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Advanced Statewide Calibration of MEPDG for NMDOT

Interim Report

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14. Abstract

This report details the activities and results for the advanced calibration project that were accomplished during the first part of the study. Field testing and materials collection from three sites (two United States (US) 54 sites, one US285/Interstate 40 (I-40) interchange) out of five proposed sites have been completed so far. The dynamic modulus (|E^*|) laboratory testing has been done. Laboratory Fatigue Endurance Limit (FEL) and resilient modulus, M_r testing have not been started yet. Few binder testing have been conducted. It can be noted that few field testing such as Dynamic Cone Penetrometer (DCP) and Clegg hammer testing, Falling Weight Deflectometer (FWD) testing on subgrade and Base layers are additional to the originally proposed work, however in-depth data analysis have not been performed yet.

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PREFACE

The research reported herein reviews information related to the advanced calibration of the Mechanistic-Empirical Pavement Design Guide (MEPDG). This includes the material collection, data collection, and literature review needed to carry out this goal.

NOTICE

The United States government and the State of New Mexico do not endorse products or manufacturers. Trade or manufactures' names appear herein solely because they are considered essential to the object of this report. This information is available in alternative accessible formats. To obtain an alternative format, contact the NMDOT Research Bureau, 7500B Pan American Freeway NE, PO Box 94690, Albuquerque, NM 87199-4690, (505)-841-9145

DISCLAIMER

This report presents the results of research conducted by the authors and does not necessarily reflect the views of the New Mexico Department of Transportation. This report does not constitute a standard or specification.

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INTRODUCTION

In the first half of this project the beginning tasks have been started or completed. This includes literature review to assist in advanced calibration, material collection from four construction projects in various districts, laboratory and field data collection on the asphalt concrete layers, laboratory and field data collection on subgrade and base layers, the beginnings of dynamic modulus testing, and supporting tests for modulus of resilience testing.

BACKGROUND

Advanced calibration is a significant task that needs extensive literature review of current and past practices. The literature collected for this project is on a wide range of topics related to calibration including calibrating the mechanistic-empirical pavement design guide (MEPDG), calibrating other mechanistic-empirical design methods, the MEPDG program, models used in MEPDG for different distresses, sensitivity studies, and reliability studies. Each of these parts of the literature review contributes guidance and information important to proper calibration. The calibration studies both for MEPDG and other programs provide guidance on common problems, task ordering, and important guidance for what calibration entails. Literature on MEPDG the program is invaluable as a tool for showing general inputs, how the program works, what testing and materials are needed, internal models and what they entail, and general knowledge about the program. Sensitivity studies take the many inputs derived from the literature on MEPDG and narrows the focus to the important inputs. These studies do this by measuring how much of a change in one input affects the change in the outputted distress. This information is invaluable to material collection and testing as it shows where the focus of the research should be. Finally, reliability studies give information on how robust the predictive capacity of MEPDG is.

MATERIAL COLLECTION

Material collection is a major tenant of this study in that the materials will be tested and some stored for later evaluation. All materials used on the chosen road section were collected from each construction project. This includes material from each pavement layer from the road being constructed (subgrade, hot mix, base course) and material that is mixed together or laid into the pavement structure such as aggregates, additives, geogrid, geotextile, etc… From the literature review the researchers know that lack of material for future testing is one of the avoidable problems in calibration studies. Once the road is built it is next to impossible to obtain true to field materials and so the quantities for materials may seem excessive but when compared to the risk of not having enough material to run all the tests the larger amounts are necessary. See Table 1 for a generalized idea of the amounts and materials needed for this project.

TABLE 1 Recommended material

Material	Unit Size	Number of Units
Binder	1 gallon buckets	7-10
HMA/WMA mix	1 20-30lb bags	75-90
HMA/WMA aggregates (sand, Fines, coarse, RAP etc)	1 40-50lb bags	3 each aggregate
Subgrade	1 40-50 lb bags	20-30 bags
Base	1 40-50 lb bags	5-10
HMA cores	1 4 or 6 inch Cores	2-3
Geosynthetics/Geogrid	Sheet/Roll	3ft X 4 ft piece/one fifth

LABORATORY AND FIELD DATA COLLECTION

For this task extensive collection of data for calibration is needed. The majority of this information is obtained from the construction crews or the New Mexico Department of Transportation (NMDOT). This information includes plans, asphalt mix specifications, soil characterization testing sheets, proctor testing results, construction practices, cut versus borrow, amounts of materials and where they were obtained, compaction information, construction timeline of our section, dates of construction and when traffic is on the pavement structure, and structural information. In addition, non-destructive field testing was performed for approximate in situ moduli values, moisture contents, dry density, compaction, spatial uniformity, and other important data. Some data was obtained from the onsite NMDOT testing labs; these included mix values air voids, maximum specific gravity, proctor results, pre-construction testing results, etc… Although, the test results were provided, often the research team did a confirmation test in the lab, this happened frequently with values such as optimum moisture content and maximum dry density. Often, the values found in the UNM laboratory were close to the field values. However, the accuracy needed was such that the laboratory values were used in lieu of the provided values especially in cases where a low spatial uniformity existed.

DYNAMIC MODULUS TESTING

Dynamic modulus testing is one of the laboratory tasks that are explicitly stated in the proposal. This value is extremely important in many of MEPDG's distress models. The testing matrix calls for 4 replicate samples for a two mix construction project (four from each mix) and 8 replicate samples for a single mix construction project. So far all of the construction projects have used a single mix for both lifts so there should be a total of 8 dynamic modulus samples per project. This task has been started and so far testing of two construction projects have been began with the completion of 3 samples per project.

MODULUS OF RESILIENCE TESTING

Modulus of resilience is a second explicit laboratory testing in the proposal. Similar to the dynamic modulus the resilient modulus is used extensively in the MEPDG for the models that include failure from or intensified by weakness in the unbound layers. The proposal does not detail an exact number of samples needed. However, given the difficulty of achieving the proper moisture content and density and the natural variability of the soils no less than 3 samples of subgrade and 3 samples of base course for each project should be done, with a recommendation to do more than 4 replicate samples for each unbound layer.

LITERATURE REVIEW

The main objective of this project is to calibrate MEPDG for Level 1 and Level 2 inputs. There have been many calibrations done in the past and in different countries. We can use previous calibration studies and literature to help guide our current calibration. Previous studies can show us many things, for example, it can show us the most efficient way to rank the tasks needed for calibration, the common issues found in calibrating MEPDG or any mechanistic-empirical pavement design program, and it can also give direction on what tasks must be completed before calibration is even attempted. Some of these pre-calibration tasks are: reading and performing sensitivity studies which provide information on important inputs and needed accuracy of testing, learning calibration techniques both specific to MEPDG and in general, learning the MEPDG models and how the program works, learning about the various pavement distress models inside MEPDG their issues, problems, and inputs, and learning about the reliability of not only the calibration itself but also the reliability of the design that is being calibrated. Without these pre-tasks the project lacks direction and the research team is more apt to make simple mistakes such as not finding important data inputs and focusing on unimportant tests while neglecting essential supporting tasks.

MEPDG Calibrations

The first important pieces of literature to obtain are MEPDG specific calibration studies. These are what follow. All of these studies and thesis are done on MEPDG calibration; they range from validation of calibration to setting up for calibration to common pitfalls found while calibrating. These are probably some of the most important literature for this project and present the brunt of the information important for correctly calibrating MEPDG.

Muthadi and Kim (1) conducted a calibration study in North Carolina. The rutting and bottom up fatigue cracking were the two models calibrated in MEPDG version 1.0. For this study 53 pavement sections were selected from the Long-Term Pavement Performance database (LTPP) and North Carolina Department of Transportation (NCDOT) databases. Excel's solver was used to calibrate the rutting model. The alligator cracking model the author was not able to calibrate in the normal manner through calibration factors; and thus had to adjust the transfer equation to calibrate it. The error was greatly reduced after calibration to less than the global error. Both models developed were retained for use in later and more advanced calibration. Thirty of the pavements were from LTPP and 23 were from NCDOT. There was one problem with the damage values, LTPP and NCDOT measure damage in two different ways and thus the damage data was not able to be consolidated. Both rutting and fatigue cracking models were determined to be satisfactory after calibration. As is common a split section approach was used with 80% of the sections being used to calibrate and 20% used for validation. Trench field testing was unavailable and thus the ratios for rutting in different layers were based on the predicted values. The only questionable part of this study that may introduce error is the statistical model for fatigue.

The following equation is the fatigue transfer equation:

$$\mathrm{FC_{bottom}} = \left(\frac{6{,}000}{1 + e^{\left(C_1 C_1' C_2 C_2' \log_{10}(D*100)\right)}}\right) * \left(\frac{1}{60}\right)$$

where $C_1 = 1.0$, $C_1' = -2 * C_2'$, $C_2 = 1.0$, $C_2' = -2.40874 - 39.748 * (1+h_{AC})^{-2.856}$, h_{AC} is the height of the asphalt concrete, FC_{bottom} is the bottom – up fatigue cracking and D is the bottom – up fatigue damage. After calibration the prediction ability was poor. The authors theorize that this is from the different damage measurement used by NCDOT; to remedy this more advanced calibration should be done with more sections. The validation showed good fit and increased accuracy due to calibration. Chi-squared tests were used to confirm the calibration was successful.

Li et al. (2) detailed the steps taken by the Washington DOT to calibrate the MEPDG version 1.0 to state conditions. The article gives detailed descriptions of preparation for calibration, needed data, inputs used to simplify calibration, results, issues, and observations they found throughout the calibration process. The authors found for calibration these things would be needed 1) specific traffic data, preferably one relevant axle load spectra for the state, 2) specific climate data 3) intensive pavement structure knowledge including used binders, soil, lift depths, material, aggregates, and subgrade soils, 4) and distress data, preferably for years preceding calibration. The authors calibrated the MEPDG using a combination of split-sample and jack knife methods as recommended by the MEPDG manual. The calibration was broken down into 5 parts: Bench Testing, Model Analysis, Calibration, Validation, and Iteration. Bench testing is primarily used to verify that the program is correctly showing understood pavement behavior; for example that binder properties and asphalt layer thickness most influence longitudinal cracking. Model analysis is used to determine the importance of the different calibration factors and the correct order of calibration. This was done using both sensitivity analysis and elasticity analysis. Elasticity analysis determines what affect the calibration factor had on pavement distress models. Calibration was performed using certain sections of road used for calibration. Next these calibrated equations were validated with independent sections around the state. These steps were then repeated through iteration until a reasonable accuracy was achieved. Through this calibration many conclusions were found. The distress models except roughness calibrated; roughness had a bug that prevented accurate calibration. This may be fixed in newer versions of MEPDG. They also found that Level 1 calibration was not possible with version 1.0 of MEPDG due to processing errors.

Hoegh et al. (3) used data from a Minnesota study section (MnRoad) and other sections to calibrate the rutting model in MEPDG. This is done through extensive and detailed comparison of predicted versus actual total rutting, asphalt layer rutting, and measured rutting. From this comparison the authors determined a normal calibration, one in which the calibration factors are the only thing needed to be adjusted, was not possible. Instead a modification to the rutting model was done to completely calibrate the model. This combined with the usual calibration greatly improved its prediction ability. The approach used by the authors was to first identify sections that have enough data to contribute to the calibration, second, data was obtained for the MEPDG inputs, third, the authors ran MEPDG, fourth, compared predicted and actual damage, and then calibrated MEPDG. The main road used was a 3.5 mile stretch of highway 94. This stretch has 31 test sections and a 26,400 vehicle per day traffic. Twelve of these test sections were used in this study. The aggregate source and subgrade type was consistent through the 12 sections, the subgrade had a clay R-value of 12. Due to the variety of rutting measurement methods used throughout the years, the authors chose to use only the straight edge measurement method. After comparing actual with predicted damage many things were found to be wrong with the rutting model. Most could not be fixed by changing the calibration factors. One of the issues found was that MEPDG 1.0, the version used in this study, had a bug in it that made a significant difference when using level 2 versus level 3 design levels. When it was looked at more closely it was found that level 2 designs did not model the binder behavior correctly, whereas level 3 did. Because of this bug the investigators were forced to use level 3 for material inputs. Further comparison found that the total rutting was consistently over predicted. What would normally be easy to correct with calibration was more difficult due to the varying degrees MEPDG would over predict base and subgrade while grossly underpredicting AC rutting. There was found no relation between the sections that were under predicted and the sections with prediction close to the total rutting. Also, the rutting model suggested 50% of the total rutting in the subgrade and base occurred in the first month under 1% of the total traffic the pavement, this is obviously inaccurate. Due to the erratic combinations of wrong predictions and reasonable predictions normal calibration was impossible. To help alleviate this rutting issue the authors developed an adjusted total rutting equation. This adjustment in total rutting did help some but more adjustment was warranted. The authors suggested a new procedure for finding the total rutting. In conclusion, abnormal rutting was found i.e. that similar sections had significantly different predicted damage. The subgrade and base had unreasonably high rutting damage in the first month. Through an adjustment to the MEPDG outputs combined with calibration the issues in the rutting model were compensated for.

Hall et al. (4) summarized the initial calibration of Arkansas MEPDG. The LTPP and PMS were used for most of the inputs to calibrate. Unfortunately, the authors found their data collection to be insufficient and suggested more in depth data collection for further calibration. Microsoft Excel Solver function was found to be useful to optimize alligator cracking coefficients. Of the many sites used for calibration 20% were randomly selected to be for validation. In Arkansas asphalt treated base is used extensively. In order to model this in MEPDG the authors either modeled it as asphalt that is moisture sensitive or as a base that is temperature sensitive; both ways create erroneous results though. The typical transverse cracking was not found to occur in Arkansas and the MEPDG results reflected this; however, there is transverse cracking by other means that isn't reflected in MEPDG. The authors did not calibrate transverse, longitudinal cracking, or IRI in this study. Default values of rutting in granular base were used in the calibration. The rutting models were improved by calibration; alligator cracking was not improved. This was known because the measured cracking versus predicted cracking was still statistically different.

Momin (5) calibrated the MEPDG to local conditions of the northeastern region of the United States. This was done by analyzing 17 pavement sections as well as data from the LTPP database. The differences were compared and coefficients developed for use in the MEPDG. The study also found a good format to follow for calibrating. It started with a literature review, then analysis of design practices followed by the MEPDG, followed by the extraction and collection of data, a "development of input data for the MEPDG along with implementation guidelines", and lastly the calibration of the MEPDG for local conditions.

Pierce et al. (6) discussed how to use existing pavement management systems (PMS) to calibrate the MEPDG program. There are four aims to this study: 1) to select a single state agency to demonstrate how this would be done, 2) find how to use PMS to create a framework for data collection, 3) populate the database with agency data, and 4) to "Demonstrate the local calibration process using agency data". In order to do this the authors found the challenge of trying to combine two different data formats one from the PMS and the other from MEPDG. The calibration process was applied in MEPDG version 1.1. To be able to use PMS data for MEPDG calibration would save time and money for the New Mexico Department of Transportation (NMDOT), construction crews and the calibrators. The chosen state for this study was North Carolina. There were many challenges with calibration including a lack of data points, the subjective way North Carolina used to measure distress, and the limited data available to the calibrators. Despite these challenges they were able to successfully calibrate the MEPDG. One aspect that may affect the results they found is the validity of the pavement condition collection procedures. Another aspect that may affect the results is how the cracks are measured.

Wu et al. (7) compared the results from MEPDG with Pavement Management System (PMS) data in Louisiana. The three modes of distress evaluated were fatigue cracking, rutting, and IRI. All of these were shown to have a strong dependency on the structure of the pavement. When comparing the PMS data over time with MEPDG fatigue cracking and IRI were sufficiently close but rutting was over predicted. Part of this study was using an optimization approach to select preliminary calibration factors for MEPDG 1.1; specifically rutting calibration factors for two types of flexible pavements. For this study only the PMS data fitting the required criteria of: a) over 5 years old, b) new or full-depth rehabilitated sections, and c) sections longer than 0.5 miles. In MEPDG level 3 design inputs were used and transverse cracking was neglected due to its rarity in Louisiana pavements. Rutting and IRI damage was found to follow a normal distribution whereas fatigue cracking did not. The AASHTO criterion was thought to be overly stringent and overly conservative, which agrees with previous work.

Quintus et al. (8) calibrated and validated the MEPDG rutting model for MEPDG 1.0 and second, to find and compare alternative transfer functions and make recommendations for the next version. Three alternative transfer functions were found to be assessed and the coefficients to calibrate the original and revised MEPDG transfer functions were determined. The research was conducted by 3 different schools working together; Applied Research Associates, Inc. from Round Rock, Texas, Advanced Asphalt Technologies, LLC, from Sterling, Virginia, and the University of Maryland, College Park, Maryland. In order to calibrate the original model which was determined from projects 1-37A and 1-40, Special projects from LTPP and full length road sections from various states were used. Three type of material properties were used to asses calibration accuracy: volumetric properties used in level 3, dynamic modulus used in level 1, and repeated-load plastic deformation tests that were used mainly for the alternative transfer functions. All of the models were evaluated for accuracy between predicted and actual rutting and goodness-of-fit. 45 pavements were used for this study; the pavements were drawn out of LTPP SPS-5, SPS-6, SPS-4, Westrack Project, National Center for Asphalt Technology (NCAT) test track, MnROAD, and I-710 Long Beach, CA. The study documents the rutting models evolution from the first MEPDG model to where it is when the study was conducted. Further material is available on the internet to showcase these changes and assist in calibration. These are: the December 2005 Facilitated Workshop: Executive Summary and Minutes, User Manual for the M-E_DPM Database, Simple Performance Test System Instrumentation, Summary of Data from the Test Sections Used for the Calibration and Validation of MEPDG Version NCHRP 9-30A, and Advanced Materials Characterization and Modeling. One of the downfalls of Project 1-37A is that all RAP, Modified binders, WMA, and other similar pavement advancements are not a part of the development of the rutting model. And the researchers for 1-37A were unable to do dynamic modulus testing which constricts the results to modeling the dynamic modulus which will produce some inaccuracies. Project 9-30 was tasked with developing a detailed and practical experimental plan for calibration and validation of the MEPDG HMA models, this can be found as the NCHRP Research Results Digest 284: Refining the Calibration and Validation of HMA Performance Models: An Experimental Plan and Database. In addition to developing this guideline Project 9-30 also developed an analytical technique called jackknifing which allows calibration and validation for limited amounts of data. The two projects 9-30 and 9-30A build on the projects 1-37A and 1-40 with applying laboratory tests. This project used three phases to accomplish its objectives: First, Database extension with testing, material collection, trench studies, etc, second, Determination of appropriate alternative transfer functions and testing matrix to test the original and 3 alternative models, and third, Laboratory testing, calibration, and validation. The researchers have found several things they have learned through this study which are helpful guidelines to future calibration studies.

1. You can never have too much material and MRL (Materials Reference Library) is useful and should be supported by researchers.
2. The differences in residual errors for level 3 which predicts Dynamic Modulus vs. level 1 where Dynamic Modulus is an input are small. This means that the differences seen between actual and predicted rut that the researchers found are not caused primarily by Dynamic Modulus as originally thought. There are other parameters that influence rutting in addition to Dynamic Modulus.
3. Stress term is recommended to be implemented but the researchers found reasonable predictions in models without a stress term.
4. Do not underestimate the value of workshops, the researchers found these as invaluable for determining ideas, concepts, and actual behavior of pavement.
5. Measurement error was found to comprise a large majority of the total error. This should be looked into in future studies and something to be aware of.
6. The depth functions in the original MEPDG were found to be reasonable, although they need thickness adjustment factors to be accurate.
7. The original MEPDG aging model showed reasonable results and needs no adjustment.

Waseem and Yuan (9) calibrated rutting in MEPDG done in Ontario, Canada. The main focus is on flexible and reconstructed pavements using PMS data. Longitudinal calibration was done, which is a unique feature of this study. This means that “the local calibration parameters in the three permanent deformation models were adjusted to predict the field observed rut depths over the whole life span of the pavement section”. Constant values of rutting ratios among the different layers were decided upon. An automatic macro-based procedure was developed for this calibration. Longitudinal calibration is more accurate and comprehensive than a regular calibration adding to the ability to quantify the significance of the values. There are three models that MEPDG uses for Rutting, one for HMA or AC, one for unbound granular materials, and the third for fine-grained soil. The transfer equations for the unbound and fine materials are different scales of this equation. An interesting phenomenon that the authors found was that when using residual sum of squares method on its own multiple optimum values can be obtained instead of one unique one. With 4 variables 17 optima local minimums were found. This makes calibrating the rutting model complicated. And thus the authors took steps to prevent multiple local optimums. The main way of doing this is to reduce the determinacy of the problem by assigning constant ratios between the percentages of rutting that comes from the different layers. Unfortunately, trenching was determined to be impractical for determining this ratio. Trenching would give more accurate results but has two main problems first, it is expensive and time-consuming and second, one trenching is not sufficient and there is a need for follow ups which is more costly and time consuming. Therefore, the authors turned to previous studies on these ratios. After analyzing many studies it was found on average 57% of the rutting came from AC, 27% or rutting came from the base layer, and 16% came from the subgrade. When comparing these values to MEPDG default models a large disagreement was found. Instead of the aforementioned values, MEPDG gave an average of 20% rutting in AC, 12% granular base/subbase layer, and 68% in the subgrade. Obviously this is drastically different from the empirical results. Due to this and finding contradicting studies and models the authors decided to use the AASHTO ratios measured in 1962; which is 32% for AC, 59% in granular layers, and 9% in fine-grained soil. In calibrating MEPDG macro-excel files were linked with MEPDG to assist with calibration. Ten reconstructed pavement sections were used. Each pavement section was calibrated individually for the 5 calibration parameters and then the parameters were compared. There was large variation found in all parameters. For AC B_{AC1} (one of the calibration parameters) the value varied from 0.162 to 0.470 this is an unexpectedly large variation. The large variation is confusing and the authors don't have a reason for such extreme variance in the local parameters. A pooled residual sum of squares (RSS) minimization was performed on all 10 section but a poor fit was still seen for average values to use as the calibration parameters. In conclusion, it was found that section by section calibration was effective but the spread of calibration parameters was large.nThis variation causes difficulties in implementing the calibration.

Other Mechanistic-Empirical Design Programs Calibrations

In addition to MEPDG specific calibrations, calibrations of other mechanistic-empirical pavement design programs in other countries were looked at. As these are not specifically related to MEPDG itself these studies were used more as general information on calibrating a design program of this complexity and were not studied in depth.

Le et al. (10) analysed and calibrated of the Korean Pavement Design Guide (KPDG) which is similar to the MEPDG. The authors proposed ideas to reduce computational power to run these types of programs. One of the ways to reduce computational power is to group similar time periods based on regularity of climate and axle load magnitude characteristics. Another is to reduce the amount of points analyzed on the road to only critical points.

Saleh (11) calibrated of the Austroads Mechanical Empirical Pavement Design, specifically the calibration and testing of the adopted Shell Fatigue Transfer Function. The calibration was done using lab testing with the suggestion of future field calibration. Calibration created a 26-27% decrease in pavement layer thickness.

