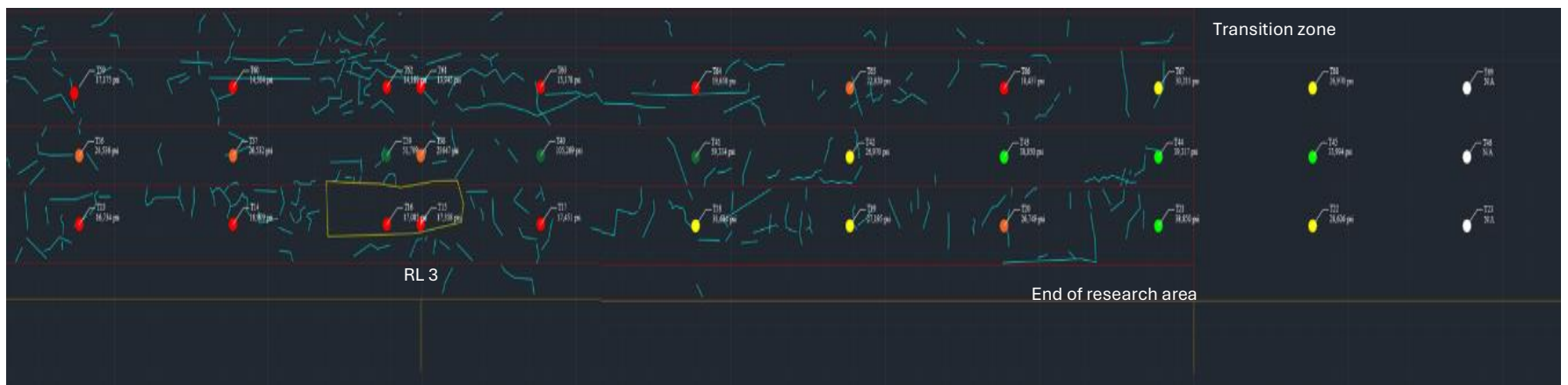
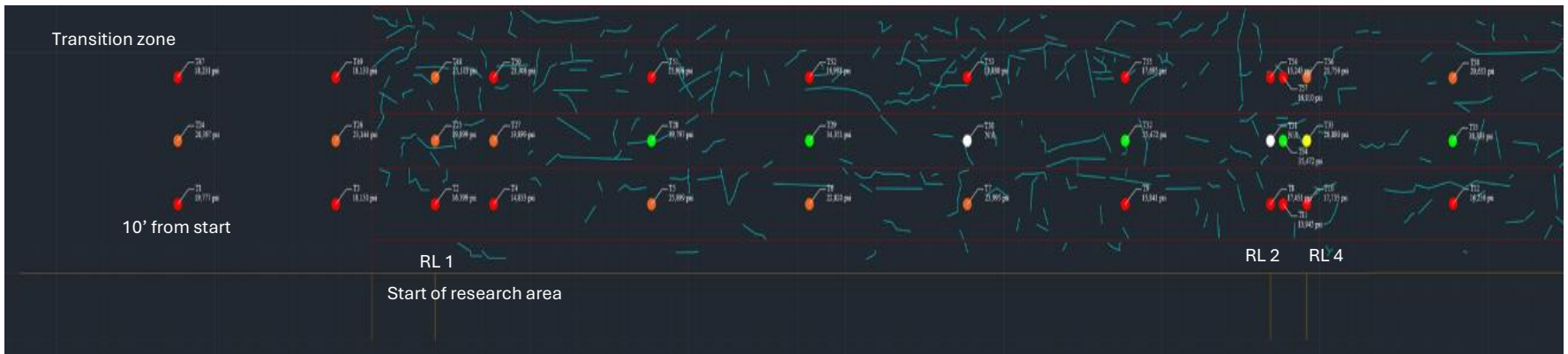
Figure 5.9: Crack Maps of (a) N1, (b) N2, and (c) N7 from 6/02/2025

Section N7 only had modulus values between 20 to 100 percentile ranges. It also had the highest number of both light green and dark green spots (57%). On the contrary, section N1 had the highest amount of red spots (41%). Sections N2 and N7 had few to no red spots with 4% and 0%, respectively. In section N1, the locations with lower modulus values were typically clustered around the areas where the cracking was most significant. However, at the locations labeled T32, T39, T41, and T42 there are exceptions to this trend. These locations in N1 have high modulus values despite being on top of or near cracking locations. This is the opposite of what is to be expected, however it only occurs in a few locations throughout the sections. This contributes to some of the variability throughout the data. The other high modulus values in N1 occur in the end transition area adjacent to N2, where no cracking is present. It is interesting to see in section N1, with the most cracking there are certain data points on or around cracks that present better modulus values than locations where no cracking is present on the surface. This also occurs in a few locations in section N2. However, the general trend that is reflected is that the LWD is able to identify lower modulus values for sections that are visually damaged.

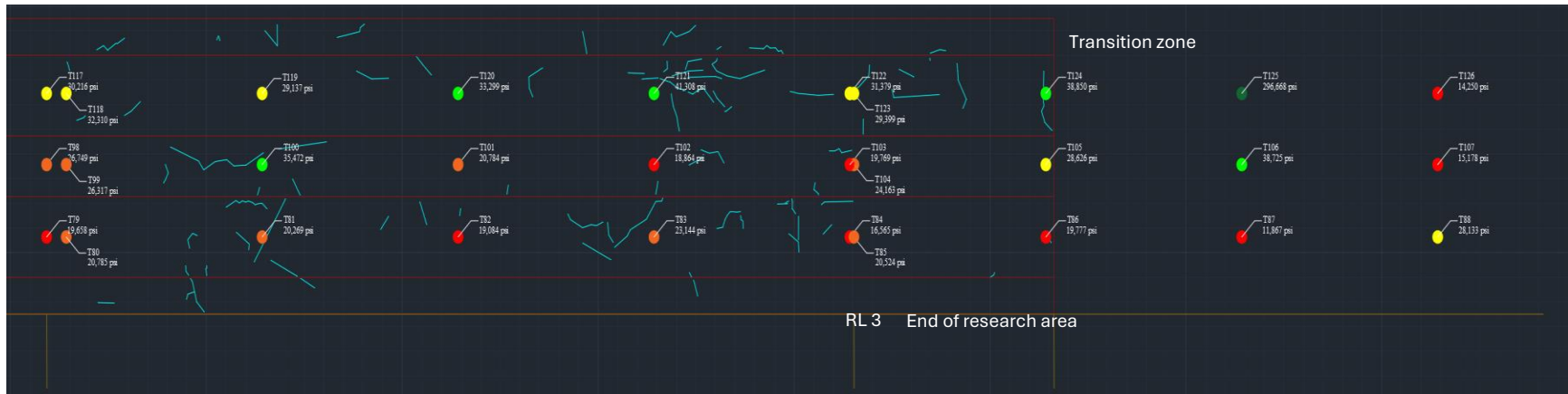
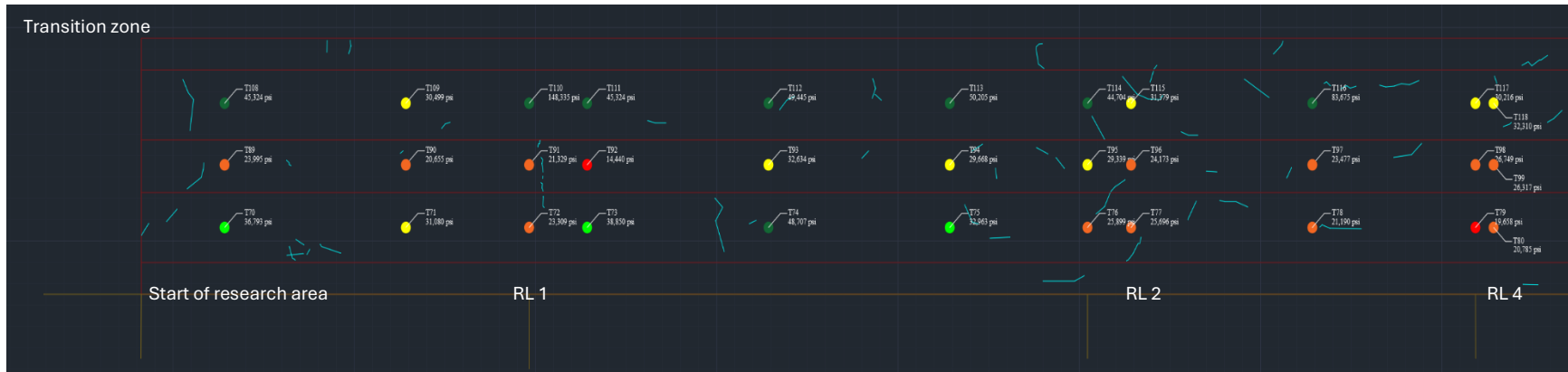
Similarly to the June testing, Figure 5.10 presents the crack maps from October testing. Table 5.7 shows the color coding legend for the October crack maps. The colors represent the same percentiles as the previous legend; however, the modulus ranges represent only the October test date LWD values.

