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PAVEMENT RESOURCE PROGRAM 2010/2011

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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).

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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. 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 00, 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 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 Unit by working with the unit staff to establish and implement a comprehensive pavement strategy toolbox that would optimize capital investment dollars by selecting the right fix at the right time on the right pavement. These strategies would be included into the Deighton Infrastructure Management System. The treatment strategies will expand on the current rehabilitation and reconstruction treatment by developing Pavement Maintenance (PM) decision trees or treatment rules, default/draft performance curves, failure criteria and timing or condition to apply the treatment (moving from Fair to Good), impact of treatment on condition (e.g., smoothness level, distress level, rutting level), performance of the activity (condition over time) based on condition of the pavement before the PP treatment.

The Pavement Resource Program agreed to work with the unit staff to finalize a new pavement sectioning methodology for use in prioritizing annual pavement program for CPM and Maintenance Operations. The Pavement Resource Program would work with the unit staff and Deighton to refine the PMS tools (analyses and reporting) and training of NJDOT staff in using the dTIMS asset management software.

The Pavement Resource Program would work with the unit staff to evaluate the use of GPS data to supplement current DMI linear referencing system for PMS field condition data collection.

Work Performed

The PRP met with Deighton and NJDOT staff during their site visit to evaluate the modifications to the dTIMS PMS and to evaluate the budget scenario results for the 2010-2020 program. The PRP worked with the NJDOT to conduct budget analyses for the NJDOT dTIMS PMS at the $300M level.

The PRP also conducted budget analyses for the NJDOT CIS unit. The PRP and the NJDOT staffs identified some issues with the dTIMS perspective tables and these tables were corrected. Based on the evaluation, the PRP and the NJDOT staffs have revised the Roadway Definition in dTIMS and developed new pavement network sections that can be verified.

The PRP evaluated the treatment triggers and reset engineering rules for Route 1 as a pilot project.

The PRP continued to refine the NJDOT Deighton dTIMS PMS user manual as the system was implemented. New perspectives, analysis variables, and expressions as well as “quick notes” that outline the annual process to modify the highway definitions, section definitions, treatment triggers, resets, and costs were added to the manual. NJDOT completed the section on pavement sectioning and NJDOT dFRAG program for the LCC Analysis Perspective programs for inclusion in the manual.

The PRP staff developed a series of performance and budget analyses for the annual CIS reports. The analyses included unrestricted and restricted pavement preservation scenarios requested by the CIS unit.

The PRP and NJDOT unit staffs developed a complete dataset to develop updated pavement performance data curves for use in the network performance and economic models. The performance models are 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 will be summarized by minor and major rehab for IRI and SDI. The models are based on regression analysis using the Excel solver program to maximize the R-squared value.

The PRP staff developed a methodology to prepare network construction program section summaries. The PRP and NJDOT staffs performed an evaluation of the recommended annual construction program from dTIMS. Certain anomalies will be discussed with Deighton staff.

The PRP also served as the NJDOT’s conduit and renewed the Deighton Software Maintenance and Support Contract for 2010 and 2011.

The PRP conducted and delivered 25 year budget analyses for the State Legislature. The PRP and NJDOT staffs determined the amount of money necessary to bring the State maintained network to 90% acceptable in 25 years ($175M/year with 2% inflation).

The PRP delivered the Rutgers Engineering Soil data and GIS maps to NJDOT. The Rutgers Engineering Soil data and GIS maps have been added to the NJDOT Intranet for use by the Department's pavement designers and other staff. The online system includes the GIS Engineering Soil layer and Rutgers Engineering Soil Manuals for each county. The NJDOT and State OIT-GIS are working to add this system to the Internet for use by anyone.

Ride Quality of New and Rehabilitated Pavements

Background

The Pavement Resource Program agreed to continue to provide technical support to the NJDOT Pavement and Drainage Management Systems Unit by working with the unit staff in establishing and implementing calibration procedures, and assist in the management of the NJDOT International Roughness Index (IRI) calibration and certification facility (based on the Texas TTI model). The Pavement Resource Program would implement and develop a Pavement Profiler Certification procedures manual for data collection and analysis of calibration of high speed profilers based on the NJDOT's standard walking profiler. The Pavement Resource Program would facilitate the training of NJDOT and industry staff on the use of walking and high speed and portable profilers to enhance pavement and bridge deck ride quality.

The Pavement Resource Program would work with the NJDOT Pavement and Drainage Management Systems Unit to calibrate, certify, and implement their walking profilers and pavement portable profilers for the NJDOT ride quality program.

The Pavement Resource Program and Advanced Infrastructure Design worked 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

At the request of the NJDOT, the PRP prepared and conducted two ride quality training classes for the Bureau of Materials and Pavement and Drainage Management and Technology staff on ride quality and profile measurements with the Surpro 2000 and profile analysis with ProVAL 3 software.

During the first quarter of 2011, the PRP took an active role in working with the Bureau of Materials in repairing their Surpro 2000 machines.

During the second quarter of 2011, the PRP assisted the Bureau of Materials to evaluate their bridge deck data collection efforts in regions North and South. The PRP worked to recalibrate the NJDOT's recently repaired Surpros as well.

The PRP located and evaluated potential Ride Quality Certification sites for NJDOT throughout the Rutgers University campuses and other locations. The PRP staff successfully prepared the NJ Turnpike traffic request paperwork to get permission to use the original site for the Spring of 2012.

During the second quarter of 2012, the NJ Turnpike site was prepared for Spring data collection. The PRP organized data collection for the Rutgers SurPro walking profiler, the NJDOT Bureau of Materials SurPro walking profilers (except Region South) and the NJDOT High Speed Profilers. The Rutgers SurPro walking profiler, the NJDOT Bureau of Materials SurPro walking profilers and the NJDOT High Speed Profilers have 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) were certified and the NJDOT High Speed Profilers (ICC and Dynatest 147) did not pass. The NJDOT High Speed Profilers (Dynatest 146) will be retested. The NJDOT Bureau of Materials SurPro walking profilers (Trenton) was analyzed once the data collection was completed. NJDOT Bureau of Materials SurPro walking profilers (South) was not been tested for certification during this quarter.

During the third quarter of 2012, the Rutgers SurPro walking profiler, the NJDOT Bureau of Materials SurPro walking profilers and the NJDOT High Speed Profilers have 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) were certified and the NJDOT High Speed Profilers (ICC and Dynatest 147) did not pass. The NJDOT Bureau of Materials SurPro walking profilers (Trenton) was analyzed.

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

Background

The Pavement Resource Program agreed to continue to provide technical support to the NJDOT Pavement and Drainage Management Systems Unit to implement the new MEPDG in NJ. The Pavement Resource Program would work with the NJDOT Pavement and Drainage Management Systems Unit to:

- Develop model calibration and training.
- Develop material databases for the proposed M-E Pavement Design guide, select NJDOT pavements to be utilized for distress evaluation and recalibration of the pavement distress models currently incorporated in the software to correspond to New Jersey materials, traffic and environmental conditions.
- Providing facilities, coordination, and instructors for training pertaining to preventative maintenance, pavement preservation and MEPDG implementation.
- Conduct laboratory and field testing of materials for characterization of pavement structures and their individual components for roadways under NJDOT jurisdiction.
- Develop a consolidated list of inputs for MEPDG for levels 1-3, organize meetings with NJDOT traffic organizations to discuss traffic data needs and modification to consultant agreement for data collection.

- 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 IRI, top down and bottom up cracking and rutting.
- Develop traffic inputs for MEPDG
- 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.
- Evaluation of urethane grouts and installation procedures and tools for undersealing of composite or concrete pavements.
- Development of a Construction Quality Assessment (Report Card-good paving practices) [from plant to end of construction]. [milling, tack/polymer joint adhesive, compaction, MTV, paver operation]
- Evaluate PMS pavement condition data collection to support MEPDG calibration.
- For the 2011 program specifically, 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.

<u>Work Performed</u>

The PRP performed tests on materials collected which will allow the modification of material coefficients and the adjustments of the MEPDG pavement distress prediction models.

Material evaluation for the Flexible Pavement Rehabilitation was completed. The evaluation consisted of the testing of field cores and loose mix material for four test sections. Additional, site specific traffic and Falling Weight Deflectometer (FWD) testing was also conducted to support in the input requirements for the MEPDG. The PRP submitted a request to the NJDOT for pavement surface distress measurements after six months of service life to begin looking at the local calibration of the distress prediction equations.

Rutgers, in conjunction with Advanced Infrastructure Design (AID), conducted field evaluation and core retrieval for the composite pavement sections. After the costs were established, the number of test sections was determined.

AID tested the composite pavement test sections recommended by the PRP. The test sections were chosen to try and look at the different overlay materials presently being used by NJDOT. These include normal Superpave, dense-graded mixtures, bottom rich intermediate course (BRIC), and stone mastic asphalt (SMA). Testing conducted by AID included:

- Visual Distress Survey;
  - Used to provide an estimate of the relative distress severity level prior to rehabilitation
- Site specific traffic collection using portable Weigh-in-Motion and Automatic Vehicle Classifier systems;
- Falling Weight Deflectometer;
  - Conducted at PCC joints/cracks to determine the vertical deflection due to the applied load, as well as the Load Transfer Efficiency (LTE)
  - Conducted at mid-slab to determine the modulus of the underlying material.
- Dynamic Cone Penetrometer;
  - Used to determine relative thickness of unbound layers, as well as develop correlations to California Bearing Ration of unbound materials
- Extracted Cores for laboratory evaluation.
  - Field cores will be used to help characterize the in-situ material, as well as provide an estimate fo the laboratory properties of the existing HMA and correlate them to the existing pavement distress levels.

Laboratory testing conducted were:

- Asphalt binder PG grading of extracted asphalt binder;
- Modulus testing of field cores;
- Coefficient of Thermal Expansion testing of the underlying concrete pavement materials;
- Overlay Tester to evaluate cracking performance.

Test locations selected by the NJDOT are as follows:

1. Rt.9 MP 70.6 to MP 81.3, Contractor is Defino
2. Rt.1 & 9 MP 45.5 to MP 47.6, Contractor is Della Pello
3. MRRC S-304, Rt. 40 and Rt.322, Contractor is Arawak
4. MRRC C-104, Rt.1, 33, 130, 175, Contractor is Earle
5. MRRC S-203, Rt.12, 22, 27, 28, 206, Contractor is TRI

Along with the three (3) “new” composite pavement sections, two (2) existing sections were evaluated. These two sections had been previously studied by Rutgers University during a composite pavement study completed in 2009. These two sections had extensive field testing and laboratory characterization of the materials, allowing for an assessment of how the pavement materials are performing over a four to five year period. Additional field cores were sampled from these locations to get an idea of how the many properties have changed over the time of the pavement life. Material testing is almost completed on the field cores extracted. The laboratory testing includes:

1. Asphalt binder stiffness properties of the extracted asphalt binder;
2. Fatigue life in the Overlay Tester for the asphalt mixture field cores; and

3. Coefficient of Thermal Expansion (CTE) of the PCC cores.

Although Rutgers University had not received an updated version of the MEPDG software, now being called DARWIN-ME, a review of the NCHRP study on the reflective cracking models for the MEPDG software platform indicated that the above field and laboratory characterization should provide the required data inputs for model calibration. The end of this summer will also mark the 1 year mark for the performance of the flexible pavement sections. Rutgers University reviewed the PMS data to evaluate if pavement distress has accumulated in any of the test sections to help begin the model calibration of flexible pavement rehabilitation design.

The PRP purchased two copies of the Darwin ME software from AASHTO. The Darwin ME software was delivered and installed on PRP laptops. The PRP staff examined the software. A training program was 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 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 worked with the NJDOT Traffic staff to examine the traffic inputs. The NJDOT traffic staff provided WIM W-4 table to the PRP. The PRP developed Excel Macros and spreadsheet to process the W-4 table to create Load Spectra tables for input into the Darwin ME software traffic inputs.

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
- Use NDE technologies in the Quality Control/Quality Assurance of HMA for NJDOT use
- Characterize vertical cracks in the pavement.

Work Performed

During the last three months of 2010, GPR data for missing sections was collected, analyzed, and inputted into the HPMA. Approximately 75% of the missing data, at this point, had been collected and 25% of that data was inputted into the HPMA.

During the next quarter (1st 2011), data collection was completed on 754 miles or 65.4% of the field task. 334 miles were analyzed and ready for loading into HPMA, which was 29% of the total.

During the second quarter of 2011, data collection was completed on 100% of the field was collected and analyzed and prepared for inputting into the HPMA.

During the third quarter of 2011, the project to evaluate dielectric variation as a indicator of air void content was started by meetings with NJDOT and an initial literature review. The proposal and work plan was submitted to NJDOT for field validation of a technique using GPR that would plot air void content vs. dielectric values. Data collection is expected to start early in the 2011 program. The ICMP also collected data to determine if a difference in the structure of adjacent lanes existed. A pilot study was performed on Rt 1. Difference in adjacent lanes was noted. The report was delivered to NJDOT and a meeting was held to discuss the findings. (APPENDIX A)

During the fourth quarter of 2011, a literature survey was performed. From the survey, others, mostly in Europe, cited success in their attempt to identify void content with GPR. They did not provide an exact methodology. Additionally, it is unknown what mix design, thickness, overlay properties (composite pavement) were being use for the HMA and so it is difficult to compare their conclusions to New Jersey conditions. The NJDOT Materials Laboratory was contacted and a site for the pilot coring was identified.

During the first quarter of 2011, CAIT representatives met the AID coring team onsite at State Route 129 near Trenton. CAIT marked locations for 4 cores and took GPR readings at each of the 4 locations. The cores were taken by AID and sent to the NJDOT laboratory for Air Void testing. Results of the Air Voids were sent to CAIT. Based only on these 4 cores and the radar data, a correlation was not identifiable between Air Void content and dielectric constant. Several factors including the small sample size could account for this lack of correlation. It was noticed that the radar signal filters were less than optimally set which could also account for the issues.

During the third quarter of 2012, the PRP conducted a variety of activities. For the characterization of rubblized concrete task, a resident engineer has been assigned and contacted about this project and the project is 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. The characterization of vertical cracks task was deleted from PSP work plan. 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. PRP’s radar van was down for mechanical repairs and therefore expect testing to be completed in the next quarter during the 2012 program.

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

Background

Quiet Pavements

The Pavement Resource Program agreed to work with the NJDOT Pavement and Drainage Management Systems Unit task force in developing criteria on the use of quiet pavements in NJ. The Pavement Resource Program would:

- Conduct a Noise study on new pavements or 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 would 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.
- Create an Access database and GIS map of highly sensitive noise areas.
- 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

Quiet Pavements

The PRP developed and submitted a Technical Memorandum encompassing the testing to date and a comparison between some of the Open Graded Friction Course (OGFC) mixtures being used in the state; both on NJDOT and New Jersey Turnpike owned and operated roadways. (APPENDIX B)

The PRP continued to test for the QPPP data, as well as looking at season (mainly temperature) influences on tire/pavement noise generation. Initial findings show that that pavement temperature has an effect on tire/pavement interface.

The PRP was able to procure a test vehicle for future work. Although test results in the past have been verified to be accurate, there were issues when the PRP was unable to obtain the same type of vehicle with similar tires to coordinate results. Different vehicles were found to create slight differences in the measured noise. Therefore, to provide consistent data over the long testing period proposed, it was essential to procure a constant test vehicle. The test vehicle was modified to measure tire and pavement temperature on a continual basis. The PRP believed that the tire and pavement temperature had a significant impact on the overall magnitude of the noise measurements on the pavement surface. Therefore, noise measurements should be 'normalized' to an average temperature for comparisons. Along with the temperature issue, CAIT also moved forward in organizing the QPPP for the NJDOT.

During the 2nd quarter of 2011, pavement noise testing ran smoothly throughout the last quarter. The main focus this quarter was to get the new testing vehicle operating properly, with the new equipment installed, and to continue to collect pavement noise for the NJ pavement noise database. Setting up the car took slightly longer than expected, partially due to inclement weather and partially due to unforeseen obstacles.

Despite weeks of rain, the pavement noise crew was able to conduct noise testing on ten sections throughout NJ to continue research for the database. Setting up the testing vehicle took a month of planning to ensure the proper materials were purchased. The planning process went through three design iterations while the car agreements were being completed by Rutgers. After a final design was settled upon, the materials were purchased and work on the car began in the beginning of March 2011. The first phase of the project required the stabilization of the floor in the trunk to support the testing equipment over a long period of time. While using the rental cars the testing crew noticed that after about two weeks, a normal trunk would be in bad shape, mostly from the movement of the car jack. The two major concerns with the new testing vehicle were the preservation of the vehicle itself and the preservation of the highly sensitive equipment that would be placed there.

In order to run the testing equipment efficiently, a second battery was installed in the truck, with a battery separator between the vehicle's primary battery and the new testing battery. The battery separator ensures that the equipment will never again drain the power from the primary vehicle, allowing the testing crew to get home after each test. After the power for the testing equipment was taken care of, each of the different components required to conduct sound testing was installed behind a false wall in the trunk. This method allows the noise testing crew to shave precious time off of the setup for each test and by running only two wires up to the front of the car to connect to the computer, this method also helps preserve the equipment, rather than having it all bounce around in the backseat or front floor of the car. The biggest problem incurred throughout the last year of testing was related to loss of power for the testing computer. Since the computer would not run the testing software without a full charge, any time there was an issue with power loss; it became almost impossible to complete testing for the day. This would become a serious issue when completing testing farther than one hour from the lab. Hard wiring the power inverter that runs the testing computer to the separate battery in the back completely eliminated the problems experienced with the testing computer. This is due to the consistent power provided the separate battery that is not affected by power surges and drawdowns associated to starting and operating the vehicle. The most recent addition to the testing vehicle is two infrared temperature probes. The installation of these probes was only made possible by having a secure and constant power source One probe is mounted in the wheel well above the right rear wheel to measure the tire temperature continuously throughout testing. The second probe is mounted just behind the right rear wheel and aimed at the pavement to measure the pavement temperature continuously throughout testing. With these two pieces of information we hope to bridge the gap between comparisons made on different pavements throughout different seasons, and to aid in the comparison of pavements measured by different noise testing groups around the country. Since CAIT's pavement noise testing group believes that the noise measured on a particular pavement on a particular day is related to the stiffness of the pavement, they are looking into determining the relationship between the actual stiffness of the roads being tested during the testing period and the resultant noise levels measured.

Although this spring was very wet, which limits the available testing time, ten roadways were tested throughout New Jersey including I-78, I-80, I-195, I-280, I-95, I-287, Rt. 202, and the Garden State Parkway. Data is currently being reviewed to determine if any changes were recorded from the previous year. Unfortunately, due to the nature of renting vehicles over the last year, there are only a few sections that can be directly compared based on the vehicle type. These comparisons are still interesting to look at.

In conclusion, for this quarter, most of the work was focused on getting the testing vehicle up and running and collecting data for the NJ pavement noise database. Now that the testing has been established and the testing vehicle will remain the same from here on out, the database will be significantly easier to build. Late last year the testing methods and capabilities were certified by the FHWA, so PRP can ensure that the measurements are acceptable across the country.

During the third quarter of 2011, pavement noise testing continued to run smoothly. The PRP noise group primarily continued monitoring the NJ test sections for the long-term pavement noise change project. All of the sections that were designated last year by the OBSI group were tested again between 6-1-11 to 10-31-11. Within these sections, the PRP have determined the different pavements found within and shown the differences in the overall levels recorded in this quarter to the initial measurements taken in 2010. The levels are all reported in A-weighted decibels (dBA) on Table 2. For most of the sections, no change was measured. This is good, because ideally there will not be a change unless there is a problem with the pavement surface. One section on I-295, was originally tested as a reinforced concrete pavement with no visible resurfacing procedures that has since been rubblized and repaved with a Novachip pavement. The difference between the old RC and the new Novachip was 4.5dBA, which was considered a significant victory for the PRP Noise group. One section, which was recently paved on I-280, was found to have a change of more than one decibel, which is still not significant as far as overall noise level is concerned, but the question of why was brought up amongst the crew. Because of this new development, the PRP Noise crew was interested to see if the reason for this change could be determined, and they went back to I-280 four times over the course of August to October. On the final time back, the PRP Noise crew was joined by the PRP GPR crew and each team tested the same locations along I-280. The final major development within this quarter was the acquisition of a fairly sensitive GPS tracking device, and the initial testing of the in-vehicle temperature probes. The initial testing provided some initial problems with how the probes were situated within the vehicle and whether or not they are measuring the temperature of the object they are pointed at, be it pavement or tire probe respectively, or whether they are measuring hot air spilling off of the brakes, out of the exhaust, or from friction created by the tire/pavement interface. Within this quarter the PRP noises crew represented PRP and the NJDOT in Portland Oregon at the TRB ADC40 summer meeting with two presentations, and at the NJDOT Showcase at the Mercer County College as a participant in the poster session.

During the first quarter of 2012, Rutgers University continued to test various pavement surfaces in New Jersey to assess their noise generating properties. Included in these sections were the AR-OGFC, MOGFC, dense-graded mixtures, and HPTO. Rutgers University will continue to evaluate these pavement surfaces, along with a concerted effort to begin more testing of pavement preservation techniques (i.e. – micro-surfacing, Novachip, HPTO, etc.).

During the second quarter of 2012, Rutgers University is continuing to conduct pavement noise measurement data to provide NJDOT with noise-reducing options with respect to pavement selection. Rutgers University collects the data quarterly in an effort to reduce possible changes due to environmental conditions. Rutgers University 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 has also begun evaluating different tire types in an effort to provide recommendations to NJDOT on "quieter" NJDOT state vehicles.

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Recycled Asphalt Pavement (RAP)

Rutgers University worked with the NJDOT to develop a High Recycled Asphalt Pavement (HRAP) specification. (APPENDIX D) The High RAP specification is hoped to be utilized by asphalt pavement contractors in New Jersey to recycle more millings to help alleviate the growing issue of stockpiling RAP materials. The benefit of using more RAP is that less virgin asphalt binder and virgin aggregate resources are required. To date, the NJDOT has allowed 15% and 25% RAP in the surface and intermediate/base course lifts. This new specification will allow contractors to use 30% RAP in the surface and up to 40% RAP in the intermediate and base course. Similar to the specialty mixes in New Jersey, the High RAP specification will contain performance testing to ensure fatigue cracking and rutting are not an issue.

During the second quarter of 2012, PRP worked with RE Pierson on their High RAP mixture designs which are proposed to be used on I 295 during the summer. RE Pierson has supplied two versions of high RAP mixtures for the surface (minimum of 20% RAP) and the intermediate (minimum of 30% RAP) courses. The first set of mix design resulted in mixtures too soft – failing the Asphalt Pavement Analyzer rutting and passing the Overlay Tester fatigue cracking. The second set of mix design performance resulted in the opposite trend, with mixtures passing the rutting but failing the fatigue cracking criteria. RE Pierson agreed to redesign the mixtures for a third time and recently submitted materials for performance testing. (APPENDIX E)

During the third quarter of 2012, 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. (APPENDIX F)

Warm Mix Asphalt (WMA)

Rutgers worked with Tilcon New York on a Warm Mix Asphalt project on Route 184. The project involved mixtures that were manufactured using a foaming system. There was a comparison between mixtures using and not using an anti-strip. The PRP also worked another project produced by RE Pierson on Rt 295 who planned on using warm mix asphalt (WMA) foaming technology. This project would be included in Rutgers University's WMA evaluation studies.

During the third quarter of 2012, PRP Rutgers University 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. Rutgers University will also be evaluating a "hybrid" WMA mixture produced by RE Pierson in the immediate future. 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. A copy of the final report is attached. Appendix (G)

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

At the request of the NJDOT, the PRP collected and analyzed GPR data on shoulders on Route 71 from mileposts zero to six. A report was completed and delivered in early 2011 (APPENDIX H). The PRP has continued to conduct the on-call testing services for HPTO and BRIC mixtures.

Along with the HPTO and BRIC (Binder Rich Intermediate Course) mixtures, the PRP tested samples of BRBC (Bottom Rich Base Course) mixtures as well. The PRP presented many of these results to the Northeast Asphalt Users Producers Group (NEAUPG) at their annual meeting. (APPENDIX I)

PRP tested and approved two new BRIC mixtures as prepared and provided by Earle Asphalt and Stavola Asphalt. The testing of their materials became ongoing throughout the length of the program.

PRP also tested and verified a Bridge Deck Water-proof Surface Course (NDWSC) mixture from Tilcon.

Rutgers completed the last remaining testing of the BRBC mixtures from the Route 295 project. A copy of the presentation on the overall performance/summary of project, which was presented a meeting of ASHE and is attached. (APPENDIX J)

PRP worked with NJDOT on conducting a study to evaluate different polymer modification techniques and compare their respective performance. The project was produced by Trap Rock Industries and include 3 types of PG76-22 asphalt binders; SBS polymer modified, ground tire rubber + SBS modified, and ground tire rubber + poly-phosphoric acid modified. The main outcome of the project would be that other modification procedures could be utilized by NJDOT which would still perform well but possibly be less expensive. Along with the physical testing, Rutgers University procured an Asphalt Pavement Analyzer “Junior” and a Flexural Beam Fatigue apparatus for both the NJDOT Bureau of Materials and the Rutgers Asphalt Pavement Laboratory. The APA is already set up in the laboratory at the Bureau of Materials, as well as a compactor to produce beam specimens for the Flexural Beam Fatigue test. The purpose of the equipment procurement was to provide NJDOT with independent Quality Assurance capabilities for the Quality Control testing that the Rutgers Asphalt Pavement Laboratory is conducting on these materials.

For the third quarter of 2011, the ICMP assisted on two on-call projects during the quarter. One was a GPR survey on Rt 78 to determine the thickness of pavement layers in the shoulder. The second was to use GPR to look for voids and wet pavement under a section of Rt 22 near a sinkhole. No additional sinkholes were identified.

For the third quarter of 2011, the PRP has been extremely busy in the on-call laboratory testing services for a number of materials recently placed by the NJDOT. These include;

1. BDWSC produced by Stavola and material placed on I-287
2. BRIC material produced by Tilcon
3. BDWSC material produced by Tilcon for:
    a. Witt Pen Bridge
    b. Contract for Della Pello
    c. I78
4. Premature rutting/flushing of Rt. 70 SMA produced by Earle Asphalt
5. Round robin testing with the NJDOT Materials Laboratory to evaluate:
    a. Volumetric properties of HMA
    b. Asphalt Pavement Analyzer testing
    c. Performance grading of asphalt binder (PG64-22)

For the fourth quarter of 2011, PRP verified Tilcon’s Binder Rich Intermediate Course (BRIC) mixture design and production materials and the Bridge Deck Waterproofing Surface Course (BDWSC) produced by Tilcon for Route 184. (Appendix K) The Rutgers Asphalt Pavement Laboratory also completed asphalt binder testing for round robin testing with the NJDOT’s asphalt binder laboratory.

For the first quarter of 2012, PRP verified a number of mixtures over the winter for asphalt contractors across the state. This included two High Performance Thin Overlay (HPTO) mixtures by Trap Rock Industries and Stavola; BDWSC from Tilcon Mt. Hope, and BRIC from Stavola and Earle Asphalt. PRP also conducted a mini-round robin with the NJDOT to evaluate the calibration of their Asphalt Pavement Analyzer (APA). PRP conducted work pertaining to an SMA mixture that showed signs of flushing and minimal rutting. Testing included Asphalt Pavement Analyzer, mixture volumetrics, and angularity testing of the fine aggregate portion of the aggregate blend.

For the second quarter of 2012, 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 Keasby, South State, Tilcon Mt. Hope, Trap Rock Industries
- Bridge Deck Water Proof Surface Course (BDWSC): Stavola Tinton Falls, Tilcon Keasby
- High RAP Mixture: South State

For the third quarter of 2012, the PRP University 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 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

Also, 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 L)

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-L)

Rutgers The State University of New Jersey

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Pavement Resource Program

FY 2010

Report

Using GPR to Determine Thickness Profiles in Adjacent Lanes

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Using GPR to Determine Thickness Profiles in Adjacent Lanes

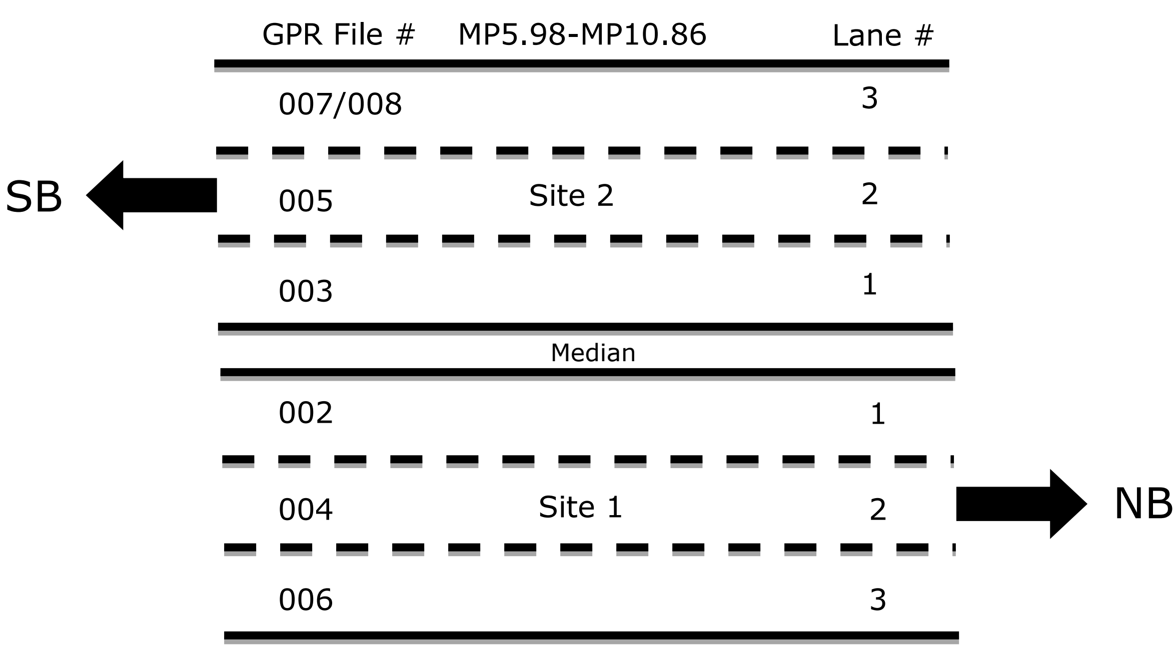
Background

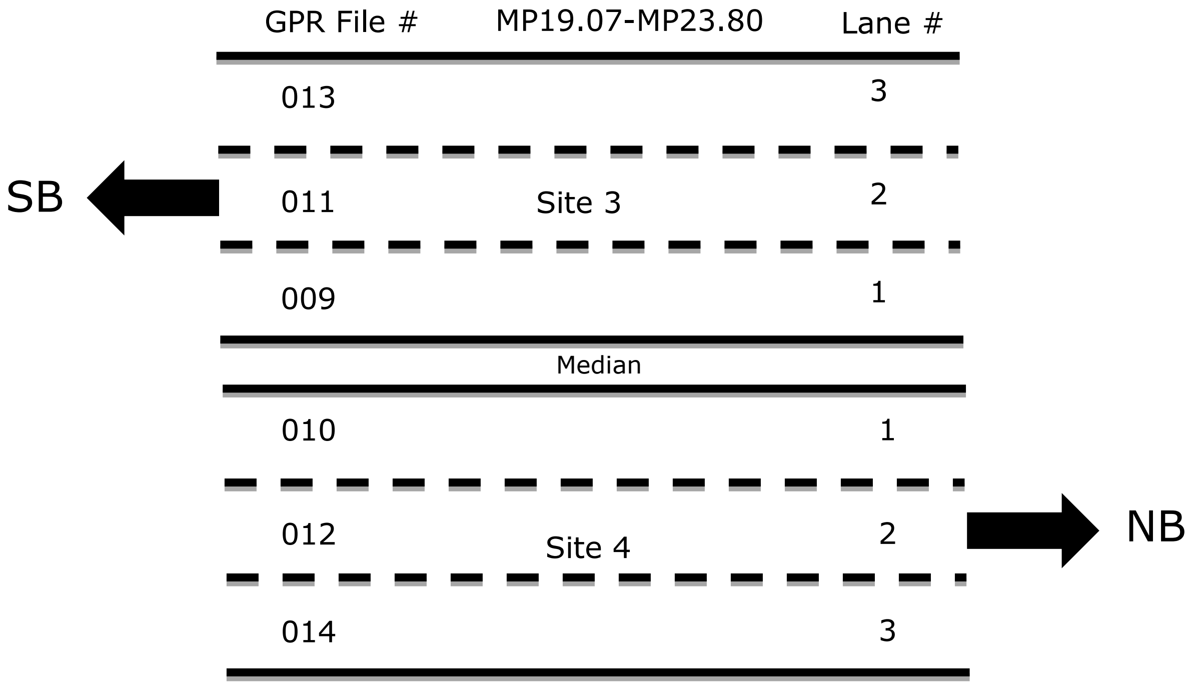
The Center for Advanced Infrastructure and Transportation (CAIT) has completed a project to characterize the layer profiles of the outside lane of all New Jersey State maintained highways. The data has been input into the HPMA software for use by designers and others. In the outside lane project no consideration was given to the possible differences in layer profile of adjacent lanes. What may have started as a two lane concrete pavement for lanes 1 & 2 may have become a composite pavement with an additional bituminous pavement added as lane 3. It is believed that the structure of the adjacent lanes may be different than the outside lane. This has caused the NJDOT to consider a GPR testing plan to address variation in pavement type in adjacent lanes and possibly the outside shoulder. It was proposed to have a pilot study to examine this variation on Route 1. This report presents the results of the pilot study.

Location & Data Collection

The pilot study was performed on two sections approximately five miles each in length. A total of four sites were identified for testing. The exact locations were from mile post 5.98 to mile post 10.86 and mile post 19.07 to mile post 23.80 in both directions. A plan view of the site locations is shown in Figure 1.

Figure 1: Site Locations





The GPR file number is the unique number of the radar pass in a lane at a site. For example GPR file # 004 is the northbound pass for lane 2 at site 1.

The same equipment and the same settings as used in the outside lane project were used in this project. The collection rate was 50 miles per hour collecting two data points per foot.

Data Analysis

As with the data collection the same methods and procedures used for analysis in the outside lane survey were used in this pilot study. The bottom of each road layer was identified using a semi-automated picking routine. The software used was RoadDoctor by Roadscanners of Finland. All files were processed independently. The results are presented as x-y scatter plots and shown on the next four figures; one for each site. The plots represent the thickness of the surface bound layers. Any PCC, granular base and subbase present are not shown in the plots.

Figure 2: Site 1

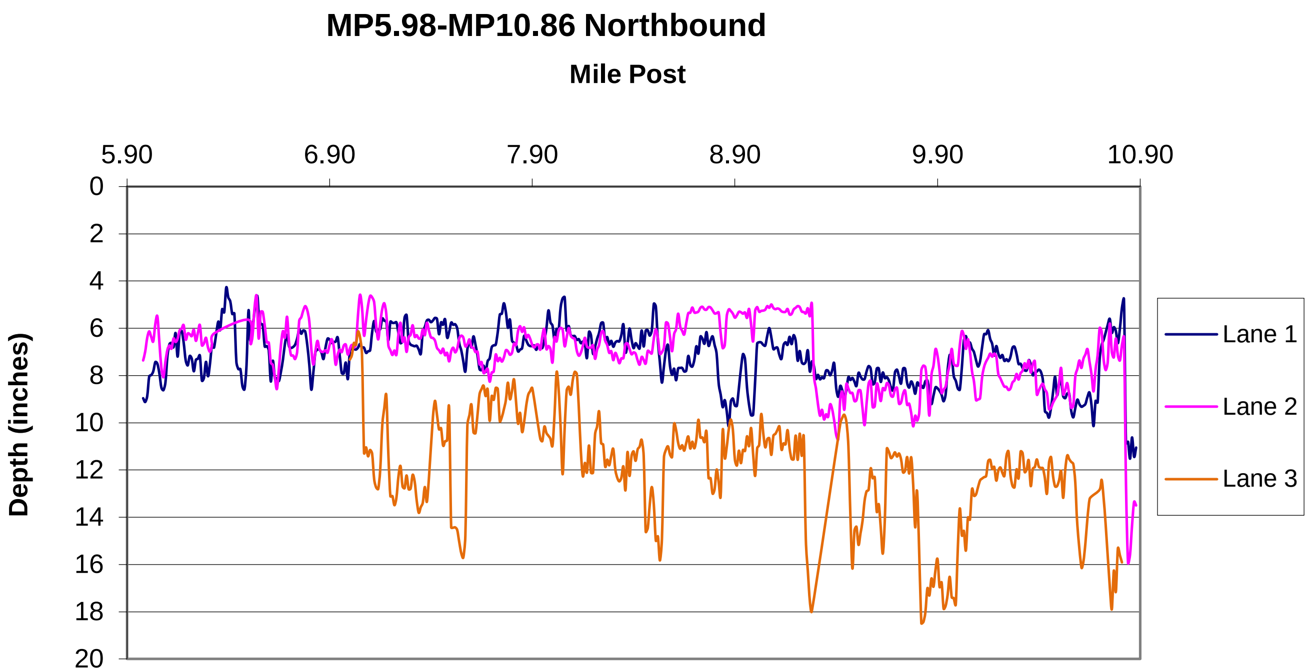


Figure 3: Site 2

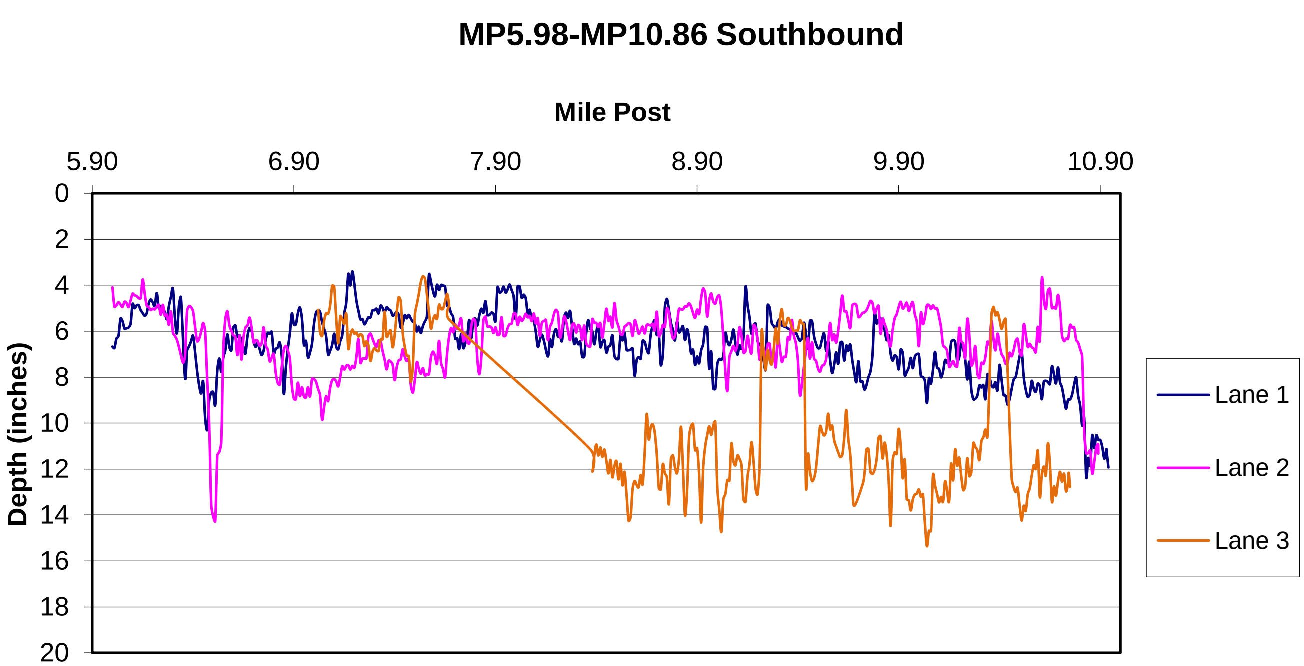


Figure 4: Site 4

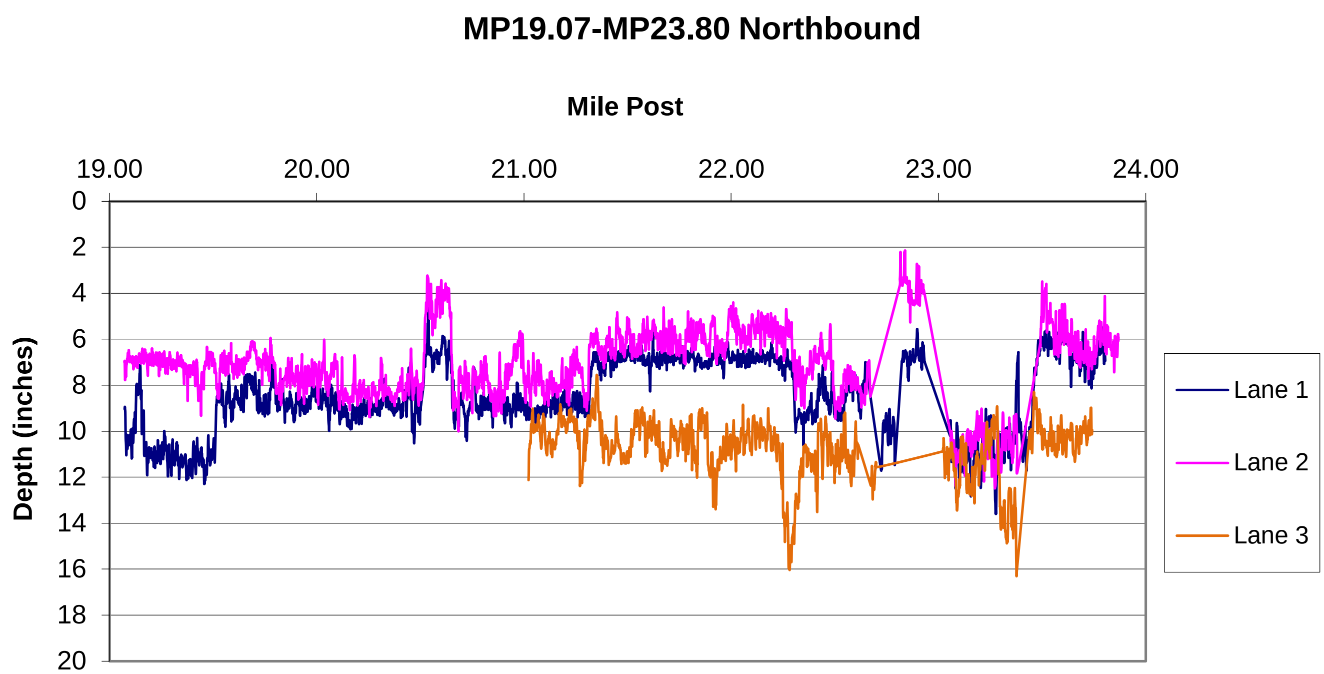
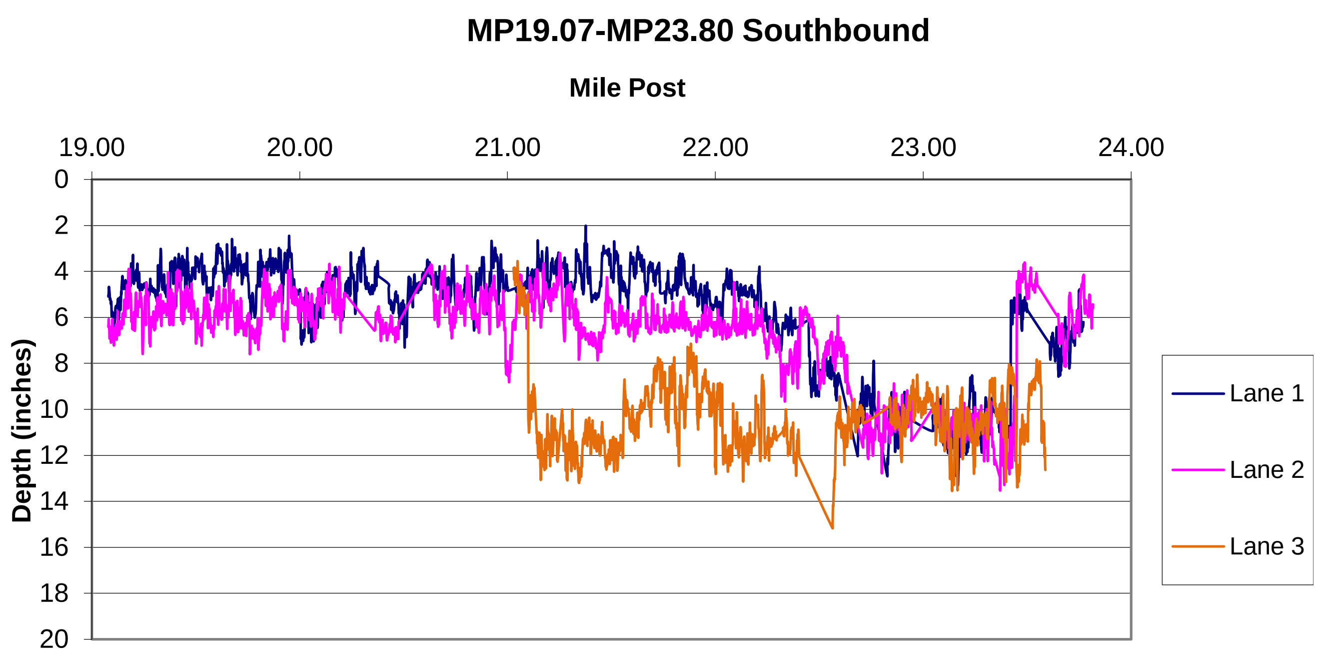


Figure 5: Site 3



As seen in the figures lanes 1&2 follow the same general profile for each site whereas lane 3 shows a different profile in each site.

Conclusions

It can be seen in the figures above that there is a difference in the outside lane 3 for each site. Therefore there is variation in layer thickness in adjacent lanes for this pilot section. Should the NJDOT wish to investigate profiles for adjacent lanes, the GPR is an effective tool for the task.

This project was successful in part because it was a pilot section. We recommend the adjacent lane survey be done on the remainder of Rt. 1. For other routes we recommend the same approach; doing the work on an identified pilot section then based on the results collecting and analyzing data on the remainder of the route.

ON-BOARD SOUND INTENSITY (OBSI) EVALUATION OF QUIET PAVEMENTS UTILIZED IN NEW JERSEY

Technical Memorandum

Submitted by

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In cooperation with

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

ABSTRACT

Quiet pavements have received much attention throughout the transportation related noise community, as a method of reducing highway related noise. This has spurred an interest to determine the noise related properties of current surface materials and surface modification techniques for asphalt and concrete pavement surfaces. Two methods, the close-proximity-method (CPX) and the on-board-sound-intensity method (OBSI), have gained success as practical methods of measuring sound properties of the tire/pavement interface, which is the largest contributing factor to highway noise from passenger vehicles moving over 30 mph (48.3 kph).

In New Jersey, concern over noise mitigation techniques and the exorbitant cost of implementing sound barriers has been steadily on the rise. The New Jersey Department of Transportation (NJDOT) became interested in noise mitigation via research completed by the Center for Advanced Infrastructure and Transportation (CAIT) and the National Center for Asphalt Technology (NCAT) utilizing the CPX method in 2005. The resultant research effort provided the NJDOT with valuable information allowing them to more appropriately select asphalt overlays that were not just durable and rut resistant, but also quieter.

In 2010, a research effort was initiated with the OBSI method (AASHTO TP76, *Method of Tire/Pavement Noise Using the On-Board Sound Intensity Method*) to measure new and old in-service pavements found in New Jersey. Data presented in this paper represent the OBSI noise measurements of typical pavement surfaces found in New Jersey. Attempts are also made to compare and evaluate the differences in noise and mixture properties of polymer modified and asphalt rubber Open Graded Friction Course mixes.

INTRODUCTION

As noise control becomes an overwhelming concern, Departments of Transportation (DOT) across the US and around the world are becoming increasingly interested at mitigating noise from the source (1,2). On highways, the controlling generation mechanism of noise is the tire/pavement interface (1). The On-Board-Sound-Intensity (OBSI) excels as a tire/pavement interface measurement technique, which has matured greatly over the last eight years, to quickly and efficiently evaluate in-service pavements (3).

The OBSI testing procedure is described in AASHTO TP 76-09 (4). As per the standard, all measurements were based on a 440 foot (134.1m) test section in a vehicle moving at 60 mph ±1mph (96.6kph ±1.6kph) with a 5 second measurement period, where one single A-weighted result is arithmetically averaged from the five second measurement period (4). Four microphones are utilized within this type of testing with two combined to create an intensity probe on the leading edge of the tire patch (the area that the tire contacts the pavement) and the remaining two microphones combined to make a second intensity probe, located on the trailing edge of the tire patch. Pulse, an acoustic engineering software program created by Brüel and Kjaer, is commonly used to do the aforementioned arithmetic averaging by utilizing the pressure difference between the microphones in each intensity probe, on the leading and trailing edge. This method utilizes sound intensity measurements near the tire pavement interface to single out the tire/pavement noise from the drive train, exhaust, and aerodynamic noise also produced by the vehicle. The close proximity of the microphones to the tire patch, the effective use of windscreens to ensure accurate measurements, and the ability to verify the quality of each measurement on the spot makes the OBSI method extremely useful.

Past research on pavement surface noise has shown that Portland cement concrete (PCC) typically produces higher noise levels from the tire pavement interface when compared to asphalt based surfaces (5). A significant amount of past pavement noise research has been focused on the possibility of mitigating highway traffic noise through pavement mix selection or surface treatment processes. Studies looking into the effectiveness of quiet pavements have compared typical dense graded hot mix asphalt (DGA) pavements to porous or semi-porous asphalts to find significant noise mitigating properties (6). In 2007, the National Center for Asphalt Technology (NCAT) completed a report comparing sound pressure level using the close proximity method (CPX) and sound intensity using the OBSI method for measurements of quiet pavements with open graded structures (7). Prior work on porous pavements has been conducted to measure the durability and effective design life of noise mitigating properties of open-graded and other functional overlays (8). To utilize open-graded friction courses (OGFC) for noise reduction at the tire/pavement interface, the structural properties of the mixes need to be analyzed for regionally specific needs (9). Such research has shown that OGFC overlays should be used cautiously in areas where the use of studded tires or snow chains during the winter season is prevalent, since it has been shown that studded tires significantly reduce the longevity of an OGFC life span (10). A significant amount of work has been completed in New Jersey on the use of functional thin lift overlays and their practicality as pavement surfaces in NJ, confirming that open graded asphalt pavements are a viable option for the NJDOT and can aid in noise mitigation (11).

Over the last five years, the New Jersey Department of Transportation (NJDOT) has received numerous noise complaints from residents who are located near several interstates and state highways. Many of these highways are aging and in need of future rehabilitation. However, after reviewing a federal highway administration (FHWA) Final Report regarding CPX tire/pavement interface noise measurements completed by The Center for Advanced Infrastructure and Transportation (CAIT) in 2005 (5), the NJDOT envisioned that the future rehabilitation of these pavement sections should address the possible functional components of the pavement as well as the required structural components. This would allow the NJDOT to address the pavement noise, possibly splash and spray, and other functional components while retaining rut resistance and durability. Therefore, if the NJDOT could quantify overall noise levels and spectral differences of pavement surface noise for in-service wearing courses in New Jersey, the NJDOT may be able to use the information in the pavement rehabilitation design process. If the NJDOT had a database of functional noise reducing pavements utilized in the state, information could be included during rehabilitation material selection to address pavement noise mitigation while retaining the other structural and functional needs during future rehabilitation.