Suh et al. (12) described a Korean study of accelerated pavement testing (APT). More specifically it is a use of APT to calibrate a laboratory rutting model. In APT a full size asphalt section is built and then a device rolls a truck wheel in a common axle configuration at a decided loading and speed. This study was conducted on a three layer AC with an AC layer of 30cm, subbase of 30cm, and subgrade of 180cm. Two deflectometers were set in the pavement at 12cm and 30cm. The following two equations were used for calibration using the APT. The first equation is for AC and the second for aggregate.

$$\frac{\varepsilon^p}{\varepsilon^r} = \beta_{r1} 10^{0.044} N^{0.185\beta_{r2}} T^{-0.708\beta_{r3}} ACV^{0.688\beta_{r4}}$$
$$\frac{\varepsilon^p}{\varepsilon^r} = \beta_{r1} 10^{0.171} N^{0.159\beta_{r2}} T^{-0.603\beta_{r3}} AV^{0.116\beta_{r4}}$$

where *N* is the number of load repetitions, T is the temperature in °C, AV is the air void in percent, β_{rj} are calibration factors. This equation, unlike the AASHTO 2002 equation, takes in account AC layer temperature, the number of load repetitions, and air void ratio. The KENLAYER program was used for multilayer elastic analysis to assist in the calibration. After testing the equation was adjusted to the following.

$$\frac{\varepsilon^p}{\varepsilon^r} = 10^{-1.85927} N^{0.3755} T^{1.5528} ACV^{-0.89602}$$

MEPDG Literature

This area of literature is important because it is essential to understand what is being calibrated. MEPDG itself needs to be understood; how the program reacts to inputs, what inputs are where, how the various pavement distresses are found, etc. In addition, the subcategory of specific distress models in MEPDG is important to know.

Newcomb and Quintus (13) explored pavement load response and performance relationships. Rutting and fatigue cracking were used as examples. The authors use the simplified relationship between rutting and vertical strain and fatigue cracking and bending strain to show how transfer equations relate stress and strain in the pavement to damage. The author also talks about how moduli are usually used to refine the transfer equations. Transfer equations are also known as performance equations, performance criteria, and failure criteria. Finally, the author brings up the idea of endurance limit, a strain limit that is small enough that the load that causes that strain may be repeated infinitely without failure of the pavement. This would create a pavement that could last forever, a perpetual pavement.

Transportation Research Board (14) suggested about the changes made to the MEPDG program up to the 0.900 version. These changes were found during the NCHRP study 1-40D. The digest mainly discusses technical improvements and enhancements.

Golalipour (15) tested and evaluated the parameters for rutting. One of the surprising conclusions found by the author is that the shear modulus and phase angle had little correlation with rutting. This parameter has been used for years to detail the rutting susceptibility of pavement. When Superpave was developed along with modified binders many studies found there was a poor correlation between this parameter and rutting. The multiple stress creep and recovery test (MSCR) has gained a lot of interest as a replacement; unfortunately it still has too many problems to become the main test for rutting. This study focuses on this test and how it can be adjusted in order for it to become a proper rutting test. To do this the role of binder mixtures, temperature, stress level, and number of cycles effect on rutting was investigated. This study brings into question using the shear modulus and phase angle for rutting in MEPDG.

Zhou et al. (16) did a validation study done in Tennessee where PMS and MEPDG results were compared using MEPDG version 1.1. The data that was not available for the materials was estimated using the witczak model. Both roughness and rutting were evaluated. It was found that AC rutting was more accurate at Level 1 and over predicted at Level 3. The method followed for calibration prior to this study was: data was collected, then the MEPDG predicted values were found, and finally actual damage was compared to the MEPDG predicted damage. For the climate inputs only complete climate files were used and not combination climate files. The traffic was back calculated out of ESALs that were obtained from the PMS. Nineteen sections in Tennessee were chosen for this study. Both design levels over predicted rutting, with the level combination of Level 1 and Level 2 inputs being the most overly predicted. The base and subgrade was considered the main source of over prediction. MEPDG also under predicted roughness. In this study MEPDG wasn't found to be sensitive enough to climate, traffic, and materials inputs.

Wen and Bhusal (17) investigated the possibility of using repeated loading of the indirect tensile test as a replacement of the FEL test and rutting prediction test. The test is done on a similar machine to the dynamic modulus test except the sample is turned 90 degrees. At intermediate temperatures it was found that fatigue could be found from cyclic IDT and good results were found. The authors determined after seeing the results, that the cyclic IDT could possibly be used for rutting and fatigue in place of traditional tests such as the FEL. More validation is needed though to strengthen this conclusion. If it can be used for this purpose it would be a great advantage to both construction and research. One test can then be used for fatigue, rutting, thermal cracking, and moisture damage.

Models in MEPDG

Priest (18) developed an accurate model for fatigue damage. Most fatigue models are developed by adjusting a basic model by calibration or shifting to fit laboratory or field data. One of the main objectives of this study is to relate binder and thickness of layers to fatigue performance. Eight test sections from NCAT were used for this study. In this study failure is determined to be 20% cracking of the design lane. Through literature research the authors found that fatigue cracking is dependent on the stiffness of the pavement. All lab testing has to be shifted to some degree to match field fatigue. The authors think there is a weak correlation between laboratory tests and field performance; this is due mainly to the large variation in shifting values. If there was a strong correlation these values would be much closer together instead the values vary from 1 to 400. After extensive literature the authors did their own study. When this report was written 3 of the sections had failed, all of them had been able to survive past their design life though. From these 3 sections a fatigue transfer function was developed. The authors took a previously designed equation and simplified and calibrated it to match the results of the failed sections. The final equation is as follows:

$$N_f = k_1 * \left(\frac{1}{\varepsilon_t}\right)^{k_2} \left(\frac{1}{E}\right)^{k_3}$$

where N_f = number of load cycles, ε_t = applied horizontal tensile strain, E = HMA mixture stiffness, K_i = regression or calibration constants. The authors found that one model was not sufficient for fatigue cracking. They needed one model for thin pavements, one for thick, and a third for the "rich bottom" section. Using these three models removed the need for a shift factor.

Xiao et al. (19) have been working on in the state of Arkansas. The objectives for this study are to: 1) find how the differences in distress definitions in LTPP and MEPDG affect calibration, 2) determine whether to classify longitudinal cracking in wheel path as alligator cracking, and 3) determine if the functions should be weighted to combine low, medium, and high sensitivities. The results the authors found was that the differences in LTPP and MEPDG are significant when calibrating MEPDG. Longitudinal cracking was found to be better taken as alligator cracking to have a more accurate calibration. Lastly, they found no data to suggest a weighting function was critical.

Wang et al. (20) used 12 full scale test sections to characterize rutting development in subgrade using HVS loading (heavy vehicle simulator). The 12 test sections were a mixture of 4 soil types and 3 moisture contents. In this study only subgrade type A-2-4 and A-4 soil types were used. The performance of the subgrade was found to be a function of soil type, moisture content, and applied stress condition. A secondary part of this study was to compare MEPDG predicted and observed rutting. MEPDG rutting model was found to have inaccuracies and the authors developed two models for the two soil types analyzed that reflected the observed rutting from the HVS testing much better. The authors surmise that with further validation and calibration these models will be an improvement over the current model. When the testing was done it was found that moisture content significantly affected deformation, the higher the moisture content the higher the deformation. Even a small change in moisture content from 18.9% to 21% moisture content dramatically affected the bearing capacity of the pavement. Also, the finer the soil type the more deformation was observed hence the authors determined that particle size is an important part of rutting. The authors thought that modification is needed in MEPDG since it did not have the sensitivity to take in account soil type or stress. Also, a relation was found between deviatoric stresses and rutting which is another factor that MEPDG does not take in account in the transfer equations. The authors determined this was another factor that needs to be in the modified model. When developing the MEPDG transfer equation for rutting the NCHRP 1-37A chose the Tseng-Lytton model for MEPDG. That equation is as follows:

$$\left(\frac{\varepsilon_0}{\varepsilon_r}\right) = \frac{\left(e^{(\rho)^{\beta}} * a_1 E_r^{b_1}\right) + \left(e^{\left(\frac{\rho}{10^9}\right)^{\beta}} * a_9 E_r^{b_9}\right)}{2}$$

$$\log \beta = -0.61119 - 0.017638 W_c$$

$$\rho = 10^9 \{\frac{C_0}{[1-(10^9)^{\beta}]}\}^{\frac{1}{\beta}} \qquad C_0 = \ln\left[\frac{\left(a_1 E_r^{b_1}\right)}{a_9 E_r^{b_9}}\right]$$

where ε_p = permanent strain; ε_o, β, and ρ = material properties; ε_r = resilient strain imposed in laboratory test to obtain material properties ε_o, β, and ρ, W_c = moisture content (%), E_r = resilient modulus (psi); a_1 (=0.15), b_1 (= 0.0), a_9 (= 20.0), b_9 (= 0.0) = constants. From this equation it can be seen that the rutting is mainly dependent on moisture content with stress and soil type playing a very small part through the resilient modulus. Due to this the A-4 soil rutting was over predicted and the A-2-4 soil was under predicted. This error is most likely due to the stress and soil type not taken into account and a small part from lack of calibration.

The following new permanent strain model was developed from the Uzan strain model.

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 \left(\frac{\theta}{P_a}\right)^{a_2} \left(\frac{\tau_{OCT}}{P_a} + 1\right)^{a_3} W_c^{a_4} N^{\left[b_1\left(\frac{\theta}{P_a}\right)^{b_2}\left[\left(\frac{\tau_{OCT}}{P_a}\right)+1\right]^{b_3} W_c^{b_4}\right]}$$

where W_c = moisture content, N = number of load repetitions; θ = bulk stress ($\theta = \sigma_1 + \sigma_2 + \sigma_3$), τ_{OCT} =octahedral shear stress, P_a = atmospheric pressure, a_i and b_i = constants. As can be seen the new permanent strain model does take in affect stress and soil type is adjusted by the a_i and b_i parameters. This model shows much more promise than the previous MEPDG model. This adjustment of the above model can create multiple models for different soil types.

Zborowski and Kaloush (21) did an evaluation of newer models and MEPDG's model for thermal cracking. The main focus is on rubber asphalt mixes and includes refinements to the IDT (indirect tensile strength and creep test) protocol, which was developed to evaluate thermal cracking resistance. A new crack depth fracture model is presented and evaluated. The model is evaluated through laboratory tests and rational from field observations, and is found to be satisfactory. Twenty-three mixes from 7 different projects with 13 conventional mixtures and 10 modified with asphalt rubber mixtures were used in this study. As part of this study the Paris Law for Crack Propagation was used to assist in the development of the model.

The model that MEPDG uses for thermal cracking is TCMODEL (thermal cracking model); in the authors experience it is inadequate to characterize thermal cracking. Although, TCMODEL is adequate for low temperature cracking, asphalt concrete mixtures, conventional binders, and distinct or extreme pavements characteristics; it is not adequate outside of these bounds, included in this group is modified asphalt concrete mixtures. The TCMODEL uses only the slope of creep compliance and tensile strength at -10^{0}C from IDT tests. The reason for this is the idea that the stronger the tensile strength at these temperatures the less susceptible to thermal cracking the pavement must be. This assumption is incorrect with modified AC's because while the modified AC has poor tensile strength from IDT tests, it has exceptional thermal cracking resistance. Naturally this brings into question the validity of using tensile strength as an indicator; and the authors theorize that total fracture energy may be a more accurate indicator. Thus the new model takes into account the total fracture energy and the IDT test is adjusted to supply this new variable. From these results and research from past years it shows that the old MEPDG thermal model gives irrational results. And this study disproves the link between high tensile strength and thermal fracture resistance.

Prozzi and Grebenschikov (22) compared the Hamburg Wheel-Tracking Device (HWTD) with the MEPDG predictions; investigating correlations, and studying the sensitivities of the different techniques. In this study the authors used two types of limestone mixes Type C and D. Type C is a coarse, dense-graded hot mix asphalt (HMA) whereas Type D is a fine dense-graded HMA. Within these two types the authors varied the volumetric properties to test the sensitivity of both HWTD and MEPDG. In the HWTD test a steel wheel is ran multiple times over a sample in temperature controlled water. The permanent deformation is recorded for 11 points on the wheel.

In the MEPDG the three volumetric properties needed are a) air void percentage, b) effective binder content, and c) specimen unit weight. The other inputs needed for MEPDG were obtained from LTPP in El Paso. This site was chosen because of the extensive data for this city. Only one binder was used PG 76-22.

The specific transfer function used in this version of MEPDG is:

$$\Delta_{p(HMA)} = \epsilon_{p(HMA)} h_{HMA} = \beta_{1r} K_z \epsilon_{r(HMA)} h_{HMA} 10^{k1r} N^{k2r\beta 2r} T^{k3r\beta 3r}$$

Where

$\Delta_{p(HMA)} =$ accumulated permanent vertical deformation in the HMA layer (in.)

$\epsilon_{p(HMA)} =$ accumalated permanent axial strain in the HMA layer $\left(\frac{\text{in.}}{\text{in.}}\right)$

$\epsilon_{r(HMA)} =$ elastic strain calculated at middepth of each HMA layer (in.)

$h_{HMA} =$ thickness of HMA layer or sublayer (in.)

$N =$ number of axle load repetitions

$T =$ mix or pavement temperature (°F)

$K_z =$ depth confinement factor

$k1r, k2r, k3r =$ global field calibration factors, and

$\beta 1r, \beta 2r, \beta 3r =$ local or mixture field calibration factors.

$K_z = (C_1 + C_2 * D) * 0.328196^D$

$C_1 = -0.1039 * H_{ac}^2 + 2.4868 * H_{ac} - 17.342$

$C_2 = 0.0172 * H_{ac}^2 - 1.7331 * H_{ac} + 27.428$

D is the depth below the surface (in.) and H_{ac} is the total HMA thickness (in.)

When evaluating the results from both HWTD and MEPDG it was found that MEPDG and HWTD agreed on the general trend of the results. Both showed that Type C performed better than Type D. However, when evaluating the sensitivity of rutting to small mixture properties and gradation changes HWTD found significant differences whereas MEPDG set at Level showed little change. MEPDG assumed that all finer gradations perform better than coarse which is not always true according to the HWTD data. Instead, the HWTD data showed some coarser samples performed significantly better than the fine samples. Some of the reasons MEPDG may have this error is a) only 4 sieves are used to represent gradation which may prove incomplete for accurate rutting predictions, b) since the air voids were within the allowable range of 93 ±1% random variations in the Hamburg samples may be responsible for this error, or c) there may also be environmental, aggregate type, and other affects. The authors suggest calibration and then reassessment with lab and field data to find if the lack of sensitivity problem persists in Level 1 design after calibration. Caliendo (23) did a study on the calibration and implementation of the MEPDG in Italy. The author calibrated it for both zero maintenance in the pavements life and for maintenance. Since MEPDG is not widely used outside of the United States the author compared the results with local equations as well as the AASHTO 1993 design guide to determine if it would be prudent to use the MEPDG for pavement design. He came to the following conclusions: 1) the MEPDG required thicker asphalt to reduce rutting, 2) the MEPDG was less conservative for low traffic then Italian methods, 3) MEPDG more closely agreed with the 1993 AASHTO guide than the Italian method, 4) it was determined that maintenance treatments should be taken into account, and 5) that field tests are needed to further confirm these results.

Huvstig (24) did a validation of a new rutting model in the Nordic countries. As a part of validating the model the shakedown theory was validated as well. In the shakedown theory the deformation according to loading is split into 3 different behavior zones depending on the applied stress. At the first level of stress there is less deformation with each loading, this is commonly considered to be compaction. The second level of stress is where the deformation is proportional to the load this is generally considered creep. And the third level is complete collapse otherwise known as shear flow. The first limit between the first and second stress level is called the plastic shakedown limit the second limit between the second and third stress level is called the plastic creep limit. As a part of this study it was found that there is a strong correlation between rutting and cracking and roughness. In addition the shear stress level was theorized to be the most important in pavement behavior. Eight road sections were chosen that are 10-20 years old LTPP roads.

As is known compaction begins happening rapidly and then evens out to a constant lower value over time. This would be fairly straightforward to predict except that over the course of the study the roads showed stress that would go beyond the plastic shakedown and plastic creep limit. This means that for these instances in time the pavement would not act in a predict Table way. Surprising the data showed that even the failure limit was exceeded, however, the pavement did not actually fail. Therefore, the authors theorized that the material had been plasticized. When the LTPP sections were evaluated the permanent deformation was mostly in the unbound materials. The following equation is the model used in MEPDG for permanent deformation.

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 * N^{a2} * T^{a3}$$

where ε_p = permanent strain, ε_r = resilient strain of the asphalt material, N = Number of load repetitions, T = temperature and a_i = non – linear regression coefficients.

Sensitivity Studies

Sensitivity studies are essential because these are the studies that guide material and data collection. Sensitivity studies look at how "sensitive" a computer program or design equation is to percent changes in inputs. These studies show which parameters have a large effect on the outputted distresses and which parameters have insignificant affect. Obviously, this is important in determining where the focus should be in data collection, material collection, and testing.

Li et al. (25) did a sensitivity analysis on a typical axle load spectra found in Washington State. The focus of the sensitivity analysis was on longitudinal cracking, alligator cracking, rutting, and IRI. Transverse cracking was ignored as it was found that axle load spectra (ALS) had no effect on it. MEPDG was found to under predict damage when compared to actual damage. Annual average daily truck traffic and annual growth rate changes affected the performance of the pavement more than ASL did. The three main conclusions found were 1) one typical ALS could be used to represent all of Washington State Department of Transportation (WSDOT) in MEPDG,

2) MEPDG is moderately sensitive to ALS for WSDOT pavement designs, and 3) MEPDG needs to be calibrated for Washington State conditions.

Sumee (26) did a thesis that explores the sensitivity of MEPDG to New Mexico inputs by advanced statistical methods. This study is conducted in three parts; first the author collected New Mexico specific data from LTPP and NMDOT, second the author ran a basic sensitivity analysis that changed one parameter at a time, and third the author took the most sensitive variables from the previous step and did advanced statistical analysis to determine sensitivity with interactions between the parameters. The first step was in order to find the ranges, distributions, and means of New Mexico parameters. The second step is to minimize time and computational need for the advanced statistical analysis, the most significant parameters were chosen according to MEPDGs' sensitivity. The third step was done using both parametric and non-parametric statistical analysis. It was found that the traffic inputs such as AADTT (annual average daily truck traffic) and percent of truck traffic in the design lane were significant parameters for all forms of pavement distress evaluated. And it was also determined that rutting, both surface and total, was the most severe and prevalent pavement distress in New Mexico. Rutting was determined to be highly sensitive to both AADTT, percent of trucks in design lane, and the bottom AC layer thickness. International roughness index (IRI), longitudinal cracking, and alligator cracking were found to be highly sensitive to bottom AC thickness. The MEPDG outputs were sensitive to HMA parameters such as thickness, percent air voids, binder content, and PG grade. Longitudinal and transverse cracking was found to be especially sensitive to base course, material type, modulus, and thickness. The water Table depth did not affect any of the parameters strongly so all output parameters were deemed insensitive to this. The other parameters that had little to no effect on the outputs were operational speed and design lane width. Due to time constraints not all parameters were able to be checked in this study. The parameters examined in this study were AC mix properties, AC thickness, base thickness, ground water Table depth, operational speed, traffic inputs especially AADTT, base material properties, and subgrade type. The most sensitive parameters for each distress in descending order of sensitivity are as follows:

Terminal IRI:

1. Bottom AC Layer Thickness
2. AADTT
3. Percent of Trucks in Design Lane
4. Type of Subgrade Material
5. Top AC Layer Thickness

Longitudinal Cracking:

1. Bottom AC Layer Thickness
2. AADTT
3. Percent of Trucks in Design Lane
4. Modulus of Base Layer
5. Percent Air Void of Top AC Layer

Alligator Cracking:

1. Bottom AC Layer Thickness
2. Percent of Trucks in Design Lane

3. AADTT
4. Percent Air Void of Bottom AC Layer
5. Top AC Layer Thickness

Transverse Cracking:

1. PG Grade of Top AC Layer
2. Type of Base Material
3. Aggregate Gradation of Top AC Layer
4. Aggregate Gradation of Bottom AC Layer
5. PG Grade of Bottom AC Layer

AC Rutting:

1. AADTT
2. Percent of Trucks in Design Lane
3. Tire Pressure
4. Bottom AC Layer Thickness
5. Traffic Growth Factor

Total Rutting:

1. AADTT
2. Percent of Trucks in Design Lane
3. Bottom AC Layer Thickness
4. Modulus of Subgrade
5. Tire Pressure

Rutting was found to be sensitive to the most parameters, which leads the author to conclude that the MEPDG rutting model is in severe need of calibration for New Mexico. After the research had been accomplished the author discussed the limitations of this study and what should be done in future MEPDG sensitivity studies. First, the author admits there is a lack of data for this study and that further studies should find more data on the ranges and distributions of the input parameters. Secondly, individual sieve analysis results were not covered in this study and thus the changes in gradations needs to be taken into account. Also, dynamic modulus, creep compliance, and tensile strength parameters would greatly enhance the accuracy of future studies. And finally given the sensitivity seen to traffic parameters it is essential for future studies to have as accurate as possible traffic data, particularly for the AADTT.

Diefenderfer (27) determined the sensitivity of the MEPDG outputs to HMA and binder inputs as well as data collection. In order to do this the author modeled two roads, an interstate and primary road. It was found that rutting fatigue between the three levels was different but not statistically significant. The fatigue for the primary route was over predicted. It was also found that fatigue error was always greater than distress error for all predictions. The study identified the different sensitivities of different failures for the level 1, 2, and 3 MEPDG analyses. The author recommended further calibration as well as local calibration and verification of rutting and fatigue predictive models.

El-Badawy (28) did a compilation of literature review on sensitivity studies and a study of their own to find how the different inputs affect the predicted damage. The factors were found for longitudinal cracking, alligator cracking, and rutting. The results of this study are from a single layer HMA. From the literature review it was found that base and subbase type and thickness had little to no effect on HMA rutting. Two structures were analyzed a 6 inch HMA, with a 10inch A-1-b base and A-6 subgrade and a similar section except the HMA is split into 3 subsections. MEPDG version 0.7 was used for this study.It was found that a thicker HMA helps prevent alligator cracking and subgrade rutting. AC fatigue was affected by both base and HMA properties. The AC rutting was found to be based only on HMA. Also, changing the volumetric properties in the upper and lower AC sub-layers had a small effect on rutting, especially when compared to the intermediate sub-layer. And in the case of longitudinal and fatigue cracking reducing the percent air voids affected the amount of damage more than increasing the binder content. Lastly, in the case of alligator cracking a change of binder content in the bottom AC sub-layer was more effective in improving performance than decreasing air voids.

