269-RU5179

PAVEMENT SUPPORT PROGRAM 2012

December 2013

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In cooperation with

New Jersey
Department of Transportation
Bureau of Research
and
U. S. Department of Transportation
Federal Highway Administration

DISCLAIMER STATEMENT

Report No. 269-RU5179	Government Accession No.	Recipient's Catalog No.
Title and Subtitle Pavement Support Program 2012		Report Date December 2013 ▪ Performing Organization Code CAIT/Rutgers
Author(s) Ali Maher, Patrick J. Szary, Nicholas Vitillo, Thomas Bennert, Nenad Gucunski, Brian Tobin, Hao Wang		Performing Organization Report No. **269-RU5179**
Performing Organization Name and Address Center for Advanced Infrastructure & Transportation (CAIT) Rutgers, The State University of New Jersey 100 Brett Road Piscataway, NJ 08854-8014		Work Unit No. Contract or Grant No.
Sponsoring Agency Name and Address N.J. Department of Transportation (NJDOT) 1035 Parkway Avenue P.O. Box 600 Trenton, NJ 08625-0600		Type of Report and Period Covered Final Report 07/01/12 – 09/30/13 Sponsoring Agency Code

Supplementary Notes

U.S. Department of Transportation/Research and Innovative Technology Administration (RITA)
1200 New Jersey Avenue, SE
Washington, DC 20590-0001

Abstract

The primary objective of the Rutgers Pavement Resource Program (PRP) is to utilize the extensive laboratory and field pavement testing equipment and staff expertise of the Pavement Resource Program in all aspects of Pavement Engineering to assist the New Jersey Department of Transportation's Pavement and Drainage Management Systems Unit in developing pavement management system strategies, innovative materials, improved pavement design tools, and advanced laboratory and field data collection equipment aimed at enhancing network condition by optimizing available capital resources. The primary goals of the current program are to:

1. Enhance the Department's Pavement Management System,
2. Provide support for implementation of Mechanistic-Empirical Pavement Design/Darwin-ME
3. Assist in the planning, design, construction and management of a NJDOT ride quality
4. Use NDT/NDE tools to examine pavement structures, enhance pavement information for pavement design, management programs, and quality assurance,
5. Develop a NJ-LTPP program to assess the pavements designed with the new M-E Pavement Design Guide (MEPDG) to determine the "as constructed" level 1 inputs for the MEPDG and enhance the predicted pavement performance models for 2010, top down and bottom up cracking and rutting, and
6. Promoting the development and implementation of tools to enhance the State's Environmental Stewardship in the Pavement area; specifically Quiet Pavements and use of Warm Mix Asphalt and Recycled Asphalt Pavement (RAP).

Key Words	Distribution Statement
Pavement Management System, MEPDG, Darwin-ME, NDE/NDT, Quiet Pavements, WMA, RAP	No Restrictions.

Security Classification (of this report)	Security Classification (of this page)	No of Pages	Price
Unclassified	Unclassified	162	

Form DOT F 1700.7

ACKNOWLEDGEMENTS

The author(s) wish to thank the New Jersey Department of Transportation (NJDOT) and the Federal Highway New Jersey Division for its support and assistance throughout the entire project.

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EXECUTIVE SUMMARY

The mission of Rutgers University's Center for Advanced Infrastructure and Transportation (CAIT) Pavement Resource Program (PRP) is to provide pavement engineering support to the New Jersey Department of Transportation (NJDOT)'s Pavement and Drainage Management Systems (P&DMS) Unit.

The activity was a partnership between federal and state transportation agencies and the academic institution of Rutgers University to provide technical and educational services to address transportation infrastructure in New Jersey. The Center supported the NJDOT by providing staff and resources to address pavement engineering, performance modeling, material characterization, operational issues, training, and other technical support as needed by the Pavement and Drainage Management Systems Unit.

The goal of the Pavement Resource Program was to assist in developing the tools and apply the resources of the Center to optimize the funds available through the NJDOT's capital program to improve the condition of New Jersey highway pavements. The condition of New Jersey's pavements has declined steadily over the past decade as available resources have been committed to other needs. The significant backlog of pavement maintenance and rehabilitation has resulted in a significant increase in vehicle operating costs to NJ motorists.

A fresh approach to pavement management using the latest technology was needed to help restore New Jersey's highway infrastructure to a state of good repair with limited available resources. The Pavement Resource Program served as an extension of the NJDOT's Pavement and Drainage Management Systems Unit and functioned as the primary research and technology arm to address the unit's needs. It was organized to rapidly respond to the Department's need for implementation of advanced pavement evaluation and asset management technologies.

The PRP worked to develop asset management tools, database architecture, material testing and evaluation, validation and implementation of new technologies, methodologies and materials. The services provided by the joint NJDOT/CAIT pavement engineering program included field and laboratory testing and evaluation, development of advanced pavement information systems, and specialized training/educational programs for NJDOT and its consulting pavement engineers.

INTRODUCTION

The primary objective of the Rutgers Pavement Resource Program (PRP) is to use the extensive laboratory and field pavement testing equipment and staff expertise of the Pavement Resource Program in all aspects of Pavement Engineering to assist the New Jersey Department of Transportation's Pavement and Drainage Management Systems Unit in developing pavement management system strategies, innovative materials, improved pavement design tools, and advanced laboratory and field data collection equipment aimed at enhancing network condition by optimizing available capital resources.

The primary goals of the current program are to:

1. Enhance the Department's Pavement Management System,
2. Provide ongoing support for implementation of Mechanistic-Empirical Pavement Design/Darwin-ME on an as needed basis to support the Department's $225 million annual paving program
3. Assist in the planning, design, construction and management of a NJDOT ride quality facility for the certification of equipment utilized by NJDOT, consultants and contractors for construction contract pay adjustments.
4. Use NDT/NDE tools to examine pavement structures, enhance pavement ; information for pavement design, management programs, and quality assurance,
5. Provide a supplemental modeling analysis for the implementation of Mechanistic-Empirical Pavement Design, and
6. Promoting the development and implementation of tools to enhance the State's Environmental Stewardship in the Pavement area; specifically by providing technical support and data collection to support the developing and NJDOT unofficial "Quiet Pavement Policy" developed by the Pavement Technologies Group and the examination of the use of Warm Mix Asphalt and Recycled Asphalt Pavement (RAP).

Task Summary

Pavement Management Systems

Background

The Pavement Resource Program agreed to continue to provide technical support to the NJDOT Pavement and Drainage Management Systems and Technology Unit by working with the unit staff in establishing and implementing a comprehensive pavement strategy toolbox that optimizes maintenance and capital investment by selecting the right fix at the right time on the right pavement.

The PRP agreed to develop an assessment plan to evaluate the latest implementation of dTIMS PMS and make recommendations for modifications where necessary. (dTIMS is anticipated to be fully implemented by June 2011 under the 2010 work plan). Based on the review, the PRP will make recommendations to refine pavement preservation treatment triggers, models, resets and costs.

The PRP agreed to develop a plan to monitor the selection of pavement preservation treatment strategies to evaluate the treatment decision trees or treatment rules, condition resets, costs, performance, life extension and failure criteria.

The PRP agreed to work with the unit staff and Deighton to finalize the NJDOT Deighton dTIMS user manual and training of NJDOT staff in using the dTIMS asset management software.

Work Performed

In the first quarter of the project, the PRP and NJDOT unit staffs developed complete dataset of updated pavement performance data curve for use in the network performance and economic models. The performance models were based on pavement treatments used on construction projects completed from 1999 through 2009. The data was separated for bituminous and composite pavements by treatment type. The models were summarized by minor and major rehabilitation for IRI and SDI. The models were based on regression analysis using the Excel solver program and built-in Excel model formats to maximize the R-squared value.

In the second quarter of the project, the PRP delivered the final report of the work it did to develop a complete dataset from the first quarter to NJDOT. The PRP began work preparing the performance and economic analysis for the NJDOT CIS unit for 2012.The PSP prepared a work plan to evaluate and implement the data from the new Pathway Profiler subsystems. The plan includes the profiler, rutting, GPS, texture lasers, distress survey, and right of way images and Pathways software.

In the third quarter, the PRP finalized the Performance and Economic Analysis for NJDOT CIS 2012 and delivered it to the NJDOT Pavement Management System unit. This report is not attached due to its confidential nature. The PRP finalized a work plan to evaluate and implement the data from the new Pathway Profiler subsystems. The plan includes the profiler, rutting, GPS, texture lasers, distress survey, and right of way images and Pathways software. The work plan was used to discuss the subsystems with the staff from Pathway, NJDOT PMS staff, and Rutgers staff. The PRP staff participated in the Pathway training on the new profiler and Pathway software. The PRP and NJDOT delivered the NJDOT dFRAG and NJDOT Construction Program Report procedures to Deighton for their consideration in refining their dTIMS version 9 software for NJDOT. The PRP is working with Deighton staff to schedule the next visit to NJDOT (April 2013) to include the implementation of the new Pavement Performance Models, and refinements to the treatment selection triggers, resets, and life extensions. Deighton will also demonstrate the new dTIMS version 9 software and discuss the transition for NJDOT.

In the fourth quarter, the PRP met with Deighton staff to incorporate the new Pavement Performance Models, and simplification of the treatment selection triggers, resets, and life extensions. Deighton staff modified the dTIMS models and established a “test” database for assessment of any further modifications. The PRP ran the Performance and Economic Analysis of the NJDOT CIS 2012 with the revised performance models and simplified pavement treatment triggers. The summary of the revised analysis was delivered to the NJDOT Pavement Management System unit. This report is not attached due to its confidential nature.

Ride Quality of New and Rehabilitated Pavements

Background

The PRP agreed to assist the Department in improving the repeatability and accuracy of quality assurance of pavement smoothness measurements by perform annual certification of NJDOT's walking, portable and high-speed pavement profilers using the pavement profiler certification test site and training NJDOT staff and industry staffs on the use of available pavement profiler tools.

The Pavement Resource Program and Advanced Infrastructure Design agreed to work with the NJDOT Pavement and Drainage Management Systems and Technology Unit to evaluate the new pavement and bridge ride quality specification on paving projects.

Work Performed

In the first quarter, the PRP continued to locate and evaluate potential Ride Quality Certification sites for NJDOT throughout the Rutgers University campuses and other locations. The Rutgers SurPro walking profiler, the NJDOT Bureau of Materials SurPro walking profilers and the NJDOT High Speed Profilers collected data on the test site and the data has been analyzed. The Rutgers SurPro walking profiler, the NJDOT Bureau of Materials SurPro walking profilers (except Region South) and the NJDOT High Speed Profilers (ICC and Dynatest 147) have been certified. The NJDOT Bureau of Materials SurPro walking profilers (Trenton) was analyzed.

In the second quarter, the PRP located a candidate testing site on Route I-295 S in the former rest area. The NJDOT will need to repave the surface to provide a qualified test site. The PSP delivered a summary report of the certification of the NJDOT walking, portable, and high speed profilers for 2012.

In the third and fourth quarters, AID delivered a final report with recommendations to refine the NJDOT current ride quality specification. The PRP agreed to refine the recommendations with NJDOT staff for implementation. The PRP developed a Ride Quality Specification Analysis Tool to refine the current ride quality specification equations. The tool allows the modification of each portion of the specification equations to assess the effects on the overall pay adjustment. The PRP used the Ride Quality Specification Analysis Tool to help the NJDOT staff refine a new RQ specification for microsurfacing.

Mechanistic-Empirical Pavement Design Guide (MEPDG)/Darwin-ME

Background

The Pavement Resource Program agreed to providing technical support to the NJDOT Pavement and Drainage Management Systems and Technology Unit to Support NJDOT's efforts to implement the latest pavement design procedures (Darwin-ME) in NJ.

The PRP agreed to work with the unit to develop model calibration and training, continue the establishment of pavement calibration sites for Darwin-ME model refinements, collect traffic data and perform material characterization (e.g., HMA binder and mixture, granular and subgrade materials) at the selected locations and run MEPDG analysis and compare analysis results to measured data.

The PRP agreed to concentrate on the continual calibration of the flexible rehabilitation distress models, as well as composite pavement (i.e. asphalt overlay on PCC) pavements. The continual calibration will utilize material collection and performance testing, while continuing to measure the pavement distress level over time. The composite pavement program will look at both field measurements of the current pavement structure, as well as collecting materials for performance testing. PRP will reach out to the Texas Transportation Institute (TTI), who is the current contractor of NCHRP Project 1-41, Models for Predicting Reflective Cracking of Hot-Mix Asphalt Overlays, to determine what the key parameters will be for proper calibration of the upcoming Mechanistic Empirical Pavement Design Guide reflective cracking models. It is proposed that a minimum of five (5) test sections will be utilized for the calibration of the Darwin-ME reflective cracking models.

The PRP agreed to work with NJDOT Traffic staff to develop traffic inputs for Darwin-ME.

The PRP agreed to work with the NJDOT to develop specifications for longitudinal joint evaluation through literature search, survey of other states, and laboratory and field trials of various products and procedures.

The PRP agreed to perform an evaluation of urethane grouts and installation procedures and tools for undersealing of composite or concrete pavements.

The PRP agreed to develop a Construction Quality Assessment (Report Card-good paving practices) [from plant to end of construction]. [milling, tack/polymer joint adhesive, compaction, MTV, paver operation]

The PRP agreed to evaluate PMS pavement condition data collection to support Darwin-ME calibration.

The PRP agreed to assist the Department and the State GIS office in implementing the Rutgers Soil Engineering GIS layer and database to complement the Soil Boring Management System, familiarize the NJDOT staff with the Rutgers Soil Engineering GIS layer for subgrade soil type locations (Rutgers Soils Maps), material characterization, and properties for use in Darwin-ME and develop a web-based software tool to make the Rutgers Soil Engineering information available to NJDOT and consultant pavement engineers.

The PRP agreed to work with the unit staff to integrate a GIS mapping of the Project Tracking System for experimental material application and preventive maintenance/pavement preservation treatments, locations, properties, and performance. This database and GIS tool will provide the means to track innovative treatment locations (constructed by Maintenance Operations personnel or contracts), and assess performance, and costs.

Work Performed

In the first quarter, the PRP used the two copies of the Darwin ME software from AASHTO to examine the data inputs and determine how it could be used by NJDOT. A training program will be developed based on the new Darwin ME software input requirements. The PRP staff has developed a plan to compare the pavement designs from the Darwin 3 and Darwin ME software. The PRP will work with Narinder to select the comparison projects from the pavement consultants pavement evaluation and design reports. The PRP is modifying the Darwin ME user manual for the NJDOT pavement design staff to facilitate the use of the new Darwin ME pavement analysis procedure. Companion Excel spreadsheets are being prepared to be used as input tables for HMA binder, modulus, and other HMA inputs.

In the second quarter, the PRP staff continued to work to compare the pavement designs from the Darwin 3 and Darwin ME software. The PSP met with the NJDOT staff to discuss materials input for new pavements for the Darwin ME software. These material inputs will be part of the Darwin ME user manual and will be provide a stand-alone Excel spreadsheets for use by pavement designer.

In the third quarter, the PRP continued to work with the material inputs and they provided input to several national surveys including NCHRP survey on the implementation of MEPDG or Darwin ME in NJ.

In the final quarter, the PRP modified the Darwin ME user manual for the NJDOT pavement design staff to facilitate the use of the new Darwin ME pavement analysis procedure. Companion Excel spreadsheets are being finalized for use as input tables for HMA binder, modulus, and other HMA inputs. These material inputs will be part of the Darwin ME user manual and will provide stand-alone Excel spreadsheets for use by pavement designers. The PRP reviewed the Darwin ME characterization of existing pavement material inputs for use in pavement rehabilitation analysis. The PRP reviewed the Darwin ME characterization of traffic data for use in pavement rehabilitations. The simplified approach will examine the use of “family” of Pavement data based on functional classes.

Non-Destructive Evaluation/Testing for Condition Assessment and QA/QC

Background

The Pavement Resource Program agreed to continue to provide technical support to the NJDOT Pavement and Drainage Management Systems Unit to:

- Work on the characterization of Rubblized Portland Cement Concrete Pavement (RPCCP)
- Provide field validation of Darwin-ME models using NDE Technologies; and
- Quality Assessment of Compaction of HMA Pavement Layers and Density or Air Voids of Longitudinal Joints.

Work Performed

For the first quarter and for the characterization of rubblized concrete task, a resident engineer has been assigned and contacted about this project and the project was expected to start in October 2012. For the Field validation of MEPDG models task, the PRP staff, including NDE and MEPDG staff, have met to discuss an appropriate work plan. For the quality assessment of compaction of HMA layers and joints using the PSPA task, it was agreed to perform that with the field validation task. And for the use of GPR to predict Air Void Content in HMA task, a meeting was held with NJDOT to identify the workplan deliverables needed. A preliminary workplan was developed and submitted to the Rutgers PSP staff for review. The work plan is being revised based on their comments. Several construction projects and resident engineers were identified for the work. A few of the Resident Engineers were contacted and plans were made for testing.

For the second quarter, the rubblization project was delayed even further. GPR to predict air void content and compaction level – During this quarter the NDT team collected GPR data on seven sites. Of the seven, good quality data was collected at four sites. Data from the remaining three sites were not good due to rain and other circumstances affecting GPR data collection. Additional opportunities for GPR data collection will be pursued until there will be at least five good sets of data. The data analysis will start with the end of November. PSPA asphalt modulus data for QA/QC and field validation of Darwin-ME models - The team collected asphalt modulus data on one site, in parallel to the GPR survey. The data were collected on approximately 600 feet long section along three survey lines and with a 10 foot test point increment. The intention was to collect data on more sections. However and unlike the GPR surveys, which are being conducted with the NDT staff in a van, the PSPA data collection is conducted with a cart and an operator. It was assessed that the conditions at the sites were not safe enough to permit the PSPA data collection.

For the third and fourth quarters and QA/QC using NDT/NDE Technologies, the PRP used GPR to predict air void content and compaction level. During this quarter the NDT team analyzed the GPR data collected on seven sites. Of the seven, good quality data was collected at five sites. Data from the remaining two sites were not good due to rain and other circumstances affecting GPR data collection. There are two parts to the analysis: 1) determination of dielectric constants at core locations, and 2) preparation of dielectric contour plots for each of the lots tested. The GPR scans at each core location exhibited very little variation in the dielectric value within several inches from the core location. Based on the received information for the recovered cores for each of the lots, and the measured dielectrics, %air void vs. dielectric constant was plotted for each material type. The majority of the data was from a 12.5m76 mix and a 12.5m64 mix, and only a few data points were from an SMA. Of the five lots, the dielectric value contour plots were completed for three of them. The dielectric value plots will be converted to %air voids plots using the relationship between the dielectric value and %air voids. For the PSPA asphalt modulus data for QA/QC and field validation of Darwin-ME models task, the asphalt modulus data collected on one site were analyzed and the data presented in terms of contour plots. In addition, the moduli obtained on an approximately 600 feet long section along three survey lines were analyzed to assess the modulus variability across the paved area. The average values of moduli in the middle section of the lane were only slightly higher, about five percent, than along the line close to the joint.

Promote the Development and Implementation of Tools to Enhance the State's Environmental Stewardship in the Pavement Area

Background

Quiet Pavements

The PRP agreed to work with the NJDOT task force in developing criteria on the use of quiet pavements in NJ. The PRP will conduct a Noise study on new pavements and rehabilitated pavements utilizing road side and at-the-source noise measurement of various pavement surfaces to determine relationships under different climatic (wind), speed, traffic levels, and geometric conditions. The PRP will continue to collect QPPP data on the "quiet pavement surfaces" for the 2nd of the required 7 year data collection program The data will be collected seasonally (4 times per year) on a minimum of 10 pavement sections to assess seasonal variations in pavement-tire noise generation. The data collected will eventually be transferred into a GIS based database to determine pavement noise "hot spots" in the state of New Jersey. The PRP will evaluate and implement the results of NCHRP study on quiet pavements.

Recycled Asphalt Pavement (RAP)

The PRP agreed to continue to perform laboratory testing to optimize the use of RAP in balancing recycling efforts with enhancing pavement performance

Warm Mix Asphalt (WMA)

The PRP agreed to promote and evaluate the use of Warm Mix Asphalt in reducing air pollution, while maintaining pavement performance

Work Performed

In the first quarter, the PRP finalized the High RAP mixture design performance testing for RE Pierson on I295. Two sets of mixture designs were finalized; a 25% RAP surface course mix and 35% RAP intermediate base course mixture. The project is set to be constructed towards the middle/end of August. The PRP conducted a WMA Implementation study for a Evotherm WMA produced by RE Pierson for a NJ RT 40 project. The testing was conducted in conformance with the NJDOT WMA Implementation specification. The "hybrid" WMA consists of a foamed asphalt with 0.2% Evotherm. The benefit of this WMA is that contractors can utilize the foamed technology to reduce production temperatures, but then get an extra "boost" in workability and moisture damage resistance, with the Evotherm product. Evotherm is a pre-approved anti-strip in a number of states across the country right now, therefore, for this mixture, it is providing a workability and anti-strip performance. The PRP continued to conduct pavement noise measurement data to provide NJDOT with noise-reducing options with respect to pavement selection. The PRP collected the data quarterly in an effort to reduce possible changes due to environmental conditions and believes that with a few more years of data, they can produce an algorithm that “normalizes” the tire-pavement noise to a constant, or average, pavement temperature for comparison purposes. The Pavement Noise Group also finished evaluating different tire types in an effort to provide recommendations to NJDOT on “quieter” NJDOT state vehicles. The study concluded that the Continental ProContact, advertised by the manufacturer to be a low rolling resistance, low CO_2, and high mileage tire, was the quietest of the four (4) different tires evaluated – all tires evaluated were advertised as “quiet” and environmentally friendly. Therefore, when possible, it is recommended that NJDOT look to retrofit all state vehicles with this type of tire.

The second quarter was very busy. The PRP analyzed and provided a final report to the NJDOT regarding the new NJDOT High RAP specification. The project resulted in using a 25% RAP mixture as the surface course and a 35% RAP mixture in the intermediate course. A final report and presentation made at the Northeast Asphalt User’s Producer Group (NEAUPG) 2012 conference is accompanying this Quarterly Report. (Appendix A) The PRP continued to evaluate different Warm Mix Asphalt (WMA) mixtures produced in New Jersey under the NJDOT WMA Pilot Program. A majority of the mixtures evaluated this quarter were a Stone Matrix Asphalt (SMA) produced with WMA technologies. The NJDOT Materials Bureau has been receiving these reports. The PSP also completed a study that summarized the fatigue cracking performance of the NJDOT’s Long Term Pavement Performance (LTPP) SPS-5 test sections. The SPS-5 sections looked at evaluating the long term pavement performance of a Virgin (no RAP) and 30% RAP mixture. Identical pavement, traffic, and climate conditions were available for a direct comparison of performance. The field investigation and laboratory testing identified the Virgin mixture as performing better in fatigue cracking when compared to the 30% RAP mixture. A report is accompanying this Quarterly Report that summarizes the evaluation. (Appendix B)

The PSP conducted a scoping study to initiate the development of a 4.75mm SMA mixture specification to be used for Pavement Preservation treatments. The mixture would utilize crumb rubber as the asphalt binder modifier. The summary report accompanies this Quarterly Report. (Appendix C)

The PRP noise group has continued to monitor the NJ long term pavement noise sections listed in Table 4.1. In addition to the long term sections, the focus has been on testing more sections statewide to build the database. These sections include a DGA on I-80, a concrete section on I-287, and a section on Middlesex Co. Rt. 522. As noise concerns were highlighted in popular media by tire companies and the evolution of quiet tires, a preliminary consumer tire noise study was conducted to better understand the different acoustic properties of three types of consumer tires on three different pavement surfaces. The consumer tire noise study evaluated 4 tires with a focus on a “quiet” tire, the Bridgestone Ecopia, a “Low-Rolling Resistance” tire, the Continental Eco-Plus, and a “winter” tire with a more aggressive tread, the Firestone Winterforce, while using the Standard Reference Test Tire (SRTT) as a reference metric. Figure 4.1 shows the overall noise levels measured for each section with each tire. The typical noise result from each pavement was recognized for all of the tires, where the concrete on I-287 was the loudest and the OGFC on I-78 was the quietest. On each pavement, there were differences noticed between each tire, where the Firestone Winterforce was the loudest and the Continental Eco-Plus was typically the quietest. Figure 4.2 shows the one-third octave band measurements recorded for the SRTT. It shows the typical spectral patterns associated with each pavement type. Figure 4.3, 4.4, and 4.5 highlights the spectral patterns measured for the Continental Eco-Plus, the Firestone Winterforce, and the Bridgestone Ecopia respectively. By comparing the different spectral patterns we can determine differences in noise quality between the different tires that were tested.

The second project, which is partially completed, involves an evaluation of micro-surfacing and chip seal surfaces. The chip seal surfaces were recently placed in Branchburg Township, Somerset County, on 3 different local roads. Testing was conducted at 25mph, 45mph, and 60mph, space permitting. 25mph was utilized to evaluate the micro-surface for the Otto and Briar Road sections, where the speed limit was 25mph. Otto Road had enough space to also conduct testing at 45mph, which was used to relate the measurements to the other chip seal paved on River Road. The speed limit on River Road was 45mph; the testing was conducted at 45mph and one test area enabled a 60mph test on that surface. The 60mph section enables a comparison to the interstate test sections that have been conducted thus far. Figure 4.6 shows the overall values recorded for the chip seal sections at 60mph, 45pmh, and 25mph, while Figure 4.7 shows the one-third octave band assessment of the same sections. The overall noise levels show that the chip seal surfaces at 60mph were loud compared to the NJ interstate surfaces, while the two roads were similar to each other at 45mph. The spectrum analysis also shows that there was some difference between the two chip seals between 1000 Hz and 5000 Hz. More testing would be needed to determine the cause of this discrepancy.

Five test sections were provided by DOT that had been recently micro-surfaced. Thus far, two of the sections have been tested. To show the comparison between thin-lift in-service pavements in NJ, various sections have been compared below. Figure 4.8 shows the measured overall values for the micro-surfaced pavements tested thus far, as well as a high-performance thin lift overlay on I-280, a polymer modified Open-Graded Friction Course on the Garden State Parkway, an Asphalt Rubber Open-Graded Friction Course on I-78, and the Chip Seal tested at 60mph in Branchburg on River Road. Figure 4.8 shows the overall value comparison of the thin-lift surfaces measured in NJ. The chip seal was the loudest, followed by the recently micro-surfaced sections on I-287 and I-24. The OGFC and HPTO sections were significantly quieter by 5-7 dBA. Figure 4.9 shows the one-third octave band for the thin-lift sections. The Branchburg chip seal was the loudest overall, which is emphasized by the elevated low-end levels from 400-1000 Hz on the frequency spectrum. The micro-surfaced sections followed similar spectral trends resembling fairly loud DGA surfaces, while the OGFC pavements show the attenuation of high frequencies in the upper register from 630 Hz to 5000 Hz and the small aggregate size associated with the HPTO showed less pounding noise at the lower end from 400-1250 Hz and more high-pitched stick-slip noise at the higher end from 2000-5000Hz.

The micro-surfacing was completed on 2 lanes of the westbound direction on Rt-24 and in all three lanes on both directions of the I-287 section. Figure 4.10 shows the overall comparison of differences between the right, center, and left lanes for the micro-surfaced sections, while Figure 4.11 shows the one-third octave band spectrum analysis for the same sections. Each micro-surface exhibited high overall levels. The left lane, presumably less traveled on each road, was slightly less loud. On I-287, the center and right lanes showed similar overall levels, but had reduced levels in the left lane. Since the micro-surfacing section on I-287 was just north of the I-80 interchange, loaded trucks are increasing the rate of wear on this surface in the right and center lanes. Figure 4.11 shows that the most affected frequencies on the one-third octave band spectrum for the right lane of Rt-24 and the right and center lanes of I-287 was the high end from 1250 Hz to 5000 Hz, showing that the surfaces were possibly becoming more smooth with traffic loading.

Table 4.1: NJDOT Test Pavement Noise Test Sections and Respective Mileposts

GSP	6-11
GSP	22-28
GSP	38-48
GSP	58-63
GSP	97-102
GSP	117-124
I-78	10-26
I-78	34-45
I-80	45-53
I-95	2-8
I-195	8-12
I-280	7-11
I-287	58-60
I-295	61-49
Rt-202	14-18