To accomplish this, a pavement noise evaluation was conducted for the NJDOT in the spring of 2010 using the OBSI method to measure the tire/pavement interface noise of various asphalt pavement surfaces in-service in New Jersey. The pavements measured were chosen as representative samples of typical surface courses and functional thin lift overlays found in the state of New Jersey. The tested sections included two PCC pavements, three DGA pavements each paved in one year increments, two asphalt rubber open graded friction course (AROGFC) paved two years apart, and two polymer modified open graded friction courses (MOGFC), also paved two years apart.

OBJECTIVES

In the age of sustainability, an often forgotten aspect of environmental stewardship is reducing traffic noise generated at the tire/pavement interface. Pavement surfaces and materials can be selected by state agencies not just for structural reasons, but also for functional purposes, such as noise mitigation. The main objective of the research study was to develop a database of pavement noise measurements for various pavement surfaces in New Jersey, and to compare the noise mitigation properties of polymer-modified and asphalt rubber modified open graded friction course mixtures.

TESTING PROCEDURES

The test data presented in the paper was measured using a Chevy Malibu test vehicle. The test vehicle weighed 4,200 lbs with a full tank of fuel, all testing equipment, and two technicians. Before and after testing, the microphones were verified using a Larson Davis CAL200 94 dB signal generator. Before and after testing, the Standard Reference Test Tire (SRTT) was measured for hardness at every quadrant of the tire across each tread using a type A tire durometer (12). After the tire was mounted to the vehicle, the microphone spacing and placement was measured to ensure the distances were within the AASHTO TP76 specifications (4). Windscreens were always used while testing in New Jersey to ensure the least amount of wind generated interference. A picture of the final mounting arrangement is shown in Figure 1. The mounting, setup, and calibration of the equipment was done in the field, as close to the test site as possible, and if a second calibration was required, it was completed in the field. All of the

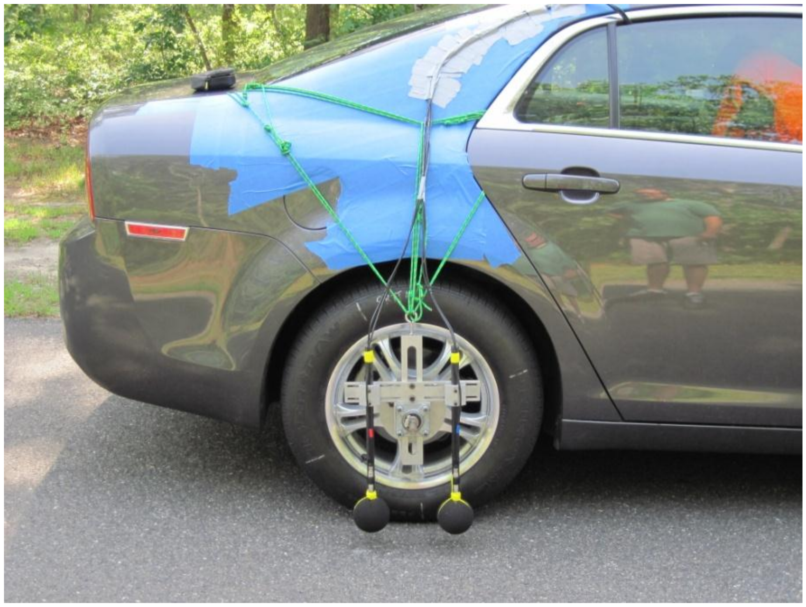

Figure 1 – On-Board Sound Intensity (OBSI) Mounting System

testing and results utilized for this paper were completed in the right lane, at 60 mph ± 1mph (96.6 ± 1.6kph). Any possible interference, such as a large truck passing by, a sound wall, or an overpass, was recorded by the technician during testing, and considered during post-analysis. The coherence between the two microphones on each intensity probe and PI spectrum were monitored during each test to ensure the validity of each measurement. A complete record of written data was compiled in conjunction with each measurement completed in the field.

TEST SECTIONS

Test locations (sites) are designated within this paper as the interstate or highway being tested, where a known material exists, typically designated by mile markers. Each test location was broken down into test sections by utilizing the mile marker signs as start points for measurements. Each test location was chosen either from a list of noise complaints provided by the NJDOT or because the pavement at that location was a functional overlay that could possibly provide noise reduction benefits. Each test section was a 440 ft (134.1 meters) long section where a 5 second noise measurement was completed. A test section was designated at each mile marker and sometimes at each half mile marker, based on the length of the test site. If the test site was less than 5 miles long, the half mile markers were usually used to gather more data about each pavement type. Using the mile marker signs ensured that the measurements were taken at the same start point during each consecutive measurement, required less time to set up test sections overall, and more easily allowed for pavement distinction based on pavement records provided by NJDOT that typically reference mile markers. The test sections evaluated in the study are described in the following sections.

NJ Rt. 202

The Rt. 202 location was paved in 2007 with a 12.5mm mix, with a PG76-22 asphalt binder (12.5H76). This test site was located from mileposts 14 to 18 southbound in Hunterdon County

NJ. Rt. 202 is a 2 lane, 55 mph state highway in Readington Township, NJ with a southbound annual average daily traffic of 18,790 (AADT) in 2008.

NJ Rt. 3

The Rt. 3 location was a 12.5mm PG76-22 (12.5H76) mix that was paved in 2008. The Rt. 3 test site was 3 miles long between mileposts 3 and 6 both northbound and southbound in Passaic County NJ. Rt. 3 is a 3 lane, 55 mph state highway used as a major thoroughfare between the George Washington Bridge and I-80 which had an average annual daily traffic of 124,050 (AADT) in 2008.

Garden State Parkway (GSP)

Three test locations were selected on the Garden State Parkway (GSP), all of which were 2 lane, 65 mph zones. A 12.5mm PG76-22 (12.5H76) mix which was paved in 2009 was tested between mileposts 38 to 48 northbound and southbound. The site connected Egg Harbor Township, NJ to Port Republic City, NJ. A polymer modified open graded friction course (MOGFC) that was paved in 2007 was tested between milepost 24 to 26 in Upper Cape May Township in Cape May County. A second MOGFC which was paved in 2009 was tested between mileposts 41 to 48 southbound, from Galloway Township NJ, to Port Republic City, NJ. The last available average annual daily traffic count suggests that in 2007, all three sites had 4712 AADT.

Interstate 95 (New Jersey)

The location tested on I-95 was an asphalt rubber open graded friction course (AROGFC) mix that was paved in 2007. The site stretched through Mercer County, between mileposts 4 and 8 both northbound and southbound from Ewing, NJ to the intersection of I-295 near Lawrenceville, NJ. The I-95 site was a 3 lane, 65 mph zone, with an average annual daily traffic of 84,079 (AADT) in 2008.

Interstate 280 (New Jersey)

On I-280 the test site was a roughly ground Portland cement concrete (PCC) in Essex County, NJ. The site ranged from milepost 6 -13 eastbound and westbound. This part of I-280 is a major connector between I-80 and I-95, which is a 4 lane, 65 mph zone with an average annual daily traffic of 102,482 (AADT), in 2008. The site was in the middle of a rehabilitation project when testing occurred. The PCC there was treated with a rough surface grind to reduce the bumps produced by joint heaving. The project will be receiving an AROGFC wearing course in the summer of 2010.

Interstate 295 (New Jersey)

On I-295, a 7 mile test site with a well worn PCC was tested. The I-295 location was in Burlington County, on a 3 lane, 65 mph zone, which ranged from Springfield Township to Bordentown Township from mile markers 49 to 56, northbound and southbound. This part of I-

295 is just south of the connection to Rt. 1, I-95, and the City of Trenton. The average annual daily traffic measured in 2007 for the test section was 65,000 vehicles.

Interstate 78 (New Jersey)

On I-78, an asphalt rubber open graded friction course (AROGFC), which had a PG64-22 binder modified with asphalt rubber, was paved in 2009. The tested site was a 6 miles long, 4 lane, 65 mph zone in Somerset County that ranged from mileposts 34 to 41 both westbound and eastbound, with an average annual daily traffic of 70,904 (AADT) in 2008. The section ranges from Bernards Township, NJ to Warren Township, NJ at the border of Somerset and Warren County.

ON-BOARD SOUND INTENSITY TEST RESULTS

The results displayed for the OBSI measurements were recorded in A-weighted decibels. The overall OBSI level is the arithmetic average of the recorded sound intensity levels at each one third octave band frequency. Each spectrum chart shows the recorded sound intensity level relative to a one third octave band spectrum. Table 1 shows the mix design information related to the I-78 AROGFC, the I-95 AROGFC, and the Garden State Parkway MOGFC mixes.

Results for the tire/pavement interface noise collected in the spring of 2010 are shown in Figure 2. Figure 2 shows the overall sound intensity levels in dBA of each test site from the highest on the left to the lowest on the right. Each site shown on Figure 2 is representative of a distinct pavement surface type. The overall sound intensity levels for each site were calculated by averaging each valid measurement from each 440 foot (134.1 meter) test section found within each test location. A single standard deviation for each material was included to show the accuracy of the averaging procedure for each material. This methodology was applied to ascertain an overall material characterization of each material. As seen with the standard deviation bars, the more test sections within each test site, the more variation that was found in the material, which represents the reality of materials due to variable states of disrepair, as well as the ability of testing the identical wheelpath area on repeat runs. As seen in Figure 2, at an overall level of 107.4 dBA, the rough ground PCC from I-280 was the loudest section tested, while the quietest section tested was the GSP 2009 MOGFC section which had an overall OBSI level measured at 96.8 dBA. The PCC pavements tested were over 4 dBA higher than the loudest DGA, the Rt. 3 12.5H76 site which was paved in 2008.

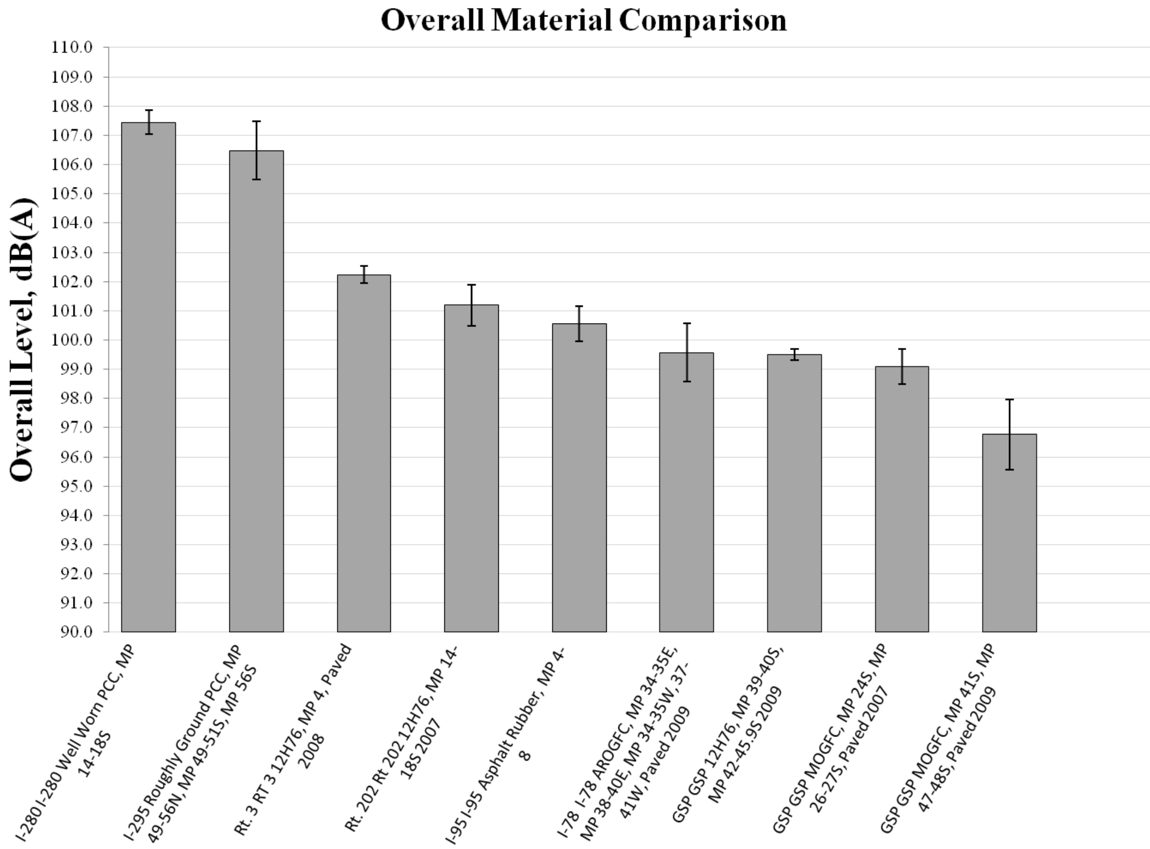

Figure 2 – On-Board Sound Intensity (OBSI) Measurement for Selected Test Sections

Figure 3 shows the spectrum analysis of the roughly ground PCC tested on I-280, and the aged PCC tested on I-295. The spectrum analysis shown in Figure 3 shows the difference between the sound intensity levels at one third octave band spectrum. The roughly ground PCC was almost 1 dBA louder overall, but the spectrum showed that it was louder from the 1600hz one third octave band and higher. The surface of the PCC on I-280 was 2 - 3 dBA louder than the I-295 PCC. The surface of the I-280 PCC was treated with a rough grind to alleviate any disparities in joint elevation, where joint heave had occurred due to improper undersealing procedures. This grinding appears to have reduced the peak seen in the I-295 PCC around 800hz, but it is suspected that the intense macro-texture led to an increase of the overall noise level. Overall, the PCC sections that were tested in the spring of 2010 were louder than every other pavement tested.

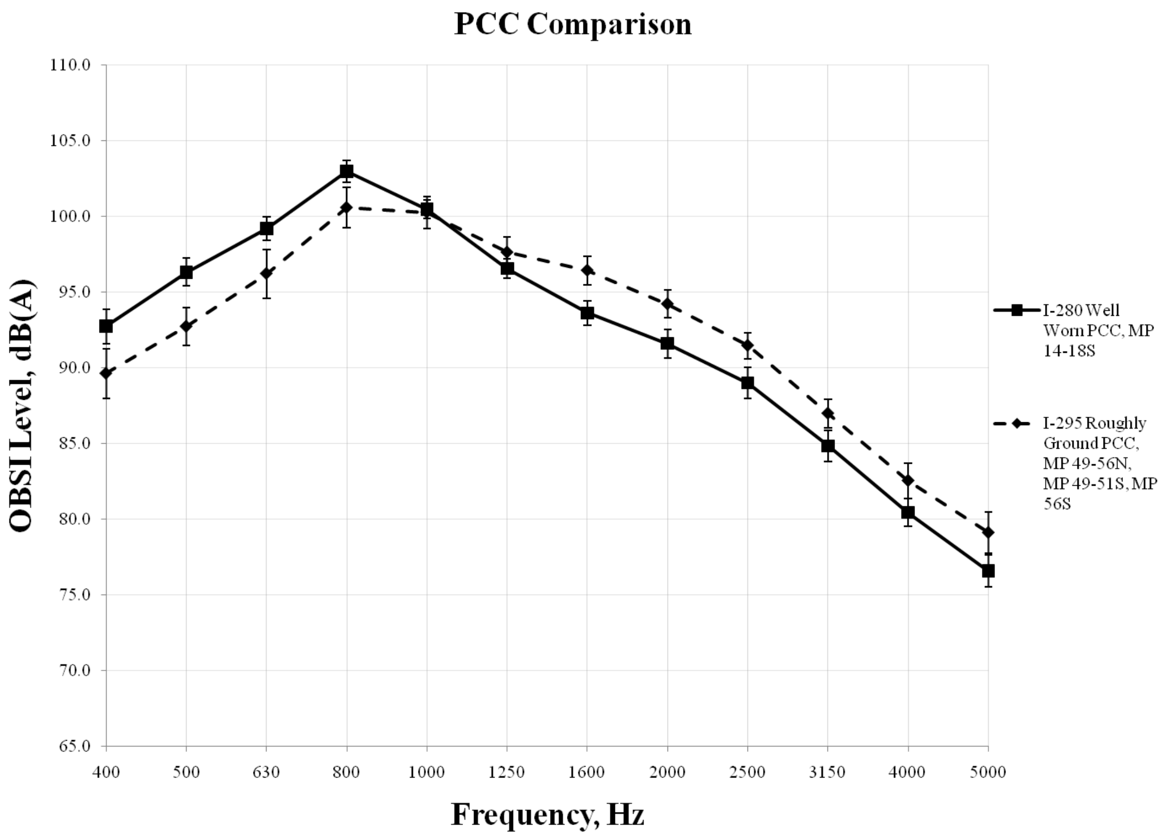

Figure 3 – Noise Spectrum Analysis for Concrete Test Sections

Figure 4 shows the spectrum analysis of the Rt. 3 12.5H76, the Rt. 202 12.5H76, and the GSP 12.5H76 materials. The DGA's measured in the Spring of 2010 all showed similar spectral trends. As shown in Figure 2, the Rt. 3 12.5H76 had an overall OBSI level of 102.2 dBA, the Rt. 202 12.5H76 had an overall OBSI level of 101.2 dBA, and the GSP 12.5H76 had an overall OBSI level of 99.5 dBA. Figure 4 shows that the pattern was similar for each surface, although the magnitude was different. It is hypothesized that even though the Rt. 3 DGA from 2008 was louder than the Rt. 202 DGA from 2007, the traffic load on the Rt. 3 DGA was over 6.5 times as much as the Rt. 202 section, possibly indicating that the additional traffic may have increased the exposure of aggregate in the wheelpath area on the Rt. 3 12.5H76 sections. The GSP 12.5H76 spectrum showed lower sound intensity levels between the 800hz to 1600hz one third octave band levels, which leads us to believe that the air voids have not seen the amount of traffic that the other two dense graded asphalts have seen, which is supported by the 4712 AADT.

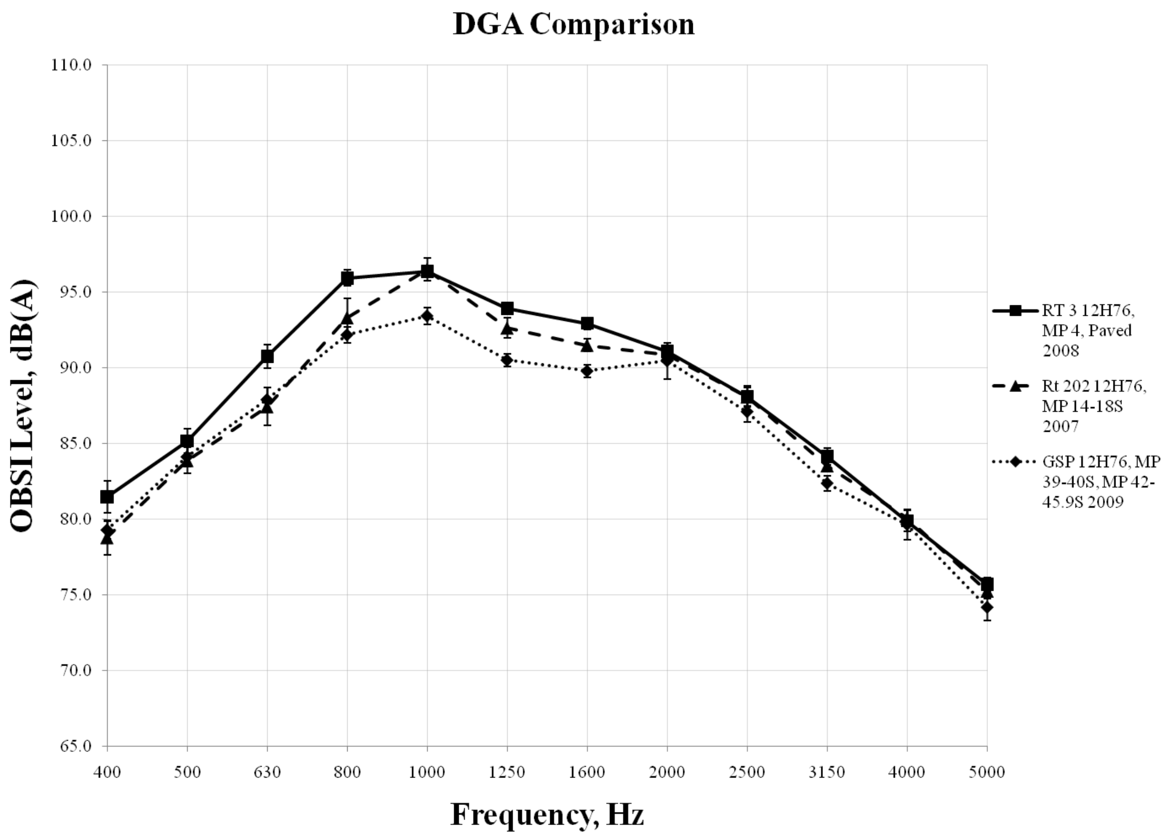

Figure 4 – Noise Spectrum Analysis for Dense-Graded Asphalt Mixture Sections

Figure 5 shows the spectrum analysis from the I-95 AROGFC and the I-78 AROGFC test sites. The I-95 AROGFC was paved in 2007 and the I-78 AROGFC was paved in 2009. The overall level of the I-95 AROGFC was 100.5 dBA and the overall level of the I-78 AROGFC was 99.6, as seen in Figure 2, which is not a large difference. Figure 5 on the other hand, shows the spectrum analysis of the two AROGFCs, which suggests that there are some differences between the 500hz to 1000hz frequencies and between the 2000hz to 3150hz frequencies on the one third octave band spectrum. Since both roads see a similar amount of annual daily traffic, it seems that the I-95 AROGFC which is two years older than the I-78 AROGFC, the mixture properties themselves were further reviewed. Table 1 lists the job mix formula (JMF) information for the OGFC test sections in the study. Table 1 clearly indicates a large difference in design air voids between two AROGFC mixes, with the I-78 AROGFC designed to accommodate almost 11% more air voids by volume. This disparity in volumetric design is most likely the reason for the differences in the one third octave band spectrum.

Regardless, the spectrum does suggest that although the two AROGFC pavements have had some time to age, they still retain the properties of a “quiet pavement”, which has reduced sound pressure levels in the 1000hz to 4000hz range on the one third octave band spectrum, where the average human range is more negatively affected.

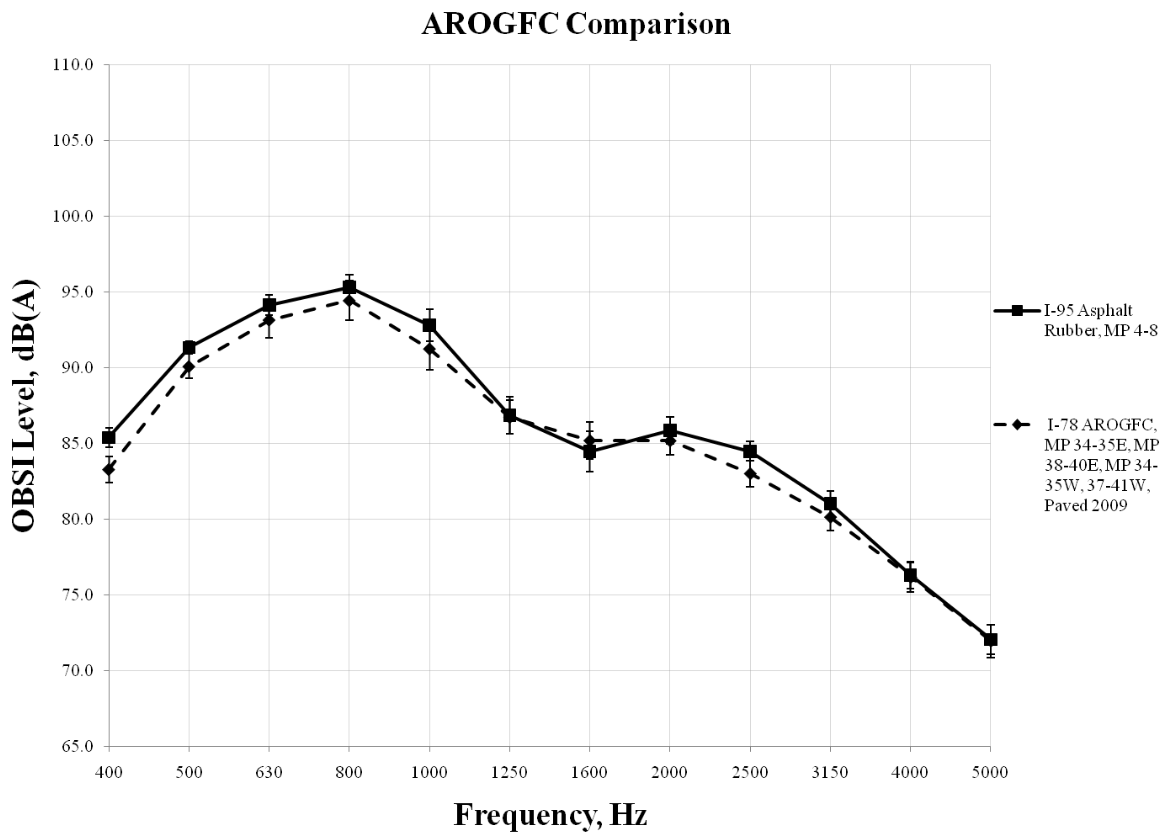

Figure 5 – Noise Spectrum Analysis for Asphalt Rubber Open Graded Friction Course (AROGFC) Sections

Table 1 - Mix Design Gradations for Polymer Modified Open Graded Friction Course and Asphalt Rubber Open Graded Friction Course Sections

Parameter	I-78 AROGFC	I-95 AROGFC	MOGFC (2007)	MOGFC (2009)
12.5mm	100	100	100	100
9.5mm	97.0	94.4	90.1	88.9
4.75mm	31.0	31.0	31.0	27.2
2.36mm	7.8	7.0	10.0	11.5
1.18mm	5.8	4.0	8.8	8.9
.600mm	4.0	4.0	7.2	7.0
.300mm	3.1	3.0	5.8	3.9
.150mm	2.0	2.0	4.6	3.3
.075mm	0.9	2.0	3.5	3.0
PG Grade	64-22	64-22	76-22	76-22
PB, %	8.5	8.6	5.7	5.7
Air Voids, %	25.5	14.1	19.7	21
VMA, %	39.4	31.1	N/A	N/A
Other	16% Crumb Rubber	19.5% Crumb Rubber	.3% Cellulose Fiber	.3% Cellulose Fibers + Anti Stripping Additive

Figure 6 shows the spectrum analysis for the MOGFC from the GSP, which was paved in 2009, and the MOGFC from the GSP which was paved in 2007. The MOGFC between mile marker 24S, 26-27S, was paved in 2007, while the MOGFC between mile marker 41S, 47-48S was paved in 2009. In Figure 2, the overall dBA level of the older 2007 MOGFC was 99.1 dBA, while the 2009 MOGFC, which was tested the following day and under identical environmental conditions, was 96.8 dBA. The MOGFC mix designs shown in Table 1 are similar in asphalt binder grade, design air void percentage, and gradation. Therefore, since the materials for both sections are extremely similar, the differences in the noise spectrum analysis is most likely a function of the aging (general stiffening) and minimal surface wearing due to accumulated vehicle passes.

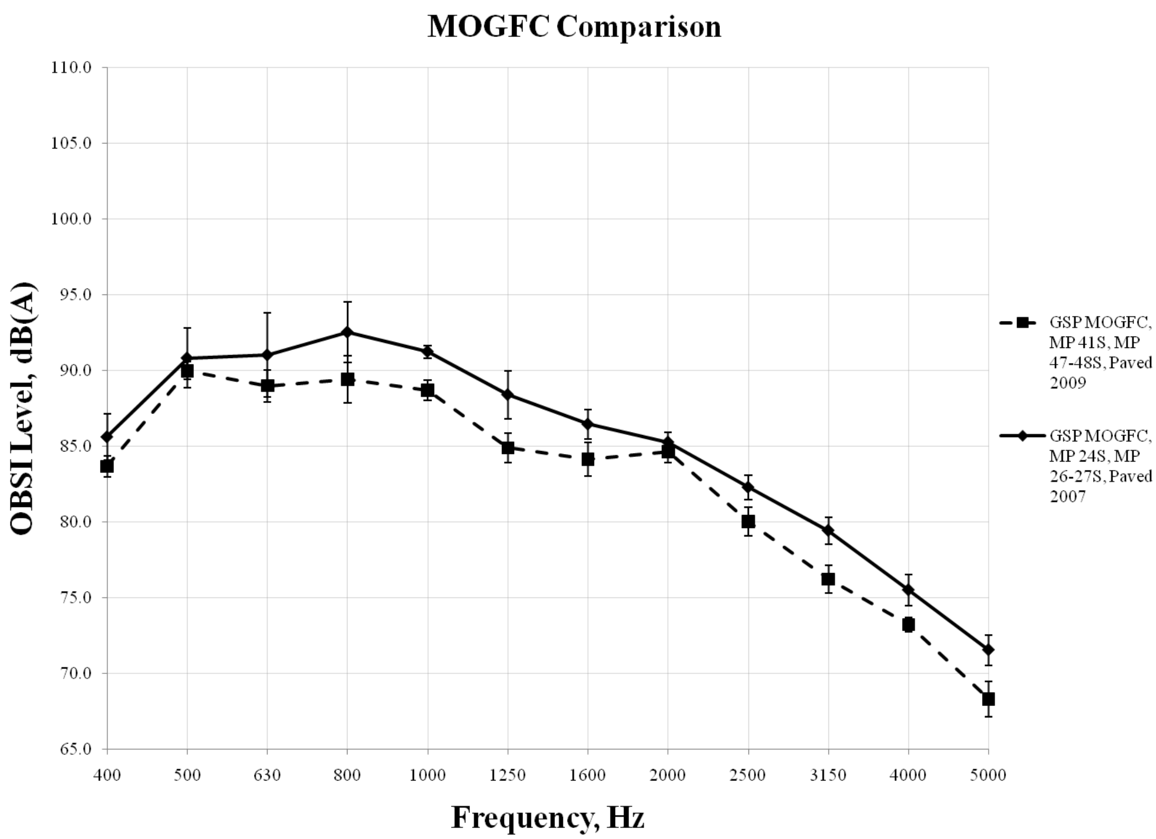

Figure 6 – Noise Spectrum Analysis for Polymer Modified Open Graded Friction Course (MOGFC) Sections

Dense Graded Asphalt (DGA) vs. Open-Graded Friction Course (OGFC)

When comparing the three DGA mixes with the OGFC mixes, it is easy to see that although the overall sound levels portrayed on Figure 2 suggest that the materials are somewhat similar, the spectrum analysis shows differently. Figure 7, which shows the OGFC mixes compared to the DGA mixes, begins to explain why OGFC materials in NJ are perceived as quieter. The DGA mixes, including the newest GSP 12.5H76, had noticeably higher measured sound intensity levels, from the 1250hz one third octave band frequency and higher. Even around 2500hz, where the older AROGFC had an average sound intensity 84.5 dBA, the newest DGA was almost 3 dBA higher at 87.1 dBA. At the low end of the measured spectrum, both the MOGFC and AROGFC mix types have higher recorded values. In the case of the GSP 12.5H76, the recorded intensity level at 500hz was 84.1 dBA, while the 2 year old AROGFC from I-95 was recorded at 91.4 dBA, which is a significant difference. Given this data, it is suspected that although the 500hz - 630hz one third octave band frequencies are higher for the New Jersey open graded mixes, the tone is less annoying to the receiver than the dense graded mixes which all have elevated A-weighted decibels from the 1250hz - 5000hz one third octave band center frequency range. It is hypothesized that the increased air void content of the OGFC surfaces are creating this attenuation of noise in the higher frequency range.

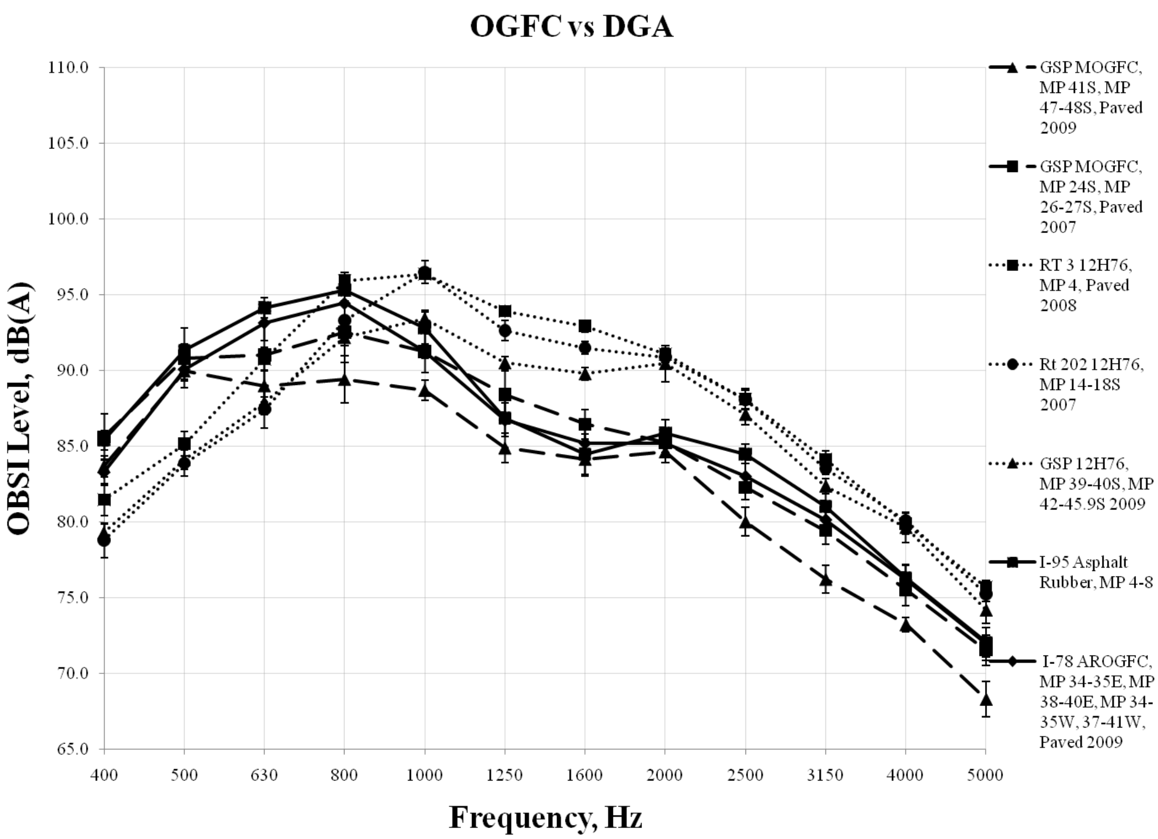

Figure 7 – Noise Spectrum Analysis for Open Graded Friction Course Mixes vs. Dense Graded Mixes

AROGFC vs. MOGFC

The comparison of the AROGFC to MOGFC results is interesting. As shown in Figure 2, the AROGFC sections were both louder overall. The 1997 AROGFC from the I-95 section overall OBSI level was measured at 100.5 dBA and the I-78 AROGFC overall OBSI level was measured at 99.6 dBA, while the 1997 MOGFC overall OBSI level was measured at 99.1 dBA and the 2009 MOGFC overall OBSI level was measured at 96.8 dBA. The important note to make about the difference between the AROGFC and MOGFC materials is the relative change in the noise spectrum properties over time. In comparing the test sections evaluated to date, it appears that the MOGFC became significantly louder over the three years of in-service life, while the AROGFC seems to have changed an insignificant amount over the 3 year in-service life. Given that the AADT on the AR sections were over 16 times higher, one would expect to see more of a change in the AROGFC, which is not apparent from the measurements. Continued testing of these sections over time is planned to continually monitor these trends.

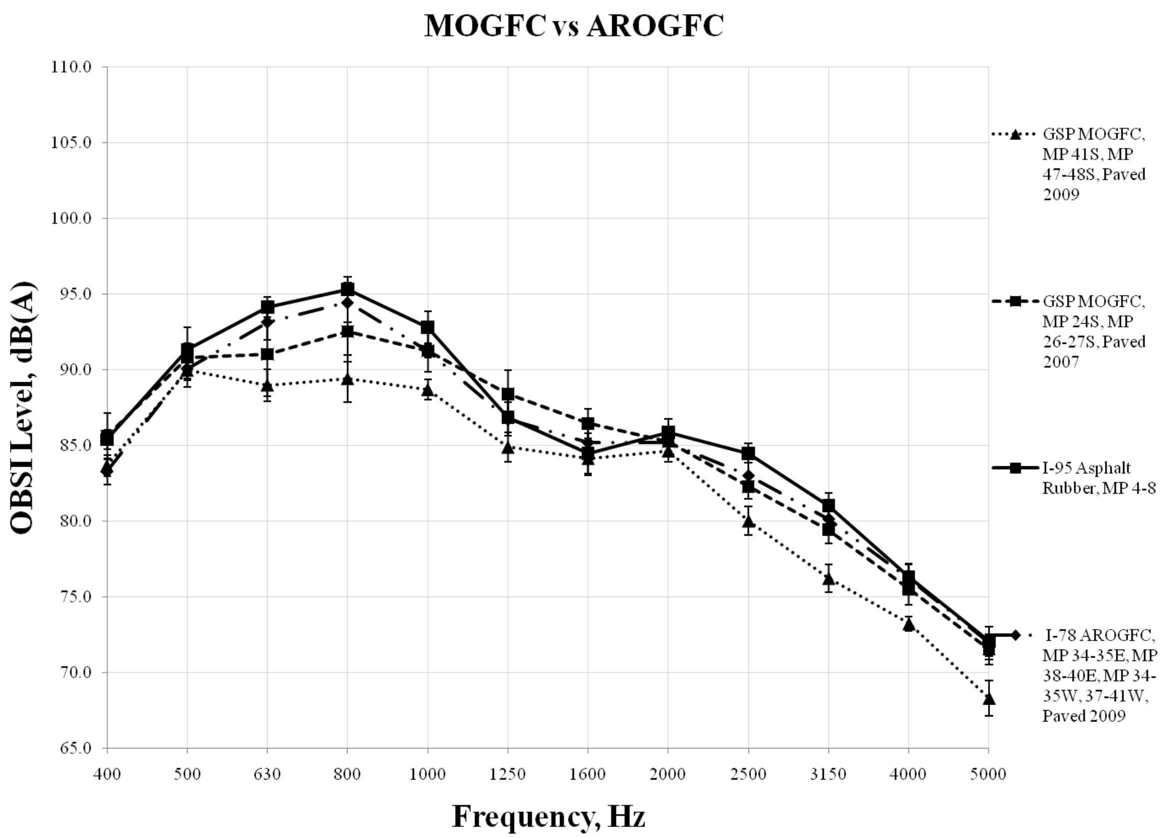

Figure 8 – Noise Spectrum Analysis for MOGFC vs. AROGFC

SUMMARY AND CONCLUSIONS

In conclusion, an OBSI testing program was initiated with the intention of creating a "quiet pavement" database for the state of New Jersey including the overall sound intensity measurements and the spectrum analyses for each material. To begin the program with help from the NJDOT, several pavement surfaces typically found in New Jersey were tested with the OBSI method. The pavement surfaces were chosen to gather a representation of the pavement types currently in-service within the state. Functional overlays that had been suspected of having properties of quiet pavements were included in this study. The materials tested and presented within this paper were primarily chosen in order to provide the NJDOT with additional information for material selections regarding noise properties, to be used in conjunction with other functional and structural information for rehabilitation projects in the future.

Initial findings show that based on the test sections evaluated in this study, in-service PCC pavements are considerably louder than the asphalt pavements in service in the state of New Jersey. It can also be seen that the dense graded asphalt sound intensity levels seem to be dependent on their location, which is presumed to be due to their respective traffic patterns, traffic volume, and age differences. As stated earlier, the initial hypothesis for the discrepancies in dBA levels with the DGA's ageing could possibly be due to general wearing in the wheelpath area, as well as the possible initiation of surface distress creating a higher ratio of macro-texture which negatively affects the sound intensity levels.

The analysis of the open-graded surface courses showed some promising results along the lines of possible aging affects with different open graded pavements. Of the pavement sections tested to date, the data suggests the MOGFC pavements in NJ have resulted in the lowest sound intensity levels, with the lowest overall sound level measured to date being 96.8 dBA on the Garden State Parkway MOGFC. However, when comparing the three year old MOGFC, there is a significant increase in noise level measured, possibly due to aging and wearing characteristics of this material. Therefore, based on the preliminary data, it would suggest that the AROGFC may be a better option over the MOGFC pavements. Even though the initial sound intensity level of the AROGFC was not as low as the MOGFC, it appears that the asphalt rubber additive may help to maintain the pavement for a longer service life by limiting the detrimental effects of asphalt age hardening, while retaining the noise mitigating properties over time.

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Title: An Evaluation of Consumer Tire/Pavement Noise Utilizing the On-Board Sound Intensity Method

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ABSTRACT

Recently tire manufacturers have been advertising consumer tires that exhibit “quiet” behavior. A comprehensive tire/pavement noise evaluation was conducted to evaluate any quantitative differences in noise generated at the source by three consumer tires including a Bridgestone Ecopia™ “quiet” tire, a Continental ProContact™ with Eco-Plus Technology low-rolling resistance tire, and a Firestone Winterforce™ “Quiet” winter tire compared to a Standard Reference Test Tire using the On-Board Sound Intensity method. Due to design constraints, two different tire pressures were tested for each of the consumer tires, first to ensure proper comparison to the Standard Reference Test Tire, then to ensure that the tires were tested under performance specifications to maximize any benefits of each design. In order to avoid a bias from pavement type three distinctly different pavement surfaces were selected including a Portland Cement Concrete pavement, an Open Graded Friction Course and a Dense Graded Asphalt pavement.

This paper discusses the testing methodology, the morphology of each tire, the overall noise levels for the different tires on each pavement, and the one-third octave band spectra for each of the tires on each of the pavements. The final comparison of the tires evaluated has the tires listed in rank order from quietest being the Continental, then the Bridgestone, then the SRTT, with the Firestone being the loudest.

INTRODUCTION

Recently tire manufacturers have been advertising "quiet" tires as additions to their line of "green" tires. These claims have not only attained the attention of consumers but also researchers who have been investigating highway noise at the source. To quantify how loud or quiet these "quiet" tires are at the tire pavement interface, a small but comprehensive study was initiated utilizing the On-Board Sound Intensity (OBSI) method. By designing a controlled experiment with three different test pavements and four different tires, researchers were able to measure the noise generated at the tire/pavement interface for each of the tires and determine the discrete differences between each of the consumer tires. Each tire was related to a Standard Reference Test Tire (SRTT) to show the relative noise levels for each pavement.

METHODOLOGY

A comprehensive testing plan was developed to isolate the discrete differences in the tires themselves. The OBSI testing performed throughout this study adhered to the AASHTO TP 76-11 (*1*) specification when possible and exceeded it in such instances as the addition of taking tire tread depth measurements and an increased number of durometer hardness measurements on the tire. Testing utilizing tires other than the SRTT and testing performed at tire inflation pressures other the specified 30 ±1psi (206.7 ± 6.89 kPa) (*1*) were the two main deviations from the test specification but were deemed acceptable to fit the scope of the study. The rear passenger wheel of the test vehicle was slightly heavier than the specified recommended weight, but consistency was maintained throughout the experiment at approximately 900 pounds (410 kg).

Pavements

The layout of the OBSI testing sections consisted of three different test sections that provided three distinctly different pavement surfaces. The first section was a Portland cement concrete pavement (PCC) on I-287 near Wanaque, NJ. The elevation on the I-287 section ranged between 260 - 320 feet above sea level. The average annual daily traffic (AADT) measured in this section was 67,187 in 2008. The second section was an Open Graded Friction Course (OGFC) functional asphalt overlay on I-78 near Basking Ridge, NJ. The elevation on the I-78 section ranged between 230 - 400 feet above sea level. The AADT near the I-78 section was recorded at 41,268 in 2008. Finally, the third section was a Dense Graded Asphalt (DGA) located on I-80 near Hope, NJ. The elevation on I-80 ranged between 480 – 500 feet above sea level. The AADT near the I-80 section was 46,532 in 2009. Each of these sections provided as many constants as could be provided within NJ test sections, while still providing three distinct wearing courses, with similar elevation and similar local traffic loading.

Tires

Four different tires were utilized for this tire study. The Standard Reference Test Tire (SRTT) typically used in the OBSI method was chosen as a general control for testing since it is the standard tire used in AASHTO TP-76 11 (*1*). The first tire chosen for investigation was the Bridgestone Ecopia™ which the manufacturer purported to be a "quiet" tire (*2*). The second tire chosen, due to general consumer interest to save money on fuel and reduce their carbon footprint was the Continental ProContact™, which is purported by the manufacturer to be a low rolling resistance, low CO_2, and high mileage tire (*3*). Finally, to investigate a more aggressive tread pattern, the fourth tire chosen was the Firestone Winterforce™ which similarly is purported by the manufacturer to be quiet while still affording extra traction in winter conditions due to its aggressive tread pattern (4). In order to enable equal comparisons between all four tires, a single size of P225 60R16 was used since it is the size of the SRTT (*5*). The same model 16 inch (406.4 mm) rim manufactured by American Racing was utilized to mount each tire. Each tire was professionally mounted and balanced at a local third party tire service center. The four tires can be seen below in Figure 1; from left to right is the Firestone Winterforce™, Bridgestone Ecopia™, SRTT, and the Continental ProContact™. The serial numbers and build dates for each tire can be found in Table 1 below.

FIGURE 1 A visual tread comparison of each tire.

Table 1 Tire Manufacturer Data

Tire Name	Serial Number	Build Date (Week)	Build Date (Year)
SRTT	ANX0EVUU	15th	2009
CONTINENTAL	P5X33X5	33rd	2011
FIRESTONE	VNX3WW68	48th	2011
BRIDGESTONE	OBX0E26	7th	2012

Before the tires were used, the hardness of each tire was determined following the ASTM D-2240-05 specification (*6*), which required a minimum of 5 hardness measurements per tire per test. For the purposes of this study, each tread pattern was measured at a minimum of 5 standard locations across the tread pattern and repeated radially around the tire at 5 equally spaced intervals. A minimum of 25 measurements per tire were recorded, well in abundance of the specification. In addition to hardness, tread depth was measured by following the ASTM F421-07 specification (*7*). Fifty or more tread depth measurements were recorded for each tire at discrete tread block locations. Before the tires were used for OBSI testing, a 300 mile (483 km) loop was followed with each tire to condition each tire evenly. Finally after OBSI field-testing was completed on each tire, the hardness and tread depth measurements were repeated to record any change.

OBSI Methodology

On each roadway tested, a minimum of six and maximum of ten 440' (134.2 m) sections were utilized to ascertain a respectable average for each pavement type. One discrete 440' (134.2 m) control test section was utilized on each pavement to provide additional field data throughout testing including ambient temperature. A minimum of three measurements were taken at each 440' (134.2 m) test section although often up to six measurements were collected for better representations of each section and tire. The weight of the vehicle was measured on site before the commencement of each test period to determine the passenger rear tire weight. Although the newer recommended weight for the right rear tire was exceeded slightly, the weight was kept as standardized as possible and the same combination of noise technicians was used throughout the experimental period. Table 2 below shows the weights collected over the course of testing.

TABLE 2 Tire Loads Prior to Each Test (lbs.)

Date	Road	Driver Front	Passenger Front	Driver Rear	Passenger Rear
6/11/2012	I-287	1265	1225	903	902
6/11/2012	I-287	1269	1201	964	944
6/13/2012	I-287	1260	1207	905	897
6/14/2012	I-287	1259	1194	892	915
6/14/2012	I-287	1256	1244	895	892
6/15/2012	I-78	1251	1231	848	937
6/15/2012	I-78	1242	1219	909	880
6/15/2012	I-78	1237	1229	899	913
6/18/2012	I-78	1276	1205	814	919
6/18/2012	I-78	1225	1239	890	909
6/19/2012	I-78	1226	1251	880	912
6/19/2012	I-78	1247	1222	903	901
6/20/2012	I-80	1242	1225	905	902
6/20/2012	I-80	1210	1263	904	890
6/20/2012	I-80	1197	1263	897	891
6/21/2012	I-80	1236	1249	874	904
6/21/2012	I-80	1230	1257	879	903
6/26/2012	I-80	1222	1270	869	902
6/26/2012	I-80	1222	1255	877	907

Tire Impressions

To help evaluate the differences between the tires, tire impressions were made using a tactile pressure film. The film, distributed and analyzed by Sensor Products Inc., was Fujifilm Prescale® Ultra Low pressure film which is capable of measuring contact pressures from 28-85psi (172-586 kPa). The measurements taken on the pressure film were conducted with the driver and testing technician in the car with all of the equipment installed as it would be to collect OBSI measurements. The measurement time, date, exposure time, ambient temperature, humidity, inflation pressure, tire name, tire serial number position, and tread direction in relation to the sheet were recorded. The exposure time and relative humidity were important for the Topaq® analysis, which was a computer analysis provided by Sensor Products Inc. The Topaq® analysis was conducted as part of package purchased with the pressure film. It provided the average contact pressure, the contact area, the total area of the tire patch, and the force exerted. Figure 2 below illustrates the visual pressure grid provided along with the Topaq® analysis. The SRTT is located at the top center of the figure. For the remainder of the figure the left column of images the imprints taken with the tire inflated to 30 psi (207 kPa) and the right column of images are the impressions taken at the tire inflation pressure of 44 psi (303 kPa). Following the SRTT in top down order is the Continental ProContact™, Firestone Winterforce™, and finally the Bridgestone Ecopia™. Impressions for both 30 and 44 psi (207 and 303 kPa) were taken because testing was conducted at both inflation pressures for all tires but the SRTT. This is because AASHTO TP 76-11 (*1*) calls for the testing tire to be inflated to 30 psi ± 1 psi (207 ± 6.89 kPa). However the recommended inflation pressure for each consumer tire was 44 psi (303 kPa). Thus to thoroughly investigate the noise quality of each of the tires, both the AASHTO TP 76-11 (*1*) testing pressure and the manufacturers designed inflation pressures were utilized.