Orobio and Zaniewski (29) studied the sensitivity of material properties inputs in MEPDG. The authors uses a space-filling approach with Latin Hypercube method to pick the values and Standardized regression coefficients to compare the values; along with Gaussian Stochastic processes and metamodels to evaluate and categorize relative importance. It was decided to purely use Level 3 inputs in MEPDG, which may limit the applicability of the study. Because of the methods used for this study all variables could be tested concurrently which saves time and computing need. The properties tested that had significant affect were binder content, as-built air voids, Poisson’s ratio, surface shortwave absorption of asphalt layers, and resilient modulus of the subgrade. Two structures were evaluated: Structure 1, has 3 AC layers on top of a permeable asphalt treated base, on top of subgrade; thus there are 5 layer inputs 3 for the asphalt layers, 1 for the base, and the last for subgrade properties. Structure 2 has 4 layers; a dense graded AC, on top of an aggregate base, on top of an aggregate subbase, and lastly the subgrade.

The evaluated parameters for structure one are:

1) Layer 1-surface shortwave absorptive, effective binder content, air voids, total unit weight, Poisson’s ratio, thermal conductivity, and heat capacity
2) Layer 2-effective binder content, air voids, total unit weight, Poisson’s ratio, thermal conductivity, and heat capacity
3) Layer 3-effective binder content, air voids total unit weight, Poisson’s ratio, thermal conductivity, and heat capacity
4) Layer 4-effective binder content, air voids, total unit weight, Poisson’s ratio, thermal conductivity, and heat capacity
5) Layer 5- Poisson’s ratio, coefficient of lateral pressure, modulus, average tensile strength at 14^0F, and mix coefficient of thermal contraction.

For structure 2 the following input parameters were studied:

1) Layer 1-surface shortwave absorptive, effective binder content, air voids, total unit weight, Poisson’s ratio, thermal conductivity, and heat capacity.
2) Layer 2-Poisson’s ratio, coefficient of lateral pressure, and modulus.
3) Layer 3-Poisson’s ratio, coefficient of lateral pressure, and modulus.
4) Layer 4-Poisson’s ratio, coefficient of lateral pressure, modulus, average tensile strength at 14 °F, and mix coefficient of thermal contraction.

It was found that 13 of the 30 input parameters for structure 1 had a significant impact on IRI; 6 of which were negatively affecting (when the parameter got larger the damage got smaller). Eleven of the 30 parameters had a significant effect on rutting 5 of which are negative. And 13 out of 30 had a significant effect on cracking. For structure 2; 9 out of 18 were found to be significant for IRI, 10 were significant for rutting, and 7 were significant for cracking.

Surprisingly Poisson's ratio had a significant impact on some of the damage. This is interesting because Poisson's ratio is normally determined to be insignificant and thus a default parameter is used in design. This study also clearly shows that the structural and material properties can have a significant effect on the MEPDG outputs and pavement performance. In conclusion the IRI was found to be most affected by resilience modulus and as-built air voids. Rutting was most effected by Poisson's ratio and resilient modulus. And cracking was most affected by as-built air voids, resilient modulus of the subgrade, and effective binder content. The author believes that MEPDG could be vastly improved if statistical software was integrated into it. If this was done, the statistical analysis used in this study would be more available. One drawback to this study was its limitation to level 3 inputs neglects the impact of the Dynamic Modulus of AC, which is a critical value for advanced calibration.

Schwartz et al. (30) did a final report for a full sensitivity study done on MEPDG. A one to one and global sensitivity were done on most of the input parameters for MEPDG. This study was accomplished on 5 pavement types, 5 climates, and 3 traffic levels. The evaluated inputs were general traffic inputs such as AADTT and design speed, layer thickness', material properties, groundwater depth, and geometric parameters such as lane width. Twenty-five to thirty-five parameters were chosen through conventional wisdom and the one to one sensitivity analysis. In this study a normalized sensitivity index was used to compare the parameters. The NSI was defined as "the percentage change of predicted distress relative to its design limit caused by a given percentage change in the design input." This essentially means the sensitivity becomes a ratio of percentage change of the parameter vs. percentage change of the distress, with the limits of the parameter being the range for that climate and material, and the limit of the distress was from 0 to the maximum allowable distress. The bound surface layer such as the HMA or JCPC was consistently the most sensitive parameter for all types of pavements. There were three steps accomplished in this study: first, a one to one sensitivity analysis was done to determine which parameters were important enough to be included in the intensive global sensitivity analysis, second, the important parameters were put through a global sensitivity analysis, and third the results were compared with previous studies. The global sensitivity was accomplished both through an ANN (neural network) and RSM (response surface method).

For just HMA the following trends were seen:

1. The HMA properties were the most consistently sensitive parameters
2. Subgrade and base properties were not as consistently sensitive parameters
3. The longitudinal cracking, alligator cracking, and AC rutting saw more sensitivity in their input parameters than seen in the IRI and thermal cracking parameters.
4. The sensitive design inputs for thermal cracking had very little overlap with the sensitive design inputs for longitudinal cracking, alligator cracking, AC rutting, total rutting, and

IRI. This is to be expected as MEPDG has thermal cracking as mainly a response to climate.
5. Thermal cracking did not occur in MEPDG with the correct binder. Thus the binder had to be adjusted to test the sensitivity of this distress.

Additionally shortwave absorption was found to be a sensitive input however there is little research in this area on testing for this parameter and how to accurately input it. In conclusion for all pavements:
1. 10,000 ANN evaluations show very well-defined single peaks
2. Bound surfaces had consistently the highest sensitivities
3. Sensitivities did not vary across the climates but magnitude of the severity of the distress did

In conclusion for HMA:
1. HMA properties, being the bound surface, were the only consistently highest sensitivities. These were Dynamic Modulus, Thickness, Shortwave absorptions, Poisson's Ratio, and δ/α.
2. Longitudinal cracking, alligator cracking, and AC rut sensitivities were higher than IRI and thermal cracking sensitivities.
3. Although δ and α have very high sensitivities these also have very small ranges which contributes significantly to the high sensitivity values seen.
4. The sensitivity of air voids and effective binder content are beyond the sensitivity of these values when considered purely in their effect on Dynamic Modulus.
5. Little to no thermal cracking was seen with the correct binder.
6. Thermal conductivity and heat capacity were found to be sensitive parameters for longitudinal cracking and to a lesser extent to alligator cracking and AC rutting. It also might be a sensitive parameter for granular layers but MEPDG does not allow this input. Note that these properties are difficult to test for and quantify.
7. Moderately stiff foundations with HMA were sensitive to operational speed for longitudinal cracking and AC rutting, this is most likely from interactions with Dynamic Modulus aging over time in model though.

In addition to the above conclusions there were some surprising conclusions that the author did not expect to encounter:

1. Poisson's ratio was a highly sensitive parameter
2. HMA unit weight was a highly sensitive parameter
3. Thermal cracking was sensitive to δ

The authors do not know why these results were found. The Poisson's ratio sensitivity is concerning due to the fact that this is difficult to quantify and is usually a default value.

Cooper et al. (31) did a sensitivity study on MEPDG. The sensitivity analysis was done at three levels low, medium, and high with 5 input parameters: traffic level, HMA thickness, dynamic modulus, base course thickness, and subgrade type. A full factorial design was done using the statistical analysis software PROC FACTEX. This sensitivity analysis was done with both single factors and multiple factors. This study has a 2-fold purpose first to do a full factorial analysis and second to identify interactive effects. Many studies have looked into sensitivity but neglected multiple factors interacting together; the authors consider this to cause the studies to be incomplete. Other sensitivity studies also had the drawback of being based on only one project creating shallow results. Cooper et. al. looked at three methods of sensitivity 1) screening methods (that only give the general ranking of the parameters), 2) local sensitivity analysis (only one variable is varied at a time), and 3) global sensitivity analysis (varying multiple factors together to determine interactive affect). Mainly 2 interacting factors were looked at as 3 interacting factors is too rare to consider.

To maintain simplicity the authors analyzed a basic three layer system with base, subgrade, and HMA. The HMA was varied through 3 types of typical Louisiana mixtures. After the analysis was completed the following results were obtained:

1. Combined parameters had more affect than non-combined
2. Traffic Level was the main factor in all types of distresses analyzed
3. In IRI the second most determining factor was HMA thickness and the third was subgrade strength. For combined affects, first was traffic combined with HMA thickness, second was traffic combined with dynamic modulus, and third was base thickness and subgrade type
4. In fatigue cracking, second single factor was HMA thickness and third was base thickness. The combination factors was first traffic and HMA thickness, second was traffic and base thickness, and third was traffic and subgrade
5. Total rutting was secondly affected by HMA thickness and thirdly dynamic modulus. And the combined factors were, first traffic and HMA thickness, secondly traffic and subgrade type, and thirdly traffic and base thickness
6. For AC rutting dynamic modulus was second most affective and third was HMA thickness. For combined affect first was traffic and dynamic modulus and second was traffic and HMA thickness
7. Top down or longitudinal cracking was secondly affected by base course and thirdly by dynamic modulus, and the combination was firstly traffic and base thickness and secondly base thickness and subgrade type

The longitudinal cracking function was brought into question by the authors due to the fact that MEPDG bases it on a traditional model that gives the base course thickness more affect than would be expected in the field.

Orobio and Zaniewski (32) did a MEPDG sensitivity analysis using Latin Hypercube, rank transformation, standardized regression, and Gaussian stochastic methods. The main focus of this study is traffic inputs and their affect on roughness, rutting, and bottom-up cracking. This study takes in account techniques specifically for complex computer codes. This study also uses a space-filling approach that spreads the inputs over a predetermined input space. The Latin Hypercube method was used to find these values in the input space. In addition, metamodelling techniques using multiple regressions with standardized regression coefficients and gausian stochastic processes to characterize importance of inputs. However, the metamodels are only applicable and reliable when the data is linear and presents a good fit; which is why it is combined with rank transformation which does not have this limitation. And Gaussian is the third form used in this study to cover the rest of the data found. Two structures were used for this study. Structure 1 is a

3 layer AC on top of a permeable asphalt treated base. Structure 2 is a dense graded AC, on top of an aggregate base, and aggregate sub-base. The outputs were analyzed through metamodels and rank transformation and then Gaussian was used to verify the results. Eight of the ten inputs studied showed significant affect on the corresponding damage. Standardized regression coefficients were used to rank the sensitivity since this is a unitless variable. Non-linearities were observed in the pavement structure 1 for rutting and IRI damage. The inputs of percent truck in design direction, percent heavy vehicles, percent truck in design lane were positively significant. For structure one the two-way AADT, traffic growth, dual tire spacing have the highest affect. And traffic wander and operational speed had a negatively significantly affect for structure 1. Axle width and mean wheel location did not have a significant effect on pavement performance. It was also recommended that due to the insignificant effect of axle width and mean tire location these inputs can be left as level 3 inputs. The most influential factors for both structures is traffic wander, percent heavy vehicles, percent truck design direction, dual tire spacing, traffic growth, and two-way AADT. Because of the importance of these factors it is critical when calibrating to use local values that are as close to the true values as possible.

Reliability Studies

And lastly, reliability studies show the strength of the prediction of the program. MEPDG can design at different reliabilities depending on the importance of the parameter. There are two types of reliabilities one is how reliably the program can predict what will happen in a certain confidence range and the second is the reliability in terms of probability that the predicted performance will match the actual performance in the field. The reliability of MEPDG itself is of most importance here. If the program says it is using a reliability of 90% for a certain distress design but due to the programs equations it is impossible to have above an 80% reliability that is important to know.

Baderscher and Pretsch (33) brought into doubt the practice of using non-selectivity of sensors in an array. And they highlight the dangers of inadequate data-selection methods. Many studies the authors found took good data but by misapplying or using the wrong methods come up with erroneous results. One of the common errors is the researcher does not understand an important assumption in the least squares method. The least squares method assumes that the data is homoscedastic; this means that the variance of the different data points is the same. Another common error is fitting data that has been transformed. This is a mistake because the transformed data is not going to have the same relationships as the untransformed data. Most importantly the transformed, or linearized data, is no longer homoscedastic. Once the homoscedasty is lost the data cannot be evaluated with a linear fit. Another common error is misusing the correlation coefficient. This is the R value calculated in excel and most programs. And it is assumed that the closer to one the value is the better the fit is. The meaning of the correlation coefficient is entirely ignored though. The correlation coefficient is: "It (the correlation coefficient) indicates the strength and direction of a linear relationship between two random variables on the basis of a two-dimensional normal distribution" and uses the following equations:

$$\frac{dP}{dx_1 dx_2} = \frac{1}{2\pi\sigma_1\sigma_2\sqrt{1-\rho^2}} e^{\frac{-1}{2(1-\rho^2)}\left(z_1^2-2\rho z_1 z_2-z_2^2\right)} \quad with\ z_1 = \frac{x_1-\mu_1}{\sigma_1} \quad and\ z_2 = \frac{x_2-\mu_2}{\sigma_2}$$

where, ρ is the correlation coefficient for the two random variables, x_1 and x_2, which have the means μ_1 and μ_2 and the variances σ_1^2 and σ_2^2 with P standing for the probability

Another error is extensive use of non-selective sensors. Using these strongly amplifies errors. When combined in an array these errors are amplified even more. Another method that can create errors is using data handlers without evaluating the results. Otherwise, you can end up with miracles, results that could not possibly happen. Another error from data handlers is creating chance correlations; i.e. correlations that do not necessarily exist in real life but can be thought to. Chance correlations can be minimized by keeping the number of parameters consistently lower than the degree of freedom and the number of parameters should not approach the number of observations.

Haider et al. (34) looked at how weigh in motion (WIM) errors change the ALS and how that in turn affects the predictions and reliability in MEPDG. The authors broke the errors into systematic errors and random errors and then investigated the influence of these errors on the design reliability for different distresses. There is a complete breakdown of WIM errors in this report. It was found by the authors that there needed to be a lower threshold for negative bias in MEPDG and a reliability analysis to assure sufficient reliability against cracking.

Tarefder et al. (35) determined the reliability for subgrade strength and flexible pavement design. Of most importance is determining the R value used as a MEPDG input. The authors used Risk 4.5 statistical software to accomplish this aim. The 3 main objectives are 1) to compare AASHTO and MEPDG reliabilities, 2) Evaluate the R value for reliability, and 3) to determine alternative designs to mitigate different distresses. The study used data from 6 sections in New Mexico. The authors concluded that the probabilistic Design Method overdesigned compared to AASHTO (1993). Analyzing the reliability revealed that a single R value for all inputs leads to an inefficient design compared to multiple reliability variables assigned to their corresponding inputs. And increasing the minimum R value for subexcavation does not meet the design requirements efficiently. It was also found that the effect of binder PG was negligible on bottom up cracking but substantial on rutting reliability.

Smith and Diefenderfer (36) compared the default traffic data with specific traffic data in terms of prediction accuracy. There are two main purposes for this study: first, to develop specific site data and second, to compare predicted distress from typical values and site-specific data to determine the difference in design. It was found that longitudinal and fatigue cracking had little difference between default and site-specific inputs. The authors also found that when considering monthly adjustment factors, vehicle class distribution factor, and number of axles per truck no statistical difference were found. Rutting difference between the two data inputs was statistically different though. In addition, axle load spectra were found to be significantly different when comparing specific and default data.

DISTRICT PROJECTS

In order to calibrate MEPDG for New Mexico we need construction projects from around the state. It was decided to have at least one construction project per district to fully represent the variety of material and construction techniques in the different areas of the state. At the time of this report three districts had a representative construction project chosen. Each project had a representative station within the project where the majority of the material collection and field testing was accomplished. This was done to ensure that when calibrating there was one set of data inputs for one set of pavement distress. Originally, testing and material collection was done over an area of 2-3 miles. After considering the consequences of a large area with varying material and distress conditions, it was determined that the testing and material areas be confined to a 500-700 foot section.

Non-destructive field testing was added to the calibration project. These tests give us a field comparison for the laboratory values obtained, an idea of the spatial uniformity in the horizontal and vertical direction, water content, density, compaction, and other useful information. The tests that were performed were dynamic cone penetrometer testing (DCP), falling-weight deflectometer testing (FWD), Clegg Hammer or Impact Hammer testing, and densometer testing. DCP Testing is performed by using a rod with a standard sliding weight attached to the top and a cone to penetrate the soil on the bottom. The weight is lifted up and dropped from a standard height which causes the cone at the bottom of the device to be forced into the ground. The weight is dropped multiple times till there are enough blows to determine the soil characteristics or the cone has reached a depth of interest. With each blow the new depth of the device is recorded. In the field the total number of blows varied from 20 to 40. The depths and corresponding blow numbers are then plotted in excel where a best linear fit is applied. If the soil is uniform there should exist only one slope. However, this is not always the case as we will see later. If the soil has discontinuities then the data is split into lines with uniform slope. The slope is considered the DCP value and is usually measured in mm/blow or in./blow.

This value has been related to modulus of resilience of the soils in the current edition of MEPDG using the following equations (37):

$$CBR = \frac{292}{DCP^{1.12}}$$
$$Mr = 2555 * CBR^{0.6}$$

where DCP is the dynamic cone penetrometer value in in./blow CBR is the California Bearing Ratio, used as an intermediate value in these equations Mr is the Modulus of Resilience Value in psi The modulus of resilience values were calculated in this way for all the DCP tests done in this report. Also, due to the more fragile nature of the DCP equipment this test was only done on the subgrade. This test was performed in accordance with ASTM Standard D6951/D6951M-09.

The second non-destructive test we did was Clegg Hammer or Impact Hammer. In this test a standard weight is attached to an acceleration measuring device. The weight is dropped from a specified height one or two times in order to create a pre-disturbed area for the hammer to hit. This is done because before the seating blows there is loose soil and excessive variability on the contact surface not related to the impact value of the soil. Although the standard says to perform four drops of the hammer at one testing spot, we performed 7 drops. We did this because it was theorized that four drops may be insufficient to create a good seating platform and a greater, more representative value may be missed without the extra blows. All of the blows were recorded and analyzed. The value obtained from the measuring device attached to the hammer is the deceleration in units of 10's of gravities or g. The standard for this test is ASTM D5874-02. As this is an impact test it was only done on the unbound layers of the pavement structure.

The FWD test is a powerful test for finding approximate elastic moduli values for all pavement layers. Once again, we have a weight dropped on the surface of the asphalt structure. This weight is high enough to produce deflection in the surface of the road; sensors radiating out from the point where the weight is applied measure this deflection at varying distances. These deflections are combined to create what is commonly called the deflection basin. From the individual deflections or the basin itself the modulus value for all the layers below the surface can be determined. These calculations are usually accomplished using a computer program that analyzes and manipulates the data according to inputs and engineering judgment of the analyzer.

This test provides the most information given it can be done on all three layers and can give information at greater depths then either of the previous methods. The major drawback to this test is the equipment is significantly bigger and takes more training to operate. The FWD for this project was obtained and operated by NMDOT personal. The standard for this test is ASTM D4694-09.

The final testing that was done on the unbound layers was nuclear densometer testing. This test was operated by trained personal that does this testing for the individual construction projects. Given the amount of training needed for testing that uses nuclear principles this was not a test that the research team could safely do. From this testing we were able to obtain the moisture content, compaction, and dry density of the subgrade and base course during most of our field testing.

Each construction project had its own construction team, material collection, data collection, field testing, and timing. The districts that have a construction project where the majority of field research objectives are met are district 2 with two reconstruction projects, district 5 with one intersection reconstruction project, and district 3 with one interstate reconstruction project. Even though district 2 has two projects one of the projects was used strongly for troubleshooting and training and so the amount of data and field testing is not complete.

The project in district 3 is on Interstate 40 approximately 30 minutes west of Albuquerque. This is an extensive project that was used for a previous study and thus has extensive data collection done. This is a major advantage as some of the values of temperature, climate, stress, stain, etc. will be known with a much greater accuracy than can be obtained in other projects. These values can be advantageous to calibration in the short and long term. As this project was accomplished before the current calibration project, obtaining the data from the previous researchers is still underway.

US54 SOUTH CN G3A92

This project is in the southern part of New Mexico approximately 25 minutes south of Carrizozo on US54. The construction company in charge was FNF from Arizona. This project consisted of realigning the one-way highway from milepost (MP) 94.23 to 107.10. As such, the old highway was milled and used as reclaimed asphalt pavement (RAP) for the base course and hot mix asphalt (HMA) on the new road. The entire section was constructed as a new pavement structure from the subgrade up.

The construction was done as follows:

1. Subgrade was tested and the estimated R-value was obtained and checked for compliance, the soil was then either replaced with borrow or compacted to 100% for a depth of 6 inches.
2. Base course was laid by first laying a three inch lift of virgin aggregate and then a 3 inch lift of RAP. The two layers were then mixed in the field to an assumed mix ratio of 50/50 RAP and virgin aggregate.
3. HMA was laid in two lifts of 3 inches each for a total depth of 6 inches

It should be noted that the technical panel disagreed with the construction team means of mixing the RAP and virgin aggregate for the base course. The true mix ratio may be different then the 50/50 that is expected; in addition, some subgrade may have been caught by the construction equipment and mixed into the base course causing further inconsistencies. The dates of interest for this project are March 4th through March 24th, 2014.This was the first project the research team did material and data collection on and thus there are some inconsistencies when compared to the later projects. First, there are several collection and testing sections, whereas in the later projects there is one main station where the majority of collection and testing takes place. Figure 1 shows how spread out the testing and material collection for this project is.

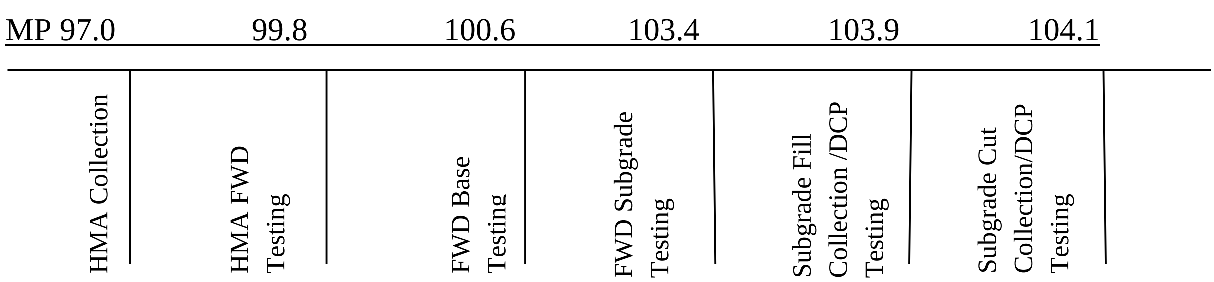

FIGURE 1 Testing and material collection schematic for US54 South

The spread of testing and material collection is 7.1 miles; and the FWD testing was spread over 3.6 miles. While not ideal, this did provide significant data as to the spatial uniformity of the unbound layers. Also, note that the FWD testing MP did not correspond with any sample collection MPs. Only the DCP tests were done on collected materials for this project.

Materials Collected

The structure for this pavement is simple with only subgrade, base course, and two layers of HMA with both layers the same mix. Due to the simplicity the amount of materials collected was straightforward. The additional material collected was subgrade material in the area we did DCP testing on borrow material. The material collected for this project is in Table 2.