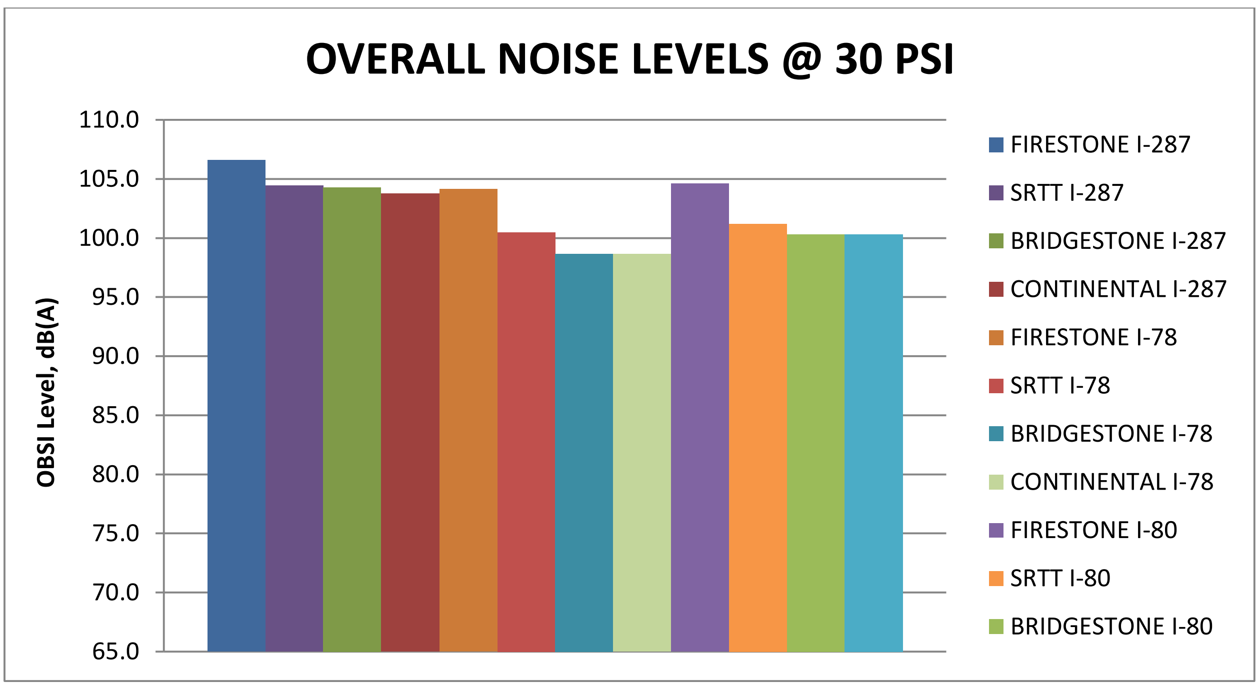


Figure 4.1: Consumer Tire Study Overall Noise Levels

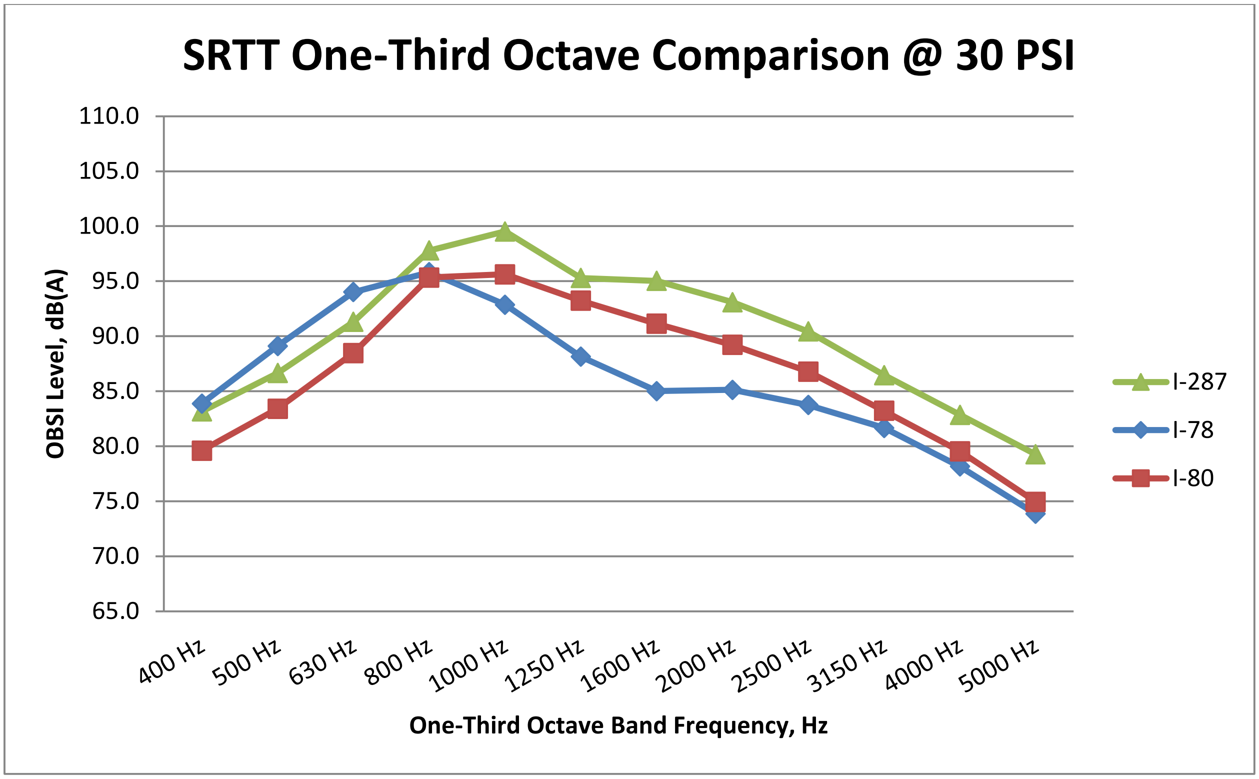


Figure 4.2: Consumer Tire Study, 30 PSI SRTT

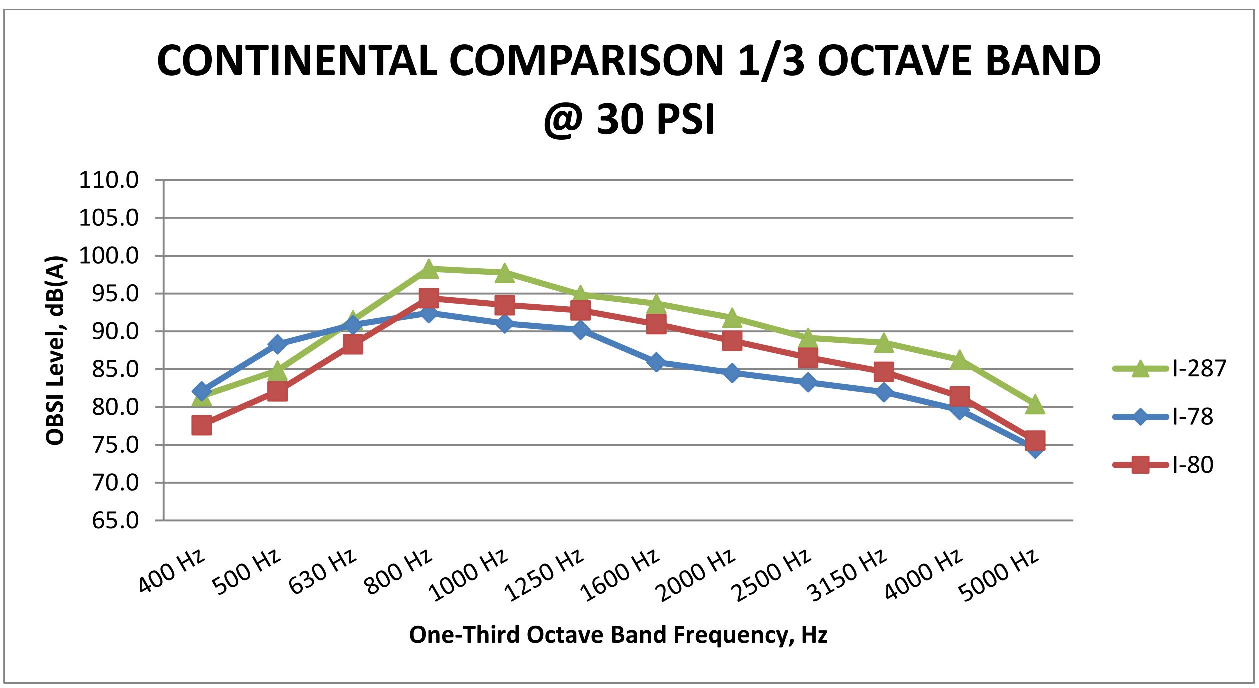


Figure 4.3: Consumer Tire Study, 30 PSI Continental Eco-Plus

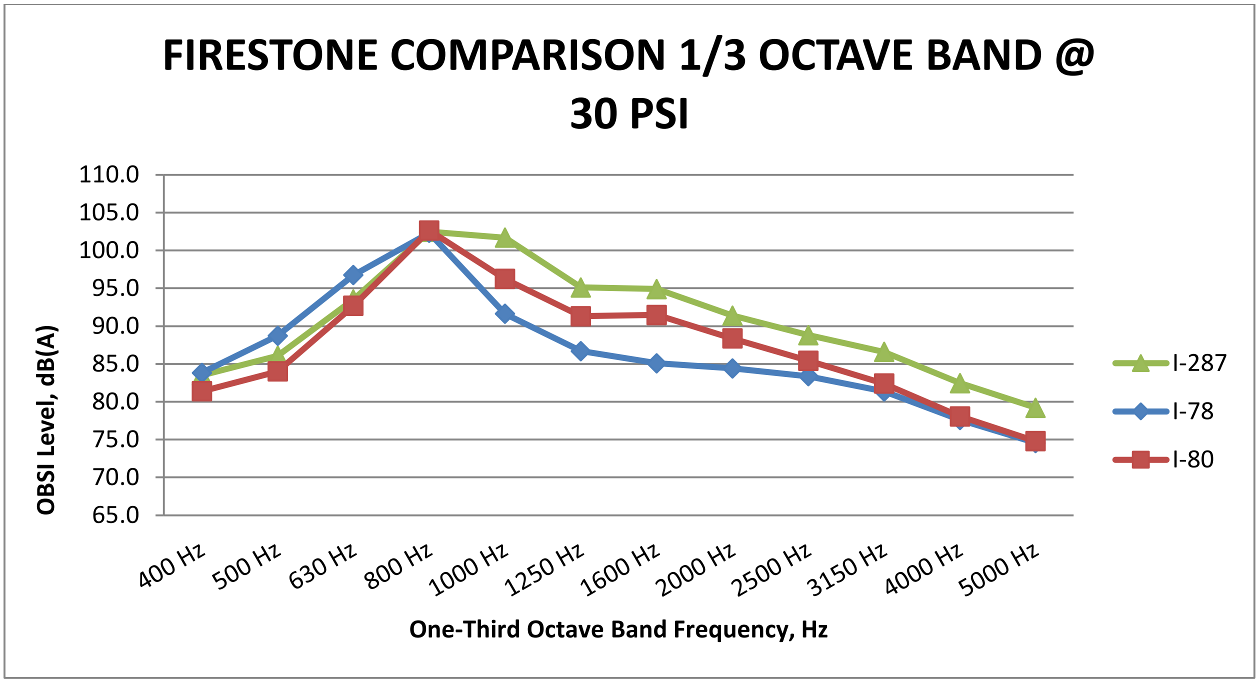


Figure 4.4: Consumer Tire Study, 30 PSI Firestone Winterforce

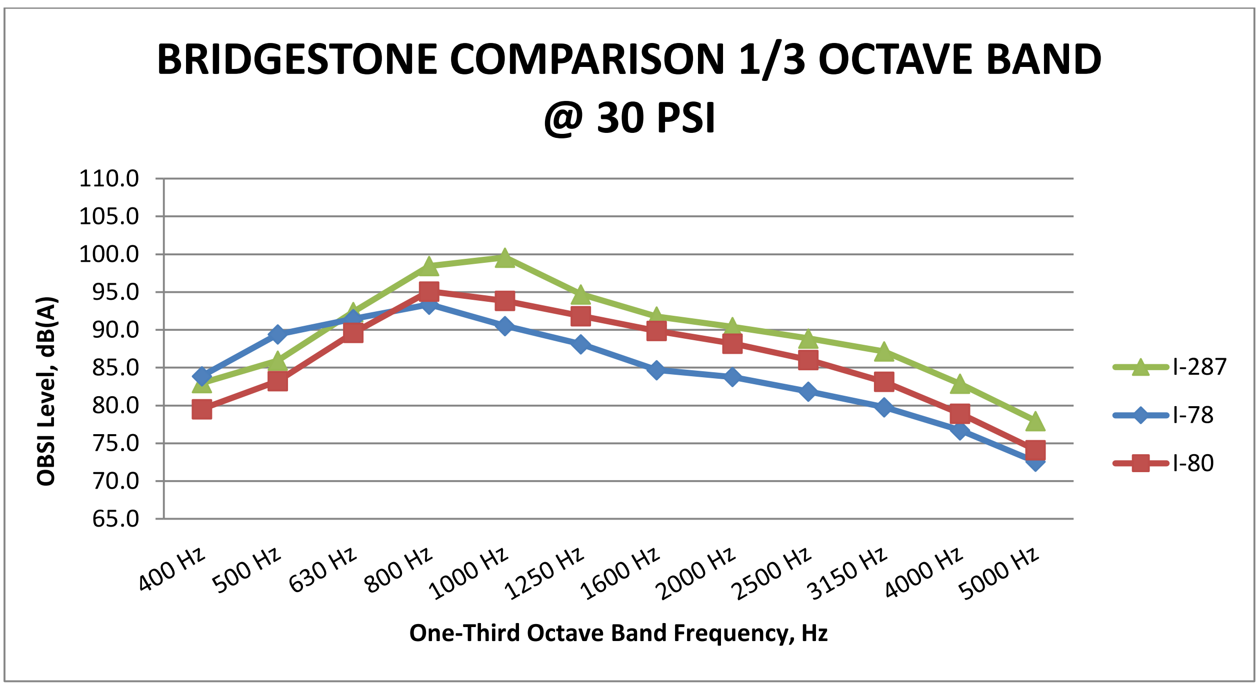


Figure 4.5: Consumer Tire Study, 30 PSI Bridgestone Ecopia

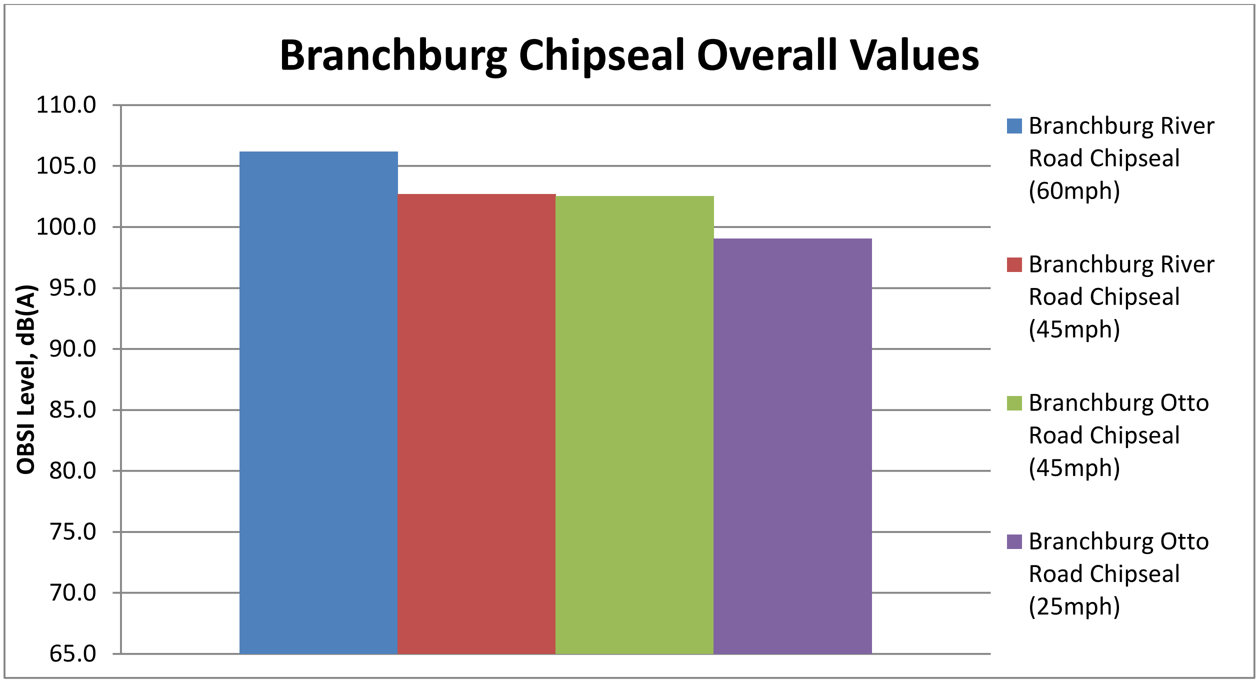


Figure 4.6: Chip Seal Overall Values

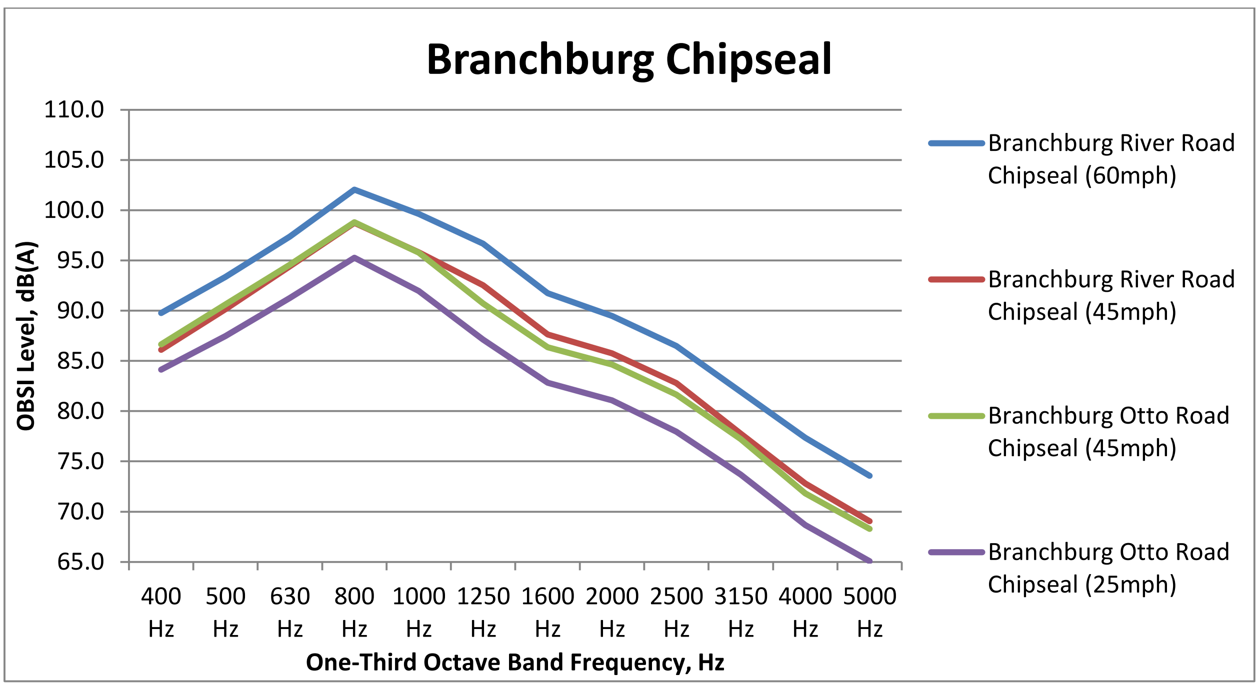


Figure 4.7: Chip Seal Spectral Comparison

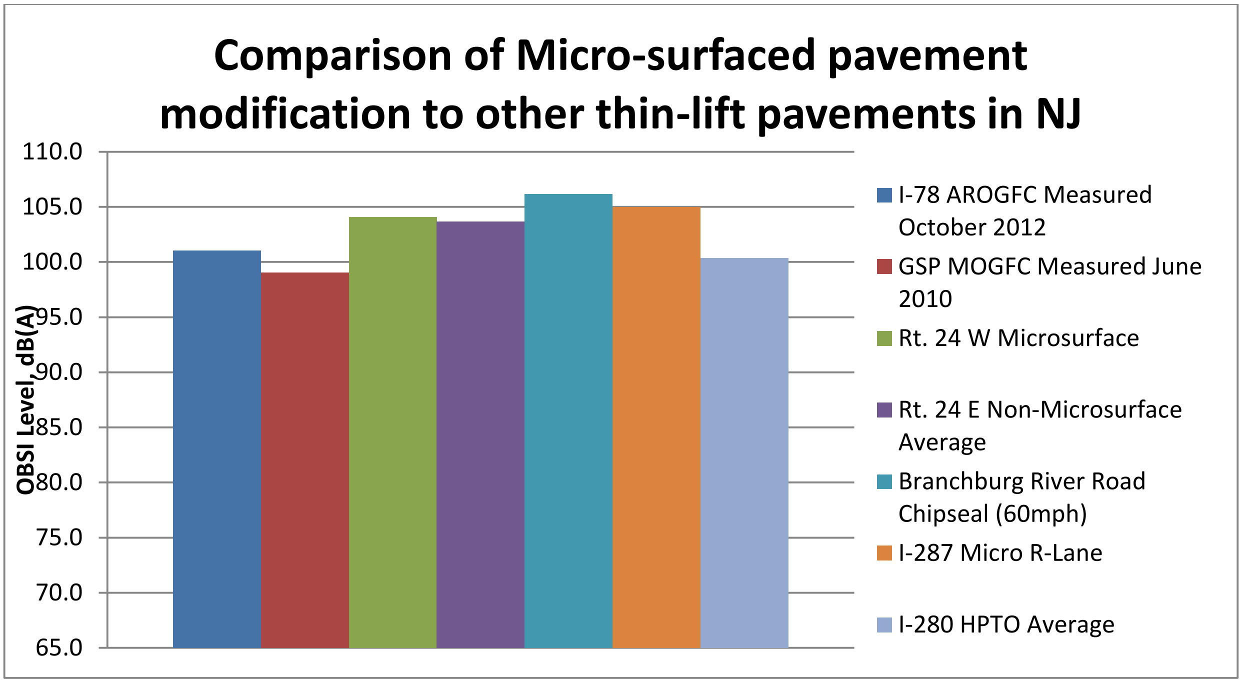


Figure 4.8: Overall values measured for thin-lift pavements in NJ

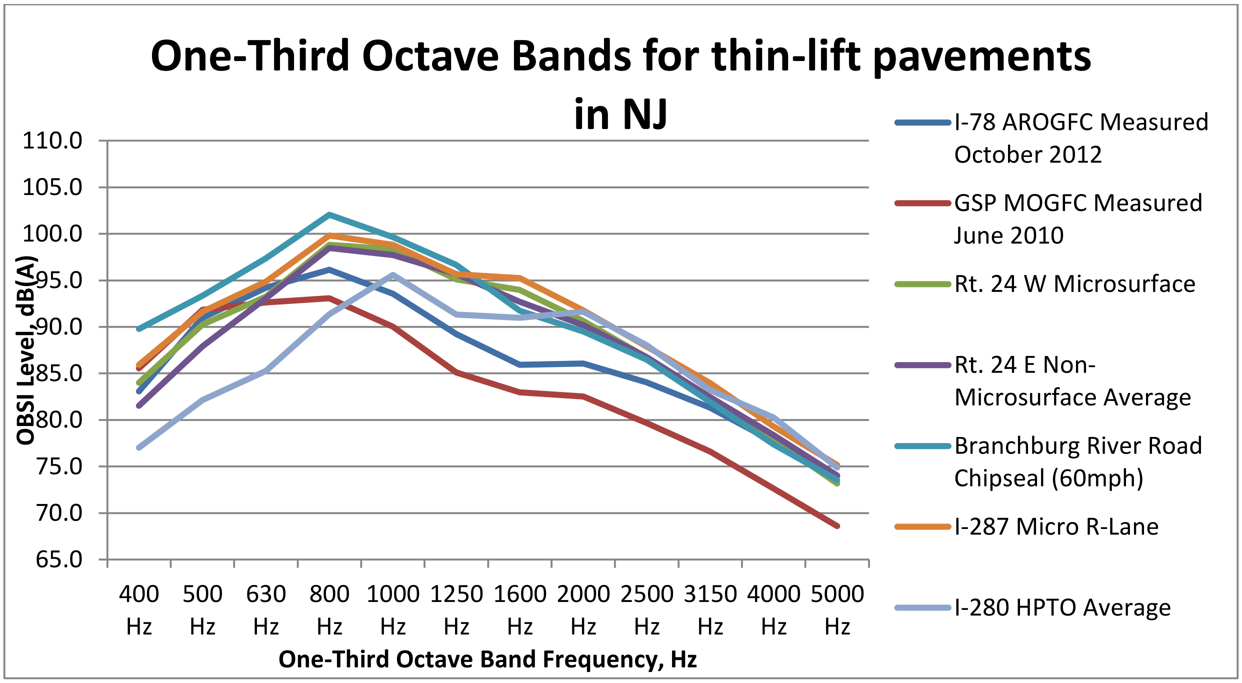


Figure 4.9: Spectral analysis for thin-lift pavements in NJ

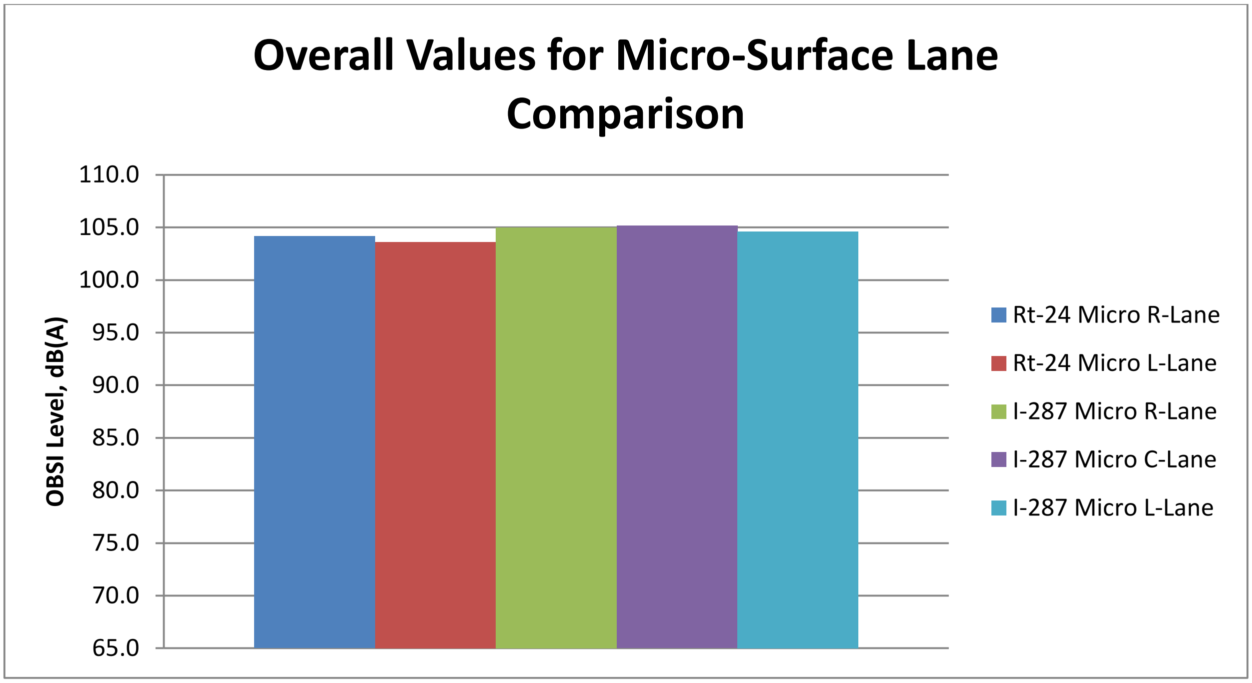


Figure 4.10: Overall level lane variation for micro-surfaced sections

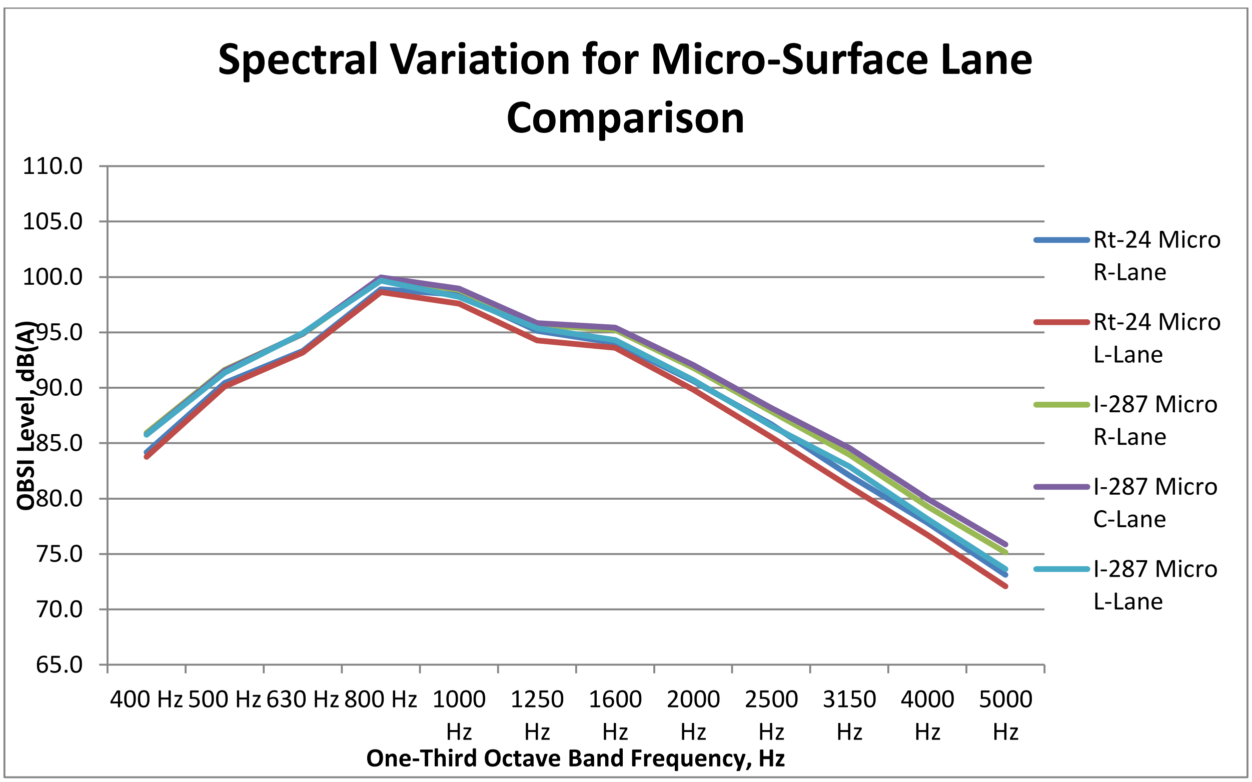


Figure 4.11: Spectral Variation for micro-surface lane comparison

In the third quarter, the PRP began the mixture design work for the asphalt rubber gap-graded (ARGG) mixture for possible use as a pavement preservation/rehabilitation treatment. A 30 mesh crumb rubber was procured for blending at the laboratory. Optimum asphalt content determination has begun and it is hopeful some initial designs will be completed soon. The noise testing program continued this quarter. The pavement noise group is compiling research and developing a report that summarizes the pavement noise properties of NJ pavement preservation treatments. These include OGFC, micro-surfacing, Nova-chip, and asphalt rubber chip seals. The noise study of pavement preservation surfaces will hope to compliment the current research being conducted for the Pavement Management group.

In the fourth quarter, the PRP began the mixture design work for the asphalt rubber gap-graded (ARGG) mixture for possible use as a pavement preservation/rehabilitation treatment. However, the during the design phase work, the gyratory compactor has broken down and we are in the process of procuring a new compactor. The noise testing program continued this quarter. The pavement noise group is compiling research and developing a report that summarizes the pavement noise properties of NJ pavement preservation treatments. These include OGFC, micro-surfacing, Nova-chip, and asphalt rubber chip seals. The noise study of pavement preservation surfaces will hope to compliment the current research being conducted for the Pavement Management group. The Pavement Management group has also provided the PRP with skid resistance values for these test sections to help compliment the noise measurements. It is hopeful that a function rating system can be developed using both surface noise and skid resistance measurements to help select pavement preservation treatments.

Supplemental Modeling Analysis for Implementation of Mechanical-Empirical Pavement Design

Background

The NJDOT Pavement Research Program agreed to work with the unit to evaluate the in-situ pavement structure capacity using advanced analysis of deflection basin data and different backcalculation algorithm and models. The results will provide accurate layer modulus input for pavement overlay design and enhance the implementation of mechanistic-empirical method for pavement rehabilitation and pavement overlay design. Proposed Deliverable: The preliminary analysis will perform modulus backcalculation using the existing FWD deflection data provided by NJDOT. Analysis will be conducted to evaluate the sensitivity of overlay design thickness subject to the modulus values backcalculated from different methods. The preliminary analysis results will identify areas where further research is needed in the FWD data analysis.

The NJDOT Pavement Support Program agreed to enhance the implementation of mechanistic-empirical pavement design method by comparing DARWIN-ME with other available software and models, such as CalME, MnPave, PerRoad, and performance models developed from recent NCHRP and pool fund studies. This will help the unit improve the understanding of pavement performance models and also contribute to the development of performance-related specification (PRS) for quality assurance.

Proposed Deliverable: The literature review will be conducted by comparing the features, capabilities, and limitations of different M-E software and models. The results will be summarized in a technical report and submitted to NJDOT.

Work Performed

The PRP reviewed the current available mechanistic-empirical pavement overlay design tool that are developed by state DOTs in addition to the AASHTO DARWin-ME. This includes WinFLEX (IDDOT), FPS 21 (TXDOT), EVERPAVE (WSDOT), MnPAVE (MNDOT). Table 4.2 compares the features of the currently available M-E design tools developed by state DOTs and as well as the new AASHTO DARWin-ME. The comparisons mainly focus on flexible pavement and overlay design. Although the M-E design tools developed by state DOTs follow the similar M-E design principle, they are relatively simple in the design inputs and easy for implementation compared to DARWIN-ME. The procedures and features of these different design tools were reviewed and compared in the aspects of traffic loading, characterization of asphalt and unbound material, climate, structure model, performance prediction, and reliability.

The PRP conducted a case study to compare the overlay design results using different design methods. The selected design methods include the AASHTO 1993 empirical design method and the mechanistic-empirical design method using EVERPAVE, MnPAVE, and DARWIN-ME. In the case study, existing pavement structures and the FWD deflection data were extracted from an early published NJDOT research report - Development of FWD Procedures Manual (FHWA-NJ-2009-005).

There are a lot of differences between the empirical AASHTO approach and the M-E design method, including the input parameters on traffic, material, environment and the existing pavement condition. The major difference is that the AASHTO 1993 method estimates the effective structure number of existing structure for overlay design; while the backcalculated modulus of each existing pavement layer is used in the M-E design. In addition, the design criteria used by the empirical method and the M-E method are fundamentally different. The AASHTO 1993 guide designs pavements to a single performance criterion, the present serviceability index (PSI), while the M-E design method considers the specific performance criteria (e.g., rutting, cracking). It is expected that these factors will result in different overlay design thicknesses.

Figures 4.12 and 4.13 show the overlay design thickness using different methods. The case study results show that the overlay design thickness based on the empirical AASHTO approach is much smaller than the ones designed with M-E methods. Generally, the overlay thickness designed with DARWN-ME is the greatest among the ones designed with different methods. As expected the overlay design thickness increases as the traffic level increases regardless of the design method.

There are a lot of inputs required in the DARWin-ME to conduct overlay design. It is important to know how these inputs affect the design results. It is expected that the sensitivity analysis results will help identify the significant factors that affect the overlay design results in the DARWin-ME.

The PRP evaluated the effect of existing pavement condition and overlay material property on AC overlay design and performance. The newest AASHTO design software DARWin-ME was used to conduct the analysis under various traffic and structure combinations. The factors considered in the analysis include the modulus of exiting layers, the rut depth of existing layer, the interface condition between AC overlay and existing pavement, and the properties of AC overlay (performance grade and Poisson's ratio). In addition to overlay thickness design, pavement performance analysis was conducted to see the effect of existing pavement condition and overlay material property on individual distresses.

Figures 4.14-4.17 show the most significant results from the analysis. Several findings were concluded from this study. First, the sensitivity of overlay design thickness to the condition of the existing AC layer depends on the existing AC layer thickness and design traffic. Second, the existing condition of base and subgrade has no significant influence to the overlay performance and design. Third, the performance analysis results indicate that the modulus of existing AC layer and interface bonding condition have more significant effects on fatigue cracking than on rutting potential. (Appendix D)

Table 1 Features of Mechanistic Empirical Design Tools for Flexible Pavements

Agency	IDDOT	TXDOT	WSDOT	MNDOT	AASHTO
Software/ Tools	WinFLEX 2006	FPS 21	EVERPAVE	MnPAVE	DARWin-ME
Level of complexity	Low			Medium	High
Traffic	ESAL	ESAL	ESAL	ESAL and axle load spectrum	Axle load spectrum
Asphalt Material	Design modulus adjusted by temperature	Design modulus	Design modulus adjusted by temperature	Design modulus or mixture volumetrics	Three level inputs
Unbound Material	Elastic modulus	Elastic modulus	Stress-dependent modulus	Elastic modulus	Three level inputs
Structure	Layered Elastic (CHEVRON)	Layered Elastic	Layered Elastic (WESLEA)	Layered Elastic (WESLEA)	Layered Elastic (JULEA)
Climate	Seasonal adjustment	NA	Seasonal adjustment	Seasonal adjustment	Enhanced Integrated Climatic Model
Performance prediction	Fatigue cracking and HMA rutting	Fatigue cracking and HMA rutting; subgrade shear failure	Fatigue cracking and HMA rutting	Fatigue cracking and HMA rutting	Fatigue and top-down cracking; HMA and subgrade rutting; roughness
Reliability	NA	NA	Adjustment to design ESAL	Monte Carlo simulation	Normal distribution for each distress
Overlay design module	Yes	Yes	Yes	Yes (FWD analysis included)	Yes

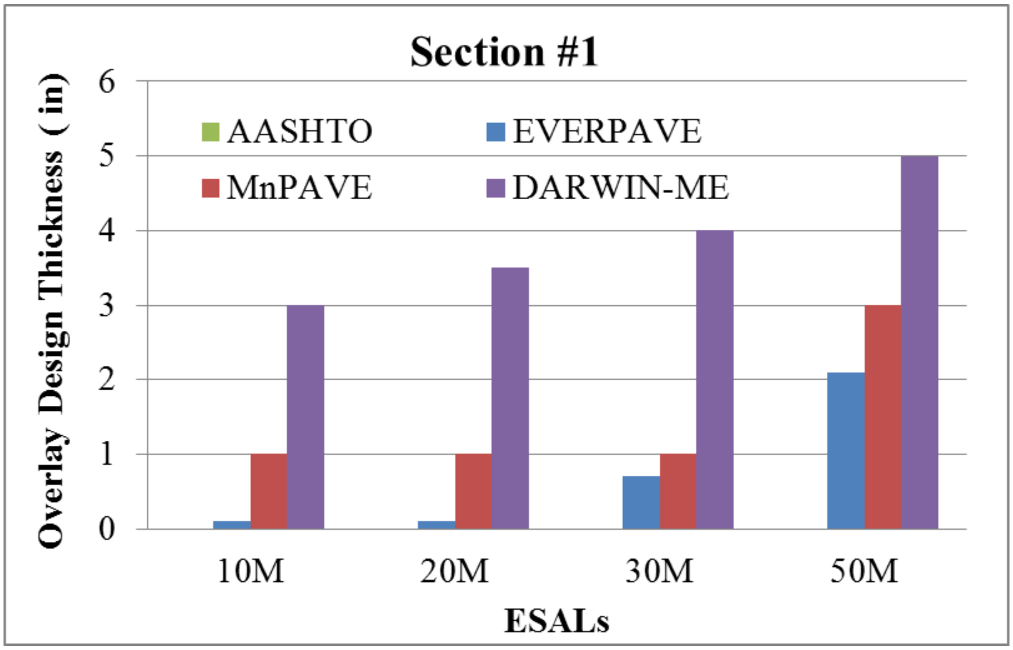


Figure 4.12 Overlay Design Thickness Using Different Approaches (Section #1)

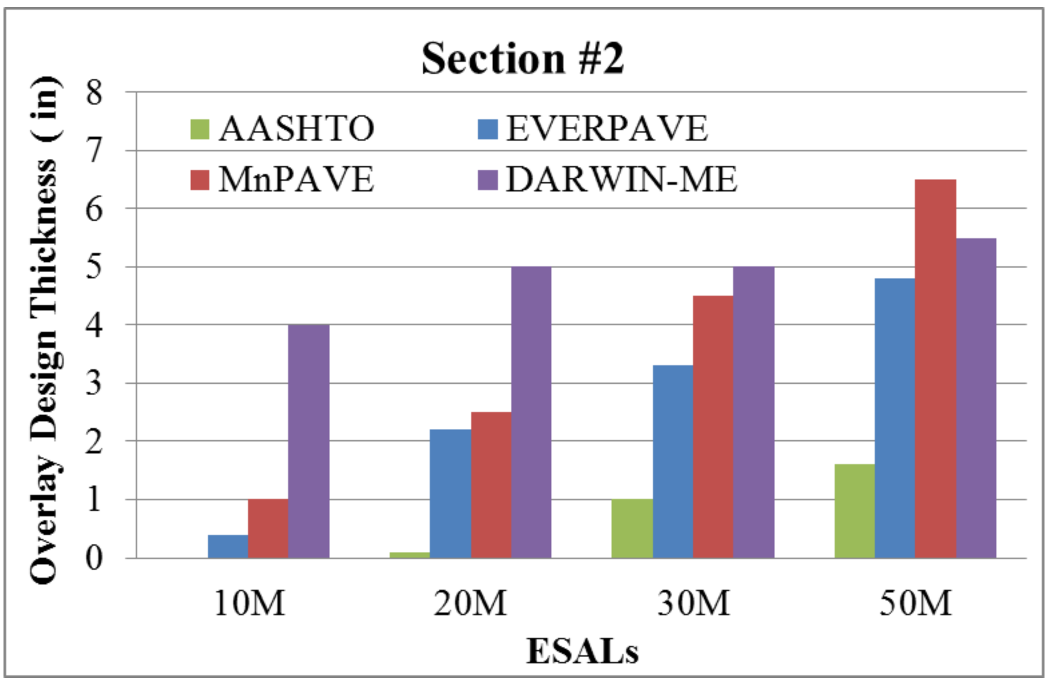


Figure 4.13 Overlay Design Thickness Using Different Approaches (Section #2)

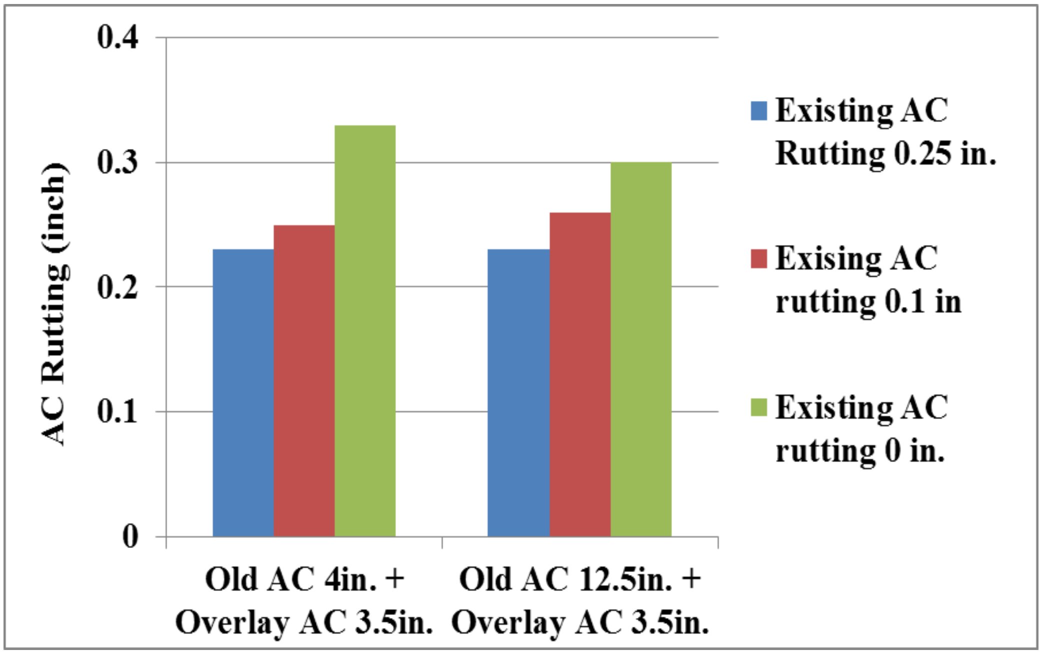


Figure 4.14 Effect of Existing AC Layer Rutting on AC Rutting

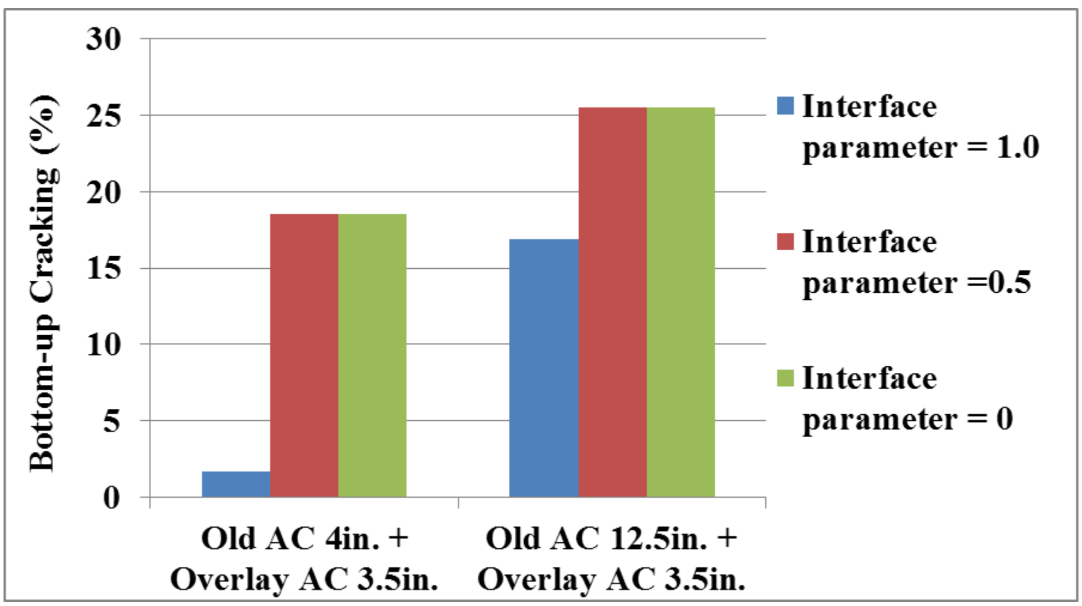


Figure 4.15 Effect of Interface Coefficient on Overlay Fatigue Cracking

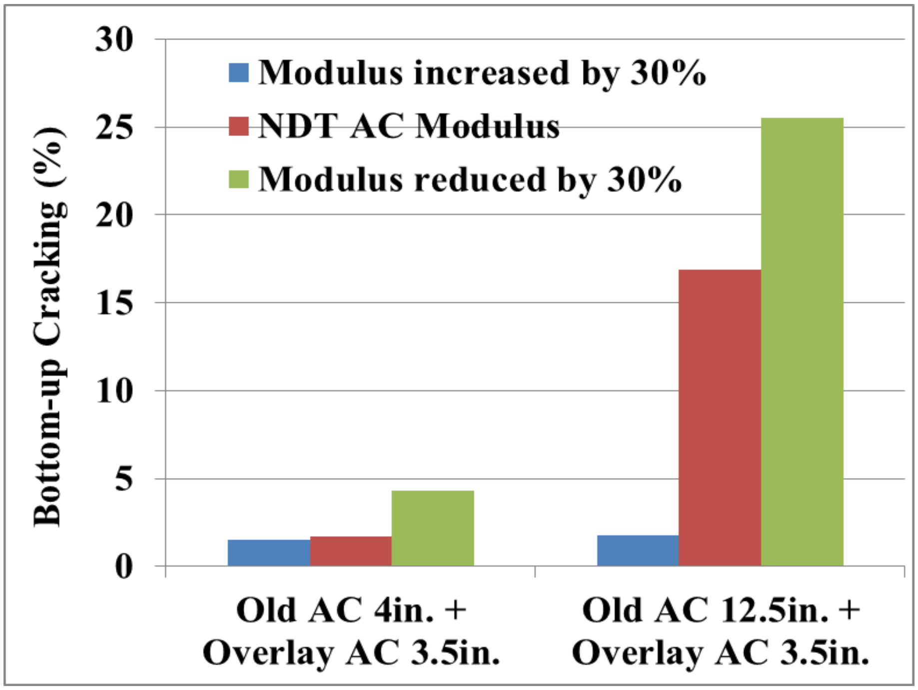


Figure 4.16 Effect of Existing AC Layer Modulus on Overlay Fatigue Cracking

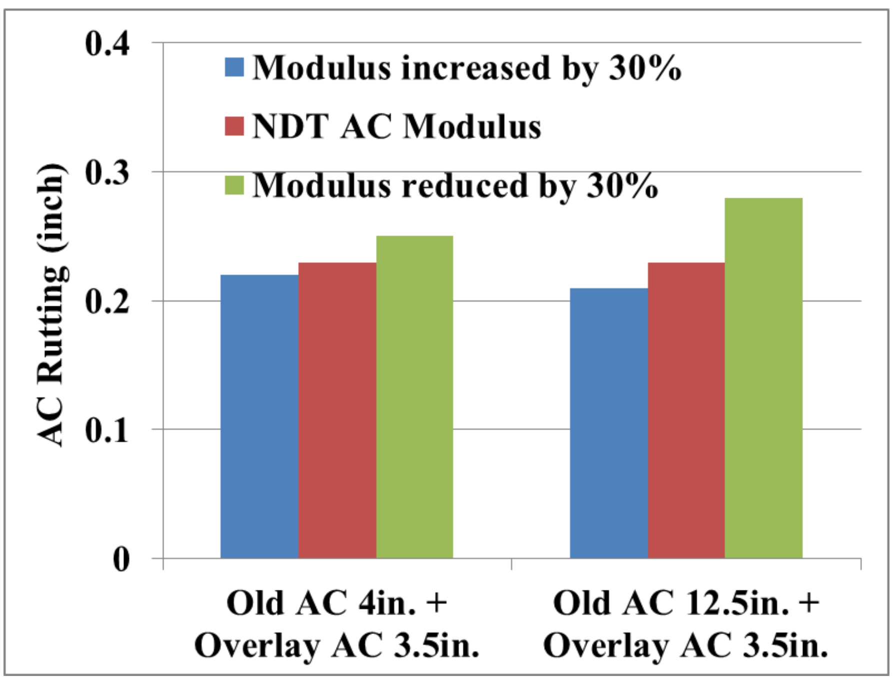


Figure 4.17 Effect of Existing AC Layer Modulus on Overlay Rutting

On Call Testing and Materials Testing Services

Background

The Rutgers Asphalt Pavement Laboratory is a valuable and useful asphalt research laboratory that could assist the NJDOT with some of their technical needs. The PRP agreed to provide timely testing as needed by the NJDOT.

The Pavement Resource Program has developed a number of performance-related specifications for pavement construction materials and houses a number of high speed, non-destructive evaluation tools that can be used to assess the in-situ properties of pavements and bridge decks. In the past, both the NJDOT Bureau of Materials and Pavement and the Drainage Systems and Technology Unit have used the laboratory and field evaluation capabilities of CAIT to provide quality analysis techniques in support of the NJDOT activities.

The PRP staff will respond to 90% of requests within one day and develop an appropriate work plan. Based on requests from NJDOT, PRP staff will provide support for PMS analysis, pavement materials testing, MEPDG and profiler inquires, and NDE field testing. Infrastructure Condition Monitoring Program (ICMP) will respond to NDE field evaluation upon NJDOT request within 3 days.

Work Performed

During the first quarter, the PRP has verified a number of asphalt mixtures for the NJDOT. They included;

- Bottom Rich Intermediate Course (BRIC): Tilcon Keasby, South State, Tilcon Mt. Hope, Trap Rock Industries
- Bridge Deck Water Proof Surface Course (BDWSC): Tilcon Keasby, Tilcon Mt. Hope
- High RAP Mixture: RE Pierson

A paper was prepared and delivered to NJDOT to summarize the factors that contribute frost damage by looking at capillary rise, frost penetration, and frost susceptible soils. The second part identifies the locations of frost susceptible soils and weak subgrade soils in NJ and the third part provides some solutions or treatments for frost susceptible soils and weak subgrades. (Appendix E)

During the second quarter, the PRP has verified, and is also in the process of verifying a number of asphalt mixtures for the NJDOT. This includes;

- Bottom Rich Intermediate Course (BRIC): Tilcon Mt Hope, Stavola
- Bridge Deck Water Proof Surface Course (BDWSC): Tilcon Keasby, Stavola
- High Performance Thin Overlay: Barrett Asphalt

During the third quarter, the PRP has verified, and is also in the process of verifying a number of asphalt mixtures for the NJDOT. This includes;

- Bridge Deck Water Proof Surface Course (BDWSC): Tilcon Mt. Hope, Stone Industries, and Stavola Tinton Falls

A report was completed and provided to the NJDOT regarding the use of fiber-modified asphalt mixtures. Two plant produced asphalt mixtures were produced by Trap Rock Industries; a 12.5M64 with Forta Fi fibers and a 12.5M64 with no fibers. Stiffness, permanent deformation and fatigue cracking of the mixtures were assessed. The testing showed that although the stiffness and permanent deformation properties were similar, the asphalt mixture with no fibers achieved better fatigue resistance. This was found in both the crack initiation (Flexural Beam Fatigue) and crack propagation (Overlay Tester) mode of testing, as well as both short and long-term aged. The report is attached. (Appendix F)

During the fourth quarter, The PSP has verified, and is also in the process of verifying a number of asphalt mixtures for the NJDOT. This includes:

- Bridge Deck Water Proof Surface Course (BDWSC): National Paving, South State, and Stavola Tinton Falls

A technical brief was provided to NJDOT Bureau of Materials that summarizes the influence of compacted air voids and the rutting and cracking performance of BRIC mixtures. The final report illustrated that as air voids increases, the rutting and cracking resistance of the mixtures decreases. (Appendix G)

CAIT is also working with NJDOT on evaluating the premature failure of an HPTO mixture placed on Rt 322. CAIT met with NJDOT and AID to discuss the coring and testing plan. Cores were procured from AID on 5/22 and initial testing has begun. Upon initial review, it appears that 50% of the cores showed that the HPTO lift was not bonded with the underlying lift.

CONCLUSION

The Pavement Resource Program at the Center for Advanced Infrastructure and Transportation at Rutgers University was pleased to participate as an extension and partner with the New Jersey Department of Transportation to perform a variety of tasks put before them.

APPENDICES (A-G)

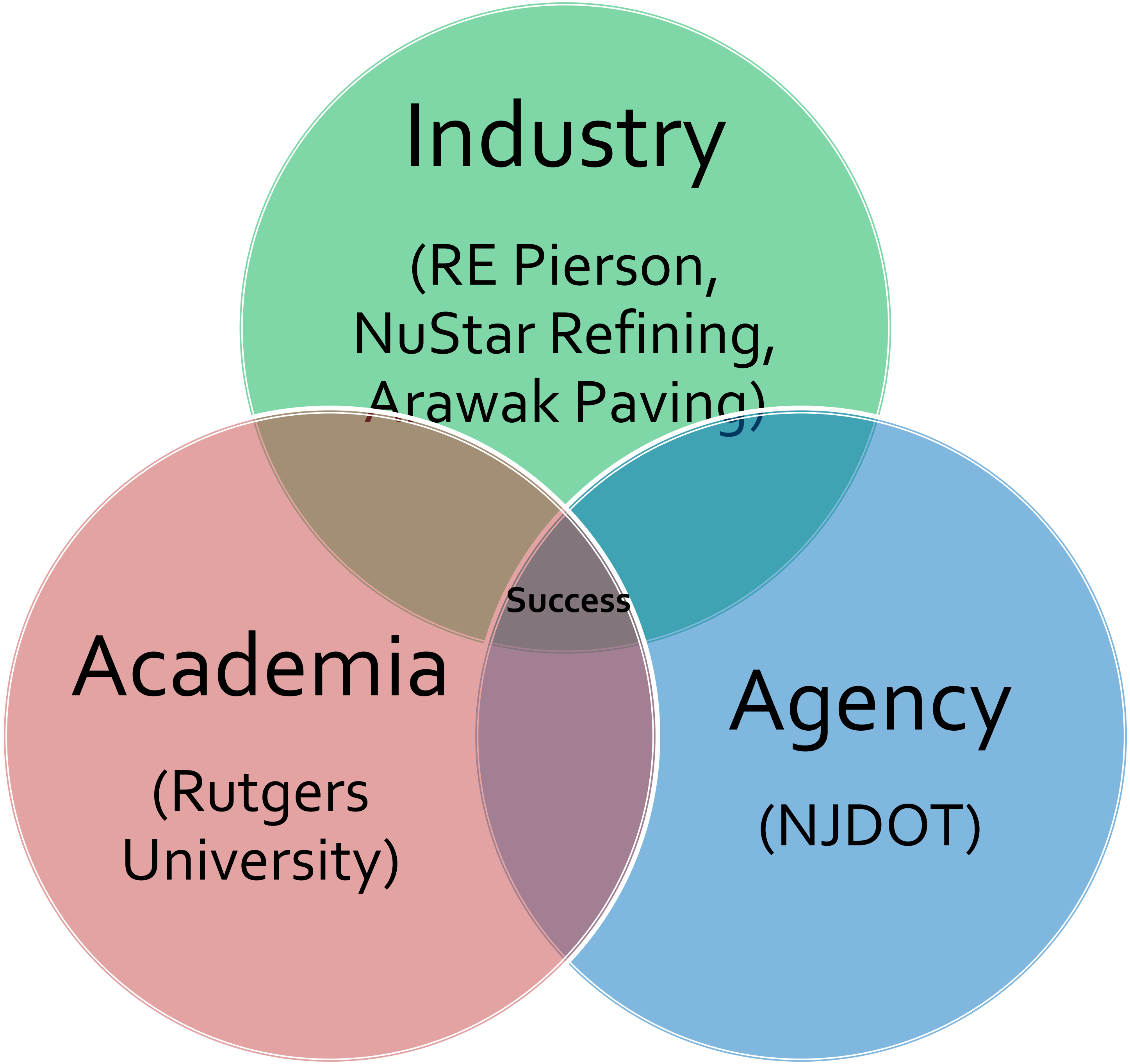
NJDOT High RAP Specification and Implementation – I295

Northeast Asphalt User Producer Group (NEAUPG)
October 24th & 25th, 2012
Philadelphia, PA

Thomas Bennert, Ph.D.
Rutgers University

Acknowledgements

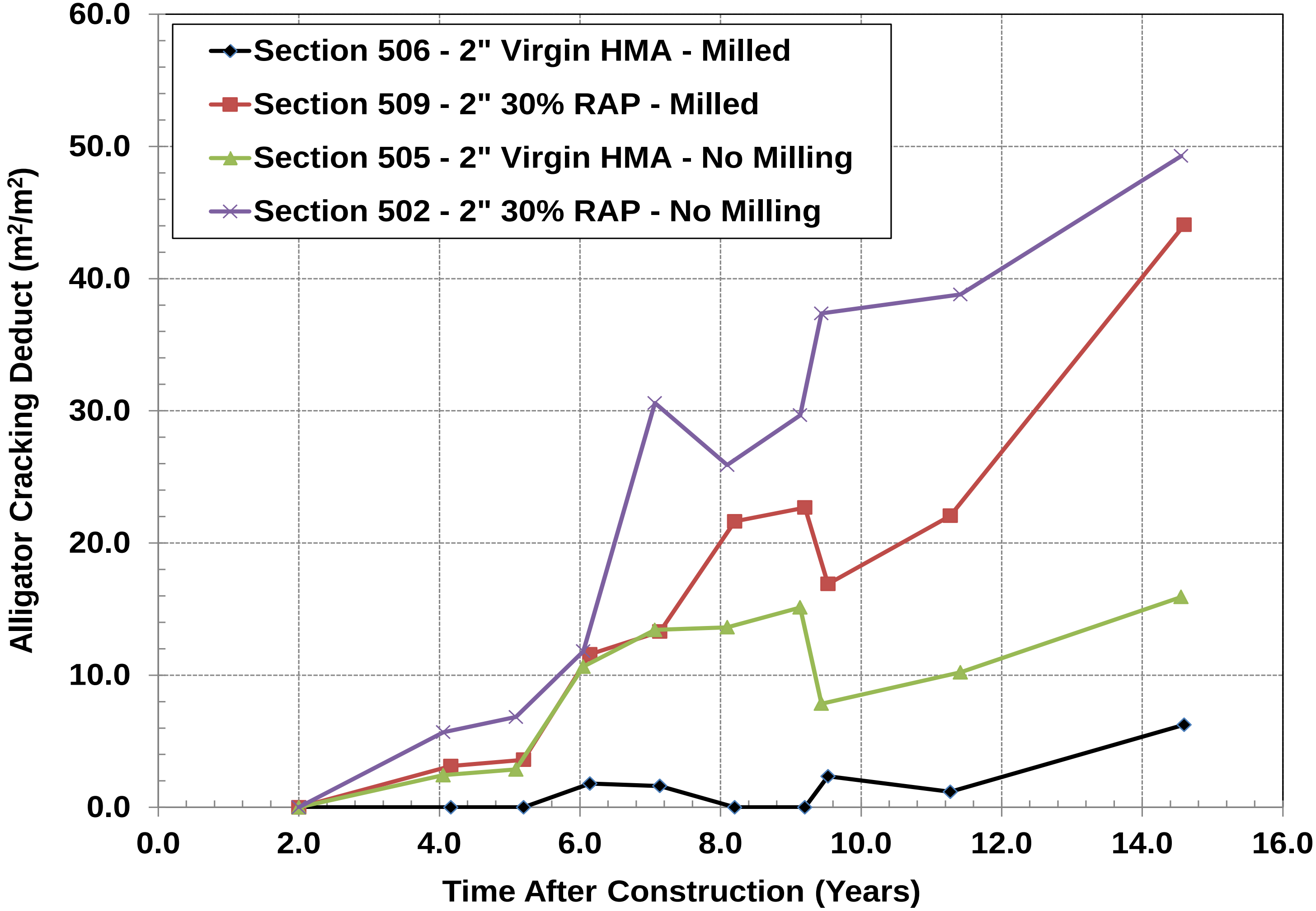
- Dan Karcher – R.E. Pierson
- Eileen Sheehy, Robert Blight, Don Matlock - NJDOT
- Frank Fee and Karissa Mooney – NuStar Asphalt


Industry
(RE Pierson,
NuStar Refining,
Arawak Paving)
Success
Academia
(Rutgers
University)
Agency
(NJDOT)

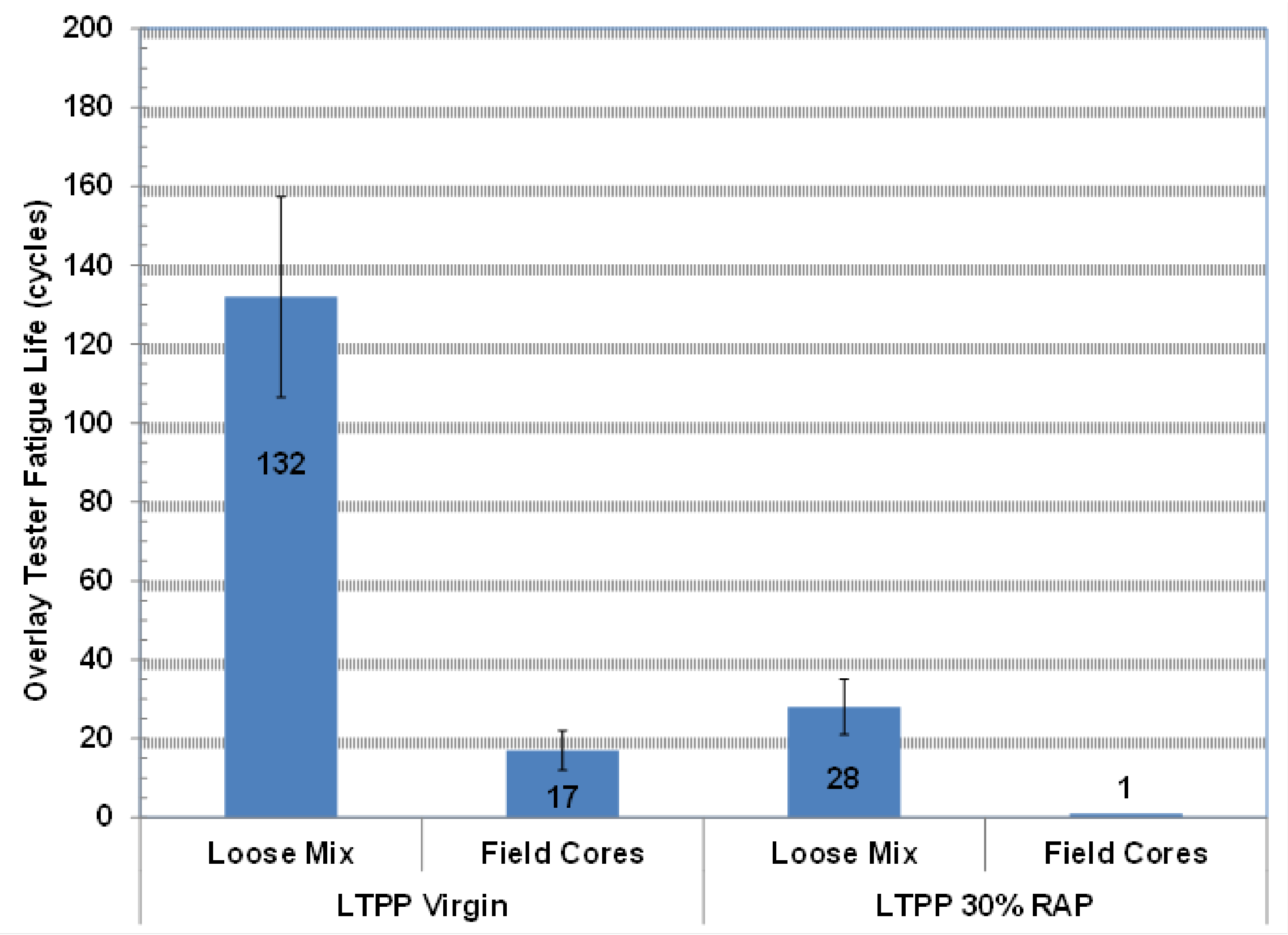
Background

- In 2008, NJDOT began evaluating higher RAP mixtures
  - Under the classification of "research pilot studies"
- Some immediate issues were brought up
  - Proper AC determination of RAP
  - Ignition oven correction factors
  - Need of softer binder to maintain -22°C low temp?
    - Were blending charts right way? Extraction/recovery?
  - Mixture tests indicated higher RAP had fatigue issues – especially Overlay Tester (crack propagation)

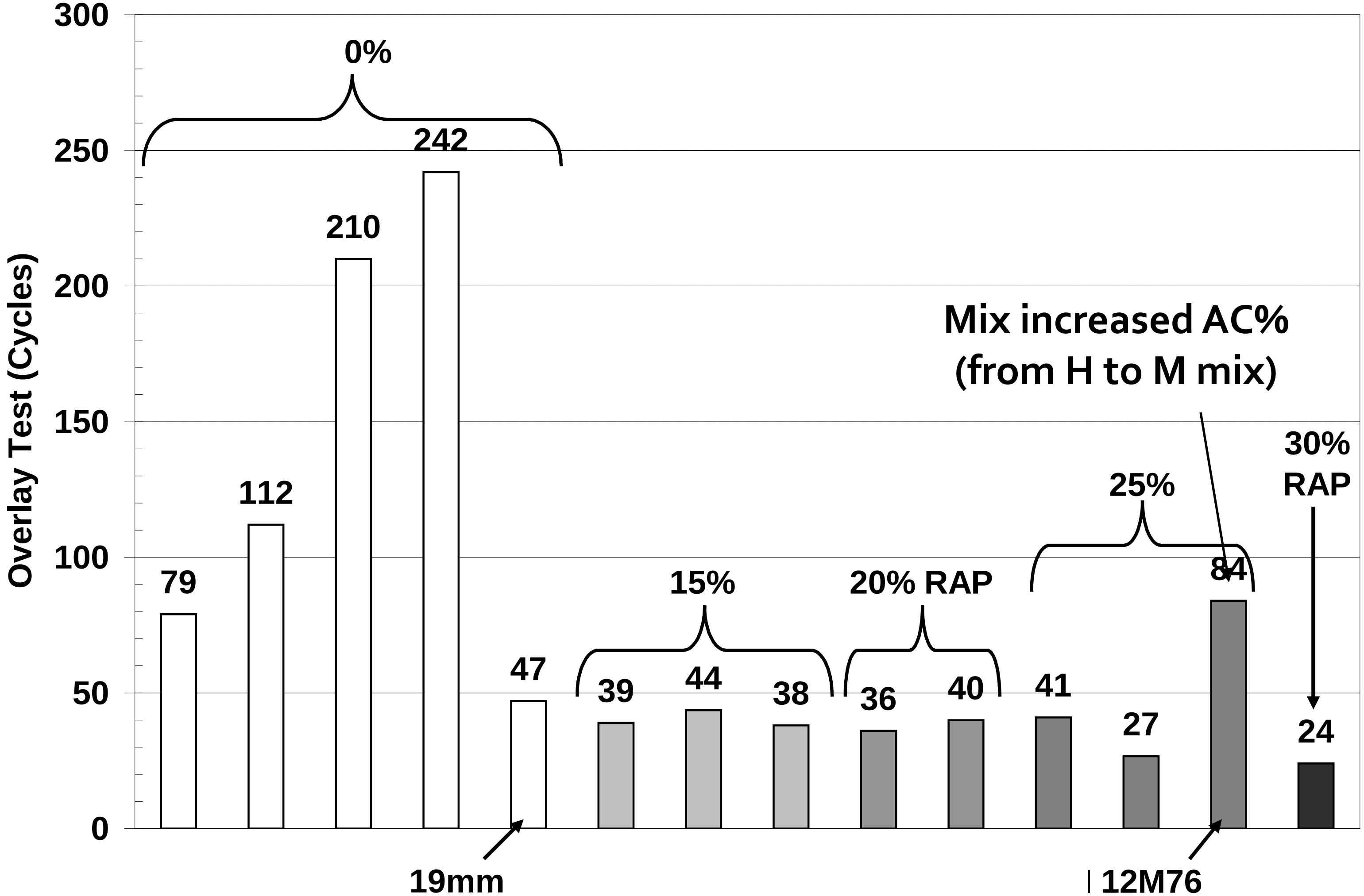
Initiation vs Propagation – NJ SPS-5



Initiation vs Propagation – Overlay Tester for NJ SPS-5



Overlay Tester – 2008 to 2009



Average Results for Overlay Tester (2008 to 2009)

- 0% RAP = 138 cycles
- 15% RAP = 40 cycles
- 20% RAP = 38 cycles
- 25% RAP = 40 cycles
- 30% RAP = 24 cycles (only 1 mix – 19mm)

2010 NJDOT Higher RAP Projects (25% RAP Surface Course Mixes)

- Rt 206 – production and construction data met specifications
  - Holding water in 2011 – Maintenance 2012
- I-80 – issues with volumetrics throughout first half of project
- I-78 – compaction issues resulted in high in-place air voids and poor ride
- South Jersey Maintenance Roadway Repair Contract (#1)
  - Could not get mix verified through plant
- South Jersey Maintenance Roadway Repair Contract (#2)
  - Only project not to report issues

Back to the Drawing Board!