**Table 5.7: October Crack Map Legend**

<i>Color</i>	<i>Percentile</i>	<i>Modulus Range, psi</i>
Red	0-20%	0-19,801
Orange	20-40%	19,801-26,837
Yellow	40-60%	26,837-32,962
Light Green	60-80%	32,962-41,837
Dark Green	80-100%	41,837-296,667
White	N/A	No Reading



(a)



(b)



(c)

Figure 5.10: Crack Maps of (a) N1, (b) N2, and (c) N7 from 10/20/2025

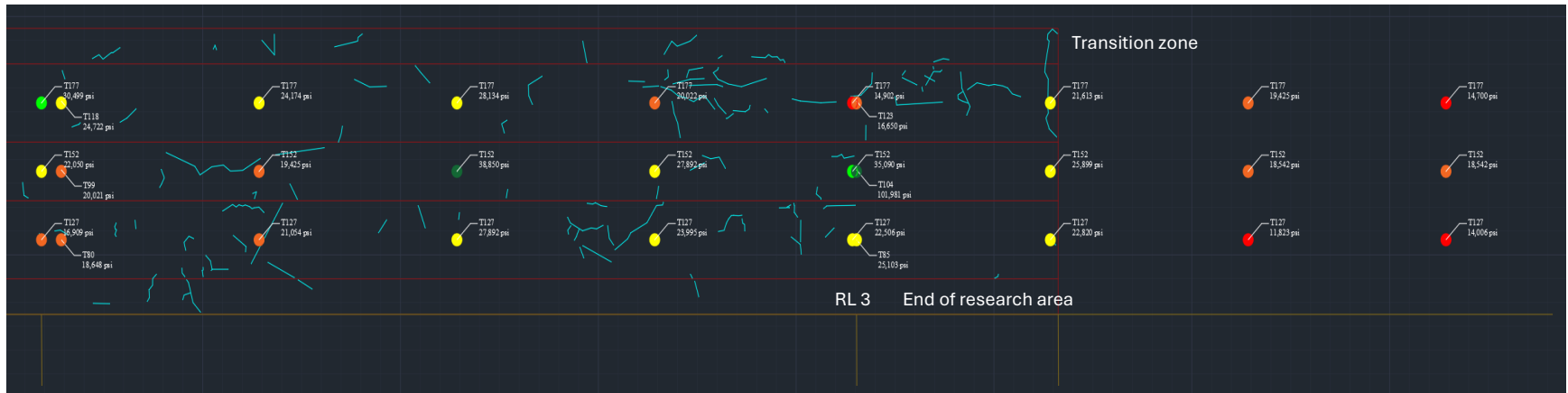
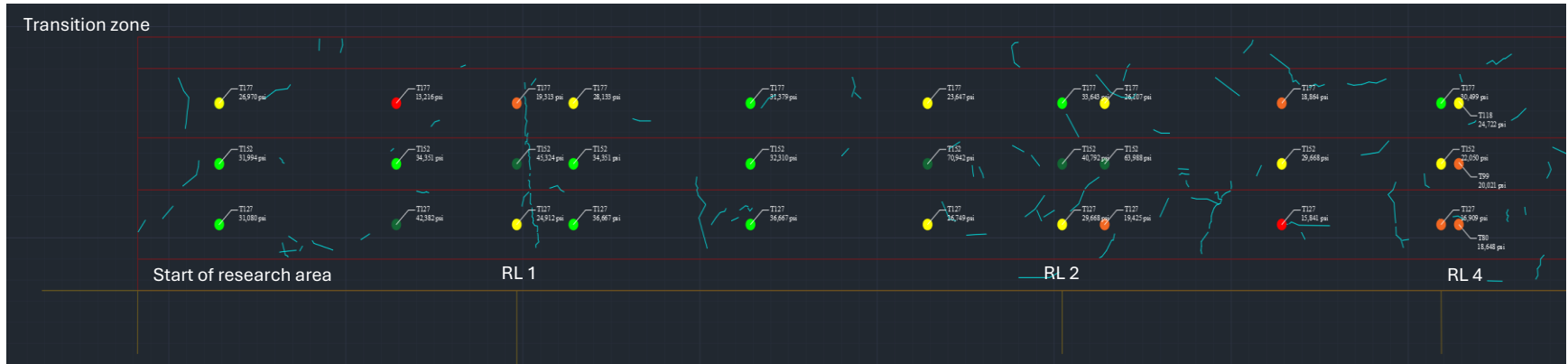
The same format that was used on the June crack maps was used in the October crack maps. This is both noticeable in the modulus values and the crack map. The last test date was in March. All testing was done the same as it was in June and October. Table 5.8 presents the same percentiles as before, but with the data specifically from March.

**Table 5.8: March Crack Map Legend**

<i>Color</i>	<i>Percentile</i>	<i>Modulus Range, psi</i>
Red	0-20%	0-16,235
Orange	20-40%	16,235-21,274
Yellow	40-60%	21,274-30,215
Light Green	60-80%	30,215-38,849
Dark Green	80-100%	38,849-141,884
White	N/A	No Reading

The crack maps for this test date are shown in Figure 5.11. The distress trends with N1 being the most distresses, N2 having medium distress, and N7 having none are still consistent.





(b)



(c)

Figure 5.11: Crack Maps of (a) N1, (b) N2, and (c) N7 from 3/2/2026

Throughout all three test dates the LWD modulus was consistent with trends within the sections. However, in the sections such as N1 and N2, the visible increase in cracking helps to understand the difference between each test day. Another important finding from the three test dates that is specifically seen in the crack maps and the tables representing the data is the variation prominent in N7 and in the B offset. The range of data under the dark green label, which is predominantly seen in N7 data is a significantly larger range than the other data points. This coincides with the larger standard deviations within the averages of N7 data. This is likely due to the lack of deflection being produced by the LWD on the asphalt surface due to the stiffness. This introduces a limitation for using the LWD on the asphalt surface when the data is not able to be repeatable and meaningful.

The development and use of the crack maps is beneficial in establishing visual connections and understanding between the structural and visual conditions of the pavement sections. From these maps it is evident that when the sections, such as N1, experience distresses their modulus values will decrease as the deflections increase. This relationship also allows for an understanding of the magnitude of the modulus from the LWD. Due to the testing done on three sections with varying distress levels, high, medium, and none, a range from those three sections was established. From the three test dates Table 5.9 shows the 25<sup>th</sup>, 50<sup>th</sup>, and 75<sup>th</sup> percentile for all the test dates. From all three test dates the values from section N1 with the most distress tended to be around the 20<sup>th</sup> percentile. Section N2 was between the 20<sup>th</sup> and 60<sup>th</sup> percentiles. Section N7 that did not present any distress on all three test days stayed in the 60<sup>th</sup> to 100<sup>th</sup> percentiles.

**Table 5.9: 25<sup>th</sup>, 50<sup>th</sup>, and 75<sup>th</sup> Percentiles of Modulus Values**

<b>Test Date</b>	<b>Modulus, psi</b>		
	<b>25<sup>th</sup></b>	<b>50<sup>th</sup></b>	<b>75<sup>th</sup></b>
3/2/2026	17,085	26,003	36,667
10/20/2025	20,785	29,803	38,850
6/2/2025	17,085	26,003	32,310
Average	18,319	27,270	35,942

An average 20<sup>th</sup> percentile value is 18,319 psi. Relating this to the distresses and section produced results around this value, it is likely that the section contains high distress levels. Similarly, the average 50<sup>th</sup> percentile value is 27,270 psi. Sections containing values around this number may have some medium levels of distress. Whereas the 75<sup>th</sup> percentile average value was 35,942 psi. Sections with values around this or higher may not have any distresses.

### **5.3: LWD Comparisons**

During testing in October and March, three different LWD devices were all tested on section N1 over the same time frame. The three LWD devices that were tested were the Zorn ZFG 3.1 that was used for all other NCAT research, the ZORN ZGF 3000 and the Olson, which were tested by the Georgia DOT (GDOT). All three LWDs fall under the same ASTM standard; ASTM E2835-21 “Standard Test Method for Measuring Deflections Using a Portable Impulse Plate Load Test Device”. Out of the three LWDs tested, the NCAT Zorn was the newest model of LWD used. By testing the three different devices on the same section with similar temperatures, comparisons were able to be made between the different devices and the resulting modulus values. The parameters for each device such as the plate diameter (300 mm) and drop weight (10 kg) were kept consistent for each of the three devices. The three devices are shown in Figure 5.12.