TABLE 2 Material collection for US54 South

Material	Unit	Number of Units	Milepost Collected
HMA	20lb Bags	70	97.0
Subgrade Fill	50lb Bags	3	103.9
Subgrade Cut	50lb Bags	7	104.1
Base	50lb Bags	3 each type	Stockpile
Aggregates	50lb Bags	3 each type	Stockpile
Binder	2 Quart Cans	2	Hot Mix Plant
Cores	3in. ht. by 4 in. dia.	3	Testing Station

Data Collected from Laboratory and Field

FWD Testing Results

FWD testing was done on three points spread out on a three and a half mile section (MP 99.8-103.36) of the project on subgrade, base course, and full depth HMA. As mentioned previously this project had much higher distances between testing locations and thus the FWD testing was done on the same day with subgrade being done first, followed by base course, and finally HMA. This spread gave us a good opportunity to check spatial uniformity in all three layers. When the results were compared; it appears that there was decent agreement between the three materials and the multiple testing points as shown in Tables 3 and 4.

TABLE 3 FWD comparison of 6 points on each material

		FWD on	FWD on	FWD on			
		Subgrade	Base	AC			
		6 kip	6 kip	6 kip			
		E,subg	E,subg	E,subg	Mean	SD	COV
Subgrade	0	23.32	18.82	26.87	22.20	2.26	0.10
	5	24.62	24.05	29.59	Mean	SD	COV
	10	23.40	23.87	28.57	20.82	2.57	0.12
	20	18.11	18.11	28.53	Mean	SD	COV
	25	21.77	20.67	27.73	27.76	1.51	0.05
	30	21.96	19.39	25.30			
		E,base	E,base				
Base	0	76.62	50.49				
	5	64.93	42.24	Mean	SD	COV	
	10	69.49	101.40	65.68	7.76	0.12	
	20	68.93	88.52	Mean	SD	COV	
	25	58.89	94.70	79.29	25.99	0.33	
	30	55.19	98.37				
AC	0	746.53					
	5	735.79					
	10	478.03					
	20	579.34	Mean	SD	COV		
	25	459.00	576.52	135.10	0.23		
	30	460.44					

TABLE 4 FWD comparison of values on subgrade to values on base

		FWD on	FWD on	FWD on			
		Subgrade 6 kip E,subg	Base 6 kip E,subg	AC 6 kip E,subg	Mean	SD	COV
Subgrade	0	23.32	18.82	26.87	23.01	4.04	0.18
	5	24.62	24.05	29.59	26.08	3.05	0.12
	10	23.40	23.87	28.57	25.28	2.86	0.11
	20	18.11	18.11	28.53	21.58	6.01	0.28
	25	21.77	20.67	27.73	23.39	3.80	0.16
	30	21.96	19.39	25.30	22.22	2.96	0.13
			6 kip E,base	6 kip E,base	Mean	SD	COV
Base	0		76.62	50.49	63.56	18.48	0.29
	5		64.93	42.24	53.59	16.05	0.30
	10		69.49	101.40	85.44	22.56	0.26
	20		68.93	88.52	78.72	13.85	0.18
	25		58.89	94.70	76.80	25.32	0.33
	30		55.19	98.37	76.78	30.53	0.40

The way we know this is the coefficients of variation (COV) values are good. Interestingly, it is not the subgrade that has the highest variation but instead the base course. Normally it is thought that the subgrade has the most variation with base course having less, and HMA having the least amount of variation.

Surprisingly, even with a less than desirable distance between the material points the standard deviation was low enough that the results were in the same general range. This was found to be true for mainly the subgrade. Which makes sense as this is a cut area with a soil type of A-2-4 throughout the entire testing area. However, when we looked at the base course the coefficient of variation shows a higher variation. And the HMA when looked at among the 6 points also has this difficulty; at the higher load tests this is resolved somewhat with the HMA. The base course has a large range for standard deviation and coefficient of variation however that is not resolved at higher load levels.

The lack of uniformity in the base course is a puzzling result as the base course should be more uniform than the subgrade. The only thing that could account for this is the construction teams' way of mixing the materials together in the field and possibly variation in the RAP material itself. Regardless if the variation is from the construction method or from inconsistencies in the RAP it is apparent that the base course should be collected from the testing section on the actual road instead of stockpiles to mitigate differences between field and laboratory conditions.

DCP Testing

For this construction project 3 sets of DCP testing were done; one on compacted cut subgrade, one on uncompacted borrow subgrade, and the last on compacted borrow subgrade. The two borrow sets were done on a box form used as part of the foundation for a bridge. This gave us the opportunity to test the same soil before and after compaction. This will give us a good comparison of the change in DCP values with compaction.

The cut subgrade was the most straightforward with fairly uniform DCP values for each test. Only one test of the four done in this set showed inconsistencies (test 9); the most likely explanation for this is excess rocks at this point. As can be seen from Figure 2 the other tests showed a strong linear relationship between number of blows and depth of penetration confirming that the soil itself is fairly uniform and rocks are the most likely reason for test number 9's variation from a linear curve.

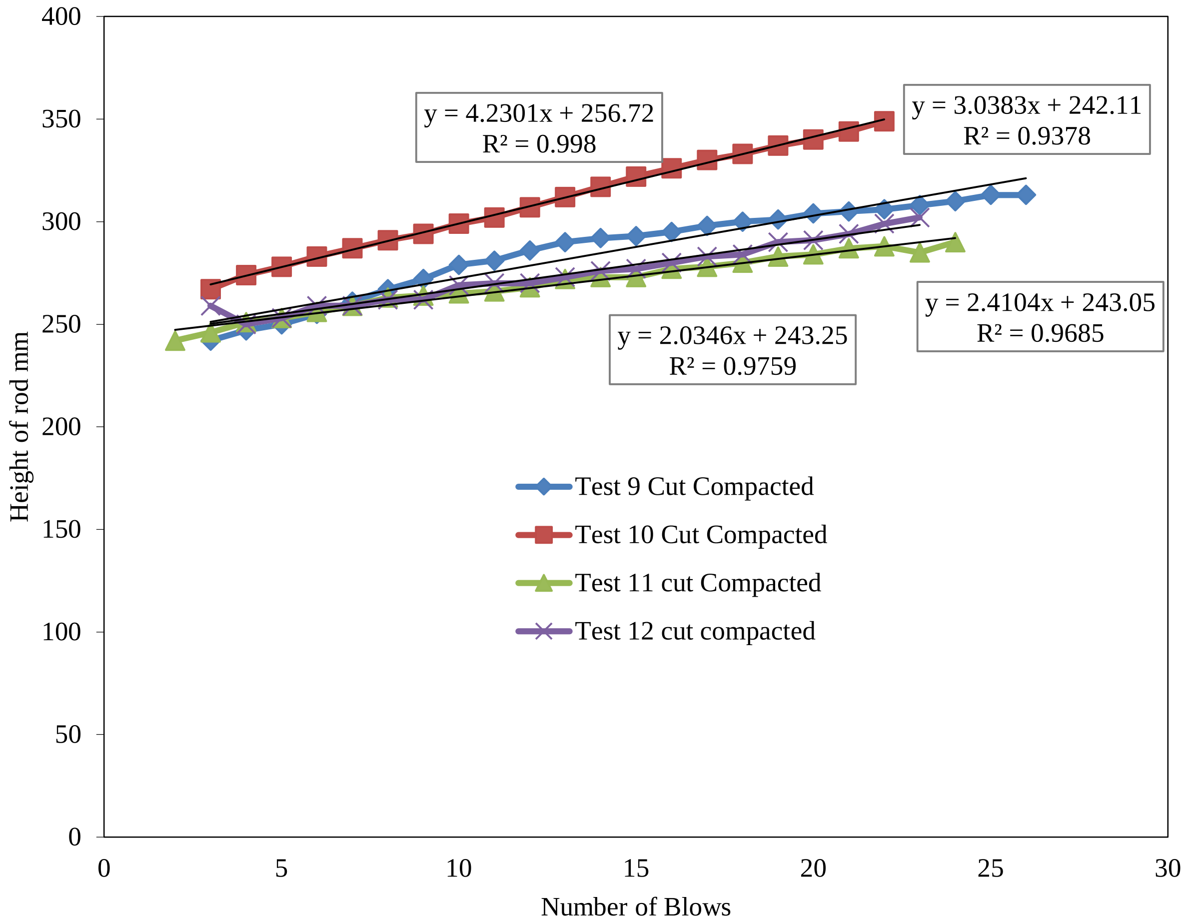

FIGURE 2 DCP test on compacted cut subgrade

From Figure 2 the DCP value can be obtained; for this test method the DCP value is the slope of the linear fit line going through the data points. The DCP values for these tests are in Table 5.

When we look at the uncompacted borrow DCP tests we see a straight line. Note, there are no rocks or pieces above #4 sieve in this material; hence the inconsistancies probably arising from rocks seen in test 9 are completely absent from these. The results from the uncompacted borrow set are in Figure 3.

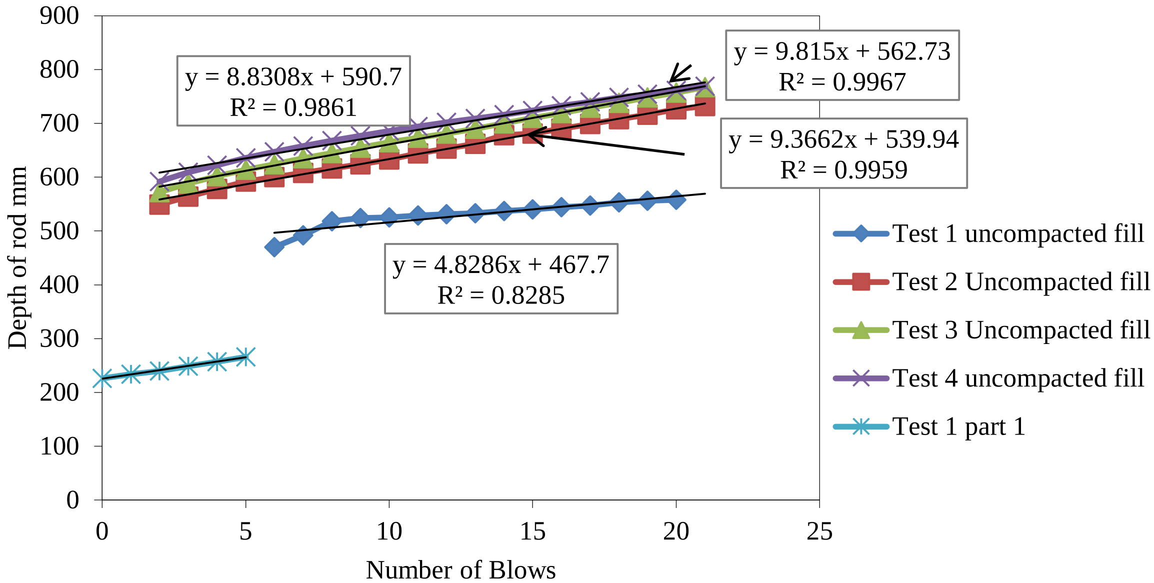

FIGURE 3 DCP test on uncompacted fill

Test 1 is the only test with inconsistencies and this was due to a malfunction with one of the pieces on the DCP. The other three tests are good and valid and show that the soil that was placed in that lift was evenly distributed and had uniform properties.

The last sets of tests showed the DCP tests after compaction of the borrow material. What is particularly interesting in this case is the DCP was done at a depth that shows not only the newly compacted soil but also the soil under the new layer. Given that both the newly compacted soil and the soil below it are supposed to be at the same compacted water content, compaction level, density, and optimum water content properties it should show little to no difference between the two layers. However, as can be seen in Figure 4 it is obvious that even though it is the same soil the two layers have some significant differences.

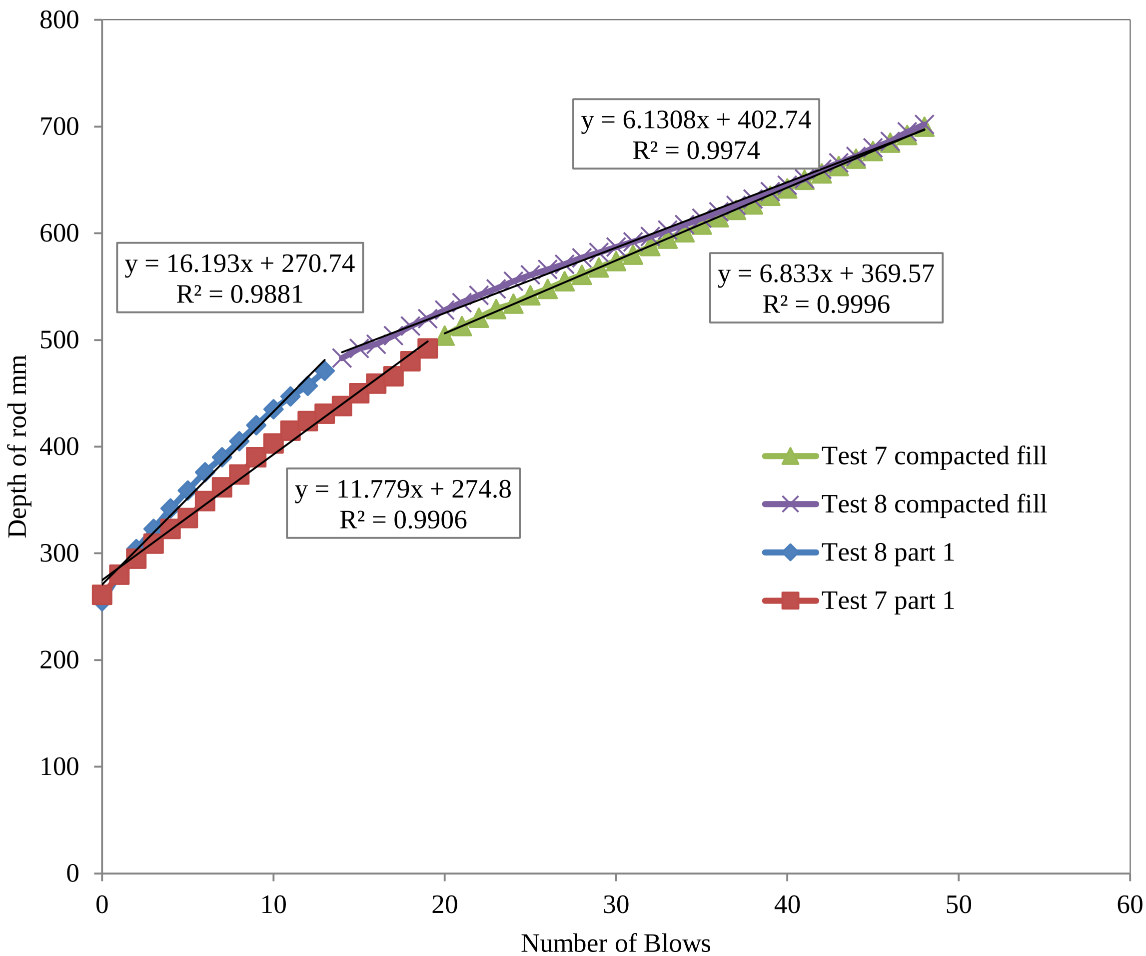

FIGURE 4 DCP test on compacted fill

All four tests show a similar difference between the newly compacted and lower layer DCP values. Whether this difference is from compaction differences, moisture content differences, or variations in the soil itself is unknown. Given the high inconsistency shown in these tests it was determined that in the future the moisture content, compaction, and density should be noted when doing all DCP tests. This result supports the idea that DCP and Modulus values can be related since the modulus is also sensitive to compaction, density, and moisture content at the time of testing. After the DCP values were found they were compared and then converted to the corresponding California Bearing Ratio (CBR) value, and using the CBR value they were converted to the M_r value using the equations from the newest version of MEPDG.

TABLE 5 DCP and Estimated M_r values for compacted cut subgrade

Direction	Material	DCP(mm)	DCP (in.)	R^2	CBR value	M_r Value (ksi)
N outer	cut	3.04	0.12	0.94	3147.59	320.7582
N inner	cut	4.23	0.17	0.998	2174.18	256.9024
S inner	cut	2.03	0.08	0.976	4947.674	420.7572
S outer	cut	2.4	0.09	0.969	4101.663	375.9819
Mean		2.93	0.12	0.97075	3592.777	343.5999
SD		0.836	0.03	0.02072	1037.463	61.318

The DCP values themselves have reasonable standard deviation and the mean value is reasonable. The M_r values are reasonable for the strength of the cut material.

TABLE 5 DCP values for fill material

Direction	Material	DCP Lift 1 (in.)	DCP Lift 2(in.)	DCP (in.) Uncompacted
N outer	fill	0.637402	0.241339	0.385827
N inner	fill	0.46378	0.268898	0.368898
S inner	fill	0.505512	0.254724	0.188976
S outer	fill	0.592126	0.255906	0.347638
	Mean	0.549705	0.255217	0.322835
	SD	0.068605	0.009753	0.078459

TABLE 6 Estimated M_r values for fill material

Direction	Material	M_r Value 1 (ksi)	M_r value 2 (ksi)	M_r value (ksi) uncompact
N outer	fill	104.24	200.21	146.07
N inner	fill	129.08	186.18	150.54
S inner	fill	121.82	193.08	235.98
S outer	fill	109.54	192.48	156.67
	Mean	116.17	192.9902	172.3157
	SD	104.2449	4.970034	36.94847

Densometer Results

The only densometer testing done on this project was on the borrow soil that was tested with the DCP apparatus. The optimum moisture content for the borrow soil tested was 9 ±3%with an optimum compaction of 100-102%. The uncompacted borrow had a moisture content of 6.618% and a compaction of 81.03. The compacted borrow soil had a moisture content of 7.6% and a percent dry density or compaction of 99.57%

Difficulties Encountered

The majority of difficulties encountered on this project were from trial and error on part of the researchers. At first only 3 bags of the compacted cut subgrade was collected; this was a problem as the needed material for modulus of resilience and proctor testing quickly used this amount and more material had to be obtained from the side of the road. This experience quickly showed how important gathering more than enough materials is. The amount of testing on this project is large and thus the material collection must be large as well. The other difficulty was the spread of the testing which made correlating DCP and FWD testing results difficult. On later projects these tests were done on the same day on the same material so we expect much better correlation with them.

US54 NORTH CN 2100250

This is the second project the research team finished. The issues seen on the previous project were resolved and enough testing was accomplished to get a good idea of the spatial uniformity. This project is very similar to the previous one. The aggregates are from the same pit for base course and asphalt concrete. They are both a SPIII mix design with 1% versabind. The RAP used for this project was from milling the road on the US54 South project. The projects are also close together with a distance of only 22 miles separating the two. The subgrade and base course both have the same AASHTO classification with the subgrade being an A-2-4 and the base course being an A-1-a. This gives an interesting opportunity as the weather and materials for the sites are so similar and the main difference is the US54 North project is a WMA (warm mix asphalt) whereas the south project is an HMA. This gives the unique opportunity to see the difference in damage from WMA to HMA. This project is approximately 20 minutes north of Carrizozo and covers mile posts 130-146. The main MP of concern for this project is MP 132.5. The majority of the field testing and material collection was done at this point. This station is 150+00 which might be of use to future researchers as the signs for the stations may be left on the fence by the constructed road.

Once again a cut section was chosen, the subgrade was cut to grade and then compacted to a depth of two feet if needed. Then the base course was laid on top to a depth of 6 inches and consisted of virgin aggregate only. The WMA was laid in two lifts of 3 inches, for a total depth of 6 inches. At the time of this report the OGFC was not laid yet; it will be laid in a 5/8 inch thick layer when weather permits. It should be noted that the first layer of WMA was laid and then had traffic on it from 9/15/14 to 10/13/14. Whether this will significantly affect the pavement life or testing results is not determinable. The first lift was laid from 8/28/14 through 9/15/14. And the second lift was laid 10/14/15 through 11/2/15. Traffic was on both lifts starting on 11/3/15. The WMA was laid using a shuttle buggy and contained 25% RAP which was added at the plant during mixing.

The testing and material collection for this project was contained to a 100 foot area except for the WMA collection. The laying of the WMA is done quickly and this made collecting from the exact station unreasonable, however, the material collected was laid on the same day with our station so it is reasonable to expect little inconsistency. In addition, the research team performed FWD testing at the station of interest and the two stations that were used to collect the material from. By examining these results, the three sections can be compared and analyzed for consistency. The mileposts of interest in this project are laid out in Tables 8 and 9.

TABLE 7 Testing stations and MP for US54 North

Testing	Station	MP
Subgrade, DCP	150+00, 150+50, 149+50	132.54, 132.55, 132.53
Base, Clegg Hammer	150+00	132.54
WMA, FWD	65+00, 90+00, 150+00	130.93, 131.4, 132.54
Base/Subgrade, FWD	150+00	132.54

TABLE 8 Material collection stations and MP for US54 North

Material Collected	Station	MP
WMA	65+00, 90+00	130.93, 131.4
Base Course	149+90, 150+00	132.538, 132.54
Subgrade	150+00, 150+50, 149+50	132.54,132.55,132.53

Materials Collected

In this project a lot of material collection was completed. The materials collected included subgrade, base course, aggregates, binder, WMA mix, versabind, NMDOT field laboratory cores, and field cores. Table 10 shows the materials collected.

TABLE 9 Material collection amounts for US54 North

Material	Unit Amount	Number of Units	Collected From
Coarse Aggregate	50lb Bag	11	Stockpile
Fine Aggregate	50lb Bag	11	Stockpile
RAP Aggregate	50lb Bag	18	Stockpile
Chip Aggregate	50lb Bag	11	Stockpile
Base Course	50lb Bag	10	Field (5 from each station)
Subgrade	50lb Bag	36	Field (12 from each station)
WMA	20lb Bag	83	Field
Binder	1 Gallon Can	9	Paving Plant
Field Cores	3 in. height., 6in. diameter	5	Field
NMDOT Lab Cores	3 in. height., 6in. diameter	47	NMDOT Lab
Versabind	~5 Gallon Bucket	1	Paving Plant

It should be noted that six of the collected bags of subgrade (two from each station collected) were transferred to the Santa Fe NMDOT research bureau for R-Value testing. This was done because the construction team using an estimated R-value based on gradation and atterbergs' limits.

Data Collected from Laboratory and Field

FWD Testing

For this project FWD testing was done on subgrade at stations 149+50, 150+00, and 150+50, on base course at 150+00, and on HMA at stations 65+00, 90+00, 150+00. Base course and subgrade testing points had the layout of starting testing at the station and making this 0 feet and moving forward to test at 5’, 10’, 20’, 25’, and finally at 30’. Subgrade had a testing schematic of 5 feet south of the station, 5 feet north of the station, and on the station tested. All testing points had four weights used (6kip, 9kip, 12 kip, and 16kip). Tables 11, 12, 13, and 14 show the results that have been analyzed already.