- In 2011, NJDOT held NJ asphalt industry to current specifications
  - 15% RAP in surface; 25% RAP in intermediate/base
- In winter 2012, Rutgers and NJDOT worked to develop a Performance-Based High RAP (HRAP) specification
  - Utilized database of performance testing results to establish performance requirements for both rutting (Asphalt Pavement Analyzer) and cracking (Overlay Tester)

NJDOT HRAP – Basic Principle

- The supplier is not held to PG grade, max. RAP content, aggregate angularity, etc.
  - Have to meet basic Superpave requirements
  - NJDOT increased VMA 1% over current specs
  - Could use softer binder, rejuvenators, WMA
- However, acceptance based on final mixture performance, based on database of typical "virgin" HMA

NJDOT HRAP

- Minimum of 20% RAP in Surface Course
- Minimum of 30% RAP in Intermediate/Base
- Lab design and plant produced material must meet rutting (APA) and cracking (Overlay Tester) requirements

Table 902.11.03-2 Performance Testing Requirements for HMA HIGH RAP Design				
	Requirement			
	Surface Course		Intermediate Course	
Test	PG 64-22	PG 76-22	PG 64-22	PG 76-22
APA @ 8,000 loading cycles (AASHTO T 340)	< 7 mm	< 4 mm	< 7 mm	< 4 mm
Overlay Tester (NJDOT B-10)	> 150 cycles	> 175 cycles	> 100 cycles	> 125 cycles

NJDOT HRAP – I295

- I295 SB – Milepost 11.26 to 14.48
- Contractor
  - Arawak Paving
- Supplier
  - R.E. Pierson
- Asphalt liquid
  - NuStar Refining

R.E. Pierson – Mix Design Prep

- Fractionated RAP

Sample No.			Fine RAP		Coarse RAP	
Sieve Size			% Passing	% Passing	% Passing	% Passing
inch	mm		#1	#2	#3	#4
50.0	2	%	100	100	100	100
37.5	1 1/2	%	100	100	100	100
25.0	1	%	100	100	100	100
19.0	3/4	%	100	100	100	100
12.5	1/2	%	100	100	100	99.3
9.5	3/8	%	100	100	94.7	94.9
4.75	No. 4	%	94.7	95.3	40.5	44
2.36	No. 8	%	72.7	74.7	25.1	27.8
1.18	No. 16	%	58.7	59.3	22.3	24.2
0.600	No. 30	%	44.6	45.9	18.7	20.8
0.300	No. 50	%	25.8	26.3	12.6	13.6
0.150	No. 100	%				
0.075	No. 200	%	9.70	9.20	5.40	5.40
Asphalt		%	6.93	7.08	3.40	3.90

83.8-18.8 (29.1)
PG82-18

R.E. Pierson – Mix Design Prep

- R.E. Pierson contracted NuStar Refining for binder.
  - Reminder – no PG grade specified
  - NuStar required to formulate binder specifically to help meet performance requirements
- R.E. Pierson designed and submitted over 5 different variations (each) of mixtures for the 9.5M76 and 12.5M64 HRAP mixtures required for the project.

Final HRAP Mix Designs

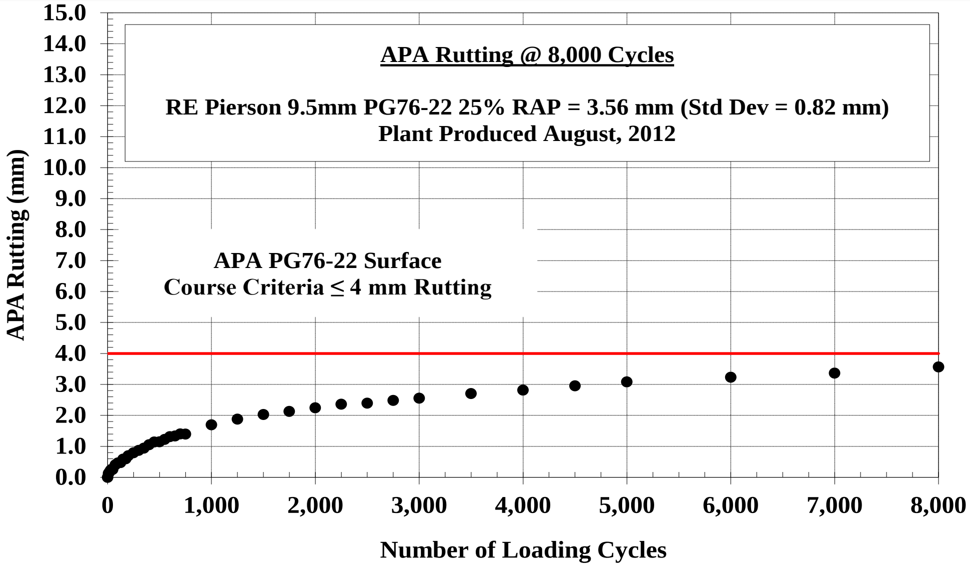
9.5M76 (SURFACE COURSE)

- 25% RAP
- 6.0% Total AC
  - 27.4% Binder Replacement
- PG70-22 (74.6-26.99)
- 25% Fine RAP Fraction Only

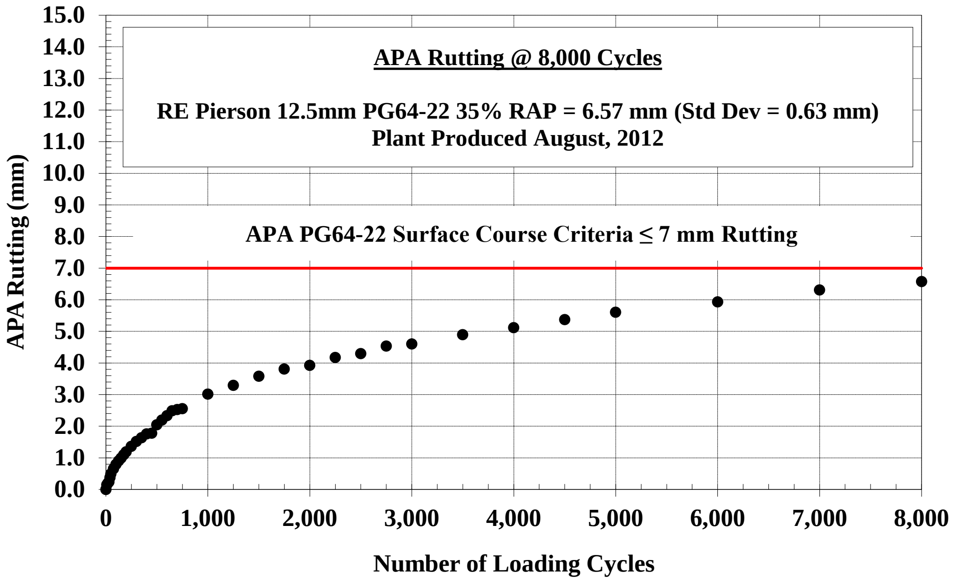
12.5M64 (INTERMED. COURSE)

- 35% RAP
- 5.8% Total AC
  - 29.7% Binder Replacement
- PG64-28 (64.8-28.29)
- 17.5% Fine RAP/ 17.5% Coarse RAP

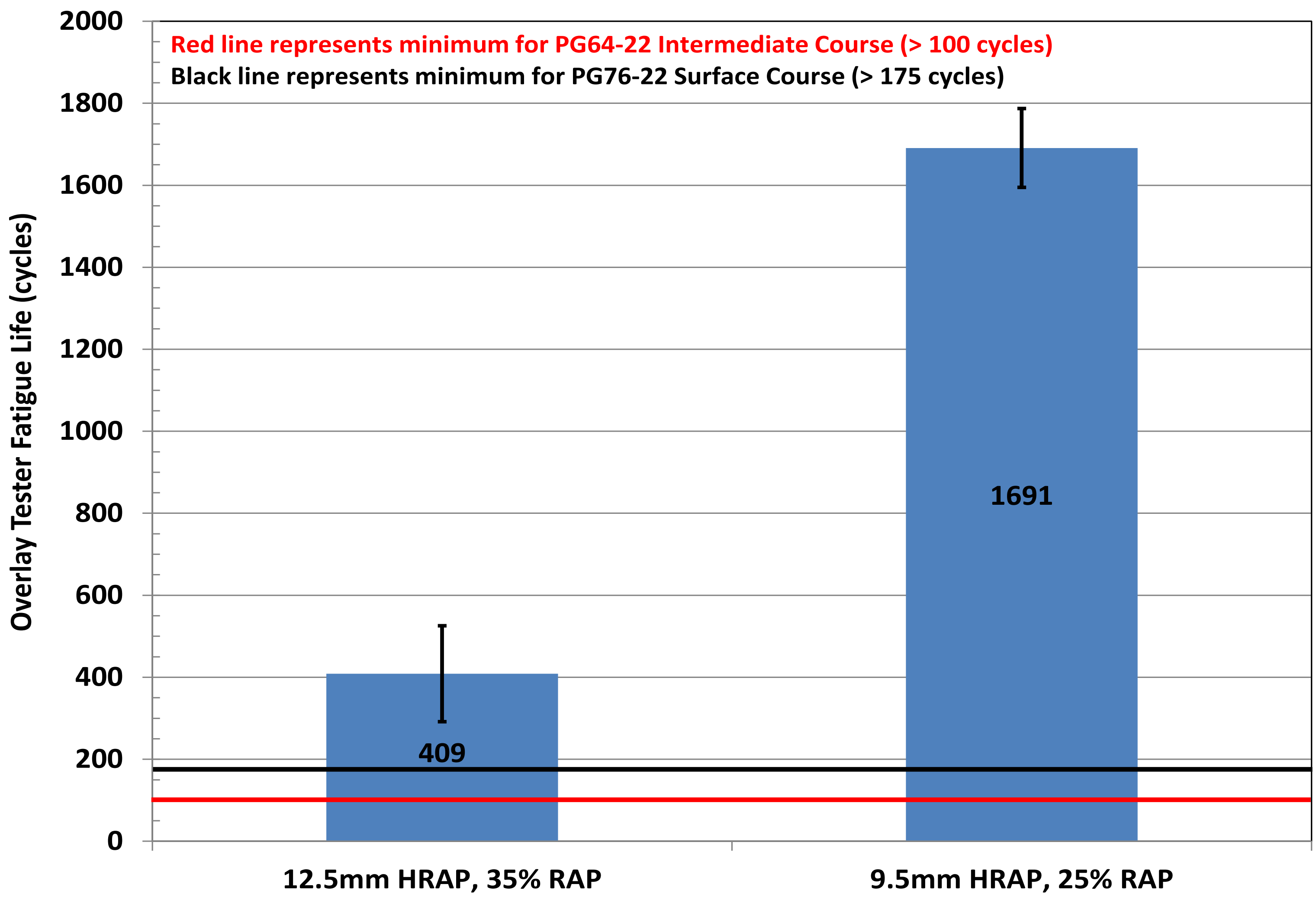
APA Rutting Performance



64°C Test Temp.; 100psi Hose Pressure; 100 lb Load Load



Overlay Tester



Final Product

Final Product

Densities

- For plant production, NJDOT allowed lower air voids in gyratories than “normal” HMA
  - 95% to 98.5% of Gmm

- 9.5M76 HRAP Cores
  - Lot #1: Average = 7.4% air voids
  - Lot #2: Average = 5.9% air voids
- 12.5M64 HRAP Cores
  - Lot #1: Average = 4.6% air voids (Full bonus)
  - Lot #2: Average = 5.7% air voids (Full bonus)
  - Lot #3: Average = 6.5% air voids

IRI

- 9.5M76 WMA
  - 11.54 – 11.26: Average = 57.8 in/mile
  - 13.93 – 11.54: Average = 37.7 in/mile
  - 14.39 – 13.93: Average = 76.9 in/mile

Ave = 57.5 in/mile

- 9.5M76 HRAP
  - 14.39 – 13.93: Average = 57.8 in/mile
  - 13.93 – 11.54: Average = 44.0 in/mile
  - 11.54 – 11.26: Average = 60.8 in/mile

Ave = 54.2 in/mile

In Summary

- NJDOT took a different approach to higher RAP mixtures
  - Put ownership on contractor/supplier to use as much RAP as possible, but need to meet mixture performance
- Collaboration between Industry, Academia, and Agency resulted in a successful project
  - Field monitoring will continue to evaluate performance

Thank you for your time!
Questions?

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Fatigue Cracking Analysis of New Jersey's LTPP SPS-5 Sections: 30% RAP vs Virgin Hot Mix Asphalt (HMA)

Submitted to:

New Jersey Department of Transportation (NJDOT)
Bureau of Materials

Conducted by:

Thomas Bennert, Ph.D.
The Rutgers Asphalt/Pavement Laboratory (RAPL)
Center for Advanced Infrastructure and Transportation (CAIT)
Rutgers University
Department of Civil and Environmental Engineering
623 Bowser Road
Piscataway, NJ 08854

ABSTRACT

In 2010, New Jersey's Long Term Pavement Performance (LTPP) SPS-5 sections closed out. Prior to the rehabilitation of these pavement sections, an extensive coring and forensic study was conducted to characterize the material properties of the Virgin and 30% RAP asphalt mixtures utilized on the project. Along with field cores, raw materials (i.e. – aggregates, binder, loose mix) was procured from FHWA-LTPP Materials Reference Library. Visual distress surveys from the LTPP database were collected and utilized to compare the mixture performance to the general field performance.

Overall, the field performance indicated that both the virgin and 30% RAP sections initiated cracking within 1 to 3 years of each other, depending on the section evaluated. However, once cracking had been initiated, the 30% RAP sections cracked at a faster rate than the Virgin sections resulting in higher crack counts, even though the 30% RAP section was using a softer binder than the virgin section (i.e. – AC-10 vs AC-20). The Overlay Tester, Disk Shaped Compact Tension (DC(T)), and Low Temperature IDT and Creep Compliance were used to characterize intermediate and low temperature cracking properties of the mixtures. Asphalt binder characterization included PG grading, master stiffness curves, and Linear Amplitude Sweep (LAS) testing to characterize the stiffness and fatigue properties of the asphalt binders. The material testing program showed that the mixture test results matched the observed field cracking performance better than the asphalt binder testing conducted on the extracted and recovered asphalt binders. The Overlay Tester and DC(T) tests appeared to be the most sensitive to the cracking performance differences between the Virgin and 30% RAP mixtures, while the LAS test appeared to rank the fatigue performance of the 30% RAP mixture better than the Virgin mixture, which contradicted the observed field performance.

INTRODUCTION

The Long Term Pavement Performance (LTPP) program started in 1987 as part of the Strategic Highway Research Program (SHRP), a 5-year applied research program funded by the 50 state agencies and managed by the Transportation Research Board (TRB). The main mission of the LTPP was to [1];

- Collect and store performance data from a large number of in-service highways in the United States and Canada over an extended period of support analysis and product development;
- Analyze these data to describe how pavements perform and explain why they perform as they do; and
- Translate these insights into knowledge and usable engineering products related to pavement design, construction, rehabilitation, maintenance, preservation, and management.

The various test sections in the LTPP program were nominated by State and Provincial highway agencies. Each of the sections was classified as being in either the General Pavement Study (GPS) or Specific Pavement Study (SPS). GPS sections were usually selected from in-service pavements designed and built according to good engineering practices by the DOT's, while the SPS sections were designed and constructed to answer specific research question [1].

Of particular interest to researchers evaluating the effect of RAP on asphalt mixture performance is the SPS-5 experiment. The SPS-5 experiment, *Study of Rehabilitation of Asphalt Concrete Pavements*, is made up of 17 projects within the United States. Each project has nine test sections consisting of a control section, where no rehabilitation was applied to the surface, and eight test sections with different combinations of the following strategies [2];

- Thin (2 Inch) and thick (5 Inch) overlays;
- Virgin and RAP mixtures used for the overlays; and
- Milled and unmilled surfaces prior to overlay placement.

The pavement distresses measured on the SPS-5 sections were fatigue cracking, longitudinal cracking in wheelpath, longitudinal cracking not in the wheelpath, transverse cracks, rutting, and roughness (IRI). Table 1 shows a summary of the results from the SPS-5 data from a LTPP Technical Brief in 2000 [2]. It should be noted that the summarized performance data shown in Table 1 only represented 5 to 10 years of service life at the time of the study [2].

In 2009, West et al. [3] presented data on the LTPP SPS-5 sections regarding the performance comparisons between the Virgin and 30% RAP mixture sections. The West et al. [3] data included an additional nine years of pavement performance data over the previous LTPP Technical Brief shown earlier. West et al. [3] concluded that;

- The location of the projects (i.e. – state or province) and the age of the pavement had a great impact on all distresses evaluated;
- Overlay thickness was also significant on pavement distress with the exception of longitudinal cracking and raveling;
- Milling prior to rehabilitation significantly decreased IRI, fatigue cracking, and transverse cracking but unfortunately increased rut depths;
- Milling did not have a significant impact on longitudinal cracking, block cracking, or raveling; and

Table 1 – Summary of SPS-5 Test Section Performance from 2000 [2]

Distress Type	Factor		
	Overlay Thickness Increasing	**Milling Surface**	**Recycled Mix**
Fatigue Cracking	Less cracking	Less cracking	No advantage over Virgin
Longitudinal Cracking in Wheelpaths	No advantage	No advantage	More cracking
Transverse Cracking	Less cracking	Less cracking	No advantage over Virgin
Longitudinal Cracking not in Wheelpaths	No advantage	No advantage	Less cracking
Rutting	No advantage	No advantage	No advantage over Virgin
Roughness	No advantage	No advantage	No advantage over Virgin

- According to the ANOVA analysis conducted by West et al. [3], mix type (Virgin vs RAP mixtures) was only significant for fatigue cracking, longitudinal cracking, and transverse cracking with most Virgin sections slightly out-performing the 30% RAP sections.

New Jersey's LTPP SPS-5 30% RAP sections were one of the test sections that showed an increased amount of fatigue/longitudinal cracking when compared to the Virgin sections. However, contrary to what would be expected based on the fatigue/longitudinal cracking results, the Virgin sections resulted in greater transverse cracking than the 30% RAP sections. West et al., [3] attempted to look at possible differences in mixture properties, such as asphalt binder content and fines content to help explain the difference in fatigue performance in the New Jersey sections, but no significant differences were found. It should also be noted that the 30% RAP mixture in New Jersey utilized a softer asphalt binder than the Virgin mixture – AC-10 vs AC-20, respectively.

Encouraged by the statistically analysis conducted by West et al. [3], as well as the fact that New Jersey's SPS-5 sections was scheduled for close out and an extensive structural rehabilitation in the summer of 2010, a research effort was undertaken to evaluate the mixture and asphalt binder properties of New Jersey's SPS-5 Virgin and 30% RAP sections. In particular, the forensic evaluation was focused on evaluating the cracking performance recorded in the field and the asphalt properties measured in the laboratory. By comparing the field and laboratory data, it is hopeful that general conclusions could be drawn that relates laboratory performance to field cracking performance, and possibly explain why New Jersey's Virgin mixture did not perform as well as the 30% RAP mixture.

OBJECTIVES

The objective of the research study was to conduct a forensic analysis of New Jersey's LTPP SPS-5 test sections to determine why significant differences were found between the Fatigue Cracking and Transverse Cracking performance of the 30% RAP and Virgin mixture sections. To help determine and possibly explain the performance differences, a battery of asphalt binder and asphalt mixture performance testing was conducted. The results of the research study will hopefully shed light on why observed cracking in the Virgin mixture sections were larger than those observed for the 30% RAP sections.

NEW JERSEY'S LTPP SPS-5 SECTIONS

New Jersey's SPS-5 sections were located on the westbound lane of Interstate 195 from East of Old York Road (station 533+30) to just East of Imlaystown-Hightstown Road (station 690+00), a distance of about 3 miles. It is a four lane divided highway with 2 lanes in each direction. The highway consisted of two 12 foot lanes with a 3 foot inside should and a 12 foot outside shoulder. The pavement structure, prior to the SPS-5 rehabilitation, consisted of 9 inches of HMA over 5 inches of granular base aggregate and 5 inches of a pit run gravel granular subbase, all overlaying a silty sand subgrade soil. Subdrains were already in-place prior to the rehabilitation, but additional subdrains were installed during the SPS-5 construction to help improve subgrade drainage. The layout plan called for the 5-Inch test sections to be placed at the east end of the project, while the 2-Inch Overlay sections were placed in the center of the project.

Cores taken during this research effort indicated that the test sections of comparable overlay thickness and paving surface type had very comparable asphalt thickness'. With the assumption that the base/subbase/subgrade materials were of identical thickness and quality, the asphalt layer thickness would be the only structural difference in the pavement structure that could influence the pavement performance. The measured asphalt layer thickness for the extracted cores (average of 8 per section) was determined as follows:

- Milled Paving Surface
  - 2-Inch Overlay: 30% RAP (Section 509) = 10.5 inches; Virgin (Section 506) = 10.8 inches
  - 5-Inch Overlay: 30% RAP (Section 508) = 13.9 inches; Virgin (Section 507) = 13.5 inches
- Unmilled Paving Surface
  - 2-Inch Overlay: 30% RAP (Section 502) = 10.4 inches; Virgin (Section 505) = 10.2 inches
  - 5-Inch Overlay: 30% RAP (Section 503) = 13.7 inches; Virgin (Section 504) = 12.6 inches

On average, the 30% RAP sections had a slightly larger asphalt layer thickness than the Virgin mixtures, except for the milled paving surface with the 2-Inch thick overlay where a 0.3 inch difference was measured.

At the time of the SPS-5 construction (1994), I-195 carried approximately 27,040 vehicles per day (2-way) with 3.75% heavy trucks for average 2-way annual daily truck traffic of 1,014 trucks. By the end of the service life of NJ's SPS-5 test sections, the AADTT had climbed to over 1,530 trucks.

FIELD OBSERVATIONS

Upon request, pavement distress measurements were provided by the LTPP Northeast contractor (Stantec Consulting) for the four different LTPP SPS-5 pavement structure types described earlier. The cracking distress values were provided in terms of low, moderate, and high severity levels, as per the protocol of the Long Term Pavement Performance (LTPP) program [4]. For this study, all three severity levels for each cracking distress type was combined through the use of deduct curves originally developed for the South Dakota Department of Transportation [5]. The benefit of using the deduct curves was to incorporate all levels of measured cracking. The equations used are shown in Equations 1 thru 4.

$$D_L = 3.4082 * P_L^{0.514} \tag{1}$$

$$D_M = 4.4575 * P_M^{0.6107} \tag{2}$$

$$D_H = 5.2064 * P_H^{0.6956} \tag{3}$$

$$D_T = D_L + D_M + D_H \tag{4}$$

Where,

D_L = low severity deduct value
D_M = moderate severity deduct value
D_H = high severity deduct value
D_T = total deduct value
P_L = recorded low severity distress
P_M = recorded moderate severity distress
P_H = recorded high severity distress

Utilizing the methodology described above, Figures 1, 2, and 3 were generated for the LTPP SPS-5 pavement sections in New Jersey. Figure 1 shows the Longitudinal Wheel Path Cracking and Alligator Cracking for both the 2-Inch and 5-Inch Overlay sections. Figure 2 shows the Transverse Cracking and Block Cracking, while Figure 3 shows the Non-Wheel Path Longitudinal Cracking for the 2-Inch and 5-Inch Overlay sections.

Longitudinal Wheelpath and Alligator Cracking Observations

Figure 1 contains the Longitudinal Wheelpath and Alligator Cracking distress observations recorded during the life of the NJ SPS-5 sections. In the 2-Inch Overlay Sections (Figure 1a), the onset of Wheelpath Longitudinal Cracking (WLC) appeared to begin at year 5 for both 30% RAP sections (milled and unmilled paving surfaces). Meanwhile, measurable WLC was not observed in the non-milled Virgin section until year 6, with no observable WLC in the milled 2-Inch Virgin section. For the 5-Inch Overlay sections (Figure 1c), WLC was only observed in the 30% RAP sections starting at approximately the 6 year mark. In all sections, “measurable” WLC stopped occurring and disappeared bafter approximately 10 years.

Alligator Cracking (AC) is basically an accelerated and more severe condition of Wheelpath Longitudinal Cracking (WLC). It is logical that as the WLC progresses with time, it would take the form of Alligator Cracking. In doing so, it would be expected that as the Alligator Cracking observations increased, the Wheelpath Longitudinal Cracking would decrease. This is the general trend observed when comparing the data in Figure 1. Although small magnitudes of Alligator Cracking can be observed in the early life of the pavement sections, it is not until the observed WLC begins to decrease that the Alligator Cracking levels significantly increase in all sections. In the 2-Inch Sections, the 30% RAP sections had a much higher Alligator Cracking level at the end of the pavement life for the milled and unmilled

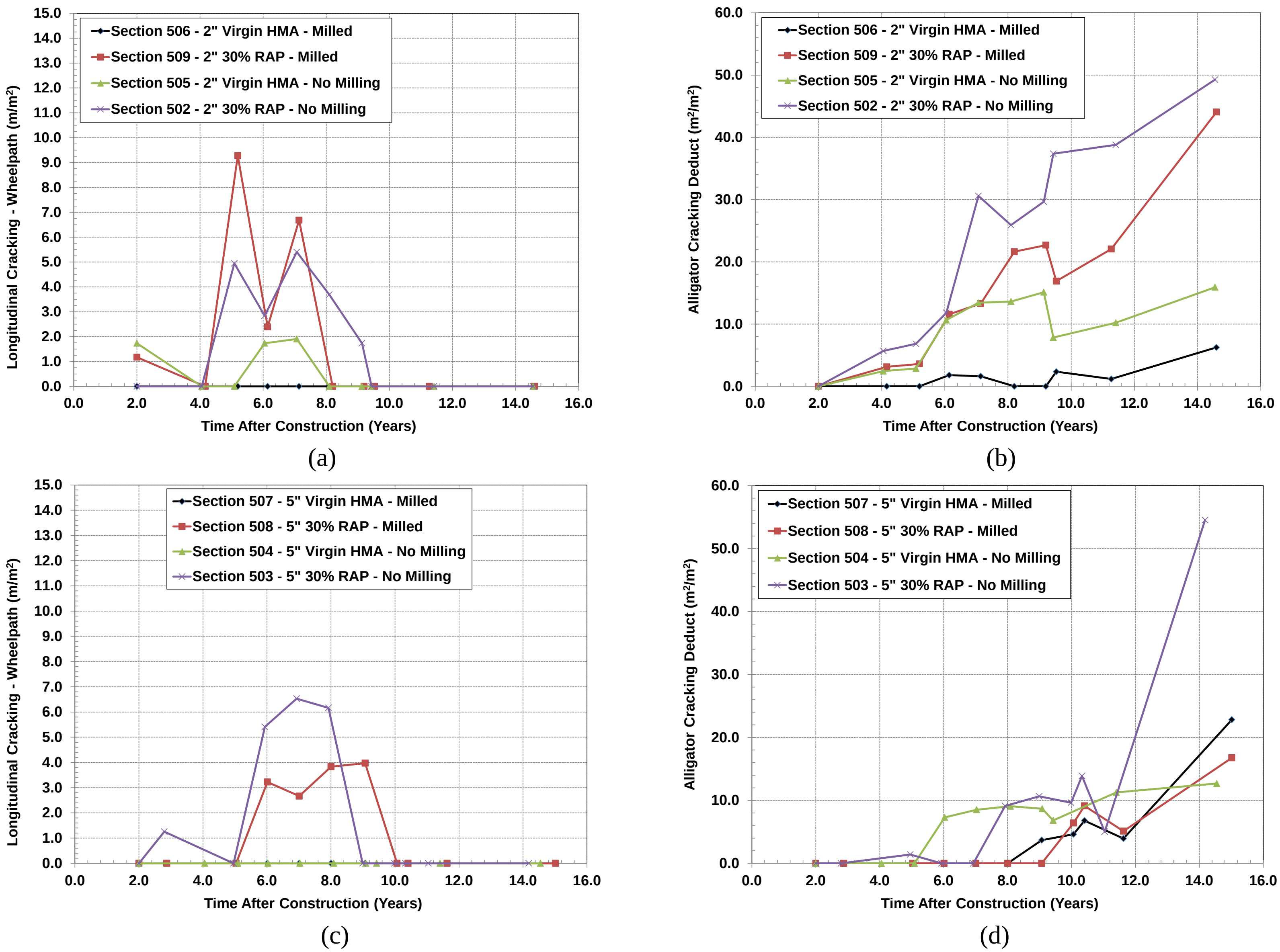


Figure 1 – Wheelpath Longitudinal and Alligator Cracking for NJ's SPS-5 Test Sections

sections, respectively. Meanwhile, in the 5-Inch Overlay Sections, the unmilled paving surface section performed similarly to the 2-Inch Overlay Sections with the 30% RAP sections having a significantly higher level of cracking. However, in the 5-Inch milled paving surface section, there was a reversal in observed performance with the 30% RAP section performing slightly better than the Virgin section.

Transverse Cracking and Block Cracking Observations

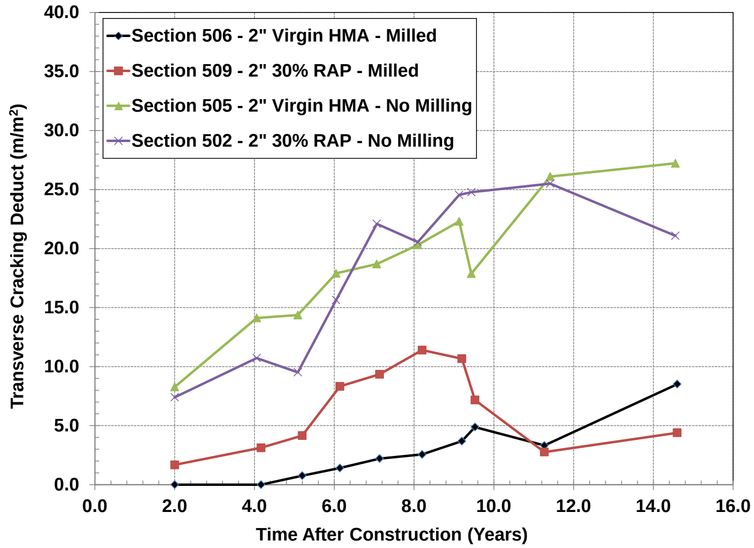
The Transverse Cracking and Block Cracking visual distress measurements for the NJ SPS-5 sections are shown in Figure 2. It should be noted that Block Cracking was not observed in the 5-Inch Overlay sections so the data is not shown in Figure 2. In general, the Transverse Cracking was observed to be greater in the umilled paving surface sections when compared to the milled paving surface. At the end of the pavement service life, the magnitude of Transverse Cracking would indicate that the 30% RAP sections had lower magnitudes of Transverse Cracking than the Virgin mixture sections. This was also observed and reported by West et al. [3]. However, the trends in the curves indicates that after approximately 10 years of service life, the observed Transverse Cracking in the RAP sections begins to decrease, while the Transverse Cracking in the Virgin Mixtures continues to increase (Figure 2a). This may be explained by the observed Block Cracking in the same sections. At the same time the decrease in Transverse Cracking was observed in the 2-Inch Overlay RAP sections, a drastic increase Block Cracking was observed (Figure 2b). Therefore, it is hypothesized that the observed Transverse Cracking in the 2-Inch Overlay 30% RAP sections did not “heal” or simply disappear but migrated into a different form of cracking distress, in this case, the “transverse” component of Block Cracking.

Meanwhile, in the 5-Inch Overlay sections, the Transverse Cracking was generally greater in the 30% RAP sections than the Virgin mixture sections, even though the initiation of Transverse Cracking for the sections all occurred around the same general timeframe. However, unlike the 2-Inch Overlay sections, there is no observed distinct decrease in Transverse Cracking, which may be the reason for the lack of Block Cracking observed in the 5-Inch Overlay sections mentioned earlier.

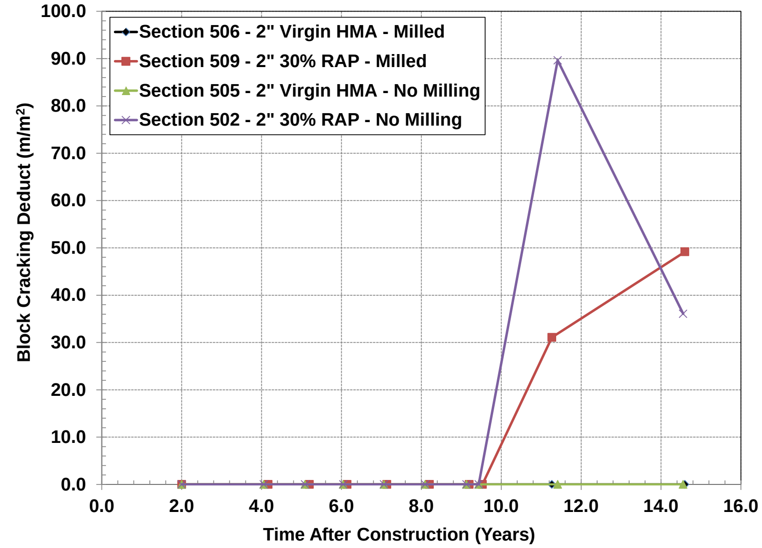
Non-Wheelpath Longitudinal Cracking Observations

The Non-Wheelpath Longitudinal Cracking observations are shown in Figure 3. The 2-Inch Overlay sections (Figure 3a) clearly show that in the first 8 to 9 years of service life, the 30% RAP sections had accumulated a greater magnitude of Non-Wheel Path Longitudinal Cracking (NWPLC) when compared to the Virgin mixtures, almost reaching a maximum Deduct value of 100.0. However, after the 9 year mark, the level of NWPLC dramatically decreases below the levels of the Virgin mixtures; very similar to what was observed in the Transverse Cracking observations. Once again, at the time period where the observed NWPLC cracking significantly decreased, the sharp increase in Block Cracking occurred (Figure 2b). Therefore, it is hypothesized that the “vertical” component of the Block Cracking observed may have actually been from the greater magnitudes of NWPLC cracking.

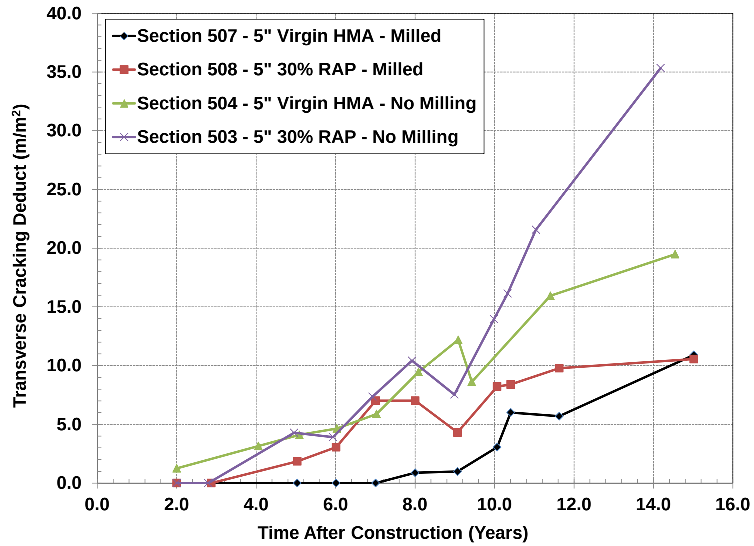
In the 5-Inch Overlay sections, NWPLC cracking is very similar for all test sections (30% RAP and Virgin) and paving surface type (milled and unmilled). All of the 5-Inch Overlay sections appear to achieve a maximum NWPLC level after approximately 8 years and essentially remained there for the remaining 7 years of service life.



(a)



(b)



(c)

Figure 2 – Transverse and Block Cracking for NJ's SPS-5 Test Sections

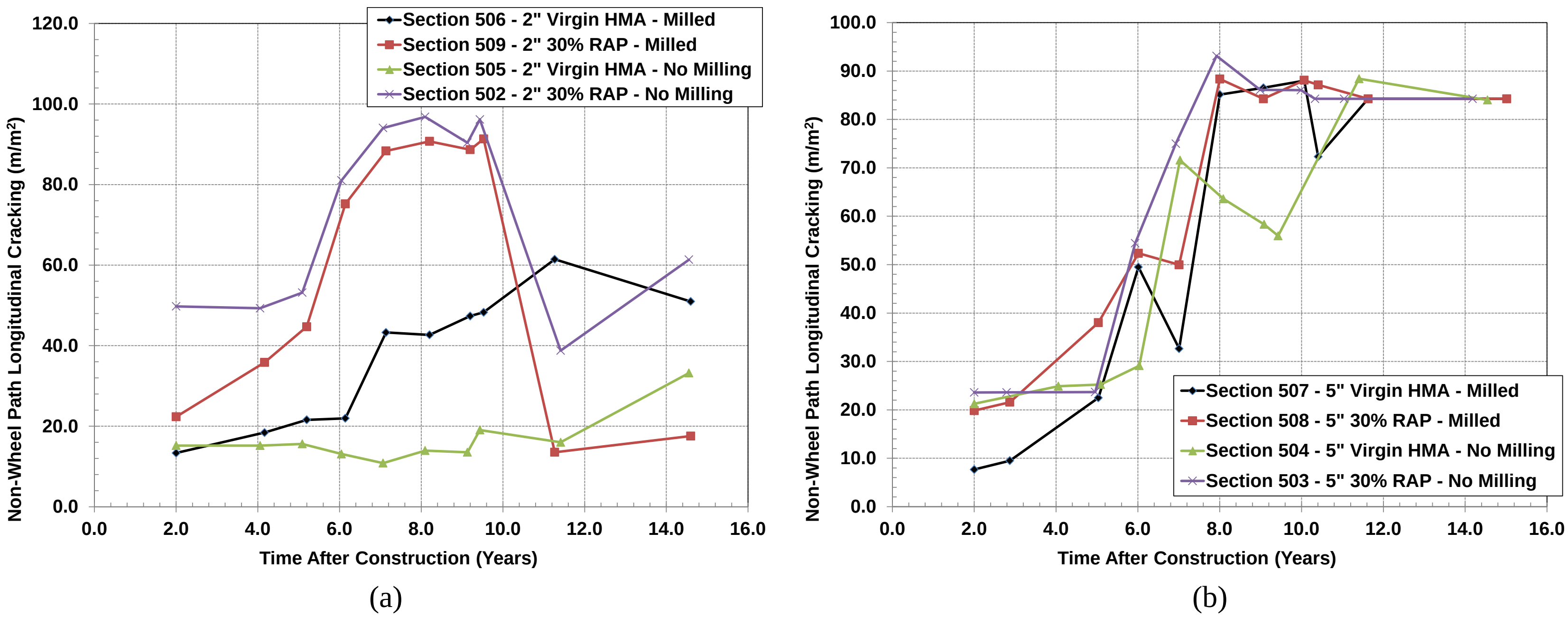


Figure 3 – Non-Wheelpath Longitudinal Cracking for NJ's SPS-5 Test Sections

Overall Field Cracking Observations

Based on the visual cracking observations conducted during the LTPP SPS-5 program, and summarized during this research effort, it is evident that the 30% RAP sections did not perform as well as the Virgin sections. In summary;

- 2-Inch Overlay Sections:
  - Higher levels of Longitudinal Wheelpath Cracking and Alligator Cracking for 30% RAP sections over the Virgin mixture sections within the same paving surface condition (i.e. – milled vs unmilled);
  - Higher levels of Transverse Cracking and Non-Wheelpath Longitudinal Cracking were observed earlier in the pavement service life. However, at approximately 10 years of service life, the observed Transverse Cracking and Non-Wheelpath Longitudinal Cracking drastically decrease, even though no pavement preservation treatments were conducted. At this same time, the observed Block Cracking for both of the 30% RAP sections sharply increased, with no observed Block Cracking in the Virgin mixture. Therefore, it is hypothesized the Transverse and Non-Wheelpath Longitudinal Cracking in the 30% RAP sections reached a critical condition and transformed into Block Cracking. Similar "transformations" can be observed in the Wheelpath Longitudinal Cracking, as this parameter reached a critical level and then disappeared, resulting in rapidly rising levels of Alligator Cracking. This observation of "compensatory differences", where a reduction in one cracking distress generally coincides with an increase in another cracking distress, has also been noted by others [6].
- 5-Inch Overlay Sections:
  - Higher levels of observed Wheelpath Longitudinal Cracking and generally higher levels of Alligator Cracking within the same paving surface condition (i.e. – milled vs unmilled) for the 30% RAP mixture compared to the Virgin mix. For the unmilled condition, the Alligator Cracking was over 4 times greater in the 30% RAP section than the Virgin mix section. Meanwhile, in the milled paving surface condition, the Alligator Cracking in the Virgin mixture was 1.4 times greater than that observed in the 30% RAP section.
  - Regarding Transverse Cracking, for the unmilled paving surface condition, the 30% RAP mixture had almost two times the observed cracking levels as the Virgin mixture. Meanwhile, almost identical Transverse Cracking levels were observed in the milled paving surface condition section.
  - The Non-Wheelpath Longitudinal Cracking was very similar for both the 30% RAP and Virgin mixtures.

The general findings indicate that the 30% RAP sections did not perform as well as the Virgin mixture with respect to cracking performance, especially when comparing the performance within each paving surface condition, contradicting some of the general findings presented by West et al. [3]. It appears one of the main reasons for the contradiction was this "compensatory difference" discovered with some of the cracking distresses. To help determine why the 30% RAP mixtures did not perform as well as the Virgin mixture, even when utilizing a softer asphalt binder, a laboratory forensic testing program was conducted.

CONSTRUCTION DETAILS

The asphalt mixtures placed on New Jersey's SPS-5 sections were produced by Trap Rock Industries in Kingston, NJ at the asphalt supplier's quarry. The asphalt plant was a portable, automated recycling drum made by Standard Havens and had a capacity of 350 tons/hr. The haul distance from the asphalt plant to field location was approximately 32 miles. The paving equipment consisted of a Barber Greene BG-760 Paver, two CAT BC 534 10-ton double drum vibratory rollers, and a Sakai 2.5 ton vibratory roller, which was only used to compact the sloped wedge of the base course HMA. Prior to the placement of the overlays, two steel broom and vacuum pump trucks were used to clean the pavement surfaces (Milled and Unmilled). Prior to paving, a 40% diluted CSS-IH emulsion tack coat was applied along the longitudinal and construction joints at a targeted application rate of 0.05 gals/yd^2.

A summary of the plant and field quality control information is shown in Table 2. The data shows that total asphalt content for both the Virgin and 30% RAP mixtures were almost identical, with only slight deviations between the aggregate gradations. The Virgin mixture utilized an AC-20 asphalt binder supplied by Citgo Asphalt in Paulsboro, NJ, while the 30% RAP mixture utilized a softer binder, AC-10, supplied by ELF in Petty's Island, NJ. According to the Job Mix Formula, the RAP constituted 1.5% of the total asphalt in the 30% RAP mixture, resulting in an approximate percent binder substitution of 32%.

Table 2 – Quality Control and Construction Information for Virgin and 30% RAP Sections

Sieve Size (mm)	Percent Passing (%)			
	Virgin Mix		30% RAP Mix	
	Average	Std. Dev.	Average	Std. Dev.
25	100	0	100	0
19	98.4	0.81	99.5	0.67
12.5	91.2	1.66	92.1	1.72
9.5	84.7	2.33	83.7	2.45
4.75	58.2	2.80	53.4	2.91
2.36	44.8	1.36	41.5	3.03
1.18	34.5	1.51	33.4	2.07
0.6	27.1	1.40	26.4	1.21
0.3	17.1	0.73	16.8	0.66
0.15	--	--	--	--
0.075	6.20	0.43	6.45	0.65
Asphalt Content (%)	4.75	0.18	4.79	0.17

NJ SPS-5 Sections	Paving Surface	Mix Type	Average Air Voids (%)	Ave. Temperature Plant/Laydown (^{o}F)
502 & 503	Unmilled	30% RAP	7.2	295/260
504 & 505	Unmilled	Virgin	6.4	310/283
508 & 509	Milled	30% RAP	5.9	295/273
506 & 507	Milled	Virgin	5.1	305/287

Average core densities taken immediately after construction are shown in Table 2 as well. Table 2 indicates that on average, better densities were achieved on the milled surface than the unmilled paving surface. The results also indicate that on average a 0.8% reduction in compacted air voids was achieved with the Virgin mixtures when compared to the 30% RAP mixtures for the same paving surface (i.e. Milled or Unmilled surface). The better compacted mat may have been a result of higher compaction temperatures for the Virgin sections as compared to the 30% RAP sections, also shown in Table 2. Additional construction details can be found in [7].