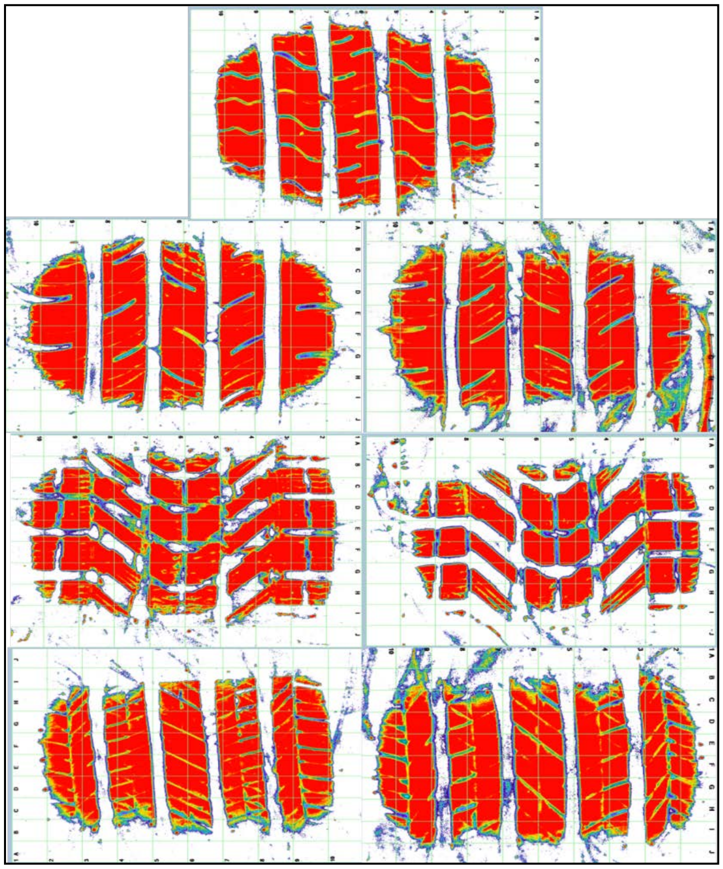

FIGURE 2 A visual representation of each tire pressure impression.

RESULTS

Tire Changes

The changes in hardness from before and after testing are shown below in Table 3. As seen in Table 2 the Bridgestone had the largest change in hardness measurements followed by the Continental. The minimal changes in the SRTT and Firestone can be deemed as negligible since the scale of the actual durometer is only measured out to whole number deviations. It is also interesting to note the SRTT was consistently the hardest tire, which is most likely attributed to oxidative aging since the SRTT had a significantly earlier build date.

TABLE 3 Durometer Hardness Measurements

Tire	Before	After	Change
SRTT	67.7	67.1	-0.6
Continental Eco-Plus ProContact™	63.6	62.0	-1.6
Firestone Winterforce™	59.7	59.4	-0.3
Bridgestone Ecopia™	59.2	57.1	-2.1

The before and after tread depth measurements are shown below in Table 4. The Continental ProContact™ experienced the most tread wear at the completion of the OBSI testing. The technician that inspected the tires also noted that the Continental exhibited more wear and did not appear to be in as stable of a condition as the other tires.

TABLE 4 Tread Depth Measurements (inches)

Tire	Before	After	Change
SRTT	0.300	0.293	-0.006
Continental ProContact™	0.311	0.300	-0.011
Firestone Winterforce™	0.411	0.404	-0.007
Bridgestone Ecopia™	0.292	0.290	-0.002

OBSI Results

The average overall OBSI levels were significantly different between the tires. Figure 3 below, shows the recorded average overall OBSI levels for all four tires on each pavement. Regardless of pavement surface, the tires consistently showed results in order of loudest to quietest with the Firestone Winterforce™ always being the loudest, followed by the SRTT, Bridgestone and finally the Continental respectively. The Firestone had the highest average overall OBSI level recorded throughout the entire study by at least 2 dB(A) at 106.6 dB(A) on the PCC pavement surface. The Continental offered the lowest average overall OBSI level recorded throughout the study on the OGFC with an average overall OBSI level of 98.6 dB(A).

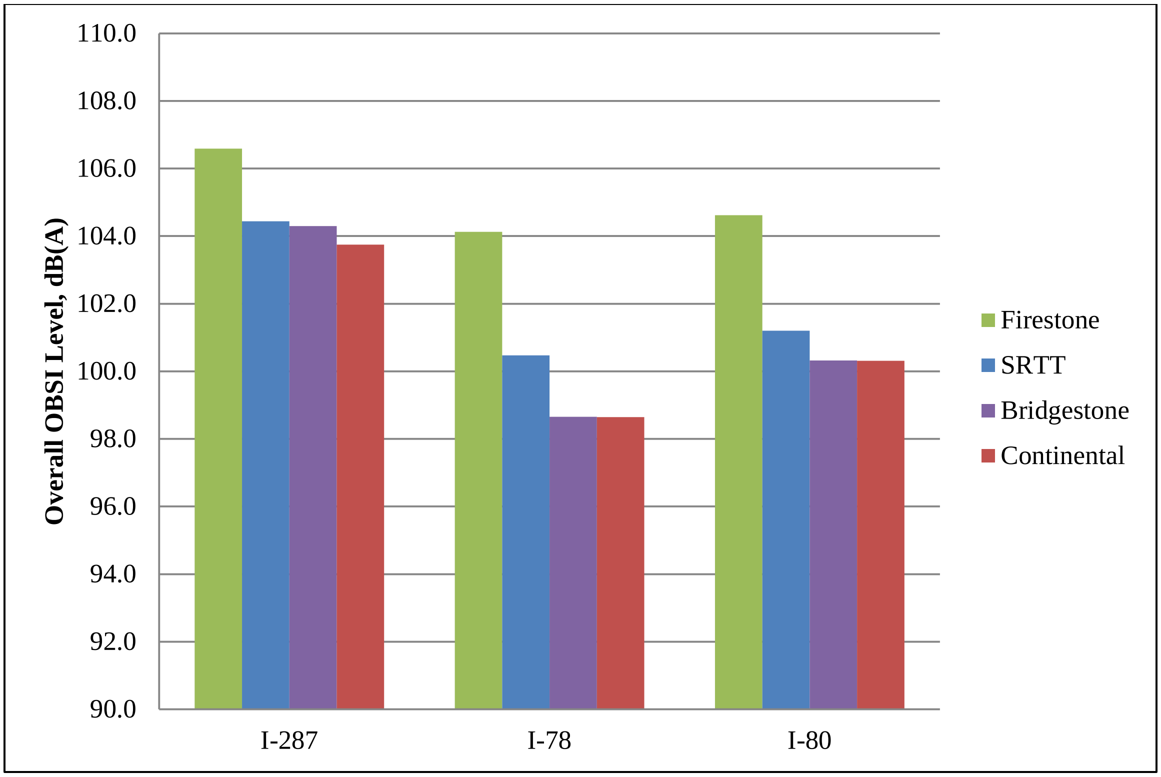

FIGURE 3 The compiled results of the average overall OBSI levels.

Figures 4 through 6 below show how each tired responded spectrally when tested on each surface type utilizing the 30 psi (207 kPa) inflation pressure. It can be seen that all of the tires with the exception of the Firestone responded similarly on the PCC. The Firestone however, appears to have a significant peak between the 630 Hz center frequency and the 1000 Hz center frequency. This peak is the dominating factor explaining the higher average overall OBSI levels for the Firestone. The comparison of the tires on the OGFC also shows the same features with the exception that the SRTT also slightly peaks in the same frequency range as the Firestone but with less amplitude. The differentiation between all the tires in this specified range reveals where the tire tread design and rubber hardness start to dominate noise generation at the source.

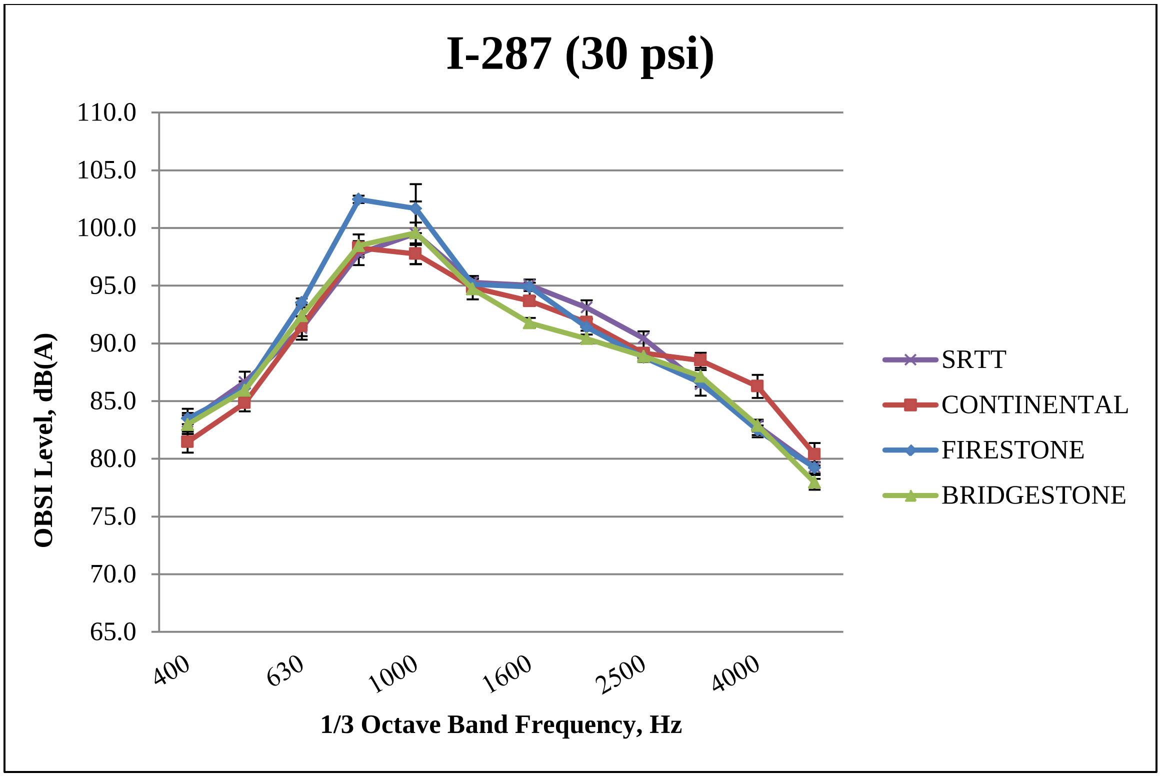

FIGURE 4 The I-287 spectral responses for each tire at 30 psi.

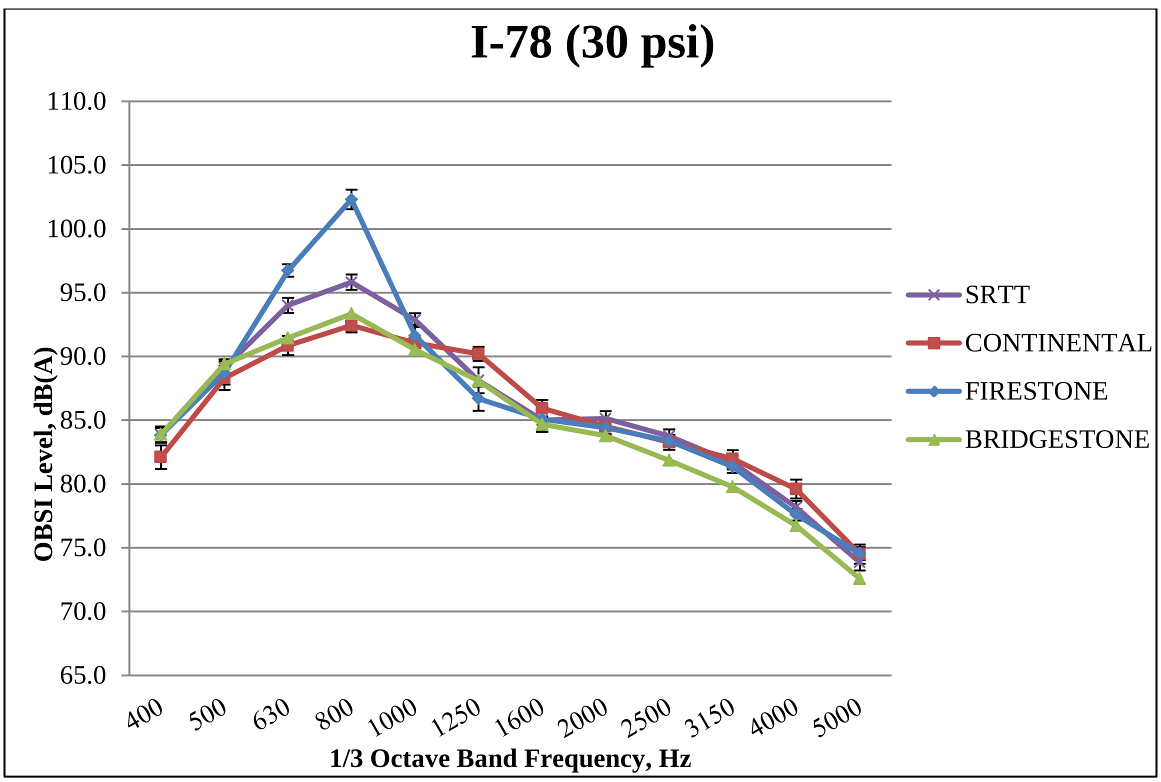

FIGURE 5 The I-78 spectral responses for each tire at 30 psi.

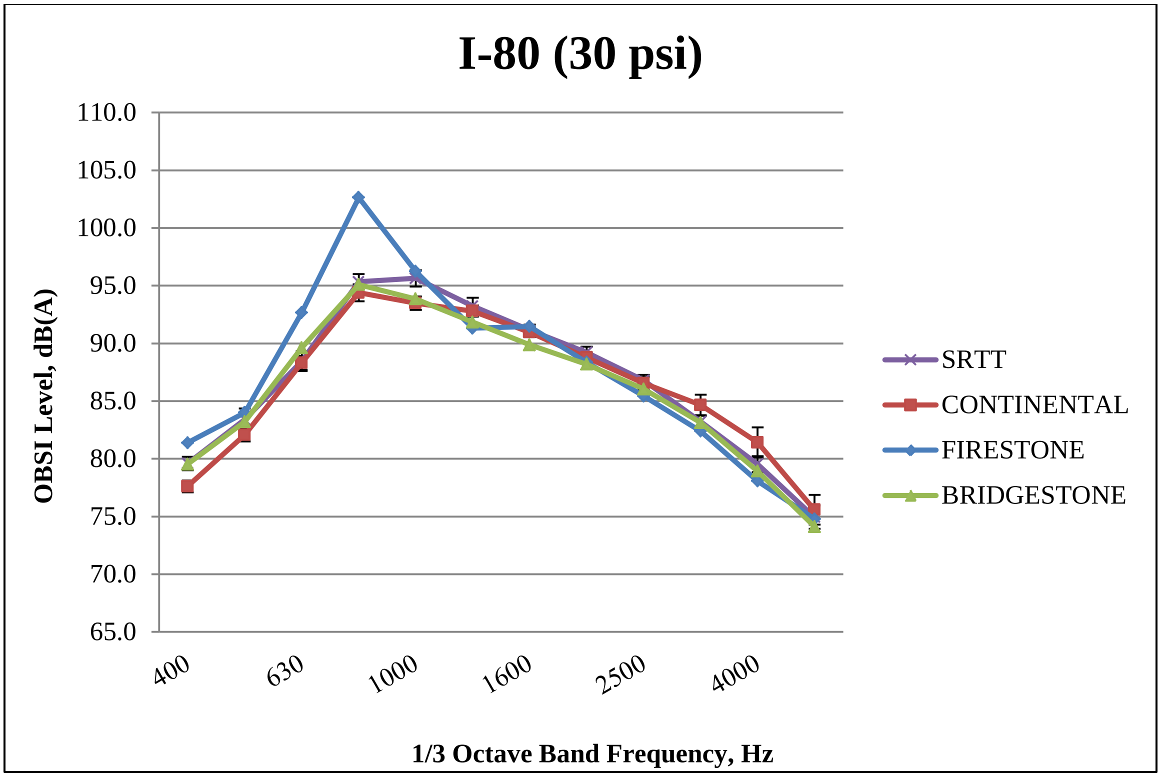

FIGURE 6 The I-80 spectral responses for each tire at 30 psi.

Figures 7 through 9 below show how each tire responded spectrally when testing using a tire pressure of 44 psi (303 kPa) on the different pavement surfaces. These figures omit the SRTT because no testing was performed using the SRTT at 44 psi (303 kPa) for this study. Similar spectral responses from each tire occurred for an inflation pressure of 44 psi (303 kPa) as previously seen for 30 psi (207 kPa) on the different pavement surfaces. However it is interesting to note that the Bridgestone and Continental both seemed to peak at the 1000 Hz center frequency similar to the Firestone due to the change in tire pressure on the PCC. This peak for the Bridgestone and Continental was not as noticeable on the asphalt pavements potentially due to the smoothness and air voids associated with the asphalt pavements tested.

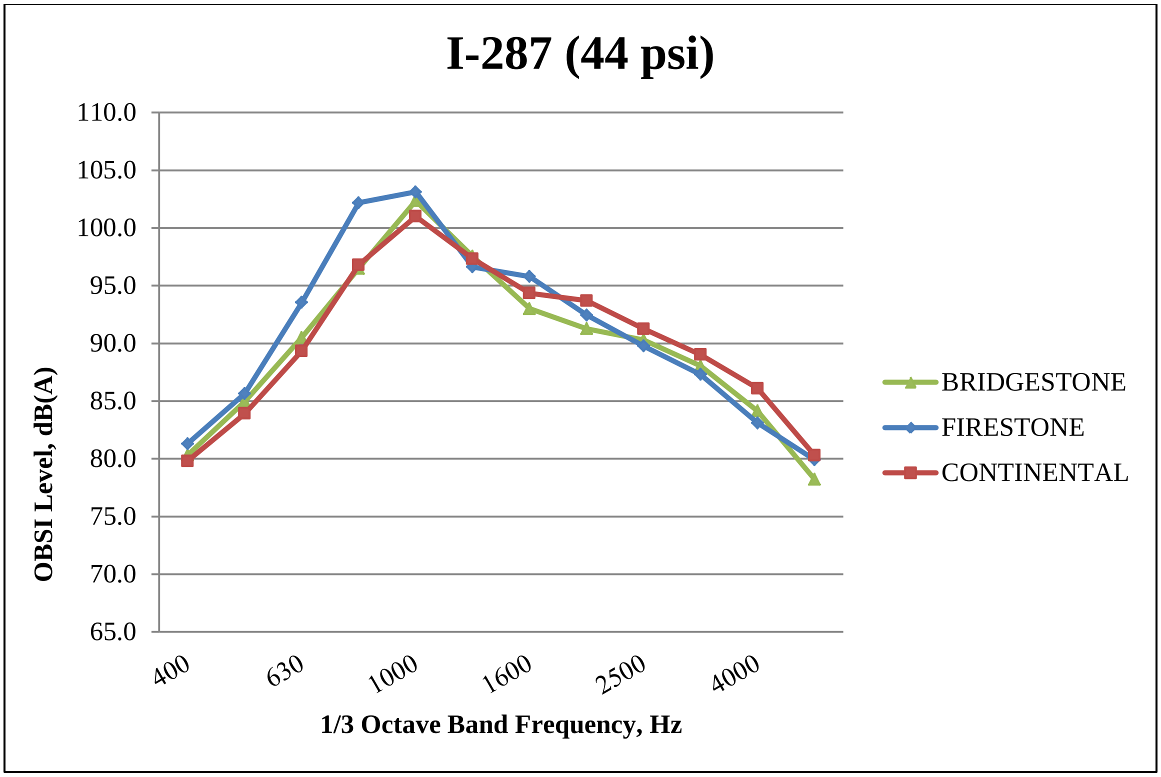

FIGURE 7 The I-287 spectral responses for each tire at 44 psi.

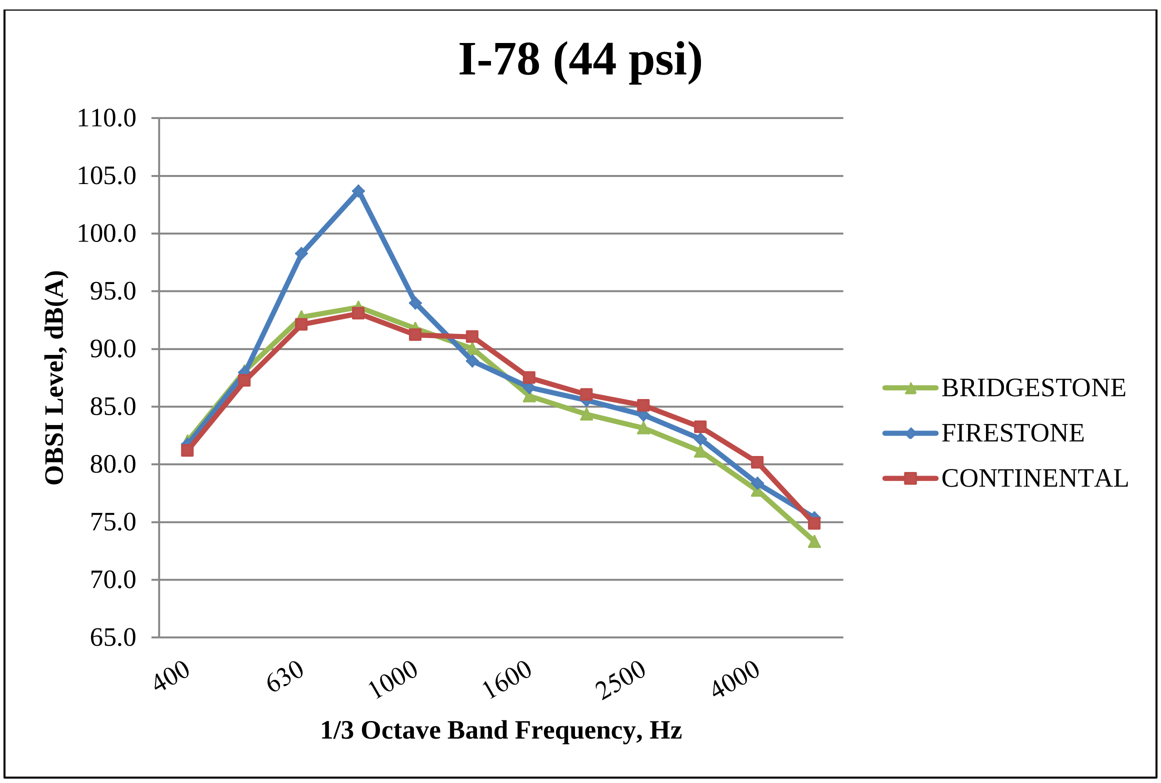

FIGURE 8 The I-78 spectral responses for each tire at 44 psi.

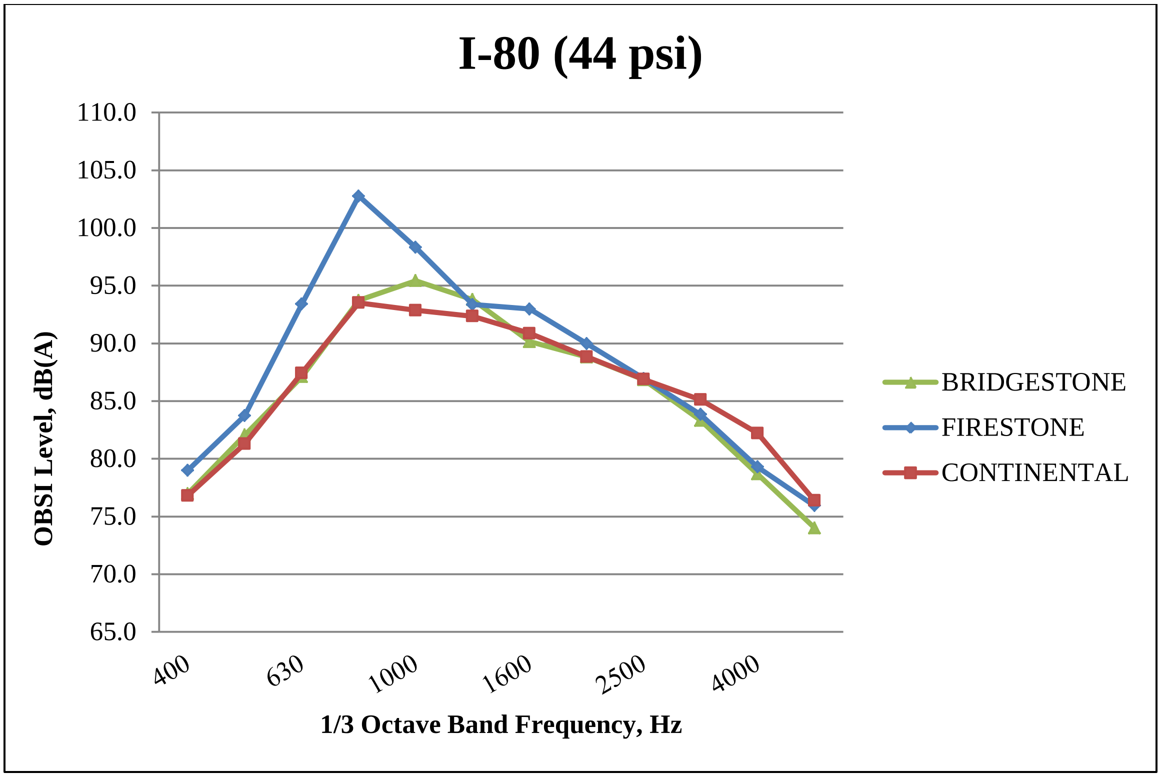

FIGURE 9 The I-80 spectral responses for each tire at 44 psi.

To investigate the Firestone further all measurements taken for each inflation pressure on each pavement surface were overlaid with each other in Figure 10 below. Figure 10 clearly demonstrates that regardless of inflation pressure or pavement surface the Firestone consistently peaked at the 800 Hz center frequency to a nearly identical dB(A) level. This clearly demonstrates that this spike can be attributed as mechanism of the tire itself, most likely the distinct aggressive tread design.

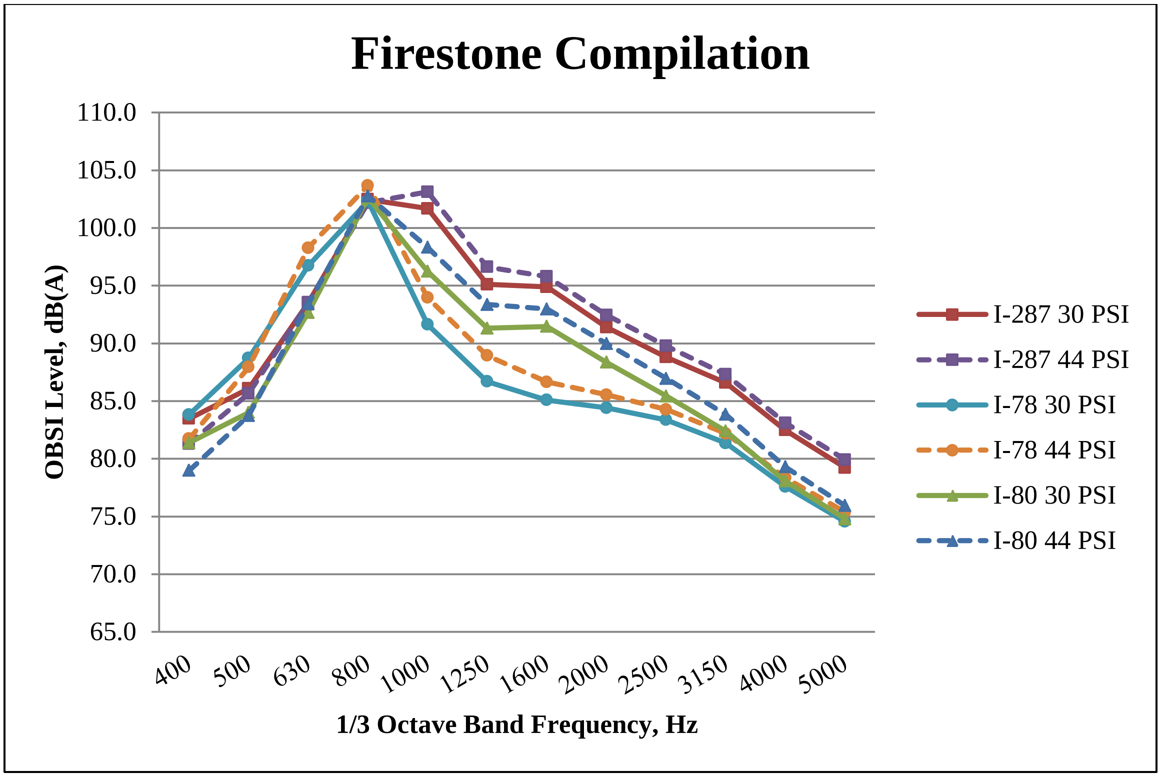

FIGURE 10 The compiled Firestone spectral responses.

Figures 11 and 12 below show how the spectral response changed due to inflation pressure changes for both the Continental and Bridgestone respectively. For each tire tested the average overall OBSI levels increased with an increase in inflation pressure. This can be explained with the Topaq® analysis due to higher pressures exerted at the tire/pavement interface.

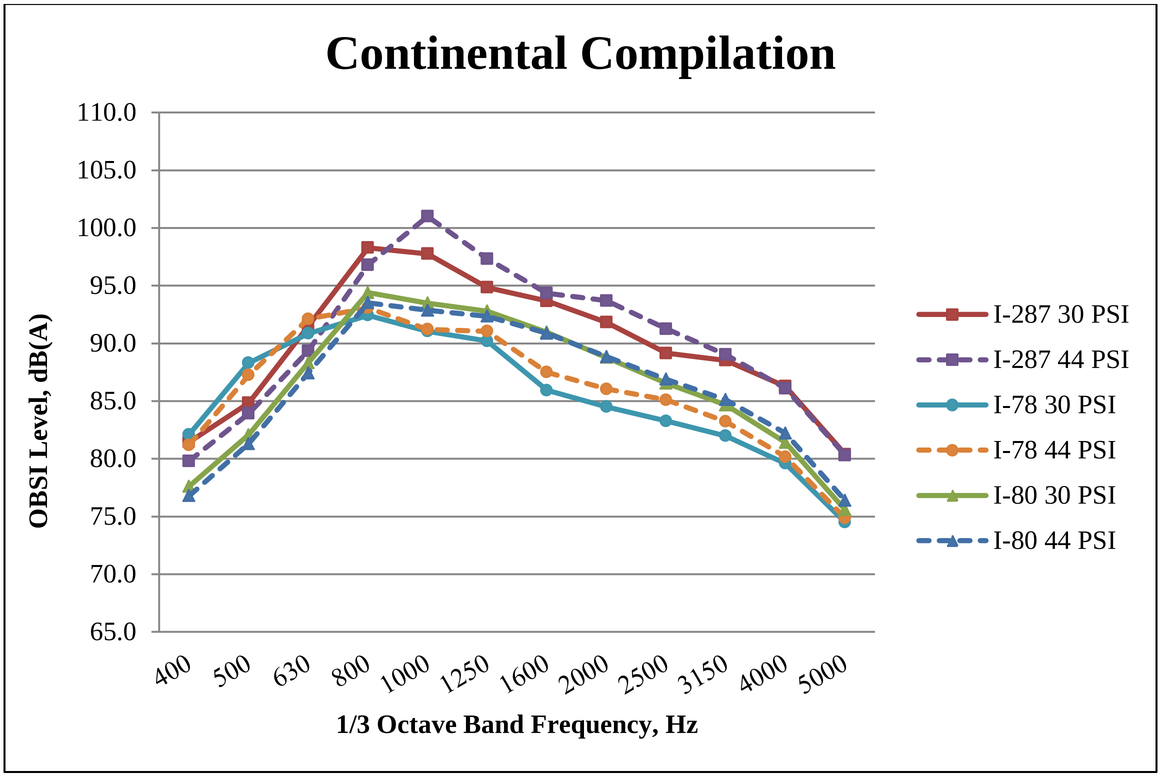

FIGURE 11 The compiled Continental spectral responses.

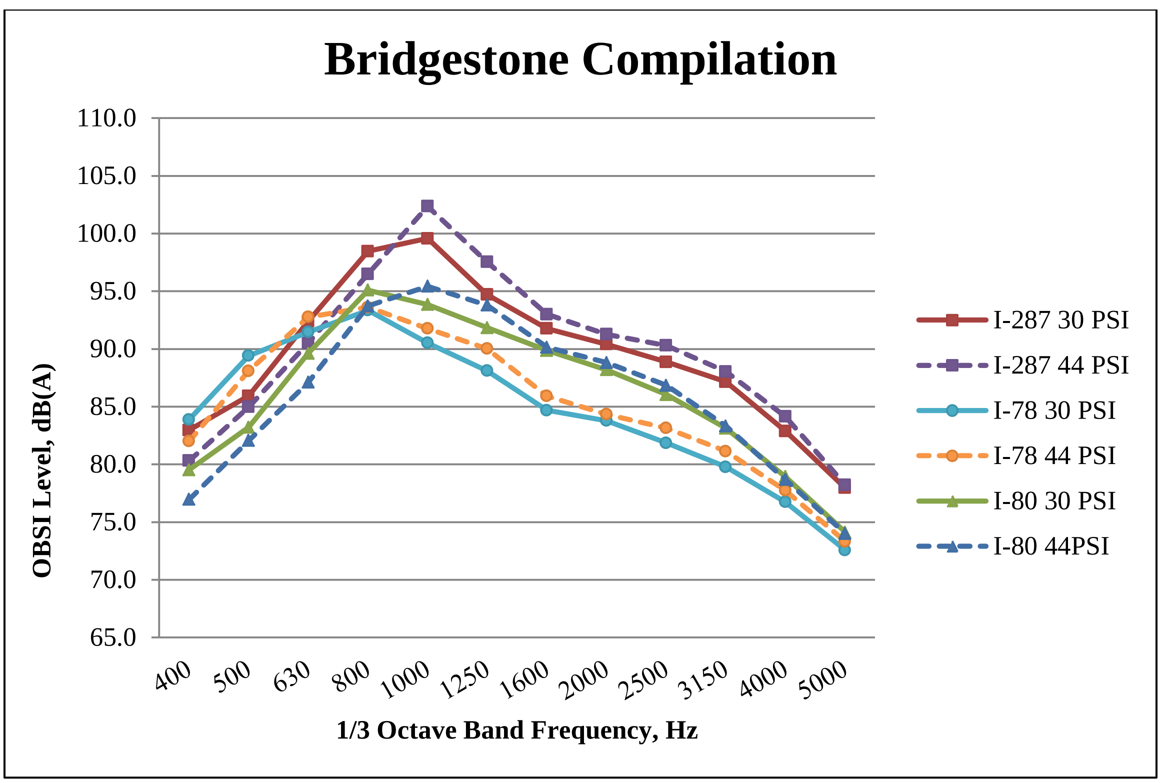

FIGURE 12 The compiled Bridgestone spectral responses.

CONCLUSIONS

In conclusion, the purpose of the study was to quantify the effective differences in specifically chosen modern consumer tires. Three tires were selected due to their manufacturer's advertised properties. A purported quiet tire, a "green" low-rolling resistance tire and a "quiet" winter tire were ultimately selected for testing. Three distinctly different pavement types were selected to investigate how the different tires would react on different surfaces to help prevent any bias. During the study, hardness and tread depth measurements were taken before and after testing to record any physical changes in the tires, tire impressions to understand the force distribution of the tire and vehicle weights prior to all OBSI testing sessions to ensure similar testing conditions for a fair comparison.

The Continental, although marketed as a low rolling resistance tire (3), was found to be the quietest tire overall by the authors. On the PCC the Continental was quieter by a minimum of .6 dB(A) when compared to any of the other tires. It was quieter on the OGFC by a minimum of .1 dB(A) when compared with any of the other tires tested. Lastly the Continental was found to be equal in overall OBSI level to the Bridgestone on the DGA with average levels of 100.3 dB(A). It should be noted however that the Continental did show the greatest tread-wear out of all of the tires, with almost double the amount of wear of the SRTT. As stated earlier the technician that inspected the tires after testing was completed also stated concern about the structural integrity of the tire.

The Bridgestone, the purported "quiet" tire (*2*), was ranked as the second quietest in average overall OBSI level out of the consumer tires tested. The Bridgestone was found to be 0.1 dB(A) quieter than the SRTT on the PCC section, 0.9 dB(A) quieter than the SRTT on the DGA pavement and 1.8 dB(A) quieter then the SRTT on the OGFC section.

The Firestone Winterforce proved to be the loudest tire of the consumer tires tested in this study. This was expected due to the aggressive tread pattern design. The Firestone proved to be minimally 2.2 dB(A) louder in average overall OBSI level on the PCC than any other tire tested. It performed similarly as the loudest tire on the DGA section with minimum average overall OBSI level of 3.4 dB(A) higher than any other tire. Finally on the OGFC section the largest difference average overall OBSI level was 3.6 dB(A) greater than any other tire tested. When analyzing the data the researchers also noticed a reoccurrence of a large spike in intensity level at the 800 Hz center frequency for the Firestone. This feature maintained similar intensity despite varying pavements, pressures and overall values which led the authors to believe that it is a design property of the tire itself.

Other than the Firestone, the tires all followed the same general trends in generated noise response. The Firestone had higher intensity levels between the 500 Hz center frequency and 1000 Hz center frequency range, which led to its higher measured average overall OBSI levels. It was also shown that regardless of the tire being tested, increased inflation pressure led to increased overall loudness. This can be potentially explained by the Topaq® analysis which showed the increased reaction pressures in Figure 2. In conclusion, the final comparison of the tires tested leaves the Continental as the quietest tested tire overall, the Bridgestone as the second quietest, the SRTT as the third quietest and finally the Firestone being loudest.

ACKNOWLEDGEMENTS

Paul Calado from Sensor Products Inc. was instrumental in the selection of the Fujifilm Prescale® Ultra Low film and for conducting the Topaq® analysis.

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6) ASTM Standard D2240, *Standard Test Method for Rubber Property-Durometer 37 Hardness*, ASTM, West Conshohocken, PA.

7) ASTM Standard F421-07, *Standard Test Method for Measuring Groove and Void Depth in Passenger Car Tires*, ASTM, West Conshohocken, PA.

SECTION 401 – HOT MIX ASPHALT (HMA) COURSES

ADD THE FOLLOWING TO 401.01:

401.01 DESCRIPTION

This Section also describes the requirements for constructing a Hot Mix Asphalt (HMA) course with required minimum amounts of Reclaimed Asphalt Pavement (RAP).

ADD THE FOLLOWING TO 401.02.01:

401.02.01 Materials

Hot Mix Asphalt HIGH RAP 902.11

ADD THE FOLLOWING SUBSECTION TO 401.03:

401.03.07 Hot Mix Asphalt (HMA) HIGH RAP

A. Paving Plan. At least 20 days before beginning placing the HMA HIGH RAP, submit a detailed plan of operation as specified in 401.03.03.A to the RE for approval. Include in the paving plan a proposed location for the test strip. Submit for Department approval a plan of the location for the HMA HIGH RAP on the project.

B. Weather Limitations. Place HMA HIGH RAP according to the weather limitations in 401.03.03.B.

C. Test Strip. Construct a test strip as specified in 401.03.03.C.

D. Transportation and Delivery of HMA. Deliver HMA HIGH RAP as specified in 401.03.03.D.

E. Spreading and Grading. Spread and grade HMA HIGH RAP as specified in 401.03.03.E. Record the laydown temperature (temperature immediately behind the paver) at least once per hour during paving. Submit the temperatures to the RE and to the HMA Plant producing the HMA HIGH RAP.

F. Compacting. Compact HMA HIGH RAP as specified in 401.03.03.F.

G. Opening to Traffic. Follow the requirements of 401.03.03.G for opening HMA HIGH RAP to traffic.

H. Air Void Requirements. Ensure that the HMA HIGH RAP is compacted to meet the air void requirements as specified in 401.03.03.H.

I. Thickness Requirements. Ensure that the HMA HIGH RAP is paved to meet the thickness requirements as specified in 401.03.03.I.

J. Ride Quality Requirements. Ensure that the HMA HIGH RAP is paved to meet the ride quality requirements as specified in 401.03.03.J

ADD THE FOLLOWING TO 401.04:

401.04 MEASUREMENT AND PAYMENT

The Department will measure and make payment for Items as follows:

Item	*Pay Unit*
HOT MIX ASPHALT ___ ___ ___ SURFACE COURSE HIGH RAP	TON
HOT MIX ASPHALT ___ ___ ___ INTERMEDIATE COURSE HIGH RAP	TON
HOT MIX ASPHALT ___ ___ ___ BASE COURSE HIGH RAP	TON

ADD THE FOLLOWING TO 902:

902.11 HOT MIX ASPHALT RAP

902.11.01 Mix Designations

The requirements for specific HMA mixtures with required minimum amounts of RAP are identified by the abbreviated fields in the Item description as defined as follows:

HOT MIX ASPHALT 12.5H64 SURFACE COURSE HIGH RAP

1. “HOT MIX ASPHALT” “Hot Mix Asphalt” is located in the first field in the Item description for the purpose of identifying the mixture requirements.
2. “12.5” The second field in the Item description designates the nominal maximum size aggregate (in millimeters) for the job mix formula (sizes are 4.75, 9.5, 12.5, 19, 25, and 37.5 mm).
3. “H” The third field in the Item description designates the design compaction level for the job mix formula based on traffic forecasts as listed in Table 902.02.03-2 (levels are L=low, M=medium, and H=high).
4. “64” The fourth field in the Item description normally designates the high temperature (in °C) of the performance-graded binder (options are 64, 70, and 76 °C). In the High RAP mixes this field will designate the mix performance requirements.
5. “SURFACE COURSE” The last field in the Item description designates the intended use and location within the pavement structure (options are surface, intermediate, or base course).
6. “HIGH RAP” This additional field designates that there will be a minimum percentage of RAP required for the mixture in 902.011.02.

902.11.02 Composition of Mixture

Provide materials as specified:

Aggregates for Hot Mix Asphalt 901.05

Use a virgin asphalt binder that will result in a mix that meets the performance requirements specified in Table 902.11.03-2. Ensure that the virgin asphalt binder meets the requirements of 902.01.01 except the performance grade. Use a performance grade of asphalt binder as determined by the mix design and mix performance testing.

Mix HMA HIGH RAP in a plant that is listed on the QPL for HMA Plants and conforms to the requirements for HMA Plants as specified in 1009.01.

Composition of the mixture for HMA HIGH RAP surface course is coarse aggregate, fine aggregate, asphalt binder, and a minimum of 20 percent Reclaimed Asphalt Pavement (RAP), and may also include mineral filler, asphalt rejuvenator and Warm Mix Asphalt (WMA) additives or processes as specified in 902.01.05. When WMA is used it must meet the requirements as specified in 902.10. Ensure that the finished mix does not contain more than a total of 1 percent by weight contamination from Crushed Recycled Container Glass (CRCG).

The composition of the mixture for HMA HIGH RAP base or intermediate course is coarse aggregate, fine aggregate, asphalt binder, and a minimum of 30 percent Reclaimed Asphalt Pavement (RAP), and may also include mineral filler, up to 10 percent of additional recycled materials, asphalt rejuvenator, and Warm Mix Asphalt (WMA) additives or processes as specified in 902.01.05. When WMA is used it must meet the requirements as specified in 902.10. The recycled materials may consist of a combination of RAP, CRCG, Ground Bituminous Shingle Material (GBSM), and RPCSA, with the following individual limits:

Table 902.11.02-1 Use of Recycled Materials in Base or Intermediate Course

Recycled Material	Minimum Percentage	Maximum Percentage
RAP	30	
CRCG		10
GBSM		5

Combine the aggregates to ensure that the resulting mixture meets the grading requirements specified in Table 902.02.03-1. In determining the percentage of aggregates of the various sizes necessary to meet gradation requirements, exclude the asphalt binder.

Ensure that the combined coarse aggregate, when tested according to ASTM D 4791, has less than 10 percent flat and elongated pieces retained on the No. 4 sieve and larger. Measure aggregate using the ratio of 5:1, comparing the length (longest dimension) to the thickness (smallest dimension) of the aggregate particles.

Ensure that the combined fine aggregate in the mixture conforms to the requirements specified in Table 902.02.02-2. Ensure that the material passing the No. 40 sieve is non-plastic when tested according to AASHTO T 90.

902.11.03 Mix Design

At least 45 days before initial production, submit a job mix formula for the HMA HIGH RAP on forms supplied by the Department, to include a statement naming the source of each component and a report showing that the results meet the criteria specified in Tables 902.02.03-1 and 902.11.03-1.

Include in the mix design the following based on the weight of the total mixture:

1. Percentage of RAP or GBSM.
2. Percentage of asphalt binder in the RAP or GBSM.
3. Percentage of new asphalt binder.
4. Total percentage of asphalt binder.
5. Percentage of each type of virgin aggregate.

Table 902.11.03-1 HMA HIGH RAP Requirements for Design

Compaction Levels	Required Density (% of Theoretical Max. Specific Gravity)		Voids in Mineral Aggregate (VMA)[2], % (minimum) — Nominal Max. Aggregate Size, mm					Voids Filled With Asphalt (VFA) %	Dust-to-Binder Ratio
	@N_{des}[1]	@N_{max}	25.0	19.0	12.5	9.5	4.75		
L	96.0	≤ 98.0	13.0	14.0	15.0	16.0	17.0	70 - 85	0.6 - 1.2
M	96.0	≤ 98.0	13.0	14.0	15.0	16.0	17.0	65 - 85	0.6 - 1.2

1. As determined from the values for the maximum specific gravity of the mix and the bulk specific gravity of the compacted mixture. Maximum specific gravity of the mix is determined according to AASHTO T 209. Bulk specific gravity of the compacted mixture is determined according to AASHTO T 166. For verification, specimens must be between 95.0 and 97.0 percent of maximum specific gravity at N_{des}.
2. For calculation of VMA, use bulk specific gravity of the combined aggregate including aggregate extracted from the RAP.

The job mix formula for the HMA HIGH RAP mixture establishes the percentage of dry weight of aggregate, including the aggregate from the RAP, passing each required sieve size and an optimum percentage of asphalt binder based upon the weight of the total mix. Determine the optimum percentage of asphalt binder according to AASHTO R 35 and M 323 with an N_{des} as required in Table 902.02.03-2. Before maximum specific gravity testing or compaction of specimens, condition the mix for 2 hours according to the requirements for conditioning for volumetric mix design in AASHTO R 30, Section 7.1. If the absorption of the combined aggregate is more than 1.5 percent according to AASHTO T 84 and T 85, ensure that the mix is short term conditioned for 4 hours according to AASHTO R 30, Section 7.2 prior to compaction of specimens (AASHTO T 312) and determination of maximum specific gravity (AASHTO T 209). Ensure that the job mix formula is within the master range specified in Table 902.02.03-1.

Ensure that the job mix formula provides a mixture that meets a minimum tensile strength ratio (TSR) of 80% when prepared according to AASHTO T 312 and tested according to AASHTO T 283. Submit the TSR results with the mix design.

Determine the correction factor of the mix including the RAP by using extracted aggregate from the RAP in the proposed proportions when testing is done to determine the correction factor as specified in AASHTO T 308. Use extracted aggregate from the RAP in determining the bulk specific gravity of the aggregate blend for the mix design.

For each mix design, submit with the mix design forms 3 gyratory specimens and 1 loose sample corresponding to the composition of the JMF. Ensure that the samples include the percentage of RAP that is being proposed for the mix. The ME will use these to verify the properties of the JMF. Compact the specimens to the design number of gyrations (N_{des}). For the mix design to be acceptable, all gyratory specimens must comply with the requirements specified in Tables 902.02.03-1 and 902.11.03-1. The ME reserves the right to be present at the time the gyratory specimens are molded.

In addition, submit nine gyratory specimens and five 5-gallon buckets of loose mix to the ME. The ME will use these additional samples for performance testing of the HMA HIGH RAP mix. The ME reserves the right to be present at the time of molding the gyratory specimens. Ensure that the additional gyratory specimens are compacted according to AASHTO T 312, are 77 mm high, and have an air void content of 6.5 ± 0.5 percent. The ME will test six (6) specimens using an Asphalt Pavement Analyzer (APA) according to AASHTO T 340 at 64°C, 100 psi hose pressure, and 100 lb. wheel load. The ME will use the remaining three (3) specimens to test using an Overlay Tester (NJDOT B-10) at 25°C and a joint opening of 0.025 inch.

The ME will approve the JMF if the results meet the criteria in Table 902.11.03-2.

Table 902.11.03-2 Performance Testing Requirements for HMA HIGH RAP Design

Test	Requirement			
	Surface Course		Intermediate Course	
	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

If the JMF does not meet the APA and Overlay Tester criteria, redesign the HMA HIGH RAP mix and submit for retesting. The JMF for the HMA HIGH RAP mixture is in effect until modification is approved by the ME.

When unsatisfactory results for any specified characteristic of the work make it necessary, the Contractor may establish a new JMF for approval. In such instances, if corrective action is not taken, the ME may require an appropriate adjustment to the JMF.

Should a change in sources be made or any changes in the properties of materials occur, the ME will require that a new JMF be established and approved before production can continue.

902.11.04 Sampling and Testing

A. General Acceptance Requirements. The RE or ME may reject and require disposal of any batch or shipment that is rendered unfit for its intended use due to contamination, segregation, improper temperature, lumps of cold material, or incomplete coating of the aggregate. For other than improper temperature, visual inspection of the material by the RE or ME is considered sufficient grounds for such rejection.

Ensure that the temperature of the mix at discharge from the plant or storage silo meets the recommendation of the supplier of the asphalt binder, supplier of the asphalt modifier and WMA manufacturer. For HMA, do not allow the mixture temperature to exceed 330°F at discharge from the plant. For WMA, do not allow the mixture temperature to exceed 300°F at discharge from the plant.

Combine and mix the aggregates and asphalt binder to ensure that at least 95 percent of the coarse aggregate particles are entirely coated with asphalt binder as determined according to AASHTO T 195. If the ME determines that there is an on-going problem with coating, the ME may obtain random samples from 5 trucks and will determine the adequacy of the mixing on the average of particle counts made on these 5 test portions. If the requirement for 95 percent coating is not met on each sample, modify plant operations, as necessary, to obtain the required degree of coating.

B. Sampling. The ME will take 5 stratified random samples of HMA HIGH RAP for volumetric acceptance testing from each lot of approximately 3500 tons of a mix. When a lot of HMA HIGH RAP is less than 3500 tons, the ME will take samples at random for each mix at the rate of one sample for each 700 tons. The ME will perform sampling according to AASHTO T 168, NJDOT B-2, or ASTM D 3665.

Use a portion of the samples taken for volumetric acceptance testing for composition testing.

C. Quality Control Testing. The HMA HIGH RAP producer shall provide a quality control (QC) technician who is certified by the Society of Asphalt Technologists of New Jersey as an Asphalt Technologist, Level 2. The QC technician may substitute equivalent technician certification by the Mid-Atlantic Region Technician Certification Program (MARTCP). Ensure that the QC technician is present during periods of mix production for the sole purpose of quality control testing and to assist the ME. The ME will not perform the quality control testing or other routine test functions in the absence of, or instead of, the QC technician.

The QC technician shall perform sampling and testing according to the approved quality control plan, to keep the mix within the limits specified for the mix being produced. The QC technician may use acceptance test results or perform additional testing as necessary to control the mix.

To determine the composition, perform ignition oven testing according to AASHTO T 308.

For each acceptance test, perform maximum specific gravity testing according to AASHTO T 209 on a test portion of the sample taken by the ME. Sample and test coarse aggregate, fine aggregate, mineral filler, and RAP according to the approved quality control plan for the plant.

Ensure that the supplier has in operation an ongoing daily quality control program to evaluate the RAP. As a minimum, this program shall consist of the following:

1. An evaluation performed to ensure that the material conforms to 901.05.04 and compares favorably with the design submittal.
2. An evaluation of the RAP material performed using a solvent or an ignition oven to qualitatively evaluate the aggregate components to determine conformance to 901.05.
3. Quality control reports as directed by the ME.