TABLE 10 FWD results for subgrade

Station (ft)	FWD on Subgrade	
	6 kip	9 kip
150+00	27.90	24.92
150+05	19.65	20.48
149+95	24.28	23.92
149+40	25.12	27.10
149+45	19.94	19.92
149+50	25.85	25.40
150+40	26.37	23.95
150+45	29.40	26.94
150+50	27.77	31.91
Average	25.14	24.95
Std.	3.21	3.41

TABLE 11 FWD results on base course

Distance (ft)	6 kip		9 kip	
	Base	Subgrade	Base	Subgrade
0	52.83	34.52	67.06	27.66
5	79.95	14.40	90.49	13.76
10	56.18	20.16	61.66	18.75
20	48.76	18.57	50.62	17.19
25	70.26	6.74	65.97	6.74
30	59.82	9.85	71.41	8.66
Average	61.30	17.38	67.87	15.46
SD	10.69	8.96	12.01	6.93

TABLE 12 FWD results on first lift of WMA

Distance (ft)	6 kip WMA	6 kip Base	6 kip Subgrade	9 kip WMA	9 kip Base	9 kip Subgrade
0	404.25	81.16	5.50	1022.87	21.80	8.91
5	170.40	72.79	9.54	672.09	19.46	13.60
10	324.03	37.65	12.85	455.81	28.29	13.07
20	300.28	35.15	11.50	201.78	88.45	6.85
25	420.70	25.42	9.00	168.87	65.13	5.85
30	305.78	24.07	15.94	352.63	22.04	14.83
Average	320.91	46.04	10.72	479.01	40.86	10.52
SD	81.87	22.53	3.27	295.13	26.42	3.47

TABLE 13 FWD results on full-depth WMA

Distance (ft)	6 kip WMA	6 kip Base	6 kip Subgrade	9 kip WMA	9 kip Base	9 kip Subgrade
0	330.46	48.18	11.92	457.83	30.11	11.21
5	294.52	81.64	13.53	295.67	85.82	11.98
10	275.18	82.24	13.26	300.29	72.89	11.68
20	254.63	61.39	14.34	268.83	47.08	13.53
25	209.44	64.01	12.39	298.26	34.14	11.75
30	233.43	69.18	10.97	322.78	42.53	9.64
Average	266.28	67.77	12.73	323.94	52.09	11.63
SD	39.67	11.85	1.11	61.89	20.38	1.15

Once again, when you look at the standard deviation for the subgrade it is highest on the base coarse FWD and smallest on the second layer of WMA. This shows that even with completely virgin aggregate in the base course and no variation due to field mixing the base course is still the least uniform spatially.

The data obtained from these tests were analyzed using ANOVA to determine the spatial uniformity. The result were in all layers over the 30 feet testing length it was determined that the variance and mean of the numbers were statistically the same. In addition to using ANOVA, F-tests, and t-tests were used to analyze the data. This is a comparison of the subgrade values from testing done on subgrade, base course, and WMA. What was found was that the only data set that had a significantly different mean was the 9-kip subgrade. At this load level the values obtained from testing on the base course and subgrade were not the same.

DCP Testing

Twelve DCP tests were done on the US54 project in the pattern shown in Figure 5. As the tests were being performed it was apparent that the subgrade was not uniform from shoulder to shoulder; along the road in the driving directions it was not noticeable during testing but it was apparent that

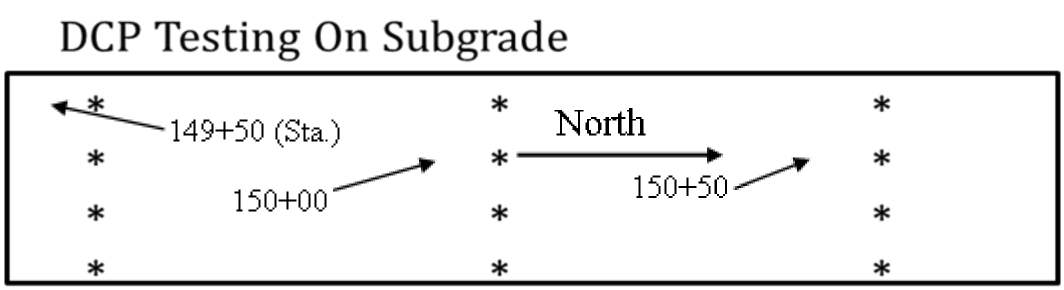

FIGURE 5 Testing schematic for DCP testing on subgrade

perpendicular to the flow of traffic the material was not uniform, with the northbound lane being stiffer than the southbound lane. The graphical results of these tests are presented in Figures 6, 7, 8, and 9.

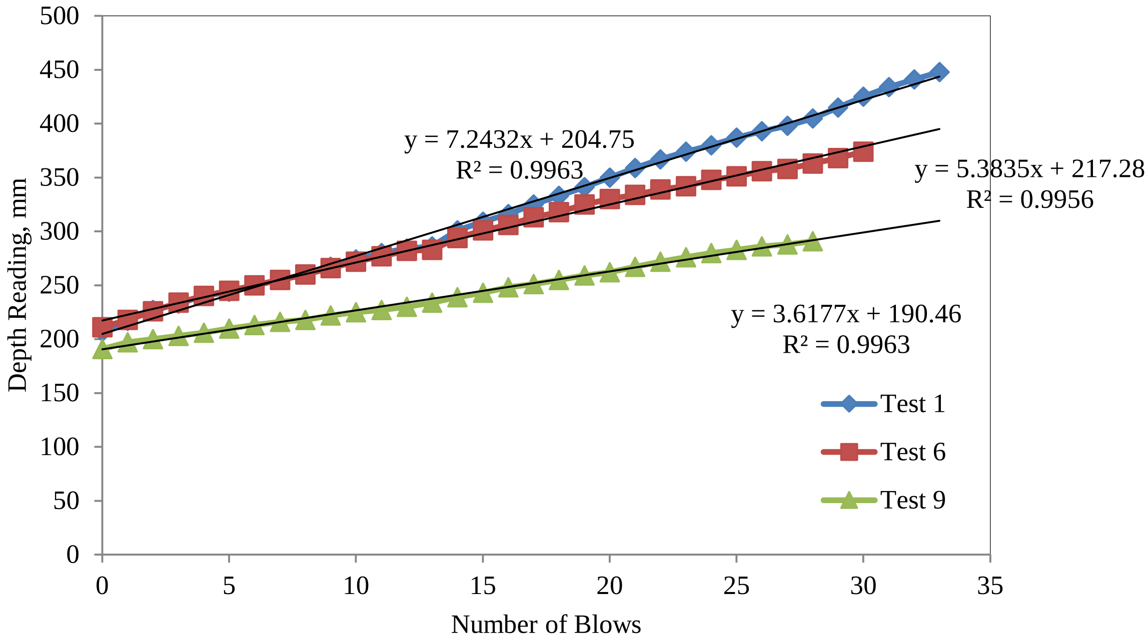

FIGURE 6 DCP testing for outside Northbound lane US54 North

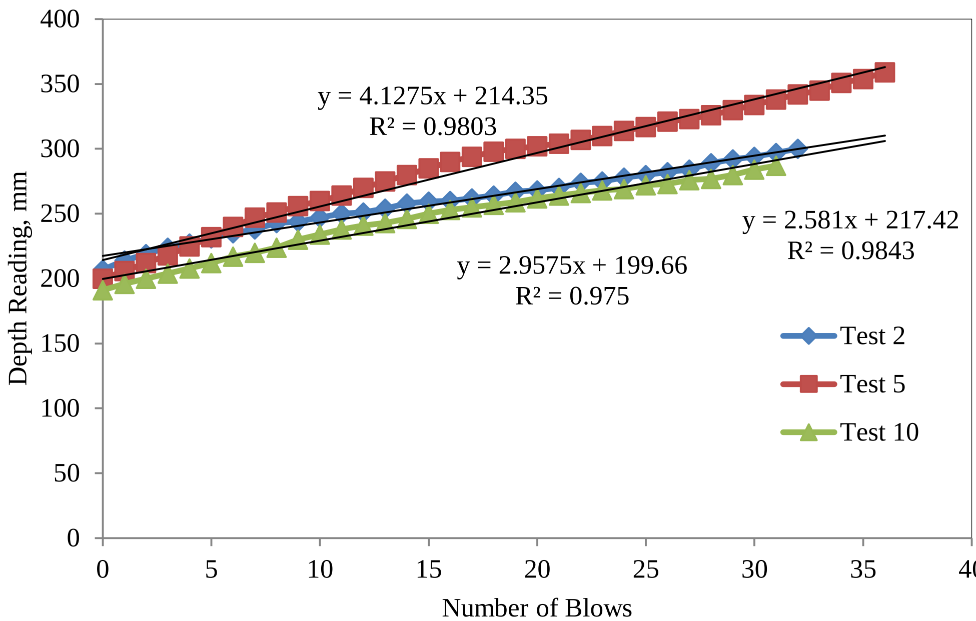

FIGURE 7 DCP for inside Northbound lane US54 North

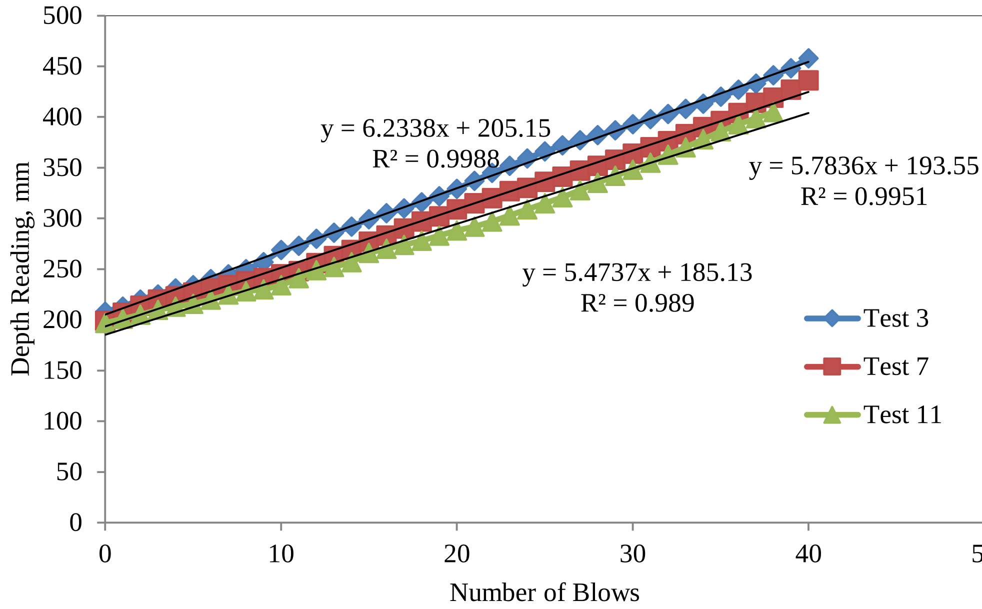

FIGURE 8 DCP for inside Southbound lane US54 North

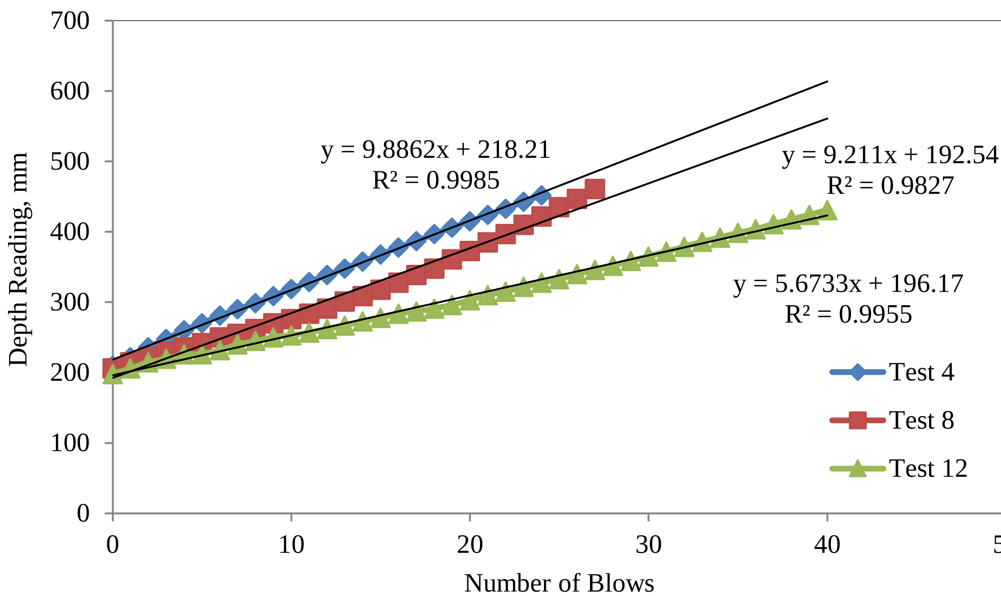

FIGURE 9 DCP for outside Southbound lane US54 North

As can be seen the DCP values all have a high R-value for their linear fit and most of the irregularities in the DCP data can be attributed to the large rocks that were in the soil. The deviations from a linear curve were determined to be rocks because during testing the DCP rod would stop penetrating each blow till the rock was broke through and it could be felt through the rod that something significantly harder than the soil was pushing the rod in the direction perpendicular to penetration.

After the DCP values were found the modulus values were calculated using the MEPDG equation as in the previous project. Those results are in Tables 15, 16, 17, and 18.

TABLE 14 DCP and Estimated M_r values for outside Northbound lane

North	outer			
Test	DCP (in.)	R-value	CBR value	M_r Value (ksi)
1	0.285039	0.996	1190.939	179.0295
6	0.211811	0.996	1660.812	218.5663
9	0.142429	0.996	2590.312	285.366
Average	0.213093	0.996	1814.021	227.6539
SD	0.058227	0	581.4731	43.88471

TABLE 15 DCP and Estimated M_r values for inside Northbound lane

North	Inner		2	3
Test	DCP (in.)	R-value	CBR value	M_r Value (ksi)
2	0.101575	0.98	3782.531	358.1463
5	0.1625	0.98	2234.74	261.1723
10	0.116437	0.975	3246.092	326.7439
Average	0.126837	0.97833333	3087.788	315.3542
SD	0.025937	0.00235702	641.7214	40.40033

TABLE 16 DCP and Estimated M_r values for inside Southbound lane

South	Inner			
Test	DCP (in.)	R-value	CBR value	M_r value (ksi)
3	0.245425	0.9988	1408.229	197.968
7	0.227701	0.9951	1531.562	208.1956
11	0.2155	0.989	1629.003	216.0449
Average	0.229542	0.9943	1522.931	207.4028
SD	0.012286	0.00404063	2.4124102	841.8802921

TABLE 17 DCP and Estimated M_r values for outside Southbound lane

South	Outer			
Test	DCP (in.)	R-value	CBR value	M_r Value (ksi)
4	0.388976	0.9985	840.7538	145.2754
8	0.362638	0.9827	909.4378	152.2842
12	0.223358	0.9955	1564.95	210.9071
Average	0.324991	0.99223333	1105.047	169.4889
SD	0.072665	0.00685144	326.4072	29.42654

After we found the values an ANOVA analysis was done to determine the spatial uniformity. As expected the northbound and southbound tests did not have the same average statistically and thus it is not spatially uniform perpendicular to traffic. When the tests were split according to station and the ANOVA analysis was done it was determined that the tests were spatially uniform in the direction of traffic for the 100 feet tested but not in the direction perpendicular to traffic flow.

Clegg Hammer Testing

Clegg Hammer testing was performed on the US54 Project on the base course in the pattern shown in Figure 10.The Clegg Hammer results for this project are presented in Tables 19 and 20. Twelve tests were done, all of them on parts of the northbound lane at various stations and parts of the

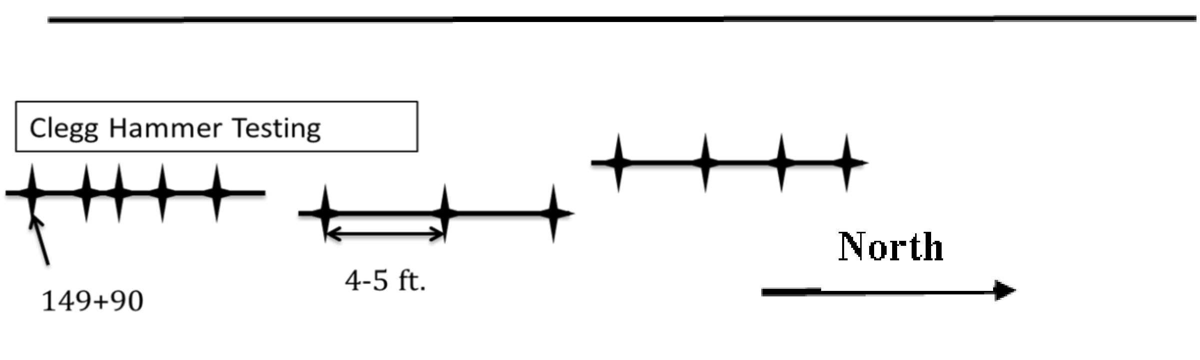

FIGURE 10 Clegg Hammer testing schematic for US54 North

lane. Seven drops were done and the highest value as well as the average were found. The highest value should be the correct impact value given that the first values are removing free floating soil from the surface and creating a good seat for the impact device.

TABLE 18 Clegg Hammer testing results for US54 North subgrade part 1

Lane	N outer	N outer	N outer	N outer	N outer	N Shoulder
Station	149+90	149+95	150+00	150+05	150+10	150+15
Test	1	2	3	4	5	6
Blow						
1	14	14.7	11.4	14.1	14.9	12.5
2	20.4	17.9	14.3	16.6	17.3	16
3	20.3	20.3	16.5	19.3	17.2	18.8
4	20.3	20.2	16.5	19.3	18.4	18.8
5	23.5	22.1	18	19.2	18.3	20.9
6	22.7	22	17.7	19	21.6	20.9
7	24.6	22	17.6	19.1	20.7	20.9
Highest	24.6	22.1	18	19.3	21.6	20.9
Average	20.828571	19.8857143	16	18.0857143	18.3428571	18.4

When the statistical analysis was run, the results in Table 21 were found. The standard deviation is 3.177 which is 17.7% of the minimum value of 18, this means that there is not a significant difference between the values.

TABLE 19 Clegg Hammer testing results for US54 North subgrade

Lane2	N shoulder	N shoulder	N Center	N Center	N Center	N Center
Station	150+20	150+25	150+30	150+35	150+40	150+45
Test	7	8	9	10	11	12
Blow						
1	15.1	15.2	15.7	20.8	16.8	15.2
2	19.4	20.3	19.8	25.9	23.1	18.7
3	20.4	20.2	23.5	30.2	25.7	20.7
4	20.3	22.5	23.4	30	25.7	20.6
5	21.8	22.4	24.6	29.9	25.6	23.2
6	22.3	22.4	22	29.8	25.5	23.1
7	23.8	22.3	23.6	25.6	25.7	23.1
Highest	23.8	22.5	24.6	30.2	25.7	23.2
Average	20.442857	20.757146	21.8	27.457146	24.014271	20.657146

TABLE 20 Clegg Hammer general statistics

	Impact Values
Mean	23.0416667
Standard Error	0.91713142
Median	22.85
Mode	24.6
Standard Deviation	3.17703645
Sample Variance	10.0935606

Densometer Results

Densometer testing was done at the three DCP testing locations for subgrade and the two testing locations for Clegg Hammer Testing. The optimum moisture content and maximum dry density as determined by the construction team was 6% and 130lb/ft^3 respectively with an optimum compaction of 96% or higher. The results from the densometer testing are shown in Tables 22 and 23.

TABLE 21 Subgrade densometer results for US54 North

Station	150+50	150+00	149+50
Moisture Content	5.8	5.75	5.1
Dry Density	108.2	110.7	111.4

TABLE 22 Base densometer results for US54 North

Station	149+90	150+00
Moisture Content	5.22	5.35
Dry Density	121.55	110.85
Compaction	93.5	85.3

Difficulties Encountered

The main difficulties encountered with this project were the large amounts of rocks in the subgrade. According to the construction team if more than 65% of the material is retained on the #4 sieve the material is considered too rocky for proctor testing and falls under different testing and evaluation criteria. Even though the material did not fulfill this requirement when a basic gradation test was done on station 149+50 it was found that one third of the material was retained on the 1 inch sieve, with some of the rocks reaching a length of 7-9inches. Given the fact that several of the rocks were too big to collect the subgrade may be in the realm of too rocky for proctors in some areas that were not tested. This makes corresponding laboratory and field conditions very difficult to correlate as both the proctor and modulus of resilience test scalps off material larger than 1 inch. This means that while the high amount of rocks adds strength there is no way to satisfactory determine the stiffness of the material in situ except by the non-destructive testing. This limits the ability of the researchers to correspond the field and laboratory found modulus' values.

US285 CN 5100411

The project for district 5 is located at the intersection of US285 and I-40 at Clines Corners, New Mexico. The project consists of reconstruction of the interchange for I-40 and US285 and reconstruction of the road up to 2 miles north of the interchange; the construction company in charge of this project is Mountain States Incorporated. The chosen section is at station 185+00 (~MP 250) this is 500 feet from the end of the project; this station was chosen for its distance from the ramp work.

The structure of the station chosen is cut subgrade of AASHTO classification A-6 or A-7 soil. The construction team strengthened some of the weak points and compacted the subgrade to a depth of 2 feet. The accepTable R-value for the subgrade on this project was a minimum of 14; the lower R-value was accepTable due to the use of geotextile and geogrid on the project. The geotextile was laid on the subgrade after compaction and the geogrid laid on top of the geotextile. After this 6 inches of base course was laid; there is a possibility that RAP was mixed with the base course but this and the percentage has not been confirmed at this time. After the base course was laid the HMA was laid in two lifts of 3.25 inches for a total depth of 6.5 inches. The HMA had a RAP content of 25% and is a SPIII. The HMA was laid down using a shuttle buggy and hopper. The HMA was laid down from February 11th through February 21rst, 2015. Traffic was on both lifts starting February 21rst, 2015. The final step will be laying down the OGFC which will take place in summer 2015 when temperatures are consistently warm enough.

One advantage to this construction project was how early the UNM research team was able to get involved. This greatly smoothed relations with the construction team. Even though construction was done quickly, due to the good communication and early involvement all the materials, data, and testing were done efficiently and easily. In addition, with the close proximity of the Moriarty project meetings the research team was able to attend all of the pertinent construction meetings and most of the meetings leading up to important tasks as well as the preconstruction and pre-pave meetings. This was an advantage as plans, specifications, contacts, data sheets, and other information was easily obtained at these meetings.

Materials Collected

This project had more extensive material collection because of the geogrid and geotextile use. The research team was able to collect subgrade, base course, HMA, geotextiles, geogrid, and field cores. Table 24 shows all the material collection done on this project.