MATERIALS

Prior to the close out of New Jersey's SPS-5 sections, an extensive field investigation was conducted. The LTPP Northeast contractor, Stantec Consulting, conducted a visual distress survey while the NJDOT's contractor, Advanced Infrastructure Design (AID) conducted Falling Weight Deflectometer (FWD), Dynamic Cone Penetrometer, and extracted field cores from the various sections. For this study, only the laboratory testing portion of the study will be discussed. With a total of eight different test sections (2 mix types x 2 overlay thickness x 2 milled types) and the contractor extracted eight cores per section, a total of 64 cores (32 for each mix type) were available for evaluation.

Along with the extracted cores, raw materials were requested from the LTPP Materials Library for evaluation. Materials procured for additional evaluation were;

- Asphalt binder from the asphalt plant's storage tanks;
  - AC-10
  - AC-20
- RAP
- Collected loose mix (1.5 5-gallon containers of each – all remaining loose mix from the project)
  - Virgin Mixture
  - 30% RAP Mixture

LABORATORY INVESTIGATION

A laboratory investigation was designed to evaluate the stiffness and fracture properties of the asphalt binders and mixtures collected. Testing was conducted on the procured LTPP Materials Library materials, providing an "initial" or "early life" look at the asphalt binder and mixture properties. Testing was also conducted on the field cores and asphalt binder extracted from the field cores, providing an "end of service" look at the asphalt binder and mixture properties.

Asphalt Binder Properties

<u>Performance Grading</u>

Performance grading of the asphalt binders was conducted in accordance with AASHTO M320, *Performance-Graded Asphalt Binder* and R29, *Grading or Verifying the Performance Grade (PG) of an Asphalt Binder*. The performance grading was conducted on the virgin materials (AC-10, AC-20, and extracted and recovered asphalt binder from the RAP), as well as the extracted and recovered asphalt binder from the cores. The extraction and recovery was conducted on field cores taken from the 5-inch overlay area allowing asphalt binder testing to be conducted on the two separate lifts; Top 2 inches and Bottom 3 inches of the field core. Asphalt binder from each core was extracted in accordance with Method A of AASHTO T164, *Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA),* and then recovered from the extract in accordance with AASHTO T170, *Recovery of Asphalt From Solution by Abson Method.* The resultant performance grading on the asphalt binders are shown in Table 3. The test results show that the RAP binder was extremely stiff and highly oxidized. The test results also show the asphalt binder in the upper 2 inches of the core was approximately one grade warmer for the high temperature PG grade. All extracted/recovered binder samples resulted in the same low temperature PG grade. The PG grade results also indicate that the AC-10 graded out to a PG58-28 and the AC-20 graded out to a PG64-22.

Table 3 – Performance Grade Properties of Tank, RAP, and Extracted/Recovered Binders

Location in Overlay	Continuous PG Grade				
	AC-10	AC-20	RAP	Virgin - Field Core	30% RAP - Field Core
Top 2"	61.5-31.5 (58-28)	65.9-23.9 (64-22)	100-1.4 (100-0)	72.3-19.2 (70-16)	77.3-19.0 (76-16)
Bottom 3"				67.6-20.6 (64-16)	72.0-21.1 (70-16)

Applied Cyclic Strain	Linear Amplitude Sweep (LAS) - Cycles to Failure			
	AC-10 (PAV)	AC-20 (PAV)	30% RAP Core	Virgin Mix Core
2.5%	35,784	13,275	56,601	37,379
5.0%	1,392	605	1,327	1,358

Asphalt Binder Stiffness

Temperature-frequency sweeps were conducted on the asphalt binder using the 4mm geometry in the dynamic shear rheometer [8]. The resultant master stiffness curves are shown in Figure 4. The figure contains the stiffness properties of two sets of binders; 1) Reheated loose mix and 2) Top 2-Inch lift of the field cores. This provides a general progression of binder stiffness from “early life” to the “end of service”. In general, the master curves show the asphalt binder at the low temperature range is fairly consistent among the four extracted asphalt binders tested. However, at the intermediate and higher test temperatures, the stiffness properties of the binders separate. The 30% RAP binder showed the greatest increase in stiffness at the intermediate and high temperature ranges, approximately doubling in stiffness. Meanwhile, little to no increase in stiffness was found between the reheated loose mix and field cores for the Virgin mixture.

Asphalt Binder Fatigue Testing – Linear Amplitude Sweep (LAS) Test

The tank samples and extracted/recovered asphalt binders were evaluated for their fatigue properties using the Linear Amplitude Sweep (LAS) test procedure [9]. The LAS Test consists of a series of cyclic loads at systemically linearly increasing strain amplitudes at a constant frequency of 10 Hz. The LAS Test utilizes the dynamic shear rheometer with the standard 8-mm parallel plate configuration. Specimen loading begins with 100 cycles of sinusoidal loading at 0.1%. Each successive loading step consists of 100 cycles at a rate of increase of 1% applied strain. The procedure also includes a frequency sweep test at a very low strain amplitude of 0.1% to obtain undamaged material properties. The resultant test data can be analyzed using viscoelastic continuum damage (VECD) concepts, which has been used extensively to model complex fatigue behavior of asphalt mixtures.

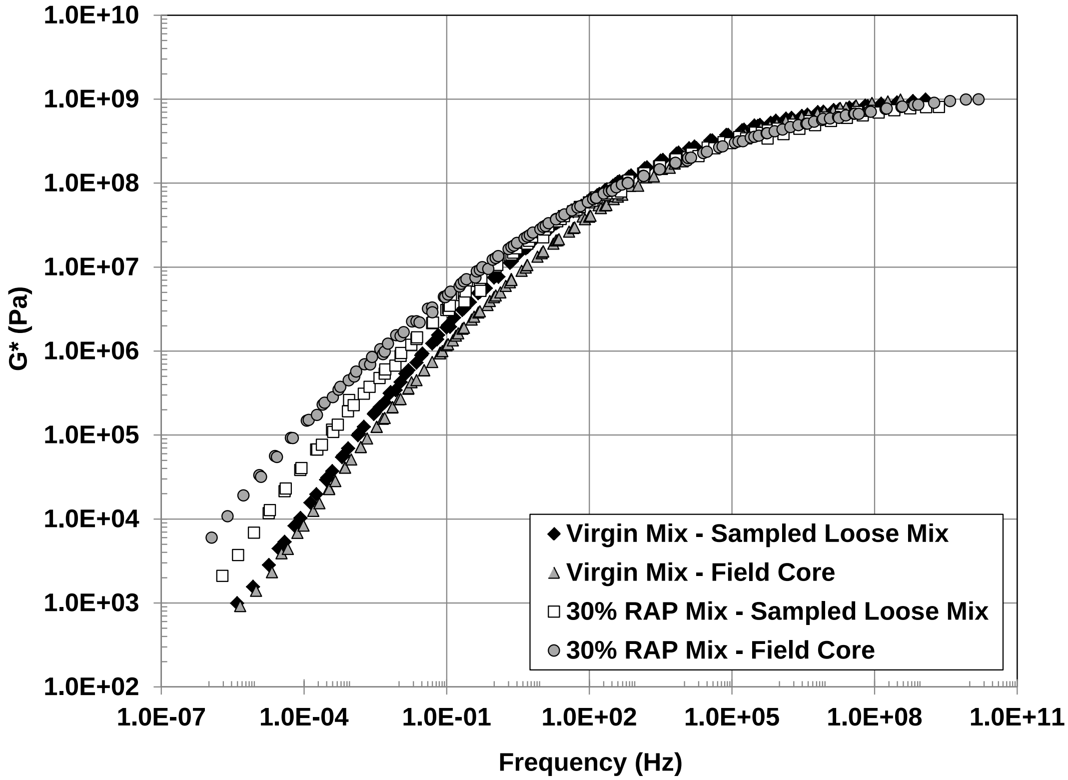


Figure 4 – Master Stiffness Curves of Extracted Asphalt Binders

The test results for the LAS testing at two different strain levels are shown in Table 3. The test results indicate that the softer binder (AC-10) would certainly have helped provide additional fatigue resistance when incorporated with 30% RAP as the AC-10 resulted in 2 to 3 times the fatigue resistance as the AC-20 binder. LAS test results of the extracted binders from the field cores also supports this statement as the asphalt binder fatigue performance appears to be better for the 30% RAP mixture (field core) than the Virgin mixture (field core), especially at the lower strain level. Unfortunately, the asphalt binder fatigue properties do not match the general field cracking observations.

Asphalt Mixture Properties

To assess the cracking potential of NJ's LTPP SPS-5 30% RAP and Virgin mixtures, stiffness and cracking tests were conducted on both reheated loose mix and field cores. Unfortunately, due to the limited amount of loose mix remaining from the 16 years of storage, as well as the limitation of obtaining field cores, a limited array of laboratory tests were able to be performed.

Prior to the laboratory study, a quick review of the Quality Control information from the initial construction showed that there were no significant differences in asphalt content, gradation, and volumetrics between the Virgin and 30% RAP mixtures [7].

Asphalt Mixture Fatigue Cracking – Overlay Tester

The Overlay Tester, described by Zhou and Scullion [10], has shown to provide an excellent correlation to field cracking for both composite pavements [10, 11] as well as flexible pavements [12]. Sample preparation and test parameters used in this study followed that of TxDOT Tex-248-F testing specifications. These include:

- 25^{o}C (77^{o}F) test temperature;
- Opening width of 0.025 inches;
- Cycle time of 10 seconds (5 seconds loading, 5 seconds unloading); and
- Specimen failure defined as 93% reduction in Initial Load.

The test results for the Overlay Tester testing are shown in Figure 5. The test results show that the Overlay Tester fatigue performance for the Virgin mixture was superior to the 30% RAP mixture for both the reheated loose mix ("early life") and field cores ("end of service life").

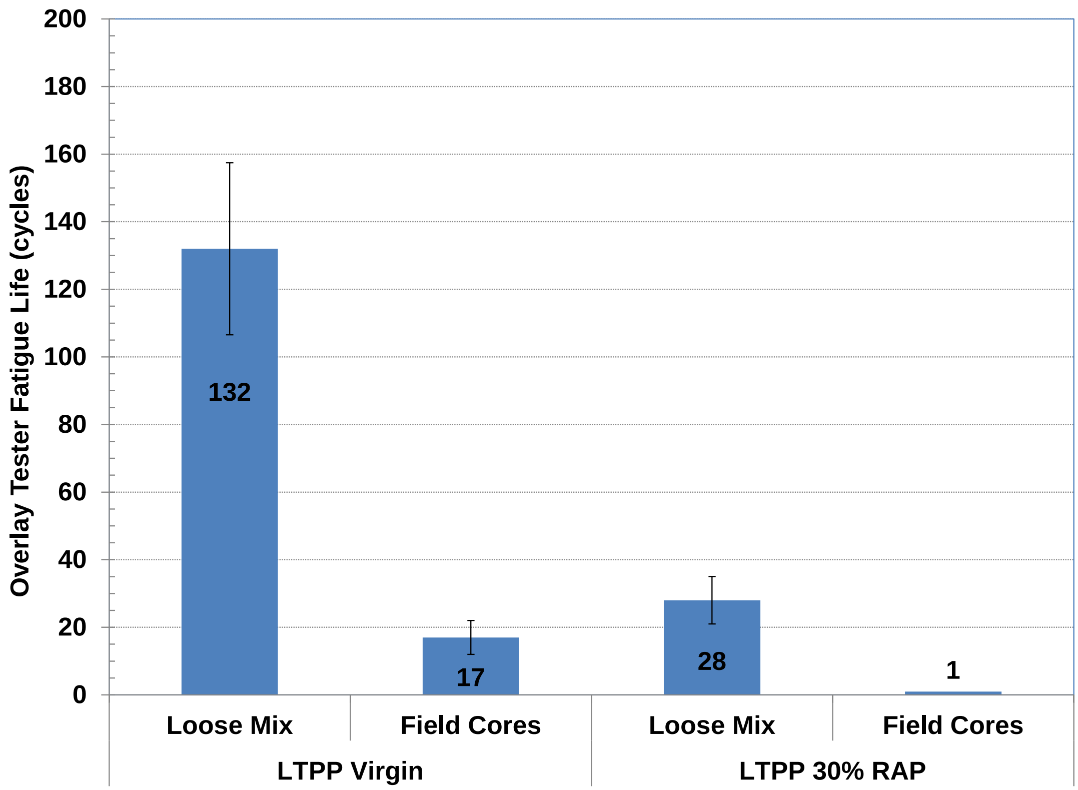


Figure 5 – Overlay Tester Fatigue Cracking Performance

Asphalt Mixture Low Temperature Cracking – Disk Shaped Compact Tension (DC(T))

The low temperature cracking potential of the asphalt mixtures were evaluated using the Disk Shaped Compact Tension (DC(T)) test in accordance with ASTM D7313-07, *Standard Test Method for Determining Fracture Energy of Asphalt-Aggregate Mixtures Using the Disk-Shaped Compact Tension Geometry*. A test temperature of -12^{o}C was chosen to correspond to the ASTM recommended test temperature for mixtures designed with a -22^{o}C low temperature PG grade. The DC(T) test is conducted using crack mouth opening displacement (CMOD) control, at a rate of 1.0 mm/min. Research by others has indicated that the fracture energy determined from the DC(T) test correlates well to low temperature cracking in asphalt pavements [13-15].

The DC(T) Fracture Energy results for the reheated loose mix and field cores are shown in Figure 6. The test results in Figure 6 show that on average the Virgin mixture resulted in larger magnitudes of fracture energy than the 30% RAP mixture for the reheated loose mix ("early life") and field cores ("end of service life"), respectively.

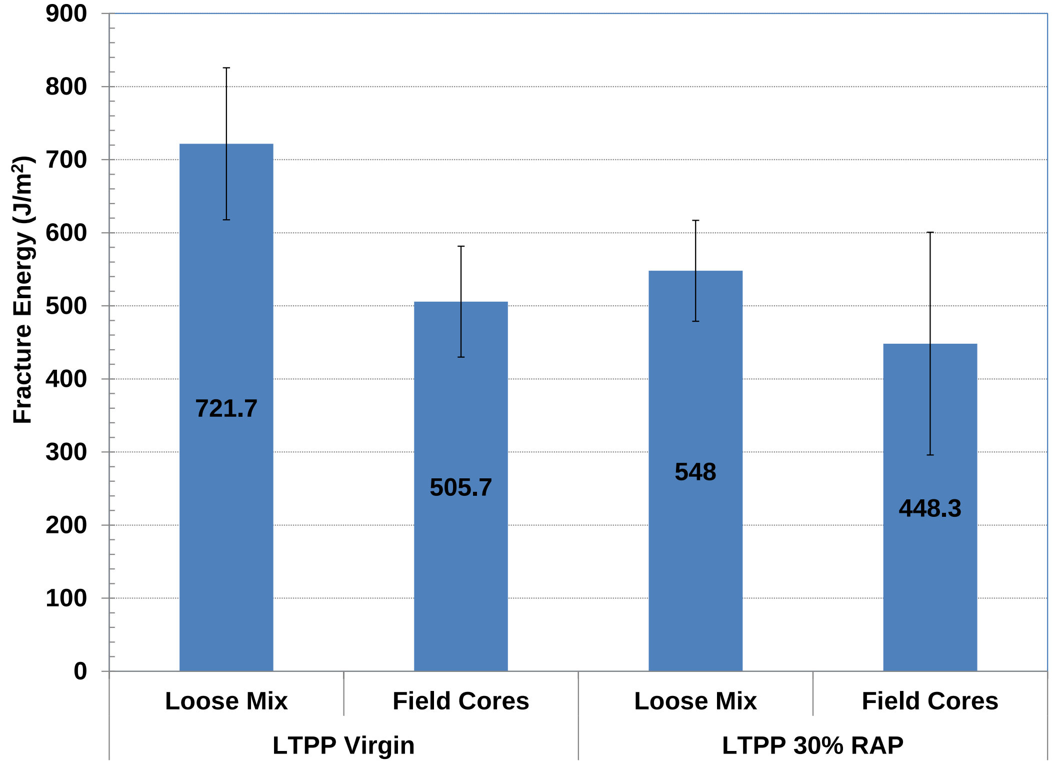


Figure 6 – Disk Shaped Compact Tension (DC(T)) Test Results

Proposed specification limits for the DC(T) test recommends the following minimum fracture energy values for different traffic levels when tested at the PG low temperature grade + 10°C [16]:

- < 10 million ESAL's: Minimum of 400 J/m^2
- 10 to 30 million ESAL's: Minimum of 460 J/m^2
- > 30 million ESAL's: Minimum of 690 J/m^2

Traffic levels during NJ's SPS-5 program would comply with the <10 million ESAL range and would require a fracture energy with a minimum of 400 J/m^2. It should be noted that the proposed specification is based on laboratory prepared mixtures long-term aged for 5 days at 85°C. Therefore, the data shown as the field cores would best represent this condition and show that the 30% RAP mixture is close to failing the criteria, which may explain the higher levels of Transverse and Block Cracking measured in the 2-Inch Overlay sections and higher levels of Transverse Cracking in the 5-Inch Overlay sections.

Asphalt Mixture Low Temperature Cracking – Creep Compliance and Indirect Tensile Strength

Similar to the Overlay Tester and DC(T) test, low temperature Creep Compliance and IDT strength testing was conducted to evaluate the low temperature cracking performance of the mixtures. Again, reheated and compacted loose mix and field cores were evaluated. The test results were analyzed using the LTSTRESS Excel® spreadsheet developed by Dr. Don Christensen. The LTSTRESS spreadsheet predicts the mixture critical cracking temperature based on the thermal cracking model TCMODEL; a mechanistic-based prediction model developed under the Strategic Highway Research Program (SHRP) and then revised to be included in the AASHTO Mechanistic Empirical Pavement Design Guide [17, 18].

The low temperature creep compliance and IDT results are shown in Figure 7. The results are fairly consistent with one another and do not show any significant differences among the different mixtures and specimen types.

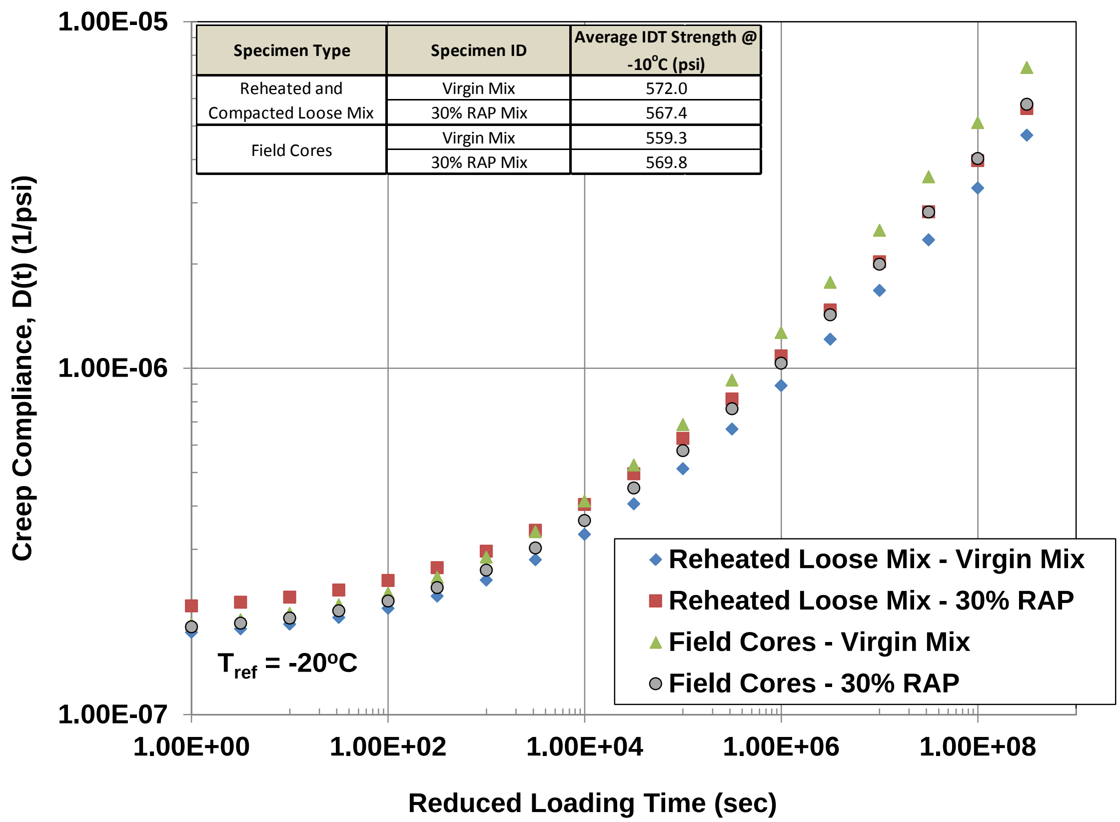


Figure 7 – Low Temperature Creep Compliance Master Curves and IDT Strength

The test results shown in Figure 7 were inputted into the LTSTRESS spreadsheet to determine the critical cracking pavement and air temperatures for low temperature thermal cracking. The resultant analysis was as follows:

- Reheated and Compacted Loose Mix
  - Virgin Mix: Pavement Temperature: -16°C / Air Temperature: -21°C
  - 30% RAP Mix: Pavement Temperature: -16°C / Air Temperature: -21°C
- Field Cores
  - Virgin Mix: Pavement Temperature: -17°C / Air Temperature: -21°C

- 30% RAP Mix: Pavement Temperature: -14^{o}C / Air Temperature: -18^{o}C

The LTSTRESS analysis indicated that no difference was found with the reheated and compacted mixtures, and was similar to the Virgin Mix field core results. However, the analysis on the 30% RAP Field Cores did result in warmer critical cracking temperatures than the Virgin Mix field cores.

SUMMARY AND CONCLUSIONS

The Long Term Pavement Performance (LTPP) SPS-5 testing program was intended to evaluate the performance of different overlay thickness, paving surface condition, and asphalt mixture type. Each of the SPS-5 locations utilized two different asphalt mixtures, a control Virgin Mix and a 30% RAP mixture, and placed using different overlay thickness and paving surface preparations. The LTPP SPS-5 test sections in New Jersey were evaluated prior to its close out in 2010 to assess the cracking performance of each mixture within the same pavement condition (i.e. – overlay thickness and paving surface preparation) and compare those results to laboratory testing. The results of the forensic study showed that:

- Overall, the 30% RAP mixture resulted in greater magnitudes of cracking when compared to the Virgin mixture within the same overlay thickness and paving surface condition, even though the 30% RAP mixture utilized a softer asphalt binder (AC-10 versus AC-20). Comparing the performance without taking into consideration these structural differences would have led to inaccurate comparisons. Although the time at which cracking was first observed (initiation) was very similar, the accumulation or propagation of cracking was greater for the 30% RAP mixture sections than the Virgin mixture sections.
- As would be expected, the magnitude of the cracking distresses was more significant in the 2-Inch Overlay sections when compared to the 5-Inch Overlay sections.
- In some of the cracking observations, the magnitude of cracking significantly decreased later in the service life. However, it should be noted that this is not due to any type of pavement rehabilitation strategy. What appears to be occurring is a particular cracking distress is progressing to a point where it "migrates" into another, more severe, cracking distress. In evaluating the cracking distress versus time, it was apparent that Wheel Path Longitudinal Cracking clearly migrated to Alligator Cracking. It is hypothesized that the same occurred with the Transverse Cracking and Non-Wheelpath Longitudinal Cracking in the 2-Inch Overlay sections. Both of these cracking distresses underwent a significant decrease in the observed cracking levels for the 30% RAP mixture at approximately the 8 to 10 year mark. However, at the same time, Block Cracking for the 30% RAP mixture in the 2-Inch Overlay section significantly increased within the same time period. This "compensatory differences" phenomenon has been observed by others [6] and is not unique for NJ's LTPP SPS-5 sections.
- Extraction and recovery of the asphalt binder from field cores of the Virgin and 30% RAP mixtures showed that the stiffness (G*) of the asphalt binders (reheated/compacted and field cores) had similar low temperature stiffness'. Differences were not found until the intermediate to higher test temperatures. At those temperatures, the 30% RAP mixture was stiffer for both the reheated/compacted and field core conditions. Binder fatigue testing using the Linear Amplitude Sweep (LAS) test procedure indicated that the 30% RAP binder performed better in fatigue than the Virgin mixture – which is a

contradiction to the observed field performance. The low temperature PG grade showed that the binders graded out to a -16^{o}C for all extracted/recovered asphalt binders.

- Mixture testing at intermediate temperatures with the Overlay Tester indicated that the Virgin mixture should perform better than the 30% RAP mixture, which matched the observed field performance. Low temperature cracking potential was evaluated using the Disk Shaped Compact Tension (DC(T)) test and the low temperature Creep Compliance and Indirect Tensile Strength. The DC(T) testing showed that on average, the Virgin mixture performed better than the 30% RAP mixture. Comparing the results of the field cores, the 30% RAP mixture DC(T) performance is "borderline" with respect to proposed criteria for the DC(T) Fracture Energy, while the Virgin mixture would have passed the criteria. Meanwhile, the analysis of the Creep Compliance and IDT data using the LTSTRESS analysis spreadsheet showed that the 30% RAP field core resulted in the warmest critical cracking pavement and air temperatures. When compared to the field cores of the Virgin mixture, the 30% RAP mixture was 3^{o}C warmer, indicating that it is more susceptible to low temperature thermal cracking than the Virgin mixture.

Overall, the mixture results compared favorably to the observed cracking in the field. Meanwhile, the asphalt binder results showed conflicting comparisons between their respective performance and the observed field cracking performance. With both the 30% RAP and Virgin sections performing similarly with respect to crack initiation, it appears that the final crack severity in the LTPP SPS-5 sections was influenced more by the crack propagation properties of the different mixtures (Virgin and 30% RAP) than the crack initiation. Crack propagation (or growth) has been shown to be significantly influenced by the fracture energy properties of the asphalt mixture. Roque et al. [19] showed that the higher the measured fracture energy of the asphalt mixture, the lower the average crack growth rates. The fracture energy results in this study, as well as others [20-22], indicates that as RAP content increases, fracture energy values generally decrease. However, with some modifications to the mixture design, fracture energy values have been shown to improve in higher RAP mixtures. In particular, work conducted at the National Center for Asphalt Technology (NCAT) indicated that when increasing the effective asphalt content in higher RAP mixtures, fracture energy values identical to virgin mixtures were achievable, while mixed results were found when using a softer asphalt binder [22]. Similar fatigue performance was also found in asphalt mixtures produced in New York State when effective asphalt contents were increased [23]. Therefore, based on the field and mixture results of NJ's LTPP SPS-5 sections, an increase in the effective asphalt content of the 30% RAP mixture may have increased the performance and mitigated the accelerated crack propagation. Future implementation of increasing the effective asphalt content in higher RAP mixtures can easily be incorporated by increasing minimum voids in mineral aggregate (VMA) values in agency specifications.

ACKNOWLEDGEMENTS

The authors would like to thank a number of participates that helped in the collection of the information presented in this paper; Stantec Consulting for providing the pavement distress measurements; Raj Dongre for providing the PG grading of the asphalt binders; Gerald Reinke for providing the 4-mm master stiffness and Linear Amplitude Strain (LAS) testing; Bill Buttlar for providing the DC(T) testing; and Jo Sias-Daniel for conducting the creep compliance and

IDT testing and analysis. Their help and support is greatly appreciated. Funding for the field coring was provided by the New Jersey Department of Transportation (NJDOT). The assistance and support of Eileen Sheehy and Robert Blight of the NJDOT Materials Bureau, and Susan Gresavage of the NJDOT Pavement Design Bureau, is greatly appreciated.

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Proposed Mixture Design Specification for a 4.75mm SMA Using Crumb Rubber Modified Asphalt Binder

- Scoping Report -

Submitted to:

New Jersey Department of Transportation (NJDOT)
Bureau of Materials

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SCOPE OF WORK

The purpose of the following report is to summarize a proposed mixture design specification and procedure for developing a 4.75mm SMA asphalt mixture using crumb rubber modified asphalt binder. The main purpose of this mixture would be for use as a Pavement Preservation mixture placed at 1-inch or less.

Upon agreement from NJDOT, the Center for Advanced Infrastructure and Transportation (CAIT) at Rutgers University will proceed to evaluate the mixture design specification utilizing local aggregates from Trap Rock Industries and Tilcon Mt. Hope.

The information presented in this report is based on a Literature Review, phone and email interviews with industry and agency engineers. Emails were also sent to both Trap Rock Industries and Tilcon Mt. Hope to ask whether the general aggregate blend proposed would be feasible.

AGGREGATE BLEND

Based on the Literature Review and information collected, two 4.75mm SMA mixtures had been proposed; 1) Maryland State Highway Administration (MDSHA) and 2) National Center for Asphalt Technology (NCAT). The gradation bands for each of the proposed mixture designs are shown in Table 1. The gradation band proposed by NCAT appears to be more restrictive than the MDSHA gradation band. Although the MDSHA has an asphalt content requirement, the NCAT specification does not.

Table 1 – Proposed Gradation Bands for NJDOT 4.75mm SMA

Sieve Size (mm)	% Passing	
	NCAT	MDSHA
9.5	100	100
4.75	90 - 100	80 - 100
2.36	28 - 65	36 - 76
1.18	22 - 36	---
0.6	18 - 28	---
0.3	15 - 22	---
0.15	---	---
0.075	12 - 15	2 -12
Asphalt Content	---	5 - 8

Fibers are not proposed for the 4.75mm SMA as previous testing has indicated that the fine graded nature of the 4.75mm SMA has enough surface area and mastic to limit the draindown potential. A design gyration level of 75 gyrations is recommended, as this is consistent with current NJDOT SMA design, as well as what was recommended in the NCAT research.

DESIGN AIR VOIDS

In their study, NCAT looked at design air void ranges between 4 to 6%. The general concern is that excessively high optimum binder contents may result at the traditional 4% target air void level due to the high surface area of the gradation and the need for a potentially high VMA. Meanwhile, the MDSHA recommendation was the traditional 4% air void level. Rutting tests should be conducted on the 4.75mm SMA to verify whether or not the 4% air voids design results in excessive asphalt contents that would jeopardize the stone-on-stone contact and make the material susceptible to pushing/shoving.

VOIDS IN COARSE AGGREGATE (VCA)

Similar to other gap-graded asphalt mixtures, it is important to ensure stone-on-stone contact exists. Gap-graded mixtures rely on stone-on-stone contact for rutting resistance and may be unstable when too much mastic, asphalt binder or air resides between the aggregate particles.

Traditionally VCA is determined using the coarse aggregate fraction of the aggregate blend. However, since coarse aggregate is commonly defined as the material coarser than the 4.75mm sieve, it was recommended to use the 1.18mm sieve (#16 sieve) as the breakpoint sieve in determining VCA. Therefore, it is recommended that only aggregates retained on the 1.18mm sieve (#16 sieve) be used to determine VCA.

DRAINDOWN

Research at NCAT has indicated that fibers are not required with a 4.75mm SMA due to the high surface area of the fine aggregates. However, draindown still needs to be evaluated. Due to the finer aggregate structure, a 2.36mm mesh basket is required. Conventional SMA mixtures by the NJDOT utilize a 4.75mm mesh basket, which would be too coarse and allow some of the finer aggregates to fall through.

INITIAL INDUSTRY CONVERSATIONS

A look at the recommended gradation bands for the 4.75mm SMA showed that to consistently produce an aggregate blend that met the gradation band requirements, a #9 (1/4”) stone is required. At this time, CAIT is aware of only three quarries in NJ that are producing this stone size; 1) Trap Rock Industries (Kingston, NJ), 2) Tilcon Mt. Hope, and 3) Tilcon Oxford. An initial look at the Trap Rock and Tilcon Mt. Hope aggregates are shown in Figures 1 and 2. The aggregates produced by Trap Rock Industries is capable of meeting both recommended gradation bands. Meanwhile, the Tilcon Mt. Hope aggregates are a little coarse on the 4.75mm sieve. However, at the time of this report, the Tilcon Oxford aggregates were not available for evaluation. A more detailed blend analysis will be conducted using the QC gradations from the respective quarries moving forward.

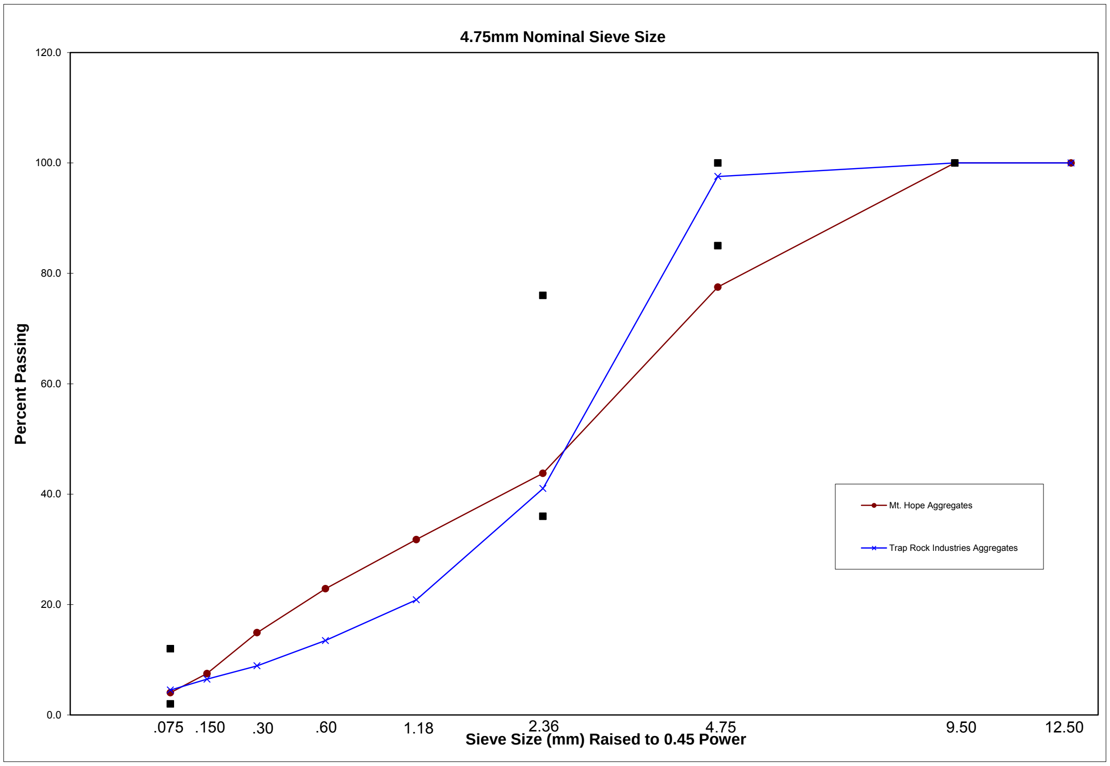

Figure 1 – MDSHA 4.75mm SMA Gradation Band

CAIT also reached out to Mike Jopko and Wayne Byard of Trap Rock Industries to ask how they perceived the recommended gradation bands of NCAT and the MDSHA. Both Mike Jopko and Wayne Byard preferred the MDSHA gradation band as they felt it was less restrictive.

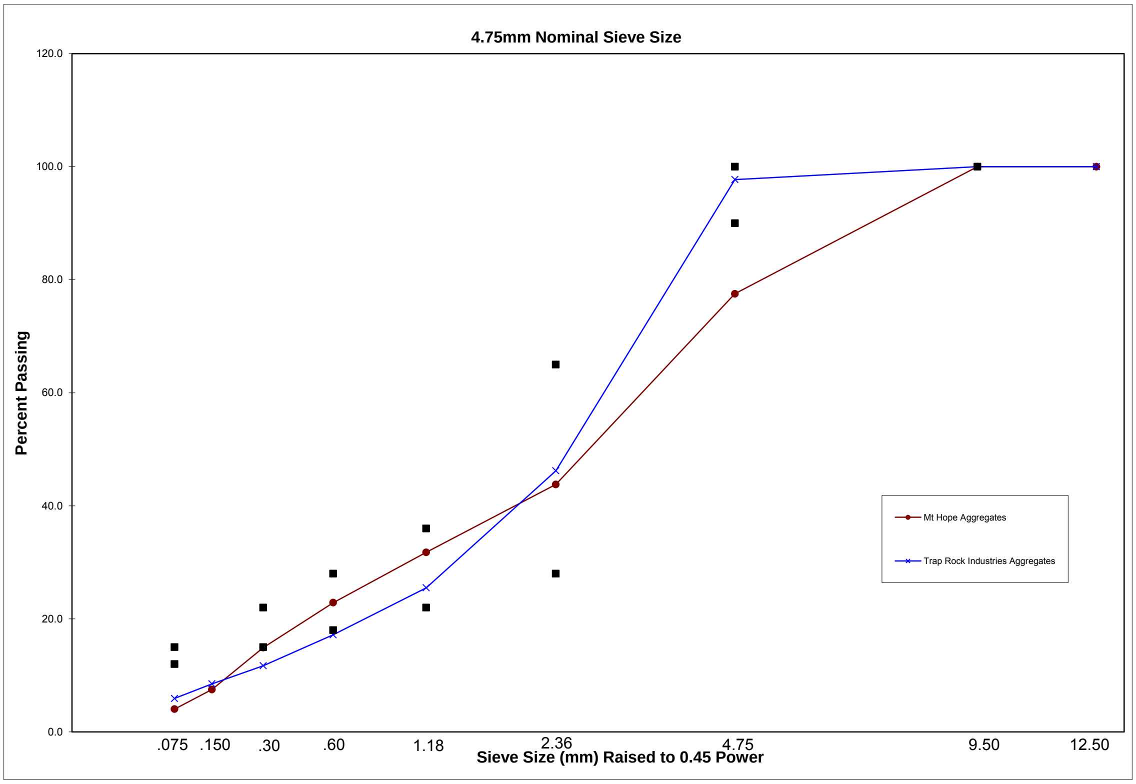

Figure 2 – NCAT 4.75mm SMA Gradation Band

RECOMMENDATIONS MOVING FORWARD

Based on the information collected to date, CAIT is recommending moving forward to verify a 4.75mm SMA can be produced and perform adequately. Below are recommendations CAIT is making to NJDOT to move forward:

- 75 Design Gyration Level
- Asphalt Binder: PG76-22 and CRM Modified Binder (30 Mesh or finer)
  - Mesh size will most likely need to be verified
- MDSHA Gradation Band or slightly modified version
- Performance testing during research study
  - APA (rutting)
  - Overlay Tester and Beam Fatigue (fatigue cracking)
  - Tensile Strength Ratio (TSR)
- Aggregate sources
  - Trap Rock
  - Tilcon Mt. Hope (or Oxford)
- If 4% design air voids results in high APA rutting, 5% will be evaluated

Overview of Mechanistic Empirical Design Tools for Flexible Pavements

Prepared by Dr. Hao Wang, Rutgers University

Mechanistic-empirical (M-E) pavement design methods represent one step forward from empirical design methods. The mechanistic-empirical approach is a hybrid approach including two steps. Firstly, mechanistic models are used to calculate pavement responses with assumptions and simplifications (i.e., homogeneous material, small strain analysis, static loading, and linear elastic theory). Secondly, empirical models are used to fill in the gaps that exist between the calculated pavement responses (stress and strain) and the performance of pavement structures (rutting, cracking, etc.).

A newly developed Mechanistic-Empirical Pavement Design Guide (MEPDG) (now DARWIN-ME) has been developed based on the research outcome from NCHRP 1-37A project that includes various design modules for evaluating and designing pavements. However, implementing the MEPDG will require significant material testing efforts and a local calibration and verification on the empirical performance models used in the software. In order to implement the M-E design approach in a quick manner, several state DOTs have developed their own M-E design procedures and tools, such as Texas DOT, Idaho DOT, Washington DOT, and Minnesota DOT. Although the M-E procedures developed by state DOTs follow the major M-E design principle, they are relatively simple in the design inputs and easy for implementation.

Based on the review of literature and DOTs' pavement design websites, Table 1 compares the features of the currently available M-E design tools developed by state DOTs and as well as the new AASHTO DARWIN-ME. The comparisons mainly focus on flexible pavement and overlay design.

Table 1 Features of Mechanistic Empirical Design Tools for Flexible Pavements

Agency	IDDOT	TXDOT	WSDOT	MNDOT	AASHTO
Software/ Tools	WinFLEX 2006	FPS 21	EVERPAVE	MnPAVE	DARWIN-ME
Level of complexity	Low			Medium	High
Traffic	ESAL	ESAL	ESAL	ESAL and axle load spectrum	Axle load spectrum
Asphalt Material	Design modulus adjusted by temperature	Design modulus	Design modulus adjusted by temperature	Design modulus or mixture volumetrics	Three level inputs
Unbound Material	Elastic modulus	Elastic modulus	Stress-dependent modulus	Elastic modulus	Three level inputs
Structure	Layered Elastic (CHEVRON)	Layered Elastic	Layered Elastic (WESLEA)	Layered Elastic (WESLEA)	Layered Elastic (JULEA)
Climate	Seasonal adjustment	NA	Seasonal adjustment	Seasonal adjustment	Enhanced Integrated Climatic Model
Performance prediction	Fatigue cracking and HMA rutting	Fatigue cracking and HMA rutting; subgrade shear failure	Fatigue cracking and HMA rutting	Fatigue cracking and HMA rutting	Fatigue and top-down cracking; HMA and subgrade rutting; roughness
Reliability	NA	NA	Adjustment to design ESAL	Monte Carlo simulation	Normal distribution for each distress
Overlay design module	Yes	Yes	Yes	Yes (FWD analysis included)	Yes

The following sections describe the details of each M-E design tool for flexible pavement and overlay design.

IDDOT WINFLEX

The WINFLEX is developed through a research project conducted at the University of Idaho in conjunction with Idaho Transportation Department. The latest version is the WINFLEX 2006.

In the WINFLEX 2006, Traffic is estimated in 18-kip equivalent single axle loads (ESALs). The environmental parameters that are considered to affect the variation of the pavement properties are: temperature (for asphalt bound layers) and moisture (for unbound materials). The proposed overlay design procedure uses this schematic for the AC Modulus-Temperature adjustment and equation modified from the original SHRP equation (Eq. 1). The default exponent (n) in WINFLEX is 0.35, which is typical for conventional asphalt mixes. The user is allowed to choose a new equation where the exponent (n) and the slope can be entered. The actual Intercept can then be calculated based on the provided modulus at the test temperature for the design case in consideration.

$$E^{n} = \left(F_{cept} - slope * T_{p}\right) \qquad \text{(Eq. 1)}$$

The seasonal shift factors for the modulus of unbound material in the WINFLEX database were developed for specific Idaho zones. For layers that are subjected to freezing conditions, the moduli values become extremely large. In this condition, an upper frozen value would be suggested.

The WINFLEX adopted the CHEVRON program, which was developed by the Chevron research company as a routine to handle the calculations of critical stresses and strains. In flexible pavement the critical tensile strains are located at the bottom of the asphalt layers. The critical compressive strains are located at the top of the subgrade.

Since different loading conditions for each season will be encountered, Miner’s hypothesis is the one most often used, which allows an accumulation of damage from the various load conditions to be combined into one damage number. Similar to other M-E design methods, pavement damage analysis was conducted for fatigue cracking and rutting in AC layer, respectively. The user can select the cracking or rutting model from a series of available damage models that are embedded in the program.

Reference*: Bayom, F.M., WINFLEX2006 - Technical Background for Program Development, Final Report for Idaho Transportation Department, 2006*

Website*: http://www.webs1.uidaho.edu/ce475/Files/SOFTWARE%20Files/Software.htm*

MNDOT MnPAVE

MnPAVE is a computer program that combines known empirical relationships with a representation of the physics and mechanics behind flexible pavement behavior. The mechanistic portions of the program rely on finding the tensile strain at the bottom of the asphalt layer, the compressive strain at the top of the subgrade, and the maximum principal stress in the middle of the aggregate base layer.

MnPAVE consists of three input modules: Climate, Structure, and Traffic. Climate contains a map of Minnesota where more specific location data can be entered. MnPAVE calculates season lengths and temperatures for each location using data from surrounding weather stations. The default traffic load type in MnPAVE is the ESAL. However, load spectrum can be also used to categorize the expected traffic by axle type and load range

MnPAVE has three design levels: Basic, Intermediate, and Advanced. The level is selected based on the amount and quality of information known about the material properties and traffic data.

- The basic level uses default design modulus values adjusted for seasonal variations in moisture and temperature.
- The intermediate level corresponds to the amount of testing data currently required for MnDOT projects. Mix design information such as asphalt binder content and gradation are required to estimate the HMA dynamic modulus.
- The advanced level requires the determination of modulus values for all materials over the expected operating range of moisture and temperature.

MnPAVE simulates traffic loads on a pavement using a Layered Elastic Analysis (LEA) called WESLEA. All layers are assumed to be isotropic in all directions and infinite in the horizontal direction. The fifth layer is assumed to be semi-infinite in the vertical direction. Material inputs include layer thickness, modulus, Poisson’s ratio, and an index indicating the degree of slip between layers. MnPAVE assumes zero slip at all layer interfaces. Other inputs include load and evaluation locations. Loads are characterized by pressure and radius. The LEA program calculates normal and shear stress, normal strain, and displacement at specified locations.

MnPAVE output includes the expected life of the pavement, the damage factor based on Miner's Hypothesis. Reliability has been incorporated into the latest version using Monte Carlo simulation. There is also a research mode of MnPAVE that allows more features and more flexibility in entering data.

Reference: MnPAVE User's Guide, Minnesota Department of Transportation, 2012

Website: http://www.dot.state.mn.us/app/mnpave/index.html

TXDOT FPS

The Flexible Pavement System (FPS) is mechanistic-empirically (M-E) based design software routinely used by the Texas Department of Transportation (TxDOT) for: (1) pavement structural (thickness) design, (2) structural overlay design, (3) stress-strain response analysis, and (4) pavement life prediction (rutting and cracking). FPS 19W is the mechanistic-empirical flexible pavement design program that has been in use by the Texas Department of Transportation (TxDOT) since the mid-1990s. FPS 21 is the most recent version of this design system developed by the Texas Transportation Institute for TxDOT.

The FPS design approach is based on a linear-elastic analysis system, and the key material inputs are the backcalculated modulus values of the pavement layers. For in place materials, these are obtained from testing with the Falling Weight Deflectometer and processing the data with backcalculation software such as MODULUS 6. For newly placed materials, realistic average moduli values for the main structural layers in typical Texas pavements are supplied based on user experience, with recommended values also available in TxDOT's online pavement design guide.

The FPS design process is comprised of the following two steps: (1) generate a trial pavement structure with proposed FPS design thicknesses, and (2) check this design with additional analysis routines, which include mechanistic design check, which computed fatigue life and subgrade rutting potential, and the Modified Texas Triaxial criteria, which evaluates the impact of the anticipated heaviest load on the proposed pavement structure.

- The mechanistic design check computes horizontal tensile strain at the bottom of the lowest HMA layer and the maximum vertical compressive strain at the top of the subgrade. Standard models are available to convert these values into the number of standard 18-kip load applications until either cracking or subgrade rutting failure occurs. Currently the mechanistic design check is not required for pavement design approval (with the exception of pavements deliberately designed as "perpetual"), but it should be run for informational purposes on all HMA designs.
- The Modified Texas Triaxial criteria were developed to prevent a shear failure in the subgrade soil under the heaviest wheel load anticipated for the pavement section. Results of the analysis will recommend the total combined thickness of granular base, stabilized materials, and HMAC surface to prevent shear failures in the subgrade. Currently the Triaxial check is mandated for all flexible pavement designs developed for TxDOT maintained highways; however the results can be waived with justification by the approving engineer.