D. Acceptance Testing and Requirements. The ME will determine volumetric properties at N_{des} for acceptance from samples taken, compacted, and tested at the HMA plant. The ME will compact HMA HIGH RAP to the number of design gyrations (N_{des}) specified in Table 902.02.03-2, using equipment according to AASHTO T 312. The ME will determine bulk specific gravity of the compacted sample according to AASHTO T 166. The ME will use the most current QC maximum specific gravity test result in calculating the volumetric properties of the HMA HIGH RAP.

The ME will determine the dust-to-binder ratio from the composition results as tested by the QC technician.

Ensure that the HMA HIGH RAP mixture conforms to the requirements specified in Table 902.11.04-1, and to the gradation requirements in Table 902.02.03-1. If 2 samples in a lot fail to conform to the gradation or volumetric requirements, immediately initiate corrective action.

The ME will test a minimum of 1 sample per lot for moisture, basing moisture determinations on the weight loss of an approximately 1600-gram sample of mixture heated for 1 hour in an oven at 280 ± 5°F. Ensure that the moisture content of the mixture at discharge from the plant does not exceed 1.0 percent.

Table 902.11.04-1 HMA HIGH RAP Requirements for Control

Compaction Levels	Required Density (% of Theoretical Max. Specific Gravity)	Voids in Mineral Aggregate (VMA), % (minimum)					Dust-to-Binder Ratio
		Nominal Max. Aggregate Size, mm					
	@Ndes[1]	25.0	19.0	12.5	9.5	4.75	
L, M	95.0 – 98.5	13.0	14.0	15.0	16.0	17.0	0.6 - 1.3

1. As determined from the values for the maximum specific gravity of the mix and the bulk specific gravity of the compacted mixture. Maximum specific gravity of the mix is determined according to AASHTO T 209. Bulk specific gravity of the compacted mixture is determined according to AASHTO T 166.

E. Performance Testing for HMA HIGH RAP. Provide five (5) 5-gallon buckets of loose mix to the ME for testing in the Asphalt Pavement Analyzer (APA) and the Overlay Tester device. Ensure that the first sample is taken during the construction of the test strip as specified in 401.03.07.C. Thereafter, sample every lot or as directed by the ME. If a sample does not meet the design criteria for performance testing as specified in Table 902.11.03-2, the Department will assess a pay adjustment as specified in Table 902.11.04-2. If a lot fails to meet requirements for both APA and Overlay Tester, the Department will assess pay adjustments for both parameters. The Department will calculate the pay adjustment by multiplying the percent pay adjustment (PPA) by the quantity in the lot and the bid price for the HMA High RAP item.

Table 902.11.04-2 Performance Testing Pay Adjustments for HMA HIGH RAP

	Surface Course		**Intermediate Course**		**PPA**
	PG 64-22	**PG 76-22**	**PG 64-22**	**PG 76-22**	
APA @ 8,000 loading cycles, mm (AASHTO T 340)	t ≤ 7 7 > t > 10 t ≥ 10	t ≤ 4 4 > t > 7 t ≥ 7	t ≤ 7 7 > t > 10 t ≥ 10	t ≤ 4 4 > t > 7 t ≥ 7	0 – 1 – 5
Overlay Tester, cycles (NJDOT B-10)	t ≥ 150 150 > t > 100 t ≤ 100	t ≥ 175 175 > t > 125 t ≤ 125	t ≥ 100 100 > t > 75 t ≤ 75	t ≥ 125 125 > t > 90 t ≤ 90	0 – 1 – 5

Laboratory Characterization of a NJDOT High RAP (HRAP) Project I295

Submitted to:

New Jersey Department of Transportation (NJDOT)
Bureau of Materials

Conducted by:

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ABSTRACT

In August 2012, the New Jersey Department of Transportation (NJDOT) implemented a High Recycled Asphalt Pavement (HRAP) specification that allows asphalt suppliers/contractors to increase the allowable percentage of RAP in asphalt pavements. Under current NJDOT specifications, the maximum allowable RAP in the surface and intermediate/base course was 15% and 25%, respectively. However, under the HRAP specification, the asphalt suppliers/contractors are encouraged to use a minimum of 20% RAP in the surface and 30% RAP in the intermediate/base. In order to do so, the resultant mixtures must pass a set of rutting and fatigue cracking laboratory performance testing to ensure the mixtures will perform up to the NJDOT expectations. Table 1 shows the testing required during mixture design and plant production with the full specification found in Appendix A of the report.

Table 1 – Laboratory Performance Requirements for NJDOT HRAP Specification

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

This report summarizes the laboratory testing requirements of the NJDOT High Recycled Asphalt Pavement (HRAP) specification, as well as other laboratory characterization testing to provide an overall assessment of the mixtures' performance.

MATERIALS

The HRAP mixtures were designed and produced by RE Pierson. The mixture design summaries for a 9.5M76 HRAP and 12.5M64 HRAP mixtures can be found in Appendix B of the report. The 9.5M76 HRAP was placed as the surface course mixture and contained 25% RAP. The 12.5M64 HRAP was placed as the intermediate course mixture and contained 35% RAP. The laydown contractor for the project was Arawak Paving Company.

Asphalt binder for the mixtures was provided by NuStar Asphalt Refining in Paulsboro, NJ. The binder properties for each of the mixtures used is shown in Table 2. The 64 HRAP-1 asphalt binder was used in the 12.5M64 HRAP mixture, while the 76 HRAP-1 asphalt binder was used in the 9.5M76 HRAP mixture. In summary, the resultant true grade and PG of the asphalt binders were:

- 64 HRAP-1: 64.8-28.29 (PG64-28)
- 76 HRAP-1: 74.6-26.99 (PG70-22)

Table 2 – Asphalt Binder Properties Used in HRAP Mixes and Produced by NuStar Asphalt (Paulsboro, NJ)

Sample ID	2012-PHIL-002466-001	2012-PHIL-002466-002
Date	6-20-2012	6-18-2012
Sample Location	RE Pierson	RE Pierson
Sample Designated As	64 HRAP-1	76 HRAP-1
	Original Binder	
COC Flash, °F	260	263
SG @ 77°F	1.014	1.026
SG @ 60°F	1.020	1.032
API Gravity	7.2	5.7
LBS/GAL	8.497	8.590
Viscosity @ 135°C	398	1025
Viscosity @ 165°C	109	265
Test Temperature	64	70
Phase angle (DELTA)	85.4	73.7
G*/Sin Delta	1.10	1.79
Test Temperature	70	76
Phase angle (DELTA)	86.8	75.8
G*/Sin Delta	0.54	0.98
Fail Temp	**64.8**	**75.8**
	RTFO Aged Binder	
Test Temperature	64	70
Phase angle (DELTA)	82	70.4
G*/Sin Delta	2.44	3.54
Test Temperature	70	76
Phase angle (DELTA)	84	72.8
G*/Sin Delta	1.16	1.90
Fail Temp	**64.8**	**74.6**
Mass Change, %	-0.341	-0.286
	PAV Aged Binder	
Test Temperature	19	25
Phase angle (DELTA)	49.3	50.8
G*Sin Delta	4850	2870
Test Temperature	16	19
Phase angle (DELTA)	46.3	45.6
G*Sin Delta	7340	6490
Fail Temp	**18.8**	**21**
Test Temperature	-18	-12
Creep Stiffness @ S60	290	152
M-VALUE @ S60	0.332	0.374
Test Temperature	-24	-18
Creep Stiffness @ S60	584	344
M-VALUE @ S60	0.267	0.308
Fail Temp	**-18.3**	**-17**
PG Classification	**64-28**	**70-22**
True Grade	**64.8-28.29**	**74.6-26.99**
Degree Stretch	**93.09**	**101.59**

RAP Properties

Prior to the production of the material, RE Pierson provided the NJDOT with samples of the fractionated (coarse and fine) recycled asphalt pavement (RAP) proposed to be utilized in the HRAP mixtures. The NJDOT conducted extraction/recovery on coarse and fine RAP, and then followed with performance grading (AASHTO M320) of the recovered asphalt binder and washed/mechanical gradations (AASHTO T30) on the remaining aggregates. The test results for the RAP are shown in Tables 3 and 4.

Table 3 – RAP Asphalt Binder Performance Grade (Source: Fractionated Fine RAP)

Abson Recovery on R. E. Pierson Fine RAP					
	Temperature	G*/sin(delta)	Delta	Results	Fail Temperature
Viscosity (Brookfield)	135C			1825	
DSR RTFO	64 C	28.68	73.32		
	70 C	12.77	76.57		
	76 C	5.81	79.53		
	82 C	2.709	82.06		83.27 C
DSR PAV Original	25 C	5065	45.13		
	28 C	3590	47.65		25.11 C
*DSR PAV after PAV	25 C	7473	38.8		
	28 C	5576	41.06		29.12 C
Bending Beam	Temperature	Stiffness	m-value		
BBR Original	(-6) C	106	0.374		
	(-12) C	247	0.309		
	(-18) C	555	0.254		(-18) C
*BBR after PAV	(-6) C	155	0.324		
	(-12) C	335	0.272		(-12) C
NOTE: * After Recovery put 50 Grams in Pan and sent through PAV for 20 Hours then tested.					
Final Results: Original DSR passed at (82 C) and PAV at (22 C), BBR at (-12 C) making it a (82-22)					
For PAV: DSR PAV passed at (31 C), and BBR at (-6 C) making it a (82-16)					

As shown in Table 3, when treating the RAP binder as a "virgin" asphalt binder, the Performance Grade of the recovered RAP asphalt binder indicates that it is a PG82-16.

The aggregate gradation and asphalt binder properties of both the Coarse and Fine Fractionated RAP stockpiles are shown in Table 4. The results indicate that the fine fraction contained approximately 7% asphalt binder, while the coarse fraction contained approximately 3.7% asphalt binder (on average)

Table 4 – Gradation and Asphalt Content Properties of RE Pierson HRAP Mixtures
Recycled Asphalt Pavement (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	N0. 8	%	72.7	74.7	25.1	27.8
1.18	N0. 16	%	58.7	59.3	22.3	24.2
0.600	N0. 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

ASPHALT MIXTURE PERFORMANCE TESTING

NJDOT HRAP Requirements

In accordance with the NJDOT HRAP mixture specifications, the mixtures must meet rutting and fatigue cracking performance criteria using AASHTO T340 (Asphalt Pavement Analyzer) and the Overlay Tester (NJDOT B-10), respectively. The performance requirements of the mixtures were shown earlier in Table 1. Based on Table 1, the following performance requirements were imposed on the mixtures:

- 12.5M64 HRAP (Intermediate Course):
  - APA Rutting < 7.0 mm;
  - Overlay Tester Fatigue > 100 cycles
- 9.5M76 HRAP (Surface Course):
  - APA Rutting < 4.0 mm;
  - Overlay Tester Fatigue > 175 cycles

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

Compacted asphalt mixtures were evaluated for their 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 test results for the APA testing are shown as Figures 1 and 2. The test results show that both the surface and intermediate met the APA rutting requirements of the NJDOT HRAP specification.

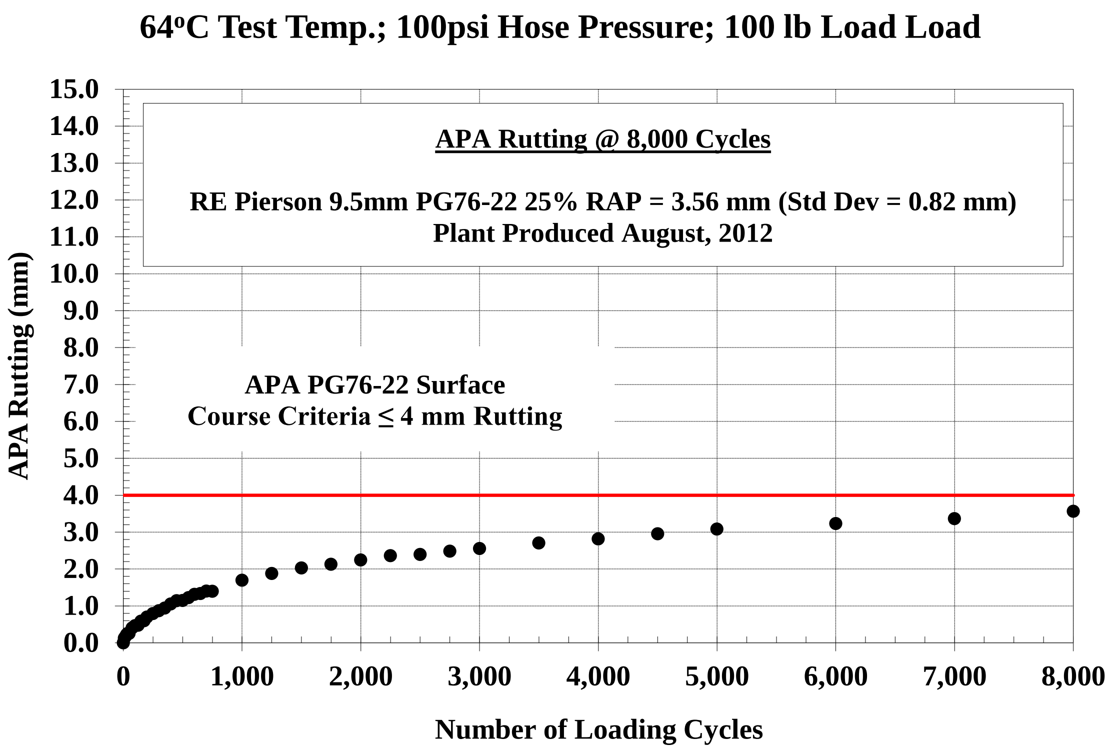


Figure 1 – Asphalt Pavement Analyzer (APA) Test Results for I295 HRAP Project – 9.5M76 Surface Course

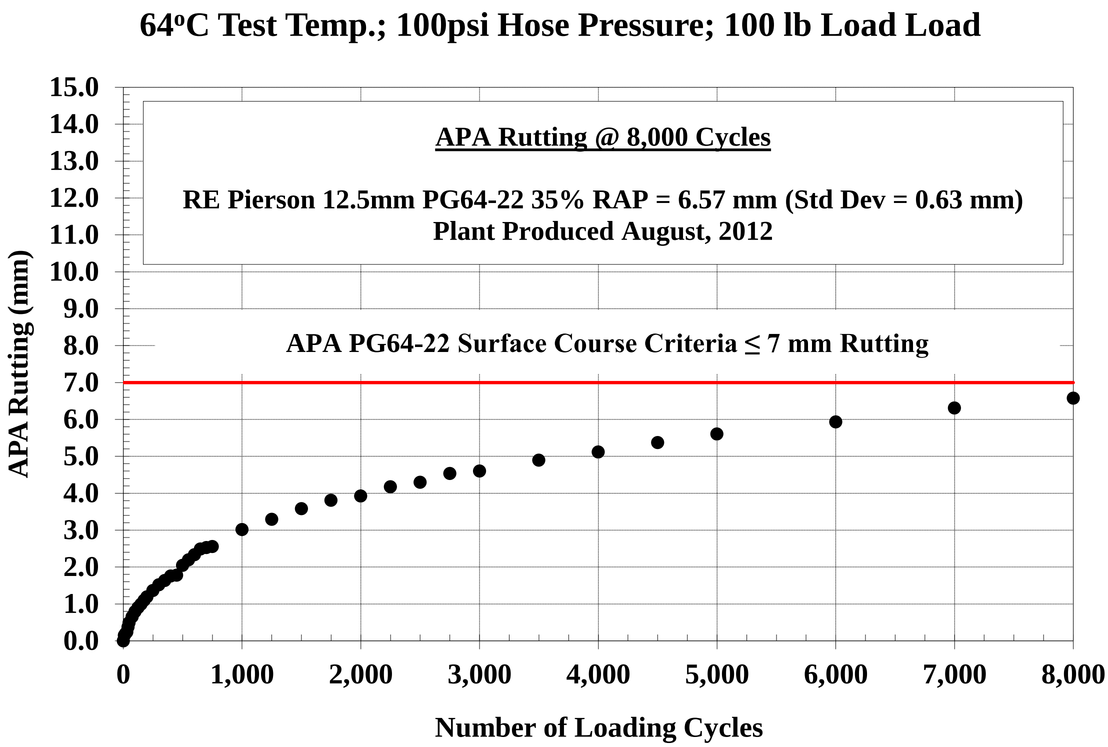


Figure 2 – Asphalt Pavement Analyzer (APA) Test Results for I295 HRAP Project – 12.5M64 Intermediate Course

Overlay Tester (NJDOT B-10) – Fatigue Cracking Evaluation

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). Sample preparation and test parameters used in this study followed that of TxDOT Tex-248-F testing specifications. These include:

- 25°C (77°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 shows the test results of the Overlay Tester fatigue evaluation. The results show both mixtures far exceeded the minimum requirements in the NJDOT HRAP specification shown earlier in Table 1. The 12.5M64 HRAP mixture resulted in 409 cycles to failure while the 9.5M76 HRAP resulted in 1,691 cycles to failure.

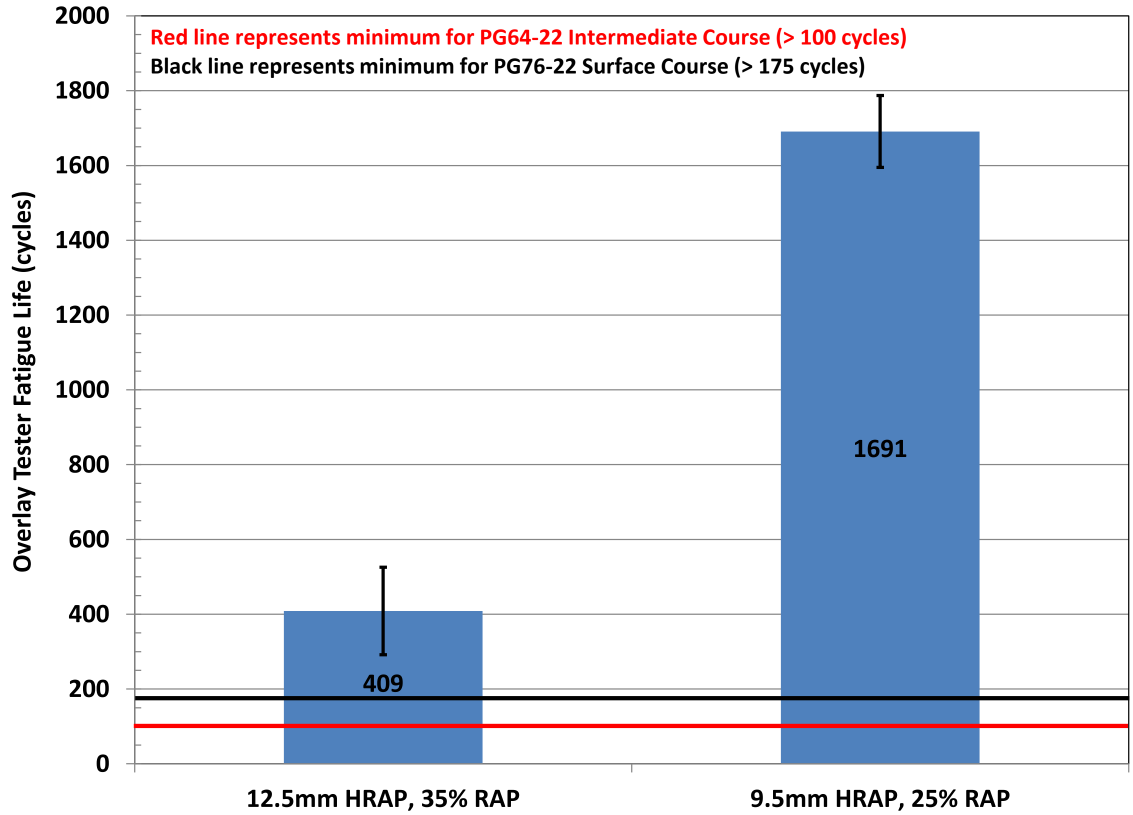


Figure 3 – Overlay Tester Results for I295 HRAP Project

Additional Mixture Testing

Additional laboratory testing was conducted to further evaluate the mixtures properties of the HRAP asphalt mixtures. In particular;

- Mixture Stiffness – AMPT Dynamic Modulus (AASHTO TP79)
- Rutting Susceptibility – AMPT Repeated Flow (AASHTO TP79)
- Moisture Damage Potential
  - Tensile Strength Ratio, TSR (AASHTO T283)
  - Hamburg Wheel Tracking (AASHTO T324)

<u>Dynamic Modulus (E*) – Mixture Stiffness</u>

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^{o}C using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz.

Figure 4 – 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^{o}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)

The resultant Master stiffness curves of the HRAP mixtures placed on I295 are shown in Figure 5. The stiffness curves show that the mixtures are relatively close to one another with respect to mixture stiffness. However, there are some differences, especially at the lower loading frequency (i.e. – higher test temperature). This would indicate that the 9.5M76 is stiffer at higher temperatures than the 12.5M64.

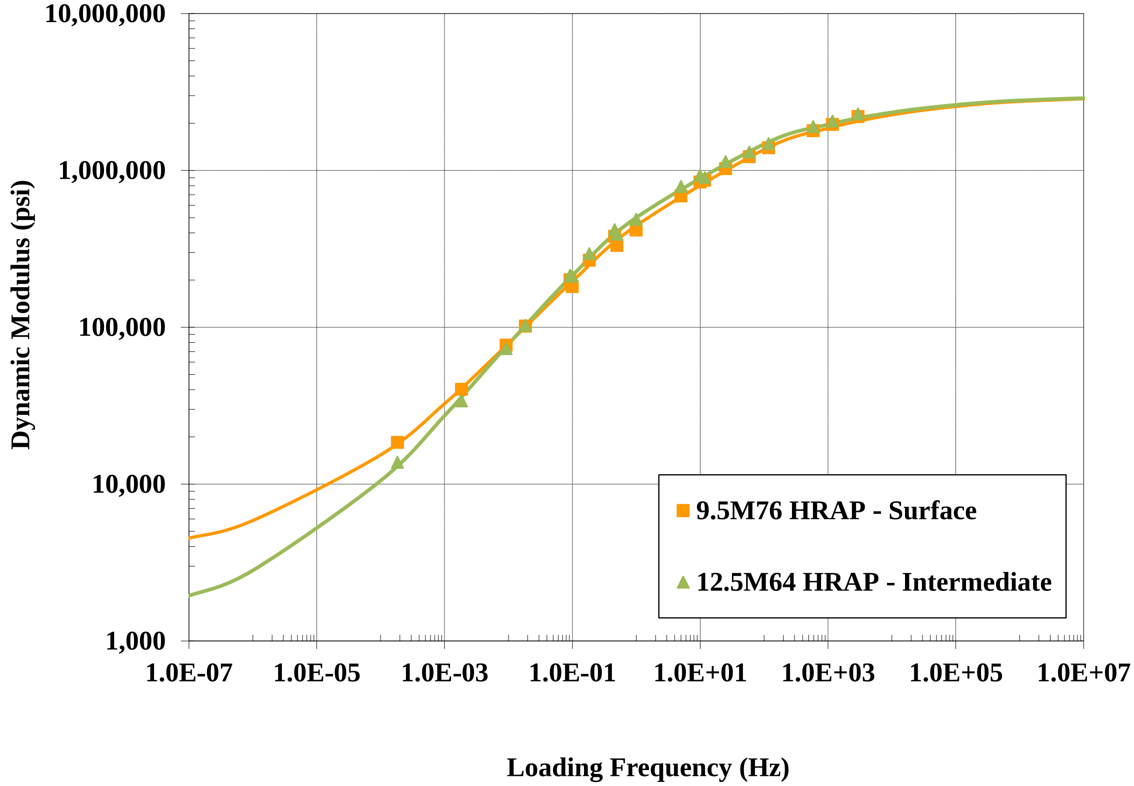


Figure 5 – Master Stiffness Curves of 9.5M76 and 12.5M64 HRAP Mixtures

<u>Repeated Load – Flow Number Test</u>

Repeated Load permanent deformation testing was measured and collected in uniaxial compression using the Asphalt Mixture Performance Tester (AMPT) 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. Testing was conducted until a permanent vertical strain of 5% or 10,000 cycles was obtained.

The test results from the Flow Number test show the same trend as both the APA tests and the high temperature stiffness measured during the Dynamic Modulus test, where the 9.5M76 HRAP mixture resulted in a greater resistance to rutting (permanent deformation) than the 12.5M64 HRAP mixture.

Table 5 – Repeated Load (Flow Number) Testing Summary

AMPT Flow Number (AASHTO TP79) 54^{o}C, 600 kPa Deviatoric Stress (NCHRP 9-33 Specs)			
Mix Type	**Sample ID**	**Flow Number (cycles)**	**Cycles to Achieve 5% Strain**
RE Pierson 9.5mm HRAP	#1	413	1,112
	#2	409	1,137
	Average	***411***	***1,125***
RE Pierson 12.5mm HRAP	#1	182	545
	#2	150	419
	Average	***166***	***482***

Under NCHRP Project 9-33, tentative criteria were established that recommended minimum Flow Number values for minimum ESAL levels. Table 6 contains those tentative recommendations. Based on the proposed criteria from the NCHRP research, the 9.5M76 HRAP mixture would be rated for pavements of 10 to 30 million ESAL's traffic levels. Meanwhile, the 12.5M64 HRAP mixture would be rated for 3 to 10 million ESAL's.

Table 6 – 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

<u>Resistance to Moisture-Induced Damage</u>

The resistance to moisture damage was evaluated using both the tensile strength ratio (TSR) test procedure and the wet Hamburg Wheel Tracking Test (AASHTO T324). The test procedures and results are discussed below.

Tensile Strength Ratio, TSR (AASHTO T283)

Tensile strengths of dry and conditioned asphalt samples were measured in accordance with AASHTO T283, *Resistance of Compacted Asphalt Mixtures to Moisture Induced Damage*. The TSR values and IDT strengths are shown in Table 7. The test results show that both mixtures resulted in passing TSR values (i.e. – greater than 80%) with the

12.5M64 HRAP achieving a slightly higher TSR value than the 9.5M76 HRAP. It should also be noted that further review of the tensile strengths show that the measured tensile strengths for the 9.5M76 HRAP mixture were actually higher than the 12.5M64 HRAP, even though the average TSR values were lower.

Table 7 – Tensile Strength Ratio (TSR) Values for HMA and WMA

9.5M76 HRAP			
Specimen Type	Indirect Tensile Strength (psi)		Average TSR (%)
	Dry	Conditioned	
AASHTO T283 Conditioned	161.8	142.6	***91.4%***
	158.4	143.7	
	151.2	144.4	
	157.1	***143.6***	

12.5M64 HRAP			
Specimen Type	Indirect Tensile Strength		Average TSR (%)
	Dry	Conditioned	
AASHTO T283 Conditioned	115.5	109.4	***94.8%***
	116.0	110.1	
	115.7	***109.8***	

Wet Hamburg Wheel Track Test (AASHTO T324)

Hamburg Wheel Track tests were conducted in accordance with AASHTO T324, *Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA)*. Test specimens were tested at a test temperature (water) of 50^{o}C. For comparison purposes, the number of cycles to reach 0.5 inches (12.5 mm) of rutting is commonly used for comparison purposes and for some state agency pass/fail specifications. Although the NJDOT does not have a specification for the Hamburg test, the criteria for the Texas Department of Transportation is often utilized. For a PG64-22 asphalt binder, the mixtures must achieve a minimum of 10,000 cycles before achieving 0.5 inches of rutting. Meanwhile, for a PG76-22 asphalt binder, the mixture must achieve a minimum of 20.000 cycles before achieving 0.5 inches of rutting.

The test results for the Hamburg Wheel Tracking are shown in Figures 6 and 7. The results indicate that the 9.5M76 HRAP mixture resulted in 11,422 cycles before achieving 0.5 inches of rutting. Meanwhile, the 12.5H64 HRAP mixture achieved 7,652 cycles.

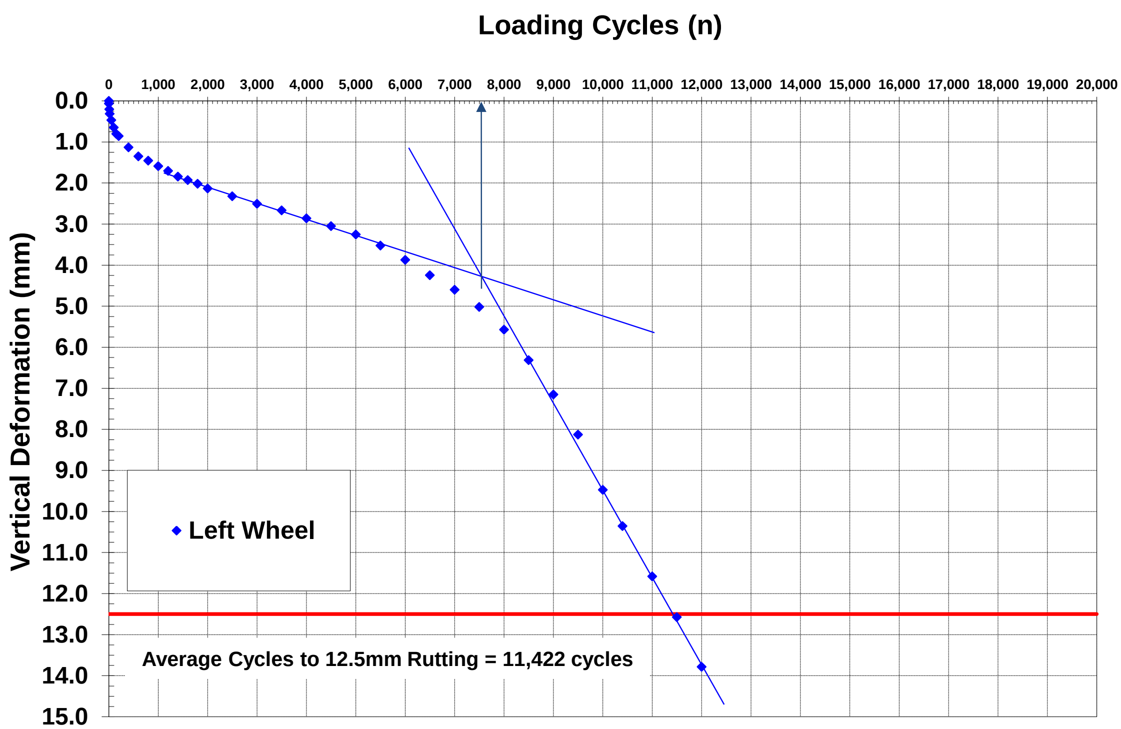


Figure 6 – Hamburg Wheel Tracking Results for 9.5M76 HRAP

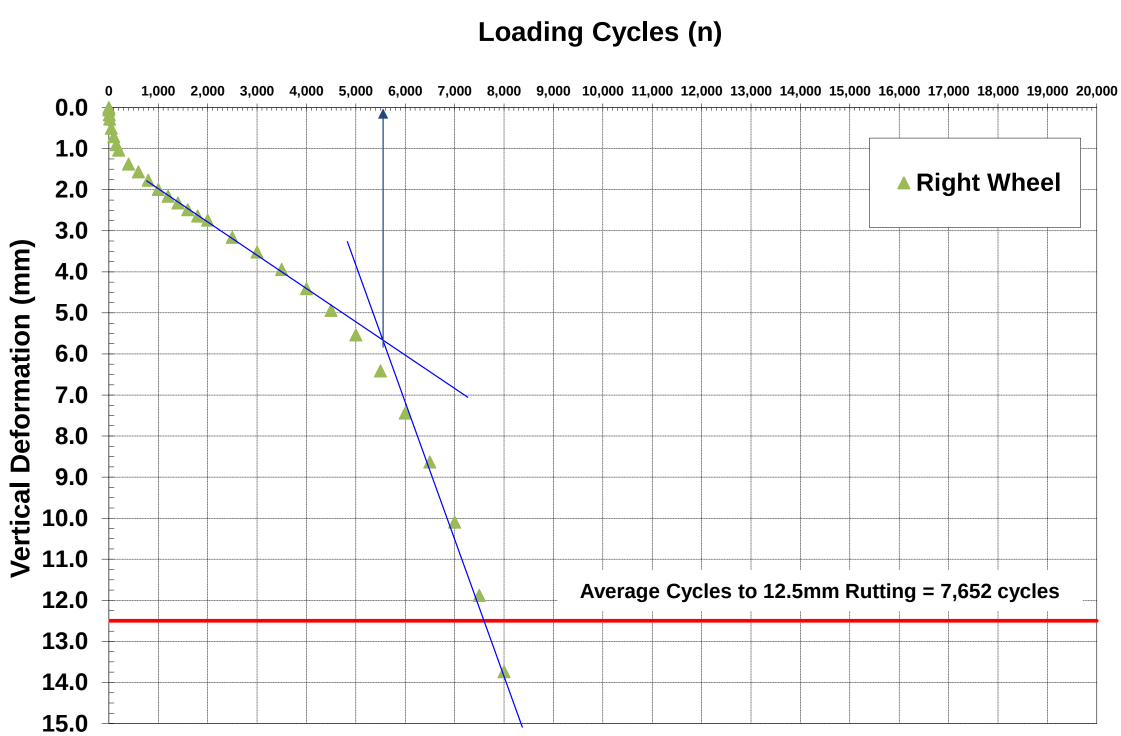


Figure 7 – Hamburg Wheel Tracking Results for 12.5M64 HRAP

FIELD CORE DENSITY

Field cores were taken by the NJDOT to determine the compacted air voids of the 9.5M76 HRAP and 12.5M64 HRAP mixtures. As with all of NJDOT asphalt pavements, field cores are used to determine pay adjustments for the construction of the asphalt pavement. The results of the field cores are shown in Tables 8 and 9.

Table 8 contains the compacted density information for the 9.5M76 HRAP mixture. The test results show some inconsistencies with the maximum specific gravity (Gmm) for both Lots, which may have attributed to higher compacted air voids and the resultant pay disincentives for both of the 9.5M76 Lots.

Table 8 – Compacted Density of 9.5M76 HRAP Field Cores

9.5M76 HRAP (25% RAP)					
Lot #1			Lot #2		
Core ID	G_{mm}	Air Voids (%)	Core ID	G_{mm}	Air Voids (%)
#1	2.530	4.4	#1	2.543	5.0
#2	2.560	8.6	#2	2.554	6.3
#3	2.549	8.6	#3	2.553	8.0
#4	2.563	8.0	#4	2.562	6.0
#5	2.531	7.2	#5	2.534	4.0
Average	*2.547*	*7.4*	*Average*	*2.549*	*5.9*
Range	*0.033*	*4.2*	*Range*	*0.028*	*4.0*

Table 9 contains the compacted density information for the 12.5M64 HRAP field cores. The compacted densities were much more consistent for the 12.5M64 HRAP field cores than the 9.5M76 HRAP field cores shown earlier. It should be noted that all three Lots of the 12.5M64 HRAP resulted in pay incentives with Lots #1 and #2 receiving a full bonus.

Table 9 – Compacted Density of 12.5M64 HRAP Field Cores

12.5M64 HRAP (35% RAP)					
Lot #1			Lot #2		
Core ID	G_{mm}	Air Voids (%)	Core ID	G_{mm}	Air Voids (%)
#1	2.561	4.7	#1	2.569	4.7
#2		4.1	#2		6.3
#3		3.9	#3		6.0
#4		4.6	#4		5.6
#5		5.8	#5		6.1
Average	*N.A.*	*4.6*	*Average*	*N.A.*	*5.7*
Range	*N.A.*	*1.9*	*Range*	*N.A.*	*1.6*
Lot #3					
Core ID	G_{mm}	Air Voids (%)			
#1	2.576	7.3			
#2		5.7			
#3		5.5			
#4		7.4			
#5		6.6			
Average	*N.A.*	*6.5*			
Range	*N.A.*	*1.9*			

APPENDIX A – NJDOT HRAP Specification

SECTION 401 – HOT MIX ASPHALT (HMA) COURSES

ADD THE FOLLOWING TO 401.01:

401.01 DESCRIPTION

This Section also describes the requirements for constructing a Hot Mix Asphalt (HMA) course with required minimum amounts of Reclaimed Asphalt Pavement (RAP).

ADD THE FOLLOWING TO 401.02.01:

401.02.01 Materials

Hot Mix Asphalt HIGH RAP 902.11

ADD THE FOLLOWING SUBSECTION TO 401.03:

401.03.07 Hot Mix Asphalt (HMA) HIGH RAP

A. **Paving Plan.** At least 20 days before beginning placing the HMA HIGH RAP, submit a detailed plan of operation as specified in 401.03.03.A to the RE for approval. Include in the paving plan a proposed location for the test strip. Submit for Department approval a plan of the location for the HMA HIGH RAP on the project.

B. **Weather Limitations.** Place HMA HIGH RAP according to the weather limitations in 401.03.03.B.

C. **Test Strip.** Construct a test strip as specified in 401.03.03.C.

D. **Transportation and Delivery of HMA.** Deliver HMA HIGH RAP as specified in 401.03.03.D.

E. **Spreading and Grading.** Spread and grade HMA HIGH RAP as specified in 401.03.03.E. Record the laydown temperature (temperature immediately behind the paver) at least once per hour during paving. Submit the temperatures to the RE and to the HMA Plant producing the HMA HIGH RAP.

F. **Compacting.** Compact HMA HIGH RAP as specified in 401.03.03.F.

G. **Opening to Traffic.** Follow the requirements of 401.03.03.G for opening HMA HIGH RAP to traffic.

H. **Air Void Requirements.** Ensure that the HMA HIGH RAP is compacted to meet the air void requirements as specified in 401.03.03.H.

I. **Thickness Requirements.** Ensure that the HMA HIGH RAP is paved to meet the thickness requirements as specified in 401.03.03.I.

J. **Ride Quality Requirements.** Ensure that the HMA HIGH RAP is paved to meet the ride quality requirements as specified in 401.03.03.J

ADD THE FOLLOWING TO 401.04:

401.04 MEASUREMENT AND PAYMENT

The Department will measure and make payment for Items as follows:

Item	*Pay Unit*
HOT MIX ASPHALT ___ ___ ___ SURFACE COURSE HIGH RAP	TON
HOT MIX ASPHALT ___ ___ ___ INTERMEDIATE COURSE HIGH RAP	TON
HOT MIX ASPHALT ___ ___ ___ BASE COURSE HIGH RAP	TON

ADD THE FOLLOWING TO 902:

902.11 HOT MIX ASPHALT RAP

902.11.01 Mix Designations

The requirements for specific HMA mixtures with required minimum amounts of RAP are identified by the abbreviated fields in the Item description as defined as follows:

HOT MIX ASPHALT 12.5H64 SURFACE COURSE HIGH RAP

1. “HOT MIX ASPHALT” “Hot Mix Asphalt” is located in the first field in the Item description for the purpose of identifying the mixture requirements.
2. “12.5” The second field in the Item description designates the nominal maximum size aggregate (in millimeters) for the job mix formula (sizes are 4.75, 9.5, 12.5, 19, 25, and 37.5 mm).
3. “H” The third field in the Item description designates the design compaction level for the job mix formula based on traffic forecasts as listed in Table 902.02.03-2 (levels are L=low, M=medium, and H=high).
4. “64” The fourth field in the Item description normally designates the high temperature (in °C) of the performance-graded binder (options are 64, 70, and 76 °C). In the High RAP mixes this field will designate the mix performance requirements.
5. “SURFACE COURSE” The last field in the Item description designates the intended use and location within the pavement structure (options are surface, intermediate, or base course).
6. “HIGH RAP” This additional field designates that there will be a minimum percentage of RAP required for the mixture in 902.011.02.

902.11.02 Composition of Mixture

Provide materials as specified:

Aggregates for Hot Mix Asphalt 901.05

Use a virgin asphalt binder that will result in a mix that meets the performance requirements specified in Table 902.11.03-2. Ensure that the virgin asphalt binder meets the requirements of 902.01.01 except the performance grade. Use a performance grade of asphalt binder as determined by the mix design and mix performance testing.

Mix HMA HIGH RAP in a plant that is listed on the QPL for HMA Plants and conforms to the requirements for HMA Plants as specified in 1009.01.

Composition of the mixture for HMA HIGH RAP surface course is coarse aggregate, fine aggregate, asphalt binder, and a minimum of 20 percent Reclaimed Asphalt Pavement (RAP), and may also include mineral filler, asphalt rejuvenator and Warm Mix Asphalt (WMA) additives or processes as specified in 902.01.05. When WMA is used it must meet the requirements as specified in 902.10. Ensure that the finished mix does not contain more than a total of 1 percent by weight contamination from Crushed Recycled Container Glass (CRCG).

The composition of the mixture for HMA HIGH RAP base or intermediate course is coarse aggregate, fine aggregate, asphalt binder, and a minimum of 30 percent Reclaimed Asphalt Pavement (RAP), and may also include mineral filler, up to 10 percent of additional recycled materials, asphalt rejuvenator, and Warm Mix Asphalt (WMA) additives or processes as specified in 902.01.05. When WMA is used it must meet the requirements as specified in 902.10. The recycled materials may consist of a combination of RAP, CRCG, Ground Bituminous Shingle Material (GBSM), and RPCSA, with the following individual limits:

Table 902.11.02-1 Use of Recycled Materials in Base or Intermediate Course

ycled Material	Minimum Percen	Maximum Percen
P		
CG		
SM		

Combine the aggregates to ensure that the resulting mixture meets the grading requirements specified in Table 902.02.03-1. In determining the percentage of aggregates of the various sizes necessary to meet gradation requirements, exclude the asphalt binder.

Ensure that the combined coarse aggregate, when tested according to ASTM D 4791, has less than 10 percent flat and elongated pieces retained on the No. 4 sieve and larger. Measure aggregate using the ratio of 5:1, comparing the length (longest dimension) to the thickness (smallest dimension) of the aggregate particles.

Ensure that the combined fine aggregate in the mixture conforms to the requirements specified in Table 902.02.02-2. Ensure that the material passing the No. 40 sieve is non-plastic when tested according to AASHTO T 90.

902.11.03 Mix Design

At least 45 days before initial production, submit a job mix formula for the HMA HIGH RAP on forms supplied by the Department, to include a statement naming the source of each component and a report showing that the results meet the criteria specified in Tables 902.02.03-1 and 902.11.03-1.

Include in the mix design the following based on the weight of the total mixture:

1. Percentage of RAP or GBSM.
2. Percentage of asphalt binder in the RAP or GBSM.
3. Percentage of new asphalt binder.
4. Total percentage of asphalt binder.
5. Percentage of each type of virgin aggregate.

Table 902.11.03-1 HMA HIGH RAP Requirements for Design

ompaction Leve	Required Density % of Theoretical Max. Specifi Gravity)		Voids in Mineral Aggregate (VMA)[2], % (minimum) Nominal Max. Aggregate Size, mm					'oids Filled Wit ısphalt (VFA) %	Dust-to-Binder Ratio
	@N_{des}[1]	@N_{max}	25.0	19.0	12.5	9.5	4.75		
L	96.0	≤ 98.0	13.0	14.0	15.0	16.0	17.0	70 - 85	0.6 - 1.2
M	96.0	≤ 98.0	13.0	14.0	15.0	16.0	17.0	65 - 85	0.6 - 1.2

etermined from the values for the maximum specific gravity of the mix and the bulk specific gravity of the compacted mixture. Maxir ific gravity of the mix is determined according to AASHTO T 209. Bulk specific gravity of the compacted mixture is determ rding to AASHTO T 166. For verification, specimens must be between 95.0 and 97.0 percent of maximum specific gravity at N_{des}.
calculation of VMA, use bulk specific gravity of the combined aggregate including aggregate extracted from the RAP.

The job mix formula for the HMA HIGH RAP mixture establishes the percentage of dry weight of aggregate, including the aggregate from the RAP, passing each required sieve size and an optimum percentage of asphalt binder based upon the weight of the total mix. Determine the optimum percentage of asphalt binder according to AASHTO R 35 and M 323 with an N_{des} as required in Table 902.02.03-2. Before maximum specific gravity testing or compaction of specimens, condition the mix for 2 hours according to the requirements for conditioning for volumetric mix design in AASHTO R 30, Section 7.1. If the absorption of the combined aggregate is more than 1.5 percent according to AASHTO T 84 and T 85, ensure that the mix is short term conditioned for 4 hours according to AASHTO R 30, Section 7.2 prior to compaction of specimens (AASHTO T 312) and determination of maximum specific gravity (AASHTO T 209). Ensure that the job mix formula is within the master range specified in Table 902.02.03-1.

Ensure that the job mix formula provides a mixture that meets a minimum tensile strength ratio (TSR) of 80% when prepared according to AASHTO T 312 and tested according to AASHTO T 283. Submit the TSR results with the mix design.

Determine the correction factor of the mix including the RAP by using extracted aggregate from the RAP in the proposed proportions when testing is done to determine the correction factor as specified in AASHTO T 308. Use extracted aggregate from the RAP in determining the bulk specific gravity of the aggregate blend for the mix design.

For each mix design, submit with the mix design forms 3 gyratory specimens and 1 loose sample corresponding to the composition of the JMF. Ensure that the samples include the percentage of RAP that is being proposed for the mix. The ME will use these to verify the properties of the JMF. Compact the specimens to the design number of gyrations (N_{des}). For the mix design to be acceptable, all gyratory specimens must comply with the requirements specified in Tables 902.02.03-1 and 902.11.03-1. The ME reserves the right to be present at the time the gyratory specimens are molded.

In addition, submit nine gyratory specimens and five 5-gallon buckets of loose mix to the ME. The ME will use these additional samples for performance testing of the HMA HIGH RAP mix. The ME reserves the right to be present at the time of molding the gyratory specimens. Ensure that the additional gyratory specimens are compacted according to AASHTO T 312, are 77 mm high, and have an air void content of 6.5 ± 0.5 percent. The ME will test six (6) specimens using an Asphalt Pavement Analyzer (APA) according to AASHTO T 340 at 64°C, 100 psi hose pressure, and 100 lb. wheel load. The ME will use the remaining three (3) specimens to test using an Overlay Tester (NJDOT B-10) at 25°C and a joint opening of 0.025 inch.

The ME will approve the JMF if the results meet the criteria in Table 902.11.03-2.

Table 902.11.03-2 Performance Testing Requirements for HMA HIGH RAP Design

Test	Requirement			
	Surface Course		Intermediate Course	
	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

If the JMF does not meet the APA and Overlay Tester criteria, redesign the HMA HIGH RAP mix and submit for retesting. The JMF for the HMA HIGH RAP mixture is in effect until modification is approved by the ME.

When unsatisfactory results for any specified characteristic of the work make it necessary, the Contractor may establish a new JMF for approval. In such instances, if corrective action is not taken, the ME may require an appropriate adjustment to the JMF.

Should a change in sources be made or any changes in the properties of materials occur, the ME will require that a new JMF be established and approved before production can continue.

902.11.04 Sampling and Testing

A. General Acceptance Requirements. The RE or ME may reject and require disposal of any batch or shipment that is rendered unfit for its intended use due to contamination, segregation, improper temperature, lumps of cold material, or incomplete coating of the aggregate. For other than improper temperature, visual inspection of the material by the RE or ME is considered sufficient grounds for such rejection.

Ensure that the temperature of the mix at discharge from the plant or storage silo meets the recommendation of the supplier of the asphalt binder, supplier of the asphalt modifier and WMA manufacturer. For HMA, do not allow the mixture temperature to exceed 330°F at discharge from the plant. For WMA, do not allow the mixture temperature to exceed 300°F at discharge from the plant.

Combine and mix the aggregates and asphalt binder to ensure that at least 95 percent of the coarse aggregate particles are entirely coated with asphalt binder as determined according to AASHTO T 195. If the ME determines that there is an on-going problem with coating, the ME may obtain random samples from 5 trucks and will determine the adequacy of the mixing on the average of particle counts made on these 5 test portions. If the requirement for 95 percent coating is not met on each sample, modify plant operations, as necessary, to obtain the required degree of coating.

B. **Sampling.** The ME will take 5 stratified random samples of HMA HIGH RAP for volumetric acceptance testing from each lot of approximately 3500 tons of a mix. When a lot of HMA HIGH RAP is less than 3500 tons, the ME will take samples at random for each mix at the rate of one sample for each 700 tons. The ME will perform sampling according to AASHTO T 168, NJDOT B-2, or ASTM D 3665.

Use a portion of the samples taken for volumetric acceptance testing for composition testing.

C. **Quality Control Testing.** The HMA HIGH RAP producer shall provide a quality control (QC) technician who is certified by the Society of Asphalt Technologists of New Jersey as an Asphalt Technologist, Level 2. The QC technician may substitute equivalent technician certification by the Mid-Atlantic Region Technician Certification Program (MARTCP). Ensure that the QC technician is present during periods of mix production for the sole purpose of quality control testing and to assist the ME. The ME will not perform the quality control testing or other routine test functions in the absence of, or instead of, the QC technician.

The QC technician shall perform sampling and testing according to the approved quality control plan, to keep the mix within the limits specified for the mix being produced. The QC technician may use acceptance test results or perform additional testing as necessary to control the mix.

To determine the composition, perform ignition oven testing according to AASHTO T 308.

For each acceptance test, perform maximum specific gravity testing according to AASHTO T 209 on a test portion of the sample taken by the ME. Sample and test coarse aggregate, fine aggregate, mineral filler, and RAP according to the approved quality control plan for the plant.

Ensure that the supplier has in operation an ongoing daily quality control program to evaluate the RAP. As a minimum, this program shall consist of the following:

1. An evaluation performed to ensure that the material conforms to 901.05.04 and compares favorably with the design submittal.
2. An evaluation of the RAP material performed using a solvent or an ignition oven to qualitatively evaluate the aggregate components to determine conformance to 901.05.
3. Quality control reports as directed by the ME.

D. **Acceptance Testing and Requirements.** The ME will determine volumetric properties at N_{des} for acceptance from samples taken, compacted, and tested at the HMA plant. The ME will compact HMA HIGH RAP to the number of design gyrations (N_{des}) specified in Table 902.02.03-2, using equipment according to AASHTO T 312. The ME will determine bulk specific gravity of the compacted sample according to AASHTO T 166. The ME will use the most current QC maximum specific gravity test result in calculating the volumetric properties of the HMA HIGH RAP.

The ME will determine the dust-to-binder ratio from the composition results as tested by the QC technician.

Ensure that the HMA HIGH RAP mixture conforms to the requirements specified in Table 902.11.04-1, and to the gradation requirements in Table 902.02.03-1. If 2 samples in a lot fail to conform to the gradation or volumetric requirements, immediately initiate corrective action.

The ME will test a minimum of 1 sample per lot for moisture, basing moisture determinations on the weight loss of an approximately 1600-gram sample of mixture heated for 1 hour in an oven at 280 ± 5°F. Ensure that the moisture content of the mixture at discharge from the plant does not exceed 1.0 percent.

Table 902.11.04-1 HMA HIGH RAP Requirements for Control

ompaction Leve	Required Density (% of Theoretical Max. Specific Gravity)	Voids in Mineral Aggregate (VMA), % (minimum)					
		Nominal Max. Aggregate Size, mm					
	@Ndes[1]	25.0	19.0	12.5	9.5	4.75	st-to-Binder Ra
L, M	95.0 – 98.5	13.0	14.0	15.0	16.0	17.0	0.6 - 1.3

letermined from the values for the maximum specific gravity of the mix and the bulk specific gravity of the compacted mix imum specific gravity of the mix is determined according to AASHTO T 209. Bulk specific gravity of the compacted mixtu :mined according to AASHTO T 166.