TABLE 23 Clines Corners material collection

Material	Unit	Number of Units
Subgrade	50-75 lb Bags	15
Base Course	50-75 lb Bags	8-9
Aggregates-3/8 inch	50-75 lb Bags	3
Aggregates-7/8 inch	50-75 lb Bags	3
Aggregates-Crusher Fines	50-75 lb Bags	3
Aggregates-RAP	50-75 lb Bags	3
HMA	20-30 lb Bags	39 +10 From Ramp Paving
Geogrid	Roll	1/5
Geotextile	-	Approximately 3’X4’ piece
Field Cores First Lift	3 inch height by 6 inch diameter	3
Field Cores Second Lift	3 inch height by 6 inch diameter	3

The geogrid and geotextile were collected in case in the future there is future testing to do and to provide information on the type of geogrid and geotextile used.

Data Collected from Laboratory and Field

DCP Testing with Frozen Soil

The first set of DCP testing that was done on this project were performed in less than ideal conditions. The construction team was aiming to finish construction of our section before the winter shut down; this meant that the subgrade was first ready at the beginning of winter with the temperature at 13 °F and the felt temperature equal to -2 °F. Due to these frigid temperatures with blowing snow and wind only 4 DCP tests were accomplished. This makes an in depth spatial analysis impossible but does give us interesting data. The subgrade results frozen and under normal conditions can be compared. The results from these tests are in Figures 11 and 12.

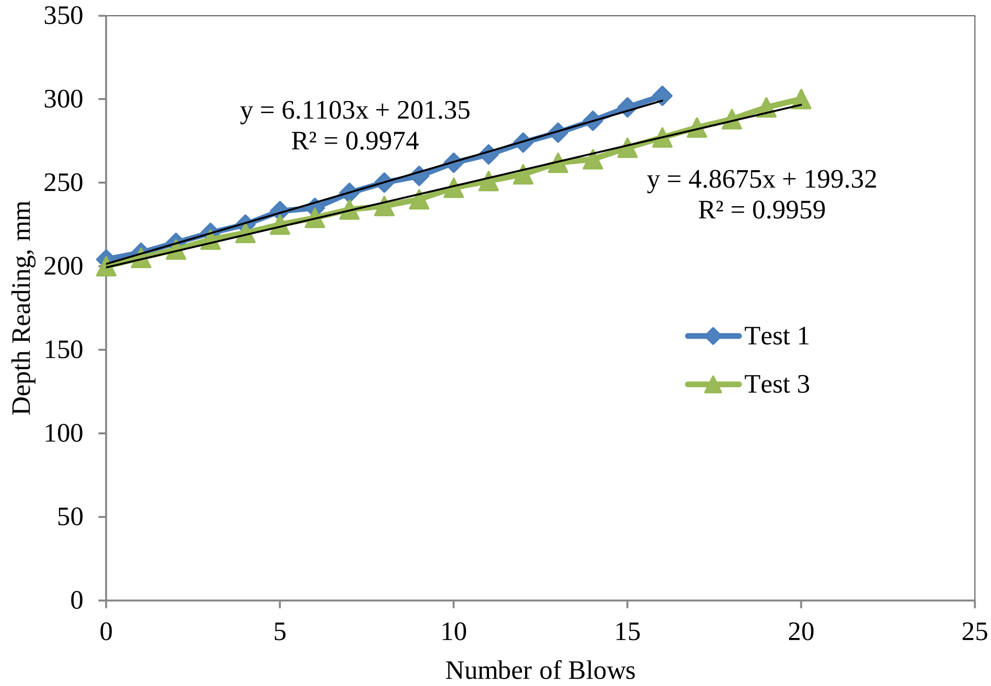

FIGURE 11 DCP tests for Clines Corners part 1

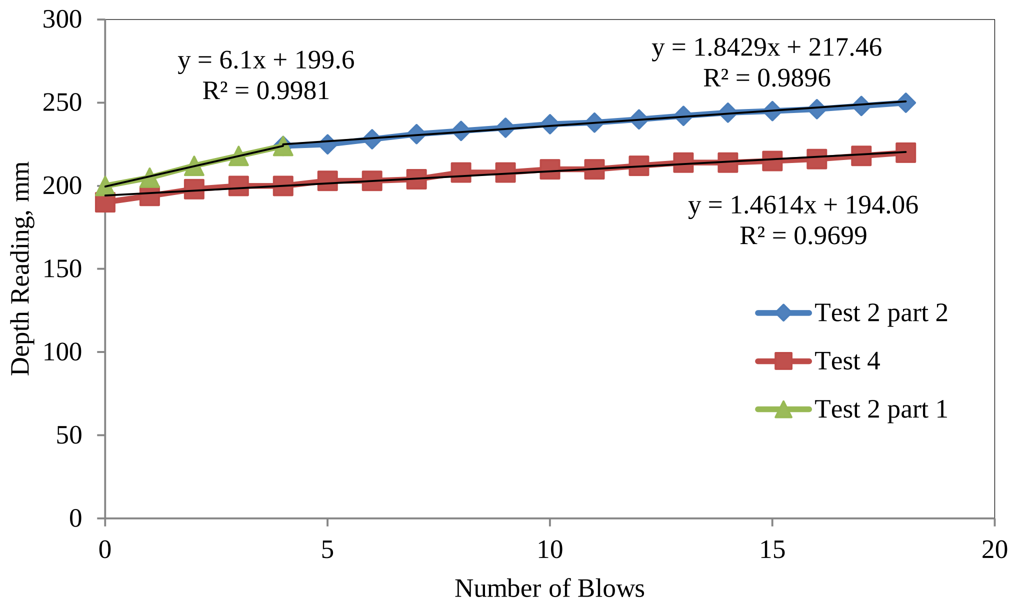

FIGURE 12 DCP for Clines Corners part 2

As expected the subgrade has much lower DCP values due to the frozen soil; this translates to much stiffer soil. The Clines Corner soil is typically a weak A-6 or A-7 soil, from the higher DCP values it is apparent that the water within the soil is frozen creating a much harder soil. The Modulus values were calculated for these DCP values as well and are in Table 25.

TABLE 24 DCP and Estimated M_r results for Clines Corners

US285	Clines	Corners		
Test	DCP (in.)	R-value	CBR value	M_r value (ksi)
1	0.240551	0.9974	1440.225	200.6546
2	0.072441	0.9896	5523.326	449.481
3	0.191654	0.9959	1857.65	233.7595
4	0.05748	0.9699	7156.842	525.0779
Average	0.140531	0.9882	3994.511	352.2433
SD	0.077703	0.0109633	2420.132	138.1525

DCP Testing under normal testing conditions

Four DCP tests were done on the compacted subgrade for this project and the schematic is shown in Figure 13.

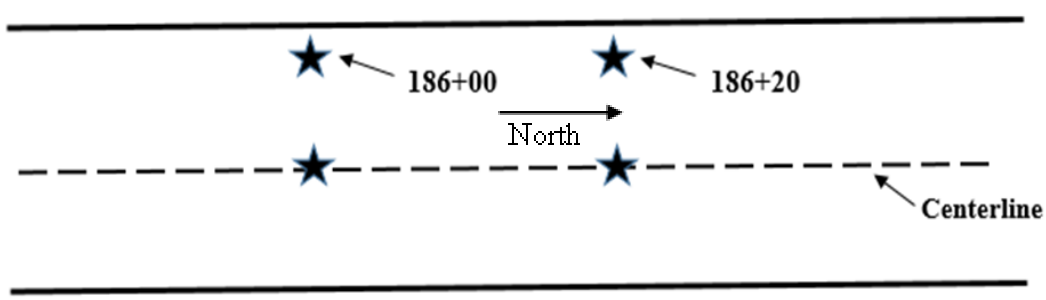

FIGURE 13 DCP testing schematic for Clines Corners

The DCP test results are shown in Figures 14 and 15. As can be seen the four tests are very close together in value. Unfortunately, due to time constraints and the still cold weather the number of tests done was significantly less than for the other projects. This limits our ability to evaluate spatial uniformity to other test methods.

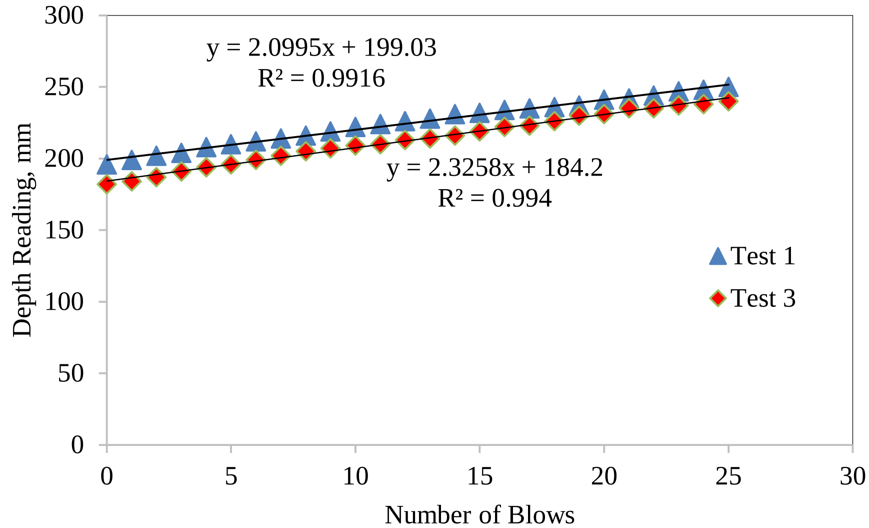

FIGURE 14 DCP results for outer part of lane

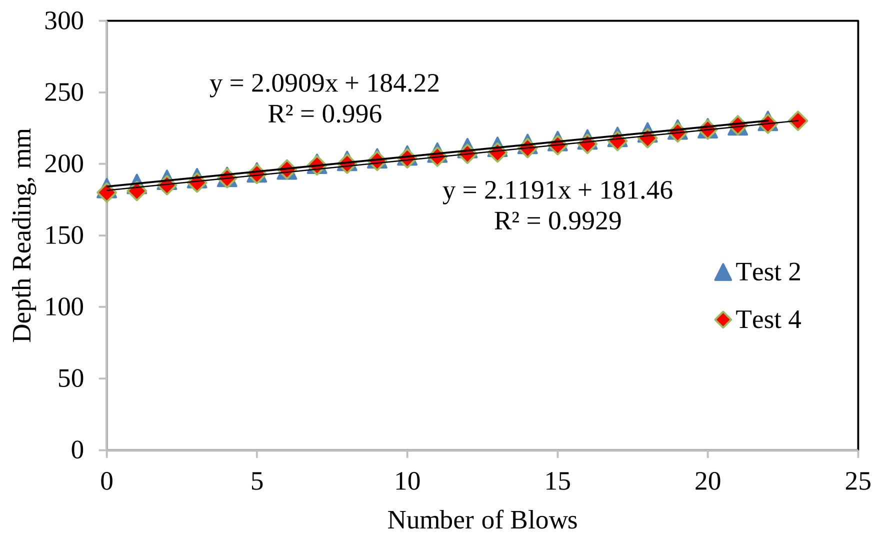

FIGURE 15 DCP for center of road

When the testing results were obtained from the excel graphs once again the modulus values were calculated using the equations from MEPDG. The results are shown in Tables 26 and 27.

TABLE 25 DCP and Estimated M_r for outer part of lane

North	Outer			
Test	DCP (in.)	R-value	CBR value	M_r MEPDG (ksi)
1	0.082657	0.9916	4764.604	411.3457
3	0.091732	0.99	4239.923	383.5356
Average	0.087195	0.9928	4502.264	397.4407
SD	0.004537	0.00	262.3405	13.90506

TABLE 26 DCP and Estimated M_r for center of lane

North	Center			
Test	DCP (in.)	R-value	CBR value	M_r value (ksi)
2	0.082319	0.996	4786.558	412.4819
4	0.083429	0.9929	4715.275	408.7851
Average	0.082874	0.99445	4750.916	410.6335
SD	0.000555	0.00155	35.64189	1.848393

Clegg Hammer Testing on Frozen Soil

Also Clegg Hammer testing was done on the day when the adverse weather conditions existed. Due to how quickly the Clegg Hammer Test can be done the team was able to do 6 tests in the adverse weather conditions. As can be seen all of the six tests had reasonably close results except test number three; this test might be an outlier caused by ice or a piece of rock in the testing area. In Tables 28, 29, and 30 the results are shown along with statistical analysis with and without test 3. The standard deviation is greatly reduced from 11% of the mean to 6% of the mean without test 3 which supports the idea that test 3 may be an outlier.

TABLE 27 Clegg Hammer results for Clines Corners subgrade

Test	1	2	3	4	5	6
Station	186+00	186+04	186+08	186+00	186+04	186+08
Lane direction	N	N	N	S	S	S
Blow #						
1	21.1	10.9	34.2	14.6	19	20.3
2	24.6	25.8	34.2	19	21	25.2
3	24.6	28.2	39.3	20.2	22.6	28.3
4	26.3	28.2	39.3	20.2	22.6	28.3
5	26.3	29.1	40.7	21.2	23.4	28.3
6	26.3	28.3	43.8	21.1	23.3	29.6
7	26.2	28.2	43.8	21.2	23.4	30.4
Highest	26.3	29.1	43.8	21.2	23.4	29.6
Average	25.0571429	25.52857	39.32857	19.64286	22.18571	27.2

TABLE 28 Clegg Hammer general statistics 1

Impact Values with test 3	
Mean	28.9
Standard Error	3.260061
Median	27.7
Mode	#N/A
Standard Deviation	7.985487
Sample Variance	63.768

TABLE 29 Clegg Hammer general statistics 2

Impact Values without test 3	
Mean	25.92
Standard Error	1.619074
Median	26.3
Mode	#N/A
Standard Deviation	3.620359
Sample Variance	13.107

Once again we see a significant stiffening of the soil from the cold temperatures. The soil from this test is stiff enough to be around the stiffness for the base course in the US54 North project. This is probably due to water freezing in the clay structure creating a much stiffer material than would be normally. Given that this soil is supposed to be an extremely weak A-6 or A-7 soil the freezing temperatures must be the main reason for this strength.

Clegg Hammer Testing on Normal Soil

On this project Clegg Hammer testing was performed on both subgrade and base course. The testing pattern for these tests is shown in Figures 16 and 17.

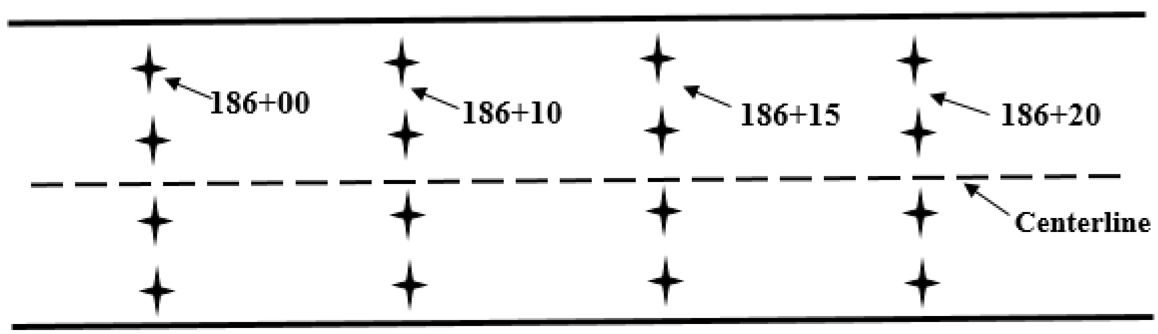

FIGURE 16 Clines Corners Clegg Hammer testing schematic for base

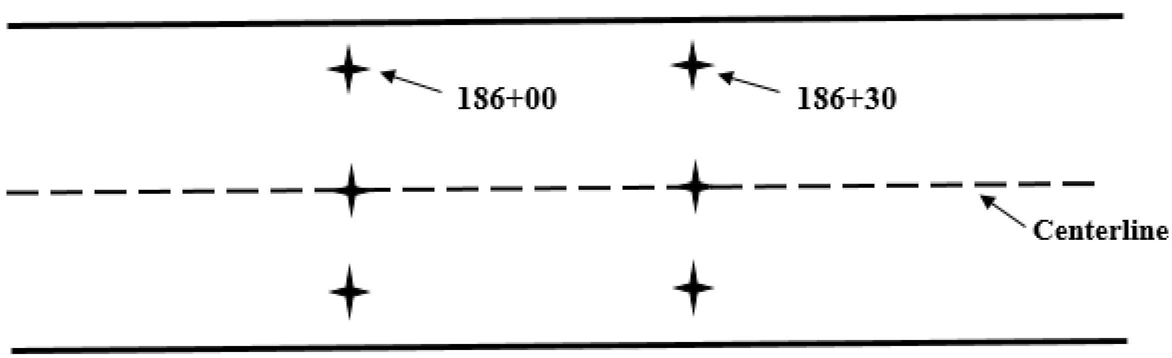

FIGURE 17 Clegg Hammer testing schematic on subgrade

The Clegg Hammer results for the base course material are presented in Tables 31 and 32. Sixteen tests were done on base course. Once again Seven drops were used to determine the Impact value.

TABLE 30 Clegg Hammer values for base part 1

Lane	Driving Outer	Driving inner	Passing Outer	Passing Inner	Driving Outer	Driving inner	Passing Inner
Station	186+00	186+00	186+00	186+00	186+10	186+10	186+10
Test	1	2	3	4	5	6	7
blow							
1	13.8	21.5	17.3	20	12.9	17.6	16.1
2	19.9	35.4	28.8	28.6	20.1	27.8	24.8
3	22.9	35.4	31.4	29.4	23.2	31.3	22.6
4	21.8	38.8	33.9	35.1	24.8	30.7	27.6
5	24.1	36.5	33.9	35.1	24.8	30.7	27.6
6	21.3	36.4	33.9	35.1	24.8	34.5	30.9
7	23.9	36.4	33.8	35	27.5	37	29.3
8							
9							
Highest	24.1	38.8	33.9	35.1	27.5	37	30.9
Average	21.1	34.34	30.42	31.18	22.58	29.94	25.55

TABLE 31 Clegg Hammer values for base part 2

Passing Outer	Passing Outer	Passing Inner	Driving inner	Driving Outer	Driving Outer	Driving inner	Passing Inner	Passing Outer
186+10	186+15	186+15	186+15	186+15	186+20	186+20	186+20	186+20
8	9	10	11	12	13	14	15	16
20.9	20.3	23.8	16.6	15.4	17.5	20.8	18.8	17
32	33.6	27.4	28.1	24.5	24.8	28.3	28.8	23.3
34.2	33.6	29.8	30.5	28.7	27	32.6	28.8	28.2
34.2	41.1	32	34	28.7	28.2	32.6	28.8	30.8
37.8	41	32	33	31	28.1	34.8	31.4	30.8
37.8	41	34.9	35.3	30.9	29.7	34.8	31.3	31.5
32.4	41	34.9	35.3	32.7	31.9	34.5	32	32.8
33.4	41.2	34.9		32.7			34.9	34.9
							34.8	34.8
37.8	41.2	34.9	35.3	32.7	31.9	34.8	34.9	34.9
32.8375	36.6	31.21	30.4	28.07	26.74	31.2	29.95	29.34

A simple statistical analysis was done on the values obtained from the base course. The standard deviation is only 12% of the mean so the values have reasonable uniformity.

TABLE 32 Clegg Hammer base course general statistics

Impact Values	
Mean	34.10625
Standard Error	1.041011878
Median	34.9
Mode	34.9
Standard Deviation	4.16404751
Sample Variance	17.33929167

The results for subgrade are shown in Table 34. Only six tests were done on the subgrade, once again due to weather issues, the wind-chill during testing was significant so the research team had limited time to be outside conducting the tests.

TABLE 33 Clegg Hammer values for subgrade

Lane	N Outer	S Outer	S Outer	N Center	N Center	N Outer
Station	186+30	186+00	186+30	186+00	186+30	186+00
Test	1	2	3	4	5	6
blow						
1	20.6	26.5	19.7	20.3	22.5	25.4
2	20.6	32.5	22.9	23.4	25.2	32
3	25.4	32.5	26.2	25.5	26.9	34.4
4	25.4	35.7	26.2	25.5	28.3	34.5
5	24.9	37.6	26.3	25.4	28.3	34.6
6	26.6	41	26.3	25.3	28	38.1
7	26.6	40.9	26.9	25.3	28	38.3
Highest	26.6	41	26.9	25.5	28.3	38.3
Average	24.3	35.2	24.9	24.3	26.7	33.9

TABLE 34 Statistics for Clegg Hammer all values

Impact Values With test 2 and 6	
Mean	31.1
Standard Error	2.7
Median	27.6
Mode	#N/A
Standard Deviation	6.7
Sample Variance	45.388

TABLE 35 Statistics for Clegg Hammer excluding test 2 and 6 values

Impact Values Without test 2 and 6	
Mean	26.8
Standard Error	0.577
Median	26.75
Mode	#N/A
Standard Deviation	1.1
Sample Variance	1.3

All of the tests values are around the same value except test 2 and 6. It is possible there is a rock in this area but unlikely given the gradation and soil type. The statistics were calculated with and without these tests values. As can be seen from Tables 35 and 36 the standard deviation, sample variance, and standard error are reduced significantly when these tests are discarded. Unfortunately, there are too few tests to confidently say that these two are outlier values. There may be a trend where these tests show a non-uniformity or they may be on rocks we cannot know without more data.

TABLE 36 Clegg Hammer for Clines Corners base course

Test	1	2	3	4	5	6
Station	186+00	186+04	186+08	186+00	186+04	186+08
Lane direction	N	N	N	S	S	S
Blow #						
1	21.1	10.9	34.2	14.6	19	20.3
2	24.6	25.8	34.2	19	21	25.2
3	24.6	28.2	39.3	20.2	22.6	28.3
4	26.3	28.2	39.3	20.2	22.6	28.3
5	26.3	29.1	40.7	21.2	23.4	28.3
6	26.3	28.3	43.8	21.1	23.3	29.6
7	26.2	28.2	43.8	21.2	23.4	30.4
Highest	26.3	29.1	43.8	21.2	23.4	29.6
Average	25.0	25.5	39.3	19.6	22.1	27.2

Interestingly the impact value for the base course on the US54 project and the subgrade on the Clines Corners project is fairly close to the same values. With the base coarse being less stiff than the subgrade. This is an interesting result because the base course is supposed to be the stiffer material especially given that the subgrade used for the Clines Corners is very soft. One explanation for this could be that the subgrade soil is a clayey type material and thus the water may have frozen producing a stronger material than when at higher temperatures. The other explanation could be that the Clegg Hammer does not have the accuracy or sensitivity to adequately approximate moduli values. Or that the compaction done on this project was better than the compaction done on the base course in the US54 project.

Densometer Results

Densometer results were obtained for all the days DCP and Clegg Hammer testing were performed. The proctor results the construction team used for determining compaction and moisture content compliance were 132lb/ft^3 dry density and a moisture content of 8.3%. The results are in Tables 38 and 39.