Reference: *Liu, W. and T. Scullion, Flexible Pavement Design System FPS 21: User's Manual, Texas Department of Transportation, 2011*

Website: *http://pavementdesign.tamu.edu/downloading.htm*

WSDOT EVERPAVE

EVERPAVE is a windows-based computer program developed by Washington DOT for the use of flexible pavement overlay design. EVERPAVE is based on the multilayered elastic analysis program. The existing layer moduli required in EVERPAVE are backcalculated from FWD deflection basins using EVERCALC, FWD backcalculation software developed by Washington DOT.

EVERPAVE calculates pavement responses using the WESLEA layered elastic analysis program (provided by the Waterways Experiment Station, U.S. Army Corps of Engineers). The pavement system model is multi-layered elastic using multiple wheel loads (up to 20). The program can analyze hot mix asphalt (HMA) pavement structure containing up to five layers and can consider the stress sensitive characteristics of unbound pavement materials. The consideration of the stress sensitive characteristics of unbound materials can be achieved through adjusting the layer moduli in an iterative manner by use of stress-modulus relationships.

The determination of the overlay thickness is based on the required thickness to bring the damage levels to an acceptable level under a design traffic condition. The traffic input is in terms of 18,000-lb ESALs. The damage levels are based on two primary distress types, fatigue cracking and rutting in the HMA layer, which are the most common criteria for mechanistic analysis based overlay design.

The HMA modulus is corrected for temperature according to data for typical Washington mixtures. The mean monthly air temperatures (MMAT) are required as inputs and converted to mean monthly pavement temperatures (MMPT). The unstabilized base, subbase, and subgrade layers can be adjusted for seasonal effects.

Reliability can be incorporated into the design procedure in a very appropriate and similar manner as used in the AASHTO Guide. Basically, reliability is simplified as a multiplier to the estimated ESALs in the design period.

Reference: Washington State DOT, EVERSERIES USER'S GUIDE Pavement Analysis Computer Software and Case Studies, 2005

Website: http://www.wsdot.wa.gov/Business/MaterialsLab/PavementGuide.htm

AASHTO DARWIN-ME

The mechanistic-empirical pavement design guide (MEPDG) was released in draft form at the conclusion of NCHRP 1-37A project in April, 2004. In 2011, DARWIN-ME was released as the next generation of AASHTOWare® pavement design software, which builds upon the MEPDG, and expands and improves the features in the accompanying prototype computational software.

In the MEPDG, structural responses (stresses, strains and deflections) are mechanistically calculated based on material properties, environmental conditions, and loading characteristics. These responses are used as inputs in empirical models to predict pavement performance. The accuracy of empirical models is a function of the quality of the input information and the calibration of empirical distress models to observed field performance. The distresses considered for flexible pavements are: rutting, (bottom-up fatigue cracking, longitudinal (top-down) cracking, thermal cracking, and roughness.

The MEPDG has a hierarchical approach for the design inputs, defined by the quality of data available and importance of the project, including:

Level 1 - Laboratory measured material properties are required (e.g., dynamic modulus for asphalt concrete, nonlinear resilient modulus for unbound materials). Project-specific traffic data is required (e.g., vehicle class and axle load distributions);

Level 2 - Inputs are obtained through empirical correlations with other parameters (e.g., resilient modulus estimated from CBR values) and state-wide traffic data;

Level 3 - Inputs are selected from a database of national or regional default values according to the material type or highway class (e.g., soil classification to determine the range of resilient modulus, highway class to determine vehicle class distribution).

The steps to design a pavement structure using DARWIN-ME are as follows:

1) Define a trial design for specific site subgrade support, material properties, traffic loading, and environmental conditions;

2) Determine design criteria and reliability level for acceptable pavement performance at the end of the design period (i.e., acceptable levels of rutting, cracking, and roughness);

3) Calculate monthly traffic loading and seasonal climate conditions (temperature gradients in AC layers, moisture content in unbound layers and subgrade) – *internal step*;

4) Compute structural responses (stresses, strains and deflections) for each axle type and load and for each time period throughout the design period using the material properties in response to environmental conditions – *internal step*;

5) Calculate predicted distresses (e.g., rutting, cracking, and roughness) throughout the design period using the calibrated empirical performance models – *internal step*;

6) Evaluate the predicted performance of the trial design against the specified reliability level. If the trial design does not meet the performance criteria, the design (thicknesses or material selection) must be modified and the calculations are repeated until the design is acceptable.

Reference: ARA, Inc., Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures, NCHRP 1-37A Final Report, TRB, Washington, D.C., 2004

Rutgers The State University of New Jersey

Center for Advanced Infrastructure and Transportation -CAIT

Pavement Resource Program

Using Subbase Soil Layer to Combat Frost Damage and Weak Subgrade Soils

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July, 2012

Introduction

This paper summarizes the use of Subbase layer or soil stabilization to mitigate or eliminate the effects of frost damage or weak subgrade soil on pavement performance.

The first section addresses the three conditions that **<u>must</u>** be present to cause frost heaving and associated frost damage problems:

- *source of water*
- *subfreezing temperatures in the soil (frost penetration) and*
- *the presence of frost-susceptible soils;*

The second section discusses the use of the Rutgers Soil Engineering Series to identify the locations of frost susceptible soils and weak subgrade soils.

The third section recommends the use of subbase layers or forms of soil stabilization to mitigate the effects of frost penetration and frost susceptible soil materials and weak subgrade soils in reducing pavement performance.

Frost Action and Damage

Effect of Frost Action on Pavement Performance

Frost action within or beneath the pavement can cause differential heaving, surface roughness and cracking, blocked drainage, a reduction in bearing capacity during thaw periods and softening of the granular base, subbase and subgrade soil layers. These effects range from slight to severe, depending on types and uniformity of subsoil, regional climatic conditions (i.e., depth of freeze), and the availability of water. The molar volume of water expands by about 9% as it changes phase from water to ice at its bulk freezing point.

One effect of frost action on pavements is frost heaving caused by crystallization of ice lenses in voids of soils containing fine particles. As shown below, three conditions **must** be present to cause frost heaving and associated frost action problems:

- ***source of water***
- ***subfreezing temperatures in the soil (frost penetration) and***
- ***the presence of frost-susceptible soils;***

The presence of frost-susceptible soil with a pore structure that promotes capillary flow is essential to delivering water to the ice lenses, as they form.

If these conditions occur uniformly, heaving will be uniform; otherwise, differential heaving will occur, causing surface irregularities, roughness, and ultimately cracking of the pavement surface. Figure 1 (*Yoder and Witczak – Principles of Pavement Design*) illustrates this phenomenon

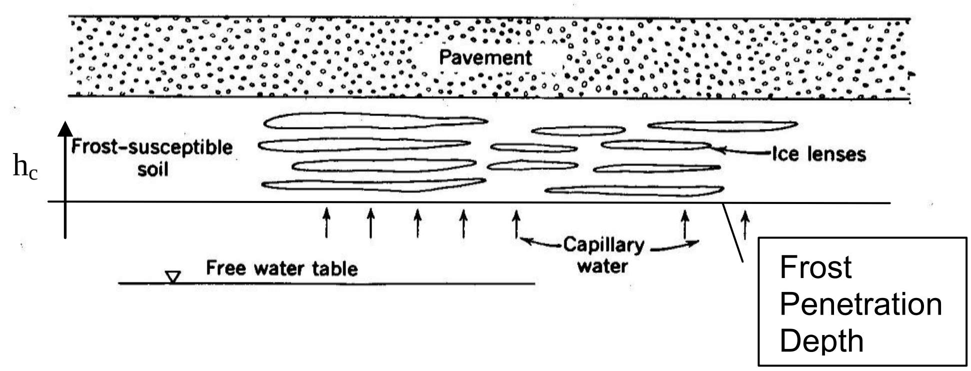


Figure 1. Mechanics of Frost Damage

A second effect of frost action is thaw weakening. The bearing capacity may be reduced substantially during mid-winter thawing periods, and subsequent frost heaving is usually more severe because water is more readily available to the freezing zone. In more-southerly areas of the frost zone, several cycles of freeze and thaw may occur during a winter season and cause more damage than one longer period of freezing in more-northerly areas. Spring thaws normally produce a loss of bearing capacity to well below summer and fall values, followed by a gradual recovery over a period of weeks or months. Water is often trapped above frozen soil during the thaw, which occurs from the top down, creating the potential for long-term saturated conditions in pavement layers. The ice lenses and thaw weakening can also loosen the aggregate base, subbase, and subgrade materials causing permanent reduction in the bearing capacity of the soil aggregate. Figure 2. (*Jumikis – The Frost Penetration Problem in Highway Engineering*) provides an illustration of the frozen and thawed zones in the pavement structure and surface sources of water to promote ice lens formation.

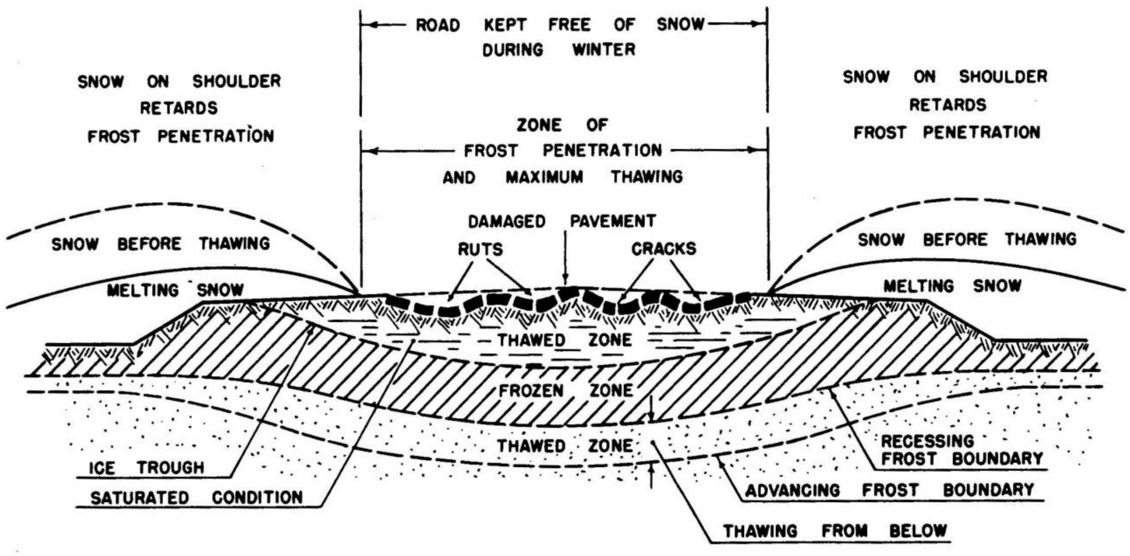


Figure 2. Frozen and Thawed Zones within and below the Pavement Structure

Sources of Water

The greatest potential of frost heave and ice lenses formation exists when the ground water table is relatively close to the surface or close to the freezing horizon within or below the pavement structure. The ice lenses will grow rapidly if the soil is subject to high thermo-osmosis capillary potential. Figure 3 illustrates Ice lens formation due to thermo-osmosis which is the process of moisture migration due to thermal potential (e.g., thermal gradient).

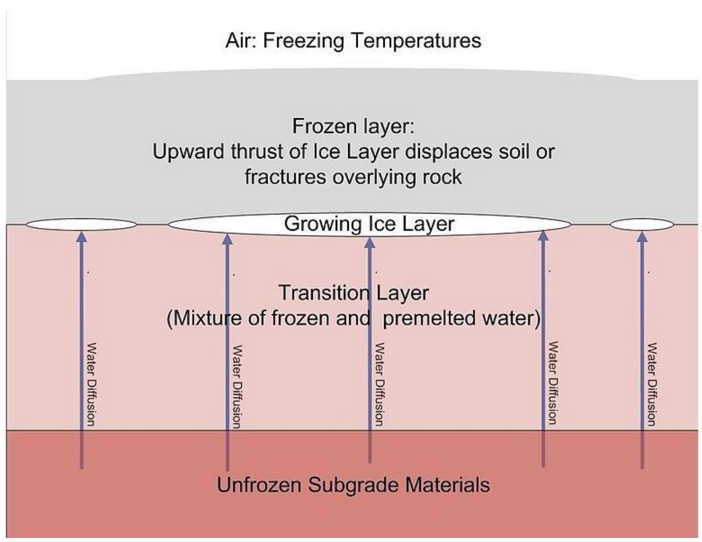


Figure 3. Frozen, Transition and Unfrozen Layers

The height of capillary rise can be estimated as

$$h_c = \frac{C}{eD_{10}}$$

Where:

h_c = height of capillary rise
C = Constant (0.1 to 0.5 cm^2)
e= void ratio = volume of the voids/volume of the solids = n/(1-n) [n= porosity]
D_{10}= Hazen effective size of the particles with 10% passing (in cm).

The critical height of capillary rise varies inversely with the D10 size of the soil. Figure 4 and 5 illustrate the particle size for D_{10}.

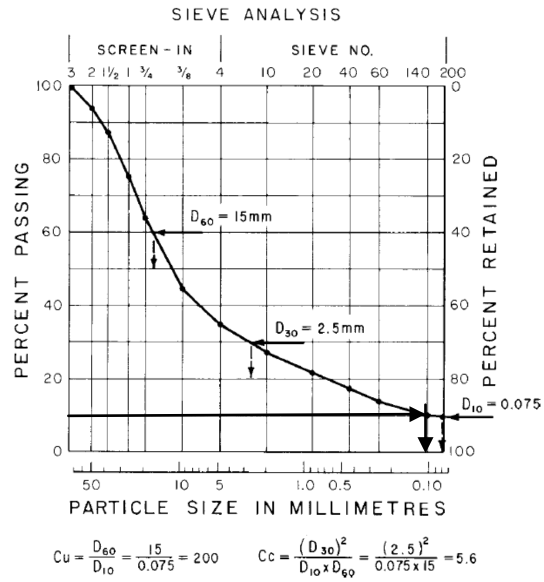


Figure 4. Particle Size Distribution

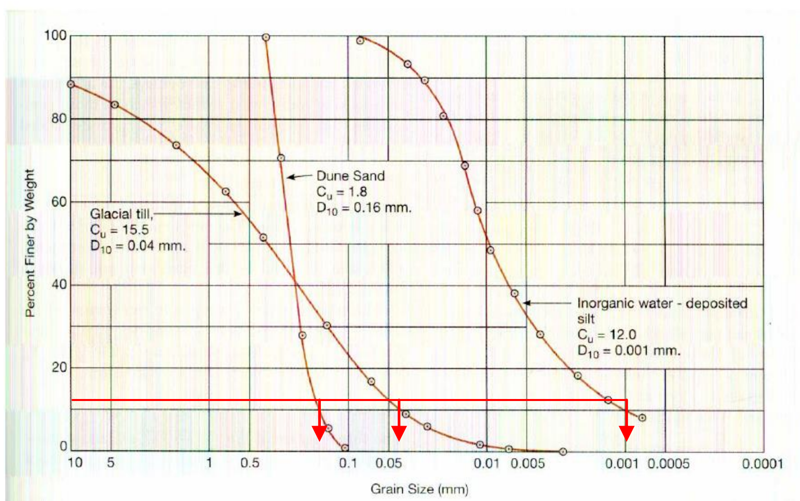


Figure 5. Example of D_{10} size for Gravel, Sand and Silt Soil Aggregates

Although many fine sands are potentially frost-susceptible, the height of capillary rise may be so low as to minimize or completely stop frost action. Tables 1 and 2 provide a range of void ratio and capillary rise values typically provided in the literature.

Table 1. Summary of the Porosity and Void Ratios of Typical Soil Aggregates

Soil Type	Porosity	Void Ratio
Gravel	0.25-0.4	0.33-0.66
Sand	0.25-0.5	0.33-1.0
Silt	0.35-0.5	0.54-1.0
Clay	0.4-0.7	0.66-2.33

Table 2. Estimates of Capillary Rise of Typical Soil Aggregates

Soil Type	Capillary Rise (Inches)
Coarse Sand	3/4 - 2
Sand	4 - 14
Fine Sand	14 - 27
Silt	27 - 59
Clay	78 - 160+

(Gruszczenski – Determination of Realistic Estimate of the Actual Formation of Product Thickness using Monitoring Wells)

Surface infiltration, particularly at the pavement edge, is another potential source of water for frost heaving. However, when freezing starts and a layer of ice exists below the pavement, the water supply will be cut off by the ice layer itself. Nevertheless, adequate surface drainage should be recognized as a prerequisite to the design against damage due to frost action. Additional water may also be available from the pavement edges especially in cuts to feed the ice lenses formation.

Frost Penetration

FROST HEAVE

The term frost heave refers to a raising of a portion of the pavement as a direct result of the formation of ice crystals in a frost-susceptible subgrade or base course. The mechanics of the frost-heaving phenomenon are extremely complex and include many factors. The water will have a strong affinity to the ice lenes with a result that water is continuously drawn to the ice crystals that are initially formed. In addition, if the soil is highly susceptible to capillary action, ice crystals will continue to grow until ice lenses begin to form; the lenses in turn grow until frost heaving results.

Estimated Frost Penetration

Over the years, the National Oceanic and Atmospheric Administration (NOAA) have developed and published climatic maps containing historical freezing index and frost penetration values, as well as the number of freeze-thaw cycles in the form of contour maps. The freezing index is defined as the cumulative number of degree-days when air temperatures are below and above 32 degrees ^{o}F. A pavement that is at 31 ^{o}F for 10 days or at 22 ^{o}F for 1 day have a cumulative freezing index of 10 degree-days. A cumulative plot of degree days versus time results in a curve such as shown in Figure 6. Since the data are accumulated, it is not necessary to begin the plot on any particular day, but, rather, the plot can be started on any convenient date. The difference between the maximum and minimum points on the cumulative degree-day plot has been termed the freezing index. The freezing index, in turn, has been correlated with depth of frost penetration.

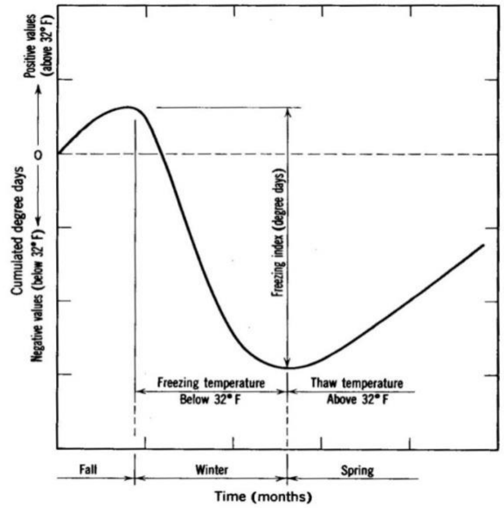


Figure 6. Cumulative Plot of Degree-Days and Frost Index

Figure7 provides a map of the cumulative freezing index for the United States.
New Jersey primarily has freezing index between 0 and 1000 with a small portion between 1000 and 2000.

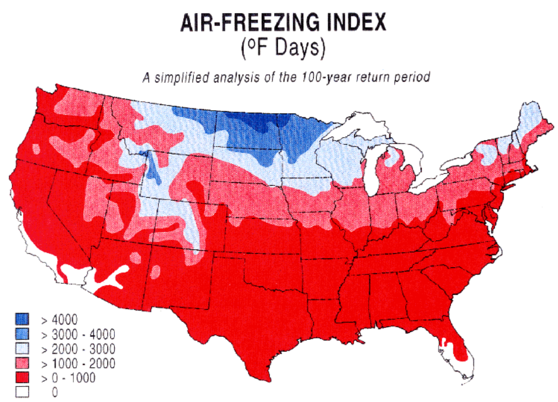


Figure 7. Map of the cumulative freezing index for the United States
Using Air freezing index to estimate Pavement Freezing index

The Corps of Engineers has determined an empirical curve which relates depth of frost penetration to freezing index for a well-drained, non-frost-susceptible base course. These data can be used to estimate depth of penetration under pavement kept free of snow and ice. Figure 8 can be used to estimate the frost penetration depth based on the air freezing index.

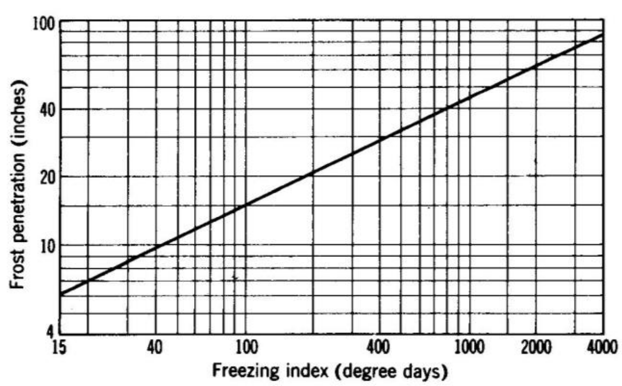


Figure 8. Estimation of Frost Penetration based on cumulative freezing index for the United States (Corps of Engineers)

Figure 9. provides estimated Frost Penetration Depths in the United States. NJ has estimated Frost Penetration Depths between 20 and 50 inch.

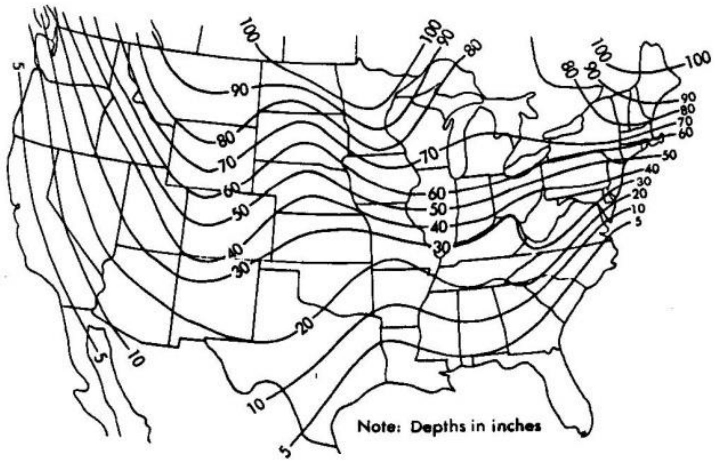


Figure 9. Estimate of Frost Penetration Depths in the United States.

More detailed freezing index for various part of New Jersey can be obtained from the National Oceanic and Atmospheric Administration (NOAA) website. Table 3 provides a listing of locations throughout New Jersey. Based on this data, NJ has a minimum freezing index of 415 degree-days and maximum of 1345 degree-days.

Table 3. Air Freezing Index of NJ Locations

Air Freezing Index- USA Method (base 32^{o} Fahrenheit)							
Air Freezing Index Return Periods (^{o}F-Days) & Associated Probabilities (%)							
	State and Station Name	**Station Number**	**Lat. (Deg. - Min.)**	**Long. (Deg. - Min.)**	**Elev. (feet)**	**Mean Annual Temp. (^{o} F)**	**100 Year (99%)**
New Jersey							
	CHARLOTTEBURG	281582	N4102	W07426	760	48.1	1086
	ESSEX FELLS SERV BLDG	282768	N4050	W07417	340	50.8	909
	FLEMINGTON 1 NE	283029	N4031	W07451	140	51.0	896
	FREEHOLD	283181	N4016	W07415	194	52.7	646
	GLASSBORO	283291	N3942	W07507	135	53.9	557
	HAMMONTON 2 NNE	283662	N3939	W07448	85	53.9	544
	HIGHTSTOWN 1 N	283951	N4017	W07431	100	52.8	641
	INDIAN MILLS 2 W	284229	N3948	W07447	100	52.8	580
	JERSEY CITY	284339	N4044	W07403	135	52.7	618
	LAMBERTVILLE	284635	N4022	W07457	60	53.2	640
	LITTLE FALLS	284887	N4053	W07414	150	52.4	686
	LONG BRANCH 2 S	284987	N4019	W07401	15	52.9	539
	LONG VALLEY	285003	N4047	W07447	550	49.0	1053
	MILLVILLE FAA AIRPORT	285581	N3922	W07504	68	54.0	506
	MOORESTOWN	285728	N3958	W07458	55	53.2	564
	MORRIS PLAINS 1 W	285769	N4050	W07430	400	50.3	922
	NEWARK WSO	286026	N4042	W07410	11	54.2	533
	NEWTON	286177	N4103	W07445	565	48.4	1230
	PEMBERTON 3 E	286843	N3958	W07438	80	53.3	571
	PLAINFIELD	287079	N4036	W07424	90	52.9	606
	SHILOH	288051	N3928	W07518	120	54.6	415
	SOMERVILLE 3 NW	288194	N4036	W07438	160	51.7	873
	SUSSEX 1 SE	288644	N4112	W07436	390	48.1	1345
	TRENTON WSO	288883	N4013	W07446	56	54.0	484
						min	415
						max	1345

Heat Flow Through the Pavement Structure and Subgrade Soils

Foremost among the factors affecting soil temperature are source and amount of heat given to (or leaving) the soil. The primary source of heat is radiation of the sun. Heat transferred to the soil by conduction is comparatively less. Latitude of the location has an important bearing on the amount of heat absorbed per unit area of surface. Other factors such as dust and water vapor in the atmosphere will also affect the quantity of heat that is absorbed by the soil. Soil freezing depends to a large extent upon the duration of depressed air temperatures.

Heat transferred from the soil to the atmosphere must pass through the pavement. The effect of type of cover in regard to both quantity and color has been known for some time. Frost penetration is deeper and its disappearance faster under bare ground than under grass cover since grass acts as an insulating layer to the soil. The temperature of soil under dark objects, such as a flexible pavement, is higher than the natural soil, whereas that under white objects is lower. Unless the air temperatures are very low, the depth of freezing under snow cover is quite limited. Because of these limitations, correlations that have been established between freezing index and depth of frost penetration must be used with some degree of caution.

The heat conduction through the pavement structure and subgrade soils can be expressed by

$$Q = KiAT = K\left(\frac{t_1 - t_2}{x}\right)AT$$

where
Q = quantity of heat flow
t_1, t_2 are the temperatures at elevation 1 and 2 in the pavement structure
i = thermal gradient $(t_1-t_2)/x$< where x is thickness in feet
A= pavement surface area, ft^2
K=thermal conductivity (BTU per ft^2 per hour per degree ^oF per foot)
T = time

Figure 10 provides an illustration of the net heat flow at the pavement surface that varies with material, cloud cover, surface (grass, soil, pavement material, etc.).

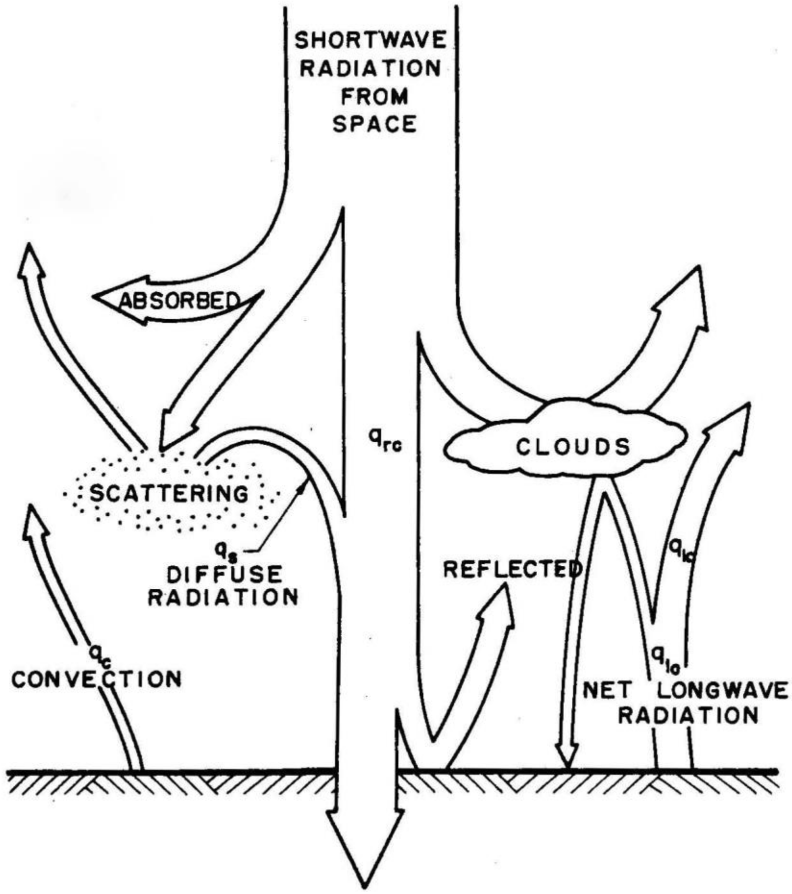


Figure 10. Illustration of the net heat flow at the pavement surface

The following outlines more precise calculations of frost penetration depths. Knowledge of the physical and thermal properties of the pavement materials and subgrade materials are necessary.

Depth of Frost Penetration

The performance of pavements in frost-affected regions depends to a large degree on the depth of frost penetration. Prediction of the maximum depth of frost can be accomplished in several ways, including correlation of field penetration data with temperature data and theoretical formulas and charts.

Several formulas have been presented for predicting depth of frost penetration. The first, known as the Stefan equation, is derived by equating the fundamental equations of heat flow and storage. While Stefan's equation provided reasonable estimates in northern climates like Canada, it has been shown to over predict frost penetration depths in temperate zone like NJ. The modified Berggren equation uses Stefan's formula and adds a correction factor to address the latent heat omitted in the Stefan's equation. The equation, presented below, is based on the assumption that the only heat flow that need be considered is that represented by the latent heat of fusion of the soil water; and time T is converted to days.

$$z = \lambda \sqrt{\frac{48\,K\,F}{L}}$$

where Z = depth of penetration in a homogeneous mass (ft)

λ= adjustment factor

K= thermal conductivity (Btu's per square foot per degree Fahrenheit, per foot, per hour)

F= degree-days

L= volumetric heat of latent fusion (BTU per ft^3)

Example

The following example problem explains the thermal terms and the use of the chart in estimating the Frost Penetration Depth in pavement and soil layers

Step 1 - Determine the pavement freezing index, F

This can be determined for graphic illustrations or tables from NOAA.
For this example problem, we will use the average for NJ (727 degree-days)

Step 2 – Determine the duration of the freezing period, t, in days, and the mean annual air temperature.

Figure 11 provides an illustration of the freezing period, t, in days for NJ.

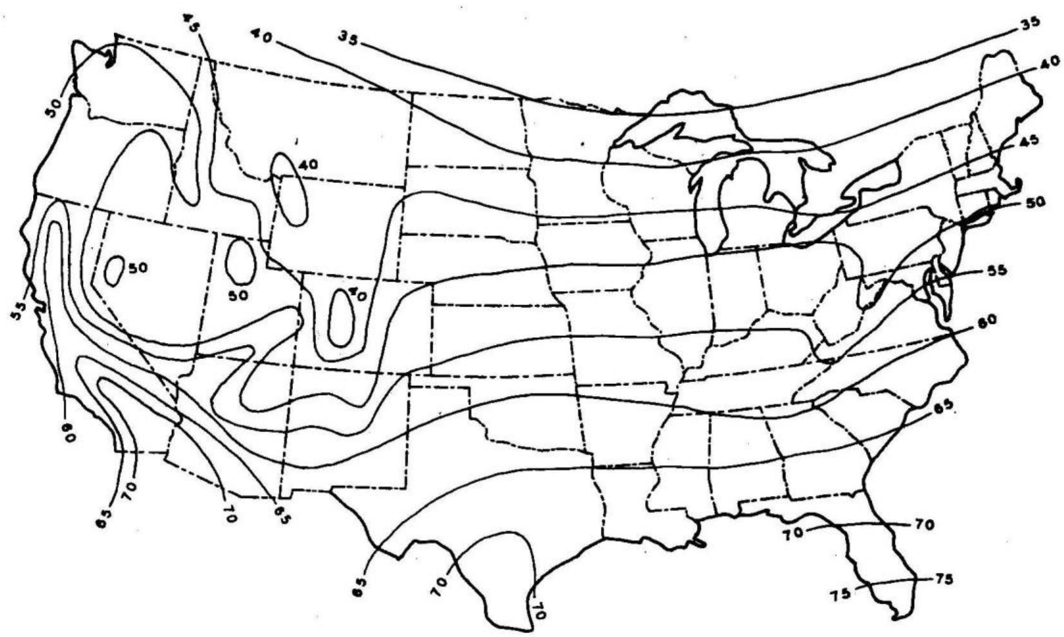


Figure 11. provides an illustration of the mean annual air temperature for NJ.

The mean annual air temperature for NJ is 52.5 ^{o}F.

Vo= mean annual air temperature minus 32 ^{o}F.
Vo= 52.5-32= 20.52 ^{o}F

Figure 12 illustrates the duration of the freezing period, t, in days for different parts of NJ.

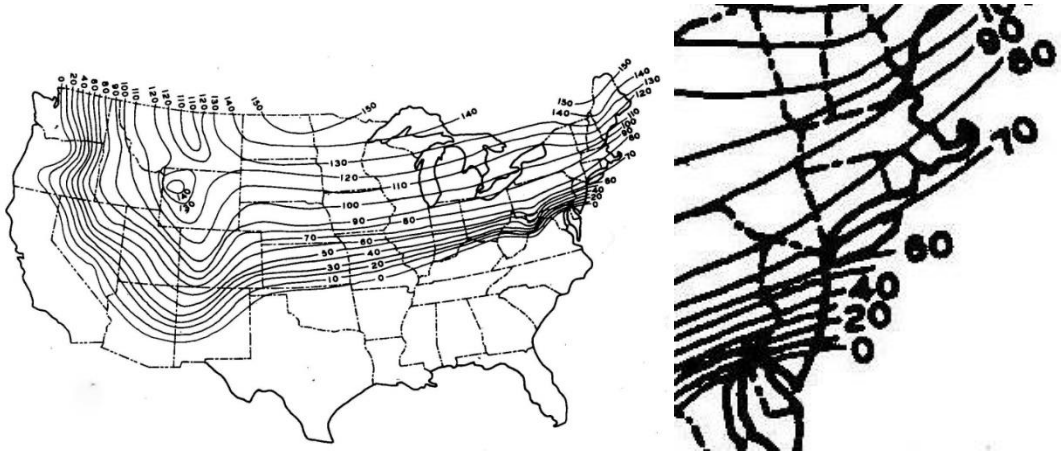


Figure 12. Map of the freezing period, t, in days for NJ.

Step 3 Determine the Thermal properties for the pavement and subgrade materials.

The physical characteristics of the soil itself determine to a large extent its ability to conduct and absorb heat, and, therefore, behavior of soils under depressed air temperatures are variable. Rate of heat transfer depends upon soil moisture content, density and many other factors.

Table 4 provides a summary of the thermal properties that are pertinent in heat transfer problems in soils.

Table 4 Summary of the thermal properties that are pertinent in heat transfer problems in soils.

Symbol	Term	Units or Equation	Typical Values
k	Coefficient of thermal conductivity	Btu per hr per ft per deg °F	Asphalt concrete = 0.84 Portland cement concrete = 0.54
c	Specific heat	Btu per lb per °F	Water = 1.0 Ice = 0.5 Soil minerals = 0.17
C	Volumetric heat	Btu per ft^3 per °F	Asphalt concrete = 21 PCC = 28
L	Latent heat	Btu per ft^3	One pound of water yields 143.4 Btu on freezing

The dry density and moisture content of the pavement materials in Table 5 are based on NJDOT pavement materials research for natural gradations provided in Table 6.

Table 5 Pavement Structure and Material Properties

thickness	Materials	Dry density, γ_d	Moisture content, ω
3 inch	Bit Concrete	--	--
6 inch	Agg Base Course	141	4
21.5 inch	Subbase	130	4
	Subgrade	108	18

Table 6. NJDOT Material Properties for DGABC and Subbase Materials and Typical Values of Maximum Density and Optimum Moisture for Common Subgrade Types of Soil (using AASHTO T 99)

Region of NJ	Soil Gradation Type	NJDOT I-3		DGABC	
		γ_d (pcf)	w (%)	γ_d (pcf)	w (%)
North Region	Natural Gradation	131	4	141	4
	High End	138	3.5	127.3	4.1
	Middle Range	131	4	143.9	6
	Low End	114	6	140.9	7.6
Central Region	Natural Gradation	112.5	4	136.5	6.4
	High End	134	4.75	129.1	4.2
	Middle Range	129	6.5	144.3	7.3
	Low End	115	8	141.1	8.5
South Region	Natural Gradation	106	6	N.A.	N.A.
	High End	120.5	3		
	Middle Range	120.5	6		
	Low End	110	10		

Unified Soil	Soil Description	Range of Max. Densities kg/m^3 (lbs/ft^3)	Range of Optimum Moisture (%)
CH	Highly Plastic Clays	1200-1680 (75-105)	19-36
CL	Silty Clays	1520-1920 (95-120)	12-24
ML	Silts and Clayey Silts	1520-1920 (95-120)	12-24
SC	Clayey Sands	1680-2000 (105-125)	11-19
SM	Silty Sands	1760-2000 (110-125)	11-16
SP	Poorly-graded Sands	1600-1920 (100-120)	12-21
SW	Well-graded Sands	1760-2080 (110-130)	9-16
GC	Clayey Gravel w/ sands	1840-2080 (115-130)	9-14
GP	Poorly-graded gravels	1840-2000 (115-125)	11-14
GW	Well-graded Gravels	2000-2160 (125-135)	8-11

The thermal conductivities, k, is based on the unfrozen and frozen material properties from Figure 13. The dry density and moisture content of the soil properties in tables 6 and 7 were used to determine the thermal conductivity values for each layer.

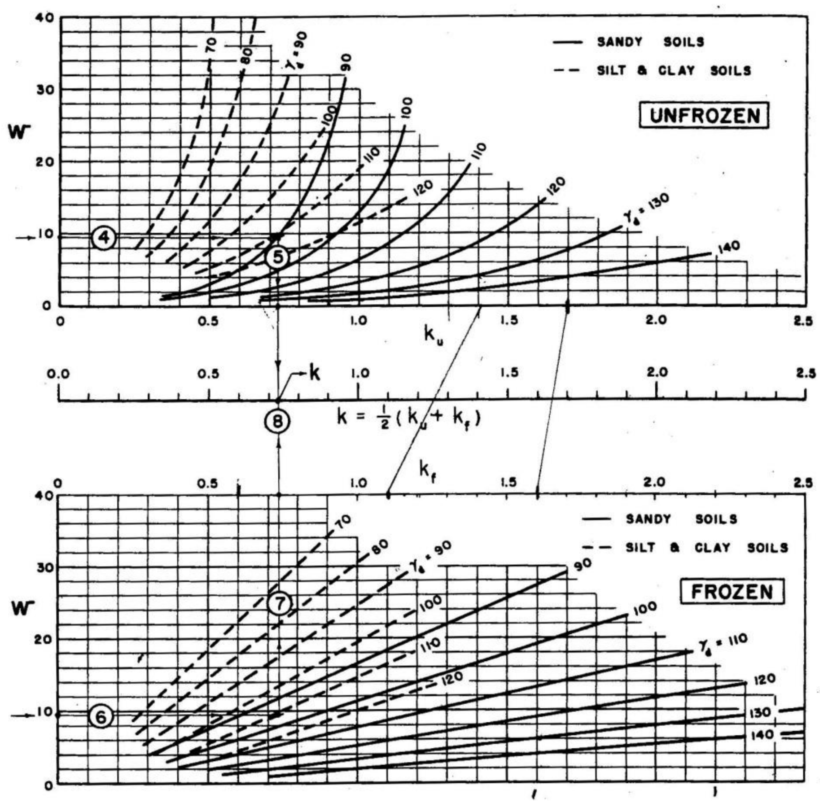


Figure 13. Thermal Conductivity of Soil Materials

The overall k = $\frac{k_u+k_f}{2}$

The Volumetric Heat, C is based on the following two equations:

C_u= $\gamma_d(0.17 + \frac{\omega}{100})$

C_f= $\gamma_d(0.17 + \frac{0.5*\omega}{100})$

The overall Volumetric Heat, C = $\frac{C_u+C_f}{2}$

The Latent Heat, L= $1.434 * \omega * \gamma_d$

Table 7 provides a summary of the Material and Thermal Properties of Pavement and Subgrade Soils, and the Frost Penetration calculation.

Table 7. Summary of the Material and Thermal Properties of Pavement and Subgrade Soils, and the Frost Penetration calculation.

mean annual temp	Est surface Frost Index	Est Z, inch	Est Z, ft	Thickness, inch	thickness, ft	Material	dry density, pcf	Moisture content, percent	ku	kf	k	Cu	Cf	C	L
52.5	727	35	2.92	3	0.25	Bituminous concrete			0.84	0.84	0.84			28	0
v_o				6	0.50	Aggregate Base	141	4	1.7	1.6	1.65	29.61	26.79	28.2	808.78
20.5				21.5	1.79	Subbse	131	4	1.4	1.15	1.275	27.51	24.89	26.2	751.42
t			0.38		0.38	Subgrade	108	18	0.9	1.1	1	37.8	28.08	32.94	2787.70
70				Est Frost Depth	2.92										
L/k (eff)=	0.235102041		d1	d2	d3	d4	total								
		0.297619048	0	404.388	1346.287	1045.386	2796.061	832.1610119							
		0.303030303		202.194	1346.287	1045.386	2593.867	786.020303							
		1.405228758			673.1435	1045.386	1718.5295	2414.927075							
		0.375				522.693	522.693	196.009875							
								4229.118265							
							L/k (eff)=	994.274335							
					total										
Cwt=	7.00	14.10	46.94	12.35	80.3941667	28									
Lwt=	0.00	404.39	1346.29	1045.39	2796.06	959									
α=	1.974														
μ=	0.299														
λ=	0.58	fig 5													
Z=	3.4	ft													
	41	inch													

Step 4 Compute an effective $(\frac{L}{k})eff$

$$\left(\frac{L}{k}\right)eff = \frac{2}{Z^2}[\frac{d_1}{k_1}(\frac{L_1 d_1}{2} + L_2 d_2 + L_3 d_3 + L_4 d_4)$$
$$+\frac{d_2}{k_2}\left(\frac{L_2 d_2}{2} + L_3 d_3 + L_4 d_4\right)$$
$$+\frac{d_3}{k_3}\left(\frac{L_3 d_3}{2} + L_4 d_4\right)$$
$$+\frac{d_4}{k_4}\left(\frac{L_4 d_4}{2}\right)]$$

Z=Estimated Frost Penetration Depth = $d_1+d_2+d_3+d_4$

From the Average Freezing Index (727 degree-days) and Figure 8, Estimated Frost Penetration Depth = 2.92 ft (35 inch).

Material Layer	Thickness
Bituminous Concrete	d_1 = 0.25 ft
Base	d2 = 0.5 ft
Subbase	d3 = 1.79
Subgrade	d4 = Z- ($d_1+d_2+d_3$) = 0.38 ft
Estimate Frost Penetration	2.92 ft

$$\left(\frac{L}{k}\right)eff = \frac{2}{2.92^2}[\frac{0.25}{0.84}(\frac{(0)(0.25)}{2} + (808.78)(0.5) + (751.42)(1.79) + (2787.7)(0.38))$$
$$+\frac{0.5}{1.65}\left(\frac{808.78}{2} + (751.42)(1.79) + (2787.7)(0.38)\right)$$
$$+\frac{1.79}{1.275}\left(\frac{(751.42)(1.79)}{2} + (2787.7)(0.38)\right)$$
$$+\frac{0.38}{1}\left(\frac{(2787.7)(0.38)}{2}\right)]$$

$$\left(\frac{L}{k}\right)eff = 994.27$$

Step 5 Compute weighted values of C and L within estimated depth of frost penetration for multiple layer system

$C_{wt} = \frac{C_1 d_1 + C_2 d_2 + C_3 d_3 + C_4 d_4}{Z} = \frac{(28)(0.25)+(28.2)(0.5)+(26.2)(1.79)+32.94(0.38)}{2.92} = 28$

$L_{wt} = \frac{L_1 d_1 + L_2 d_2 + L_3 d_3 + L_4 d_4}{Z} = \frac{(0)(0.25)+(808.8)(0.5)+(751.4)(1.79)+(2787.7)(0.38)}{2.92} = 959$

Step 6 Compute the effective values of α and μ from the following equations

$\alpha = \frac{v_o}{F} t = \frac{20.5}{727} 70 = 1.974$

$\mu = \frac{Cwt\ F}{Lwt\ t} = \frac{(28)(727)}{(959)(70)} = 0.3$

Step 7 Determine the Correction Coefficient λ from Figure 14

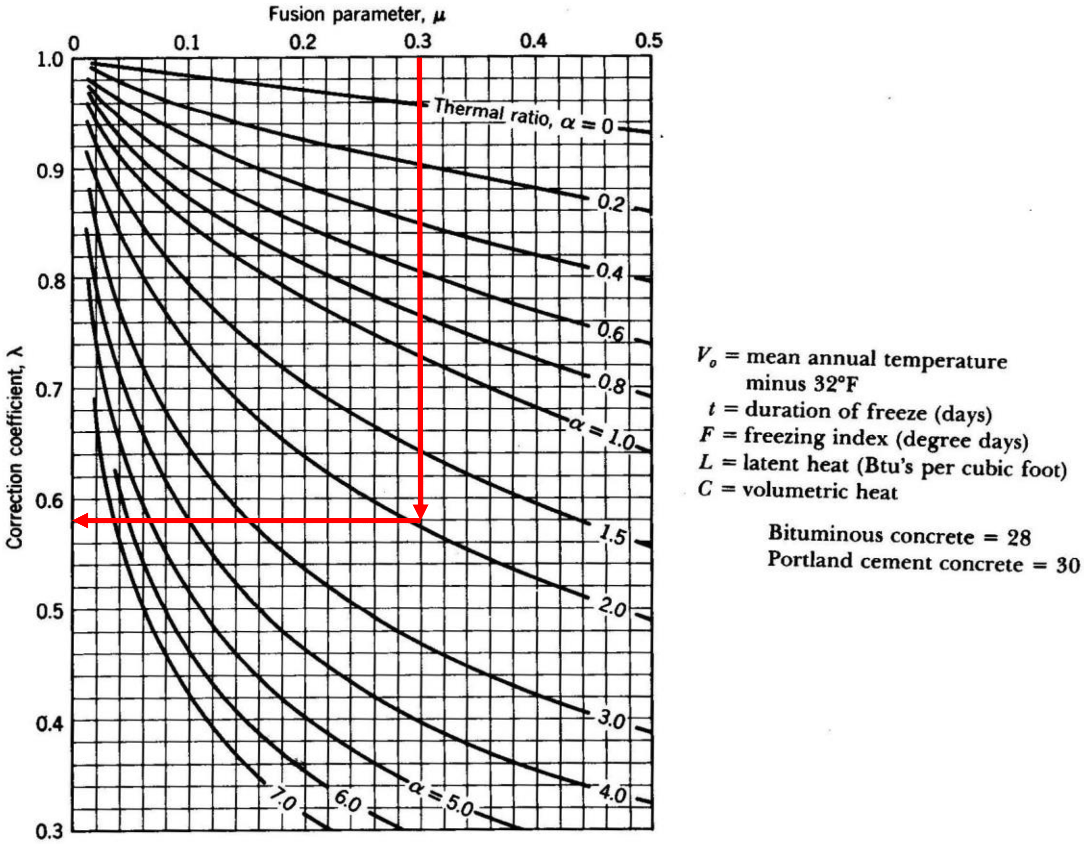


Figure 14. Correction Coefficient for the modified Berggren formula (from Aldrich, Highway Research Board Bulletin 135 Frost Penetration Below Highway and Airfield Pavements

Based on the α and μ values, λ value from Figure 14 equals 0.58.

Step 8 Compute the depth of Frost Penetration

$$z = \lambda \sqrt{\frac{48\ F}{\frac{L}{k}eff}} = 0.58\sqrt{\frac{48\ (727)}{994.27}} = 3.4\ ft = 41\ inch$$

Frost-Susceptible Soils

Results of studies made by the Corps of Engineers have indicated that frost-susceptible soils include all inorganic soils that contain greater than 3 percent by weight particles finer than 0.02 millimeter. Frost-susceptible soils have further been placed into several categories according to degree of susceptibility (Table 8 and Figure 15). The F1 materials are the least susceptible to frost action and are all gravelly soils with between 3 and 20 percent finer then 0.02 millimeter. The F2 materials include the sands with between 3 and 15 percent finer than 0.02 millimeter, and F3 group includes gravelly and sandy soils not included in F1 and F2 and clays with plasticity indices of more than 12, whereas the F4 group includes all silts, silty sands, lean clays, and most varved clays.