*(a) GDOT Olson LWD*



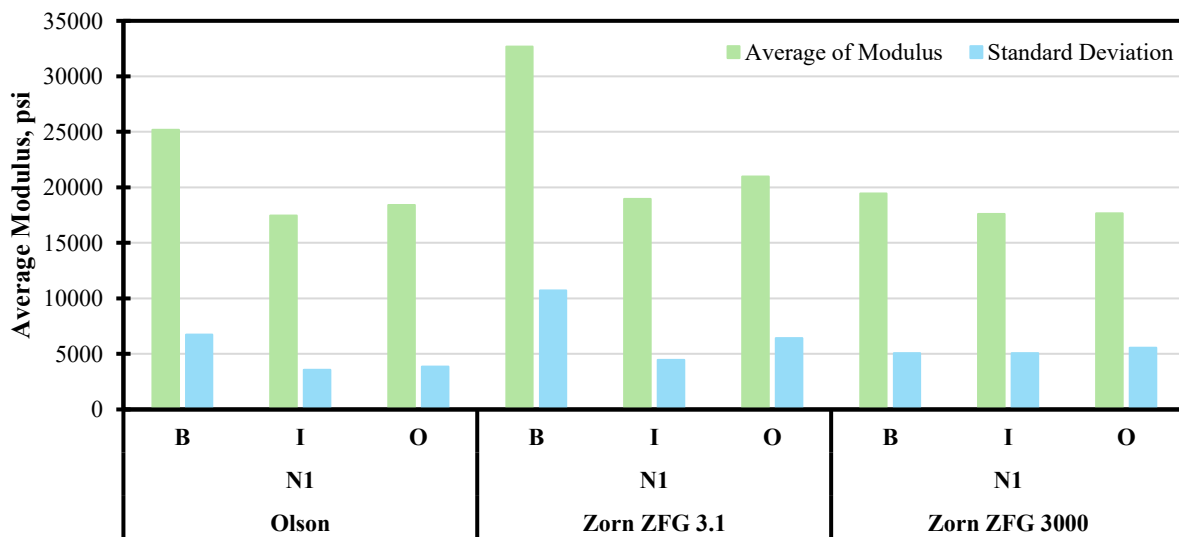
*(b) GDOT Zorn LWD*

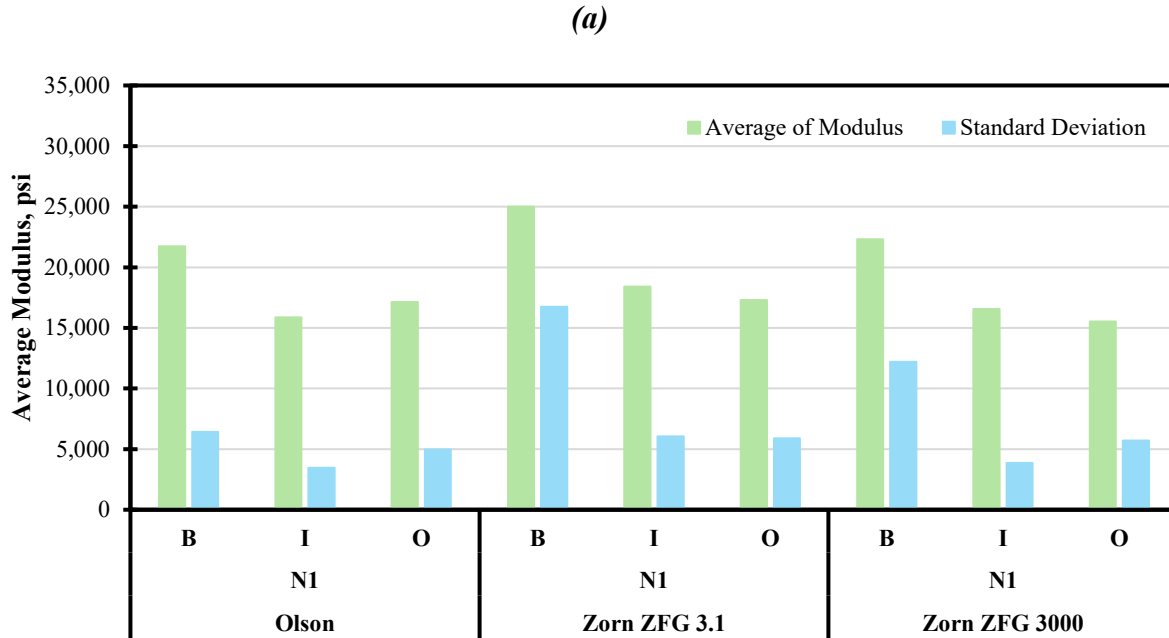


(c) NCAT Zorn LWD

Figure 5.12: Three LWDs Tested on Section N1

Figure 5.13 shows the average modulus for the three offset values in N1 that were tested by the three LWD devices in October and March.





**(b)**

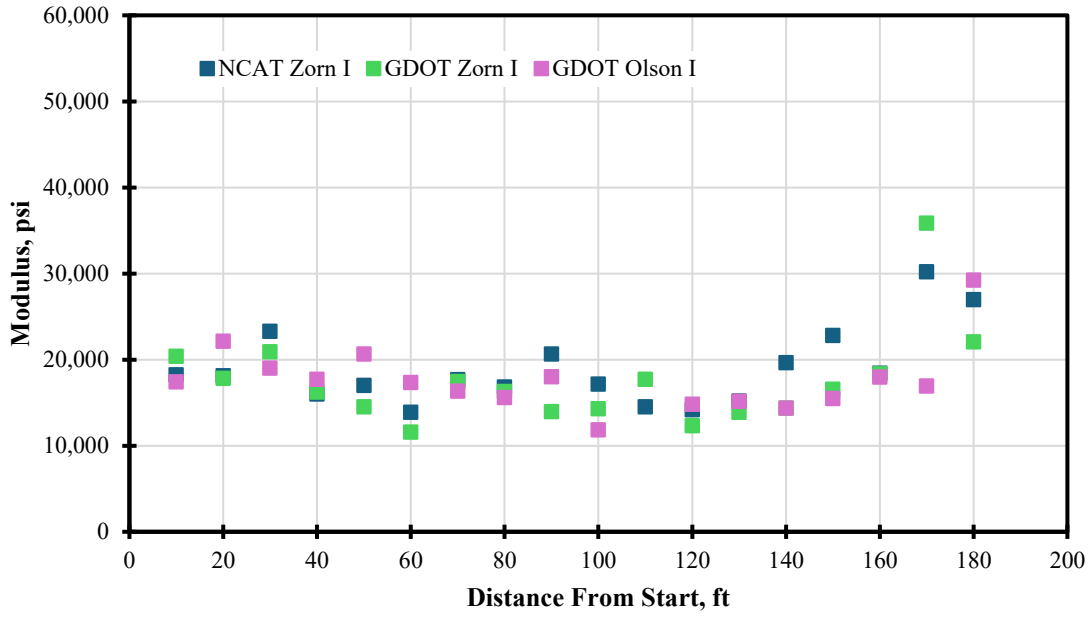
**Figure 5.13: Comparison of Three LWD Devices on N1 on (a) 10/20/2025 and (b) 3/2/2026**

In Figure 5.13, the main difference between the results from the three devices on both testing days was the average value from the B offset varies from each device. The average modulus in the B wheelpath that was recorded by the Zorn ZFG 3.1 is significantly higher than that recorded by the Zorn ZFG 3000 and the Olson. However, the I and O wheelpaths produced similar results. All three devices picked up the same trends that showed the B wheelpath having the highest modulus, the I wheelpath being the lowest, and the O wheelpath being in the middle. Table 5.10 reports the average modulus values for each wheelpath that was produced by the three LWDs. The NCAT Zorn ZFG 3.1 recorded the highest values for all three wheelpaths.

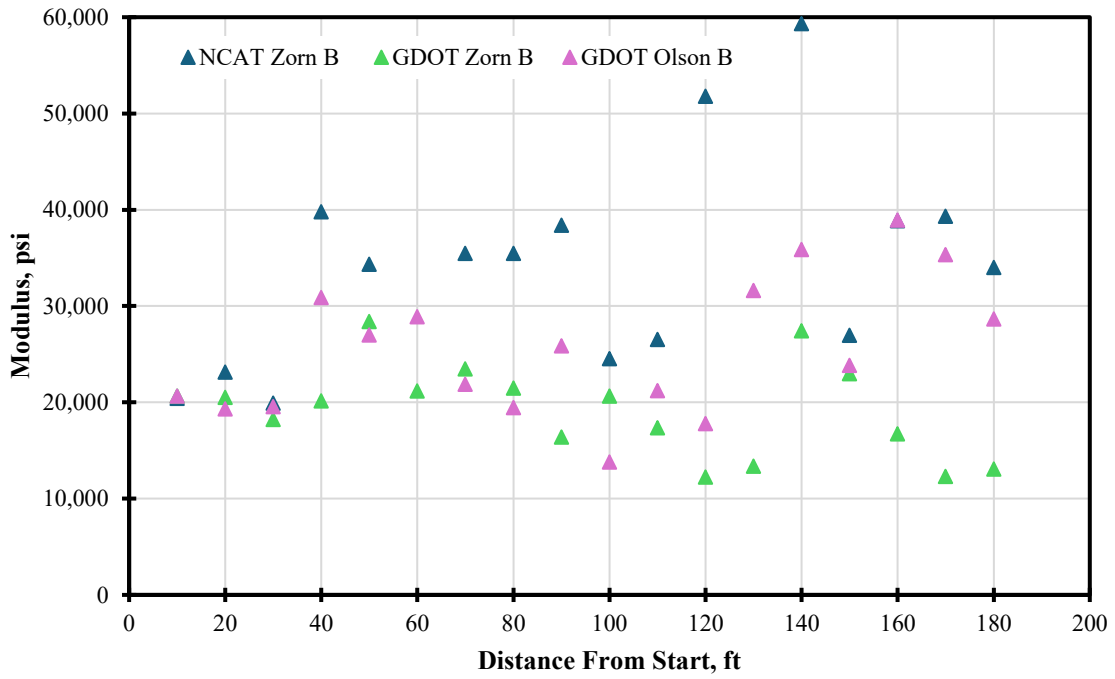
**Table 5.10: Average Modulus by Wheelpath for Each LWD**

<i>LWD Type</i>	<i>Average Modulus, psi</i>					
	<i>10/20/2025</i>			<i>3/2/2026</i>		
	<i>O</i>	<i>B</i>	<i>I</i>	<i>O</i>	<i>B</i>	<i>I</i>
<i>Zorn ZFG 3.1</i>	20,987	32,667	18,950	17,294	25,001	18,411
<i>Zorn ZFG 3000</i>	17,651	19,466	17,608	15,518	22,317	16,551
<i>Olson</i>	18,426	25,183	17,475	17,127	21,739	15,861

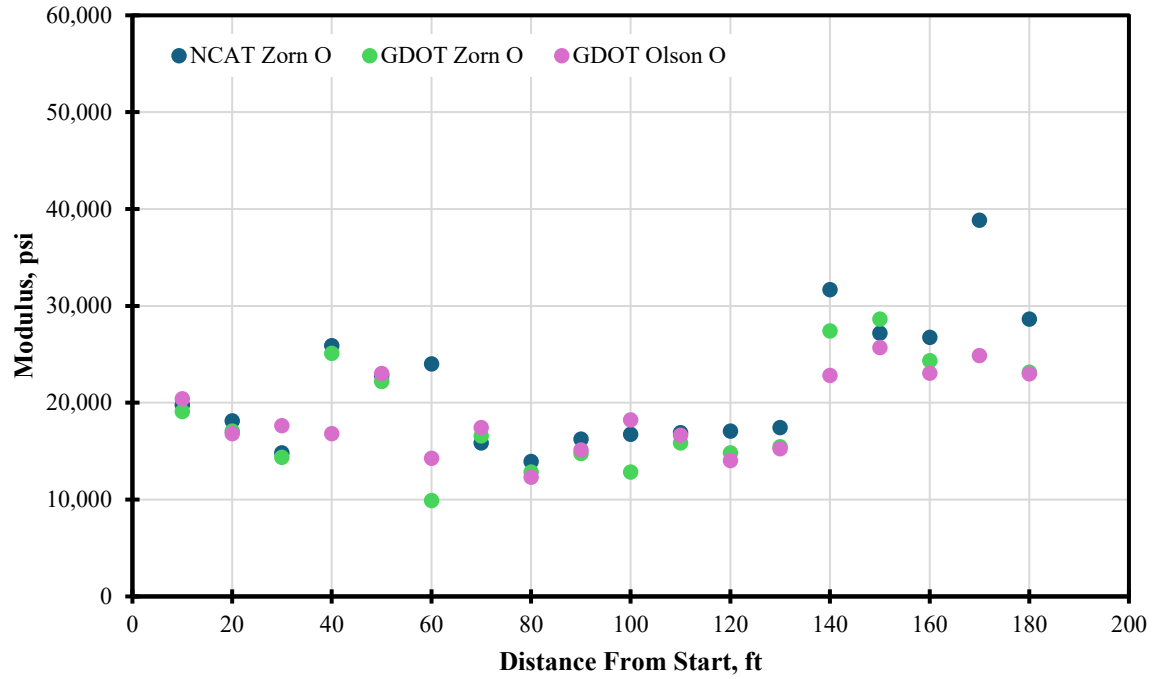
In Figures 5.14 and 5.15 the data are broken down by wheelpath offset (I, B, O) into three plots showing the distance from the start of the section versus the modulus at that testing location. The I and O wheelpaths data from each of the three devices stays relatively consistent throughout the entire section for both testing dates. The B wheelpath shows variations throughout the entire section for all three devices. This is more prominent on the test date in October. In March there was a lot less variation in the between wheelpath average modulus values. In the I and O wheelpath there is a trend near the end of the section at the start of the transition area where the modulus increases. This trend was picked up by all three devices. Figures 5.14 and 5.15 show a graphical representation of the modulus values in each wheel path over the length of the section from the October and March test dates, respectively.