TABLE 37 Subgrade Densometer results

Date	1/29/15	1/29/15	11/12/14
Moisture Content (%)	5.1	5.0	3.9
Dry Density (lb/ft^3)	130.5	130.4	130.0
Compaction (%)	99	99	98

TABLE 38 Base Densometer Results

Test #	1	2	3	4	5	6	7
Moisture content (%)	4.0	3.6	3.4	3.4	4.7	4.1	4.2
Dry Density (lb/ft^3)	141.8	142.5	141.1	139.7	143.6	140.9	143.4
Compaction	100.2	100.7	99.7	98.7	101.5	99.6	101.3

On the date of 11/12/14 the temperature was 13 °F, thus the accuracy of the densometer testing is in doubt. It is not commonly accepted to use densometer results below 40 °C due to the inaccuracies that accumulate at such low temperatures. The moisture content of 3.9% is most likely incorrect and shows more about how the densometer reads a soil below freezing more than an actual measurement. Most of the water in the soil at this temperature is probably ice, and given the unknown of how the densometer will read ice both the density and moisture content values are questionable. The other readings from 1/29/15 while still showing a very high density given the soil are most likely reliable. These tests were done above the freezing point of water. The base course values all make sense for the range of moisture content and dry density of a base course.

Difficulties Encountered

This project had did not have any major difficulties with it. The early communication with the construction team made material and data collection easy and efficient. This project was constructed extremely quickly so the early communication made maintaining a 200-500 foot collection area feasible. Without proper communication the research team would have been forced to abandon keeping the majority of material and data collection in one area.

DYNAMIC MODULUS TESTING

Dynamic modulus testing was performed on the two projects on US54. Three samples from each project was made and tested according to AASHTO T 342. The dynamic modulus testing and master curve calculations were done. It would be possible for the research assistants on this project to do the test but the accuracy of the test would be less and the test would have taken significantly longer to complete.

US54 SOUTH

Figure 39 shows the dynamic modulus of US54 South AC.

TABLE 39 Dynamic Modulus results for US54 South

Conditions		Specimen 1		Specimen 2		Specimen 3		Modulus		Phase Angle (Degrees)	
Temperature, °C	Frequency, Hz	Modulus, MPa	Phage Angle, degree	Modulus, MPa	Phage Angle, degree	Modulus, MPa	Phage Angle, degree	Avg. Modulus, MPa	CV, %	Avg. P. Angle, degree	Standard Dev, degree
-10	25	50261.88	2.5	33938.59	0.9	45669.42	6.3	43289.96	19.45	3.2	2.77
-10	10	49458.26	5.8	29850.26	5.8	49658.09	5.8	42988.87	26.47	5.8	0.00
-10	5	47807.76	5.7	28786.07	5.6	48219.50	5.0	41604.44	26.69	5.4	0.38
-10	1	42125.11	7.2	26376.83	4.9	43973.90	6.8	37491.95	25.79	6.3	1.23
-10	0.5	42803.32	3.2	24358.11	7.7	44104.45	8.8	37088.63	29.78	6.6	2.97
-10	0.1	38336.47	8.3	22316.74	9.5	39036.74	5.7	33229.98	28.46	7.8	1.94
4.4	25	37427.72	0.6	25688.65	0.3	36040.72	7.9	33052.36	19.41	2.9	4.30
4.4	10	37574.24	8.4	22079.04	9.4	36423.68	8.1	32025.65	26.96	8.6	0.68
4.4	5	35215.56	9.2	20893.93	9.0	34158.29	8.0	30089.26	26.52	8.7	0.64
4.4	1	30908.04	10.9	17700.54	10.6	28718.31	11.4	25775.63	27.46	11.0	0.40
4.4	0.5	28916.04	14.8	15650.04	14.3	25983.87	10.1	23516.65	29.63	13.1	2.58
4.4	0.1	22922.83	15.1	13120.05	15.9	22108.80	14.6	19383.89	28.06	15.2	0.66
21.1	25	18001.20	16.3	12060.34	12.7	21160.37	15.4	17073.97	27.06	14.8	1.87
21.1	10	15009.74	20.7	11249.39	18.8	18548.50	17.1	14935.88	24.44	18.9	1.80
21.1	5	13474.04	21.5	9874.14	20.2	16442.06	18.1	13263.41	24.80	19.9	1.72
21.1	1	9431.20	26.5	6760.39	24.5	11593.34	21.4	9261.64	26.14	24.1	2.57
21.1	0.5	8400.21	27.4	5641.35	25.9	9913.38	22.6	7984.98	27.13	25.3	2.46
21.1	0.1	6089.21	30.2	3702.70	29.4	6753.27	27.1	5515.06	29.09	28.9	1.61
37.8	25	8574.39	33.6	5328.92	32.1	10369.92	12.3	8091.07	31.58	26.0	11.89
37.8	10	6950.60	27.1	4107.65	32.5	8203.71	24.6	6420.65	32.69	28.1	4.04
37.8	5	5424.32	28.2	3046.09	33.5	5967.75	27.6	4812.72	32.29	29.8	3.25
37.8	1	3156.39	33.9	1695.26	34.5	3620.50	29.7	2824.05	35.58	32.7	2.62
37.8	0.5	2509.56	34.3	1340.59	34.3	2966.18	29.4	2272.11	36.90	32.7	2.83
37.8	0.1	1597.78	30.8	892.72	30.4	2126.80	24.4	1539.10	40.23	28.5	3.59
54.4	25	2607.37	34.3	1669.94	33.3	3153.82	37.9	2477.04	30.30	35.2	2.42
54.4	10	1862.58	30.9	1146.56	31.3	2345.26	32.6	1784.80	33.79	31.6	0.89
54.4	5	1484.22	28.6	879.42	29.8	1913.89	31.1	1425.84	36.45	29.8	1.25
54.4	1	952.89	25.6	527.75	26.2	1160.66	29.1	880.44	36.64	27.0	1.87
54.4	0.5	780.79	24.3	409.62	24.9	967.96	27.4	719.46	39.50	25.5	1.64
54.4	0.1	586.23	20.2	300.12	19.3	695.32	23.7	527.22	38.71	21.1	2.32

It should be noted that at the temperatures of 37.8 °C and 54.4 °C the Coefficient of variation is equal to or greater than 30%, this is typical not accepTable and shows too much variation in the three test samples

Mastercurves

After the preliminary results were found the master curves were calculated using the AASHTO procedure as shown in Figures 18-21 and Table 41.

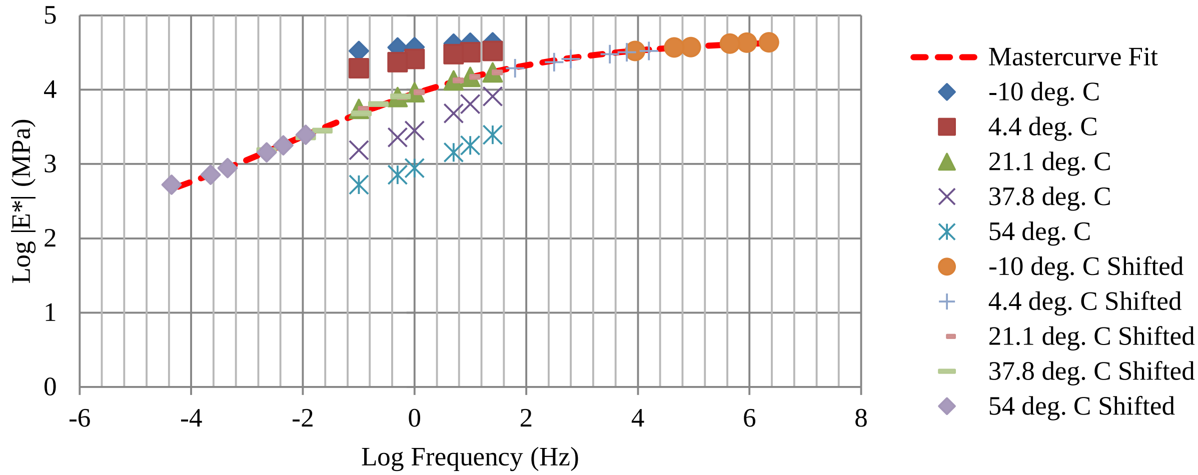

FIGURE 18 Data for mastercurves US54 South

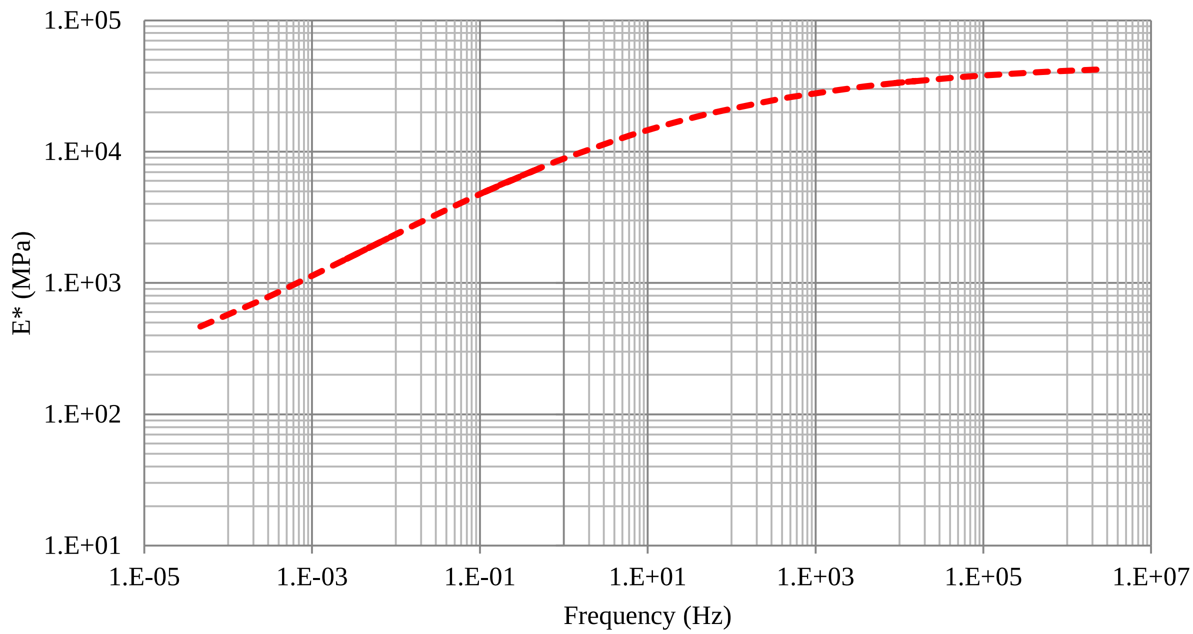

FIGURE 19 Mastercurve in logarithmic scale for US54 South

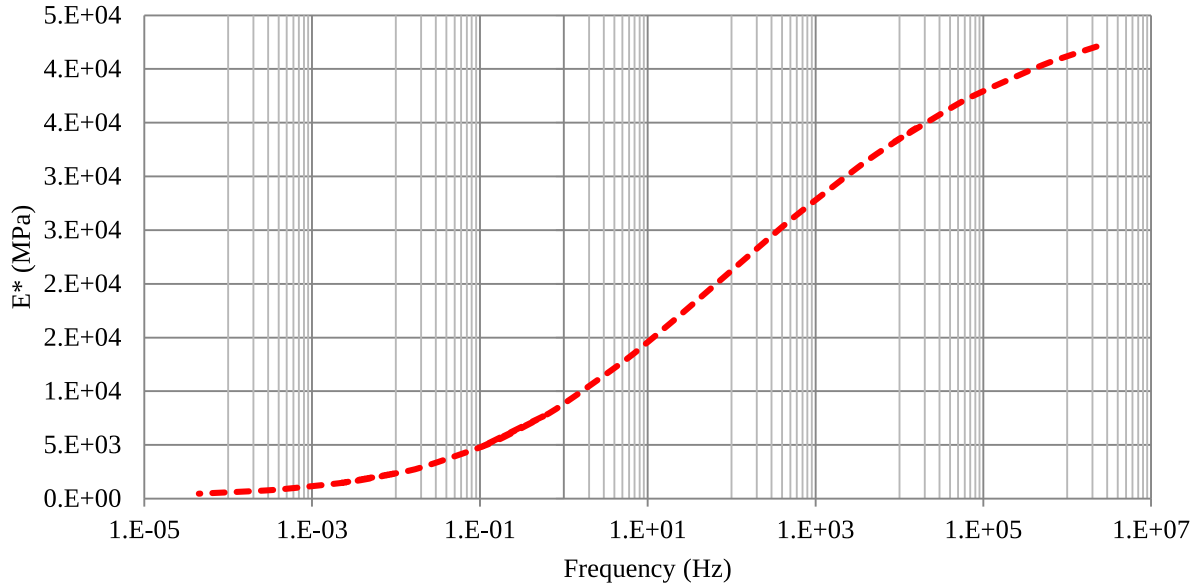

FIGURE 20 Mastercurve in semi-logarithmic scale for US54 South

TABLE 40 Mastercurve parameters for US54 South

	Reference Temperature	α	β	δ	γ
SI System	21.1° C	2.83	-1.05	1.85	-0.45
English System	70° F	2.83	-1.05	1.02	-0.45

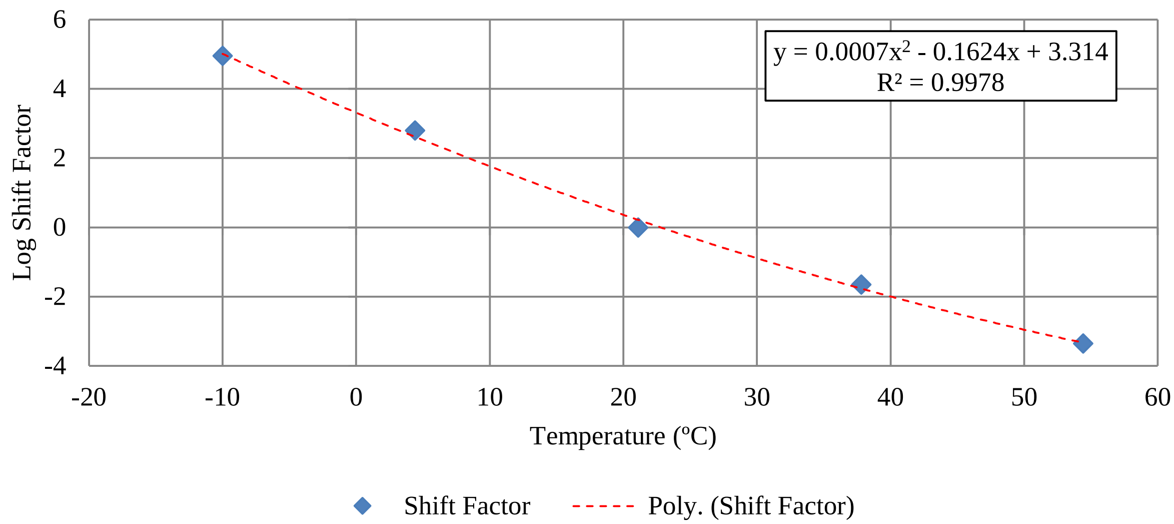

FIGURE 21 Shift factor curve for US54 South

US54 NORTH

For this project 3 samples were also tested and the results are shown in Table 42.

TABLE 41 Dynamic Modulus Results for US54 North

Conditions		Specimen 1		Specimen 2		Specimen 3		Modulus		Phase Angle (Degrees)	
Temperature, °C	Frequency, Hz	Modulus, MPa	Phage Angle, degree	Modulus, MPa	Phage Angle, degree	Modulus, MPa	Phage Angle, degree	Avg. Modulus, MPa	CV, %	Avg. P. Angle, degree	Standard Dev, degree
-10	25	53006.39	5.1	33406.71	0.3	42285.58	13.9	42899.56	22.88	6.4	6.90
-10	10	44462.52	7.1	35986.89	6.2	48152.68	5.2	42867.36	14.55	6.2	0.95
-10	5	43105.32	6.8	34836.96	5.8	46604.51	5.4	41515.60	14.56	6.0	0.72
-10	1	39871.25	6.7	31228.55	6.5	43576.17	6.7	38225.32	16.58	6.6	0.12
-10	0.5	36507.99	9.1	31020.34	4.5	40008.01	4.8	35845.45	12.64	6.1	2.57
-10	0.1	34339.49	9.3	27644.98	8.1	36975.64	9.3	32986.70	14.58	8.9	0.69
4.4	25	32232.96	1.8	25218.43	7.1	32490.29	18.7	29980.56	13.76	9.2	8.64
4.4	10	30305.13	11.0	25871.53	10.0	34129.82	10.6	30102.16	13.73	10.5	0.50
4.4	5	27605.97	11.5	23990.78	10.8	33016.50	9.8	28204.42	16.11	10.7	0.85
4.4	1	22971.17	15.1	20249.24	12.7	26545.83	13.6	23255.41	13.58	13.8	1.21
4.4	0.5	22087.53	15.8	18685.68	11.3	25530.49	18.0	22101.23	15.49	15.0	3.42
4.4	0.1	16920.17	19.2	14705.42	16.4	18930.79	18.1	16852.13	12.54	17.9	1.41
21.1	25	14175.32	24.1	13800.11	18.7	17896.93	20.4	15290.78	14.81	21.1	2.76
21.1	10	12479.03	24.6	11864.18	20.1	15928.43	20.2	13423.88	16.32	21.6	2.57
21.1	5	11575.53	26.0	10437.89	21.7	13763.16	21.8	11925.53	14.17	23.2	2.45
21.1	1	8448.25	30.9	7332.66	25.3	9133.64	26.8	8304.85	10.95	27.7	2.90
21.1	0.5	7600.58	29.4	6269.28	28.6	7391.27	28.8	7087.04	10.10	28.9	0.42
21.1	0.1	4362.87	33.8	4042.43	29.4	4749.56	29.4	4384.96	8.07	30.9	2.54
37.8	25	7558.96	31.3	6207.43	29.7	5901.72	35.1	6556.04	13.45	32.0	2.77
37.8	10	5247.33	32.6	4668.17	29.9	4217.73	33.5	4711.08	10.96	32.0	1.87
37.8	5	4030.42	33.0	3575.94	30.2	3242.39	33.7	3616.25	10.94	32.3	1.85
37.8	1	2526.31	30.8	1970.68	34.3	1723.68	33.6	2073.55	19.83	32.9	1.85
37.8	0.5	2110.26	29.2	1564.19	33.1	1354.59	32.0	1676.35	23.27	31.4	2.01
37.8	0.1	1506.11	23.6	1072.24	24.5	857.41	27.3	1145.26	28.85	25.1	1.93
54.4	25	2278.64	34.7	1501.44	38.7	1712.20	34.5	1830.76	21.95	36.0	2.37
54.4	10	1677.02	32.2	1054.47	34.5	1263.24	29.2	1331.58	23.80	32.0	2.66
54.4	5	1444.63	30.5	725.31	32.1	1008.15	26.9	1059.37	34.21	29.8	2.66
54.4	1	1162.65	23.7	472.19	26.9	650.40	22.5	761.75	47.06	24.4	2.27
54.4	0.5	985.39	21.8	427.57	23.0	525.24	21.5	646.06	46.11	22.1	0.79
54.4	0.1	737.58	18.1	290.61	18.0	405.64	17.2	477.94	48.56	17.8	0.49

Once again it should be noted that the coefficient of variation is too high for most of the modulus values obtained at 37.8 °C.

Mastercurves

After the data was analyzed the following mastercurves and shift factors were obtained. All of the mastercurves used a 70 °F reference temperature.

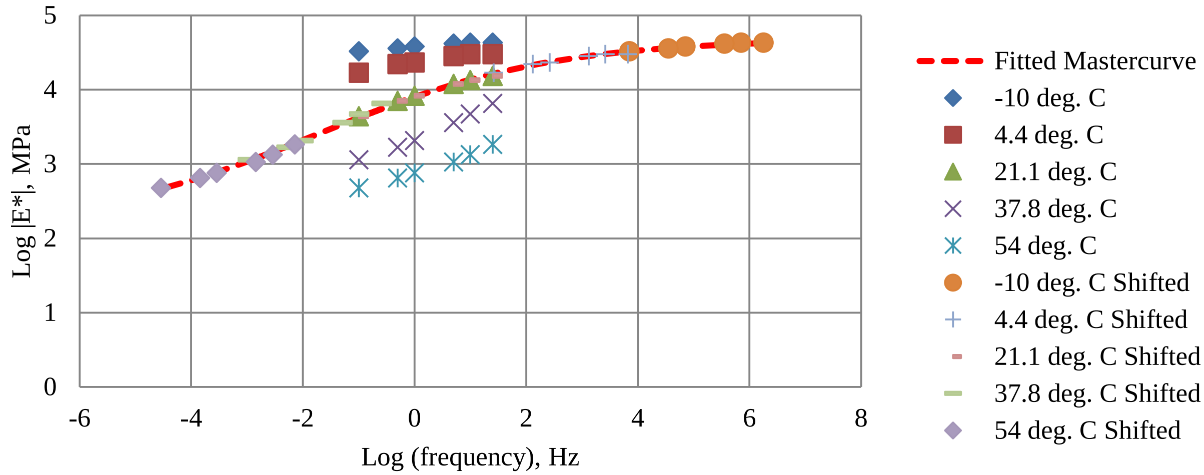

FIGURE 22 Data for mastercurve for US54 North

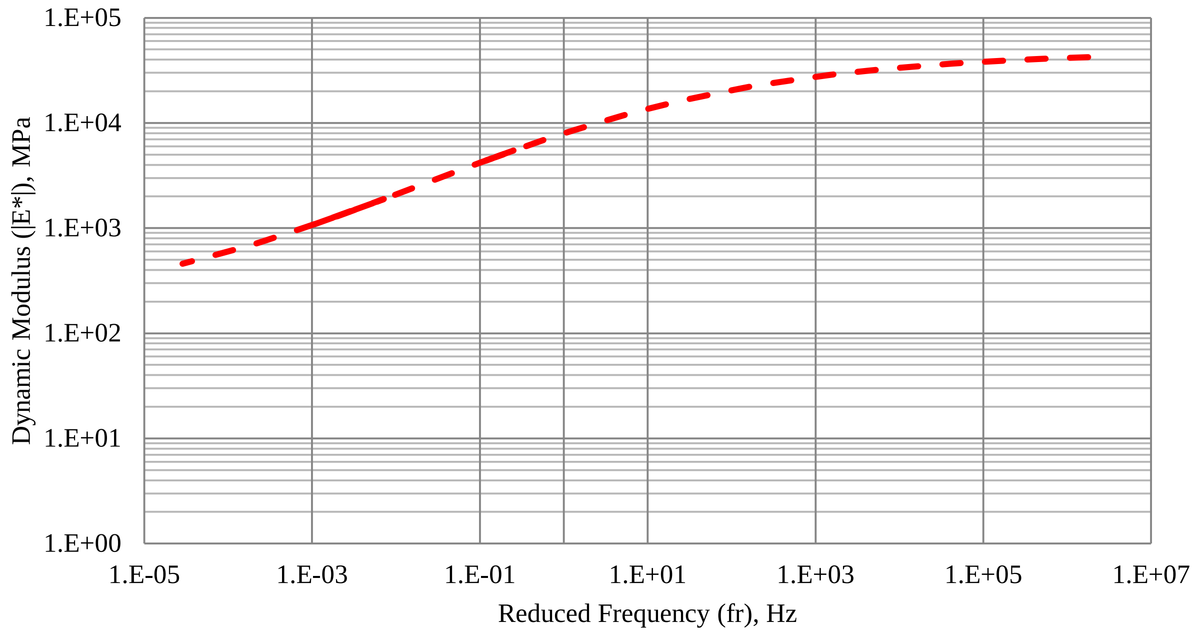

FIGURE 23 Master curve for reference temperature of 70 °F logarithmic scale

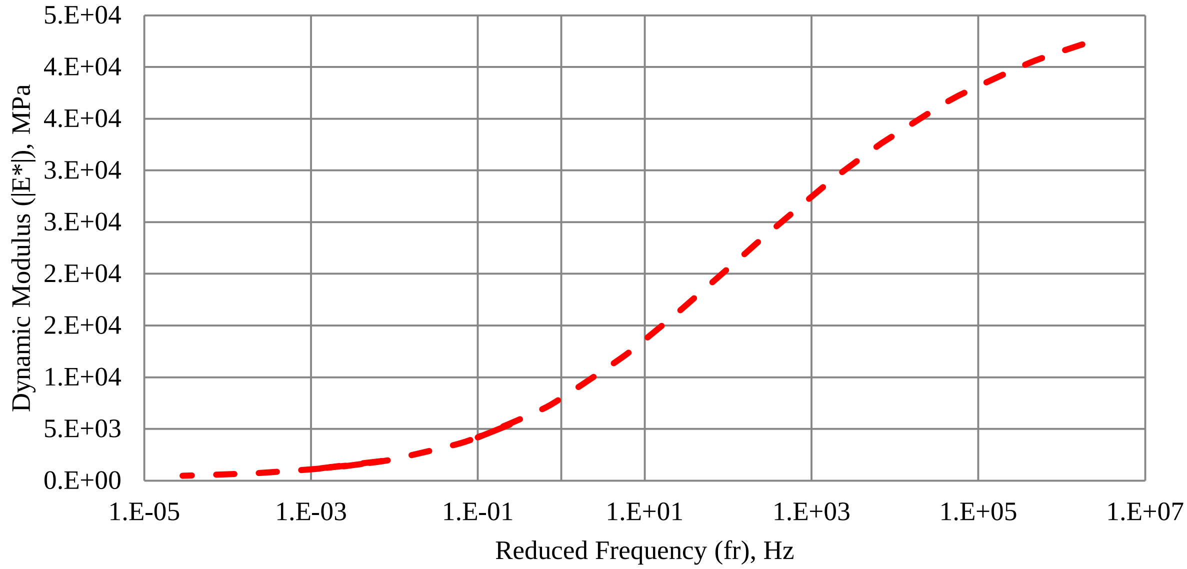

FIGURE 24 Semi-logarithmic scale mastercurve

TABLE 42 Parameters for mastercurve

	Reference Temperature	α	β	δ	γ	a_1	a_2
SI system	21.1° C	2.53	-0.81	2.15	-0.48	0.132	0.00077
English system	70° F	2.54	-0.81	1.30	-0.48	0.072	0.00022