Frost heaving requires a frost-susceptible soil, a continual supply of water below (a water table) and freezing temperatures, penetrating into the soil. Frost-susceptible soils are those with pore sizes between particles and particle surface area that promote capillary flow. Silty and loamy soil types, which contain fine particles, are examples of frost-susceptible soils. Many agencies classify materials as being frost susceptible if 10 percent or more constituent particles pass through a 0.075 mm (No. 200) sieve or 3 percent or more pass through a 0.02 mm (No. 635) sieve.

Non-frost-susceptible soils may be too dense to promote water flow (low hydraulic conductivity) or too open in porosity to promote capillary flow. Examples include dense clays with a small pore size and therefore a low hydraulic conductivity and clean sands and gravels, which contain small amounts of fine particles and whose pore sizes are too open to promote capillary flow. Little to no frost action occurs in clean, free draining sands, gravels, crushed rock, and similar granular materials, under normal freezing conditions. The large void space permits water to freeze in-place without segregation into ice lenses. Conversely, silts are highly frost susceptible.

The condition of relatively small voids, high capillary potential/action, and relatively good permeability of these soils accounts for this characteristic.

TABLE 8. Frost-susceptible Soils'(NCHRP 1-37A)

Frost Group	Degree of Frost Susceptibility	Type of Soil	Percentage Finer that 0.75mm (#200) by wt	Typical Soil Classification	AASHTO Classification
F1	Negligible to low	Gravely Soils	3-10	GC, GP, GC-GM,GP-GM	A-1-b
F2	Low to medium	Gravelly Soils	10-20	GM, GC-GM, GP-GM	A-3
		Sands	3-15	SW, SP, SM, SW-SM, SP-SM	
F3	High	Gravelly Soils	Greater than 20	GM-GC	A-2, A-6, A-7
		Sands, except very fine silty sands	Greater than 15	SM, SC	
		Clays PI > 12	—	CL, CH	
F4	Very High	Very Fine Silty Sands	Greater than 15	SM	A-4, A-5
		Clays PI<12	—	CL, CL-ML	

		Varved Clays and Other Fine Grained, Banded Sediments	—	CL, ML, SM, CH	

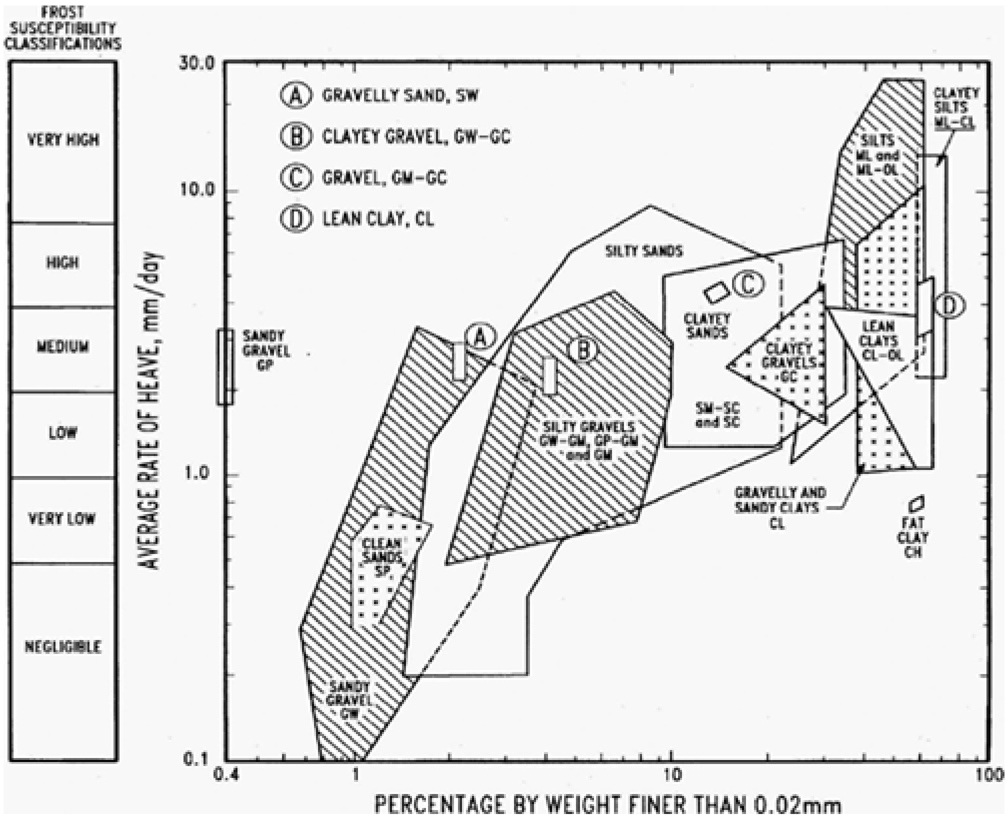


Figure 15. Graphical Identification of Frost Susceptible Soils

In general, the degree of frost susceptibility can be explained by two hydraulic properties of soils:

Capillarity — the soil's ability to pull moisture by capillary forces. The smaller the pore size distribution of a pore network, the greater the driving force (capillary action)
and the greater the capillarity.

Permeability — the soil's ability to transmit water through its voids. The permeability of any material is heavily dependent on the connectivity of its pore network. For example, if a material contains many tortuous pores that abruptly end, it will have less permeability than a material with very open pores that pass completely and directly through the material. The more connected and the larger the pore network is, the greater the permeability.

The relation of these properties to frost susceptibility is visualized in Figure 16.

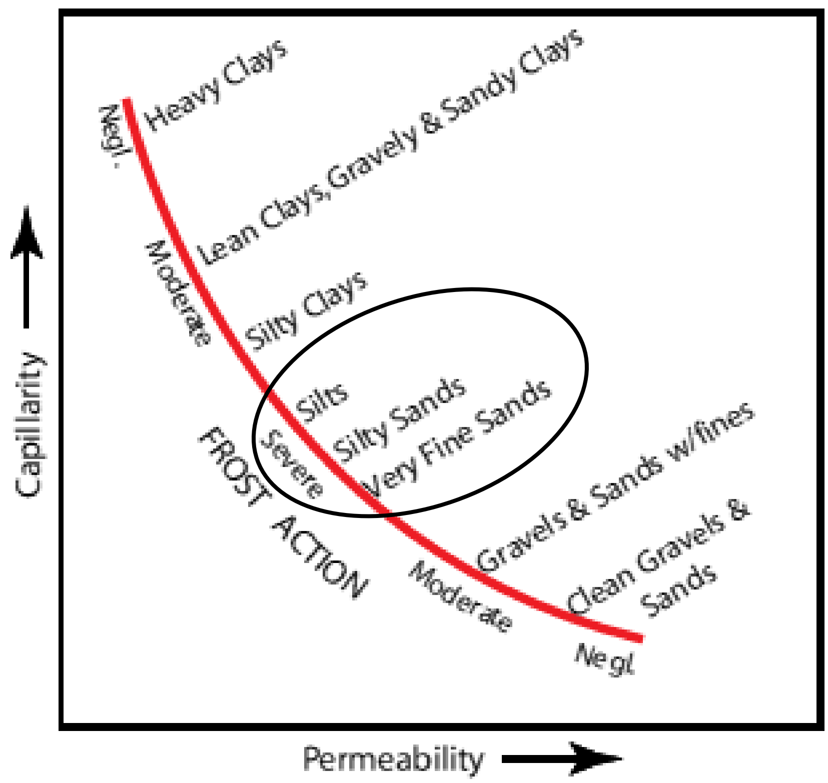


Figure 16. Relationship between Frost Action and Hydraulic Soil Properties

Clays are cohesive and, although their potential capillary action is high, their capillary rate is low. Although frost heaving can occur in clay soils, it is not as severe as for silts, since the impervious nature of the clays makes passage of water slow. The supporting capacity of clays must be reduced greatly during thaws, even in the absence of significant heave.

Thawing usually takes place from the top downward (solar energy) and bottom up (geothermal energy), leading to very high moisture contents in the upper strata above the frost zone.

A groundwater level within 1.5 m (5 ft) of the proposed subgrade elevation is an indication that sufficient water will exist for ice formation (perched groundwater level). Homogeneous clay subgrade soils also contain sufficient moisture for ice formation, even with depth to groundwater in excess of 3 m (10 ft). However, the magnitude of influence will be highly dependent on the depth of the freezing front (*i.e.,* frost depth penetration). For deep frost penetration, groundwater at even a greater depth could have an influence on heave.

As stated initially, in order to have frost damage, three conditions **must** be present to cause frost heaving and associated frost action problems:

- *source of water*
- *subfreezing temperatures in the soil (frost penetration) and*
- *the presence of frost-susceptible soils;*

The most distinguishing factor for identifying a pavement frost hazard condition is water supply. Since the depth of the water table varies, and the frost penetration depth varies from year to year; the frost susceptible nature and the related capillary action of the subgrade soil materials are the only constants that can contribute to the frost damage under the pavement section.

The conditions associated with a <u>high frost hazard potential</u> include
1. A water table within 3 m (10 ft) of the pavement surface (depth of influence depends on the type of soil and frost depth).
2. Observed frost heaves in the area.
3. Inorganic soils containing more than 3% (by weight) or more grains finer than 0.02 mm (0.8 mils) in diameter according to the U.S. Army Corps of Engineers.
4. A potential for the ponding of surface water. The occurrence of soils between the frost zone within or beneath the pavement with permeabilities high enough to enable seepage to saturate soils within the frost zone during the term of ponding.

The conditions associated with a <u>low frost hazard potential</u> include
1. A water table greater than 6 m (20 ft) below the pavement surface (again, could be much shallower depending on the type of soil and frost depth).
2. Natural moisture content in the frost zone low versus the saturation level.
3. Seepage barriers between the water supply and the frost zone.
4. Existing pavements or sidewalks in the vicinity with similar soil and water supply conditions and without constructed frost protection measures that have experienced frost damage.
5. Pavements on embankments with surfaces more than 3 – 6 feet above adjacent grades (provides some insulation and a weighting action to resist heave).

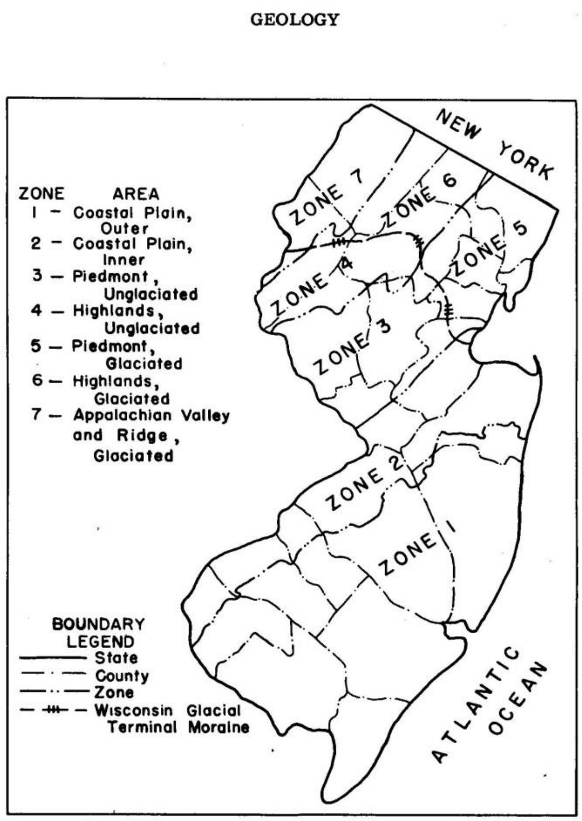
Location of Frost Susceptible Soils and Weak Subgrade Materials in New Jersey

Rutgers Engineering Soil Survey Series

The Rutgers Engineering Soil Survey Series consists of 22 reports that detail the soil types, properties, and locations throughout New Jersey. Report 1 summarizes the soil environment and methods used to identify the soil zones or polygons. Reports 2 through 21 provides details on the soil types and engineering properties found in each of New Jersey's 21 counties. Report 22 provides soil summaries and location of soils in county reports as well as description of the nomenclature used to symbolically identify the soils. Figure 17 illustrates the county report numbers.

Report 1 Soil Environment and Method of Research

State Geology Zones
New Jersey consists of seven geological regions illustrated in Figure 17.



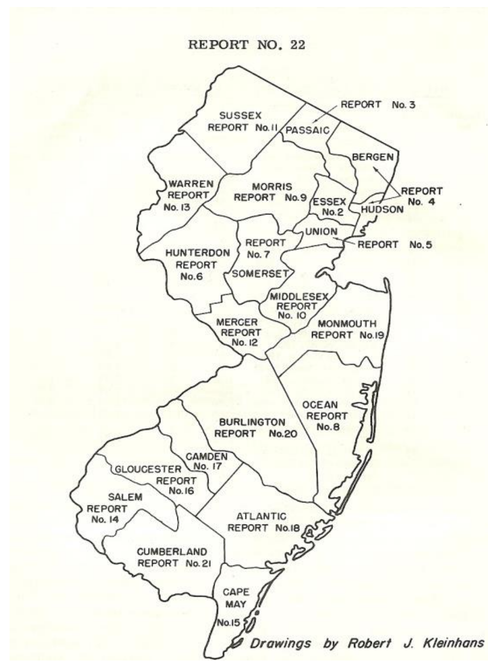


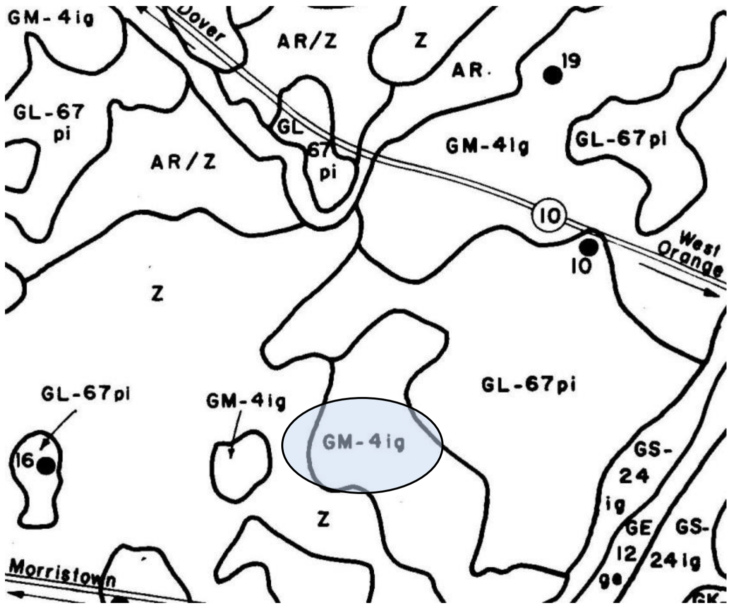
Figure 17. New Jersey's Geologic Zones and County Report Numbers

Soil Maps

The soil maps use the following notation to identify the soil types and provide input to the soil engineering properties.

EXPLANATION OF SOIL MAP IDENTIFICATION Symbols

The soil nomenclature used on the map is a shorthand method developed to explain the soil geology, AAHSTO soil engineering properties, drainage condition, and special symbols to identify unique conditions. The shorthand has four parts, the Geologic symbol, the AAASHTO Classification Range, Drainage Conditions, and Special situations.



<u>Geologic</u> – <u>Textural</u> (AASHTO Classification Range) <u>DRAINAGE</u> <u>SPECIAL</u>

Example GM-4 ig

The line width separating the soil polygons has an accuracy of Map Details (500ft). It represents a transition between soil materials.

GEOLOGIC SYMBOLS — The first part of the soil code designates the type of geologic formation on which the soil occurs. Within any specific climatic zone the geologic designation, in addition to defining the nature of the underlying formation, establishes the probable land form and strongly implies surface drainage characteristics. The letter symbol*; for geologic formations and types are charted in Figure 18. Explanations and definitions of the symbols appear in Figure 18 and Table 9 Geologic Notation.

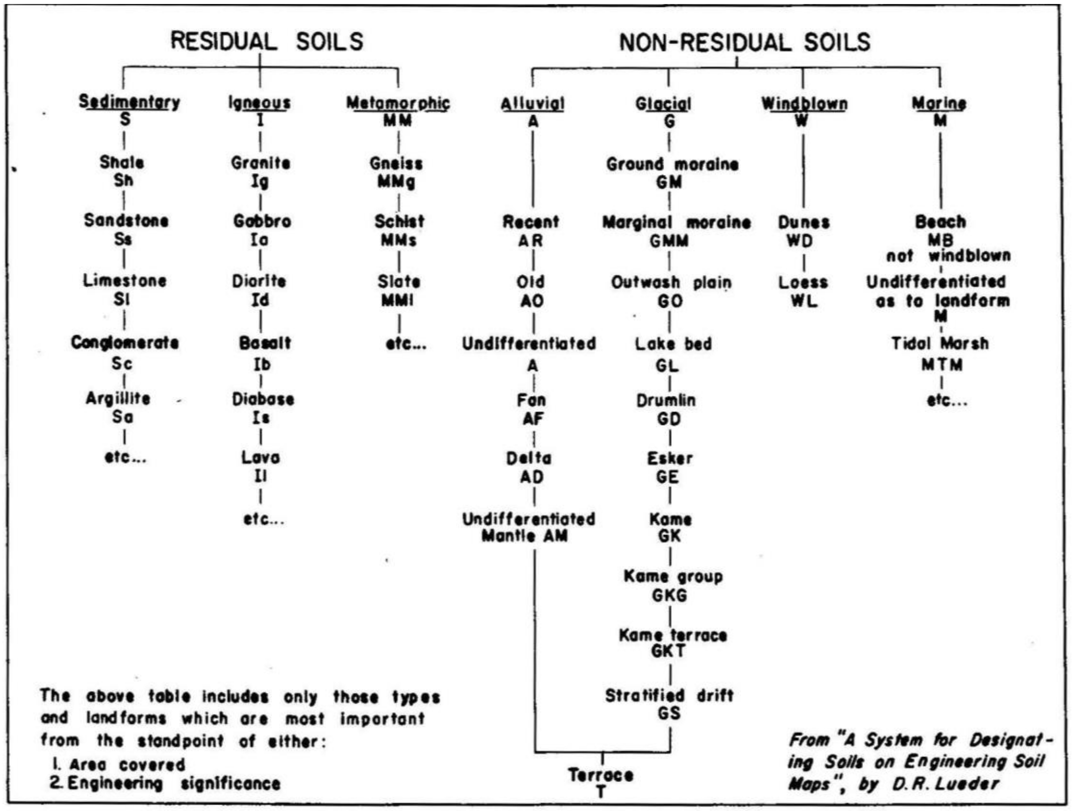


Figure 18. Geologic Notation

Table 9. Codes for Geologic Symbols

DESCRIPTION OF MATERIALS DENOTED BY MAP UNITS

AM- This symbol designates extensive areas of unconsolidated alluvial material which occurs as a discontinuous surface mantle in the Coastal Plain. The associated soil texture ranges from sandy gravel (AM-12), through silty sand (AM-23), and gravelly sand-silt (AM-24), to silt (AM-4).

AM-12 — This soil is present generally on ridges, hills and high areas and also forms some small terraces near streams. AM-12 is an excellent source of sand and gravel. The higher deposits are silty whereas those near streams are almost entirely silt-free, coarse sand and gravel. Topographic position and permeable structure provide for good internal and surface drainage. Many borrow pits are operated in AM-12 areas.

AM-23 — The AM-23 material is usually present bordering streams in quantities directly proportional to stream size), in broad, sandy plains between lower stream courses, and as sloping plains near sea level adjacent to tidal marsh. AM-23 is primarily silty sand with large areas of almost uniform medium sand. Its loose permeable structure promotes internal drainage. AM-23 provides a satisfactory source of sandy borrow material and is also an important source of concrete sand and filter sand.

AM-24 — The AM-24 occurs as broad, rolling, elevated plains. Small areas occur at lower elevations, some adjacent to tidal marsh and some bordering or within AM-4 areas, where erosion has removed the silt (AM-3) cover. This soil is a mixture of silt, sand and gravel. Drainage is usually good because of the elevated position of the larger areas, gently sloping ground surface and fairly open structure. AM-24 is a major source of good earth borrow material. It is particularly satisfactory for constructing soil roads. Numerous borrow pits are present.

AM-4 — This material occurs as extensive flat plains well above the surrounding terrain. Some small level areas are present near sea level adjacent to tidal marsh. AM-4 areas have a minimum of surface drainage features such as erosion gullies. This material is typically a uniform silt from four to eight feet deep overlying silty sand and gravel. Drainage is good because of relative topographic position and the porous natural structure of the soil. Pits in AM-4 areas furnish excellent top soil from the upper part and silty sand

and gravel borrow from the lower part.

AO- This symbol designates stratified older alluvium (second and third bottom) present as higher terrace and flood plain deposits along streams which are subject to flooding, usually at infrequent highwater stages. Large AO deposits occur in the Piedmont along Ambrose and Bound Brooks and the Raritan River and its branches. Small deposits occur along many streams in the rest of the northern part of the state.

Although the ground-water table is fairly shallow, the relatively elevated position and open structure of much of the material causes the surface and upper parts to remain fairly dry, particularly during the summer months. Local deposits of sand, gravel and good quality top soil are present in some AO areas. A general rule is that the coarser material is present along streams having steeper gradients, and beneath the silty surface soil along other streams.

AR - Recent alluvium (first bottom sediment) is shown as AR. These poorly drained, level lowlands, invariably adjacent to streams, are subject to flooding by seasonal high water. AR deposits, in the northern part of the state, are composed of stratified clay, silt, sand, gravel and even cobbles and boulders. The coarser alluvium usually borders the more swiftly flowing streams. In southern New Jersey the AR material is mostly silt and sand with some gravel. As a result of the prevailing low level surface in the Lower Coastal Plain, numerous long, wide AR areas are present. In many places the AR material is intermixed with tidal marsh, swamp and other poorly drained soil types. Recent alluvium is usually rich in organic matter and numerous deposits can be considered as sources of top soil, humus and even peat.

F - This symbol designates either filled areas (man-made land) or areas having man-made drainage control.

GD - Glacial drumlins are located in northern New Jersey and they are mapped as GD-24 and GD-42. These smoothly rounded, elongated hills are composed of an unconsolidated, unstratified accumulation of compact till. The included soil mass consists of various textures from clay to boulders. Surface drainage is good; however, internal drainage is usually poor.

GD-24 — This soil is mostly a clayey silt with much intermixed sand and gravel. Drumlins mapped as GD-24 are in Sussex County and they are a source of low grade borrow material.

GD-42 — In Essex County, the soil of the drumlins contains a high percentage of silt and therefore are shown as GD-42.

GE- This symbol indicates glacial eskers. Because the eskers are composed of a high percentage of sand and gravel, they are mapped GE-12. Eskers occur typically as narrow, fairly continuous, winding ridges of stratified drift which are a few hundred yards to several miles long. Several eskers are mapped in Bergen, Union, Morris and Sussex Counties. The largest is located at Florham Park in Morris County. The basal center part of an esker usually contains coarse sandy gravel whereas finer gravel and sand are present above the center part. This material, in turn, is overlaid by silty sand and silt, particularly along the flanks (see Fig. 4-10). The larger eskers are potential sources of good borrow material. Silt-free sand and gravel for concrete mix usually can be obtained. Drainage is invariably good both externally and internally because of the ridge land form and open structure of the material.

GK- GK designates glacial kames which occur in that part of the state north of the terminal moraine. The terminal moraine extends northerly from Perth Amboy to Denville and west to Belvidere. These well drained kame deposits occur as individual small hills (GK-12), or as a group of small hills (GKG-12), or as fields and groups of small hills (GKF-12). Rounded kame hills usually rise above valley floors or stratified drift plains. A typical kame may have a base diameter of one-eighth to one-quarter mile and a height of 50 to 150 feet. Kames are composed mostly of silt-free sand and gravel in discontinuous, inclined stratified layers These deposits are a source of high grade borrow material for use as concrete sand and aggregate. Exceptions to this latter are kames that occur in the Piedmont. A large percentage of incorporated weak shale particles may be present.

GL - This symbol designates material which was deposited in ponds and lakes formed during the glacial period and which now exists as swampy areas. (Shown as swamp on the Geology Map.)

GL — The lake-bed material, which is primarily peat and black or dark organic muck, is indicated by the GL symbol. This material is present in Sussex and northern Warren Counties as poorly drained, flat swamp and meadow land. Larger areas at Great Meadows and along the Wallkill River are ditched and farmed intensively for market produce. Material from other such deposits is excavated, dried and sold as humus.

GL-67 — This designates the lake-bed deposits in Union, Somerset, and Morris Counties. This material is primarily clay and silty clay with varying thicknesses of peat at the surface in many places. The GL-67 occurs mostly as poorly drained, flat areas along the Passaic River and in the adjacent swamps in the bed of the former glacial Lake Passaic. This large lake occupied the area between the Watchung Mountains and the Highlands. Thick deposits of pottery clay are present locally.

GM- Excluding the relatively small total area of recessional moraine, drumlins, eskers, kames, lake-bed sediments, stratified drift and bedrock outcrops, the entire area north of the terminal moraine is covered with ground moraine of a variable thickness. This moraine or till is a mixture of clay, silt, sand, gravel and boulders. The till forms a surface mantle a few feet to many feet thick. A rolling land form is typical with surface and internal drainage varying from poor to good.

GM-24 — This symbol designates the more desirable GM material for borrow purposes. It is largely a silty, gravelly sand with included cobbles and boulders. Local pockets of sand are present. This type of GM occurs in Essex, Passaic, Bergen, Hudson, Sussex and Warren Counties. Drainage conditions are fairly good in most locations. Some till areas in Passaic County are mapped GM-12 and GMX-24. These symbols are intended to indicate either extra-coarse till or deposits of a somewhat variable texture.

GM-4 — This ground moraine contains a high percentage of silt with some intermixed clay, sand, gravel and boulders. Drainage conditions vary from poor to good. Large, gently rolling areas occur in and near low swamp regions, whereas hummocky, steeply sloping deposits are along valley sides and on higher slopes. The GM-4 till is present in Essex, Union, Morris and Middlesex Counties and in a few small poorly drained areas in Warren County. Some areas in Essex and Passaic Counties are shown as GM-42 to indicate the predominance of silt in the till.

GM-46 — This symbol designates the low-lying, poorly drained moraine which contains a high percentage of fines. Some deposits are in depressions and contain concentrations of silt and silty clay. This type of ground moraine occurs in the glaciated Piedmont of New Jersey.

GMC -The symbol GMC-46 is used to designate early drift of the Jerseyan and Illinoian glacial stages. This drift occurs south of the terminal moraine in Morris, Warren, Hunterdon and Somerset Counties. The more extensive deposits are present in the main limestone valleys, on the gneiss near the terminal moraine in Warren and Morris Counties and on the Triassic sediments of northern Somerset and northeastern Hunterdon Counties. The early drift is characterized by its compact structure and its well-weathered condition. Included gneissic cobbles are apt to crumble readily under little pressure. The GMC-46 occurs as rolling valley bottom deposits, extensions of slopes and on flat upland areas. Surface drainage is fair to good, whereas internal drainage is usually impeded by the clayey B horizon. GMC-46 deposits are sources of common borrow material in some areas.

GMM -This map symbol represents the terminal and recessional moraine deposits of the Wisconsin stage of continental glaciation. The terminal moraine forms an almost continuous, hummocky and rolling- topped ridge from one-quarter to two miles wide. It extends across northern New Jersey from Perth Amboy northerly through Summit to Denville and west to the Delaware River at Belvidere. It is markedly broken only at Morristown by the Whippany River and at Chatham by the Passaic River. The recessional moraine is essentially the same as the terminal moraine except that the deposits are much smaller and form discontinuous ridges and scattered deposits well north of the terminal moraine in Sussex and Bergen Counties.

GMM-24 — This map unit indicates the soil mixture — varying proportions of clay, silt, sand, gravel and boulders — which constitutes the terminal and recessional moraines. The moraine material of the Piedmont was largely derived from weathered shale, sandstone, conglomerate and basalt, whereas that of the Highlands and Appalachian Valley and Ridge was derived mainly from the older formations in those areas. Surface drainage is fairly good on much of the GMM-24, but internal drainage is impeded in many places and water collects temporarily or seasonally in numerous kettle holes or depressions which dot the surface. Some of the terminal moraine in Essex County is mapped GMM-42 to indicate the presence of large percentages of silt.

GO- Areas mapped GO are south of the terminal moraine. This is stratified glacial outwash and consists of sorted and intermixed gravel, sand and silt. The coarser soils are near the terminal moraine, whereas the percentage of included fines increases farther from the terminal moraine. These deposits occur as terraces along many streams flowing from the glaciated region and as gently sloping out- wash plains in front of the terminal moraine.

GO-12 — This map unit designates the granular outwash — mostly gravel and sandy gravel. It usually occurs near or abutting the terminal moraine or as terrace deposits along streams. GO-12 is ideal borrow and is suitable for concrete mix and similar uses.

GO-24 — Large deposits of GO-24 occur adjacent to the front of the terminal moraine as broad, gently sloping outwash plains extending for considerable distances to the south, and as large terraces along the Delaware River. Large outwash plains in front of the terminal moraine are at Belvidere, Succasunna, from

Morris Plains to Chatham, and from Scotch Plains to Metuchen. This material is satisfactory for borrow and constitutes a large, valuable source of sand.

GO-4 — This indicates the silt phase of the outwash. Extensive GO-4 areas are a part of the outwash plain near Plainfield, Dunellen and Metuchen. The GO-4 is primarily uniform silt overlying gravelly, silty sand. These silt plains are excellent areas for farming and are sources of good topsoil.

GS - This map symbol includes all stratified glacial drift, other than eskers and kames, north of the terminal moraine. The major GS deposits occur as large terraces along the Delaware River; as flood plains and valley fill along streams, particularly the Pompton, Passaic and Hackensack Rivers; and as deltaic deposits like that at North Church. The terraces are fairly flat-topped, bench-like features; the flood plains are broad and level, often with channel scars; the valley fill occurs as small terraces and as mounds of drift; and the deltaic deposits are thick, steep-sided and flat-topped.

GS-12 — Large deposits of GS-12 are at Netcong, North Church and scattered over the entire glaciated part of New Jersey. This material is highly valued as quality borrow and is used extensively for concrete mix and other uses requiring the best materials. Drainage is excellent both internally and externally. Thick deposits often extend below the ground-water table and require dredging for removal from such pits.

GS-24 — Large deposits of GS-24 occur in all counties north of the terminal moraine. The broad flood plain along the Pompton River and the extensive outwash near Lafayette, Sussex County, are GS-24. Such deposits provide some of the major sources of sand in the northern part of the state. Thick deposits of GS-24 often are dredged well below the ground-water table.

GS-4 — Small, low areas of silty drift in Passaic, Bergen, Hudson and Union Counties are mapped as GS-4 to indicate the prevailing silty texture of the material.

GS-42 — This map unit is used in Essex County to indicate the predominance of silt over sand and gravel. Most of the GS-42 is satisfactory for use as common borrow.

GS-46 — Poorly drained low areas and depressions, with concentrations of silt and silty clay, are mapped GS-46 in Essex, Passaic, Bergen and Hudson Counties.

Ib- This symbol represents the basalt flows of the Piedmont Province. The basalt is a hard, dense, fine-grained, basic igneous rock which forms prominent ridges such as the Watchung Mountains and other smaller ridges such as Long Hill and Hook Mountain. Many trap rock quarries are presently operated in the basalt. Crushed basalt is widely used as aggregate in concrete and bituminous mixes, in highway construction and for rip rap and roofing granules. Many outcrops are characterized by intensive vertical jointing which facilitates excavation.

Ib-4 — The fairly thin soil cover on the high areas and upper slopes of the basalt ridges is mapped as Ib-4. Basalt bedrock underlies the silty soil, with included basalt fragments, at a shallow depth. Drainage is fairly good because of the ridge land form and steep slopes.

Ib-46 — This map unit designates the usual type of soil associated with the basalt as shown in Fig. 3-11 (see color insert, this chapter). Thick Ib-46 accumulations are at bases of slopes and on broad upland regions. This soil is a clayey silt or a silty clay with included basalt fragments. Internal drainage is impeded by the clayey soil texture.

Ibb- This symbol designates several small volcanic necks or plugs, several yards to one-quarter mile in diameter, which occur as small prominent bedrock hills in the vicinity of Beemerville, Sussex County. This breccia is a hard, dense, basic igneous rock containing biotite and included fragments of limestone, shale and gneiss. Small amounts of glacial soil material occur in pockets on the larger hills.

Igr— A mass of high, rugged hills occurs along the New York State line between Glenwood and Owens in Sussex County. The map symbol Igr designates the coarse-grained hornblende granite constituting the bedrock in this area. Glacial drift locally forms a thin soil cover.

Ins- This symbol designates a rugged, bench-like outcrop of intrusive igneous rock (nephelite syenite), approximately two miles long, against the face of Kittatinny Mountain west of Beemerville. Included are scattered dikes of porphyritic nephelite syenite, tinguaite and bostonite, occurring in the Martinsburg shale. The syenite is rich in feldspar and would make an attractive and durable building stone.

Is — This map symbol designates the intrusive igneous rock which forms such prominent ridges in the Piedmont as Sourland and Cushetunk Mountains, Rocky Hill and the Palisades. This rock is very similar to the Triassic basalt (Ib) in composition and color but has a medium to coarse-grained texture.

Only the bedrock outcrops in Bergen County are mapped Is. Numerous trap rock quarries are present in the diabase ridges.

Is-24 — The thin rocky soil cover on several of the diabase ridges and upper slopes in Mercer County is indicated by the Is-24 map unit.

Is-46 — This map unit refers to the thicker, clayey silt and clay soil associated with the diabase in Hunterdon, Somerset, Middlesex and Mercer Counties. A large percentage of diabase fragments is generally included in the soil mass. Internal drainage is impeded by clayey soil texture.

M— The unconsolidated marine formations of southern New Jersey are designated by the letter M. Land form of the deposits tends to vary according to the texture of the various sediments.

M-23 — This light-colored soil consists primarily of well-drained, stratified, uniform sand and silty sand. Large, undulating M-23 areas are in the Lower Coastal Plain and smaller hummocky outcrops are present in the Upper Coastal Plain. M-23 materials can be used for filters, subdrains and as molding sand. Some make good concrete sand, but for the most part they are too fine. Numerous large borrow pits are present.

M-24 — Extensive areas of intermixed silt and sand are present, primarily in the Upper Coastal Plain. Land form varies from rolling to undulating. Numerous areas are either low with a poor surface runoff potential or contain sufficient silt and glauconite to hamper internal drainage. Large pits are operated in M-24 areas to obtain common earth borrow material.

M-27 — This predominantly green soil which crops out in the Upper Coastal Plain is a mixture of silt and clay with some sand. Very high percentages of glauconite are usually included. Numerous small hilly to undulating M-24 areas are present with very poor internal drainage. Several deep pits are present in Monmouth and Burlington Counties. The glauconite is used commercially as a water softener and as fertilizer.

M-3 — In Ocean County large areas of sand, with a minimum of included silt, are present and such areas are shown as M-3.

M-46 — This soil is mostly a clayey silt or laminated silty clay with very poor drainage characteristics. Lenses and layers of sand are usually present in the soil profile. Random outcrops occur near and are parallel to the Delaware River and extend from Trenton to the general vicinity of South Amboy. Several very large pits are present in Monmouth and Middlesex Counties as the M-46 material is an excellent source of clay for industrial uses.

M-67 — This symbol designates stratified deposits of blocky micaceous clay. In some places a few feet of silty soil overlie the darker- colored, impervious clay strata. Numerous large open pits are worked, mostly in the Upper Coastal Plain. This clay is used for the manufacture of brick, tile and other ceramic products.

MB- Coastal deposits of sand and gravel are designated with the MB symbol. These materials border mostly the Atlantic Ocean with some outcrops along Delaware Bay in Cape May and Cumberland Counties. The narrow (approximately one-quarter mile wide) off-shore bar usually present consists of fine to coarse sand with a little fine gravel in a few places. A short distance (from 100 to 200 feet) inland from the seaward side of the bar, a series of hummocky, well-drained dunes is usually present. The dunes are predominantly fine sand. The coarser coastal beach materials, MB-13, are in Monmouth and Ocean Counties and the finer sandy sediments, MB-3, occur farther to the south. These beach deposits are possible sources of uniform sand, both fine and coarse.

MC- In Monmouth County the symbol MC-6 identifies a significant soil condition which is associated primarily with the Navesink marl formation. This marine deposit forms extensive undulating to rolling areas which are poorly drained both at the ground surface and within the soil mass. The soil profile consists of a layer of silt overlying silty clay. The latter material usually overlies silty sand. Deeper in the profile, a glauconitic, impervious, clayey marl is present. The MC-6 material may be a potential industrial source of glauconite.

ML- In the Outer Coastal Plain some of the well-drained, sandy marine formations form prominent, high, steep-sided hills and ridges. Usually these conspicuous land forms (some are outliers) are present as groups of hills and ridges. Near the surface of many such deposits, a thick (up to 30 feet) stratum of cemented sand or gravel (ironstone) is present.

ML-12 - This soil type consists of sandy gravel with numerous lenses of sand. These coarse materials, together with included ironstone layers, overlie silty sand and sand at depths greater than ten feet. A photograph of a pit face in an ML-12 area is shown in Fig. 3-14 (see color insert, this chapter). ML-12 deposits supply large quantities of south Jersey gravel.

ML-23 - This soil is mostly sand, possibly with several feet of gravelly sand near the surface in some areas. Thick ironstone layers are also present. The ML-23 deposits are worked in numerous places for supplies of earth borrow and for uniform sand.

MMg This designates the gneisses of the Highlands in northern New Jersey. These are primarily resistant, granitoid, metamorphic rocks of various colors such as black, brown, pink and gray. The gneisses are characterized by jointing in three planes, spaced a few inches to several feet apart. The gneiss north of the terminal moraine forms high, rugged, rocky hills and ridges separated by deep valleys. South of the terminal moraine the gneissic hills are more rounded and have varying depths of weathered material accumulated as soil cover. This soil material extends to depths of many feet in some places and most of the hills have a considerable thickness of rock fragments and rubble accumulated on them. Only small areas are mapped MMg (non-soil cover) south of the terminal moraine, whereas to the north all of the gneiss not covered with glacial deposits is mapped MMg.

MMgC-24 - This map unit designates areas of rough stony land on hills and steep upper slopes in Warren County. A large percentage of the soil consists of small and medium angular rock fragments and sand particles in addition to the clayey silt fraction.

MMgC-46 - This map unit designates most of the gneissic region south of the terminal moraine. It indicates primarily the area of deep rock weathering characteristic of this region. The soil is a clayey silt with a large percentage of rock fragments. A clayey, compact B horizon is present in most areas. Deepest soil accumulations are at bases of slopes and on flatter areas, with increasing amounts of fragments and large rocks on steep slopes, hill tops and along streams. The small letter "a" is appended to the drainage symbol (MMgC-46ge"a") to indicate the normal rolling to hilly land form, whereas a small letter "b," similarly appended, indicates hills and ridges of higher relief. Surface and internal drainage are good on most high gneiss areas because of steep slopes, high percentage of rock fragments and porous soil structure. Internal drainage is usually impeded on flatter areas and water tends to remain at the surface before slowly percolating down through the clayey B horizon. Where jointing is closely spaced, suitable borrow can be obtained from the weathered bedrock. A large quarry in massive gneiss bedrock at Riverdale operates in much the same manner as a trap rock quarry.)

MMg — This symbol represents bedrock outcrops of the Hardyston sandstone. This formation, conglomeratic at the base and shaly towards the top, crops out as a narrow, discontinuous bench between tilt. gneiss and the Kittatinny limestone. The largest bedrock outcrops are in Warren County. Very little residual soil is present although ground moraine covers some of the formation. The MMq is a possible source of building stone in some places.

MTM This symbol designates both marine and fresh water tidal marsh.

MV- Several of the more glauconitic marine formations of the Inner Coastal Plain are extremely variable in their textural content and outcrop pattern. Lenses and layers of dull gray, black and dark green sandy clay and clay transgress a dull brown silty and clayey sand profile at various depths. The map unit MV-47 designates this variable soil condition, which is mostly the result of stratification. Large, low, poorly drained areas are present in Burlington, Monmouth and Middlesex Counties. In many places a thin (two to eight feet) cover of gravelly sand caps this green-black clayey material and compound map units, such as AM- 24 are commonly employed.

MX- In some areas of the Lower Coastal Plain, predominantly in Ocean County, an extremely intermingled assortment of stratified materials, consisting of gravel, sand, silt and clay in various combinations, is present.

MX-2 — This designates a deposit of either clay-coated sand grains or a mixture of gravel and clay with some sand. These materials occur as thick stratified layers which have a random outcrop pattern. Surface and internal drainage are usually imperfect. In most places the land form is hummocky and dissected. MX-2 materials are used for fill and common borrow in southern New Jersey.

MX-67 — Random outcrops of thick strata of white and yellow blocky clay with layers of sandy clay are shown with this symbol. These areas are poorly drained and the ground-water table is usually close to the ground surface. MX-67 areas are possible sources of clay for industrial uses.

R - This symbol designates a variable and/or complex geologic, soil and cultural condition.

Sa— The triassic argillite in Hunterdon, Somerset, Middlesex and Mercer Counties is referred to as Sa. This is a dense, hard, dark gray rock which forms broad low ridges and undulating areas with some steep slopes to the north. Some interbedded layers of hard shale are present. The argillite breaks into large pieces (up to two feet) whereas the shale breaks easily into smaller pieces (up to six inches). The argillite is a possible source of building stone.

Sa-4 — This symbol designates the silty soil which is developed from the weathered products of the argillite. Internal drainage in Sa-4 areas is usually poor.

Sc — This map symbol represents several bedrock formations in the northern part of New Jersey. Sc indicates the Shawangunk conglomerate (Ssg), a resistant gray quartzite and conglomerate, which forms Kittatinny Mountain. The Green Pond conglomerate (Sgp), except for its red-purple color, is similar to the Shawangunk and is also mapped Sc. This rock forms the high, prominent Green Pond, Copperas and Bowling Green Mountains in Morris and Passaic Counties. Another mountain-forming rock (Bearfort Mountain in Passaic County) mapped Sc is the Skunnemunk conglomerate (Dsk). This is a purple-red massive conglomerate containing abundant large, white quartz pebbles, alternating with beds of red quartzitic sandstone. All of these formations are mapped Sc, which indicates that they are essentially **non**-soil areas. These rocks are well indurated and extremely resistant, as evidenced by their ridge-forming tendency. The Triassic Border conglomerate (Trc) is another formation mapped Sc. Major outcrops are along the northwest border of the Piedmont from Pottersville to the Delaware River. Small outcrops are near Gladstone, on Mount Paul near Ralston, at New Vernon, near Morristown and at Montville.

Sc-46 — The large, unglaciated areas of the Border conglomerate are mapped Sc-46 because of the usual thick soil cover. This is a clayey soil with included pieces of rock, many rounded and some angular. Random, rounded hills typify these areas.

Sh - The mapping symbol Sh designates the bedrock outcrops of both the Brunswick shale (Trb) and the Martinsburg shale (Omb). The Brunswick is chiefly a soft red shale with some interbedded sandstone, which is more abundant to the northeast. This rock forms extensive rolling to undulating areas throughout the Piedmont. The Martinsburg is mostly a very fine-textured, gray to black rock with well-developed slaty cleavage. The typical slate splits easily into small, thin plates and larger thick slabs. The land form varies from a rolling surface to smoothly-rounded oval or linear hills and sharp-crested ridges. This rock crops out mostly in Sussex and the northern part of Warren Counties.

Sh-2 — This symbol designates the normal soil development in Martinsburg shale areas. The thin soil cover, from one to three feet thick, contains a very high percentage of shale fragments, as can be seen in Figure 3-16 (see color insert, this chapter). This material is suitable for embankment construction and as a source of borrow. It is also used in many places for road surfacing.

Sh-4 — The greater part of the unglaciated Piedmont of New Jersey is mapped Sh-4. This red soil, developed from the weathered Brunswick shale, is predominantly silt with a large percentage of included shale fragments. Surface drainage, in Sh-4 areas, is good but internal drainage is only fair.

Sh-67 — In Middlesex County several large, low areas of Brunswick shale are present. The poorly drained soil in these areas is a mixture of silt and clay with some shale fragments.

Shl- This map symbol represents a group of Upper Silurian and Lower Devonian formations in northern New jersey. Most of these formations crop out in northwest Sussex County and constitute all of the bedrock formations of Wallpack Ridge (except the crest-forming Esopus grit) from Flatbrook to the Delaware River. The remainder occur along the flanks of the ridges made by the Green Pond conglomerate in northern Morris and Passaic Counties. With the exceptions of the Onondaga limestone and Marcellus shale, on the northwest slope and terrace of Wallpack Ridge, all of the Shl formations are thin and largely composed of limestone and shale with some usually calcareous sandstone. The limestone occurs in fairly thick, well-defined beds. The shale varies from thin beds of gray, limy shale to thick, highly fractured black shale. Glacial deposits of varying thicknesses cover parts of the formations. Some of the shaly limestones are small potential sources of rock for the manufacture of cement.

Sl — Three formations in northern New Jersey are designated by the S1 symbol. These are the Franklin, Kittatinny and Jacksonburg limestones. These formations occur in the lower parts of valleys. In glaciated areas, the limestone often crops out as prominent valley bottom ridges, whereas in non-glaciated areas good outcrops are usually present along streams. The Franklin limestone is typically a gray to white granular limestone or calcite marble which occurs mostly in Warren and Sussex Counties. This is the rich zinc ore limestone of the Franklin area. It is also quarried, crushed and calcined at Lime Crest for agricultural and building lime. The Kittatinny limestone crops out in large areas in Warren and Sussex Counties and in small areas in Morris, Somerset and Hunterdon Counties. This is a thick formation of massive, dark gray dolomitic limestone with included shaly and siliceous beds. This limestone was formerly extensively quarried primarily to obtain lime for agricultural uses. Numerous large and small quarries and lime kilns remain as evidence of former operations. The Jacksonburg limestone crops out as a discontinuous band between the Kittatinny limestone and the Martinsburg shale. It is mostly in Warren and Sussex Counties and some in Hunterdon County. The Jacksonburg is a dark gray to black limestone and limy shale. It has been extensively used in the manufacture of cement in Warren County.

Sl-47 — Soils derived from the weathered Kittatinny and Jacksonburg limestones are represented by the S1-47 map unit. These are brown to yellow-brown, silt, silty clay and clay soils underlain at approximately four to eight feet by bedrock. Small areas of limestone are at the surface in some places, particularly along streams and on the steeper slopes. This soil has a naturally loose and permeable structure in spite of its clayey texture. S1-47 areas make good agricultural land.

Ss — This map symbol designates small areas of fine-grained, red **Newark**" sandstone (Trb) along the flanks of ridges in the Piedmont of Bergen and Passaic Counties, and the dark gray to black, fine-grained Esopus grit (Des) which forms the crest of Wallpack Ridge in Sussex County. The red, Triassic sandstone has been widely used as an easily worked building stone.

Ssh- Several formations of northern New Jersey are represented by the Ssh symbol. The High Falls formation consists primarily of red sandstone and shale which forms the high valley and secondary ridge on the backslope of Kittatinny Mountain. Scattered glacial deposits cover extensive areas of this formation (Shf). The hard, dark-gray, slaty Pequannac shale and gray Bellvale sandstone overlying the Pequannac occur in northern Morris and Passaic Counties. These formations crop out as small ridges on the bottom of a glacial drift- covered valley (Dbp). The Stockton formation is composed of arkosic sandstone with shale and conglomerate beds. It forms fairly large areas in the Piedmont of Hunterdon, Somerset, Middlesex and Mercer Counties. The light-colored sandstone is used as an attractive build stone. The occurrences along the Palisades are largely covered with glacial drift and have not been indicated on the soil map.