(a)

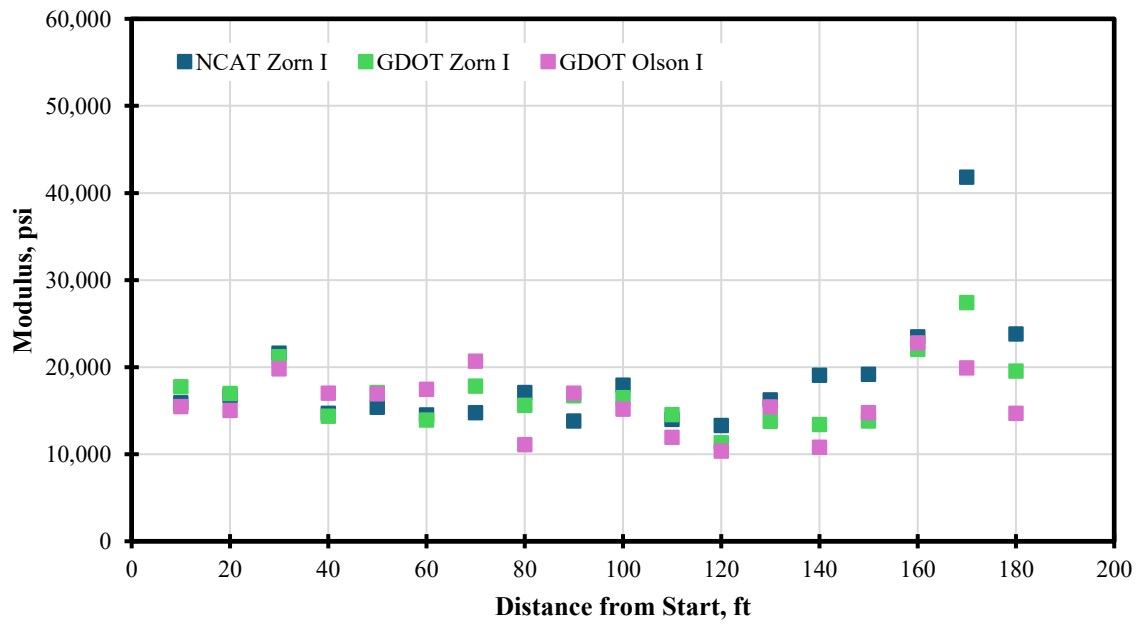


(b)

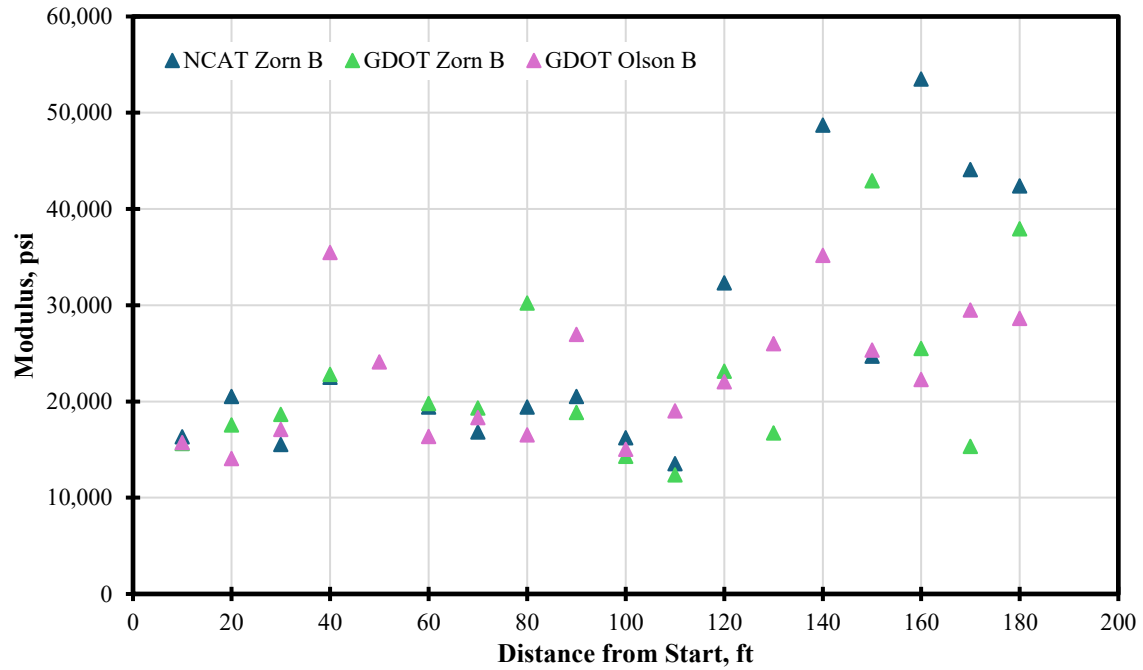


(c)

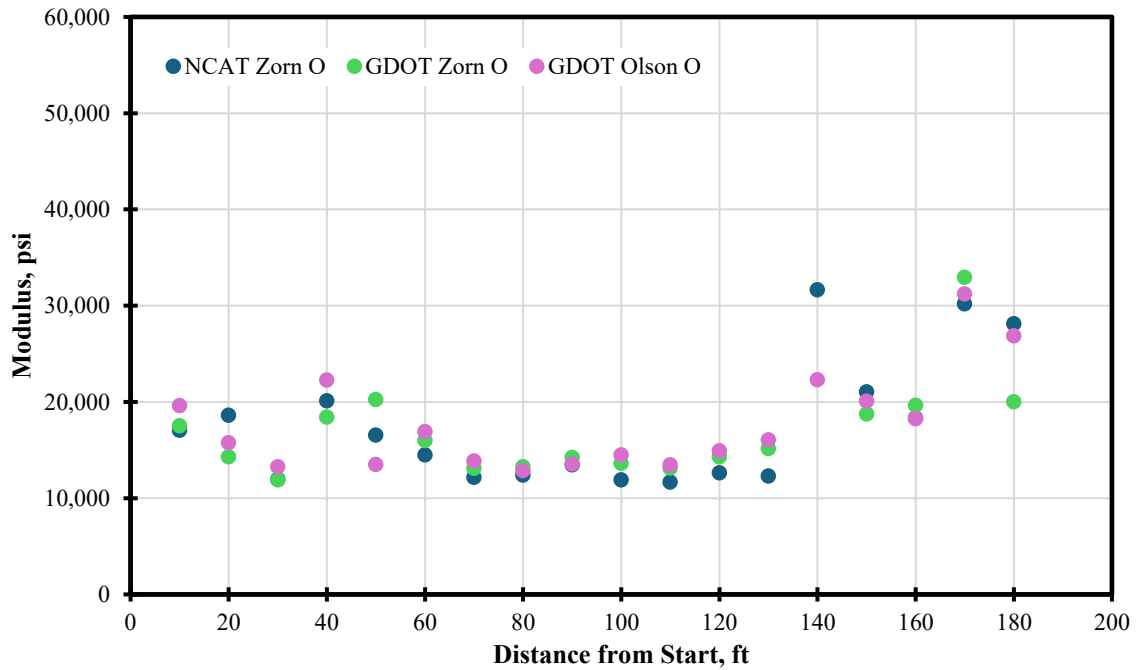
Figure 5.14: Distance From Start versus Modulus of NCAT Zorn, GDOT Zorn, and GDOT Olson for (a) I, (b) B, and (c) O wheelpaths on 10/20/2025



(a)



(b)

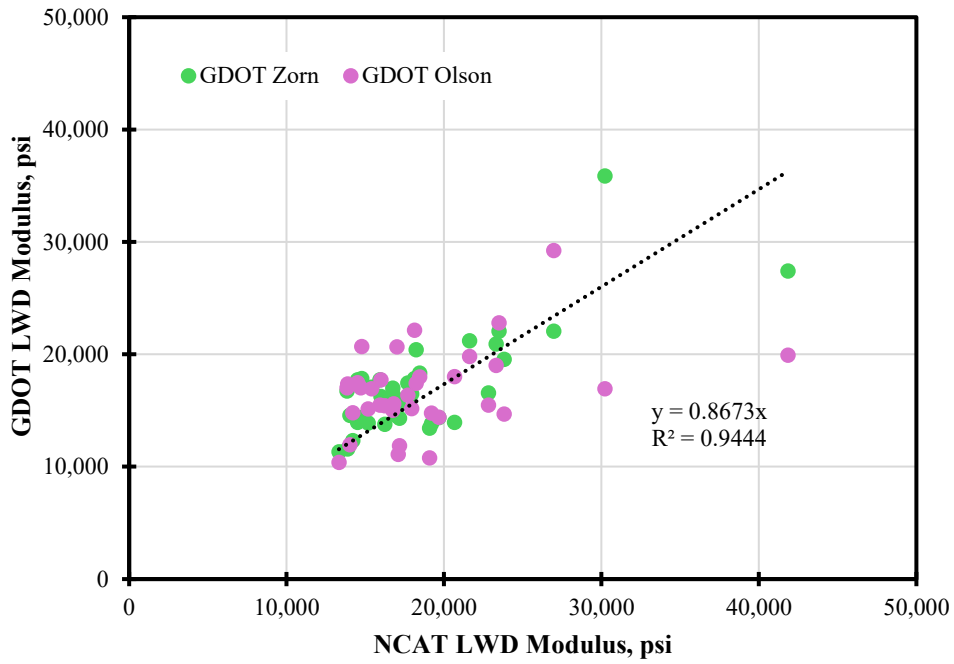


(c)