E. Performance Testing for HMA HIGH RAP. Provide five (5) 5-gallon buckets of loose mix to the ME for testing in the Asphalt Pavement Analyzer (APA) and the Overlay Tester device. Ensure that the first sample is taken during the construction of the test strip as specified in 401.03.07.C. Thereafter, sample every lot or as directed by the ME. If a sample does not meet the design criteria for performance testing as specified in Table 902.11.03-2, the Department will assess a pay adjustment as specified in Table 902.11.04-2. If a lot fails to meet requirements for both APA and Overlay Tester, the Department will assess pay adjustments for both parameters. The Department will calculate the pay adjustment by multiplying the percent pay adjustment (PPA) by the quantity in the lot and the bid price for the HMA High RAP item.

Table 902.11.04-2 Performance Testing Pay Adjustments for HMA HIGH RAP

	Surface Course		**Intermediate Course**		
	PG 64-22	**PG 76-22**	**PG 64-22**	**PG 76-22**	**PPA**
APA @ 8,000 loadin cycles, mm (AASHTO T 340)	$t \leq 7$ $7 > t > 10$ $t \geq 10$	$t \leq 4$ $4 > t > 7$ $t \geq 7$	$t \leq 7$ $7 > t > 10$ $t \geq 10$	$t \leq 4$ $4 > t > 7$ $t \geq 7$	0 – 1 – 5
Overlay Tester, cycle (NJDOT B-10)	$t \geq 150$ $150 > t > 100$ $t \leq 100$	$t \geq 175$ $175 > t > 125$ $t \leq 125$	$t \geq 100$ $100 > t > 75$ $t \leq 75$	$t \geq 125$ $125 > t > 90$ $t \leq 90$	0 – 1 – 5

APPENDIX B – HRAP MIXTURE DESIGN SUMMARY SHEETS

Mixture Design Summary – 9.5M76 HRAP

New Jersey Department of Transportation

Producers Analysis of Materials and Job Mix Formula

1 COVER
page 1 of 18

Producer - Location	R. E. Pierson Materials Corp.	Date	2/27/2012	Producer Information	
Plant TYPE	DRUM	Nom Sieve Size(mm)	9.5	Prepared By	Dan Karcher
Asphalt Grade	PG 64-22	Design EASL's (millions)	.3 < 3	Submitted By	Dan Karcher
Compact. Temp C or F	293--302	Mix Designation*	HMA 9.5M64-25R	Note* Surface course mixes prohibit the use of glass and limit RAP to 15 %.	
Mixing Temp C or F	315 - 325	Depth From Surface, mm	50		
Gyrations N Initial	7	NJDOT SERIAL Number		Signature**	
Gyrations N Des	75	Producer Mix ID		Title	QC Manager
Gyrations N Max	115	Plant Local ID		Date	

Bin or Feed No.	VIR	10% RAP	15% RAP	20% RAP	25% RAP	Components- Producer & Location
6 Fine Rap	23.5					Fine RAP - REP Bridgeport, NJ
5	43.5					# 8 - Hanson Crushed Stone Glen Mills, PA
4	8.2					Rice - Hanson Crushed Stone Penns Park, PA
2	0.9					Sand - REP Bridgeport, NJ
1	17.9					# 10 - Martin Stone Quarries Bechtelsville, PA
Virgin AC	6.00					PG 64-22 Nustar Paulsboro, NJ
Totals	100.0	0.0	0.0	0.0	0.0	* See Standard Pay Item List Of Most Common Superpave Mixes
AC From RAP						

NJDOT Review &Approval

Name

Title

Signature

Date

Tests Performed	Test Results	Min	Max	Test Specification
Sand Equivalence	85.3	45	na	D2419
% Flat and Elongated Particles	2	na	10	D4791
% Fine Aggregate Angularity	46.5	45	na	TP-33
% Coarse Agg. Angularity fr. faces	100	75/-	na	TP56
% Air Voids (Va)	4.0	na	na	T-166/T-209
% VMA	17.5	16	na	MP-2
% VFA	77	65	85	MP-2
Dust/Asphalt Ratio	0.9	0.6	1.2	MP-2
Max. Specific Gravity (Gmm)	2.529	na	na	T-168
Bulk Specific Gravity (Gmb)	2.428	na	na	T-209
% Gmm @ Nini	88	na	89	PP-28
% Gmm @ Nmax	97	na	98	PP-28
Eff. Sp. Gravity of Blend (Gse)	2.788	na	na	LB-251 b
Sp. Gravity of Binder (Gb)	1.030	na	na	T-228
S.G. of Agg. Blend (Gsb)	2.765	na	na	T84/T85
Moisture Sensitivity TSR	82%	80 %	na	T-283
Megagrams/(mm * m^2)	2.428	na	na	

Nominal Maximum Aggregate Size = 9.5

JMF	Sieve Size mm	The Zone Min	The Zone Max	Control Points Min	Control Points Max	Average of 5 Control Tolerances 2.36mm Tol	0.075 Tol
100.0	50.0			100	100	4	1.4
100.0	37.5			100	100		
100.0	25.0			100	100		
100.0	19.0			100	100		
100.0	12.5			100	100		
95.0	9.5			90	100		
64.0	4.75			0	90		
40.0	2.36	47.2	47.2	32	67	36.0	44.0
28.4	1.18	31.6	37.6	0	100		
19.8	0.600	23.5	27.5	0	100		
12.3	0.300	18.7	18.7	0	100		
7.4	0.150			0	100		
5.2	0.075			2	10	3.8	6.6
6.0	Percent AC						

Ig. Oven Agg Corr. Factor	0.58
Percent Absorbed AC	0.29

V-2001a

Cold Feed Percentages

Aggr. Size							
Percentages							

** Producer signature certifies that this mix design has been prepared in accordance with NJDOT Specifications; that all calculations have been independantly verified, and the final mix design is an accurate reflection of the test data herein.

Mixture Design Summary – 12.5M64 HRAP

New Jersey Department of Transportation

Producers Analysis of Materials and Job Mix Formula

Producer - Location	R. E. Pierson Materials Corp.	Date	2/27/2012	Producer Information	
Plant TYPE	DRUM	Nom Sieve Size(mm)	12.5	Prepared By	Dan Karcher
Asphalt Grade	PG 64-22	Design EASL's (millions)	.3 < 3	Submitted By	Dan Karcher
Compact. Temp C or F	293--302	Mix Designation*	HMA 12.5M64-35R	Note* Surface course mixes prohibit the use of glass and limit RAP to 15 %.	
Mixing Temp C or F	315--325	Depth From Surface, mm	75		
Gyrations N Initial	7	NJDOT SERIAL Number		Signature**	
Gyrations N Des	75	Producer Mix ID		Title	QC Manager
Gyrations N Max	115	Plant Local ID		Date	

Bin or Feed No.	VIR	10% RAP	15% RAP	20% RAP	25% RAP	Components- Producer & Location
6 Fine RAP	16.5					Fine Rap REP - Bridgeport, NJ
5 Medium RAP	16.5					Medium Rap REP - Bridgeport, NJ
4	10.4					# 67 - Hanson Crushed Stone Glen Mills, PA
3	25.0					#8 - Hanson Crushed Stone Glen Mills, PA
2	11.8					Rice - Hanson Penns Park, PA
1	14.1					# 10 - Martin Stone Quarries Bechtelville, PA
Virgin AC	5.80					PG 64-22 Nustar Paulsboro, NJ
Totals	100.0	0.0	0.0	0.0	0.0	* See Standard Pay Item List Of Most Common Superpave Mixes
AC From RAP						

NJDOT Review &Approval

Name

Title

Signature

Date

Tests Performed	Test Results	Min	Max	Test Specification
Sand Equivalence	85.2	45	na	D2419
% Flat and Elongated Particles	2.5	na	10	D4791
% Fine Aggregate Angularity	46.7	45	na	TP-33
% Coarse Agg. Angularity fr. faces	100	75/-	na	TP56
% Air Voids (Va)	4.0	na	na	T-166/T-209
% VMA	16.8	15	na	MP-2
% VFA	76	65	85	MP-2
Dust/Asphalt Ratio	1.0	0.6	1.2	MP-2
Max. Specific Gravity (Gmm)	2.529	na	na	T-168
Bulk Specific Gravity (Gmb)	2.428	na	na	T-209
% Gmm @ Nini	87	na	89	PP-28
% Gmm @ Nmax	97	na	98	PP-28
Eff. Sp. Gravity of Blend (Gse)	2.778	na	na	LB-251 b
Sp. Gravity of Binder (Gb)	1.030	na	na	T-228
S.G. of Agg. Blend (Gsb)	2.749	na	na	T84/T85
Moisture Sensitivity TSR	82%	80 %	na	T-283
Megagrams/(mm * m^2)	2.428	na	na	

Nominal Maximum Aggregate Size = 12.5

JMF	Sieve Size mm	The Zone Min	The Zone Max	Control Points Min	Control Points Max	Average of 5 Control Tolerances 2.36mm Tol	0.075 Tol
100.0	50.0			100	100	4	1.4
100.0	37.5			100	100		
100.0	25.0			100	100		
100.0	19.0			100	100		
94.5	12.5			90	100		
86.7	9.5			0	90		
56.4	4.75			0	100		
34.8	2.36	39.1	39.1	28	58	30.8	38.8
24.9	1.18	25.6	31.6	0	100		
17.8	0.600	19.1	23.1	0	100		
11.5	0.300	15.5	15.5	0	100		
7.2	0.150			0	100		
5.1	0.075			2	10	3.7	6.5
5.8	Percent AC						

Ig. Oven Agg Corr. Factor	0.59
Percent Absorbed AC	0.36

V-2001a

Cold Feed Percentages

Aggr. Size								
Percentages								

** Producer signature certifies that this mix design has been prepared in accordance with NJDOT Specifications; that all calculations have been independantly verified, and the final mix design is an accurate reflection of the test data herein.

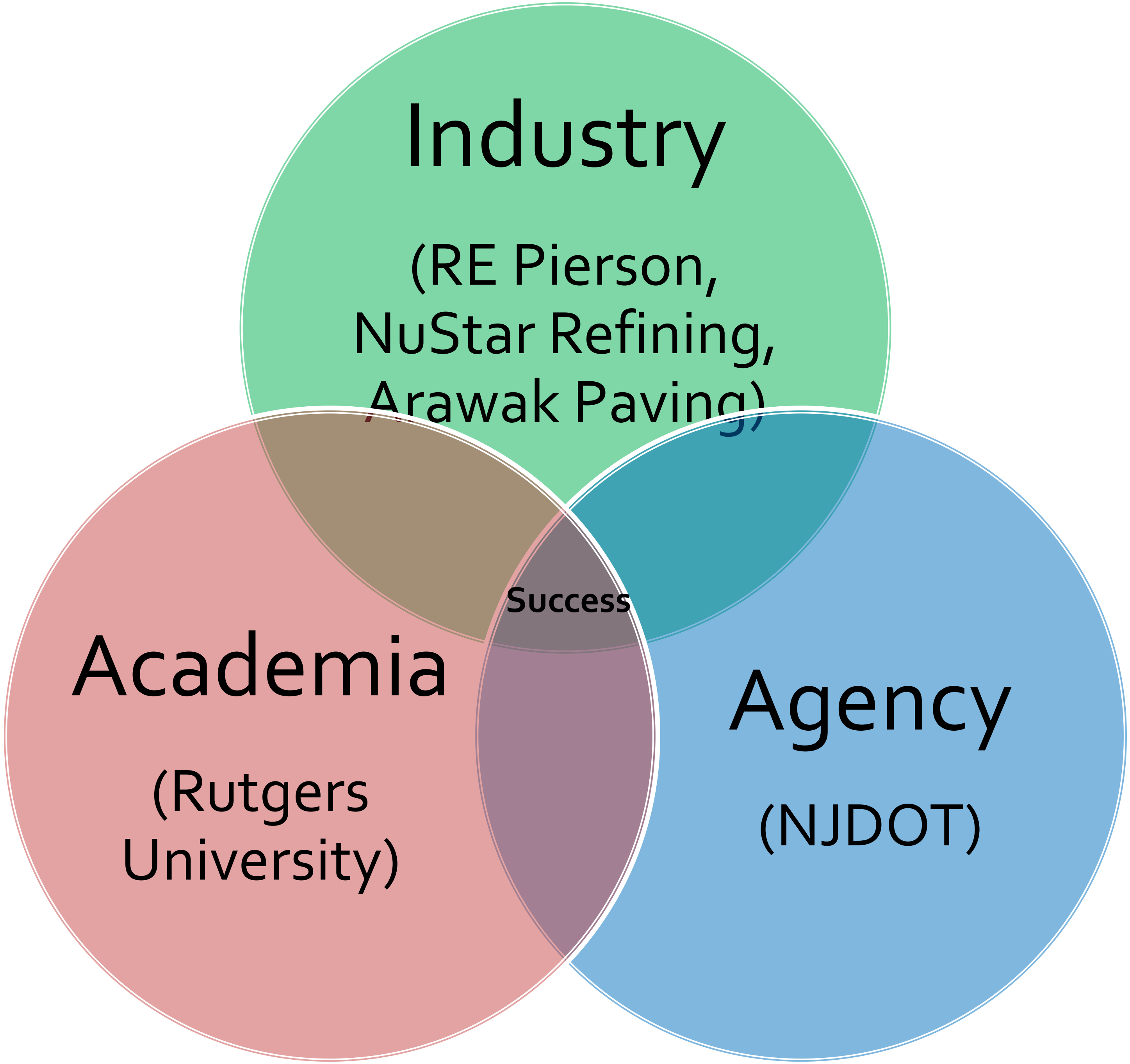
NJDOT High RAP Specification and Implementation – I295

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

Thomas Bennert, Ph.D.
Rutgers University

Acknowledgements

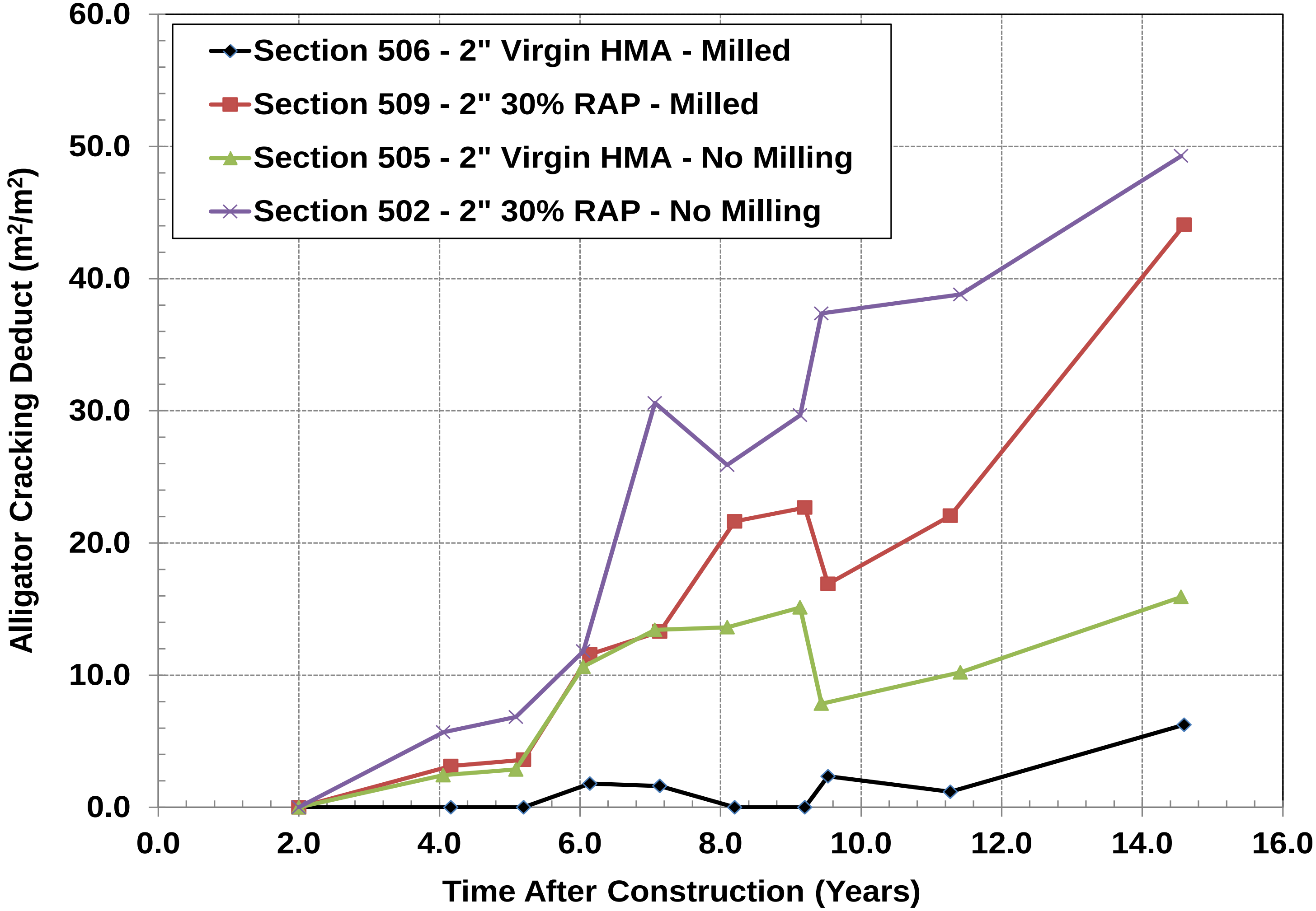
- Dan Karcher – R.E. Pierson
- Eileen Sheehy, Robert Blight, Don Matlock - NJDOT
- Frank Fee and Karissa Mooney – NuStar Asphalt


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

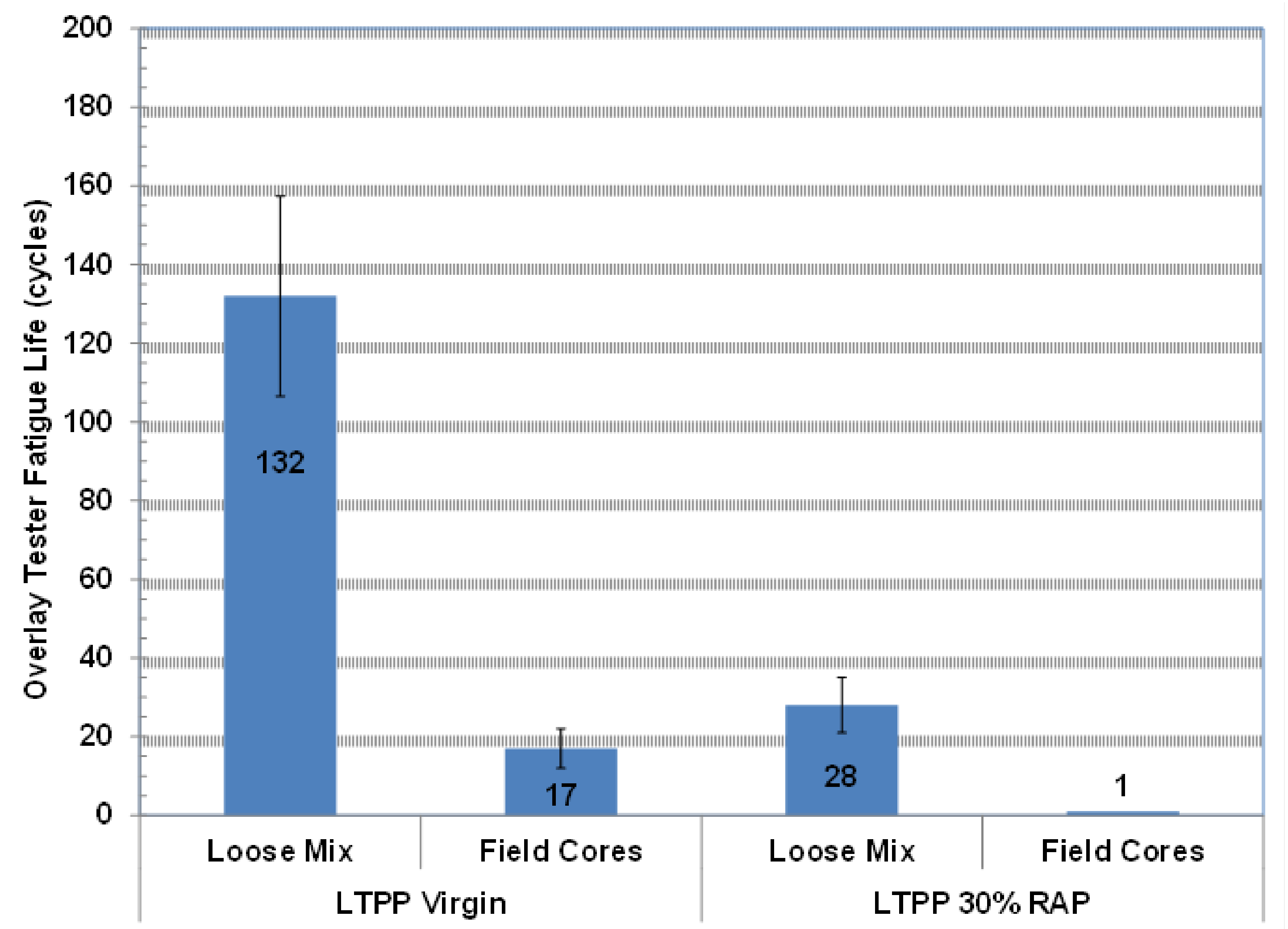
Background

- 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)

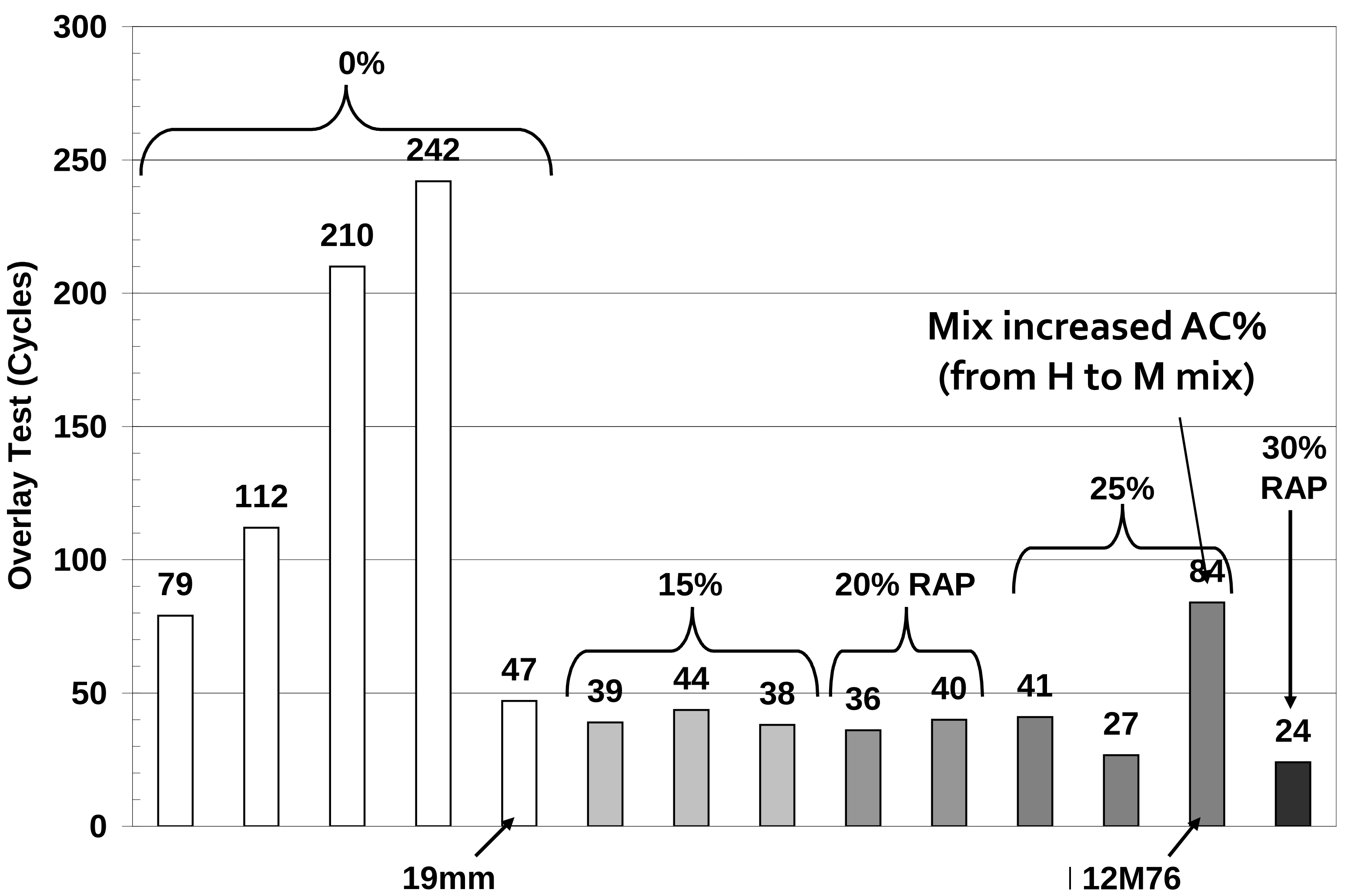
Initiation vs Propagation – NJ SPS-5



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

- <u>Minimum</u> of 20% RAP in Surface Course
- <u>Minimum</u> 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

Test	Requirement			
	Surface Course		Intermediate Course	
	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

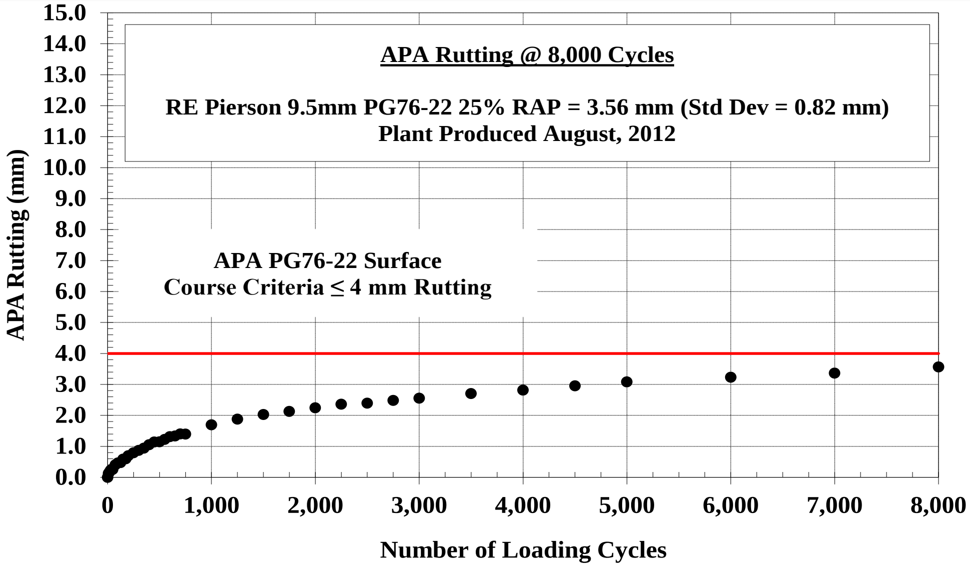
9.5M76 (SURFACE COURSE)

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

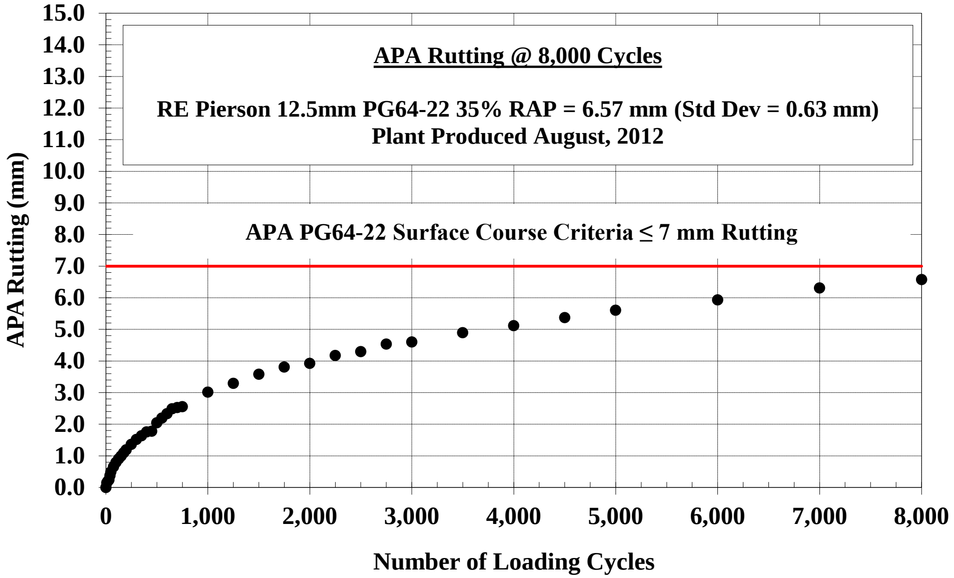
12.5M64 (INTERMED. COURSE)

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

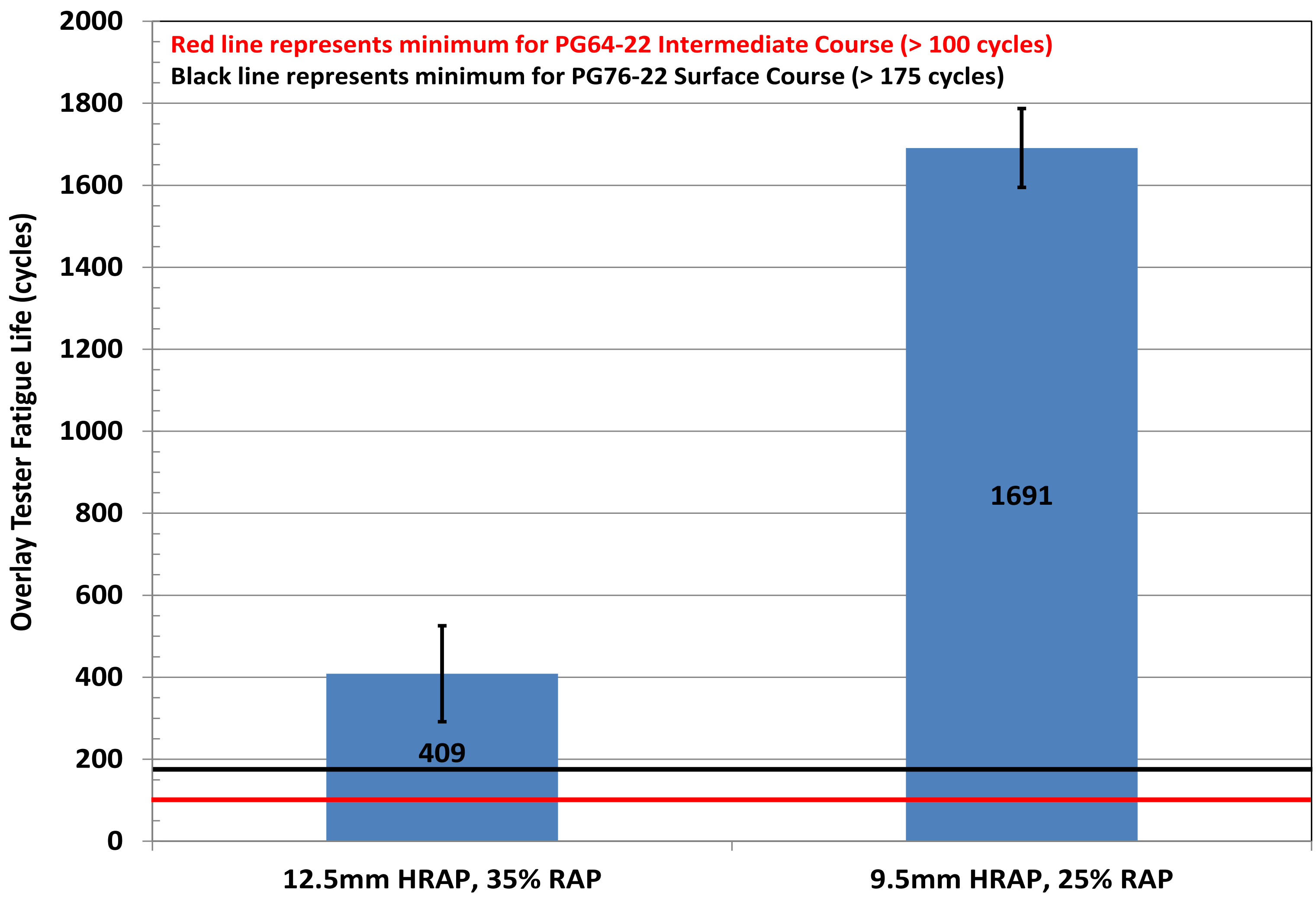
APA Rutting Performance



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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Laboratory Characterization of Laboratory Produced Warm Mix Asphalt

- RE Pierson 9.5M64 (15% RAP) -

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
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Abstract

In accordance with the NJDOT specification, the asphalt supplier must provide compacted HMA and WMA test specimens for mixture performance testing when wanting to utilize WMA on a NJDOT paving project. The testing matrix required is shown below in Table 1.

Table 1 – Test Procedure and Specimen Requirements for NJDOT WMA Implementation Projects

Performance Tests for HMA Control						
Type of Test	Test Method	Pavement Distress	Test Specimen Air Voids	Compacted Specimen Height (mm)	Number of Test Specimens	Test Temperature
AMPT E*	AASTHO TP 79	Rutting Susceptibility	6.5 ± 0.5 %	170 [1]	2	129ºF (54ºC)
Asphalt Pavement Analyzer (APA)	AASTHO TP 63	Rutting Susceptibility	6.5 ± 0.5 %	170	2	147ºF (64ºC)
Hamburg Wheel Tracking	AASTHO T 324	Moisture Damage	6.5 ± 0.5 %	170	2	122ºF (50ºC)
Tensile Strength Ratio (TSR)	AASTHO T 283	Moisture Damage	6.5 ± 0.5 %	95	4	77ºF (25ºC)
Overlay Tester	NJDOT B-10	Fatigue Cracking Potential	6.5 ± 0.5 %	170[3]	2	77ºF (25ºC)

[1] Final Cut and trimmed test specimens. Lab compacted specimens should be approximately 1.0% higher.
[2] Three specimens of 170 mm height may be used instead of the required 6 specimens of 77 mm height.
[3] Four specimens of 115 mm height may be used instead of the required 2 specimens at 170 mm height.

The 9.5M64 + 15% RAP warm mix asphalt was laboratory produced and intended for the Route 40 Resurfacing project DP#12117. The mixture was produced with 0.5% Evotherm. No additional information was provided regarding the mixing and compaction temperature of the WMA supplied for evaluation.

Plant Production Data

No plant production data was provided. It should also be noted that HMA was not provided for evaluation – only WMA.

Dynamic Modulus (E*) – Mixture Stiffness

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^{o}C using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz.

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^{o}C.

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

where:

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

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

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)

The resultant Master stiffness curves for the RE Pierson 9.5M64 WMA is shown in Figure 2.

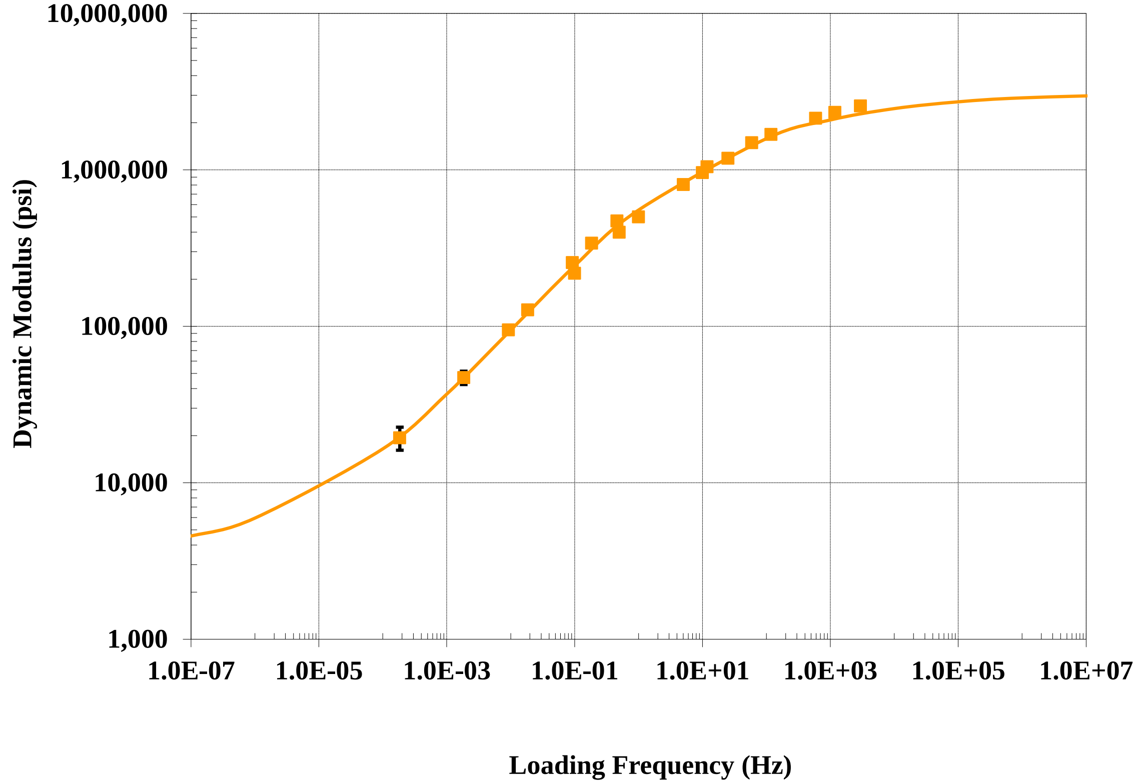

Figure 2 – Master Stiffness Curve of 9.5M64 WMA Produced by RE Pierson

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 testing were their rutting potential using the Asphalt Pavement Analyzer (APA) in accordance with AASHTO TP63, *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 test results for the APA testing are shown as Figure 3. The test results indicates that the WMA mixture achieved an APA rutting of 4.16mm, which would be below the maximum allowable rutting for a 64-22 asphalt binder mixture in the NJDOT High RAP specification.

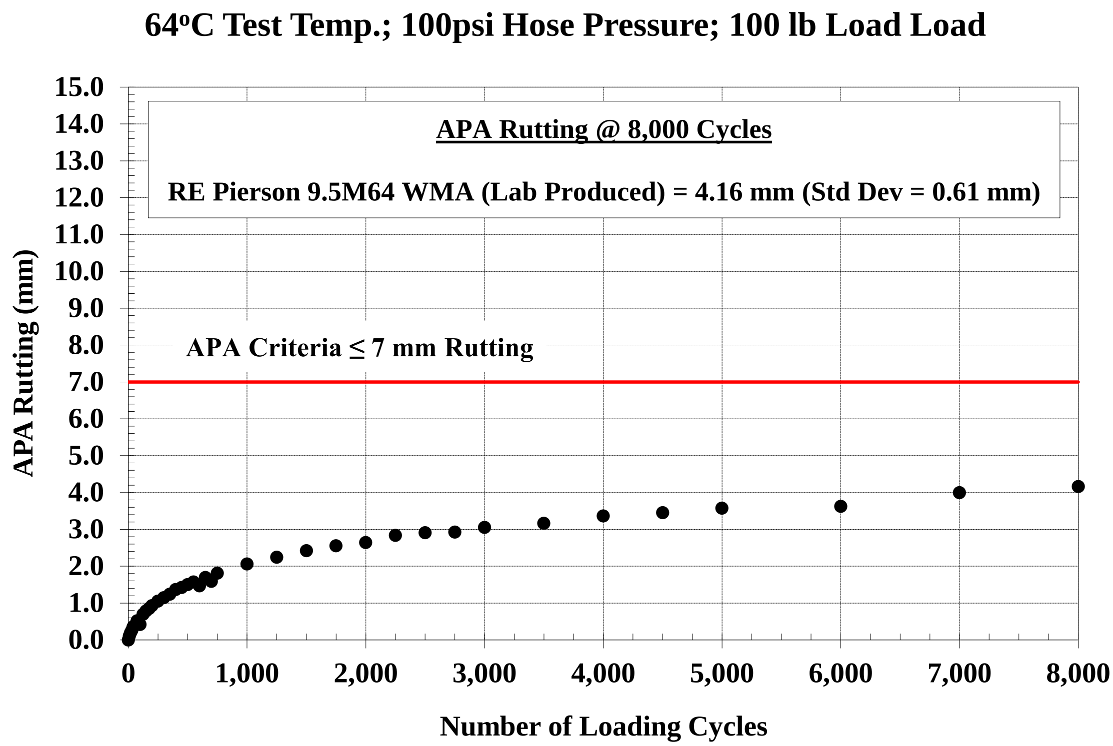

Figure 3 – Asphalt Pavement Analyzer (APA) Test Results for 9.5M64 WMA Produced by RE Pierson

Repeated Load – Flow Number Test

Repeated Load permanent deformation testing was measured and collected in uniaxial compression using the Asphalt Mixture Performance Tester (AMPT) 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^{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 9-43. Testing was conducted until a permanent vertical strain of 5% or 10,000 cycles was obtained. Table 2 contains the test results from the Flow Number testing.

Table 2 – Repeated Load (Flow Number) Testing Summary

AMPT Flow Number (AASHTO TP79) 54°C, 600 kPa Deviatoric Stress (NCHRP 9-43 Specs)			
Mix Type	**Sample ID**	**Flow Number (cycles)**	**Cycles to Achieve 5% Strain**
RE Pierson 9.5M64 WMA (0.5% Evotherm)	#1	426	1,203
	#2	326	905
	#3	257	742
	Average	***336***	***950***

Under NCHRP Projects 9-33 and 9-43, tentative criteria were established that recommended minimum Flow Number values for minimum ESAL levels. Tables 3 and 4 contain these values, respectively. Although the guidelines were developed for laboratory produced mixtures, the criterion does provide a means of assessing the general rutting performance of the mixture. Based on the proposed criteria from the NCHRP research, the WMA mixture would be appropriate for pavements of less than 30 million ESAL's.

Table 3 – 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

Table 4 – Recommended Flow Number vs ESAL Level for WMA

Traffic Level, Million ESAL's	Minimum Flow Number
<3	N.A.
3 to < 10	30
10 to < 30	105
≥ 30	415

Overlay Tester (TxDOT Tex-248-F) – Fatigue Cracking Evaluation

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). Sample preparation and test parameters used in this study followed that of TxDOT Tex-248-F testing specifications. These include:

- 25°C (77°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 are shown in Table 5.

Table 5 – Overlay Tester Results for RE Pierson 9.5M64 WMA Mix

Overlay Tester 25°C, 0.025 Inch Displacement (TxDOT TX-248F Specs)					
Mix Type	**Sample ID**	**Width (mm)**	**Height (mm)**	**Faitgue Life (cycles)**	**Average (cycles)**
RE Pierson 9.5M64 WMA (0.5% Evotherm)	#1	75.7	38.7	150	114
	#2	75.7	37.6	99	
	#3	75.8	37.8	119	
	#4	75.9	37.7	87	

Resistance to Moisture-Induced Damage

The resistance to moisture damage was evaluated using both the tensile strength ratio (TSR) test procedure and the wet Hamburg Wheel Tracking Test (AASHTO T324). The test procedures and results are discussed below.

<u>Tensile Strength Ratio, TSR (AASHTO T283)</u>

Tensile strengths of dry and conditioned asphalt samples were measured in accordance with AASHTO T283, *Resistance of Compacted Asphalt Mixtures to Moisture Induced Damage*. The TSR values and IDT strengths are shown in Table 6. The results show that the WMA mixture achieved the minimum 80% TSR value required by the NJDOT.

Table 6 – Tensile Strength Ratio (TSR) Values for RE Pierson 9.5M64 WMA

RE Pierson - 9.5M64 WMA (0.5% Evotherm)			
Specimen Type	Indirect Tensile Strength (psi)		Average TSR (%)
	Dry	Conditioned	
AASHTO T283 Conditioned	109.5	92.2	***84.2%***
	109.5	95.4	
	107.8	87.5	
	108.9	***91.7***	

Wet Hamburg Wheel Track Test (AASHTO T324)

Hamburg Wheel Track tests were conducted in accordance with AASHTO T324, *Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA).* Test specimens were tested at a test temperature (water) of 50°C. For comparison purposes, the number of cycles to reach 0.5 inches (12.5 mm) of rutting is commonly used for comparison purposes and for some state agency pass/fail specifications. For a PG64-22 asphalt binder, the mixtures must achieve a minimum of 10,000 cycles before achieving 0.5 inches (12.5 mm) of rutting.

The test results are shown in Figure 4. The test results show that the average number of cycles to reach 12.5mm rutting was 7,888 cycles with an average Stripping Inflection Point of 5,470 cycles.

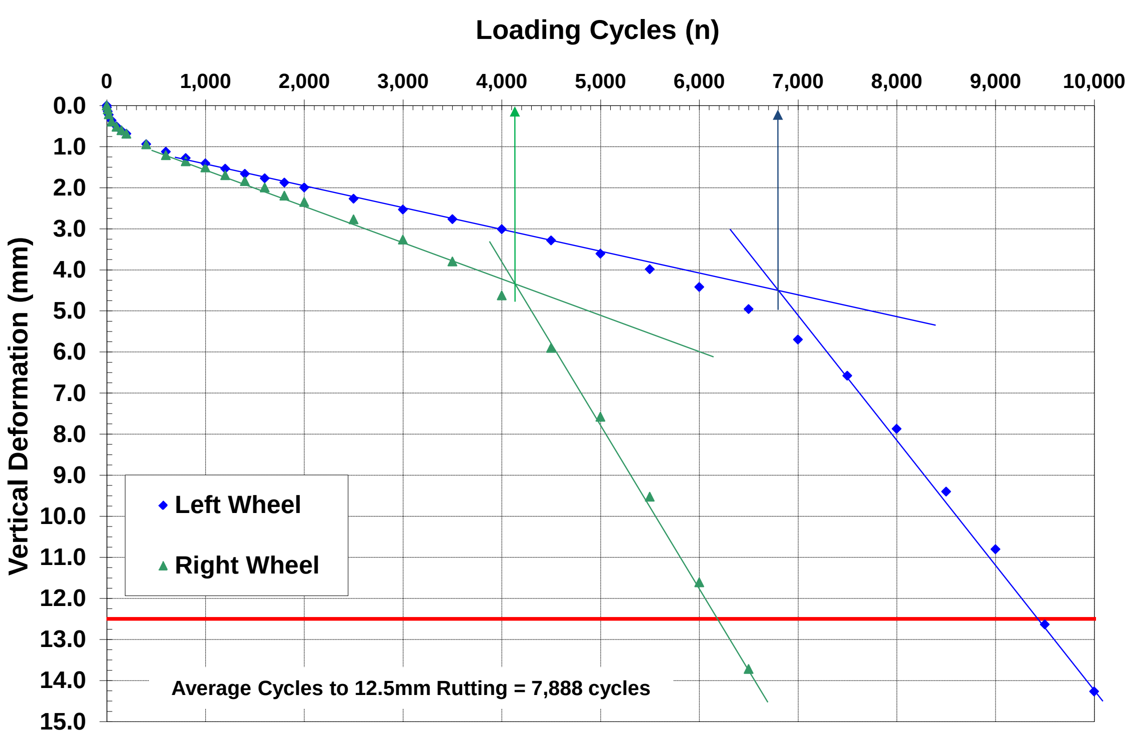

Figure 4 – Hamburg Wheel Tracking Results for RE Pierson 9.5M64 WMA

New Jersey Department of Transportation

Producers Analysis of Materials and Job Mix Formula

Producer - Location	R. E. Pierson Materials Corp.	Date	4/23/2012	Producer Information	
Plant TYPE	DRUM	Nom Sieve Size(mm)	9.5	Prepared By	Dan Karcher
Asphalt Grade	PG 64-22	Design EASL's (millions)	< 3	Submitted By	Dan Karcher
Compact. Temp C or F	243--252 F	Mix Designation*	WMA 9.5M64	Note* Surface course mixes prohibit the use of glass and limit RAP to 15 %.	
Mixing Temp C or F	265--275 F	Depth From Surface, mm	75		
Gyrations N Initial	7	NJDOT SERIAL Number		Signature**	
Gyrations N Des	75	Producer Mix ID		Title	QC Manager
Gyrations N Max	115	Plant Local ID		Date	

Bin or Feed No.	VIR	10% RAP	15% RAP	20% RAP	25% RAP	Components- Producer & Location
RAP			15.0			Fine RAP - R. E. Pierson Materials Corp.
4	45.3		48.0			# 8 - Hanson Crushed Stone Glen Mills, PA
3						
2	14.2		20.0			# 10 Martin Stone Quarries-Bechtelville,PA
1	34.9		17.0			# 10 - Martin Stone Quarries Bechtelsville, PA
Virgin AC	5.60		4.58			Nustar PG 64-22 .5% Evotherm
Totals	100.0	0.0	104.6	0.0	0.0	* See Standard Pay Item List Of Most Common Superpave Mixes
AC From RAP			1.02			

NJDOT Review &Approval

Name

Title

Signature

Date

Tests Performed	Test Results	Min	Max	Test Specification
Sand Equivalence	85.3	45	na	D2419
% Flat and Elongated Particles	2.2	na	10	D4791
% Fine Aggregate Angularity	48.9	45	na	TP-33
% Coarse Agg. Angularity fr. faces	100	95/90	na	TP56
% Air Voids (Va)	4.0	na	na	T-166/T-209
% VMA	16.0	15	na	MP-2
% VFA	75	65	78	MP-2
Dust/Asphalt Ratio	1.1	0.6	1.2	MP-2
Max. Specific Gravity (Gmm)	2.538	na	na	T-168
Bulk Specific Gravity (Gmb)	2.436	na	na	T-209
% Gmm @ Nini	86	na	89	PP-28
% Gmm @ Nmax	97	na	98	PP-28
Eff. Sp. Gravity of Blend (Gse)	2.779	na	na	LB-251 b
Sp. Gravity of Binder (Gb)	1.030	na	na	T-228
S.G. of Agg. Blend (Gsb)	2.739	na	na	T84/T85
Moisture Sensitivity TSR	82%	80 %	na	T-283
Megagrams/(mm * m^2)	2.436	na	na	

JMF	Nominal Maximum Aggregate Size = 9.5					Average of 5 Control Tolerances	
	Sieve Size mm	The Zone Min	The Zone Max	Control Points Min	Control Points Max	2.36mm Tol	0.075 Tol
100.0	50.0			100	100		
100.0	37.5			100	100	4	1.4
100.0	25.0			100	100		
100.0	19.0			100	100		
100.0	12.5			100	100		
94.6	9.5			90	100		
64.0	4.75			0	90		
40.7	2.36	47.2	47.2	32	67	36.7	44.7
28.4	1.18	31.6	37.6	0	100		
19.4	0.600	23.5	27.5	0	100		
12.8	0.300	18.7	18.7	0	100		
8.3	0.150			0	100		
5.8	0.075			2	10	4.4	7.2
5.6	Percent AC						

Ig. Oven Agg Corr. Factor	0.49
Percent Absorbed AC	0.51

V-2001a

Cold Feed Percentages

Aggr. Size								
Percentages								

** Producer signature certifies that this mix design has been prepared in accordance with NJDOT Specifications; that all calculations have been independantly verified, and the final mix design is an accurate reflection of the test data herein.