Ssh-4 — The Stockton formation is characterized in many places by a silty soil cover and is mapped as Ssh-4.

T - Some glacial stratified drift, in the form of stream terraces in Passaic County, is indicated by the symbol T-12. These deposits contain gravel and sand and are excellent sources of select borrow material. Several similar deposits in this county are irregular in shape and have surface depressions. These latter areas are mapped TX-12 on the soil map.

Z - The symbol Z, which designates swamp areas.

TEXTURE SYMBOLS (AASHTO Classification Range) — The second part of the soil code, that which identifies soil texture, utilizes an abbreviated form of the AASHTO soil classification system. This system uses the notation A-1-a to A-7-6 for textures ranging from well-graded, granular materials to clay-soils, respectively. The texture symbol used on the engineering soil maps consists of the number that follows the letter "A" in the Highway Research Board soil classification system. For example, a soil that varies from A-2-4 to A-4 is identified by the notation 24. If the soil variation falls within one group, such as A-4, the texture is indicated by the symbol 4. The controlling grain size percentages and soil consistency test values for the seven symbols used in the code system are listed in Figure 19.

Table 10. AASHTO Classification Descriptions

GRANULAR MATERIALS

Containing 35 Per Cent or Less Passing the No. 200 Sieve.

Group A-1 - The typical material of this group is a well-graded mixture of stone fragments or gravel, coarse sand, fine sand and a nonplastic or feebly plastic soil binder. However, this group includes also stone fragments, gravel, coarse sand, volcanic cinders, etc., without soil binder.

Subgroup A-1-a includes those materials consisting predominantly of stone fragments or gravel, either with or without a well- graded binder of fine material.

Subgroup A-1-b includes those materials consisting predominantly of coarse sand either with or without a well-graded soil binder.

Group A-3 - The typical material of this group is fine beach sand or fine desert blow sand without silty or clay fines or with a very small amount of nonplastic silt. The group includes also stream- deposited mixtures of poorly graded fine sand and limited amounts of coarse sand and gravel.

Group A-2 - This group includes a wide variety of "granular" materials which are border-line between the materials falling in Groups A-1 and A-3 and the silt-clay materials of Groups A-4, A-5, A-6, and A-7. It includes all materials containing 35 per cent or less passing the No. 200 sieve which cannot be classified as A-1 or A-3, due to fines content or plasticity, or both, in excess of the limitations for those groups.

Subgroups A-2-4 and A-2-5 include various granular materials containing 35 per cent or less passing the No. 200 sieve and with a minus No. 40 portion having the characteristics of the A-4 and A-5 groups. These groups include such materials as gravel and coarse sand with silt contents or plasticity indexes in excess of the limitations of Group A-1, and fine sand with nonplastic silt content in excess of the limitations of Group A-3.

Subgroups A-2-6 and A-2-7 include materials similar to those described under subgroups A-2-4 and A-2-5 except that the fine portion contains plastic clay having the characteristics of the A-6 or A-7 group. The approximate combined effects of plasticity indexes in excess of 10 and percentages passing the No. 200 sieve in excess of 15 is reflected by group index values of 0 to 4.

SILT-CLAY MATERIALS

Containing more Than 35 Per Cent Passing the No. 200 Sieve

Group A-4 - The typical material of this group is a nonplastic or moderately plastic silty soil usually having 75 per cent or more passing the No. 200 sieve. The group includes also mixtures of fine silty soil and up to 64 per cent of sand and gravel retained on No. 200 sieve. The group index values range from 1 to 8, with increasing percentages of coarse material being reflected by decreasing group index values.

Group A- 5 - The typical material of this group is similar to that described under Group A-4, except that it is usually of diatomaceous or micaceous character and may be highly elastic as indicated by the high liquid limit. The group index values range from 1 to 12, with increasing values indicating the combined effect of increasing liquid limits and decreasing percentages of coarse material.

Group A-6 - The typical material of this group is a plastic clay soil usually having 75 per cent or more passing the No. 200 sieve. The group includes also mixtures of fine clayey soil and up to 64 per cent of sand and gravel retained on the No. 200 sieve. Materials of this group usually have high volume change between wet and dry states. The group index values range from 1 to 16, with increasing values indicating the combined effect of increasing plasticity indexes and decreasing percentages of coarse material.

Group A-7 - The typical material of this group is similar to that described under Group A-6, except that it has the high liquid limits characteristic of the A-5 group and may be elastic as well as subject to high volume change. The range of group index values is 1 to 20, with increasing values indicating the combined effect of increasing liquid limits and plasticity indexes and decreasing percentages of coarse material.

Subgroup A-7-5 includes those materials with moderate plasticity indexes in relation to liquid limit and which may be highly elastic as well as subject to considerable volume change.

Subgroup A-7-6 includes those materials with high plasticity indexes in relation to liquid limit and which are subject to extremely high volume change.

CLASSIFICATION OF HIGHWAY SUBGRADE MATERIALS (40)

General Classification	Granular Materials (35% or Less Passing No. 200)							Silt-Clay Materials (More than 35% Passing No. 200)			
Group Classification	A-1			A-2							
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve Analysis % passing											
No. 10	50max										
No. 40	30max	50max	51min								
No. 200 . . .	15max	25max	10max	35max	35max	35max	35max	36min	36min	36min	36min
Character of Fraction Passing No. 40											
Liquid Limit				40max	41min	40max	41min	40max	41min	40max	41min
Plasticity Index	6 max		N.P.	10max	10max	11min	11min	10max	10max	11min	11min*
Group Index**	0		0	0		4 max		8max	12max	16max	20max
Usual Types of Significant Constituent Materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good					Fair to Poor					

Figure 19. AASHTO Classification Description

DRAINAGE SYMBOLS — The third part of the code, that part used to indicate the prevailing or average drainage conditions, expresses an estimate of sub-surface drainage, classed as excellent to very poor for estimated ground-water table depths of over ten feet to less than one foot, respectively. The estimate of ground water conditions is based primarily on the interpretation of air photo patterns supplemented, in some instances, by field observations. Table 11 lists the code symbols for drainage conditions, with descriptive terms and the corresponding estimated depths to ground-water table.

Table 11. CODE SYMBOLS FOR DRAINAGE CONDITIONS

Symbol	Type of Ground-Water Condition	Estimated Depth to Ground-water Table
e	Excellent	over 10 ft
g	Good	6 to 10 ft
i	Imperfect	6 ft
p	Poor	1 to 3 ft
v	Very Poor	0 to 1 ft

SPECIAL SYMBOLS — Special symbols are employed to denote conditions that cannot be clearly described by the three-part code system. The more common are listed in Table 12.

TABLE 12. SPECIAL SYMBOLS

C	Contrast Between Horizons: Indicates soil areas in which the B and C horizons are sufficiently dissimilar to warrant individual treatment in design and construction. The B horizon usually has more fine soil particles and is more plastic than the C horizon.
F	Fill: Often industrial or municipal waste.
ML	Land Form: Indicates high, steep-sided hills and ridges (often isolated outliers)in the outer coastal plain. These predominantly marine deposits usually have ironstone layers near the surface.
R	Variable: Denotes a range of conditions far beyond that which can be described with any degree of precision by the three-part code system. Usually the areas so labeled on the engineering soil maps are described in the corresponding county report.
X	Exceptional: Used where the code system does not accurately describe conditions. Usually explained in the county report.
Z	Swamp: Indicates areas of high ground-water table and mucky surface soil. The county report usually estimates the depth to which the mucky materials extend.
a, b	Relief: These letters appended to the MMgC map unit drainage symbol indicate two types of relief: a, the usual rolling, hilly topography; b, areas of prominent ridges and high relief.
/	Diagonal Bar: Used to separate two mapping symbols where both materials are present at the ground surface, but the individual occurrence of each is too small to permit separate mapping.
—	Horizontal Bar: Used with code symbols above and below the bar. The material described by the upper symbol appears at the ground surface and is underlaid at shallow depths by the material described by the lower symbol. The compound symbol, in the form of a fraction, is applied where the underlying material differs considerably from the surface soil and occurs close enough to the ground surface to warrant consideration in design and construction.

County Soil Survey Maps

The same Subgrade Soil types exist in more than one county. Table 13 provide a summary of the Subgrade Soil types by county. Figure 20 provides an illustration of the locations in the State that have high concentrations of Gravel, Sand, Silt and Clay.

Table 13. Location of Subgrade Soils by Type

	ATLANTIC	BERGEN	BURLINGTON	CAMDEN	CAPE MAY	CUMBERLAND	ESSEX	GLOUCESTER	HUDSON	HUNTERDON	MERCER	MIDDLESEX	MONMOUTH	MORRIS	OCEAN	PASSAIC	SALEM	SOMERSET	SUSSEX	UNION	WARREN
AM-12	✓		✓	✓	✓	✓		✓				✓	✓	✓	✓		✓				
AM-23	✓		✓	✓	✓	✓		✓				✓	✓		✓		✓				
AM-24	✓		✓	✓	✓	✓		✓			✓	✓	✓		✓		✓	✓			
AM-4			✓			✓					✓	✓	✓				✓				
AO										✓				✓		✓		✓	✓		✓
AR	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
F	✓	✓	✓	✓	✓	✓	✓	✓	✓			✓	✓	✓	✓	✓	✓			✓	✓
GD-24																			✓		
GD-42							✓														
GE-12		✓							✓					✓					✓	✓	
GK-12		✓					✓		✓			✓		✓		✓			✓	✓	✓
GKF-12																✓					
GKG-12							✓									✓					
GL																			✓		✓
GL-67														✓				✓		✓	
GM							✓									✓					
GM-12																✓					
GM-24		✓					✓		✓							✓			✓		✓
GM-4							✓					✓		✓						✓	✓
GM-42							✓									✓					
GM-46		✓					✓		✓			✓				✓				✓	
GMC-46										✓				✓				✓			✓
GMM-24		✓					✓		✓			✓		✓					✓	✓	✓
GMM-42							✓														
GMX-24																✓					
GMX-24R																✓					
GO-12							✓														✓
GO-24											✓	✓		✓				✓		✓	✓
GO-4												✓						✓		✓	
GS		✓					✓		✓							✓					
GS-12		✓					✓		✓					✓		✓			✓		✓
GS-24		✓					✓		✓			✓		✓		✓			✓	✓	✓
GS-4		✓							✓							✓				✓	
GS-42							✓														
GS-46		✓					✓		✓							✓					
GSR																✓					
GSX																✓					
GSX-12																✓					
GSX-24																✓					

	ATLANTIC	BERGEN	BURLINGTON	CAMDEN	CAPE MAY	CUMBERLAND	ESSEX	GLOUCESTER	HUDSON	HUNTERDON	MERCER	MIDDLESEX	MONMOUTH	MORRIS	OCEAN	PASSAIC	SALEM	SOMERSET	SUSSEX	UNION	WARREN
Ib		✓					✓		✓					✓		✓					
Ib-4														✓				✓		✓	
Ib-46										✓				✓	✓			✓		✓	
Ibb																			✓	✓	
Igr																			✓		
Ins																			✓		
Is		✓							✓			✓									
Is-24											✓										
Is-46										✓	✓	✓						✓			
M-23	✓		✓	✓		✓		✓			✓	✓	✓		✓		✓				
M-24			✓	✓				✓			✓	✓	✓		✓		✓				
M-27			✓	✓				✓					✓		✓		✓				
M-3			✓												✓						
M-46				✓							✓	✓	✓					✓			
M-67	✓		✓	✓	✓	✓		✓			✓	✓	✓				✓				
MB-13													✓		✓						
MB-3	✓				✓	✓															
MC-6			✓										✓								
ML-12													✓		✓						
Ml.-23			✓	✓								✓	✓		✓						
MMg		✓							✓	✓				✓		✓			✓		✓
MMgC-24																					✓
MMgC-46										✓				✓				✓			✓
IMMq										✓									✓		✓
MTM	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓	✓		✓		✓			✓	
MV-47			✓	✓				✓			✓	✓	✓								
MX-2			✓												✓						
MX-67			✓												✓						
R		✓		✓			✓		✓		✓	✓	✓	✓		✓		✓			✓
Sa											✓	✓									
Sa-4										✓	✓	✓						✓			
Sc														✓		✓			✓		✓
Sc-46										✓				✓				✓			
Sh							✓			✓	✓	✓		✓		✓		✓	✓	✓	✓
Sh-2										✓				✓					✓		✓
Sh-4										✓	✓	✓		✓				✓			
Sh-67												✓									
Shl														✓		✓			✓		
Sl										✓				✓					✓		✓

	ATLANTIC	BERGEN	BURLINGTON	CAMDEN	CAPE MAY	CUMBERLAND	ESSEX	GLOUCESTER	HUDSON	HUNTERDON	MERCER	MIDDLESEX	MONMOUTH	MORRIS	OCEAN	PASSAIC	SALEM	SOMERSET	SUSSEX	UNION	WARREN
S1-47										/				/				/			/
Ss		/					/		/							/			/		
Ssh												/		/		/			/		/
Ssh-4										/	/	/						/			
T-12																/					
TX-12																/					
Z	/	/	/	/	/	/	/	/	/		/	/	/	/	/	/	/	/	/	/	/
TOTAL	10	19	18	15	8	11	24	12	19	16	19	31	19	28	16	33	12	20	23	17	25

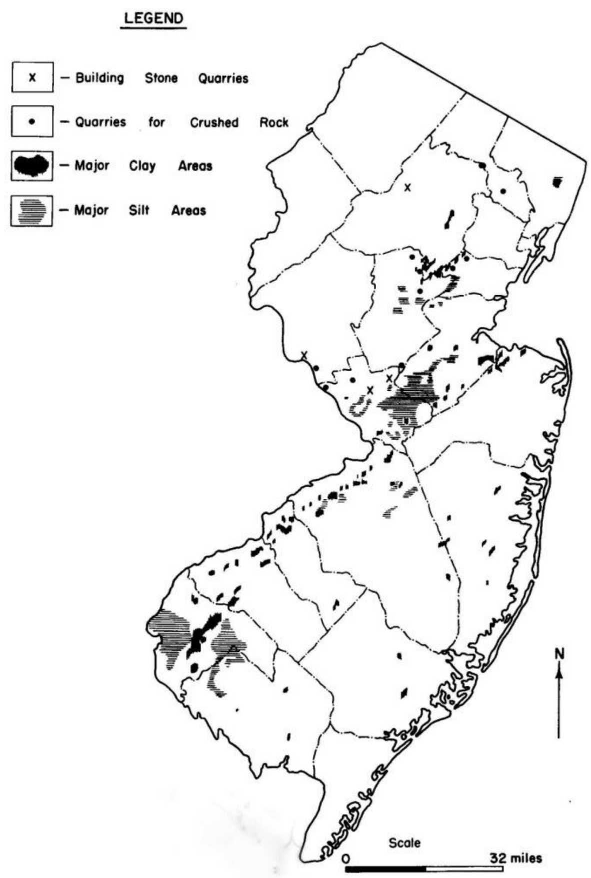


Figure 20. Illustration of locations that have large concentrations of Gravels, Sands, Silts and Clays

Problem Subgrade Soils Types –

The following table (Table 14) contains a summary of Subgrade Soils that are Frost Susceptible and benefit from Subbase soil layer to minimize the penetration of the frost layer into the Subgrade or weak soils that provide minimal Pavement Support and require a Subbase layer to reduce the microstrain levels from wheel loads.

Table 14. Frost Susceptible or Weak Subgrade Soils by Type

AM-24 - unconsolidated alluvial material

Soil Type	Silt, silty sand and silty and clayey sand and gravel.
Pavement Support	Fair to good depending upon the silt and clay content and the drainage facilities afforded in each case.
AASHTO Classification	A-2-4, A-4

AM-4- unconsolidated alluvial material

Soil Type	Silt and sandy silt with some interbedded layers of silty sand. Some gravel is commonly present throughout the profile. Usually silty sand and silty sand and gravel are present with depth. Internal drainage is imperfect to poor in the A-4 material
Pavement Support	Only fair because of the high silt content. Pavement support will be very poor in areas where the groundwater table is shallow. Pavement damage to roads, caused by detrimental frost action, is severe in areas mapped AM-4. The presence of surface water in the AM-4 material is also a contributing factor to damage by frost.
AASHTO Classification	A-4

AO – Recent Alluvial

Soil Type	Variable, but generally quite silty, with appreciable amounts of clay-sizes, and often significant accumulations of organic materials.
Pavement Support	Usually rated poor, with minor areas rated fair. High water table tends to keep these soils in a constant saturated state and therefore, a raised grade line is frequently advisable
AASHTO Classification	A-4, A-5

AR – Recent Alluvial

Soil Type	Variable, but generally quite silty, with appreciable amounts of clay-sized, and often significant accumulations of organic materials.
Pavement Support	Usually rated poor, with minor areas rated fair. High water table tends to keep these soils in a constant saturated state and, therefore, a raised grade line is frequently advisable.
AASHTO Classification	A-4, A-5

GD-24 - Glacial Drumlins

Soil Type	Clayey silt, silt and silty sand. Usually numerous pebbles and cobbles, and a few boulders, are scattered through the profile. The ground water-table is fairly deep.
Pavement Support	Variable. Fair to good under light axle loads and fair to poor under repeated, heavy axle loads. Fines content and internal drainage are governing factors.
AASHTO Classification	A-2-4, A-4 Detrimental effects of frost action should be anticipated where A-4 is predominant.

GL - Lake-Bed Material

Soil Type	Mostly organic matter. Some clay, silt and sand is intermixed with the peat and also underlies it. Poor surface and internal drainage are the result of level surface, low elevation and high ground water-table. The latter is a few feet from the surface.
Pavement Support	Very poor.
AASHTO Classification	Soil grouping by the HRB classification system is affected by the high organic content. This material is soft peat and muck with a low bearing capacity. Several samples were taken in areas mapped as GL, but test results were erratic and misleading. Therefore, no test results are tabulated in Appendix A and no engineering test values are listed for the GL map unit.

GL -67- Lake-Bed Material

Soil Type	Clay and silty clay, with some silt and sand in the lower horizons.
Pavement Support	Very poor. Poor drainage, low densities and high plasticity will probably make the use of subbase and a raised gradeline necessary.
AASHTO Classification	A-6, A-7-5 and A-7-6 predominate. A-4 and A-2-7 groups, when encountered, probably are the result of intermingling along the borders of the areas.

GM – Ground Moraine

Soil Type	Silty-loams, and sandy-silts with varying amounts of pebbles, gravel, and boulders. Below depths of 3-4 feet, the material tend more towards silty-sand.
Pavement Support	Rated as poor to very poor in the GM-46 areas. The use of subbase is advisable where other than light traffic is expected.
AASHTO Classification	A-4, A-6

GM – 46 Ground Moraine

Soil Type	Silty-loams, silty-sands and sandy-silts with varying amounts of pebbles, gravel, and boulders. Usually poor internal drainage, intermediate to poor surface drainage, moderately high capillarity, and fairly highwater-tables in the southern part of the county.
Pavement Support	Rated as good to occasionally excellent in the GM-24 and GMX-24 areas, fair to good in the better GM-42 areas, and poor to very poor in some of the GM-46 areas. In the last-mentioned case, the use of subbase is advisable where other than light traffic is expected.
AASHTO Classification	A-4, A-6

GM – 4 Ground Moraine

Soil Type	The soil in GM-4 areas is a silt or a silty sand. Drainage is imperfect because of silty soil textures, flat slopes and the relatively shallow perched ground water-table.
Pavement Support	Fair to occasionally good in GM-4 areas
AASHTO Classification	Uniformly silty to considerable depth in GM-4 areas, A-4 predominant.

GMC – 46 - Early Drift of the Jerseyan and Illinoian Glacial

Soil Type	Silts, silty clays, and silty sands, with a scattering of pebbles, cobbles and boulders. water-tables may be expected to occur at considerable depths, generally below 10 feet.
Pavement Support	Good to excellent in the C-horizon; poor to fair in the B-horizon. When making cuts, seepage and unequal pavement support should be anticipated where the subgrade surface changes from B to C-horizon.
AASHTO Classification	A-2-4 to A-4

GMM – 42 Marginal Ground Moraine

Soil Type	Silty-sands, sandy-silts, and silts with some clay and varying percentages of gravel, cobbles, and boulders.
Pavement Support	Rated as fair to good in well drained areas and poor to fair in poorly drained areas.
AASHTO Classification	A-4, A-2-4

GMX – 24 Marginal Ground Moraine

Soil Type	Silty-loams, silty-sands and sandy-silts with varying amounts of pebbles, gravel, and boulders.
Pavement Support	Rated as good to occasionally excellent in the GM-24 and GMX-24 areas, fair to good in the better GM-42 areas, and poor to very poor in some of the GM-46 areas. In the last-mentioned case, the use of subbase is advisable where other than light traffic is expected.
AASHTO Classification	A-4, A-6

GO – 4 - Stratified Glacial Outwash

Soil Type	The surface soil is a silt or sandy silt with noticeable organic accumulation, while the subsurface soil is usually silty sand, sand, gravelly sand or sandy gravel. The GO-4 soils, because of their low elevation and the predominance of silt in their upper horizons, exhibit imperfect to poor surface drainage and a shallow depth to the ground water-table.
Pavement Support	poor to fair
AASHTO Classification	A-4

GS – 4, 42 and 46 Stratified Drift

Soil Type	Silty sands, silty gravels, sandy gravels, and gravelly sands.
Pavement Support	Usually rated poor to very poor in the GS-4 and GS-46 areas. It is advisable to use subbase where other than light traffic is expected.
AASHTO Classification	A-4, A-2-4, A-6

Ib – 4, 46 - Basalt Flows

Soil Type	Silt, silty clay and clay; often containing appreciable amounts of basalt fragments.
Pavement Support	Fair under conditions of good drainage and light axle loads; poor to very poor under adverse drainage and heavy axle loads. The use of subbase is advisable.
AASHTO Classification	A-4, A-6

Is –46 Basalt Flows

Soil Type	Silts and silty clays, with frequent gravelly phases reflecting the presence of large quantities of partially disintegrated diabase. Soil classifications are quite erratic in the steeper areas due to the variable bedrock depths and variation of profile development. True water-tables are very deep, although perched water-tables may be expected in the elevated, flat areas.
Pavement Support	Fair under conditions of good drainage and light traffic; poor to very poor under more adverse drainage and traffic conditions. The use of subbase is advisable.
AASHTO Classification	A-4 to A-6

M –46 - Unconsolidated Marine Formations

Soil Type	Silt, clayey silt and silty clay with small amounts of intermixed gravel in some areas. Surface drainage is usually imperfect to poor as a result of the overall level ground surface. The fine texture of the soil is responsible for imperfect to poor subsurface drainage. Where these materials occur on level or low areas, the ground water-table is frequently at, or near, the ground surface.
Pavement Support	Poor to very poor. Raised grade lines and the use of subbase is advisable.
AASHTO Classification	A-4, A-6

M –67 - Unconsolidated Marine Formations

Soil Type	Clay and silty clay overlaid by a thin cover of silt with some intermixed gravel particles. Because of their low elevations, these areas usually have imperfect to poor surface drainage with a shallow depth to the ground water-table. Internal drainage is also poor because of heavy soil textures and the shallow ground water-table.
Pavement Support	Very poor. Raised grade lines and the use of subbase is advisable.
AASHTO Classification	A-4, A-6, A-7-5, A-7-6

MC –6 - Marine Deposit (Marl)

Soil Type	Silt, clayey silt and silty clay overlying silty sand. Usually silty clay and clay are encountered with depth. Internal drainage is characteristically poor.
Pavement Support	Very poor to imperfect. Subbase is particularly necessary at locations where cuts or low areas result In glauconitic clay and silt occurring close to or at the grade line.
AASHTO Classification	The A horizon is usually soil group A-. The B horizon is mostly group A-6 and with depth soil groups A-4, A-2-4 and even A-3 are present.

MMgC –46 - Gneissic Region

Soil Type	Silts, silty clays, and silty sands. The silty sands occur most frequently in the C-horizon, while the silty clays occur almost exclusively in the B-horizon. Water-tables are deep.
Pavement Support	The B-horizons of these soils provide fair to good support for light traffic and poor support for heavier traffic. The C-horizons provide good to excellent support under light traffic, and fair to good support under heavy traffic. When making cuts, seepage and unequal pavement support should be anticipated where the subgrade surface changes from B to C-horizon.
AASHTO Classification	A-2-4 to A-4 to A-6

MV – 47- Glauconitic Marine Formations

Soil Type	Silty and clayey sand interbedded with sandy clay.
Pavement Support	Imperfect to poor. A combination of raised grade line, use of subbase and adequate drainage structure is advisable.
AASHTO Classification	A-2-4, A-4, A-6, A-7-5 and A-7-6

MX – 67 - Stratified Materials (gravel, sand, silt and clay)

Soil Type	Clay with varying amounts of silt and sand scattered throughout the profile. Gravel stringers and layers are present in areas mapped as MX-67. Surface drainage varies from poor in areas bordering stream courses to good in the higher areas between streams. Internal drainage is very poor.
Pavement Support	Very poor. The use of base, subbase and adequate drainage facilities is advisable.
AASHTO Classification	A-4, A-6, A-7-5, A-7-6

Sa – 4 - Triassic Argillite

Soil Type	Silt, except in poorly drained areas where silty clay develops. Internal drainage is impeded by the moderately fine textures and the shallow depth to bedrock. Generally, the ground-water table in Sa-4 areas is quite deep, but the possibility of a perched water table should be anticipated in low areas.
Pavement Support	Satisfactory for light axle loads; poor to very poor under heavy, repeated axle loads.
AASHTO Classification	A-4

Sc – 46 - Unglaciated Conglomerate

Soil Type	Silt to clayey silt with many quartzite cobbles included. In the gneissic phase, a variety of pebbles, cobbles and boulders is imbedded in a dull brown to reddish-brown material. The ground water-table is fairly deep.
Pavement Support	Variable, depending upon the soil characteristics in the specific locality under consideration.
AASHTO Classification	A-4 and A-6.

Sh – 4 - Brunswick Shale

Soil Type	Silts with silty clays in the depressions. In depressions, drainage is usually impeded. Due to the predominance of silt sizes and the relatively open structure of the underlying bedrock, internal drainage is usually fair. Except in depressions, depths to water-table usually exceed 10 feet.
Pavement Support	Fair under lightly trafficked roads; poor to very poor under medium to heavily trafficked roads. In the latter case, the use of subbase is desirable. An important detrimental characteristic of these materials is a tendency to pump freely when saturated.
AASHTO Classification	A-4

Sh – 67- Brunswick Shale

Soil Type	Clayey silt, silty clay and clay. Poor surface and internal drainage with a high ground water-table.
Pavement Support	Poor to very poor. Raised grade lines and the use of subbase is advisable. Detrimental frost action and a tendency for the soil to pump freely when saturated are characteristics associated with this map unit.
AASHTO Classification	A-4 to A-7-6

Shl – Limestone and Shale

Soil Type	Usually a very thin mantle of red-brown silt. Mapped as non-soil. Good surface runoff because of the steep slopes. Downward percolation would undoubtedly be at a minimum and mainly confined to fracture and cleavage planes.
Pavement Support	Poor because of the friable nature, lack of permeability, and shallow depth to bedrock.
AASHTO Classification	--

Sl – 47 - Limestone

Soil Type	Silty clays and silts
Pavement Support	Generally poor.
AASHTO Classification	A-4 to A-7-5

MTM – Marine Tidal Marsh

Soil Type	The upper 2 to 15 feet is usually a highly compressible mixture of dark gray-brown to black, decomposed organic matter, clay and silt. This material is much deeper in areas influenced by main drainage ways. Beneath this soft liquid material is light gray sand and gravel.
Pavement Support	Inadequate. The physical characteristics of the tidal marsh deposits make them extremely susceptible to consolidation. The possibility of large settlements of embankments and other structures must be anticipated. A thorough investigation of proposed sites should be made prior to the design and construction of embankments, bridge foundations and other structures.
AASHTO Classification	A-7

Z - Swamp

Soil Type	z - Swamp: Used without additional designation. Denotes swampy areas where the ground-water table is at the ground surface most of the year, and the surface or near-surface soils are generally high In organic content. The characteristics of the material underlying the organic surface layers usually resemble, in all Important aspects, those of the surrounding map units. The map symbol Z usually includes poorly drained areas at the heads of streams, along streams above tidal influence and areas bordering tidal marsh.
Pavement Support	--
AASHTO Classification	--

F - Filled or Made Land

Soil Type	Filled or Made Land: Used without additional designation. Denotes areas where the original ground surface Is covered by varying depths of fill material. The fill may have been placed to cover unsatisfactory soil conditions or to raise the ground surface above a high ground-water table. The fill material Is frequently Industrial or municipal waste. The symbol F is also used to denote areas of cranberry bogs. This type of agricultural development has Influenced soil conditions and the relative height of the ground-water table. Much fill In Atlantic County has been placed on tidal marsh areas to raise the ground surface to the level of adjacent land surfaces, which are often sand bars. Most of this type of fill consists of hydraulically placed sand.
Pavement Support	--
AASHTO Classification	--

Treating Problematic Subgrade and Subbase Soils

(based on FHWA Geotechnical Aspects of Pavements)

Problematic soils can be treated using a variety of methods. Improvement techniques that can be used to improve the strength and reduce the climatic variation of the foundation on pavement performance include:

1. **Improvement of subsurface drainage. Removing water from the pavement structure should always be considered.**
2. Removal and replacement with better materials (e.g., thick granular layers).
3. Mechanical stabilization using thick granular layers.
4. Mechanical stabilization of weak soils with geosynthetics (geotextiles and geogrids) in conjunction with granular layers.
5. Lightweight fill.
6. Chemical Stabilization of weak soils and frost susceptible soils with admixtures.
7. Soil encapsulation.

When frost-susceptible soils are encountered, consideration should be given to the following alternatives for improving the foundation or supporting subgrade:

1. Remove the frost-susceptible soil (generally for groups F3 and F4) and replace with select non-frost susceptible borrow to the expected frost depth penetration.
2. Place and compact select non-frost-susceptible borrow materials to a thickness or depth to prevent subgrade freezing for frost susceptible soil groups F2, F3, and F4.
3. Remove isolated pockets of frost-susceptible soils to eliminate abrupt changes in subgrade conditions.
4. Stabilize the frost-susceptible soil by eliminating the effects of soil fines by three processes: a) mechanically removing or immobilizing by means of physical-chemical means, such as cementitious bonding, b) effectively reducing the quantity of soil moisture available for migration to the freezing plane, as by essentially blocking off all migratory passages, or c) altering the freezing point of the soil moisture.

   a. Cementing agents, such as Portland cement, bitumen, lime, and lime-flyash have been used to address these issues. These agents effectively remove individual soil particles by bonding them together, and also act to partially remove capillary passages, thereby reducing the potential for moisture movement. Care must be taken when using lime and lime-flyash mixtures with clay soils in seasonal frost areas since the resulting flocculated material may take on the granular nature of a silt-like material. The secondary treatment of the lime treated subgrade material with cement can reduce the susceptibility.

   b. Soil moisture available for frost heave can be mitigated through the installation of deep drains and/or a capillary barrier such that the water table is maintained at a sufficient depth to prevent moisture rise in the freezing zone. Capillary barriers can consist of either an open graded gravel layer sandwiched between two geotextiles, or a horizontal geocomposite drain. The installation of a capillary barrier requires the removal of the frost susceptible material to a depth either below frost penetration or sufficiently significant to reduce the influence of frost heave on the pavement. The capillary break must be drained. The frost susceptible soil can then be replaced and compacted above the capillary barrier to the required subgrade elevation.

5. Increase the pavement structural layer thickness to account for strength reduction in the subgrade during the spring-thaw period for frost-susceptible groups F1, F2, and F3.

Pavement design for frost action often determines the required overall thickness of flexible pavements and the need for additional select material beneath both rigid and flexible pavements. Three design approaches have been used for pavement in seasonal frost areas:

- The Complete Protection approach—requires non-frost susceptible materials for the entire depth of frost (e.g., treatment methods 1, 2, and 3 above).
- Limited Subgrade Frost Penetration approach—permits some frost penetration into the subgrade, but not enough to allow unacceptable surface roughness to develop.

• Reduced Subgrade Strength approach—allows more frost penetration into the subgrade, but provides adequate strength during thaw weakened periods. AASHTO 1993 (Appendix C) provides procedures and graphs to predict the direct effect of frost heave on serviceability loss and is treated with respect to the differential effects on the longitudinal profile of the road surface. If the frost is anticipated to be relatively uniform, then the procedures do not apply.

For the most part, local frost-resistant design approaches have been developed from experience, rather than by application of some rigorous theoretical computational method. A more rigorous method is available in the NCHRP 1-37A design procedure to reduce the effects of seasonal freezing and thawing to acceptable limits. The Enhanced Integrated Climatic Model is used to determine the maximum frost depth for the pavement system at a particular location. Various combinations of layer thicknesses and material types can be evaluated in terms of their impact on the maximum frost depth and total amount of base and select materials necessary to protect the frost susceptible soils from freezing.

Subgrade (and Subbase) Material Improvement and Strengthening

Proper treatment of problem soil conditions and the preparation of the foundation are extremely important to ensure a long-lasting pavement structure that does not require excessive maintenance. Some agencies have recognized certain materials simply do not perform well, and prefer to remove and replace such soils (e.g., a state specification dictating that frost susceptible loess cannot be present in the frost penetration zone). However, in many cases, this is not the most economical or even desirable treatment (e.g., excavation may create disturbance, plus additional problems of removal and disposal). Stabilization provides an alternate method to improve the structural support of the foundation for many of the subgrade conditions presented in the previous section. In all cases, the provision for a uniform soil relative to textural classification, moisture, and density in the upper portion of the subgrade cannot be over-emphasized. This uniformity can be achieved through soil sub-cutting or other stabilization techniques. Stabilization may also be used to improve soil workability, provide a weather resistant work platform, reduce swelling of expansive materials, and mitigate problems associated with frost heave. In this section, alternate stabilization methods will be reviewed, and guidance will be presented for the selection of the most appropriate method.

Objectives of Soil Stabilization

Soils that are highly susceptible to volume and strength changes can cause severe roughness and accelerate the deterioration of the pavement structure in the form of increased cracking and decreased ride quality when combined with truck traffic. Generally, the stiffness (in terms of resilient modulus) of some soils is highly dependent on moisture and stress state. In some cases, the subgrade soil can be treated with various materials to improve the strength and stiffness characteristics of the soil. Stabilization of soils is usually performed for three reasons:

1. As a construction platform to dry very wet soils and facilitate compaction of the upper layers-for this case, the stabilized soil is usually not considered as a structural layer in the pavement design process.
2. To strengthen a weak soil and restrict the volume change potential of a highly plastic or compressible soil-for this case, the modified soil is usually given some structural value or credit in the pavement design process.
3. To reduce moisture susceptibility of fine grain soils.

Blending of Gravel and Sand-size material can improve the soil engineering (textural) properties of problematic Subgrade and Subbase materials.

Stabilization with admixtures, such as lime, cement, and asphalt, have been mixed with subgrade soils used for controlling the swelling and frost heave of soils and improving the strength characteristics of unsuitable soils. For admixture stabilization or modification of cohesive soils, hydrated lime is the most widely used. Lime is applicable in clay soils (CH and CL type soils) and in granular soils containing clay binder (GC and SC), while Portland cement is more commonly used in non-plastic soils. Lime reduces the Plasticity Index (PI) and renders a clay soil less sensitive to moisture changes. The use of lime should be considered whenever the PI of the soil is greater than 12. Lime stabilization is used in many areas of the U.S. to obtain a good construction platform in wet weather above highly plastic clays and other fine-grained soils. **It is important to note that changing the physical properties of a soil through chemical stabilization can produce a soil that is susceptible to frost heave.** Following is a brief description of the characteristics of stabilized soils followed by the treatment procedures.

Characteristics of Stabilized Soils

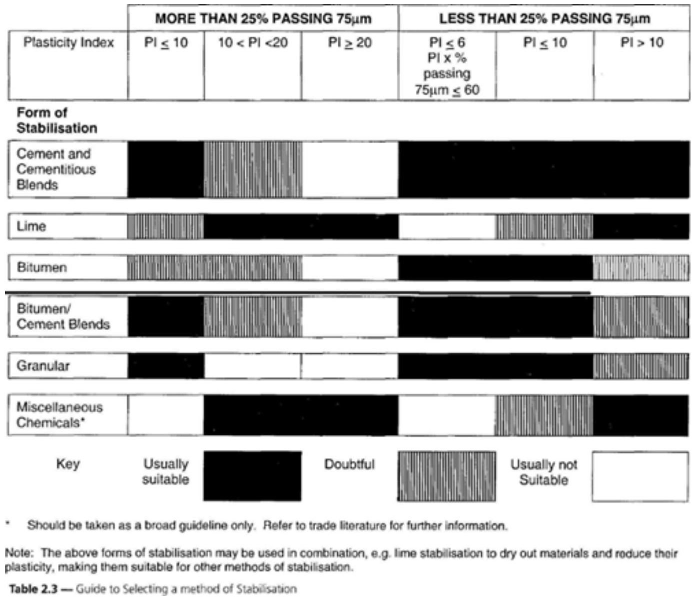
The improvement of subgrade or unbound aggregate by application of a stabilizing agent is intended to cause the improvements outlined above (i.e., construction platform, subgrade strengthening, and control of moisture). These improvements arise from several important mechanisms that must be considered and understood by the pavement designer. Admixtures used as subgrade stabilizing agents may fill or partially fill the voids between the soil particles. This reduces the permeability of the soil. Reduction of permeability may be relied upon to create a waterproof surface to protect underlying, water sensitive soils from the intrusion of surface water. This mechanism must be accompanied by other aspects of the geometric design into a comprehensive system. The reduction of void spaces may also tend to change the volume change under shear from a contractive to a dilative condition. The admixture type stabilizing agent also acts by binding the particles of soil together, adding cohesive shear strength and increasing the difficulty with which particles can move into a denser packing under load. Particle binding serves to reduce swelling by resisting the tendency of particles to move apart. The particles may be bound together by the action of the stabilizing agent itself (as in the case of asphalt cement), or may be cemented by chemical reaction between the soil and stabilizing agent (as in the case of lime or Portland cement). Additional improvement can arise from other chemical-physical reactions that affect the soil fabric (typically by flocculation) or the soil chemistry (typically by cation exchange). The down side of admixtures is that they require up front lab testing to confirm their performance and very good field control to obtain a uniform, long lasting product, as outlined later in this section. There are also issues of dust control and weather dependency, with some methods that should be carefully considered in the selection of these methods.

The zone that may be selected for improvement depends upon a number of factors. Among these are the depth of soft soil, anticipated traffic loads, the importance of the transportation network, constructability, and the drainage characteristics of the geometric design and the underlying soil. When only a thin zone and/or short roadway length is subject to improvement, removal and replacement will usually be the preferred alternative by most agencies, unless a suitable replacement soil is not economically available. Note that in this context, the use of the qualitative term "thin" is intentional, as the thickness of the zone can be described as thick or thin, based primarily on the project economics of the earthwork requirements and the depth of influence for the vehicle loads.

Admixture Stabilization

As previously indicated, there are a variety of admixtures that can be mixed with the subgrade or Subbase material to improve its performance. The various admixture types are shown in Table 15, along with initial guidance for evaluating the appropriate application of these methods. Following is a general overview of each method, followed by a generalized outline for determining the optimum admixture content requirements.

Table 15. Guide for selection of admixture stabilization method(s) (Austroads, 1998).



Lime Treatment

Lime treatment or modification consists of the application of 1 - 3% hydrated lime to aid drying of the soil and permit compaction. As such, it is useful in the construction of a "working platform" to expedite construction. Lime modification may also be considered to condition a soil for follow-on stabilization with cement or asphalt. Lime treatment of subgrade soils is intended to expedite construction, and no reduction in the required pavement thickness should be made.

Lime may also be used to treat expansive soils. Expansive soils as defined for pavement purposes are those that exhibit swell in excess of 3%. Expansion is characterized by heaving of a pavement or road when water is imbibed in the clay minerals. The plasticity characteristics of a soil often are a good indicator of the swell potential, as indicated in the following table. If it has been determined that a soil has potential for excessive swell, lime treatment may be appropriate. Lime will reduce swell in an expansive soil to greater or lesser degrees, depending on the activity of the clay minerals present. The amount of lime to be added is the minimum amount that will reduce swell to acceptable limits. Procedures for conducting swell tests are indicated in the ASTM D 1883 CBR test and detailed in ASTM D 4546.

Swell potential of soils (Joint Departments of the Army & Air Force, 1994).		
Liquid Limit	Plasticity Index	Potential Swell
> 60	> 35	High
50 - 60	25 - 35	Marginal
< 50	< 25	Low

The depth to which lime should be incorporated into the soil is generally limited by the construction equipment used. However, 0.6 - 1 m (2 - 3 ft) generally is the maximum depth that can be treated directly without removal of the soil.

Lime Stabilization

Lime or pozzolanic stabilization of soils improves the strength characteristics and changes the chemical composition of some soils. The strength of fine-grained soils can be significantly improved with lime stabilization, while the strength of coarse-grained soils is usually moderately improved. Lime has been found most effective in improving workability and reducing swelling potential with highly plastic clay soils containing montmorillonite, illite, and kaolinite. Lime is also used to reduce the water content of wet soils during field compaction. In treating certain soils with lime, some soils are produced that are subject to fatigue cracking.

Lime stabilization has been found to be an effective method to reduce the volume change potential of many soils. However, **lime treatment of soils can convert the soil that shows negligible to moderate**

frost heave into a soil that is highly susceptible to frost heave, acquiring characteristics more typically associated with silts. It has been reported that this adverse effect has been caused by an insufficient curing period. Adequate curing is also important if the strength characteristics of the soil are to be improved.

The most common varieties of lime for soil stabilization are hydrated lime [Ca(OH)2], quicklime [CaO], and the dolomitic variations of these high-calcium limes [Ca(OH)2×MgO and CaO×MgO]. While hydrated lime remains the most commonly used lime stabilization admixture in the U.S., use of the more caustic quicklime has grown steadily over the past two decades. Lime is usually produced by calcining2 limestone or dolomite, although some lime-typically of more variable and poorer quality-is also produced as a byproduct of other chemical processes.

For lime stabilization of clay (or highly plastic) soils, the lime content should be from 3 - 8% of the dry weight of the soil, and the cured mass should have an unconfined compressive strength of at least 0.34 MPa (50 psi) within 28 days. The optimum lime content should be determined with the use of unconfined compressive strength and the Atterberg limits tests on laboratory lime-soil mixtures molded at varying percentages of lime. As discussed later in this section, pH can be used to determine the initial, near optimum lime content value. The pozzolanic strength gain in clay soils depends on the specific chemistry of the soil - *e.g.*, whether it can provide sufficient silica and alumina minerals to support the pozzolanic reactions. Plasticity is a rough indicator of reactivity. A plasticity index of about 10 is commonly taken as the lower limit for suitability of inorganic clays for lime stabilization. The lime-stabilized subgrade layer should be compacted to a minimum density of 95%, as defined by AASHTO T99.

These are the result of several chemical processes that occur after mixing the lime with the soil. Hydration of the lime absorbs water from the soil and causes an immediate drying effect. The addition of lime also introduces calcium (Ca+2) and magnesium (Mg+2) cations that exchange with the more active sodium (Na+) and potassium (K+) cations in the natural soil water chemistry; this cation exchange reduces the plasticity of the soil, which, in most cases, corresponds to a reduced swell and shrinkage potential, diminished susceptibility to strength loss with moisture, and improved workability. The changes in the soil-water chemistry also lead to agglomeration of particles and a coarsening of the soil gradation; plastic clay soils become more like silt or sand in texture after the addition of lime. These drying, plasticity reduction, and texture effects all occur very rapidly (usually with 1 hour after addition of lime), provided there is thorough mixing of the lime and the soil.

Cement Stabilization

Portland cement is widely used for stabilizing low-plasticity clays, sandy soils, and granular soils to improve the engineering properties of strength and stiffness. Increasing the cement content increases the quality of the mixture. At low cement contents, the product is generally termed cement-modified soil. A cement-modified soil has improved properties of reduced plasticity or expansive characteristics and reduced frost susceptibility. At higher cement contents, the end product is termed soil-cement or cement-treated base, subbase, or subgrade.

For soils to be stabilized with cement, proper mixing requires that the soil have a PI of less than 20% and a minimum of 45% passing the 0.425 mm (No. 40) sieve. However, highly plastic clays that have been pretreated with lime or flyash are sometimes suitable for subsequent treatment with Portland cement. For cement stabilization of granular and/or nonplastic soils, the cement content should be 3 - 10% of the dry weight of the soil, and the cured material should have an unconfined compressive strength of at least 1 MPa (150 psi) within 7 days. The Portland cement should meet the minimum requirements of AASHTO M 85. The cement-stabilized subgrade should be compacted to a minimum density of 95% as defined by AASHTO M 134.

Type I normal Portland cement has been used successfully for stabilization of soils. At the present time, Type II cement has largely replaced Type I cement as greater sulfate resistance is obtained, while the cost is often the same. High early strength cement (Type III) has been found to give a higher strength in some soils. Type III cement has a finer particle size and a different compound composition than do the other cement types. Chemical and physical property specifications for Portland cement can be found in ASTM C 150. The presence of organic matter and/or sulfates may have a deleterious effect on soil cement. Tests are available for detection of these materials and should be conducted if their presence is suspected.

Conclusions and Recommendations

The Subbase soil layer has traditionally been used to provide a less-frost susceptible or non-frost susceptible layer in the pavement structure to force the frost penetration zone to go deeper into the pavement before it can facilitate the formation of ice lenses. The Subbase materials were selected to be less expensive than the aggregate base courses with a gradation and soil classification that promoted permeability and grain size distribution that would minimize capillary migration of moisture from the ground-water table.

To minimize the amount of frost damage, the total pavement thickness was calculated to be a minimum of 75% of the historic maximum frost depth for the region of the state. Since the annual frost penetration varies for year to year, the historic maximum frost depth for the region of the state was used to ensure that the non-frost susceptible pavement material in the pavement structure would not form ice lenes within the pavement structure most of the time. The thickness of the Subbase layer was usually set equal to the thickness of the aggregate base.

The second use of the Subbase soil layer was to distribute the wheel loads at the pavement surface to protect the subgrade soil layer from excessive strains that would promote rutting. The total pavement structural number including the Subbase layer is used to ensure the pavement's performance over the design period.

Realizing that there is a finite amount of acceptable soil materials for Subbase layers; other less desirable soils may need to be used for Subbase soil materials. Soil stabilization, soil grids, soil encapsulation and other techniques can be used to improve the engineering properties of these materials to maintain the overall pavement performance. Since the underlying need for the of the Subbase layer is the protection of the frost-susceptible or weak subgrade soil layers, soil stabilization, soil grids, soil encapsulation and other techniques can be also used to improve the engineering properties of the subgrade materials to maintain the overall pavement performance. While the "improved" subgrade layer may not be considered part of the pavement structure, it does reduce the structural requirements of the pavement structure.

It is recommended that the discussions in this report be considered in addressing the use of Subbase layers and consideration for stabilization techniques all aimed at maintaining the overall pavement performance over the pavement design period.

Forta Fi Fiber Mixture Evaluation

Submitted to:

New Jersey Department of Transportation (NJDOT)
Bureau of Materials

Conducted by:

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SCOPE OF WORK

The scope of the project encompassed evaluating the asphalt mixture performance of two 12.5M64 asphalt mixtures; 1) One produced with conventional/approved materials and 2) One produced with conventional/approved materials plus Forta Fi fibers. The laboratory mixture characterization was intended to determine if the material properties increased/decreased/stayed the same with the inclusion of the fibers.

The asphalt mixture was produced by Trap Rock Industries at their Kingston, NJ asphalt plant in October 2012 under NJDOT project MRRC #C-204. The asphalt mixture utilized a PG64-22 asphalt binder produced by NuStar Asphalt from Paulsboro, NJ. The Quality Control data forms from production can be found in Appendix A.