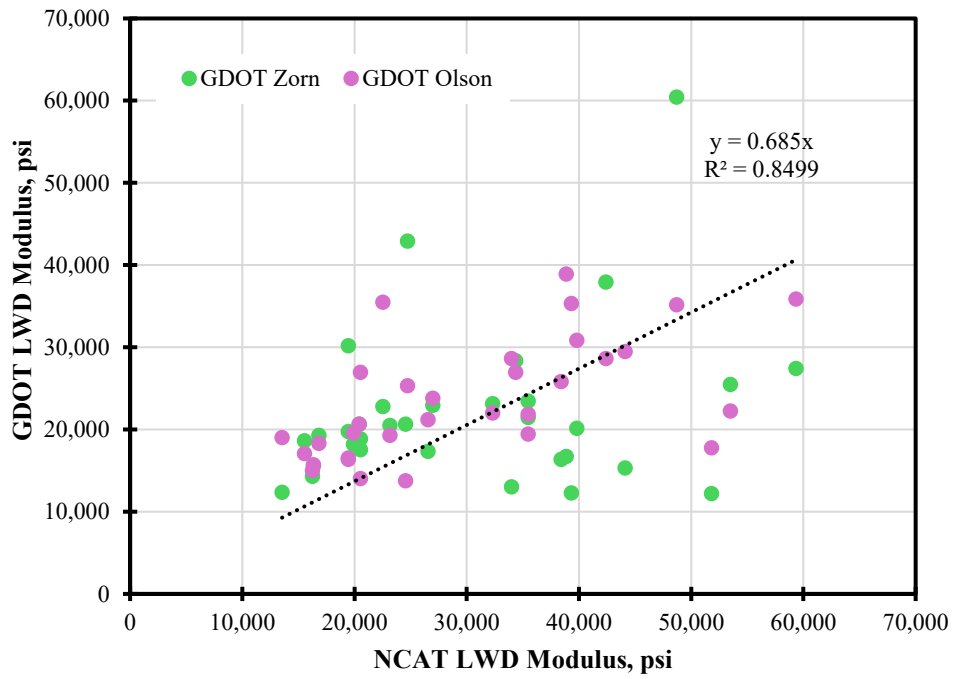
**Figure 5.15: Distance From Start versus Modulus of NCAT Zorn, GDOT Zorn, and GDOT Olson for (a) I, (b) B, and (c) O wheelpaths on 3/2/2026**

Visually, it is seen that the I and O wheelpaths all follow the same trends between the three LWDs. These two wheelpaths are subjected to the most loading and therefore the most distress and damage. The B wheelpath has the least amount of damage hence the higher modulus values picked up by all three devices. The B wheelpath also has the highest modulus values compared to the other two wheelpaths. This is a consistent trend that has been seen in all test dates and sections. All three devices did pick up this trend, despite the variability. It is likely the B wheelpath has the most variation due to the lack of distress. This is because the deflection is not significant enough in relation to the load level of the LWD and does not show repetitive results. The B wheelpath results from the March testing follow a similar trend compared to the B wheelpath from October testing. However, neither are as prominent as the I and O wheelpaths.

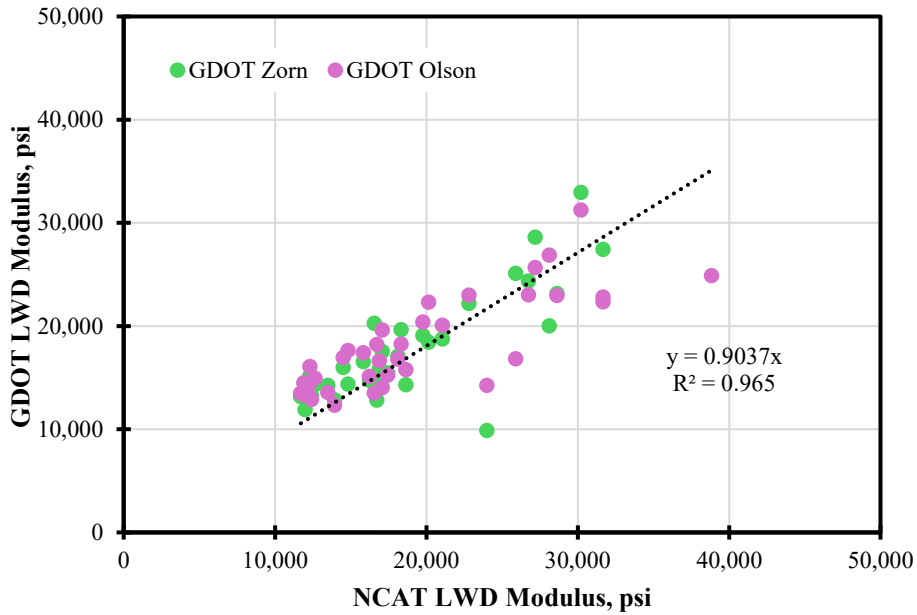
Another visual representation of the data was done by comparing the two GDOT LWDs to the NCAT LWD. This is seen in Figure 5.16. This analysis includes both test dates in each wheelpath. The three plots have a linear trendline with an intercept of zero included. The assumption was made to make the intercept 0 due to the relationship that no load would produce zero deflection from each device. With this assumption, the O wheelpath presented the best correlation with an  $R^2$  of 0.965. The I wheelpath had a slightly lower  $R^2$  (0.944) and had more variation. The B wheelpath has the greatest variability between the devices and lowest  $R^2$  value (0.849). The slopes of the I and B wheelpaths are similar to each other, this is a consistent trend throughout all testing. The two wheelpaths resulting in similar trends in both modulus and deflection with little variation when compared to the B wheelpath.



(a)



(b)



(c)

**Figure 5.16: Linear Comparisons of Both GDOT LWDs to the NCAT LWD in the (a) I wheelpath, (b) B wheelpath, and (c) O wheelpath**

Two tailed paired t-tests were performed on each of the wheelpaths on each test date comparing each of the GDOT LWDs to the NCAT LWD. This comparison was made to the NCAT LWD due to that being the device that is used throughout this research and routinely at the Test Track. A 95% confidence level was used with an alpha of 0.05. The results from this testing are shown in Table 5.11.

**Table 5.11: P-Values from Paired T-Tests**

<i>Wheelpath</i>	<i>10/20/2026</i>			<i>3/2/2026</i>		
	<i>I</i>	<i>B</i>	<i>O</i>	<i>I</i>	<i>B</i>	<i>O</i>
NCAT - GDOT Zorn	0.0583	0.00074	0.00595	0.0298	0.2822	0.5856
NCAT - GDOT Olson	0.095	0.00651	0.01492	0.0464	0.0643	0.6086

From the resulting p-values seen in Table 4.4, there are a few wheelpaths that are statistically different from the NCAT Zorn device. From the October testing, both the B and O wheelpaths from both GDOT devices are statistically different than the NCAT Zorn LWD. From the testing in March, the I wheelpath was the only offset that was statistically different than the NCAT Zorn. A globally paired t-test was also performed to ignore test date and offset and compare all results from the devices as a whole. Each GDOT LWD was again compared to the NCAT LWD. Unlike the t-tests for individual test dates and wheelpaths, bringing everything together creates more variation leading to the resulting p-values indicating statistical differences between the two devices. A 95% confidence level was also used in this analysis.

**Table 5.12: P-Values from Global Comparison**

<i>Comparison</i>	<i>P-value</i>
NCAT - GDOT Zorn	7.13*10 <sup>-5</sup>
NCAT - GDOT Olson	2.71*10 <sup>-5</sup>

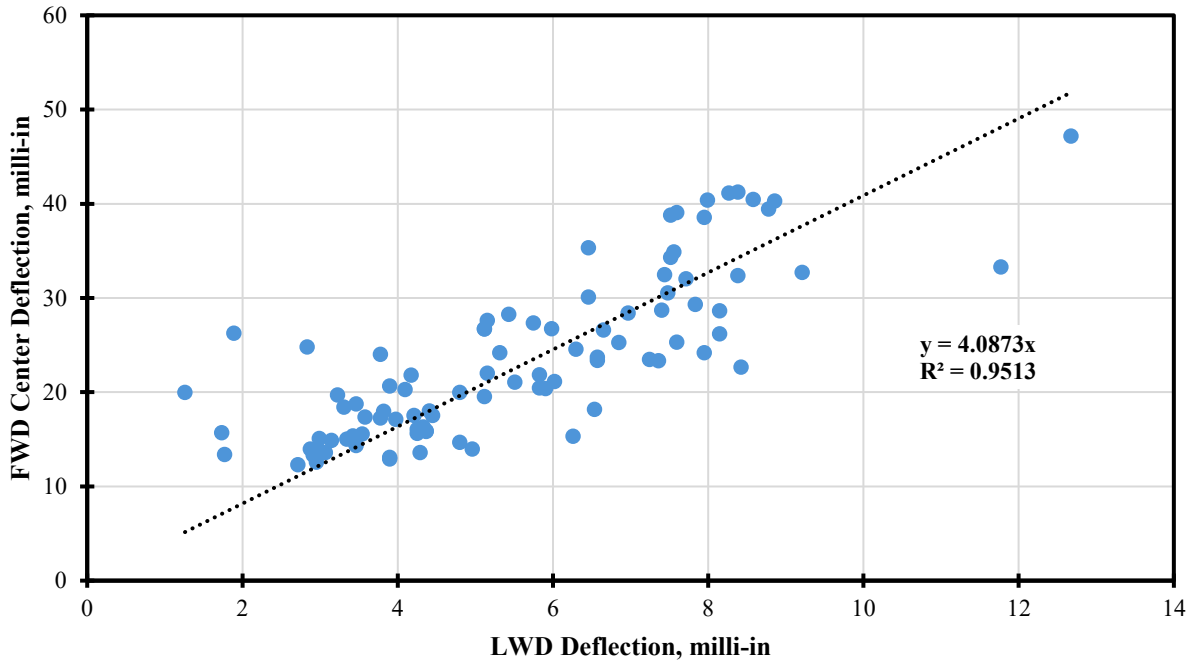
The statistical analysis from this research shows that there is variation between the three devices that were used at an all-inclusive level. The results needed to be analyzed at a deeper level (i.e. wheelpaths) to identify statistical relationships between the data. The B wheelpath is a significant contributor to creating the statistical difference between the devices. The B wheelpath is subjected to the least amount of loading compared to the other two wheelpaths leading to the increase in variability. The main takeaway from these comparisons of devices was that the NCAT Zorn produced higher values from testing compared to the other devices mainly in the B wheelpath but had similar trends in the I and O wheelpaths. Overall, it does not seem that out of the three devices, all LWDs provided reasonable results.