RUTGERS, THE STATE UNIVERSITY OF NEW JERSEY

CENTER FOR ADVANCED INFRASTRUCTURE AND TRANSPORTATION

NJ 71 Shoulder Survey

Ground Penetrating Radar Results

For Narinder Kohli

1/14/2011

NJ 71 Shoulder Survey

The Center for Advanced Infrastructure and Transportation (CAIT) at Rutgers, the State University of New Jersey was asked by the pavement design section of the New Jersey Department of Transportation (NJDOT) to perform some exploratory work on NJ 71 in Monmouth County from mile post 0.15 to mile post 5.45. NJDOT is considering roadway rehabilitation on this section of pavement but they currently do not have enough information about existing physical properties for the shoulder on this road.

Equipment description

On the morning of November 10, 2010 members of CAIT arrived on the north end of the job site to meet with traffic control staff. A 1 GHz air coupled horn antenna was used in conjunction with a SIR-20 control unit, and a collet distance measuring instrument (C-DMI) to perform the exploratory shoulder survey. The mounting equipment for the antenna is attached to a class 3 hitch on the back of the surveying van. To insure a minimal amount of bouncing of the antenna during data collection four straps are used to keep the antenna at a constant height as seen in Figure 1. Attached to the back driver side wheel is where the C-DMI is attached. Collets are placed over the lug nuts of the wheel and the mounting plate is centered using a centering star.

Figure 1: Survey Van Setup

Once the C-DMI and the 1GHz horn antenna are connected to the SIR-20 control unit, the system is ready to start warming up. It is recommended that the horn antenna continuously fires a pulse for twenty minutes until data collection can start. After the warm-up period elapsed a calibration file was taken to insure proper results. A traffic control vehicle followed behind the CAIT survey van starting at mile post 5.45 while traveling south to mile post 0.15. Once this was completed the same procedures were duplicated starting at mile post 0.15 traveling north to mile post 5.45.

Data results

Figures 2 and 3 show the results of the NJ 71 shoulder survey. In the graph the x-axis represents distance along the roadway in miles and the y-axis is a double scale. One scale is shoulder width in feet and the second scale is the calculated thickness of the surface layer in inches. When there was a shoulder width less than ten feet the antenna was not exclusively over the shoulder rendering the radar data not useful. When there was an observed shoulder width of ten feet, the horn antenna was able to be placed directly over the shoulder and a reliable depth was able to be collected. In Figures 2 and 3 the red lines represent the different shoulder widths, the blue dots represent the observed thickness of the surface layer, and the green line is the average thickness of the surface layer from mile post 1.80 to mile post 3.20. Any data collected with a shoulder width less than ten feet should not be included in this average because the antenna was either over the main roadway or the interface of the shoulder and the road.

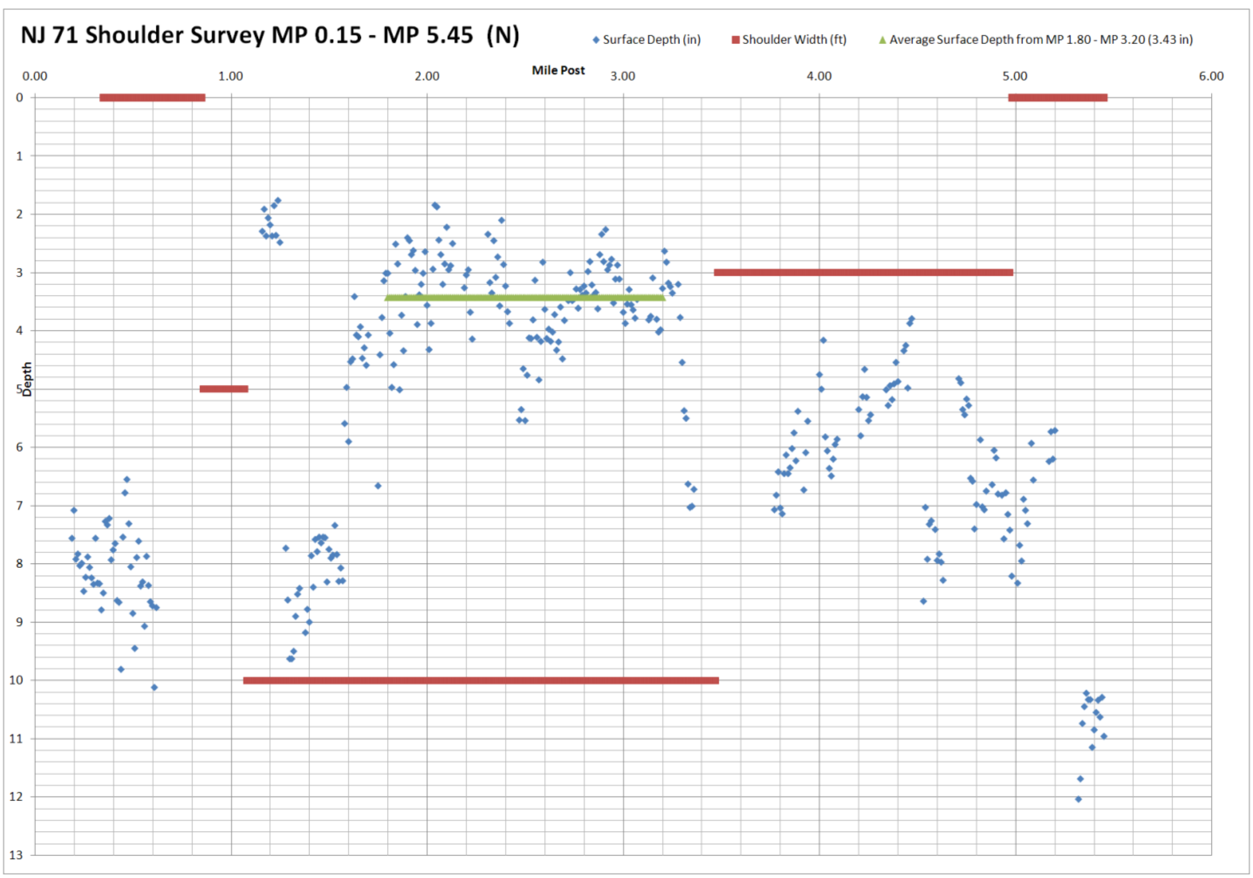

Figure 2: GPR Layer Thickness by Mile Post (NB)

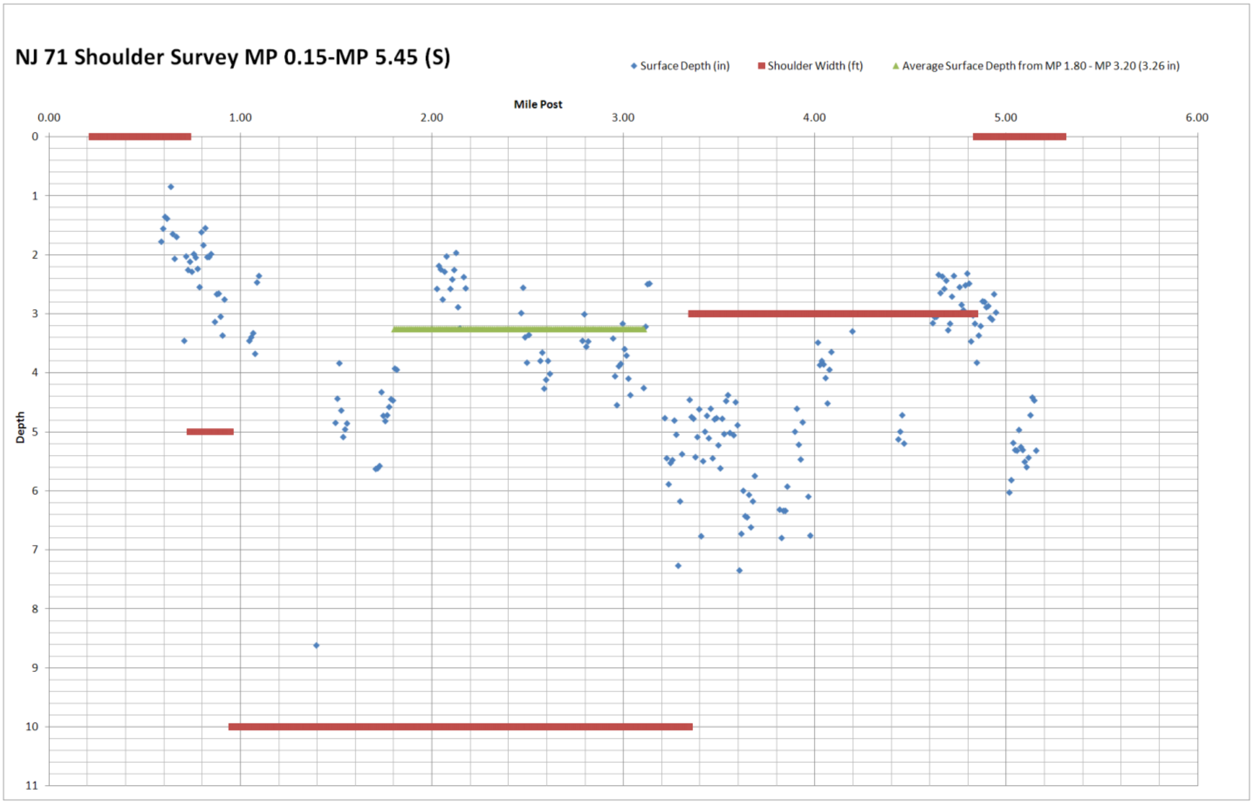

Figure 3: GPR Layer Thickness by Mile Post (SB)

The north lane of NJ 71 has an average thickness of 3.43 inches for the surface layer and the south lane has an average thickness of 3.26 inches for the surface layer.

Figures 4 and 5 represent the dielectric constants observed over distance for the north and south bound shoulders respectively. The dielectric constants show little deviation which demonstrates reliable results.

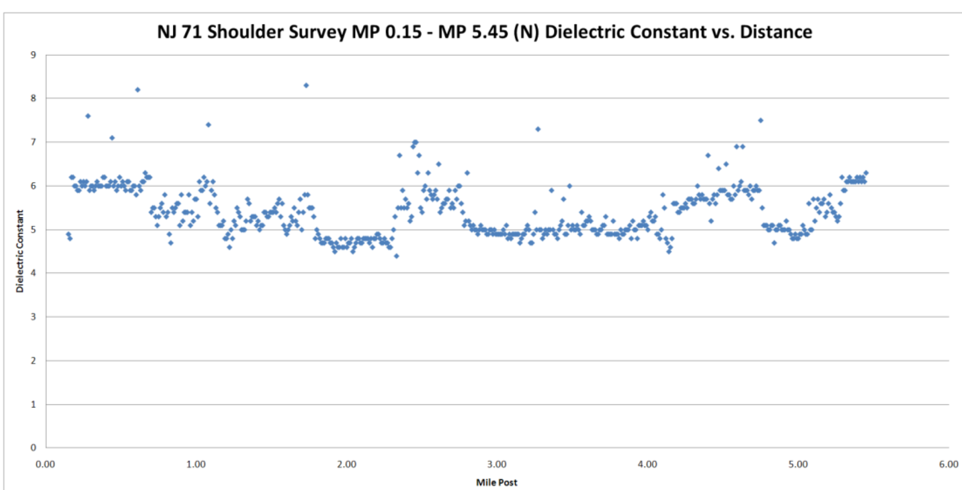

Figure 4: Dielectric Constants (NB)

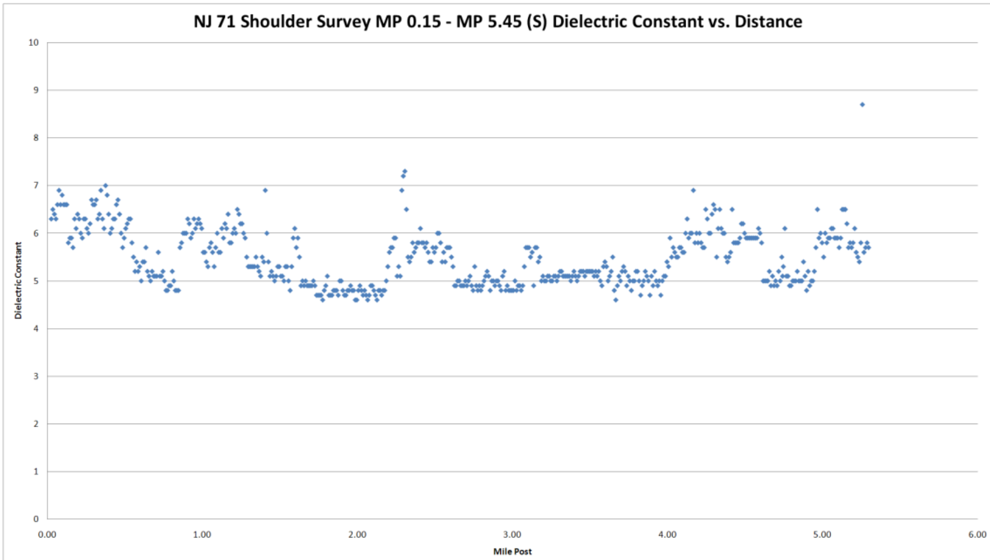

Figure 5: Dielectric Constants (SB)

Implementation of Performance-Based HMA Mixtures in NJ

Thomas Bennert, Ph.D.

Rutgers University

Center for Advanced Infrastructure and Transportation (CAIT)

CAIT RUTGERS

Acknowledgements

- Eileen Sheehy, Materials Bureau of NJDOT
- Robert Blight and Susan Gresavage, NJDOT Pavement Design and Management
- Robert Sauber, Advanced Infrastructure and Design, AID (formerly NJDOT)
- Frank Fee, NuStar Energy
- Mike Jopko, Trap Rock Industries

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Problem

- Current asphalt mixture design procedures based on volumetrics – no performance check
  - Aggregate gradation, VMA, VFA
- Asphalt binder specs provide an idea of performance but not reliable for today's asphalt mixtures
  - High RAP & RAS mixtures
  - Warm mix asphalt
  - Differences in asphalt plant production and storage
- Production issues and binder contamination (storage tank and lines) – more later

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So, Why Performance-Based Specs?

- Tests the "End Result"
- Combines the interaction of the aggregate, asphalt binder, and other additives (RAP, WMA, fibers, etc) with the plant production and storage (temperature and time)
  - Current methods looks at the components separately
- Shouldn't material actually on roadway be tested for performance?

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Performance-Based Specs – NJDOT's Specialty Mixes

- These mixtures are designed to help with a specific condition/distress on a pavement in NJ
  - Granted, some mixes may not be appropriate for other states/regions
  - Performance testing associated with mixture design phase and plant production phase

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Are these mixes designed differently?

- No – still using Superpave methods and procedures
- However,
  - Included mixture performance testing to ensure mixes are performing at required level(s)
  - Some difference in material selection (i.e. – no natural sands, different asphalt binders, change in volumetric targets)
    - MAKE SURE TO READ THE SPECIFICATIONS AHEAD OF TIME!
    - MAKE SURE TO CONTACT MATERIAL/BINDER SUPPLIERS AHEAD OF TIME!

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NJDOT Design & Acceptance

1. Perform volumetric design and NJDOT verification
2. Supply Rutgers University lab prepared loose mix (or virgin materials) for performance testing
3. Produce mix through plant and pave test strip off site
4. Sample during production and supply Rutgers University loose mix for performance testing
5. Sample and test every other Lot

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General Performance Tests Used

- Rutting Check – Asphalt Pavement Analyzer (AASHTO T340)
- Flexural Cracking Check – Flexural Beam Fatigue (AASHTO T321)
- Pavement Cracking Check – Overlay Tester (NJDOT B-10 & ASTM Spec coming)

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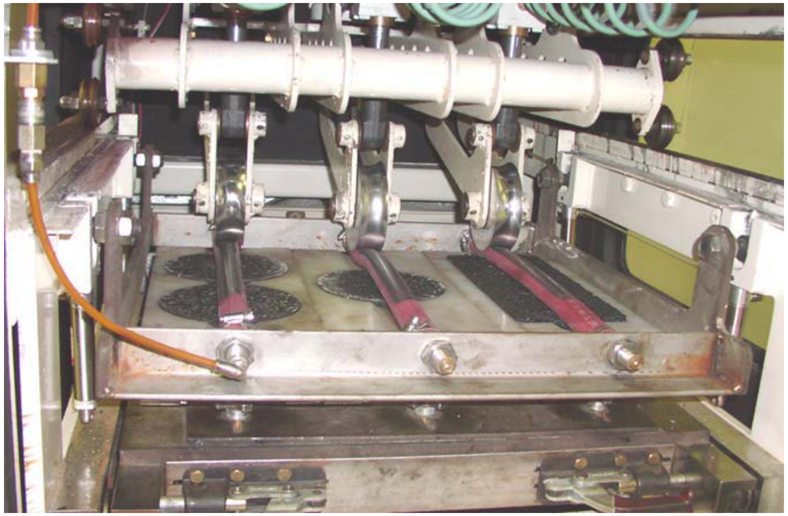
Asphalt Pavement Analyzer

- AASHTO T340
- 100 lb. wheel load;
  100 psi hose pressure
- Tested at 64^{o}C (148^{o}F) for 8,000 cycles
- Samples at specified air voids
- APA Rutting < “X” mm to pass



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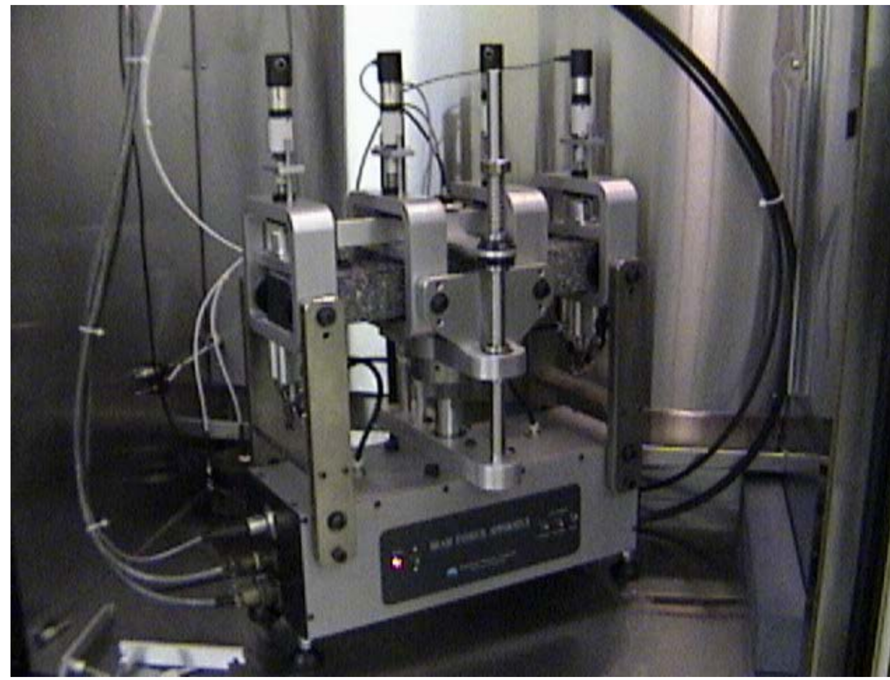
Flexural Beam Fatigue

- Flexural Beam Device, AASHTO T321
- Test mixes ability to withstand repeated bending
- Run at strain levels higher than expected field strains to accelerate testing



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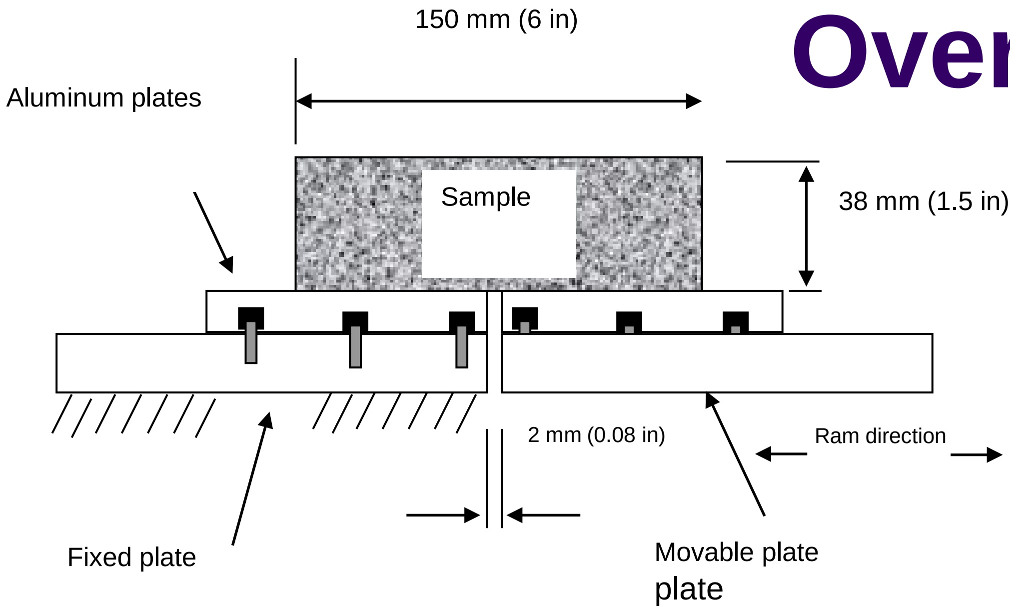
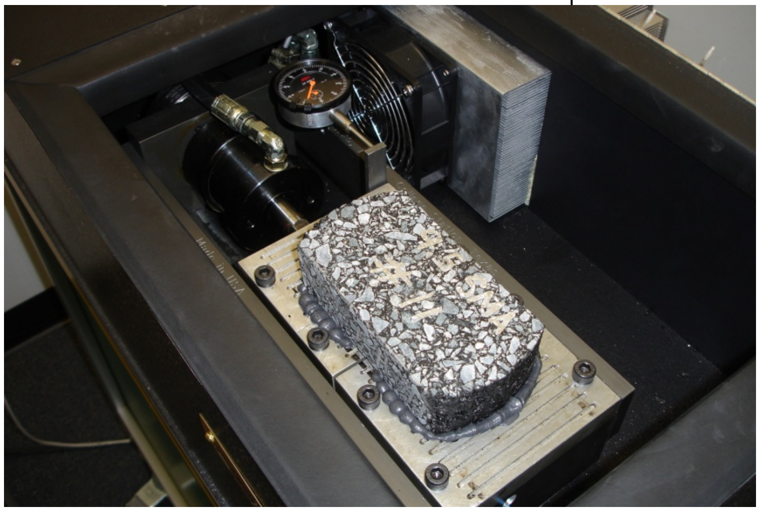
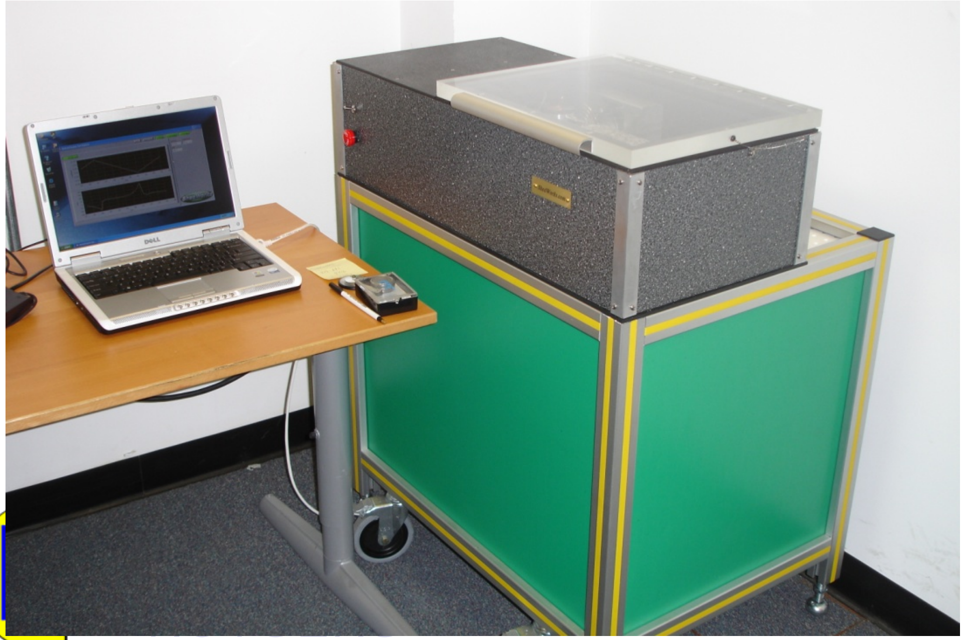
Overlay Tester







- Sample size: 6’’ long by 3’’ wide by 1.5’’ high
- Loading: Continuously triangular displacement 5 sec loading and 5 sec unloading
- Definition of failure
  - Discontinuity in Load vs Displacement curve

NJ's Performance-Based Mixes

- High Performance Thin Overlays (HPTO)
- Bridge Deck Water-proofing Surface Course (BDWSC)
- Bottom Rich Base Course (BRBC)
- Bottom Rich Intermediate Course (BRIC)

High Performance Thin Overlay (HPTO)

- Main Purpose – used as a rut-resistant and durable thin lift mix for maintenance/pavement preservation (DOT and Local Aid), as well as a superior leveling course (DOT)

HPTO

- 4.75mm Superpave
- 7% min PG 76-22 binder
- 3.5% AV @ N_{design} = 50 Gyrations
- Field Compaction: 2 - 7% mat voids
- 1” +/- Lift Thickness
  - Steel roller in static mode
- Performance Test: APA
  - APA Rutting < 4mm at 8,000 cycles

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HPTO Applications

- Thin Lift Overlay for Preventive Maintenance
- Leveling Course
- Bridge Deck Overlay
  - Small quantity

Beginning to use in conjunction with WMA to reduce potential for swelling due to PCC joint sealants and patching materials

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Bridge Deck Waterproofing Surface Course (BDWSC)

- Main Purpose – to provide a rut and fatigue resistant and impermeable bridge deck overlay mix that can be placed using static rollers (i.e. – preserving critical bridge infrastructure)

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Bridge Deck Waterproofing Surface Course (BDWSC)

- Highly Modified Mix for Bridge Decks
- Mixture Performance Testing
  - Rutting = APA
  - Cracking = Flexural Beam Fatigue
- 50 Gyrations @ 1% AV, 7% min AC
- 3% Max Air Voids in the Field

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BDWSC

- Recommended Binders: PG 76-28 to a PG 82-34 Polymer Modified Binder, or
- Concentrated Thermoplastic Polymeric Asphalt Modifier (dry mix)
- APA: < 3 mm @ 8,000 loading cycles
- Flexural Fatigue: >100,000 cycles @ 1500 microstrains (originally used 2000$\mu\varepsilon$)
- Mix Performance Tests used for final acceptance, regardless of binder grade or additive

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BDWSC
Rt.80 ACROW Bridge

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BDWSC
Rt.80 ACROW Bridge

- Life of the HMA overlay
  - Nov. 2009 – Paved 2.5” to 3.5” of HMA 12.5H76 Surface Course
  - March 26, 2010 – Opened to WB traffic
  - April 8, 2010 – Started patching HMA due to excessive and rapid deterioration – cracking and shoving
  - May 5 – 6, 2010 - Removed FAILED HMA
  - “HMA overlay practically failed immediately but was patched until more resilient mix placed”

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BDWSC
Rt.80 ACROW Bridge

RUTGERS

BDWSC
Rt.80 ACROW Bridge

CA
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BDWSC
Rt.80 ACROW Bridge

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BDWSC
Rt.80 ACROW Bridge

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BDWSC
Rt.80 ACROW Bridge

- Life of the BDWSC
  - Paved BDWSC on May 5-6, 2010
  - Opened to WB traffic immediately
  - WB Traffic on BDWSC until Dec.17, 2010
  - 7 ½ months with ZERO distress!
  - Opened to EB traffic January 2011
  - 6 months with ZERO distress!
  - ACROW temporary bridge taken down at end of 2011.
    - **1.5 years of service with no distress**

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BDWSC
Rt.80 ACROW Bridge

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BDWSC
Rt.80 ACROW Bridge

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Bottom Rich Base Course (BRBC)

- Main Purpose – base course mixture designed specifically to meet the flexural needs of a perpetual pavement (site specific)

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Bottom Rich Base Course (BRBC)

- Used summer 2010 on I-295 rubblization project to decrease the required pavement thickness.
- 19 mm Base Course mix with 5% min. of PG 76-28 binder
  - Binder grade chosen based on initial mix testing
- Fatigue Resistance - 100 μ-strain @ 100,000,000 cycles
  - Based on Endurance Limit procedure from NCHRP Project 9-38
- APA (rutting) 5mm at 8,000 cycles

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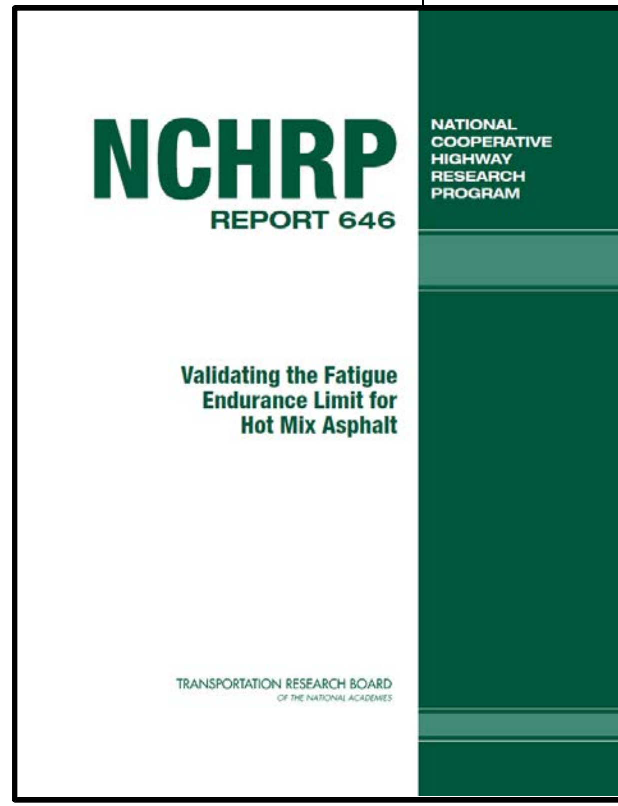
Endurance Limit from NCHRP Project 9-38

- Used methodology in NCHRP Report 646
- Conduct flexural beam fatigue at 400 and 800$\mu\varepsilon$
  - 3 samples each
- Use 95% confidence interval with a selected # of repetitions



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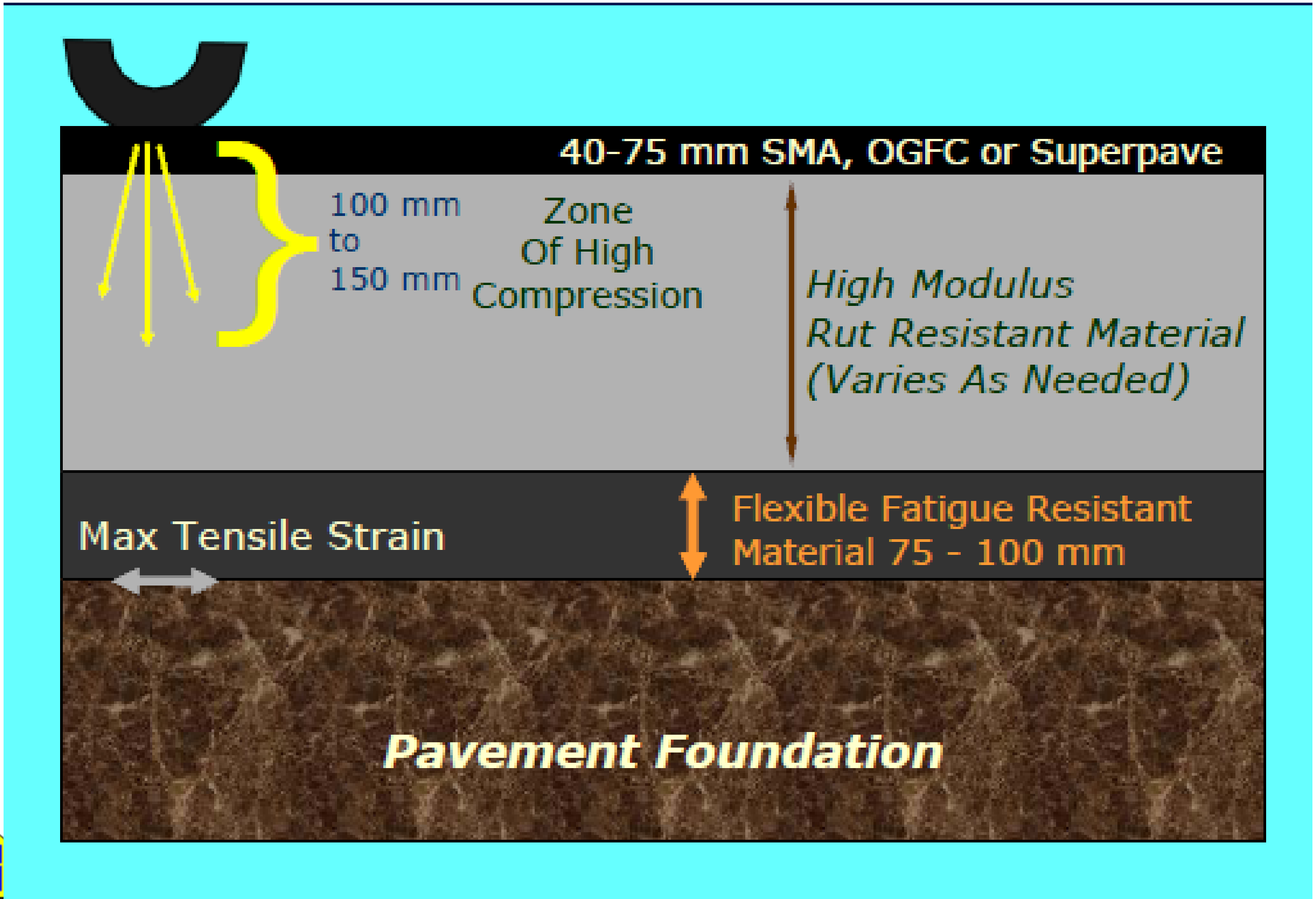
BRBC – Perpetual Pavement Design



RUTGERS

I-295 Design Methodology

- Evaluated maximum tensile strain with 8” HMA over rubblized PCC
  - Used JULEA software to estimate tensile strain
  - Resulted in 82 micro-strains (rounded up to 100 microstrains to be conservative)
- Final design pavement cross-section
  - 2” SMA Surface
  - 3” 19M76 Intermediate Course
  - 3” of NJDOT Bottom Rich Base Course
    - Designed specifically for this project
    - Utilized Endurance Limit concept

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BRBC

- 50 Gyrations @ 3.5% AV
- 2%-8% Mat Air Voids
- Full flexural fatigue suite required during mixture design and test strip production
  - 3 beams at 400 $\mu\varepsilon$ and 3 beams at 800 $\mu\varepsilon$
  - Only 3 beams at 800 $\mu\varepsilon$ during plant production (1st and every 5th Lot)

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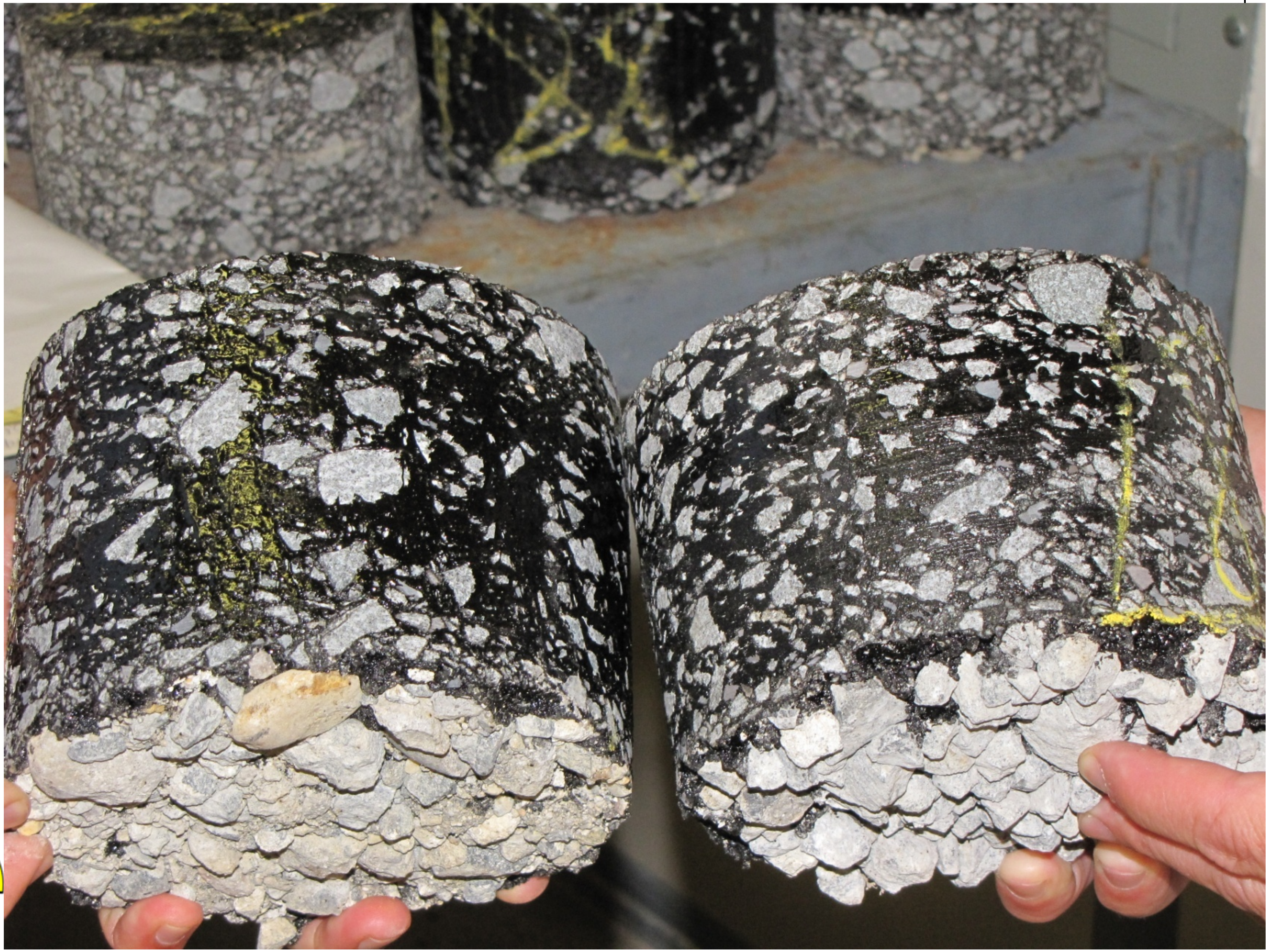
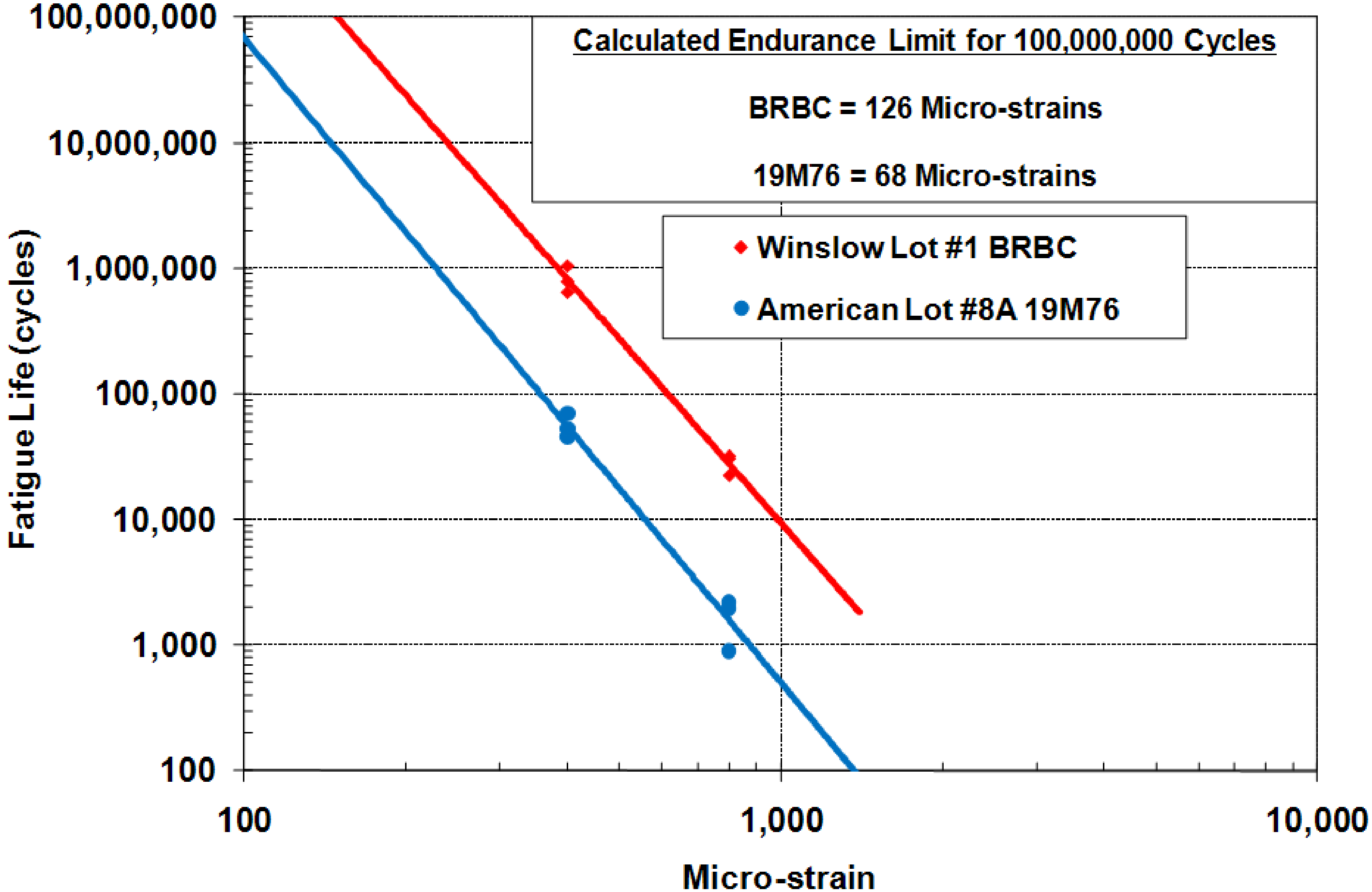
BRBC in Field

RUTGERS

BRBC Core Samples

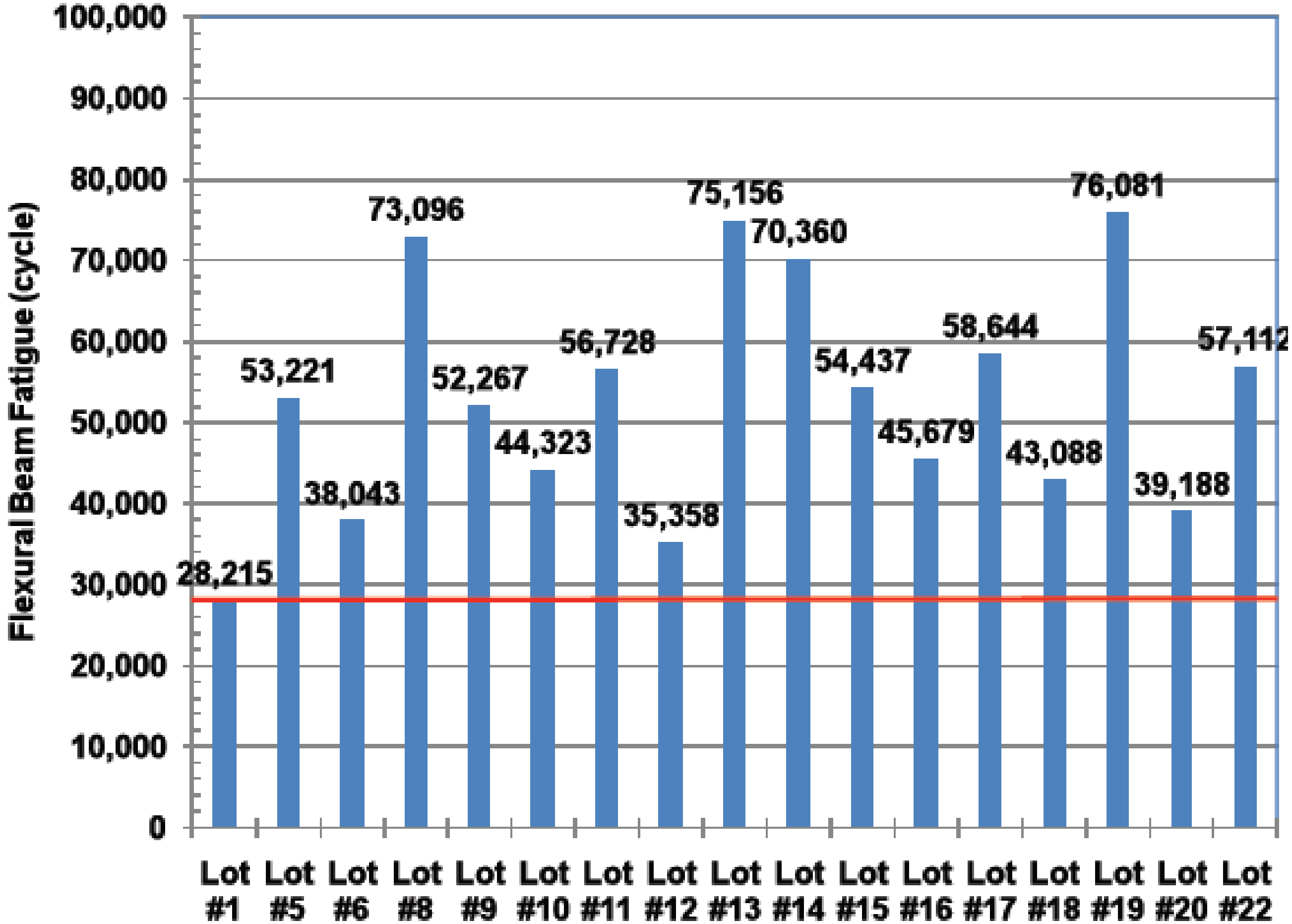


BRBC



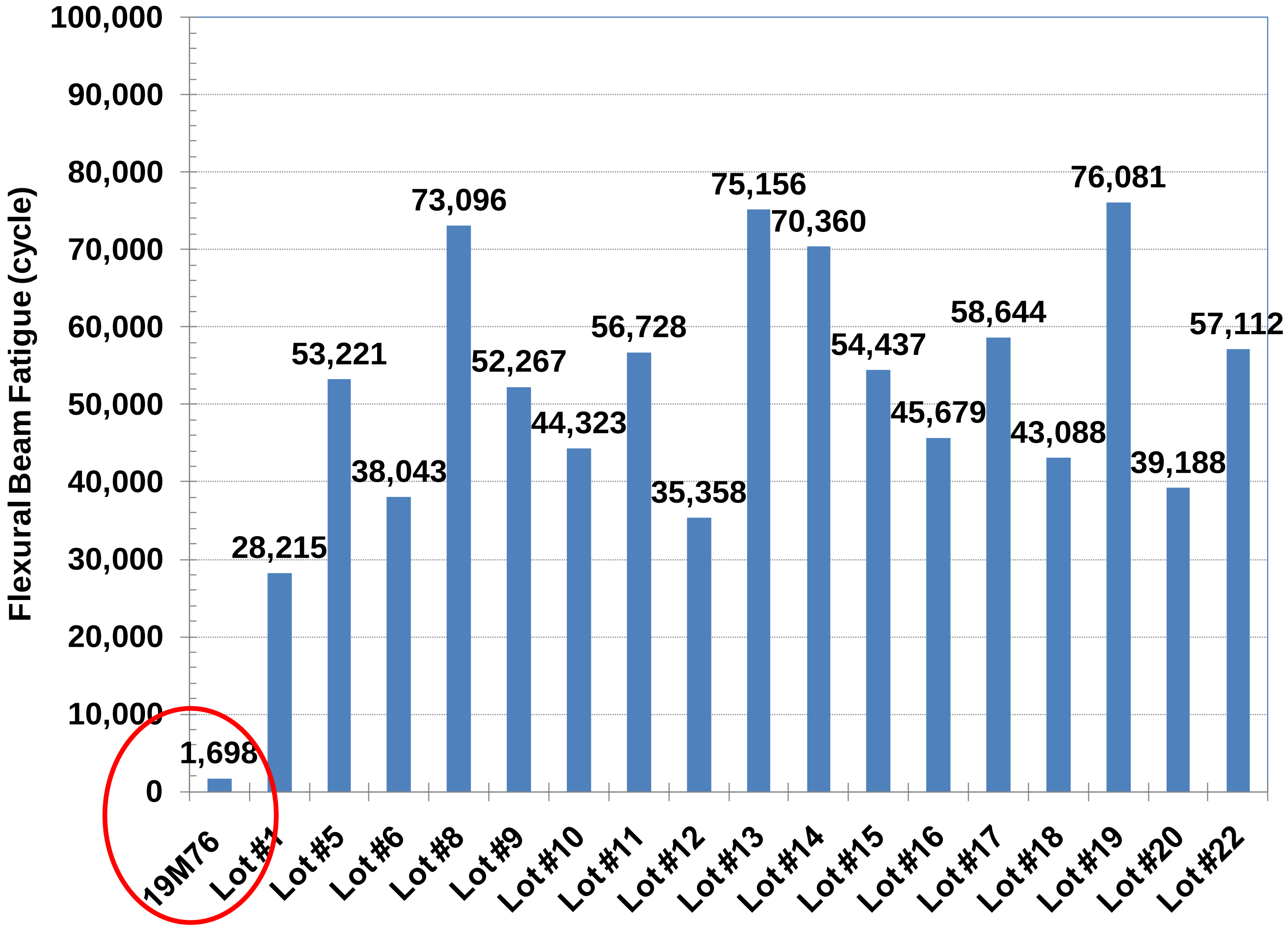
Calculated Endurance Limit for 100,000,000 Cycles
BRBC = 126 Micro-strains
19M76 = 68 Micro-strains
Winslow Lot #1 BRBC
American Lot #8A 19M76
Fatigue Life (cycles)
100,000,000
10,000,000
1,000,000
100,000
10,000
1,000
100
100
1,000
10,000
Micro-strain