Laboratory testing consisted of mixture testing that focused on the stiffness, permanent deformation, and fatigue cracking performance. The asphalt mixture testing consisted of:

- Dynamic Modulus (AASHTO TP79);
- Rutting Evaluation
    - Asphalt Pavement Analyzer (AASHTO T340)
    - Asphalt Mixture Performance Tester (AASHTO TP79)
- Fatigue Cracking Evaluation
    - Flexural Beam Fatigue (AASHTO T321)\
        - Short term and long term aged conditions
    - Overlay Tester (NJDOT B-10)
        - Short term and long term aged conditions

MIXTURE TESTING

The asphalt mixture produced by Trap Rock Industries consisted of a 12.5mm Superpave mixture designed using a gyration level of 75 gyrations and containing a PG64-22 asphalt binder. The Forta Fi fibers were added during the mixing phase in accordance to manufacturer's recommendations. After production and just before leaving the asphalt plant, the asphalt mixtures were sampled from the back of the delivery trucks and placed in 5-gallon metal containers. The containers were sealed and delivered to the Rutgers Asphalt Pavement Laboratory, where the sample containers were stored until sample fabrication and testing took place.

Prior to testing, the asphalt mixtures were reheated to compaction temperature and then compacted into the respective performance test specimens. For this study, test specimens were compacted to air void levels ranging between 6 and 7%.

Dynamic Modulus (AASHTO TP79)

Dynamic modulus and phase angle data were measured and collected in uniaxial compression using the Simple Performance Tester (SPT) following the method outlined in AASHTO TP79, *Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT)* (Figure 1). The data was collected at three temperatures; 4, 20, and 35°C using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz. Test specimens were evaluated under short-term aged conditions. Since the mixtures evaluated in the study were plant produced, it was assumed that these materials already represented short-term aged conditions.

Figure 1 – Photo of the Asphalt Mixture Performance Tester (AMPT)

The collected modulus values of the varying temperatures and loading frequencies were used to develop Dynamic Modulus master stiffness curves and temperature shift factors using numerical optimization of Equations 1 and 2. The reference temperature used for the generation of the master curves and the shift factors was 20°C.

$$\log|E^*| = \delta + \frac{(Max - \delta)}{1 + e^{\beta + \gamma\left\{\log \omega + \frac{\Delta E_a}{19.14714}\left[\left(\frac{1}{T}\right) - \left(\frac{1}{T_r}\right)\right]\right\}}} \tag{1}$$

where:

$|E^*|$ = dynamic modulus, psi
ω_r = reduced frequency, Hz
Max = limiting maximum modulus, psi
δ, β, and γ = fitting parameters

$$\log[a(T)] = \frac{\Delta E_a}{19.14714}\left(\frac{1}{T} - \frac{1}{T_r}\right) \tag{2}$$

where:

$a(T)$ = shift factor at temperature T

T_r = reference temperature, °K

T = test temperature, °K

ΔE_a = activation energy (treated as a fitting parameter)

Figure 2 shows the master stiffness curves for the short-term aged mixtures. The test results show that both mixtures have very similar stiffness properties at the short-term aged condition over the range of temperatures and loading frequencies tested.

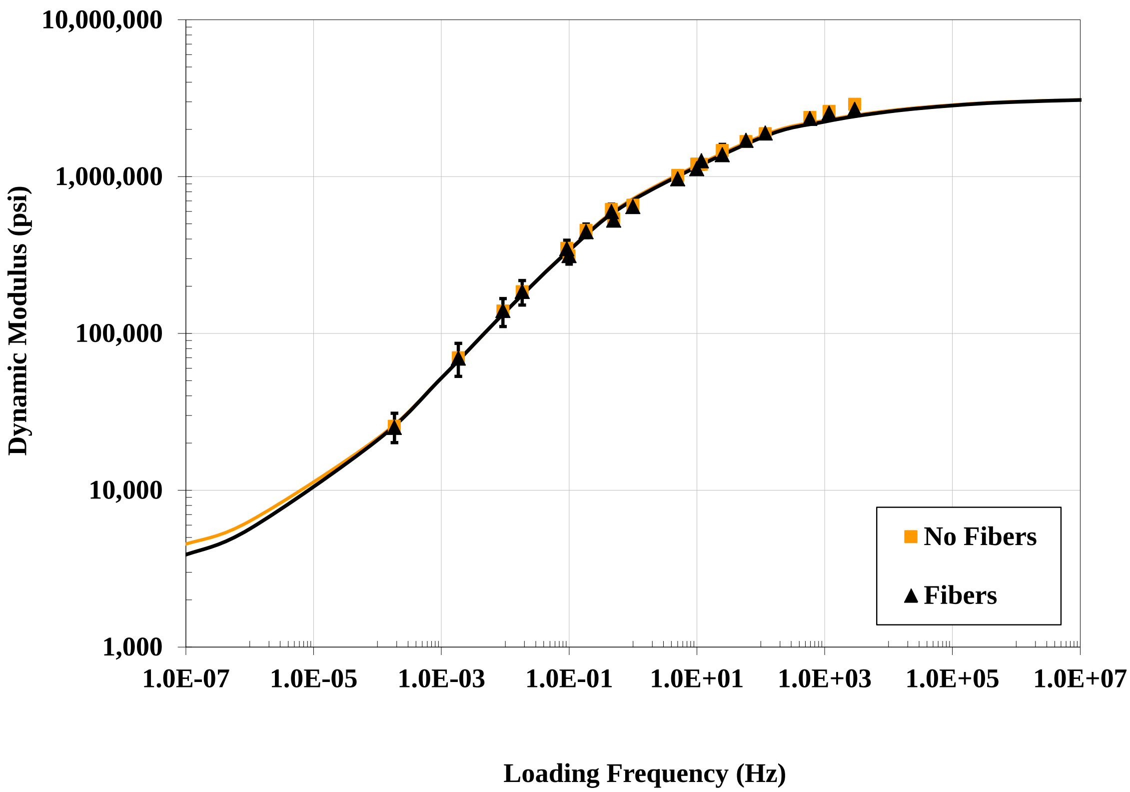

Figure 2 – Dynamic Modulus (E*) Master Stiffness Curves for Short-Term Aged (STOA) Conditions for 12.5M64 with and without Fibers

Rutting Evaluation

The rutting potential of the asphalt mixtures were evaluated in the study using two test procedures; 1) The Asphalt Pavement Analyzer (AASHTO T340) and 2) The Repeated Load – Flow Number (AASHTO TP79).

<u>Asphalt Pavement Analyzer (APA)</u>

Compacted asphalt mixtures were tested for their respective rutting potential using the Asphalt Pavement Analyzer (APA) in accordance with AASHTO T340, *Determining*

Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA). Prior to testing, the samples were conditioned for a minimum of 4 hours at the test temperature of 64^{o}C. The samples are tested for a total of 8,000 cycles using a hose pressure of 100 psi and wheel load of 100 lbs.

The APA rutting results for 12.5M64 mixtures with and without fibers is shown in Figure 3. The results indicate that the fiber modified asphalt mixture achieved a better resistance to rutting in the asphalt pavement analyzer. However, it should be noted that the final results are rather close to one another (2.70 mm and 3.14 mm, respectively).

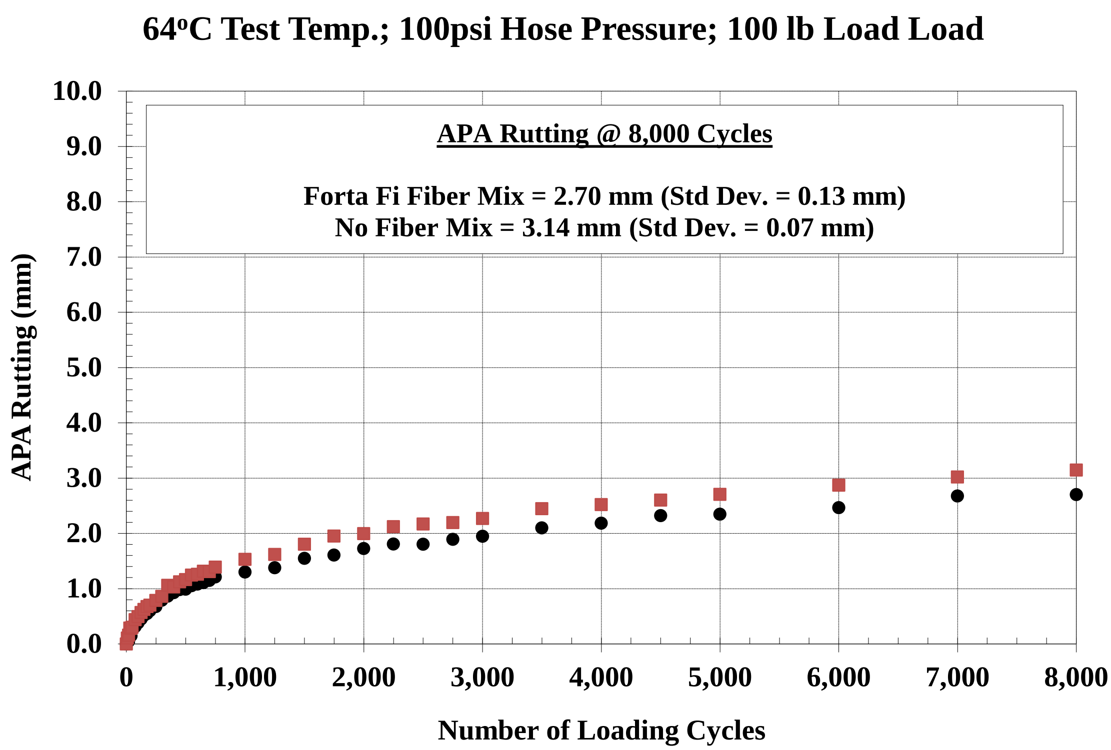

Figure 3 – Asphalt Pavement Analyzer (APA) Rutting Results of 12.5M64 Mixtures with and without Fibers

Repeated Load – Flow Number Test

Repeated Load permanent deformation testing was measured and collected in uniaxial compression using the Simple Performance Tester (SPT) following the method outlined in AASHTO TP79, *Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt (HMA) Using the Asphalt Mixture Performance Tester (AMPT)*. The unconfined repeated load tests were conducted with a deviatoric stress of 600 kPa and a test temperature of 54.4^{o}C, which corresponds to New Jersey's average 50% reliability high pavement temperature at a depth of 25 mm according the LTPPBind 3.1 software. These testing parameters (temperature and applied stress) conform to the recommendations currently proposed in NCHRP Project 9-33, *A Mix Design Manual for Hot Mix Asphalt*.

Testing was conducted until a permanent vertical strain of 5% or 10,000 cycles was obtained.

The test results for the 12.5M64 mixtures with and without fibers are shown in Table 1. The Flow Number results indicate that on average the 12.5M64 with fibers mixture resulted in a better resistance to permanent deformation than the mixture with no fibers. This is consistent with the APA results shown earlier.

Table 1 – Repeated Load – Flow Number Test Results

Mix Type	**Sample ID**	**Flow Number (cycles)**	**Cycle to Achieve 5% Strain**
Fibers	1	911	2,190
	2	1,007	3,555
	Average	***959***	***2,873***
No Fibers	1	668	1,824
	2	826	2,332
	Average	***747***	***2,078***

Under NCHRP Project 9-33, tentative criteria were established that recommended minimum Flow Number values for minimum ESAL levels. Table 2 contains these values, respectively. Based on the proposed criteria from the NCHRP research, both mixtures would be rated for ≥30 million ESAL’s.

Table 2 – Recommended Flow Number vs ESAL Level for HMA

Traffic Level, Million ESAL's	Minimum Flow Number
<3	N.A.
3 to < 10	53
10 to < 30	190
≥ 30	740

Fatigue Cracking Evaluation

The fatigue cracking properties of the mixtures were evaluated using two test procedures; 1) the Overlay Tester (NJDOT B-10) and 2) Flexural Beam Fatigue (AASHTO T321). The Overlay Tester evaluates how an asphalt mixture resists the propagation or growth of a crack once it has been initiated. Meanwhile, the Flexural Fatigue test evaluates how an asphalt mixture resists the crack from ever initiating. Both are important characteristics when evaluating the crack resistance properties of asphalt materials.

The asphalt mixtures were evaluated under both Short Term (STOA) and Long Term (LTOA) conditions. The STOA condition represents the early life of the asphalt mixture while the LTOA condition represents the asphalt mixture after 8+ years of service life according to research conducted during the Strategic Highway Research Program (SHRP). The age conditioning of the mixtures were conducted in the laboratory in accordance with AASHTO R30.

Overlay Tester (NJDOT B-10)

The Overlay Tester, described by Zhou and Scullion (2007), has shown to provide an excellent correlation to field cracking for both composite pavements (Zhou and Scullion, 2007; Bennert et al., 2009) as well as flexible pavements (Zhou et al., 2007). Figure 4 shows a picture of the Overlay Tester used in this study. Sample preparation and test parameters used in this study followed that of NJDOT B-10, *Overlay Test for Determining Crack Resistance of HMA*. These included:

- 25^{o}C (77^{o}F) test temperature;
- Opening width of 0.025 inches;
- Cycle time of 10 seconds (5 seconds loading, 5 seconds unloading); and
- Specimen failure defined as 93% reduction in Initial Load.

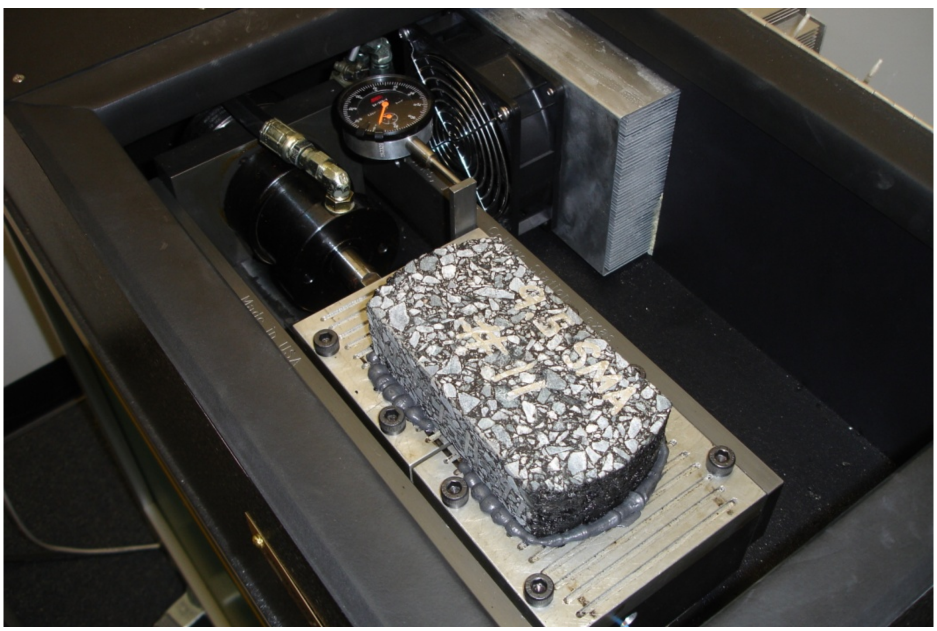

Figure 4 – Picture of the Overlay Tester (Chamber Door Open)

Figure 5 indicates that on average the 12.5M64 asphalt mixture without fibers achieved a greater resistance to crack propagation fatigue cracking than the 12.5M64 asphalt mixture with fibers when evaluated in the Overlay Tester. This was found at both the short term and long term aged conditions. A statistical analysis of the test data using the Student t-Test shows that the No Fiber mix is statistically Not Equal to the Fiber mix Overlay Tester results at a 95% confidence interval. Therefore, statistically, the No Fiber mix is better at resisting crack propagation than the Fiber mix at each aging condition, respectively.

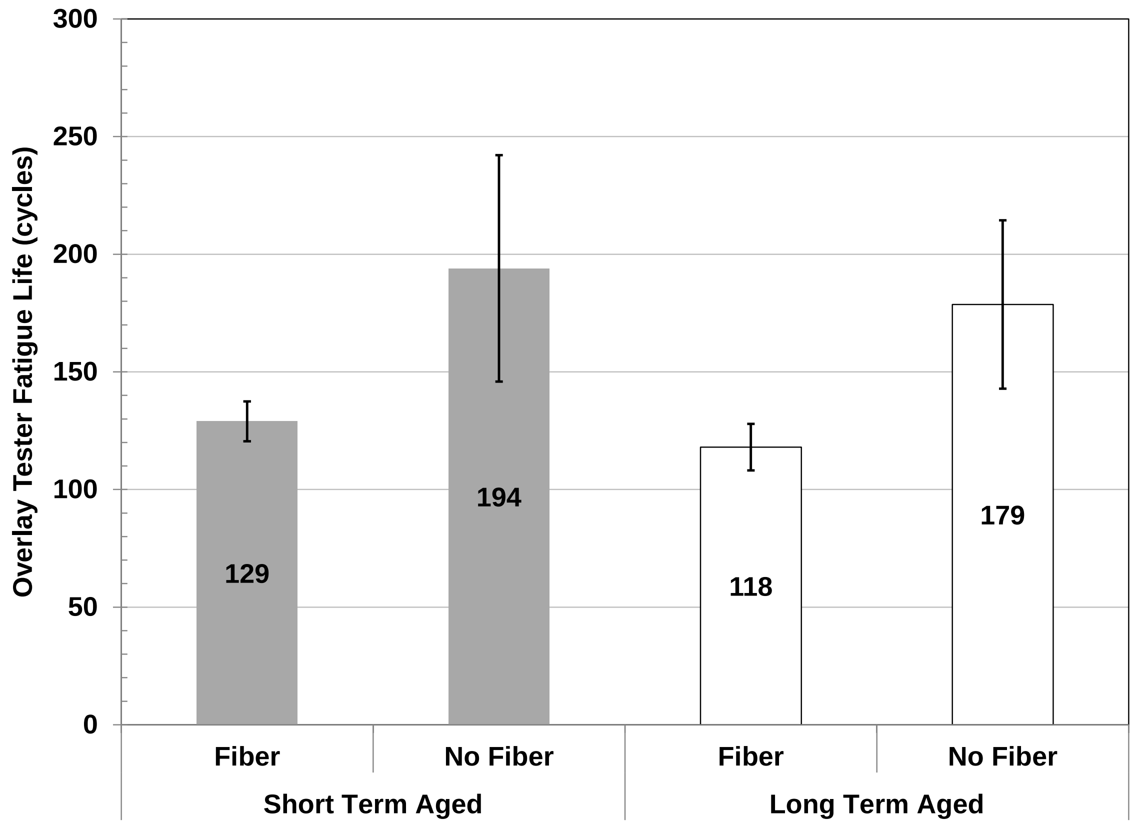

Figure 5 – Overlay Tester Results for 12.5M64 Asphalt Mixtures with and without Fibers – Short Term and Long Term Aged Conditions

Flexural Beam Fatigue (AASHTO T321)

Fatigue testing was conducted using the Flexural Beam Fatigue test procedure outline in AASHTO T321, *Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending*. The applied tensile strain levels used for the fatigue evaluation were; 500, 700, and 900 micro-strains. Samples were tested at short-term and long term aged conditions as mentioned earlier.

Samples used for the Flexural Beam Fatigue test were compacted using a vibratory compactor designed to compact brick samples of 400 mm in length, 150 mm in width, and 100 mm in height. After the compaction and aging was complete, the samples were trimmed to within the recommended dimensions and tolerances specified under AASHTO T321. The test conditions utilized were those recommended by AASHTO T321 and were as follows:

- Test temperature = 15^{o}C;
- Sinusoidal waveform;
- Strain-controlled mode of loading; and
- Loading frequency = 10 Hz;

The flexural beam fatigue test results for the 12.5M64 asphalt mixtures with and without fibers for the short-term and long term aged conditions is shown in Figure 6. The test results indicate that the 12.5M64 mixture without fibers achieved a greater resistance to fatigue crack initiation at both the short term and long term aged conditions, respectively. In fact, the Flexural Fatigue performance of the long term aged No Fiber mixture actually achieved the same Flexural Fatigue performance of the Fiber mixture at the short term aged conditions.

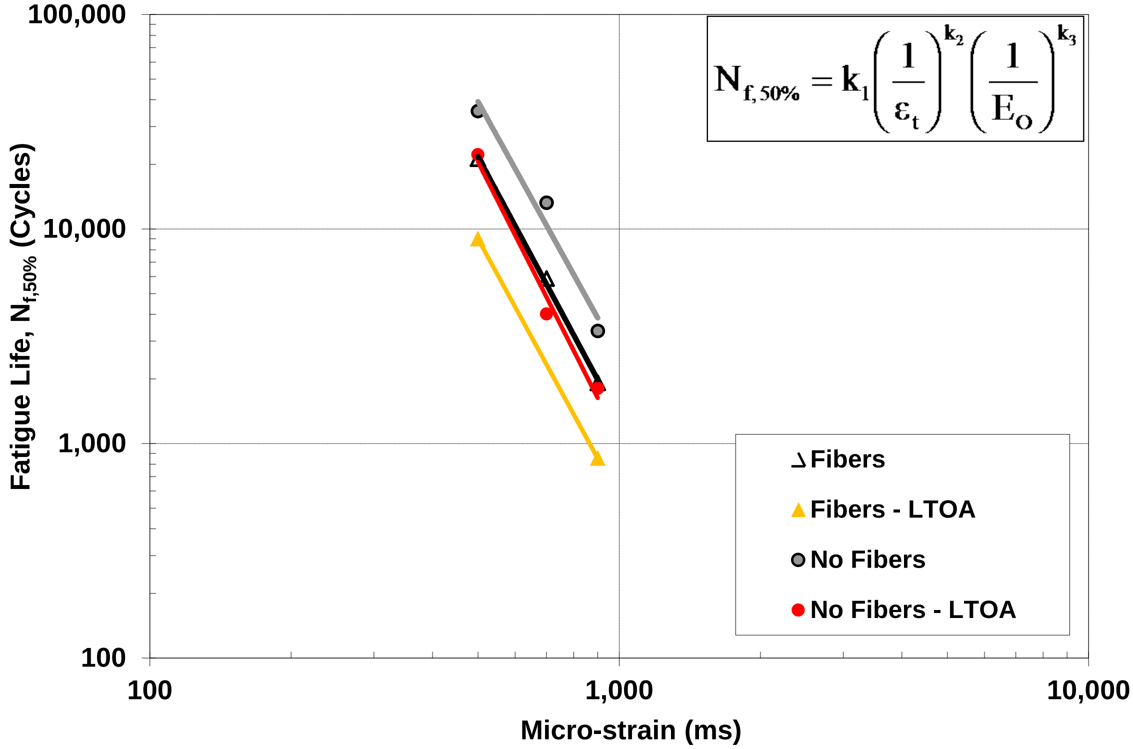

Figure 6 – Flexural Fatigue Results for 12.5M64 Asphalt Mixtures with and without Fibers – Short Term and Long Term Aged Conditions

CONCLUSIONS

A research program was developed to compare the performance of a 12.5M64 asphalt mixture with and without fibers. The test results indicated that:

- Both mixtures, with and without fibers, achieved very similar stiffness properties when measured using the Dynamic Modulus test (AASHTO TP79) at the test temperatures and frequencies shown earlier.
- On average, the 12.5M64 mixture with fibers resulted in a better resistance to rutting when compared to the Asphalt Pavement Analyzer and the AMPT Flow Number. However, when comparing the AMPT Flow Number results to the proposed NCHRP 9-33 criteria, it was found that both mixtures should be able to withstand traffic levels of 30 million ESAL's or greater.

- The 12.5M64 asphalt mixture without fibers was able to achieve a greater fatigue resistance when evaluated using the Flexural Fatigue (crack initiation) and the Overlay Tester (crack propagation). This occurred at both the short term and long term aged conditions.

Overall, it would appear that the general stiffness properties and permanent deformation resistance for the mixtures with and without fibers were similar. However, the fatigue cracking properties for the No Fiber mixture were found to be better than the Forta Fi Fiber mixture. Based on the limited testing conducted in the study, it is uncertain as to why the fatigue behavior was different. However, it is hypothesized that the addition of the fibers reduced the effective asphalt content of the mixture. The effective asphalt content is an asphalt mixture property commonly known to influence fatigue resistance and mixture durability. Additional testing would be required to verify this hypothesis.

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APPENDIX A – QC DATA

TRAP ROCK INDUSTRIES, INC.
IGNITION METHOD & GYRATORY TEST FOR COMPLIANCE

PROJECT :	NJDOT MAINT. CENTRAL C204	CONTRACTOR :	DELLA PELLO CONST. CO.	DATE :	10/19/12
PRODUCER :	TRAP ROCK INDUSTRIES	LOCATION :	KINGSTON PLANT#1	MIX NO. :	HMA 12.5M64W/FIBERS
		% AC Absorbed:	0.27	SER. NO.:	CO1DC0104

GYRATION LEVELS	Nini	Ndes
	7	75

Lot Sample No.			28					GYRATORY RESULTS TO Ndes		
Starting Temperature			538c			538c		Mix Temp.	310	310
Elapsed Time			45 mins.			45 mins.		Sp. Gr. Of Binder (Gb)	1.035	1.035
A-Sample Weight			1937.9					Specimen Ht. @ Ndes	117.2	
B-Sample Wgt. After Ignition *			1847.1					Specimen Ht. @ Nini	128.0	
C-Wgt. Loss (A-B)			90.8					Weight in Air	5306.6	
C-Wgt. Loss (Corrected for moisture)			90.3					Weight in Water	3280.6	
D-% Loss (C/A x 100)			4.66					Weight SSD	5312.2	
E-Temperature Compensation **			**			**		Bulk Sp. Gr.(Gmb)	**2.612**	
F-Calibration Factor (From Design)			0.17			0.17		Air Voids - %	**4.3**	
G-% Bitumen (D-E-F)	5.00		**4.48**			**-0.17**		% Gmm @ Ndes	**95.71**	
SIEVE SIZE		Wgt.	Ret. %	Pass %	Wgt.	Ret. %	Pass %	% Gmm@Nini (<90.5)	**87.64**	
2" (50.0mm)				100.0			100.0	Total Volume	38.284	
1-1/2" (37.5mm)				100.0			100.0	Effective AC	4.214	
1" (25.0mm)				100.0			100.0	Volume of AC	4.072	#VALUE!
3/4" (19.0mm)	100			100.0			100.0	% AC by Volume	10.635	
1/2" (12.5mm)	90-100	82.2	4.5	**95.5**			**100.0**	% VMA (min-14)	**14.9**	
3/8" (9.5mm)	0-90	246.7	13.4	**86.6**			**100.0**	% VFA	**71.3**	
No. 4 (4.75mm)		743.8	40.3	59.7			100.0	Dust / Binder Ratio	**1.3**	**#VALUE!**
No. 8 (2.36mm)	33.5-41.5	1119.1	60.6	**39.4**			**100.0**	CALIBRATION FACTOR		
No. 16 (1.18mm)		1350.3	73.1	26.9			100.0	H-Sample Weight Before	1300.0	1301.2
No. 30 (600um)		1480.7	80.2	19.8			100.0	I-Sample Weight After *	1235.2	1236.6
No. 50 (300um)		1601.1	86.7	13.3			100.0	J-Weight Loss	64.8	64.6
No. 100 (150um)		1691.3	91.6	8.4			100.0	K-% Loss(J/H x 100)	4.98	4.96
No. 200 (75um)	3.4-6.2	1748.0	94.6	**5.4**			**100.0**	L-Actual Asphalt	4.80	4.80
Minus N0. 200		99.1	5.4					M-Temperature Comp. **	**	**
Total Aggregate Weight		1847.1	550.2					N-Calibration Fac.(K-L-M)	0.18	0.16
Weight Before Wash		1847.1	Weight Before Wash			% MIX MOISTURE		MAXIMUM SPECIFIC GRAVITY (Gmm)		
Weight After Wash		1757.5	Weight After Wash			Wet Wgt.	1496.6	Sample Weight	2480.1	
Wash Loss		89.6	Wash Loss			Dry Wgt.	1496.2	Wgt. in H2O After Vac.	1571.3	
Fines In Pan		9.5	Fines In Pan			Loss	0.4	Wgt. of Displaced H2O	908.8	
Total Minus No. 200		99.1	Total Minus No. 200			% Moist.	**0.027**	Maximun Sp. Gr. (Gmm)	**2.729**	

I certify that the above test was sampled by me, and that all operations were performed in accordance with N.J.D.O.T. specifications and and procedures to the best of my knowledge .

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SIGNED : **DAVE SCOTT**

TRAP ROCK INDUSTRIES, INC.
IGNITION METHOD & GYRATORY TEST FOR COMPLIANCE

PROJECT : NJDOT MAINT. CENTRAL C204

PRODUCER : TRAP ROCK INDUSTRIES

CONTRACTOR : DELLA PELLO CONST. CO.

LOCATION : KINGSTON PLANT#1

% AC Absorbed: 0.27

DATE : 10/19/12

MIX NO. : /A 12.5M64 W/OUT FIBEI

SER. NO.: CO1DC0104

GYRATION LEVELS	Nini	Ndes
	7	75

Lot Sample No.		29	
Starting Temperature		538c	538c
Elapsed Time		45 mins.	45 mins.
A-Sample Weight		1954.1	
B-Sample Wgt. After Ignition *		1858.4	
C-Wgt. Loss (A-B)		95.7	
C-Wgt. Loss (Corrected for moisture)		95.2	
D-% Loss (C/A x 100)		4.87	
E-Temperature Compensation **		**	**
F-Calibration Factor (From Design)		0.17	0.17
G-% Bitumen (D-E-F)	5.00	**4.70**	**-0.17**

SIEVE SIZE		Wgt.	Ret. %	Pass %	Wgt.	Ret. %	Pass %
2" (50.0mm)				100.0			100.0
1-1/2" (37.5mm)				100.0			100.0
1" (25.0mm)				100.0			100.0
3/4" (19.0mm)	100			100.0			100.0
1/2" (12.5mm)	90-100	105.1	5.7	**94.3**			**100.0**
3/8" (9.5mm)	0-90	311.2	16.7	**83.3**			**100.0**
No. 4 (4.75mm)		761.4	41.0	59.0			100.0
No. 8 (2.36mm)	33.5-41.5	1130.2	60.8	**39.2**			**100.0**
No. 16 (1.18mm)		1340.9	72.2	27.8			100.0
No. 30 (600um)		1482.4	79.8	20.2			100.0
No. 50 (300um)		1602.5	86.2	13.8			100.0
No. 100 (150um)		1690.0	90.9	9.1			100.0
No. 200 (75um)	3.4-6.2	1754.1	94.4	**5.6**			**100.0**
Minus N0. 200		104.3	5.6				
Total Aggregate Weight		1858.4	553.3				

Weight Before Wash	1858.4	Weight Before Wash	
Weight After Wash	1765.7	Weight After Wash	
Wash Loss	92.7	Wash Loss	
Fines In Pan	11.6	Fines In Pan	
Total Minus No. 200	104.3	Total Minus No. 200	

% MIX MOISTURE	
Wet Wgt.	1496.6
Dry Wgt.	1496.2
Loss	0.4
% Moist.	**0.027**

GYRATORY RESULTS TO Ndes		
Mix Temp.	**310**	**310**
Sp. Gr. Of Binder (Gb)	1.035	1.035
Specimen Ht. @ Ndes	117.1	
Specimen Ht. @ Nini	127.6	
Weight in Air	5300.3	
Weight in Water	3276.1	
Weight SSD	5305.5	
Bulk Sp. Gr.(Gmb)	**2.612**	
Air Voids - %	**4.2**	
% Gmm @ Ndes	**95.82**	
% Gmm@Nini (<90.5)	**87.93**	
Total Volume	38.288	
Effective AC	4.426	
Volume of AC	4.276	#VALUE!
% AC by Volume	11.169	
% VMA (min-14)	**15.3**	
% VFA	**72.8**	
Dust / Binder Ratio	**1.3**	#VALUE!

CALIBRATION FACTOR		
H-Sample Weight Before	1300.0	1301.2
I-Sample Weight After *	1235.2	1236.6
J-Weight Loss	64.8	64.6
K-% Loss(J/H x 100)	4.98	4.96
L-Actual Asphalt	4.80	4.80
M-Temperature Comp. **	**	**
N-Calibration Fac.(K-L-M)	0.18	0.16

MAXIMUM SPECIFIC GRAVITY (Gmm)	
Sample Weight	2526.2
Wgt. in H2O After Vac.	1599.4
Wgt. of Displaced H2O	926.8
Maximun Sp. Gr. (Gmm)	**2.726**

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I certify that the above test was sampled by me, and that all operations were performed in accordance with N.J.D.O.T. specifications and and procedures to the best of my knowledge .

SIGNED : **DAVE SCOTT**

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NJDOT BRIC Mixture

Technical Memorandum – *Influence of Compacted Air Voids on Performance*

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ABSTRACT

The Center for Advanced Infrastructure and Transportation (CAIT) was requested by the NJDOT Materials Bureau to evaluate the influence of compacted air voids on the mixture performance of a NJDOT Bottom Rich Intermediate Course (BRIC) mixture. Loose mix sampled and procured from 2012 project produced and placed by Earle Asphalt was utilized in the study. Three levels of compacted air voids were targeted: 1) 2 to 3% air voids; 2) 4 to 5% air voids; and 3) 6 to 7% air voids. The range in air voids was selected to represented low, middle, and high end of the compacted air void range noted in the NJDOT specifications. The Asphalt Pavement Analyzer was used to measure the rutting potential and the Overlay Tester was used to measure the fatigue cracking potential. The tests were selected for their reputation to correlate to field performance, as well as the fact that both test methods are required in the NJDOT BRIC mix design and plant production verification.

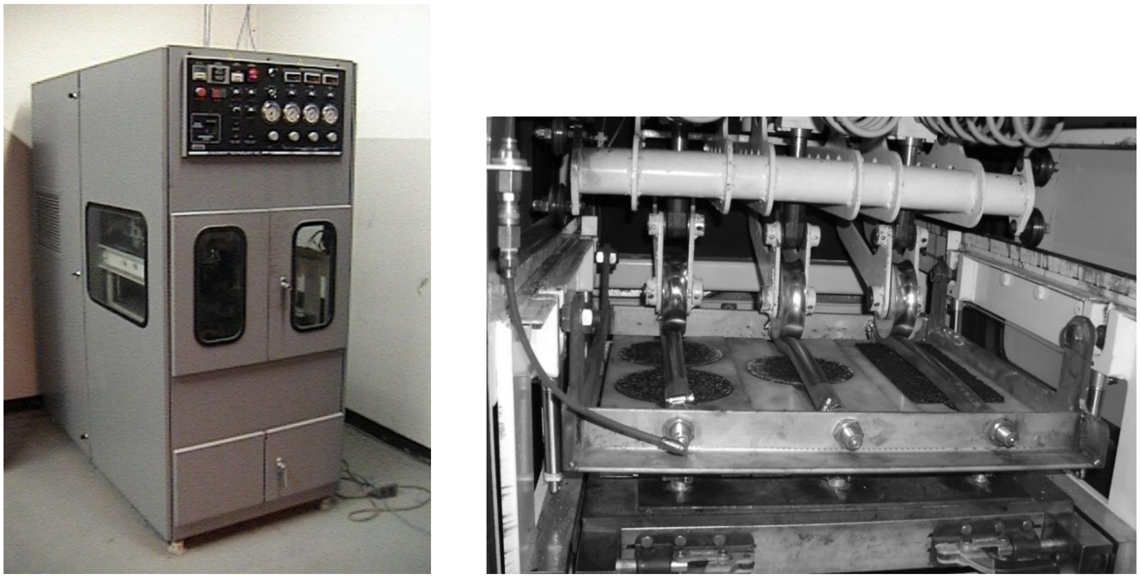
The test results indicated that the air void level for which the mixtures are compacted to does influence the test results. Higher level of compacted air voids will promote more rutting and decrease the fatigue life. Test data collected during this evaluation is shown in the following pages.

MIXTURE PERFORMANCE

Rutting Potential – Asphalt Pavement Analyzer (AASHTO T340)

The Asphalt Pavement Analyzer (APA) was conducted in accordance with AASHTO T340, *Determining Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA).* A hose pressure of 100 psi and a wheel load of 100 lb were used in the testing. Testing was continued until 8,000 loading cycles and APA rutting deformation was recorded at each cycle. The APA device used for testing at Rutgers University is shown in Figures 1a and 1b.

The test results for the BRIC mixture compacted at different air void levels is shown as Figure 2. The results indicate that as the air void level decreases, the rutting measured in the APA decreases. Since the mixture utilized in the study was loose mix sampled from a project produced and placed by Earle Asphalt, the plant production verification data is also included in the chart as additional information for comparison.



(a) (b)

Figure 1 – a) Asphalt Pavement Analyzer (APA) at Rutgers University; b) Inside the Asphalt Pavement Analyzer Device

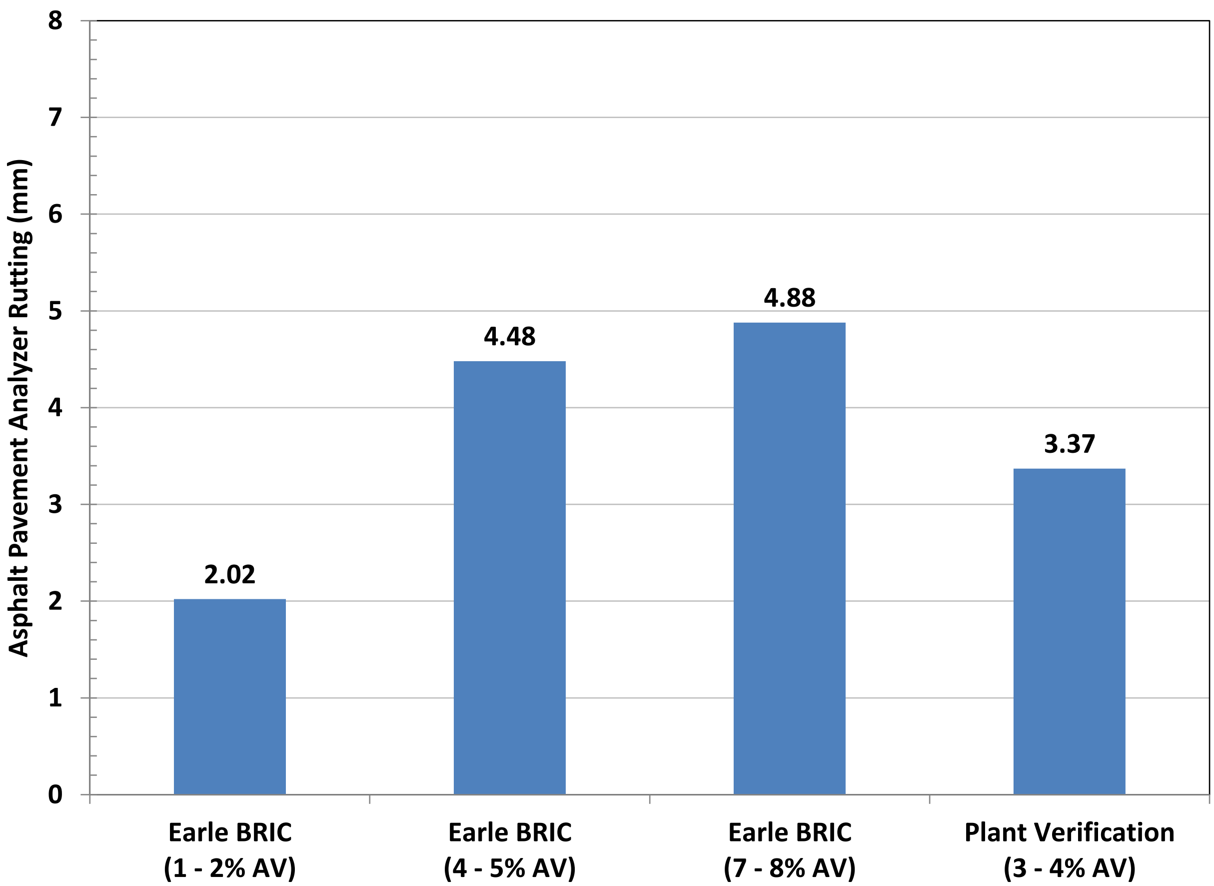


Figure 2 – Asphalt Pavement Analyzer for NJDOT BRIC Mix at Various Air Void Levels

Fatigue Cracking Potential - Overlay Tester (TxDOT TEX-248F)

The Overlay Tester, described by Zhou and Scullion (2007), has shown to provide an excellent correlation to field cracking for both composite pavements (Zhou and Scullion, 2007; Bennert et al., 2009) as well as flexible pavements (Zhou et al., 2007). Figure 3 shows a picture of the Overlay Tester used in this study. Sample preparation and test parameters used in this study followed that of NJDOT B-10, *Overlay Test for Determining Crack Resistance of HMA*. These included:

- 25^{o}C (77^{o}F) test temperature;
- Opening width of 0.025 inches;
- Cycle time of 10 seconds (5 seconds loading, 5 seconds unloading); and
- Specimen failure defined as 93% reduction in Initial Load.

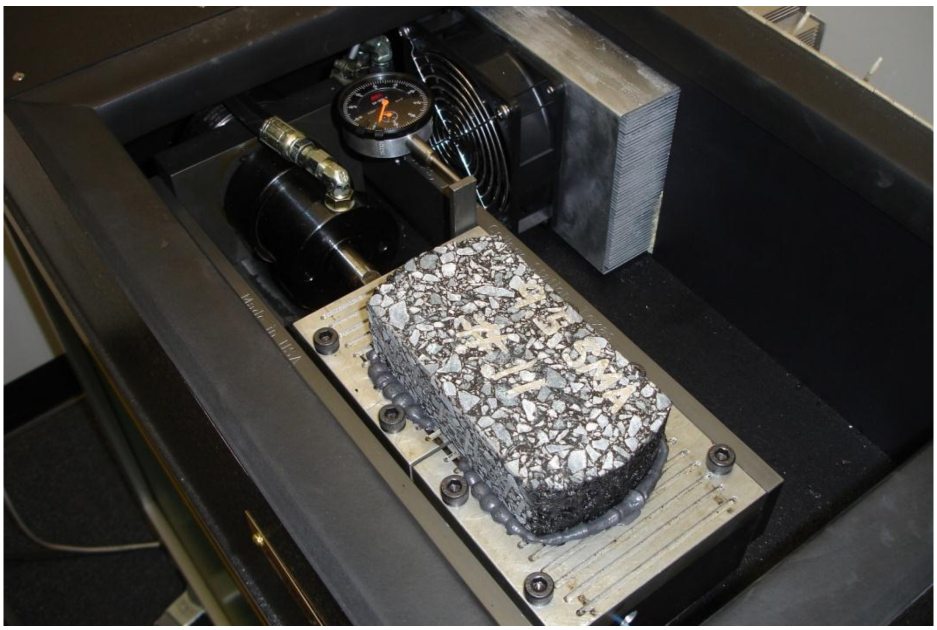


Figure 3 – Picture of the Overlay Tester (Chamber Door Open)

The test results for the mixture are shown in Figure 4. The Overlay Tester results show that as the air void level increases, the resistance to fatigue cracking in the Overlay Tester decreases.

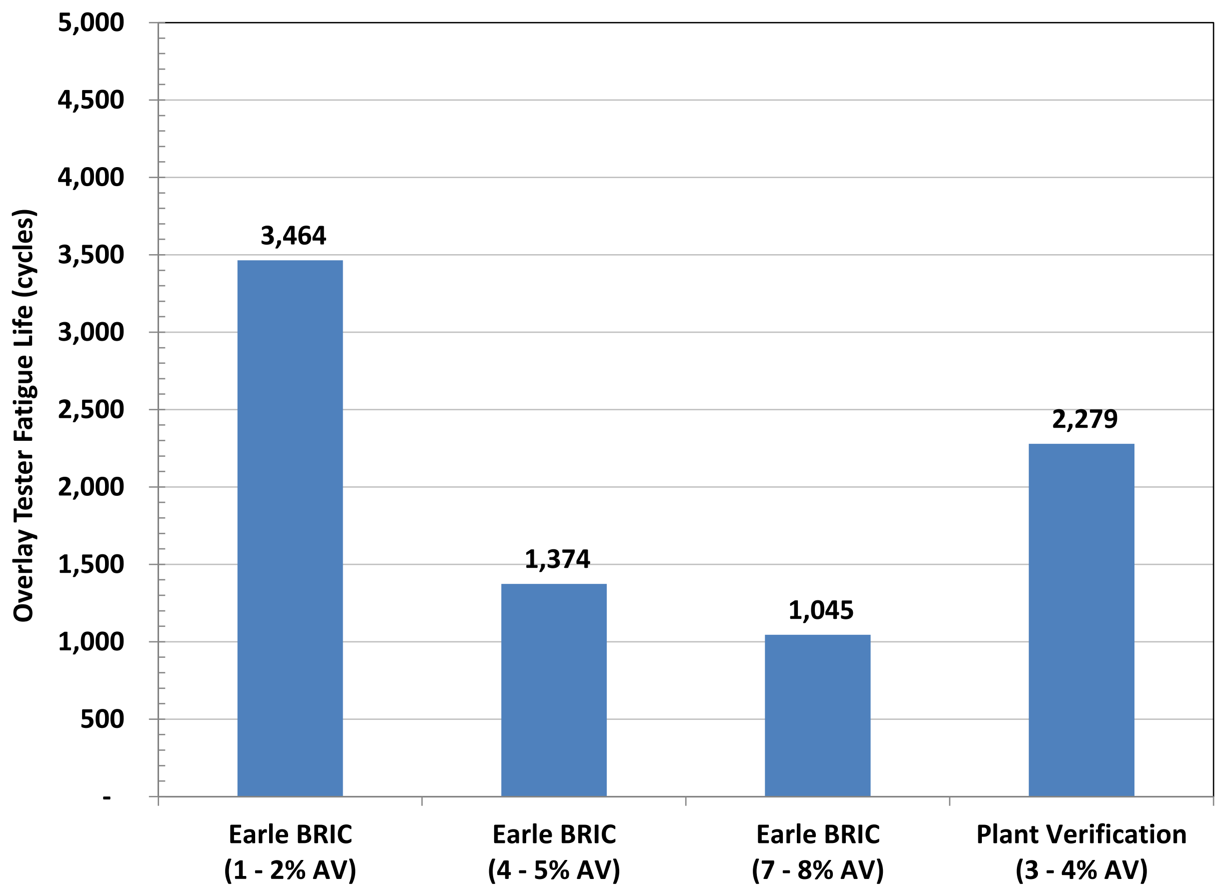


Figure 4 – Overlay Tester Results for NJDOT BRIC Mix at Various Air Void Levels

Asphalt Binder PG Grading from Extracted and Recovered Asphalt Binder

To evaluate the asphalt binder properties of the BRIC mixture tested in this study, the Center for Advanced Infrastructure and Transportation (CAIT) utilized solvent extraction and recovery procedures to extract the asphalt binder from the loose mix (Figure 5). The extraction and recovery of the asphalt binder was conducted in accordance with AASHTO T164, *Quantitative Extraction of Asphalt Binder from HMA* and ASTM D5404, *Recovery of Asphalt from Solution by Rotavapor Apparatus*. The recovered asphalt binder was then performance graded in accordance with AASHTO R29, *Grading or Verifying the Performance Grade (PG) of an Asphalt Binder*.

The resultant asphalt binder grade is shown in Table 1 and was determined to be a continuous PG grade of 79.3 – 27.6 (21.8) or a PG grade of PG76-22.

Figure 5 – Rotarvapor Solvent Extraction Set-up at Rutgers University

Table 1 – Performance Grade Results for Extracted/Recovered Asphalt Binder from Earle Asphalt BRIC Mixture

Condition	Test Method	Specification Criteria			Test Results at T_1			Test Results at T_2			T_C (°C)
					T_1 (°C)	P_1		T_2 (°C)	P_2		
Original	D 7175	G*/sind, kPa	≥	1.00	N.A.	N.A.	kPa	N.A.	N.A.	kPa	N.A.
RTFOT (D 2872)		G*/sind, kPa	≥	2.20	76	3.07	kPa	82	1.69	kPa	79.3
PAV (D 6763)	D 7175	G*sind, kPa	≤	5,000	19	6,889	kPa	22	4,913	kPa	***21.8***
	D 6648	S, MPa	≤	300	-18	314	MPa	-12	146	MPa	-27.6
	D 6648	m-value	≥	0.30	-18	0.302		-12	0.342		-28.3
					Continuous Grade =				***79.3-27.6 (21.8)***		