#### **5.4: FWD Data**

Unlike the LWD, the FWD was only run on the predetermined testing locations in each of the three offsets on the four RL locations. This means that there were twelve FWD data points within each tested section. The first two test dates were done using the Dynatest 8002 hydraulic FWD and the third test date was done using the Dynatest 8012 FFWD. Testing on all dates was done at a range of drop heights equating to loads ranging from 5,000 lbs to 12,000 lbs. The modulus was backcalculated using EVERCALC 5.0 as a three layer system representing the asphalt concrete layer, subgrade layer, and granular base layer. Modulus values were also calculated using the ZORN LWD equation. Center deflection values from the sensor directly under the FWD load plate were used for comparisons.

#### **5.5: FWD and LWD Comparisons**

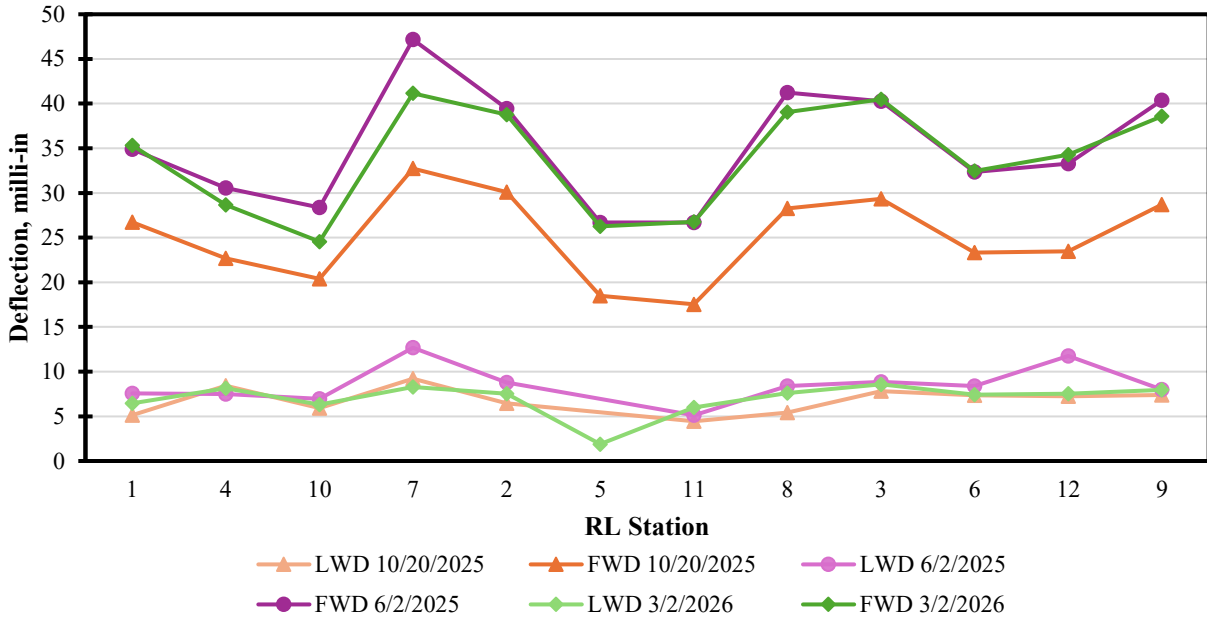
The purpose of testing the LWD and FWD on the same locations during the same time frame in each section was to establish a relationship and correlation between the two devices. The force produced by the LWD is significantly less than that produced by the FWD therefore resulting in lower produced modulus values, deflection, and zones of influence. Figure 5.17 compares the deflection produced by the LWD and the center deflection from the FWD at the load range of 8,000 to 10,000 lbs from testing in June, October, and March in all three sections. All deflection data collected on the RL locations were included in this figure.



***Figure 5.17: FWD Center Deflection vs LWD Deflection***

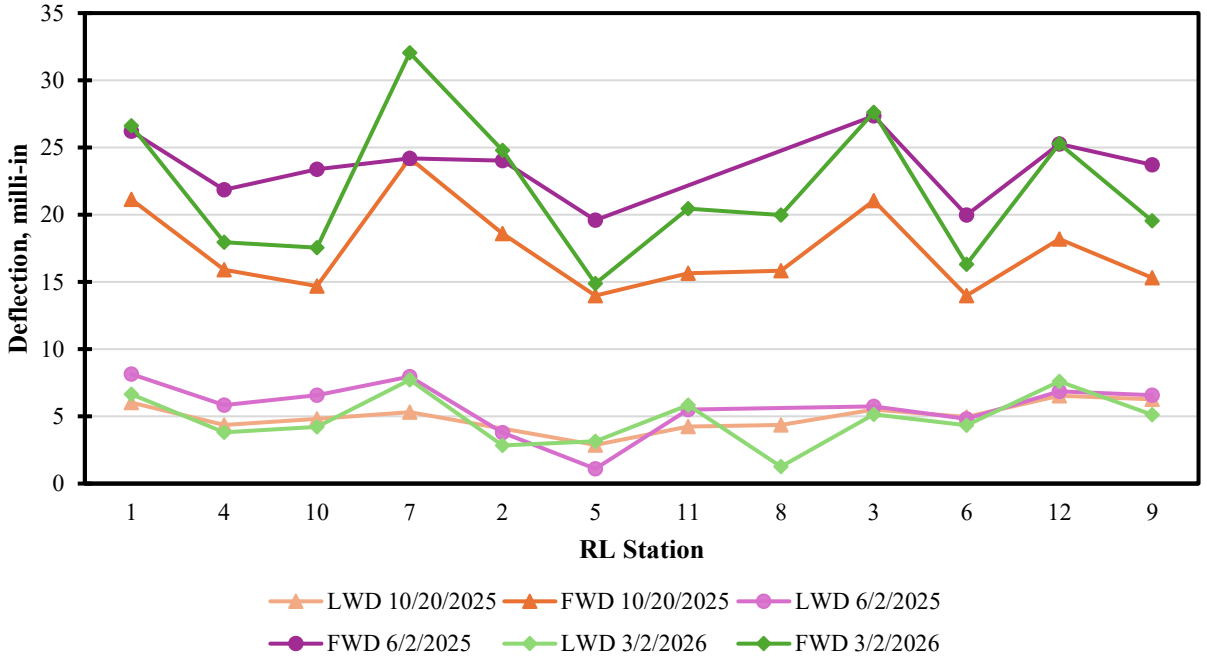
The trendline in Figure 5.17 had an intercept that was forced through zero. The resulting  $R^2$  value from these series is 0.9513, which reflects a good relationship between the two sets of deflection data despite the difference in force due to the load drops. From the deflection data, the deflection produced by the FWD in the 8,000 to 10,000 lbs of force range was around 4 times greater than the deflection values recorded by the LWD. This is due to the lower force produced by the LWD, which is only around 1,500 lbs. However, the trends produced are similar. This can be seen more clearly in Figure 5.18. Broken down by section the recorded deflection from the LWD and center deflection from the FWD are plotted for each section from each test date. Despite the LWD data being around 4 to 5 times less than the FWD, a lot of the trends are similar between the two devices. The LWD has less variation throughout testing, where the FWD shows greater differences.

### N1

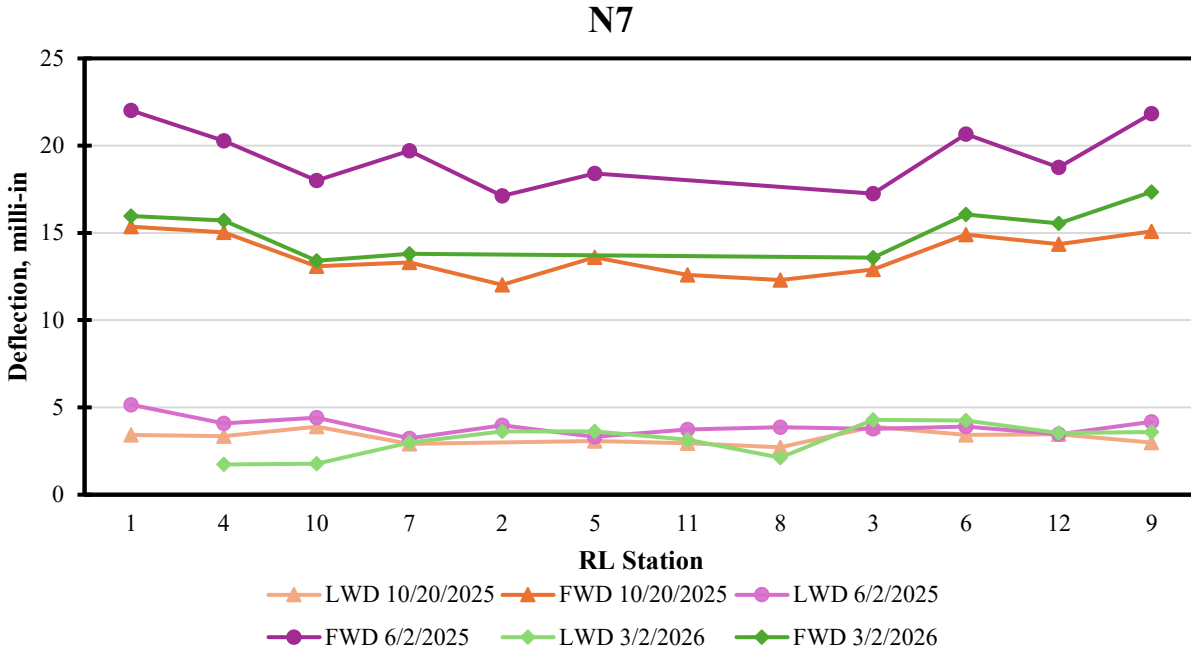


(a)