Rt.295 BRBC Fatigue Results @ 800 micro-strains

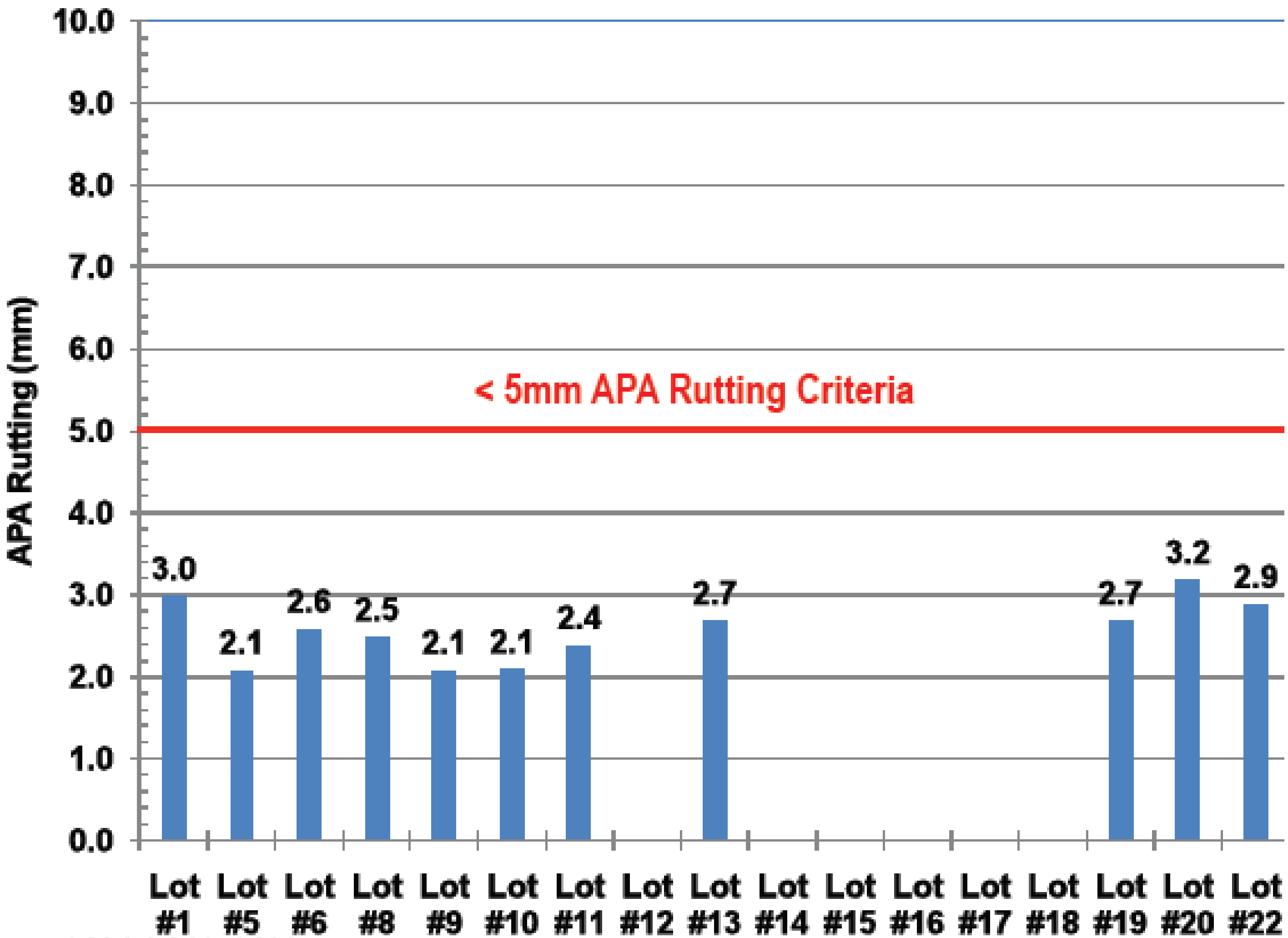


RUTGERS

Rt.295 BRBC Fatigue Results @ 800 micro-strains



Rt.295 BRBC APA Rut Results



RUTGERS

Bottom Rich Intermediate Course (BRIC)

- Main Purpose – to be placed over PCC/bottom of HMA overlay on composite pavement to withstand cracking due to horizontal joint movement (environmental) and vertical joint movement (traffic)
  - Important to note – mixture placed over BRIC still needs to be flexible enough to resist residual vertical bending

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Reflective Cracking on MA I495

Bottom Rich Intermediate Course (BRIC)

- Superpave 4.75 mm Intermediate Course with PG 70-28 binder
  - Very similar to TxDOT’s CAM mixture
- Mix performance testing required.
  - TTI Overlay Tester (reflective cracking)
  - APA (rutting)
- A number of projects proposed this year
  - 1” BRIC
  - 1.5” to 2” SMA Surface Course

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SOME THINGS TO WATCH OUT FOR

Binder Storage Tank

- Manufacturers recommend to not drain tanks below heating coils
- Therefore, always have residual binder at bottom of tank

Horizontal Tank

Vertical Tank

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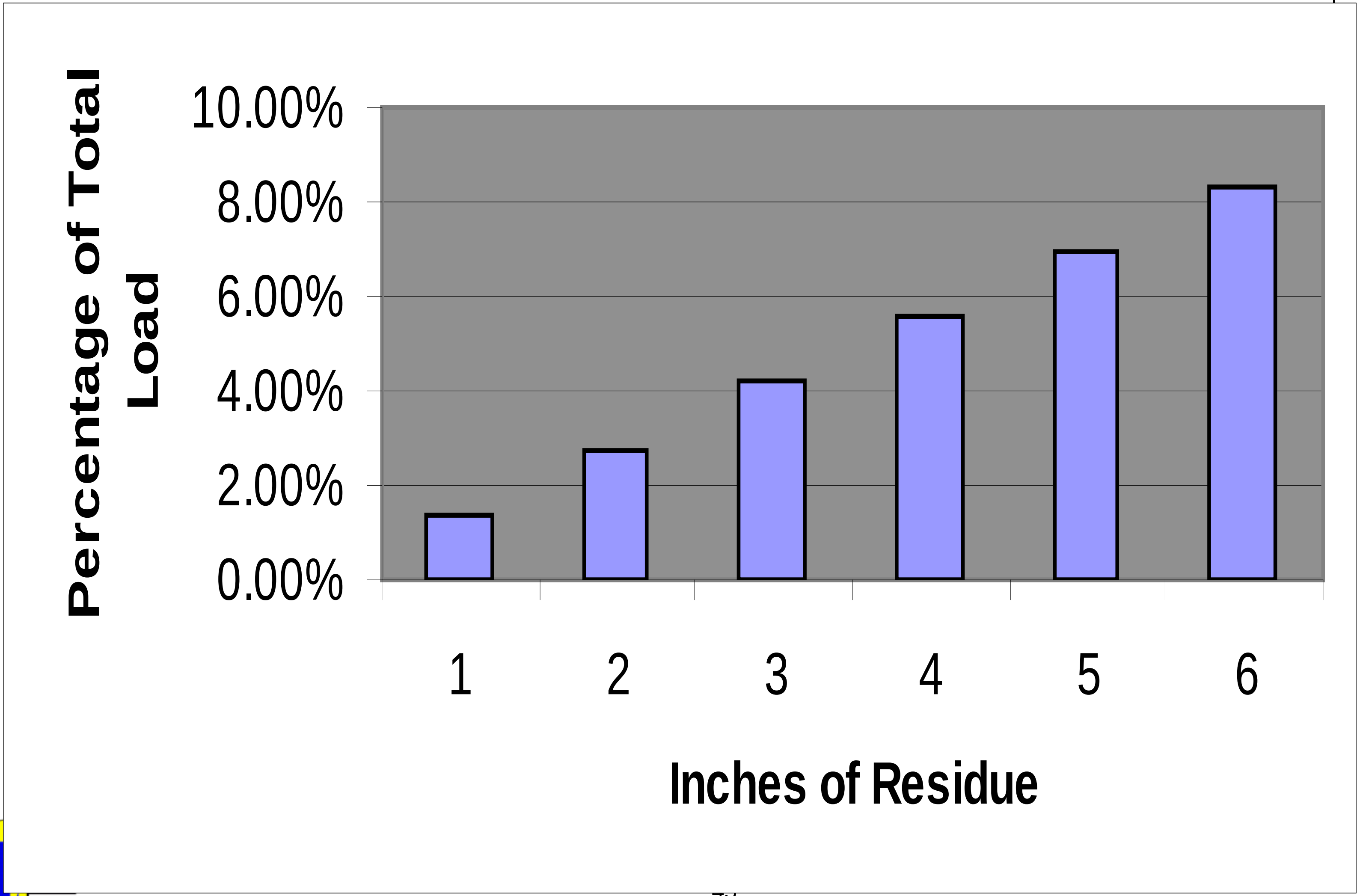
Residue as % of Load



Asphalt Lines from Storage Tanks

- Leads from storage tank to mixing vessel (drum or pug mill)
- Typical length ~ 70 ft
- Typical ID ~ 4 inches
- Equates to around 0.2 tons of residual liquid binder in the asphalt lines alone

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In Summary: Why NJ Using Performance-Based Mixes?

- Today's roadways, with high traffic and extreme climate conditions, require more than Mill 2", Pave 2" on typical HMA
- Based on performance data (lab and field), along with costs for mixes, these mixes are a "Smart Economic Investment"
- Need tools in the toolbox for all situations
- Have to make sure we use the "Right Mix, On the Right Road, At the Right Time, for the Right Price"

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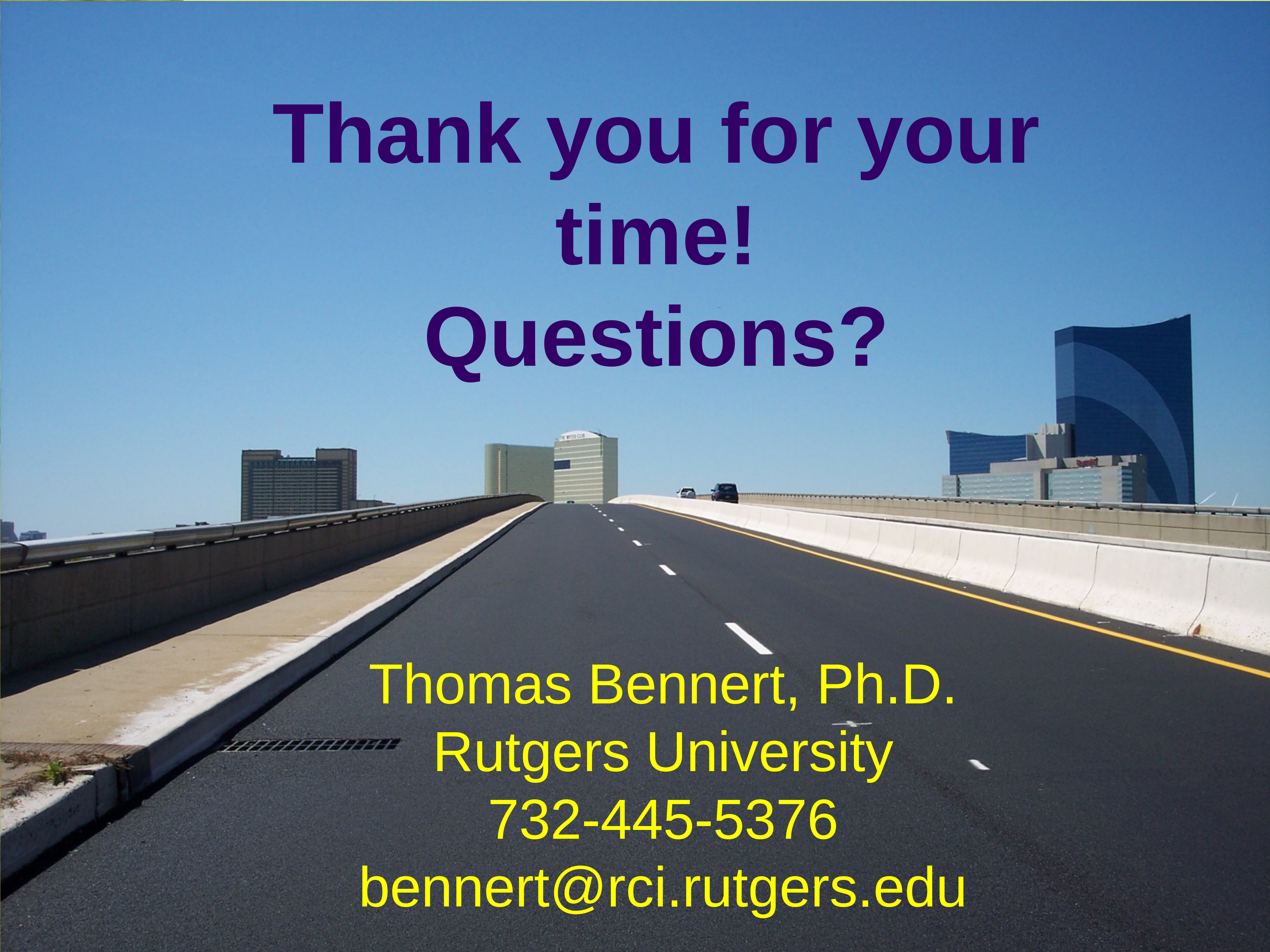

Thank you for your time!

Questions?

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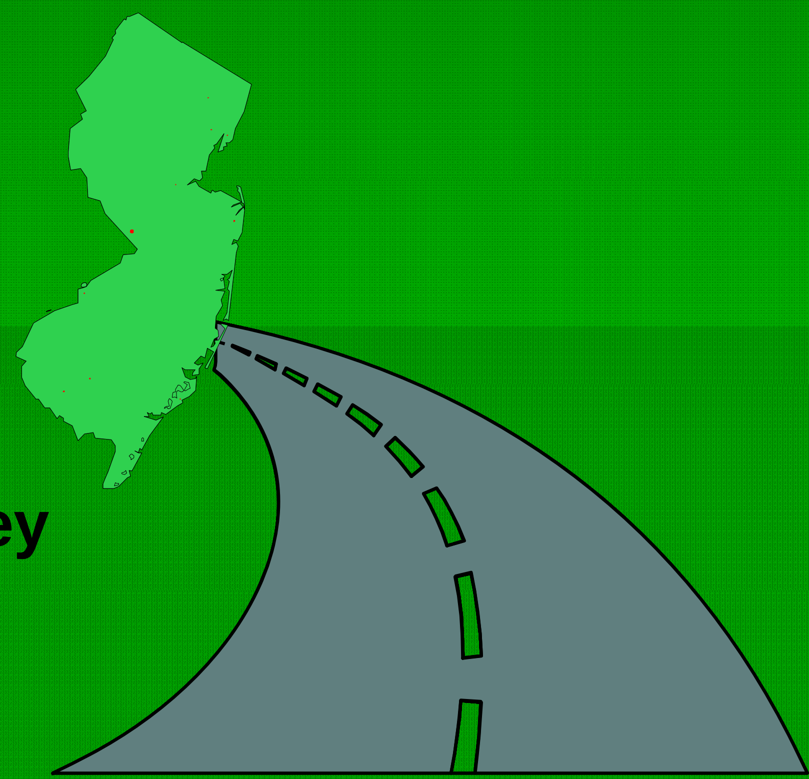
NJ's Innovative Approach for Rt. I-295 Reconstruction

Presented by:

- **Robert Sauber, NJ Asphalt Pavement Association**
- **Thomas Bennert, Rutgers University**



Presented to:

- **ASHE SNJ & ASCE South Jersey**
- **Cranbury Inn, NJ**
- **December 8, 2010**

The Problem

- Rt I-295 constructed 1972 to 1974
- Reached terminal serviceability a decade ago
- PCCP with ASR (alkali-silica reaction)
- Limited pavement program funding
- High traffic volumes that must be maintained during reconstruction

Potential Solutions

- Patch and overlay, cost $26 million
  - Short service life not cost effective
  - Ultimate fix will be more difficult
- Replace broken slabs
  - Too slow and expensive, cost overrun risk
  - Not a long term solution

Chosen Solution

- Full Closure to increase production and lower project cost, time is $$$
- Hyperbuild to reduce traffic impact and obtain public by in for full closure
- Sustainability Elements
  - Rubblization to recycle in place and reduce cost and duration
  - Engineered HMA base course to reduce total overlay thickness

Project Specifics

- Project Limits
  - Rt. I-295 NB & SB MP 45 to 57.3
  - Three12’ travel lanes with 4’ inside and 12’ outside shoulders
  - Total paved width of 52 ft each direction
- 21 structures within project limits resulting in 20 undercut locations to maintain underclearance
- Full closure limited to 59 days during summer recess when traffic was “lower”

NJDOT's prior experience with Rubblization

- Route I-295 Burlington & Camden County
  - Contractor: RE Pierson
- Route I-78 Essex & Union Counties
  - Contractor: Union Paving
- Route I-295 Gloucester County
  - Contractor: RE Pierson
- All pavements were 78' long x 12' wide, 9" thick JRCP over 12" granular subbase

Why Rubblization?

- Rubblization is a viable, rapid, and cost-effective rehabilitation method for deteriorated PCC pavements
- Rubblization $1.46/sy vs. Removal $5.76/sy
  - Average of 3 lowest I-295 bids and typical
- Rubblization is cost effective when the amount of patching exceeds approximately 10 percent of the project area (NJ)
- Lower Risk to Owner and Contractor
  - Reduced subgrade exposure to moisture damage

Rubblization Benefits

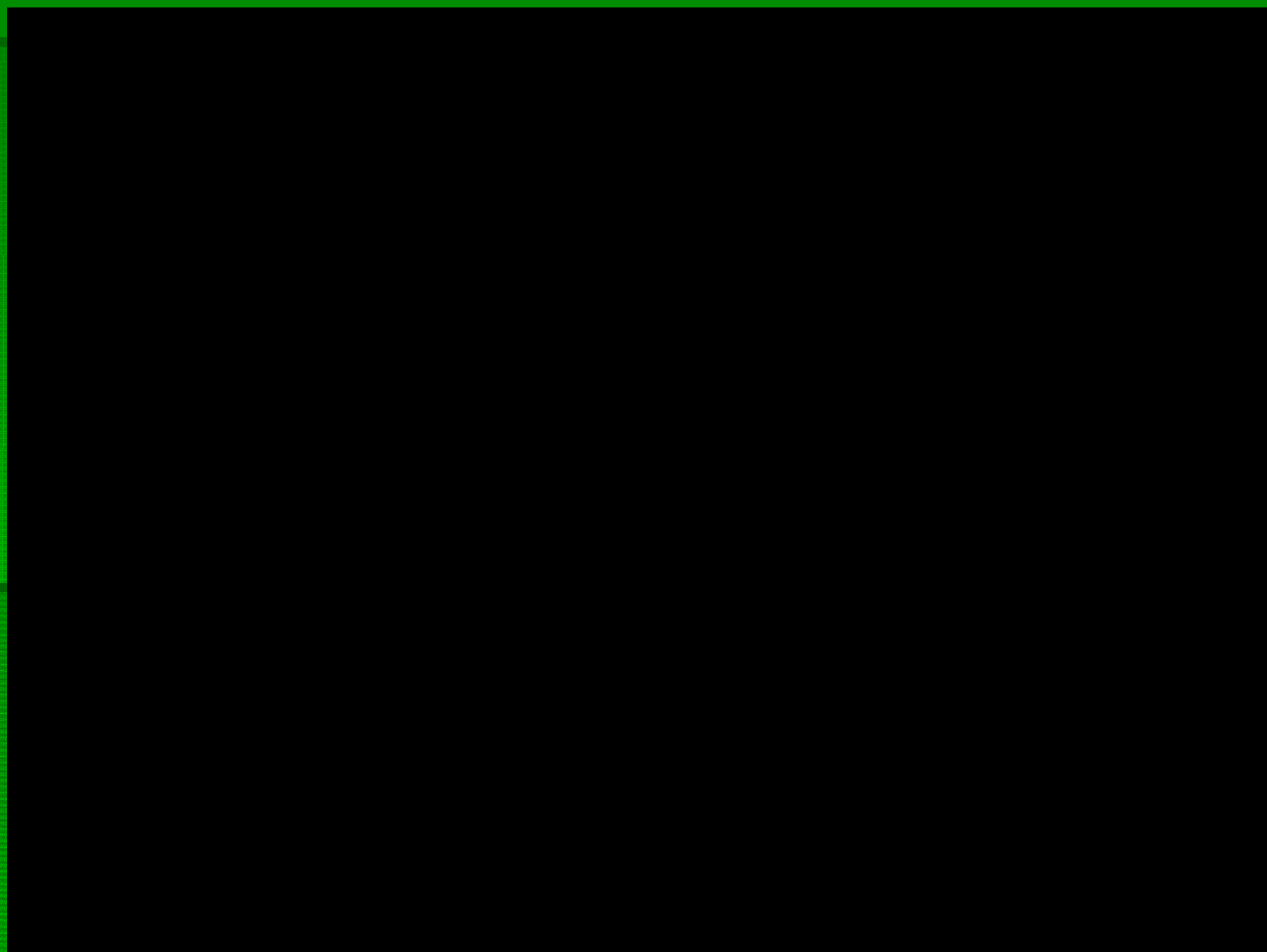
- Rubblization Saves Time
  - Typical rubblization process recycles one lane mile per day, with no material hauling
  - 4X faster than breaking, excavating, hauling and placing DGABC using traditional methods
- Rubblization Saves Money
  - Approximately 50% cost savings compared to reconstruction with PCCP
  - Approximately 33% cost saving compared to reconstruction with HMA

Sustainability Benefits

- Water Consumption: 41% Reduction
- Energy Consumption: 44% Reduction
- CO_2 Emissions: 43% Reduction
- NO_x Emissions: 26% Reduction
- PM_{10} Emissions: 48% Reduction
- SO_2 Emissions: 40% Reduction
- CO Emissions: 38% Reduction

source: RMRC case study of a NHDOT project

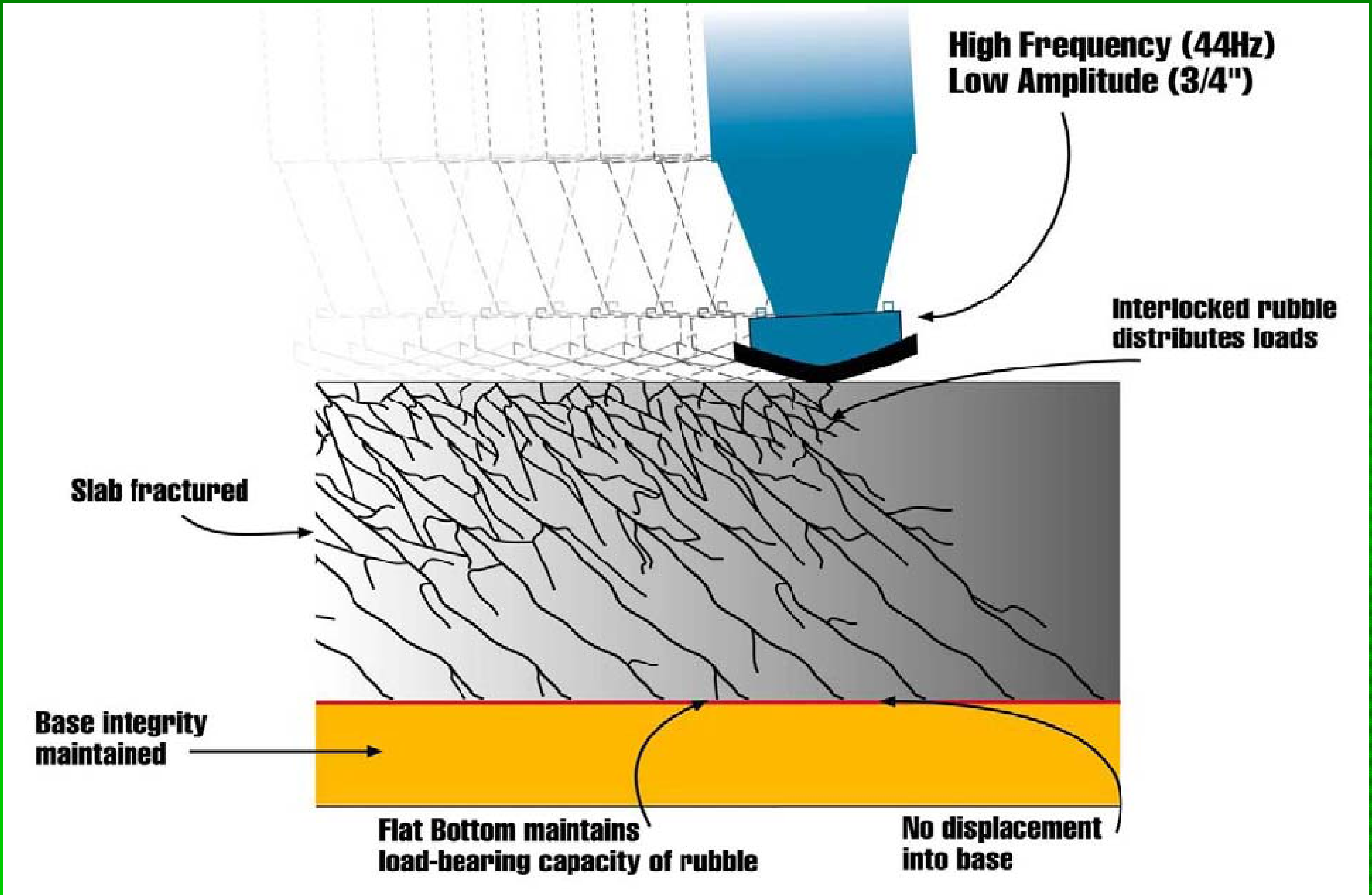
Rt I-295 Resonant Pavement Breaker




RMI
RB 500
RESONANT PAVEMENT BREAKER
RMI


RMI
TULSA, OKLA
RESONANT
PAVEMENT
BREAKER

Illustration of PCC fracturing resulting from Resonant Rubblization



Resonant Pavement Breaker

- Resonant breaker encroaches 3 to 5 feet on the adjacent lane when rubblizing the centerline
- Breaking pattern is approximately 8 inches wide and requires 18 passes to break a 12-foot lane width
- 20,000 lb wheel load and 60,000-70,000 lb weight can damage rubblized pavement

Resonant broken PCC pavement

Multi-Head Breaker (MHB)

- MHB is a self-propelled unit with multiple drop-hammers mounted at the rear of the machine
- hammers are set in two rows, and strike the pavement approximately every 4.5 in
- 1,200 lb - 1,500 lb hammers have variable drop heights and variable cycling speeds
- can break pavement up to 13 ft wide in one pass
- production is approximately 1.0 lane-mi per day
- Z-pattern steel grid roller, a vibratory roller with a grid pattern, must be used in conjunction with the MHB to complete the breaking process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295


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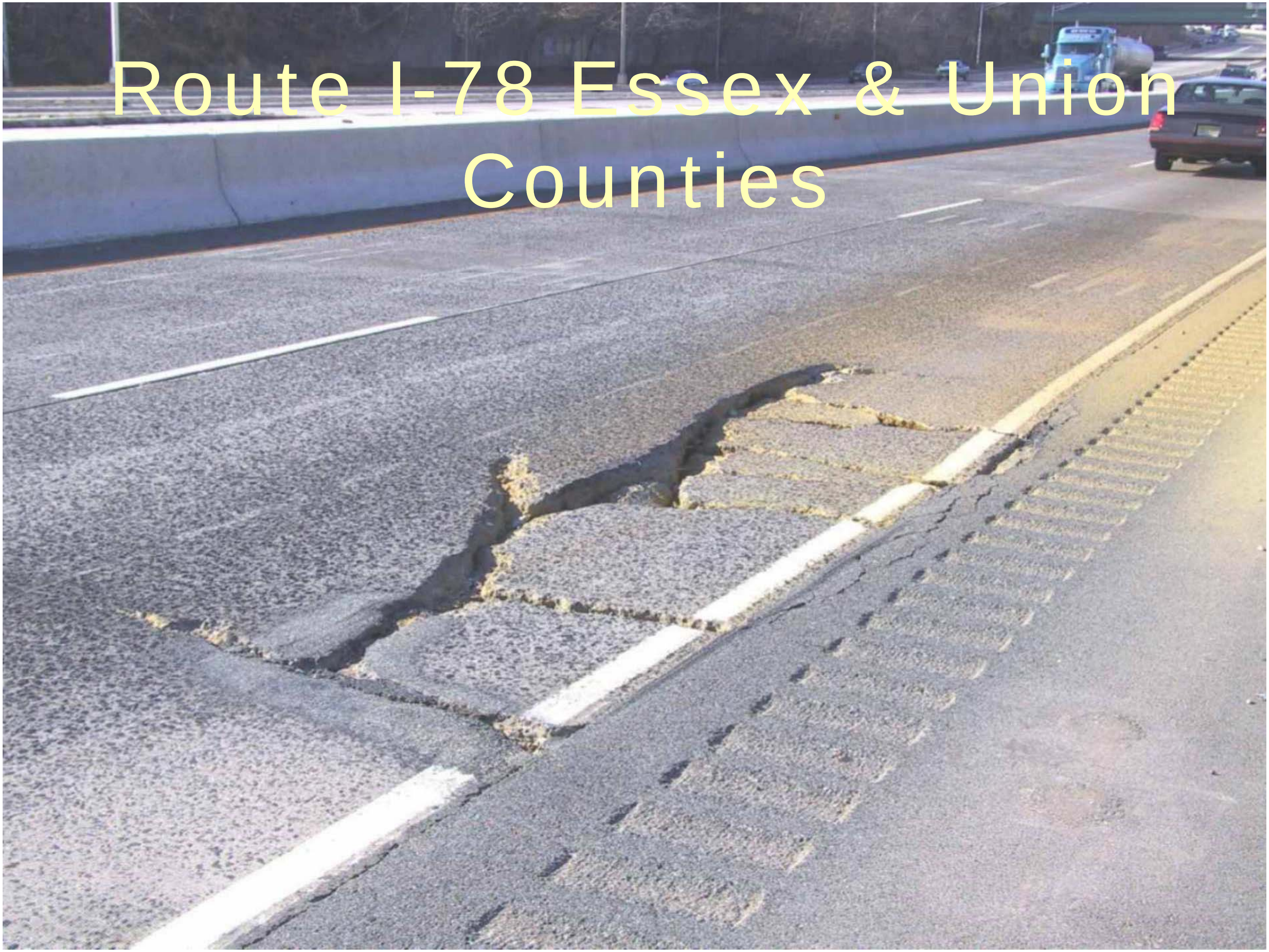
I-78 MP 54 Sept '06

Route I-78 Essex & Union Counties



Route I-78 Essex & Union Counties

Route I-78 Essex & Union

Rubblized and compacted Route I-78 PCCP


Cross section of rubblized I-78 PCCP

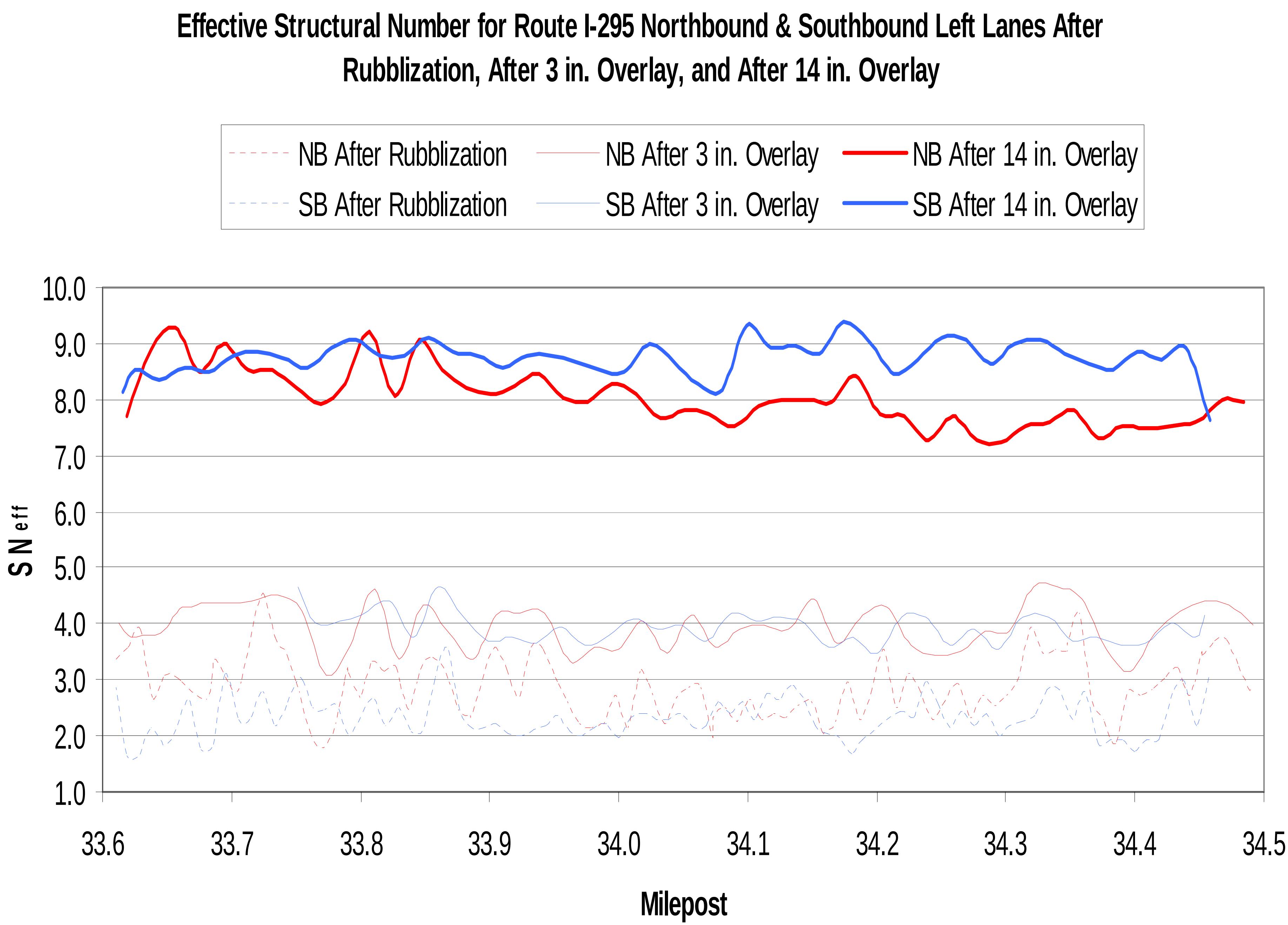

Portable Seismic Property Analyzer (PSPA) for soils

Route I-78 PSPA Test Results

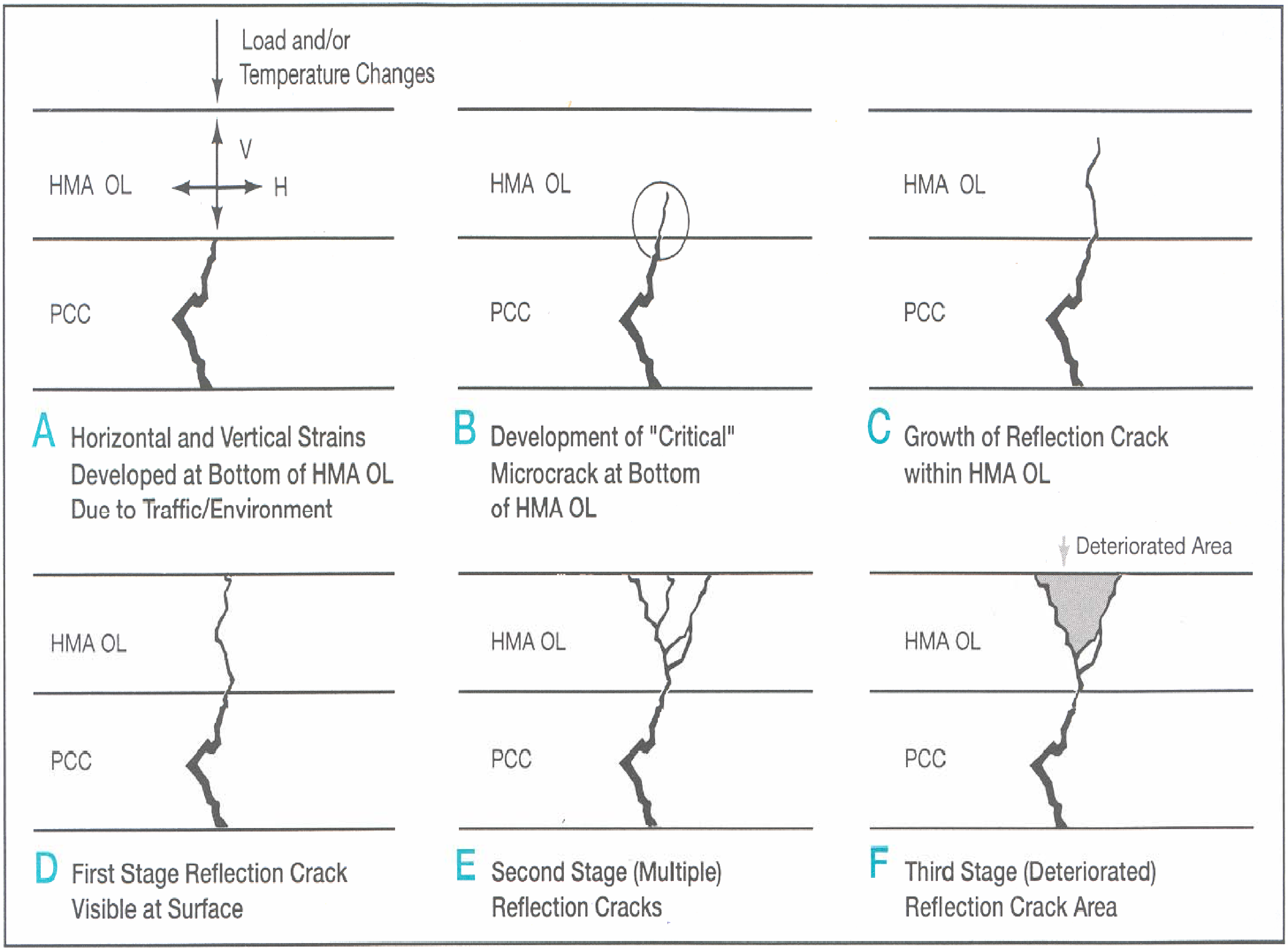
- Elastic modulus is evaluated from the average velocity of surface waves
- Seismic testing is a low strain modulus, reductions should be made to describe it as resilient modulus
- Modulus varied between 80 and 400 ksi
- Average modulus was 217 ksi

Objective of Rubblization

- **Eliminate reflection cracking in the HMA overlay by the total destruction of the existing slab action**
- **Slab is reduced to small pieces and diminished to a high-strength granular base**
- **Restoration of structural capacity is accomplished with an HMA overlay**



Effective Structural Number for Route I-295 Northbound & Southbound Left Lanes After Rubblization, After 3 in. Overlay, and After 14 in. Overlay
NB After Rubblization
NB After 3 in. Overlay
NB After 14 in. Overlay
SB After Rubblization
SB After 3 in. Overlay
SB After 14 in. Overlay
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
SNeff
33.6
33.7
33.8
33.9
34.0
34.1
34.2
34.3
34.4
34.5
Milepost


Load and/or
Temperature Changes
V
H
HMA OL
PCC
A Horizontal and Vertical Strains Developed at Bottom of HMA OL Due to Traffic/Environment
HMA OL
PCC
B Development of "Critical" Microcrack at Bottom of HMA OL
HMA OL
PCC
C Growth of Reflection Crack within HMA OL
HMA OL
PCC
D First Stage Reflection Crack Visible at Surface
HMA OL
PCC
E Second Stage (Multiple) Reflection Cracks
Deteriorated Area
HMA OL
PCC
F Third Stage (Deteriorated) Reflection Crack Area

General Rubblization Info

- Every rough, worn-out PCC pavement may not be a candidate for rubblization with an HMA overlay
- Evaluate the existing pavement, traffic, subgrade, and environmental conditions
- Understand the soil and moisture conditions for the pavement system prior to making a decision
- Most PCC pavements can be rubblized in an appropriate manner and overlaid with HMA

General Rubblization Info

- Rubblized PCCP is an order of magnitude less than a concrete slab, say 300,000 psi as compared to 3 million psi for concrete
- Rubblized modulus values are at least twice the modulus values typically used for crushed stone base

General Rubblization Info

- For thicker slabs, rubblized particles tend to be larger and interlocked stronger, leading to a higher modulus
- For thinner slabs on subgrade, reduced support results in poor particle interlock leading to a lower modulus
- No traffic (including unnecessary construction traffic) should be allowed on the fractured pavement surface

Rubblization Design Criteria

- AASHTO M-E Design Guide for Highways 150 ksi for PCCP 8 to 12 inches thick
- Asphalt Institute Airfield Project 2007
  - Slabs 6 to 8 in. thick: Moduli from 100 to 135 ksi
  - Slabs 8 to 14 in. thick: Moduli from 135 to 235 ksi
  - Slabs >14 in. thick: Moduli from 235 to 400 ksi

Rubblization Design Criteria

- 5 inches recommended as a minimum overlay thickness over rubblized PCCP
- At least two lifts of HMA are necessary to meet grading and smoothness requirements
- The first lift must be at least 3 inches to achieve compaction, lack of fine particles hinders confinement

Rubblization Design Criteria

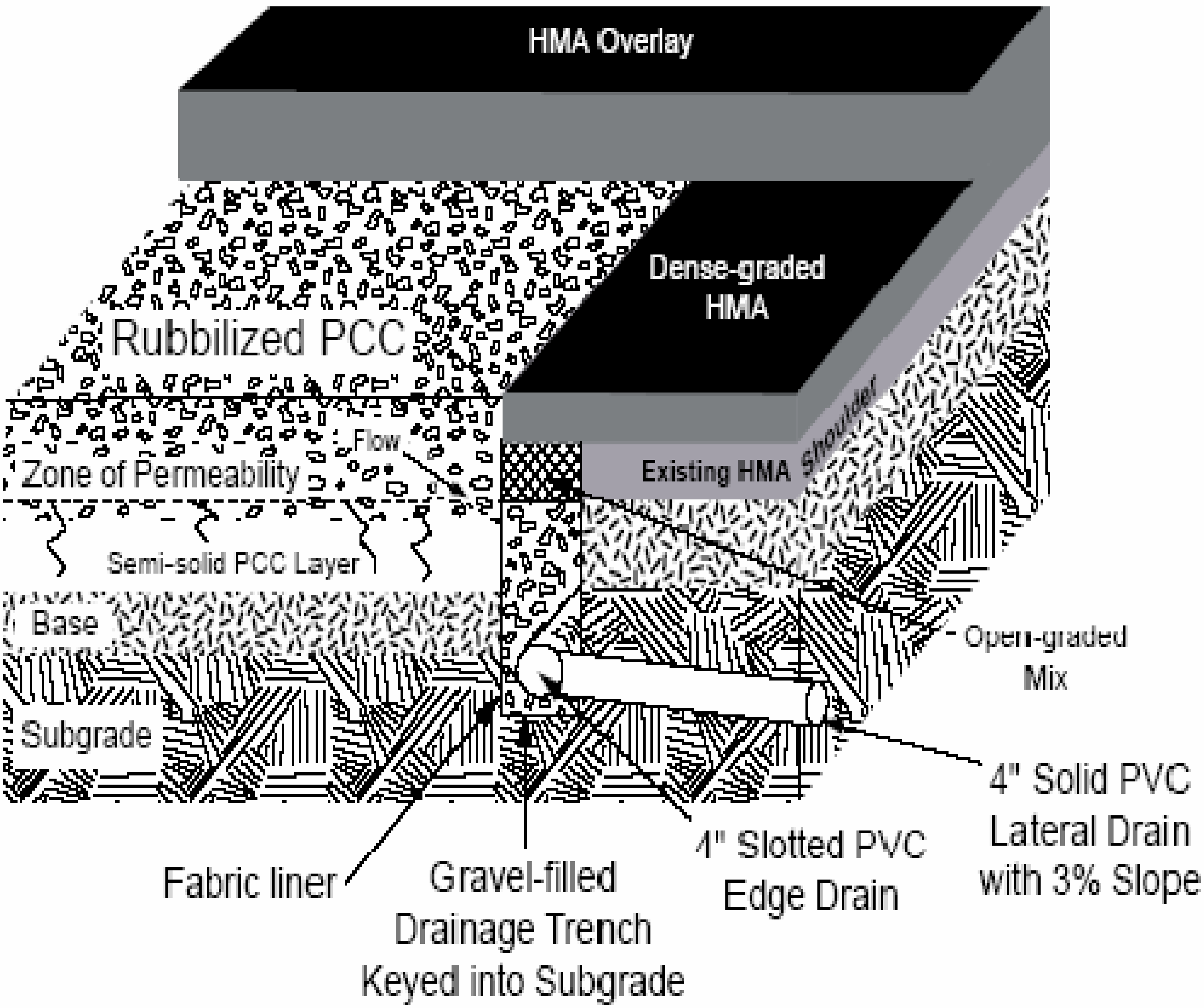
- Moduli of typical rubblized PCCP 100-400 ksi compared to crushed aggregate base with a typical range of 50-60 ksi
- Rubblized modulus (E) appears to be influenced by slab thickness; thicker slabs tended to have higher modulus
- Rubblized modulus related to the pre-rubblized PCC modulus, retained modulus
- No difference in rubblized moduli between the two types of rubblization equipment (MHB and RPB)

Rubblization Design Criteria

- Rubblized modulus is dependent on the level of rubblization, too much can reduce the concrete to a granular base with moduli in the 50-60 ksi range
- Reinforcing steel reduces the effectiveness of rubblization, may be minimal breakage below the steel, test pits necessary
- Reinforcing steel increases both the pre and post-rubblized modulus

General Rubblization Criteria

- Subgrade condition is crucial for success in rubblization. Wet subgrades and/or soils with low bearing capacity may not rubblize
- Typically no change in subgrade moduli after rubblization
- When specified, underdrains should be installed and operational at least 14 days prior to rubblization


HMA Overlay
Dense-graded HMA
Rubbilized PCC
Flow
Zone of Permeability
Existing HMA
Shoulder
Semi-solid PCC Layer
Base
Open-graded Mix
Subgrade
4" Solid PVC Lateral Drain with 3% Slope
4" Slotted PVC Edge Drain
Fabric liner
Gravel-filled Drainage Trench Keyed into Subgrade

Crack & Seat

- The Crack and Seat is only applicable to Jointed Plain Concrete
- Crack spacing (18” to 36” is typical) is a function of the overall stiffness of the existing subbase-subgrade foundation.
- Seating of the broken slabs after cracking is intended to re-establish support between the base and the fractured PCC

Break and Seat

- Suitable for JRCP, steel debonding required
- Largely replaced by rubblization because of more consistent results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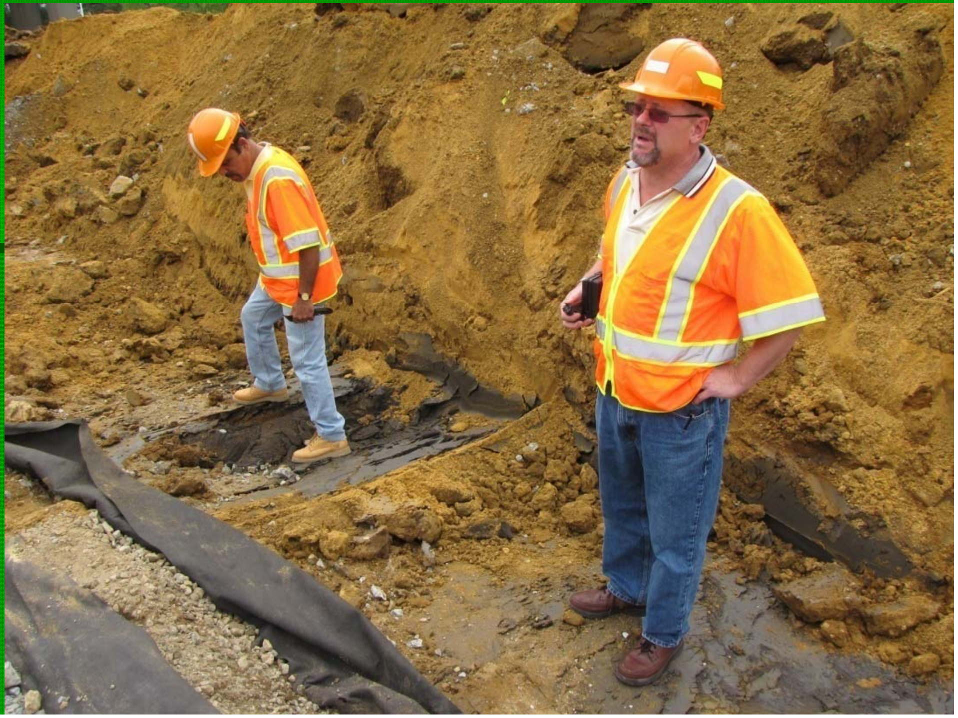
Initial Pavement Design

- Initial design conducted with 1993 AASHTO Pavement Design
  - 12” of HMA over rubblized PCCP, 4 lifts
  - Rubblized PCCP and subgrade modulus determined using FWD data from similar rubblized PCCP projects
  - The 59 day closure required extremely high HMA production and placement , up to15,000 tons/day

Problem with Design Thickness

- For 12" thickness, 2400 linear feet (+ width of bridge) of PCCP removal and box out at each structure, 100' transition per inch
- From past experience, box outs problematic because they are at low points
- Most boxes ended up requiring a 2 ft undercut (clay subgrade)
- Acid producing clay limited in-situ treatment