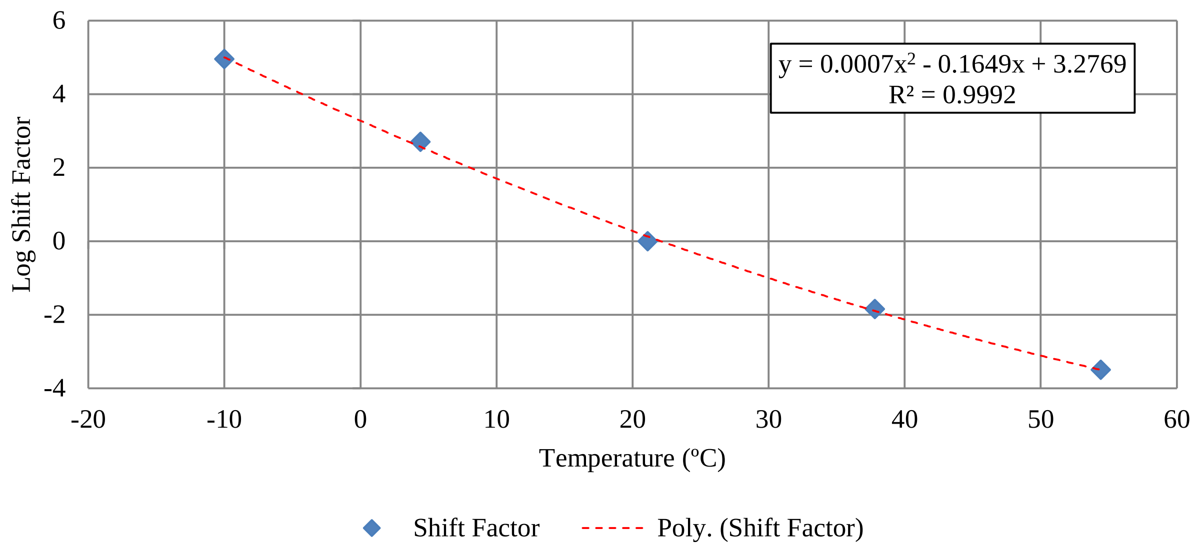

FIGURE 25 Shift factors for mastercurve

COMPARISON

Mastercurves

Several comparison Figures were done by Asif from the dynamic modulus data. In Figure 26 we see a comparison of the two mastercurves for the US54 projects.

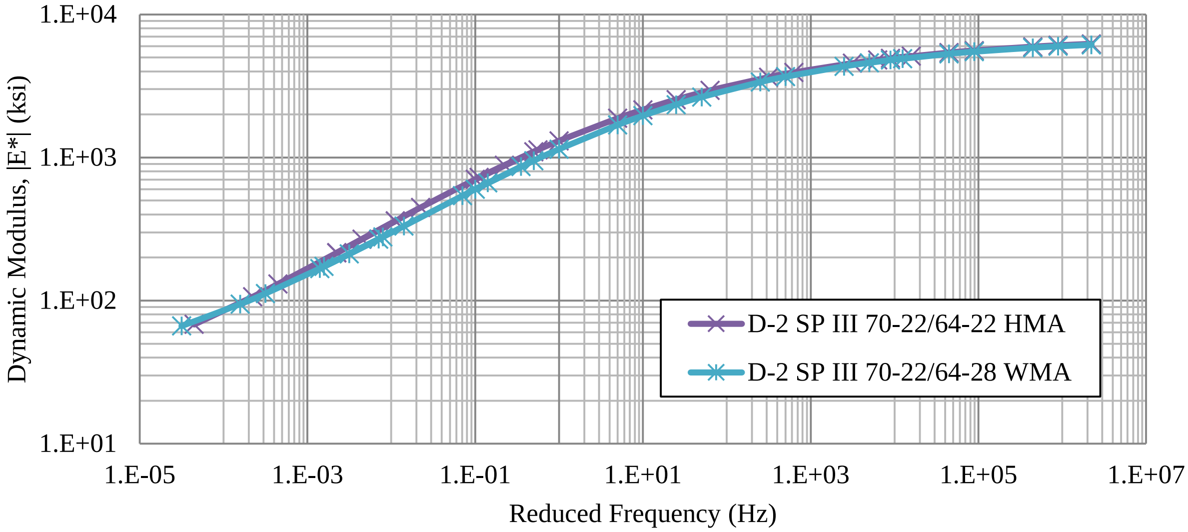

FIGURE 26 Comparison of mastercurves for US54 North and South

As can be seen from above the two master curves are almost exactly the same. This makes sense as the two mixes are similar. They both have the same aggregate pit, similar binder, same SP grade, and the same amount of RAP. The main difference is one being a warm mix and one a hot mix asphalt. Which as can be seen in the middle frequencies the hot mix (US54 South) does perform better slightly but at the higher and lower points the two are essentially equal in performance.

Comparison of Values at Different Temperatures

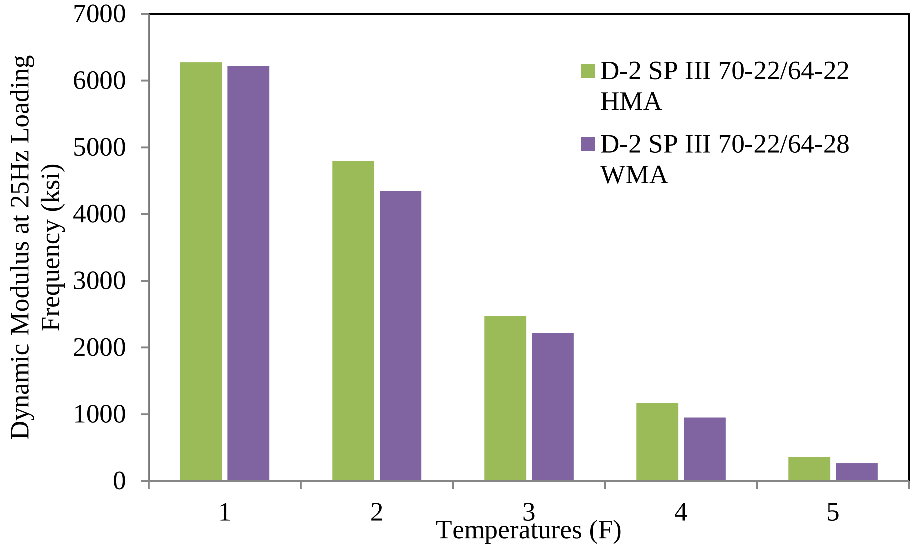

FIGURE 27 Comparison of dynamic modulus values vs. temperature for US54 North and South

In Figure 27 we are comparing the two asphalts dynamic modulus at 25 Hz at varying temperatures (14, 40, 70, 100, 130 degrees Fahrenheit or -10, 4.4, 21.1, 37.8, 54.4 degrees Celsius). The HMA as can be seen holds its stiffness a little better than the WMA. In addition, with rising temperatures there is a significant drop in stiffness; this makes sense as the higher temperature will allow the binder to flow and move easier making the mix less stiff.

MODULUS OF RESILIENCE TESTING

This test had many issues in it. The testing machine itself had some maintenance issues to resolve concerning old O-rings and inadequate screws. The manufacture of the test specimens also had issues. These were mainly issues deriving from the large amount of rocks in the subgrade, trouble finding an accurate optimum moisture content and maximum dry density, as well as difficulties working with the soil itself to create smooth, straight samples. Due to the difficulties in the modulus of resilience test itself many supporting tests were performed including modified proctor testing by the UNM research team and gradation testing.

GRADATIONS

Gradation testing was done for three purposes. First, gradation is a necessary input in the MEPDG program, second gradation testing gives us the type of modulus of resilience testing that is needed, and third, gradation testing shows the variation in the subgrade for the US54 North project where three collection points were used (stations 149+50, 150+00, 150+50). In addition, to the testing accomplished for the US54 North project, two soils were tested from the US54 South project. These were the original cut subgrade collected and the subgrade collected from the side of the road after construction was completed. This was done to see the differences between the two subgrades. Lastly, the base course from the US54 North project was tested to determine the proper modulus of resilience testing as well as provide inputs for MEPDG calibration. After testing, the gradations were compared to the AASHTO classification chart and the general group of soil was determined.

US54 South Gradation

One of the first gradations performed was on the two subgrades obtained from the US54 South project. It was expected that the two soils would be very similar.

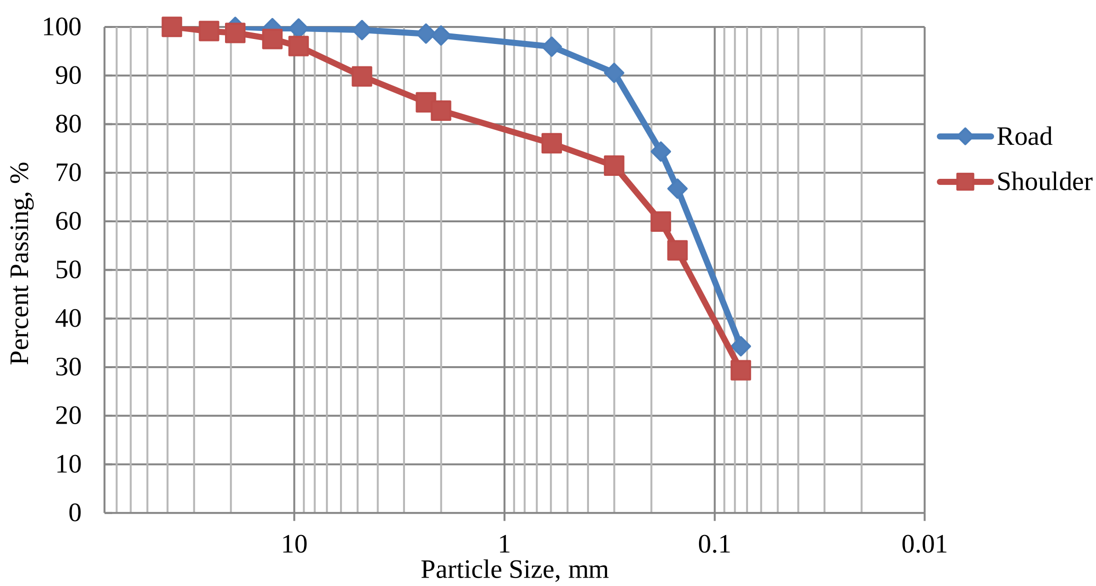

FIGURE 28 US54 South gradation curves

Unfortunately, as can be seen in Figure 28 the soil samples may be the same AASHTO classification but they are significantly different. It is not clear from the gradation alone how much this difference will affect the modulus of resilience testing and therefore it is recommended that both soils are used for determining the modulus values. Both soils were determined to be in the A-2 gradation group.

US54 North

The three gradations from the same subgrade area had similar differences. Although, all three were determined to be in the A-2 AASHTO soil group, the three tests had significantly different curves as can be seen in Figure 29. The rockiness of the cut subgrade may have contributed to such a large

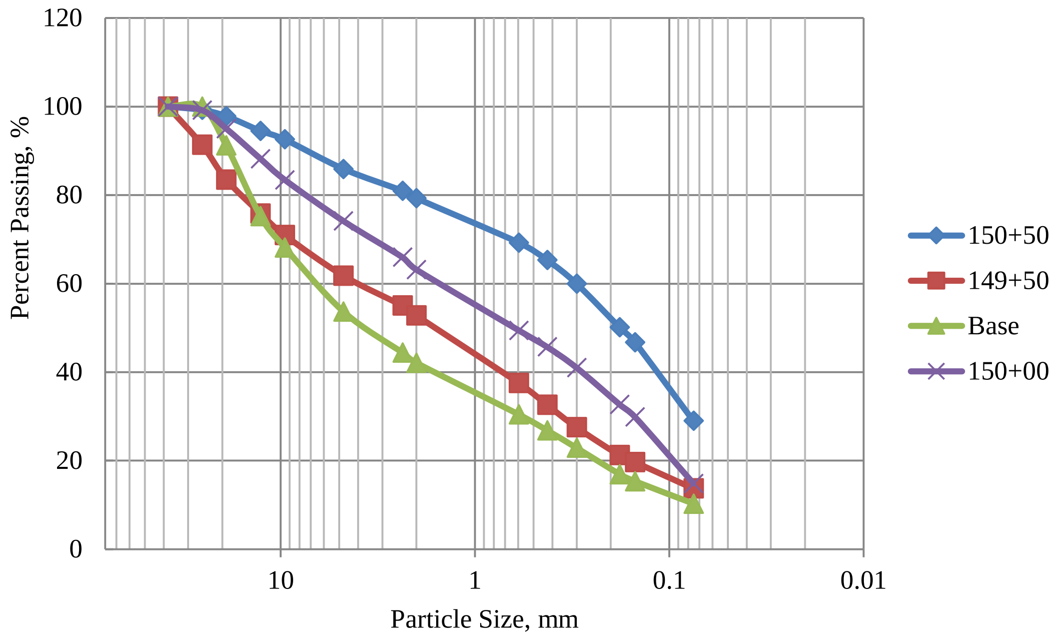

FIGURE 29 US54 North gradation curves

difference in the three soils. This difference in curves requires modulus of resilience samples to be made from all three subgrades to verify that the modulus is representative of that area. Also, the three subgrade soils were not greatly removed from the base course curve; the similarity to the base is a product of the rockiness of the soil as well. The base course has a definite difference in its curve compared to the subgrade curves, and is determined to be in the A-1 AASHTO group. The gradation results were then compared to the NMDOT specifications for gradation of base course as shown in Table 44. The base course fulfills the requirements for both Type I and Type II subgrade. The only questionable part is Type I, where the passing No. 200 sieve is supposed to be less than or equal to 10%, the UNM team found 10.3% passing the No. 200 sieve.

TABLE 43 Gradation specification comparison

Sieve Size	Type I	Type II	US54 North Base
1.0	100	100	100
¾	80-100	85-95	91
No. 4	30-60	40-70	53.6
No. 10	20-45	30-55	42
No. 200	3-10	6-15	10.3

PROCTOR

Proctor testing is essential for this project because all modulus of resilience testing is required to be done at optimum moisture content and maximum dry density. Even though the construction teams perform proctor tests the results are often too far from the point of interest for this project and need to be confirmed with a proctor performed by the UNM team. Two proctor tests were done: one for the subgrade on the US54 South subgrade and one on the US54 North subgrade.

US54 South

When the proctor test was performed the optimum moisture content was 9.8% at a maximum dry density of 131 lb/ft^3. This maximum dry density is high for a subgrade, the proctor was done as the standard specifies. Given that the values the construction team was getting were around 112lb/ft^3 it is recommended that repeat tests are done to verify these results. The higher density could be the result of the construction team using standard effort proctor hammers versus the modified hammer the UNM team uses. The proctor result is shown in Figure 30.

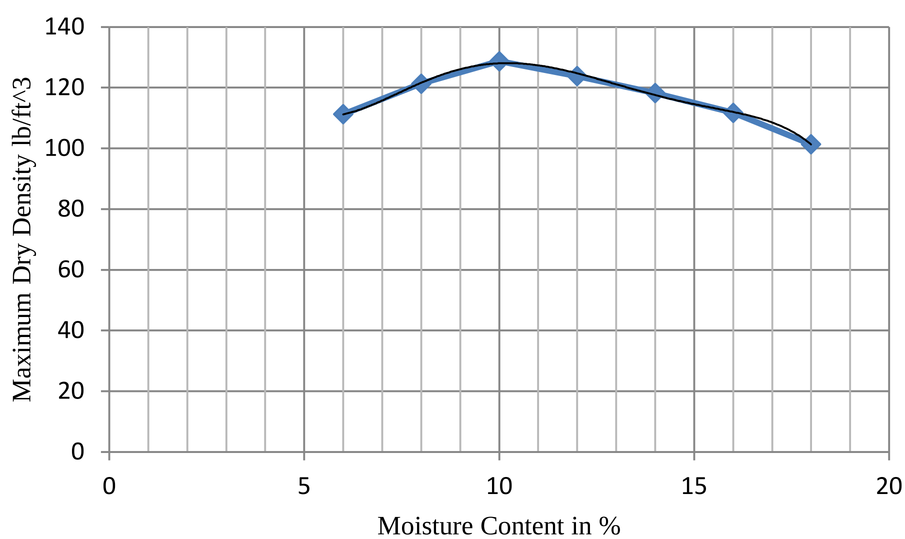

FIGURE 30 Proctor curve for US54 South

US54 North

The second proctor test was performed using the soil from the US54 North at station 150+00. This station had the least amount of large aggregates in the gradation. The proctor result is shown in Figure 31. The optimum moisture content was found to be 13.8% at a maximum dry density of 114.5lb/ft3. These values are reasonable for A-2 soil and are considered good.

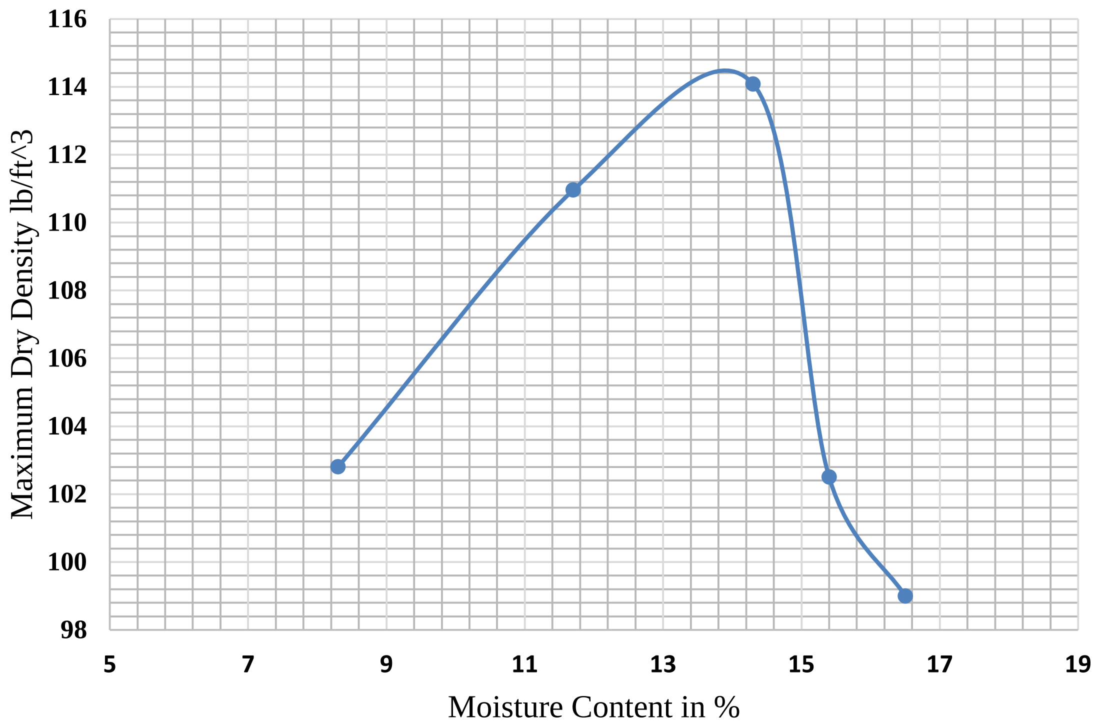

FIGURE 31 Proctor curve for US54 North at station 150+00

CONCLUSIONS

This report details the activities and results for the advanced calibration project that were accomplished during the first part of the study. Field testing and materials collection from three sites (two US 54 sites, one US285/I-40 interchange) out of five proposed sites have been completed so far. Only asphalt dynamic modulus E* laboratory testing has been done. Laboratory Fatigue Endurance Limit, FEL and resilient modulus, Mr testing have not been started yet. Few binder testing have been conducted. It can be noted that few field testing such as dynamic cone penetration (DCP) and Clegg hammer testing, FWD testing on subgrade and Base layers are additional to the originally proposed work, however in-depth data analysis have not been performed yet.

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