### N2



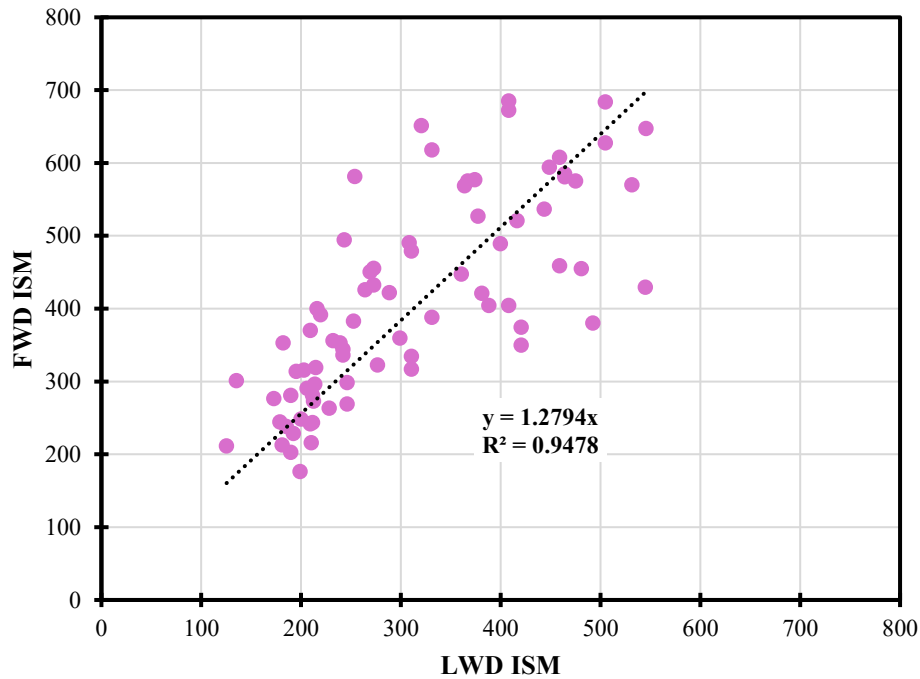
(b)



(c)

**Figure 5.18: LWD and FWD Deflection at Each RL Location in Section (a) N1, (b) N2, and (c) N7**

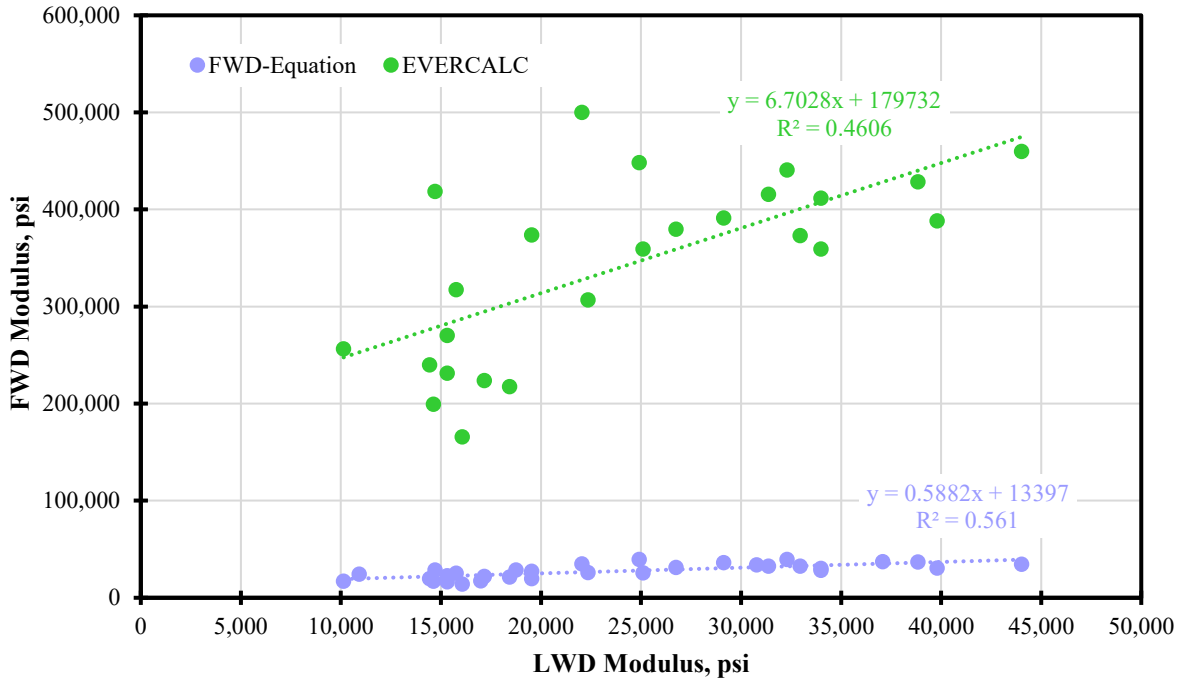
As mentioned in the literature review, one correlation metric that has been used between the LWD and FWD is the impulse stiffness modulus (ISM). This is the impact force divided by the deflection. This allows for difference in load level to be accounted for. For this analysis the deflection produced by the FWD at the load level 8,000 to 10,000 lbs was used to be consistent. Figure 5.19 shows the two ISM values plotted against each other with a trendline with an intercept set at zero. With this set interception, there is an  $R^2$  value of 0.9478, which is extremely good.



**Figure 5.19: LWD ISM vs FWD ISM**

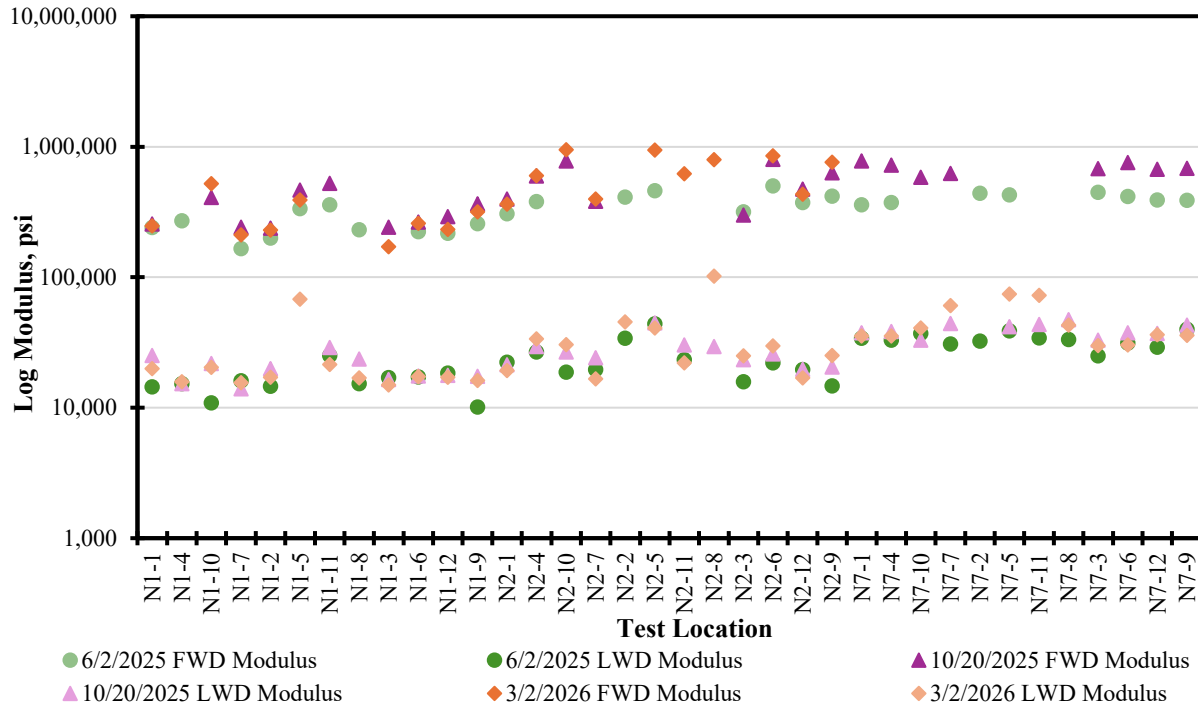
With this ISM data, a paired hypothesis test was performed to determine statistical differences between the two data sets. This test was run with a 95% confidence level. From this testing, the p-value of the two tailed test ( $3.8 \times 10^{-15}$ ) indicated that there is a statistically significant difference between the means of the two data sets. The mean of the LWD ISM being 308.5 whereas the mean of the FWD ISM was 409.14

Shown in Figure 5.20 is the relationship between the LWD modulus and the FWD modulus at the same location. The data presented only accounts for data from June testing. The series in purple is the FWD modulus that was calculated using the center deflection from the FWD input into the Equation 1 that is used in the Zorn LWD. The series in green was the backcalculated modulus value determined by using EVERCALC 5.0.



**Figure 5.20: LWD Modulus vs FWD Modulus**

By using the LWD equation, the resulting FWD values were significantly lower than that of the EVERCALC backcalculated modulus values. However, neither method produced a strong correlation between the FWD and LWD. This may be a result of lack of temperature correction or difference in applied force. As mentioned in the literature review in Chapter 2, it has been typical in other research where there was not a strong correlation between FWD and LWD modulus values, but a good correlation between the deflection values produced. Figure 5.21 shows all LWD and FWD modulus from all sections and test dates and the stark difference in magnitude of the values.



**Figure 5.21: All LWD Modulus and FWD Modulus**

As seen in Figure 5.21, all LWD modulus values are significantly lower than those of the FWD modulus values. This Figure is presented on a log scale that allows for the similarity in trends to be seen, but also the stark difference in magnitudes of the modulus values. Overall, the LWD values seem to be 10 to 30 times smaller than the FWD backcalculated modulus values. The variation within the FWD backcalculated modulus values lack the consistency of the LWD values. There generally is not a good relationship between the two sets of modulus values at the same locations. The trends may be relatively similar between the LWD and FWD, but the modulus reflected by the LWD is not representative of the typical range of asphalt pavement modulus the way the FWD is.

### **5.6: Summary of Data Under Traffic**

From the current testing that has been done with the LWD on asphalt surface layers at the Test Track, beneficial data has been acquired. The LWD has shown promising usage to identify section modulus values and distinguish the level of damage between sections. Using sections N1, N2, and N7 allowed for a range of already known damage levels, which helped to understand the relevance of the collected LWD modulus data per section. Using the LWD throughout the section allows for a more location specific representation of the pavement structural conditions. The FWD allows for changes on historic data spots and a deeper understanding of the pavement layer properties, which is extremely important and will not be replaced. However, the LWD is a very portable and versatile device to introduce a relationship between the visual deterioration and the pavement surface conditions. The sections tested were thinner (less than 6 inches) pavement sections and therefore had a better reliability for testing. The next chapter discusses the conclusions and recommendations for using the LWD at the Test Track for future testing and reconstruction efforts.

## **Chapter 6: Conclusions and Recommendations**

### **6.1: Findings and Conclusions**

Overall, the light weight deflectometer is a very versatile and beneficial device to be utilized at the NCAT Test Track. It was seen that the LWD allows for testing in multiple phases of the three year testing cycle in an easy, fast, and effective manner. From using the LWD during construction, some key findings from this research were:

- Data from the LWD can be correlated to the moisture data from the NDG during construction.
- The strongest relationship was seen between the moisture and deflection values resulting in a preliminary maximum deflection value of 0.020 inches (0.504 mm) and minimum modulus value of 6,908 psi (47.64 MPa) for the materials tested (Test Track Base).
- The use of Proctor data (optimum moisture content, dry density) would be used to establish deflection and modulus limits for materials that were not tested in this research.
- LWD testing on unbound materials produces reliable, fast, and efficient results.

During construction only 12 spots were tested per section, a recommendation would be to increase the amount of testing throughout the section to account for variability due to moisture variance or other issues on the base. From testing done during construction, there is a benefit to using the LWD as an additional QC device alongside the NDG. By using both of these devices, more characteristics and properties of the subgrade and base can be analyzed and correlated to each other. However, eventually the use of the NDG could be removed.

Post construction, for using the LWD on the asphalt surface as a structural condition assessment tool for an APT facility, some findings were,

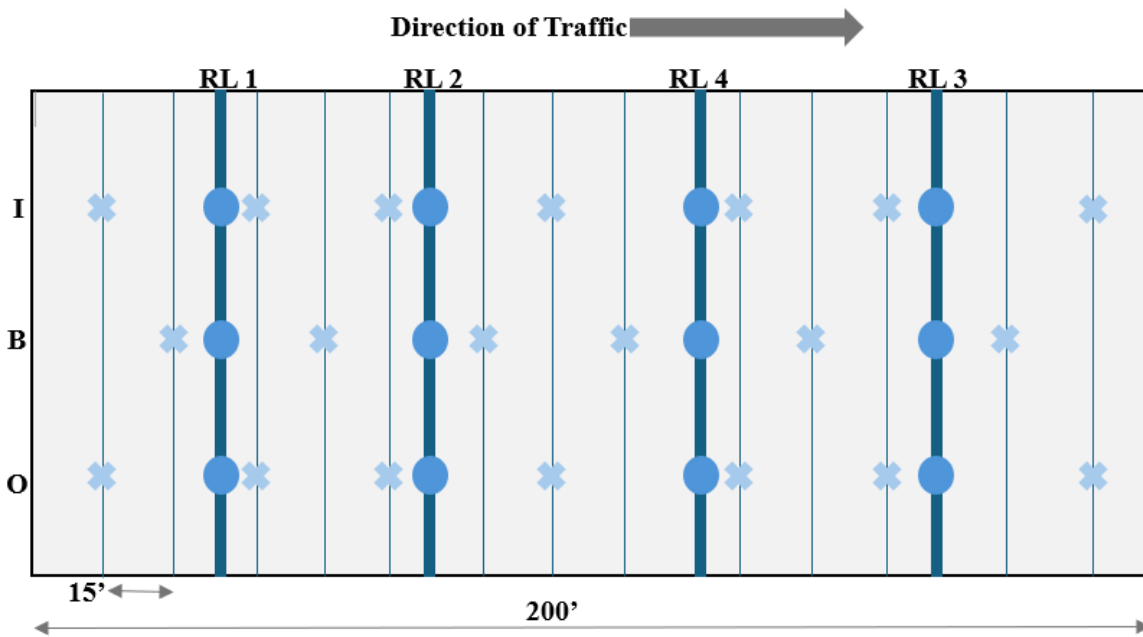
- The LWD deflection and FWD deflection results exhibit correlations. However, the FWD deflection values are 4-5 times greater than the LWD deflection due to increased load levels.
- The LWD modulus values and FWD backcalculated modulus values do not show a strong correlation.
- The LWD is a possible option for use as an APT structural assessment tool specifically for specific areas of concern on distressed areas.
- The LWD can be used on asphalt pavement layers that are relatively thin (i.e., around 6 inches).
- Sections that lack cracking distress show limitations with repeatability of LWD testing on the asphalt surface.

### **6.3: Recommendations and Future Research**

From the research done at the NCAT Test Track using the LWD during the 2024 testing cycle, more ideas for future investigations using the device have been ideated for the next reconstruction phase as well as during trafficking in the next cycle.

During reconstruction efforts, specifically on structural sections that are to be fully redone, the LWD should be used for testing the base and subgrade materials. The testing should be increased from only the 12 RL locations to 20 locations plus the 12 RLs for a total of 32 for a more encompassing evaluation of the section. If using a different material than what was used in 2024, the increased amount of testing of these materials will allow for a database to be compiled that sets baselines for the values of deflection and modulus that should be found.

A deflection value that corresponds to the optimum moisture content found from a Proctor test should be determined for each new material. Similarly to typical NDG standards, a range of deflection values should be established by using the optimum value and a range of deflection values that are of +/- 0.5% of the optimum moisture content. The LWD testing should be done in all locations that the NDG is tested to allow for consistent correlations in both QC devices. The testing schematic that was illustrated in Chapter 3 is adequate for testing alongside where the NDG is tested, however more testing locations should be included to ensure total compliance throughout the section. Since the LWD is a single plate load, more testing is needed throughout a larger section to gather information that can encompass the entire section. A recommended testing schematic is shown in Figure 6.1.



**Figure 6.1: Recommended LWD Testing Schematic of Unbound Foundational Materials**

This testing keeps the typical testing pattern of the three wheelpaths on each of the four RLs while also adding more testing throughout the beginning and ending areas in each section.

The desire to keep testing on the RLs is based on historical use of those surveyed points. For unbound foundational layers, there is not the distress like there is on the asphalt pavement surface therefore the spacing for testing increased to 15 foot increments. Also testing in a checkered pattern for the same reasoning as increasing the increment spacing. Any locations that are not included in the schematic but are visually concerning should be tested as well. Due to the ease of the LWD, adding test locations is not an issue. The quickness of running an LWD test allows for the increase in testing throughout the sections to not add a detrimental time increase compared to the NDG. By utilizing the deflection values from the LWD alongside the NDG moisture and density readings, a well-rounded QC standard that encompasses the typical practice as well as material characteristics that are relevant to pavement design can be considered. Using this method can address the failure of the asphalt surfaces that could be due to the variations in the base and subgrade materials that were not identified by the sole use of the NDG. The LWD also can be tested on areas of the section that are visually different in addition to the predetermined locations. The ease of use of the machine allows for spot testing if needed. Ultimately, it is likely that the NDG can be phased out of use once a strong, consistent practice is done with the LWD.

For post construction testing or testing during trafficking, when the LWD is used as an APT structural evaluation tool, testing needs to be performed more frequently to fully evaluate the changes in the asphalt modulus over time. The increased amount of test locations on the asphalt, similar to testing on unbound materials, is due to the LWD being a single plate load. An initial LWD reading should be done soon after construction on the asphalt pavement to establish baseline modulus and deflection values for the entire pavement section without trafficking. From this baseline the following LWD testing done on the section over time and with an increase of ESALs can help create a relationship between the modulus and the pavement distresses. By starting when

the sections are initially constructed, the changing modulus values can be monitored alongside the monitoring of the cracking and distress in each of the sections. For testing on the asphalt surface, it would be more practical to use the modulus values produced by the LWD rather than the deflection for overall evaluation to relate to the section distress. However, using the deflection values from the LWD in this instance would be useful to correlate the data to the FWD deflection values. During the testing with the LWD the pavement temperature should also be recorded to understand the effects on the viscoelastic material. The LWD was tested on three asphalt pavement sections that had very similar pavement structures. More testing could be done to see the effect pavement structure may have on LWD results when testing on the asphalt pavement.

#### **6.4 Summary**

Overall, the LWD is a beneficial device to have at the Test Track due to the variety of uses. The primary use of the LWD should be during construction on the unbound foundational pavement layers. Reiterating the objectives of this research, a testing method and minimum/maximum values were established for use of the LWD as a QC device during Test Track reconstruction efforts. A preliminary minimum modulus of 6,908 psi (47.64 MPa) should be followed as guidance for acceptance for the Test Track Base material that was analyzed in this research. Modulus values higher than this are beneficial to the stiffness of the unbound material. Other limits for Test Track materials can be established following the same methodology. Also, a method for utilizing the LWD as an APT structural condition assessment tool during the trafficking phase was established along with ranges that correlate to possible distress levels. A secondary use of the LWD at the Test Track can be used as a structural condition assessment tool during trafficking. Based on the testing

performed on sections N1, N2, and N7 values that indicate levels of distress for pavements around 6 inches. The values are presented in Table 6.1.

***Table 6.1: LWD Modulus Values and Corresponding Distress Levels***

<b><i>Distress Level</i></b>	<b><i>Average LWD Modulus, psi</i></b>
Heavy	18,319
Medium	27,270
Light	35,942

Overall, the main objective of establishing ways to implement the LWD at the NCAT Pavement Test Track were created and if used it can be curated even more for the best results.

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