Typical Box Out



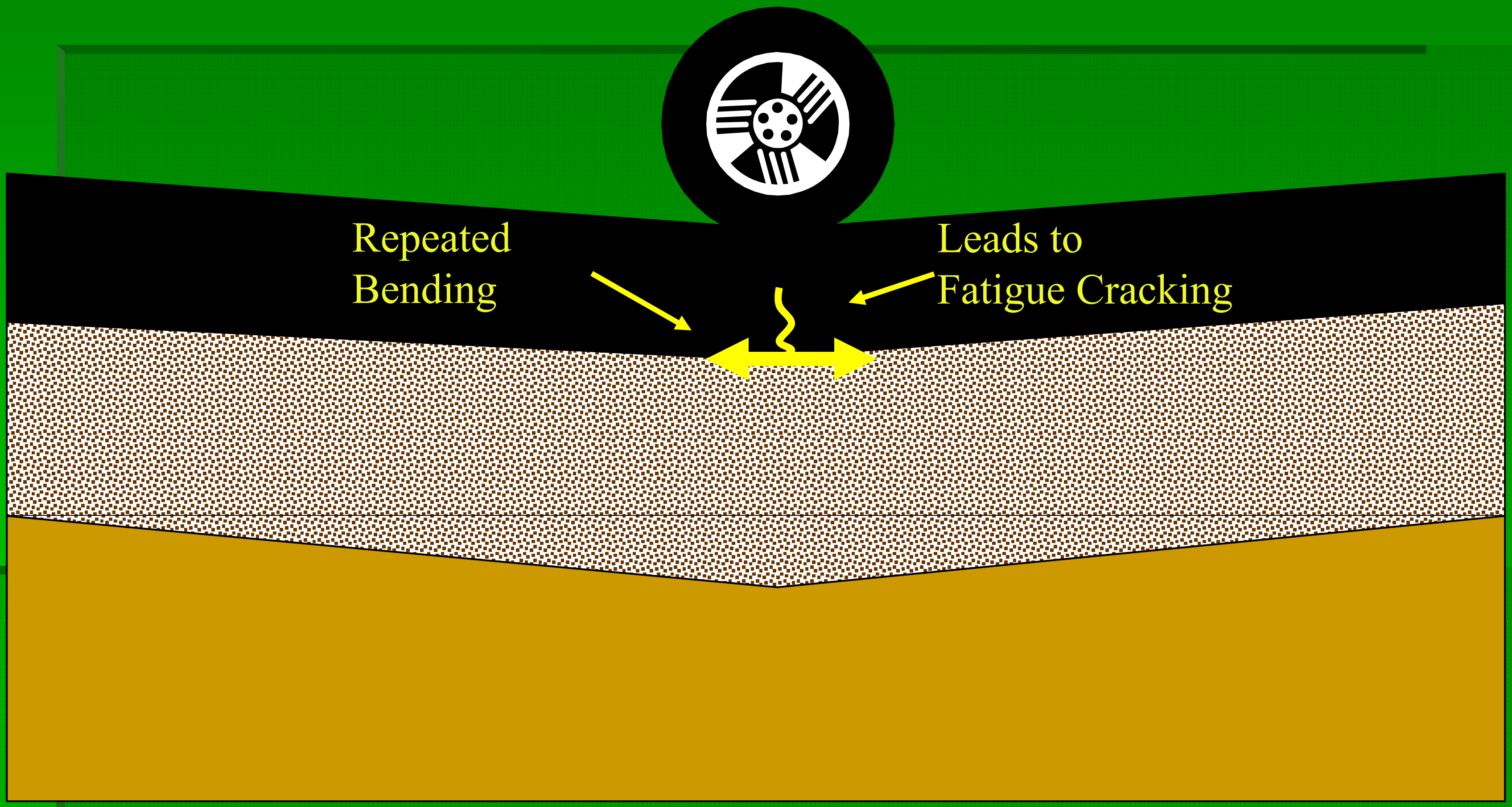
Acid Producing Clay

Rt. 295 NB Haul Road

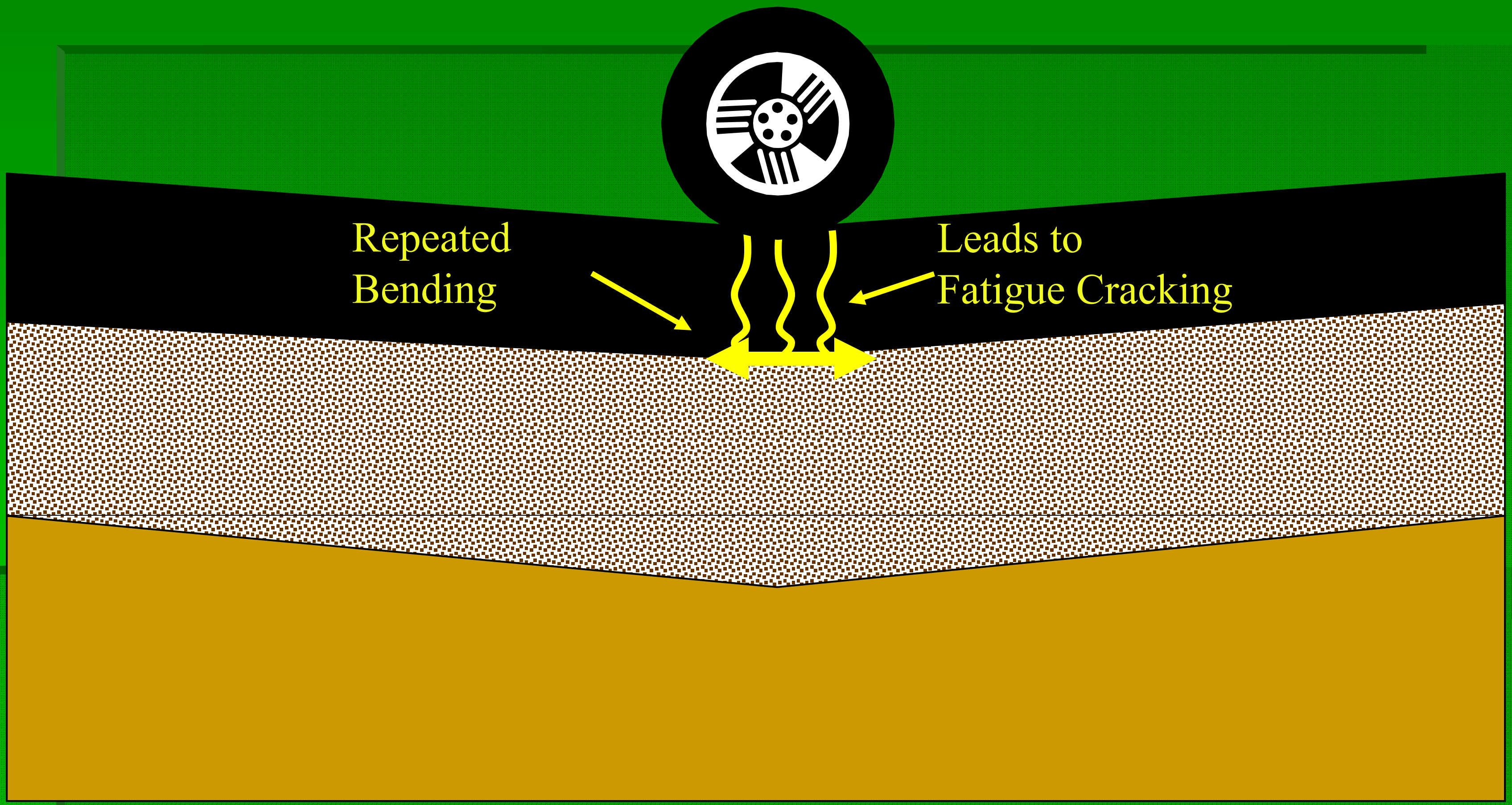
Different Design Approach

- NJDOT could save a lot of time and money from box out and undercuts if total HMA height 8 inches.
- Looked at pavement response in MEPDG (typical NJ HMA materials) and noted bottom-up cracking could be potential issue at 8 inches thick
  - Slight HMA rutting
- NJDOT decided on perpetual pavement design with “rich” bottom layer

Bottom-up Cracking



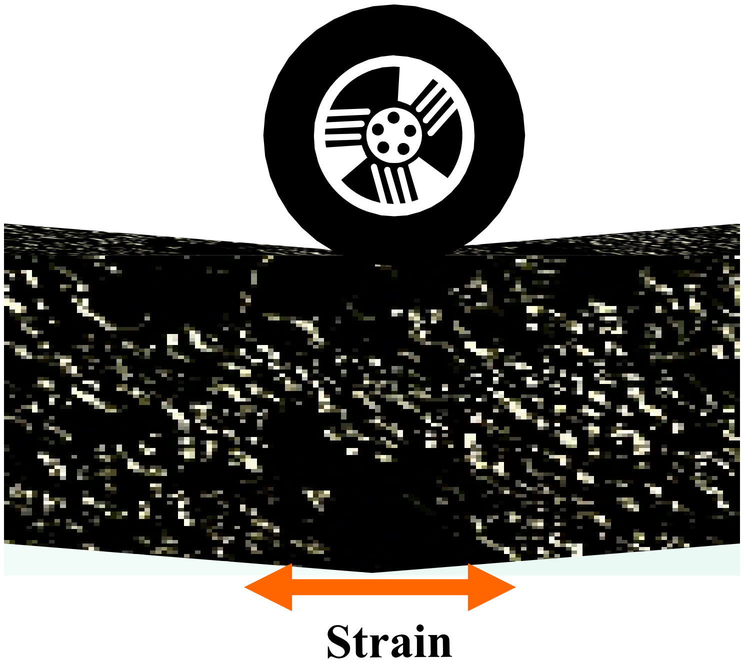
Bottom-up Cracking

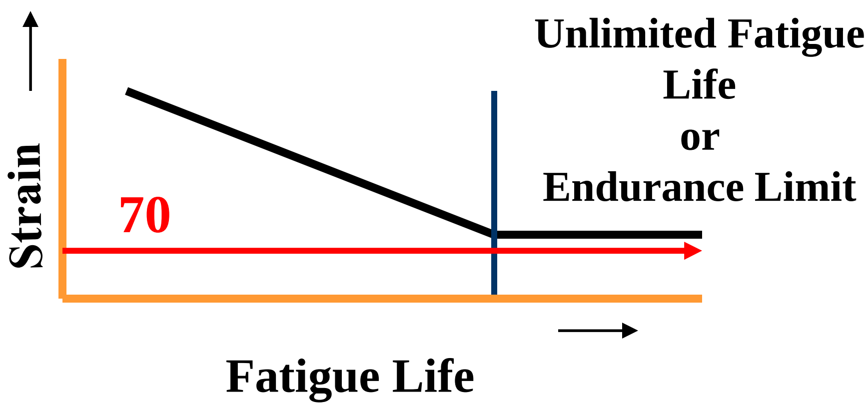


Fatigue Theory for Perpetual Pavements

High Strain = Short Life

Low Strain = Unlimited Life





Goal of Perpetual Pavement Design

- **Design the structure such that there are no deep structural distresses**
  - **Bottom up fatigue cracking**
    - **Limit tensile strain at bottom of asphalt layer**
  - **Structural rutting**
    - **Limit compressive strain at top of subgrade**
- **All distresses can be quickly remedied from surface**
- **Result in a structure with 'Perpetual' or 'Long Life'**

Surface Distresses Only

Top Down Cracking

Non-Structural Rutting

I295 – Designing for Perpetual Pavement

- Need to determine tensile strain at bottom of HMA
  - Use Elastic Layer Theory
  - Use "optimal" structure and thickness
  - Need to make sure HMA can withstand resultant tensile strain
- Need rut resistant HMA
  - New pavement section over rubblized PCC – very stiff so likelihood of structural rutting minimal – more concerned with surface rutting

Change Design Methodology

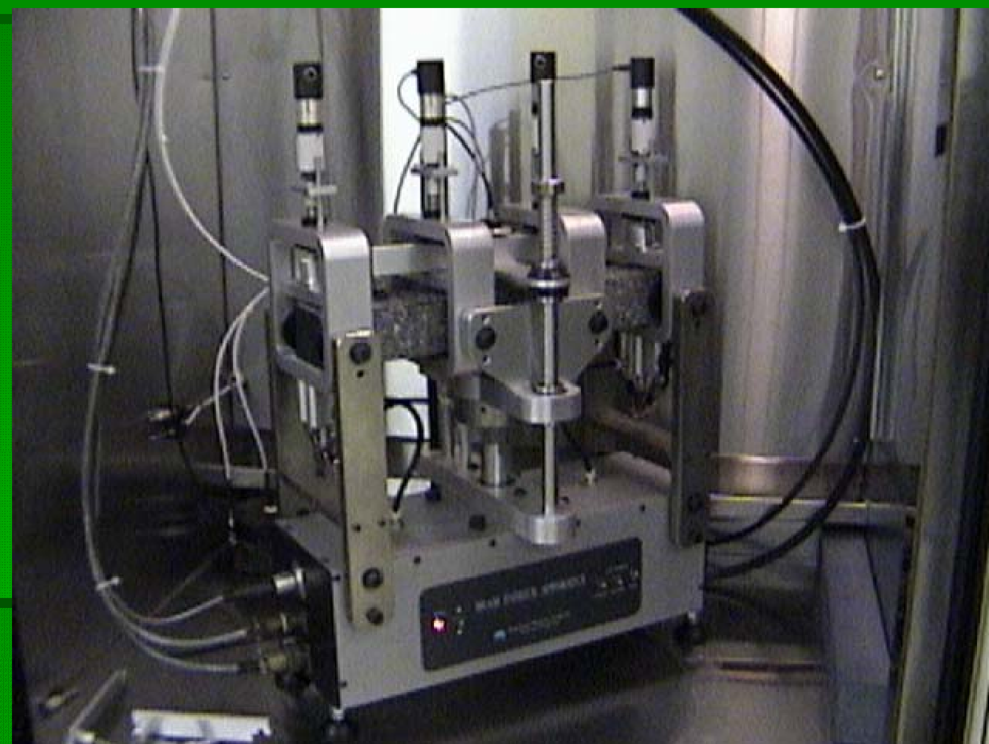
- Evaluated maximum tensile strain with 8" HMA over rubblized PCC
  - Used JULEA software – same in MEPDG
  - Resulted in 82 micro-strains (rounded up to 100 microstrains to be conservative)
- Final design pavement cross-section
  - 2" SMA Surface
  - 3" 19M76 Intermediate Course
  - 3" of NJDOT Bottom Rich Base Course
    - Designed specifically for this project
    - Utilized Endurance Limit concept

Endurance Limit

- Used methodology in NCHRP Report 646
- Conduct flexural beam fatigue at 400 and 800ms
  - 3 samples each
- Use 95% confidence interval with a selected # of repetitions

Flexural Fatigue Testing of HMA

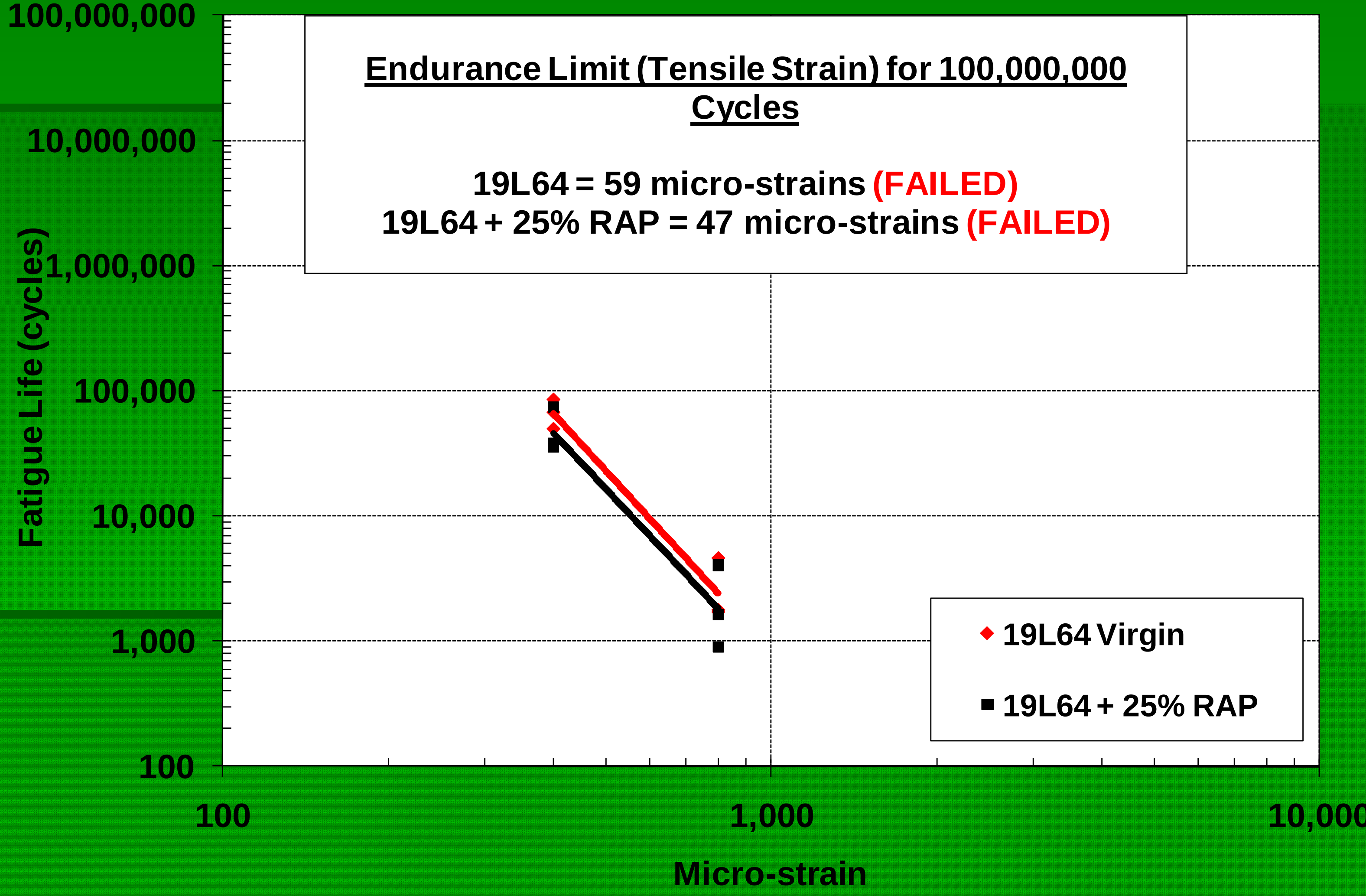
- Flexural Beam Fatigue Device, AASHTO T-321
- Tests mix's ability to withstand repeated bending
- Data = number of loading cycles to failure (Fatigue Life)
- Run at typical strain (deformation) to simulate anticipated pavement deflections



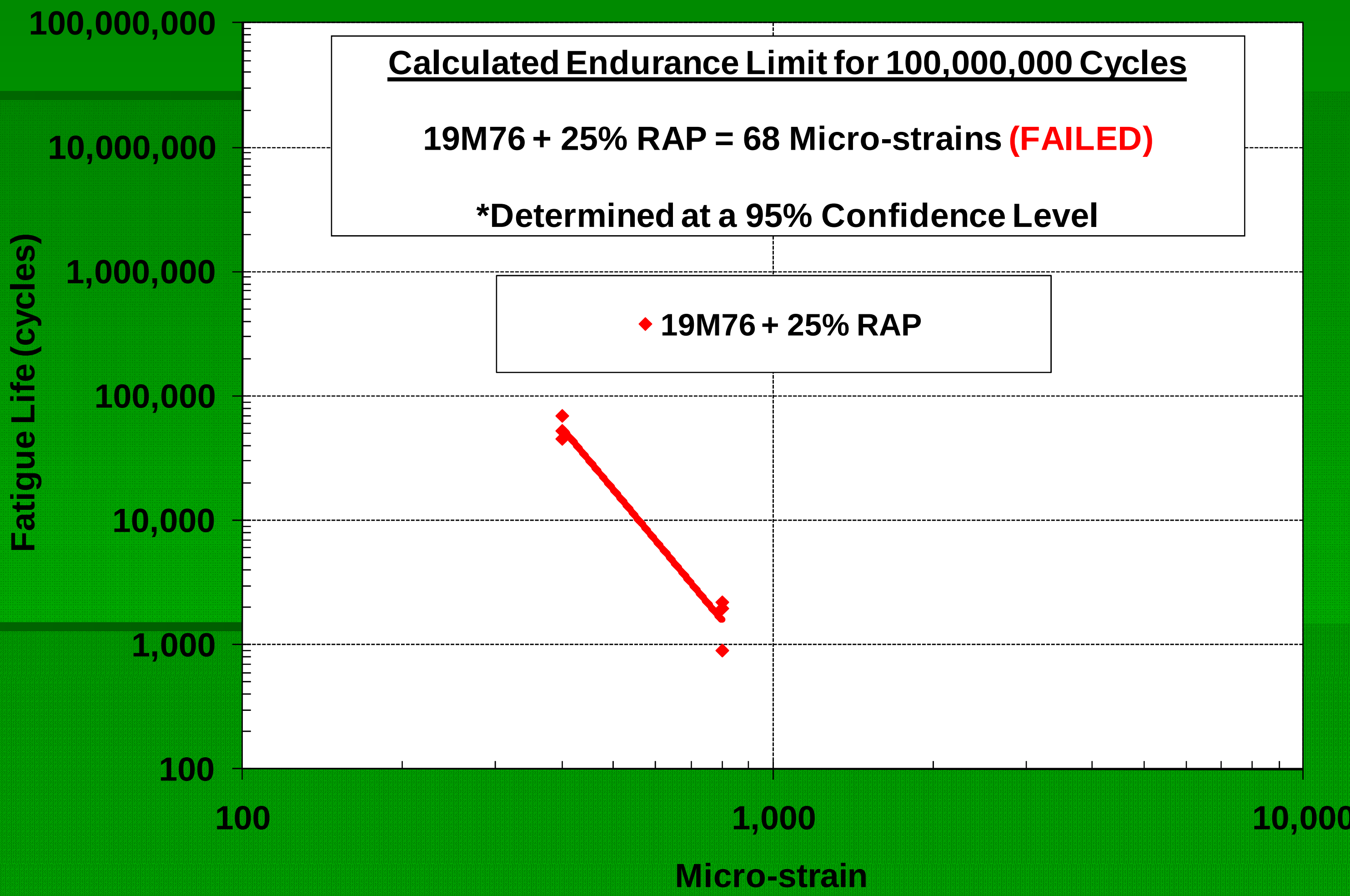
What Mix to Use?

- With performance evaluation in place, Rutgers University began testing plant produced mixes in Fall 2009
- Different base course mixes were evaluated – none were successful
  - Must achieve an Endurance Limit greater than 100 micro-strains at 100,000,000 cycles (NCHRP 9-38 had used 50,000,000 cycles)
- Required design of new mixture
  - Bottom Rich Base Course - BRBC

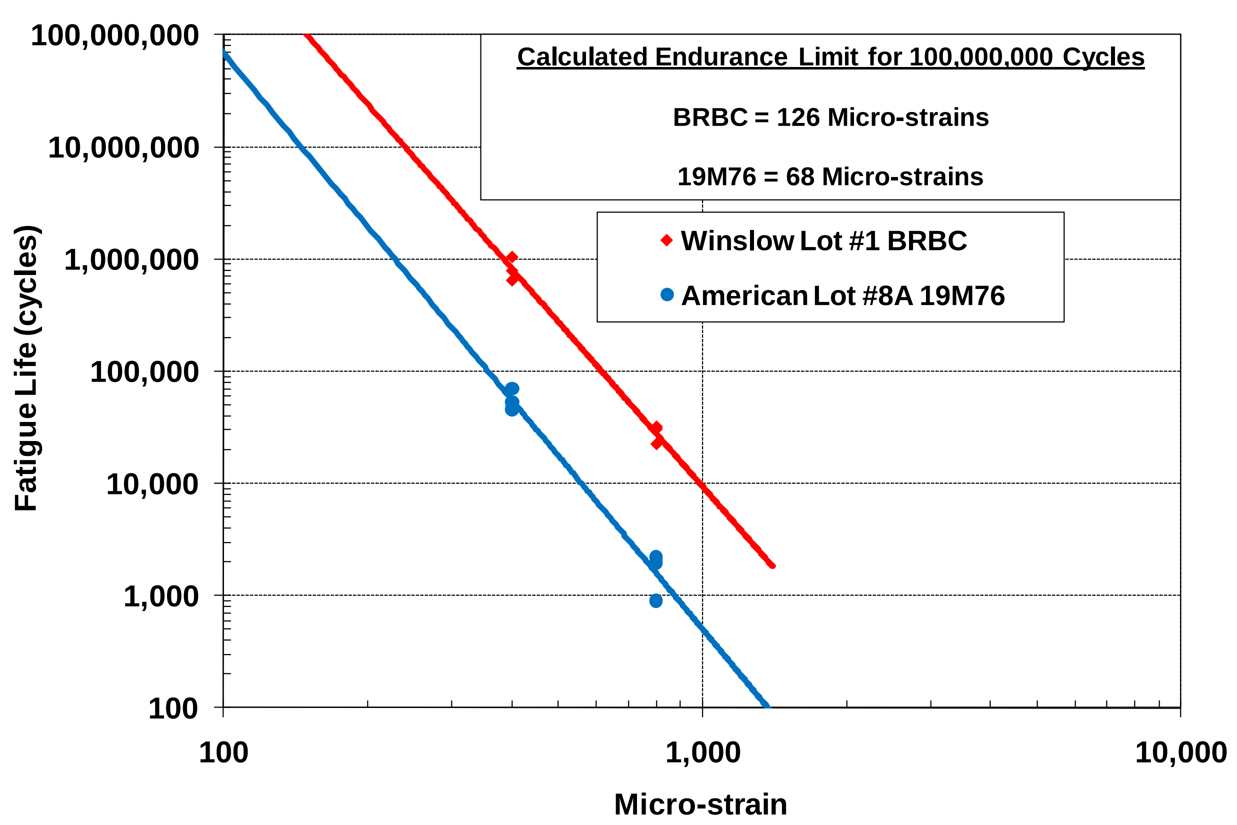
Endurance Limit – 19L64



Endurance Limit – 19M76



Endurance Limit -19M76 vs BRBC



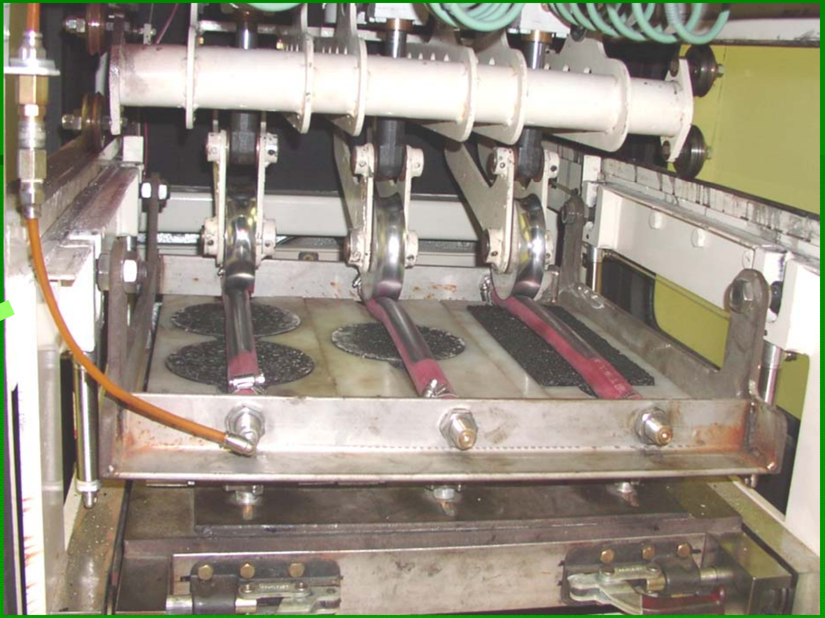
BRBC Specification

Table 902.07.03-1 BRBC Grading of Total Aggregate		
Sieve Size	**Percent Passing by Mass**	
	minimum	**maximum**
1"	100	--
¾"	90	100
½"	--	90
#8	23	49
#200	2.0	8.0
Minimum Percent Asphalt Binder by Mass of Total Mix	5.0	

Table 902.07.03-2 Volumetric Requirements for Design and Control of BRBC					
	Required Density (% of Max Sp. Gr.) @ N_{des} (50 gyrations)	**Voids Filled with Asphalt (VFA)**	**Voids in Mineral Aggregate (VMA)**	**Dust to Binder Ratio**	**Draindown AASHTO T 305**
Design Requirements	96.5	70 - 80	≥ 13.5 %	0.6 – 1.2	≤ 0.1 %
Control Requirements	95.5 – 97.5	70 - 80	≥ 13.5 %	0.6 – 1.3	≤ 0.1 %

Table 902.07.03-3 Performance Testing Requirements for BRBC	
Test	**Requirement**
Asphalt Pavement Analyzer (AASHTO TP 63)	< 5 mm@ 8,000 loading cycles
Flexural Fatigue Life of HMA (AASHTO T 321)	> 100,000,000 cycles@ 100 microstrains

Asphalt Pavement Analyzer



- AASHTO TP 63
- 100 lb wheel load; 100 psi hose pressure
- Tested at 64°C for 8,000 loading cycles

BRBC Specification

- No RAP
- No natural sand
- Binder
  - PG76-28 (NJDOT Spec)
  - RTFO Elastic Recovery > 60% @ 25°C (AASHTO T301)
- Performance Specification
  - APA and Flexural Beam
    - Testing for mix design verification and control (1st Lot and every 5th Lot after)

Required BRBC Protocol

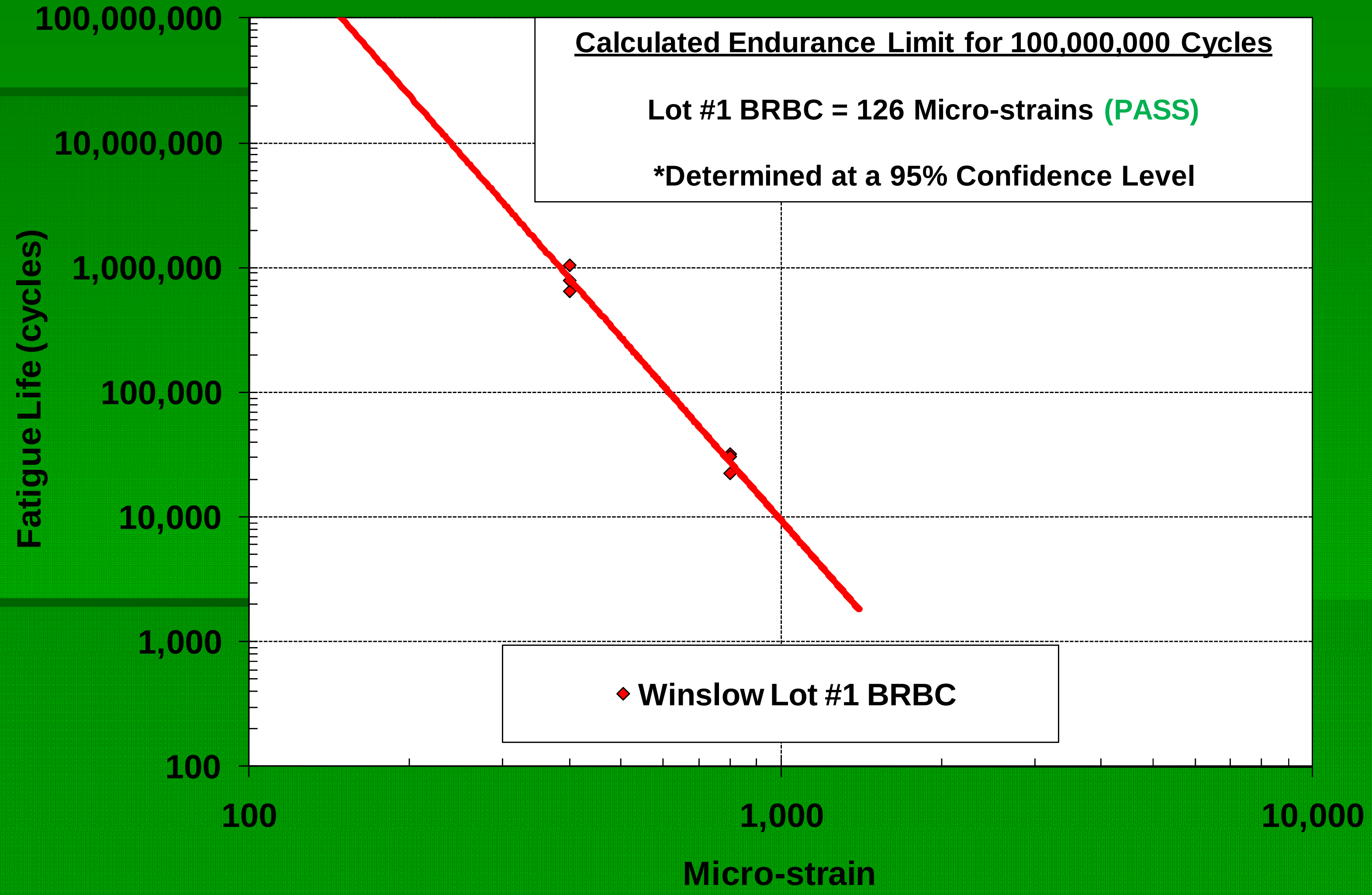
- Conduct volumetric mix design
- Supply loose mix for performance testing (fatigue and rutting)
- If pass, conduct test strip
  - Loose mix sampled and again tested (fatigue and APA)
- If pass, allowed to produce for project
  - 2 suppliers had passing designs
  - 1 supplier had failing design

General Bid Costs

- Final bid costs of BRBC equal or less than that of SMA on job

LINE NO / ITEM CODE / ALT ITEM DESCRIPTION	QUANTITY		(1) I5964 INTERCOUNTY PAVING J/V H. & UNIT PRICE	AMOUNT	(2) S6836 SOUTH STATE INC UNIT PRICE	AMOUNT	(3) C7444 CRISDEL GROUP, INC UNIT PRICE	AMOUNT
0078 408003P BOTTOM RICH BASE COURSE	177628.000	T	85.75000	15231601.00	90.90000	16146385.20	91.20000	16199673.60
0079 401099M HOT MIX ASPHALT 25 M 64 BASE COURSE	155975.000	T	54.75000	8539631.25	62.35000	9725041.25	63.40000	9888815.00
0080 401108M CORE SAMPLES, HOT MIX ASPHALT	950.000	U	72.85000	69207.50	500.00000	475000.00	112.68000	107046.00
0081 404006M STONE MATRIX ASPHALT 12.5 MM SURFACE COURSE	82228.000	T	97.75000	8037787.00	82.00000	6742696.00	97.53000	8019696.84

Endurance Limit of BRBC

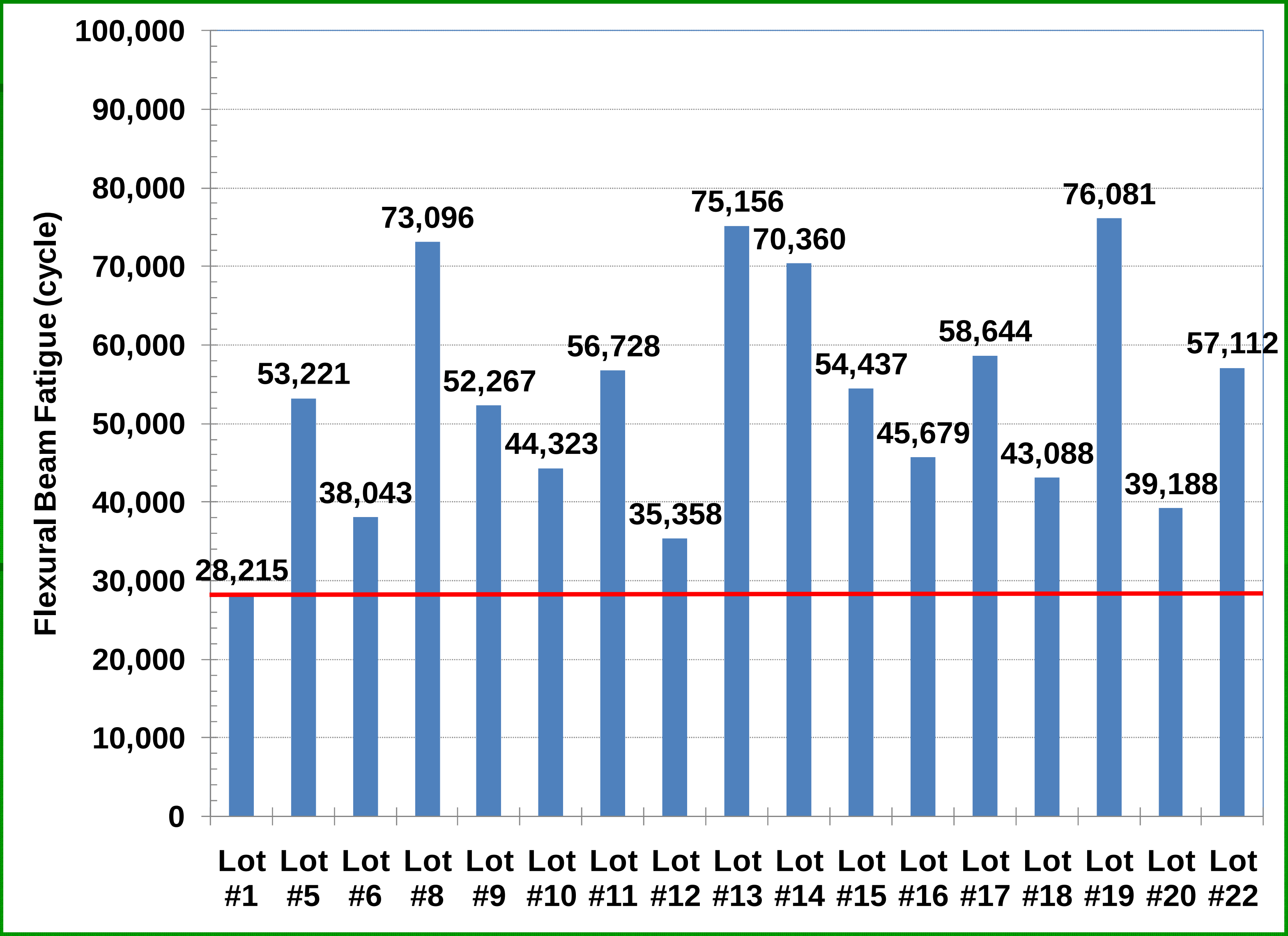


BRBC in Field

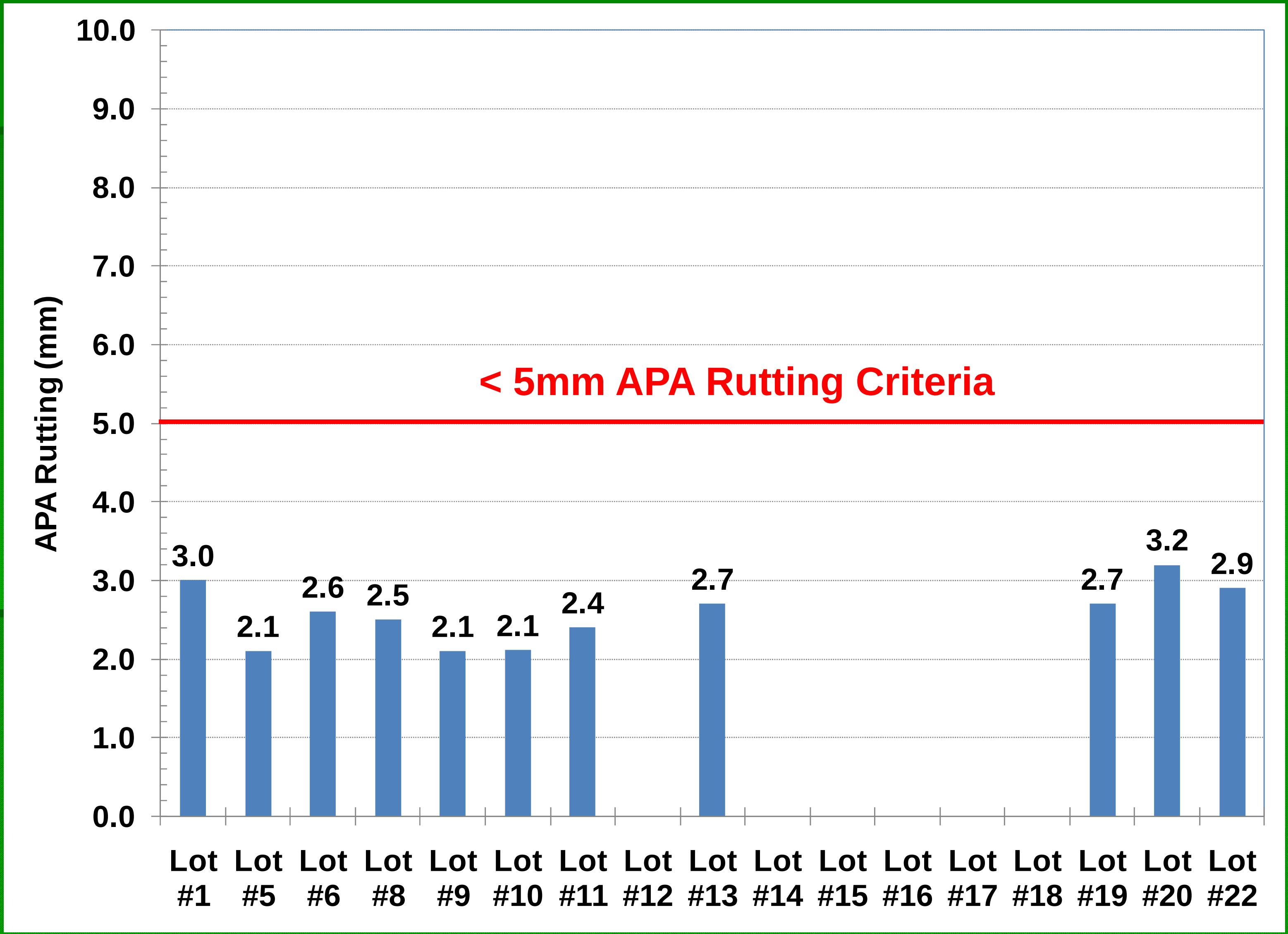
BRBC Core Sample

BRBC Core Samples

QC Test Results - Fatigue



QC Results - APA



Final Project Quantities

- As of July, total project cost: $79 million (significantly lower than engineers estimate)
  - BRBC saved 170,000 tons of HMA
  - Reduced PCCP removal and replacement by 3 miles compared to 12” thickness
  - 400’ + 400’ x 20 locations = 16,000 ft
- BRBC = 177,628 T, 3” min to 5” max
- 19M76 Intermediate = 127,078 T, 3” lift
- 12.5 SMA = 82,228 T, 2” lift
- 25M64 Base = 156,000 T

Other Innovative HMA on I295

Bridge Deck Wearing Course (BDWC)

CAIT
RUTGERS

NEAUPG
Northeast Asphalt User/Producer Group

Water Proof Wearing Course Mix

- Mix designed to provide a thin, rut and fatigue resistance mixture for bridge deck overlays
- Can be placed on bridge deck without vibratory
- Asphalt mixture must also be “water proof” or low permeability
- “Sealing older bridge structures”

Water-Proof Wearing Course

- Mix design specifications
  - N_{design} = 50 gyrations
  - Air voids @ N_{design} = 1%
    - Low permeability!
  - Requires a highly polymer-modified binder (but no binder grade specified)
    - SemMaterials Product: 76-BD
    - Rosphalt 50
  - No natural sands – stone or manufactured sands
  - Mixture performance of mix design will dictate acceptance

Water Proof Wearing Course - Specifications

Table 555.02.01-1 Job Mix Formula Requirements for BDWSC	
Sieve Size	**Percent Passing by Mass**
½"	100
3/8"	80-90
#4	55-85
#8	32-42
#16	20-30
#30	12-22
#50	7-16
#100	3-12
#200	2.0-6.0
Minimum Percent Asphalt Binder by Mass of Total Mix	7.0

Water Proof Wearing Course - Specifications

Table 555.02.01-2 Volumetric Requirements for Design and Control of BDWSC					
	Required Density (% of Max Sp. Gr.)	**Voids Filled with Asphalt**	**Voids in Mineral Aggregate**	**Dust to Binder Ratio**	**Draindown AASHTO T 305**
	N_{des} (50 gyrations)	**(VFA)**	**(VMA)**		
Design Requirements	99	90 - 100	≥ 18.0 %	0.3 – 0.9	≤ 0.1 %
Control Requirements	98 - 100	90 - 100	≥ 18.0 %	0.3 – 0.9	≤ 0.1 %

Table 555.02.01-3 Performance Testing Requirements for BDWSC	
Test	**Requirement**
APA @ 8,000 loading cycles (AASHTO TP 63)	< 3 mm
Flexural Fatigue Life (AASHTO T 321)	> 100,000 cycles

Beam Fatigue Run at 15C, 1500 micro-strains

Water Proof Wearing Course – Design Acceptance

1. Perform volumetric design and NJDOT verification
2. Supply Rutgers University loose mix for performance testing
3. Produce mix through plant and pave test strip off site
4. Sample during production and supply Rutgers University loose mix for performance testing

1st Project – Rt 87 Absecon Inlet Bridge

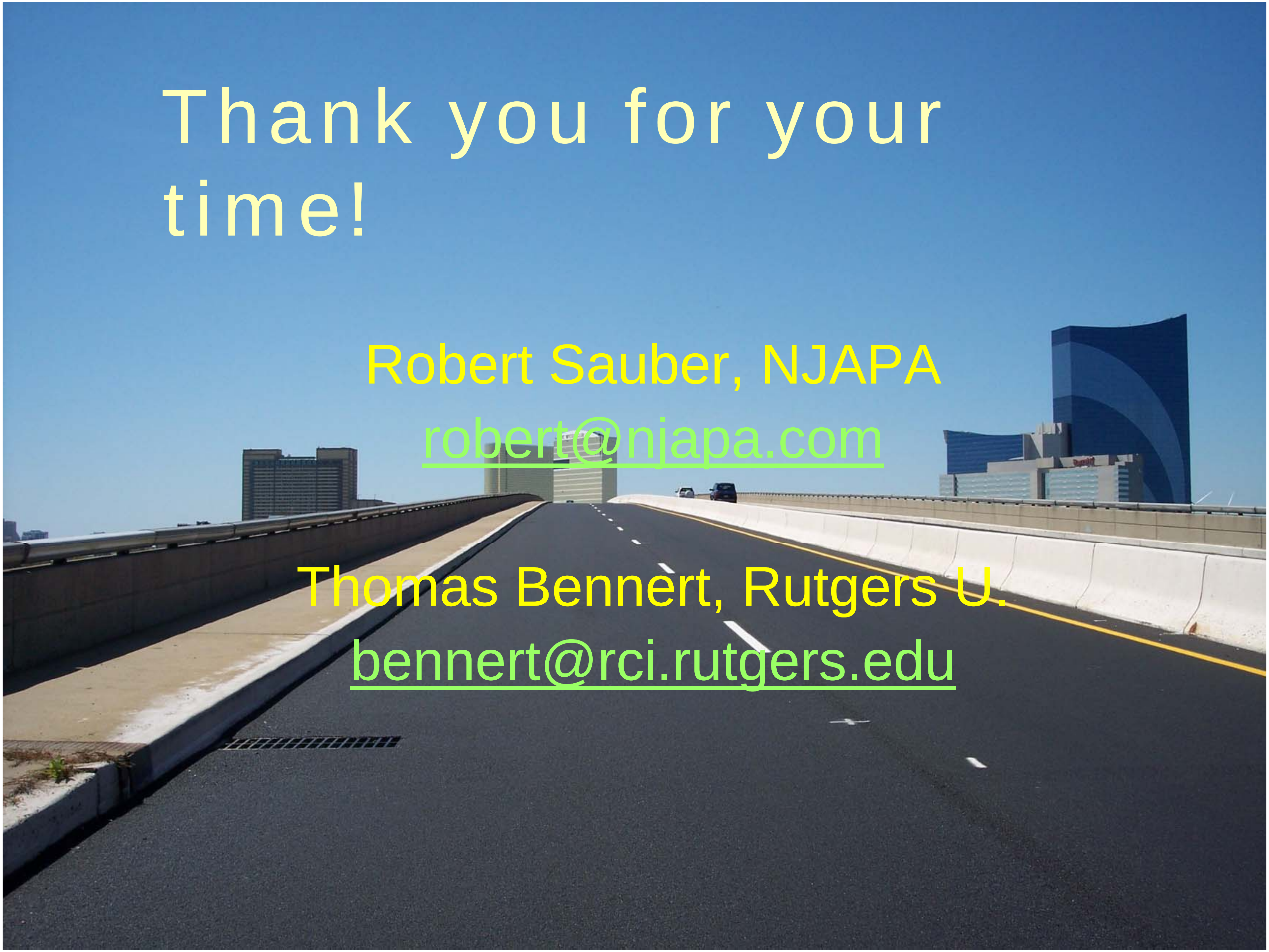
- A.E. Stone produced first BDWC mix
- 1900 tons placed and compacted to a 2-inch thickness in 2 days
- Core densities all between 2 to 4% air voids

Rt 87 Absecon Inlet Bridge – 2008 NAPA Quality in Construction Award Winner!

for Non-Typical Asphalt Project

Summary

- NJDOT utilized a performance-based approach to design and build a “perpetual pavement” out of an aging I295 PCC pavement
- Consisted of developing BRBC mix – but saved NJDOT almost $6M
  - Performance testing required for acceptance
- BDWC used to “preserve” bridge decks



Thank you for your time!

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Laboratory Characterization of a NJDOT Warm Mix Asphalt Project Rt 184

Submitted to:

New Jersey Department of Transportation (NJDOT)
Bureau of Materials

Conducted by:

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Abstract

An asphalt paving project was conducted using the NJDOT provisional specification of Warm Mix Asphalt (WMA). In accordance with the specification, the asphalt supplier must provide compacted HMA and WMA test specimens for mixture performance testing. The testing matrix required is shown below in Table 1.

Table 1 – Test Procedure and Specimen Requirements for NJDOT WMA Implementation Projects

Performance Tests for HMA Control						
Type of Test	Test Method	Pavement Distress	Test Specimen Air Voids	Compacted Specimen Height (mm)	Number of Test Specimens	Test Temperature
AMPT E*	AASTHO TP 79	Rutting Susceptibility	6.5 ± 0.5 %	170 [1]	2	129ºF (54ºC)
Asphalt Pavement Analyzer (APA)	AASTHO TP 63	Rutting Susceptibility	6.5 ± 0.5 %	170	2	147ºF (64ºC)
Hamburg Wheel Tracking	AASTHO T 324	Moisture Damage	6.5 ± 0.5 %	170	2	122ºF (50ºC)
Tensile Strength Ratio (TSR)	AASTHO T 283	Moisture Damage	6.5 ± 0.5 %	95	4	77ºF (25ºC)
Overlay Tester	NJDOT B-10	Fatigue Cracking Potential	6.5 ± 0.5 %	170[3]	2	77ºF (25ºC)

[1] Final Cut and trimmed test specimens. Lab compacted specimens should be approximately 1.0% higher.
[2] Three specimens of 170 mm height may be used instead of the required 6 specimens of 77 mm height.
[3] Four specimens of 115 mm height may be used instead of the required 2 specimens at 170 mm height.

Along with the test specimens, the asphalt supplier was to provide plant production information pertaining to the production of the WMA. This report provides a summary of the test results regarding the HMA and WMA mixtures produced for the Rt 184 paving project.

A copy of the NJDOT WMA Implementation Project specification can be found in the Appendix of the report.

Plant Production Data

The WMA was produced using a Gencor Green Machine asphalt foaming system. Plant production parameters during the WMA production were recorded as follows:

- Burner Set Point = 270^{o}F
- Discharge Temperature = 265 to 275^{o}F
- Production Rate = 275 to 300 tons per hour
- Silo Storage Time: < 2 hours

During production, aggregate moisture contents were determined as follows:

- #8 Stone = 1.9%
- #10 Stone = 4.0%

- Sand = 5.2%
- RAP = 4.1%

Three different asphalt mixtures were produced and placed on Rt 184; 1) Normal hot mix asphalt (HMA), 2) Warm Mix Asphalt (WMA) with no anti-strip, and 3) Warm Mix Asphalt (WMA) with anti-strip. Each of the mixtures were evaluated under the testing protocol shown in Table 1.

Dynamic Modulus (E*) – Mixture Stiffness

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^{o}C using loading frequencies of 25, 10, 5, 1, 0.5, 0.1, and 0.01 Hz.

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^{o}C.

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

where:

$\left|E^*\right|$ = dynamic modulus, psi

ω_r = reduced frequency, Hz

Max = limiting maximum modulus, psi

δ, β, and γ = fitting parameters

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

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)

The resultant Master stiffness curves for the HMA and WMA produced on Rt 184 is shown in Figure 2. As expected, the stiffness of the two WMA mixtures was lower than the HMA, especially at the higher test temperatures, represented in Figure 2 as the lower loading frequencies. The test results in Figure 2 also indicate that minimal differences are found at the higher loading frequencies, which represents colder testing temperatures.

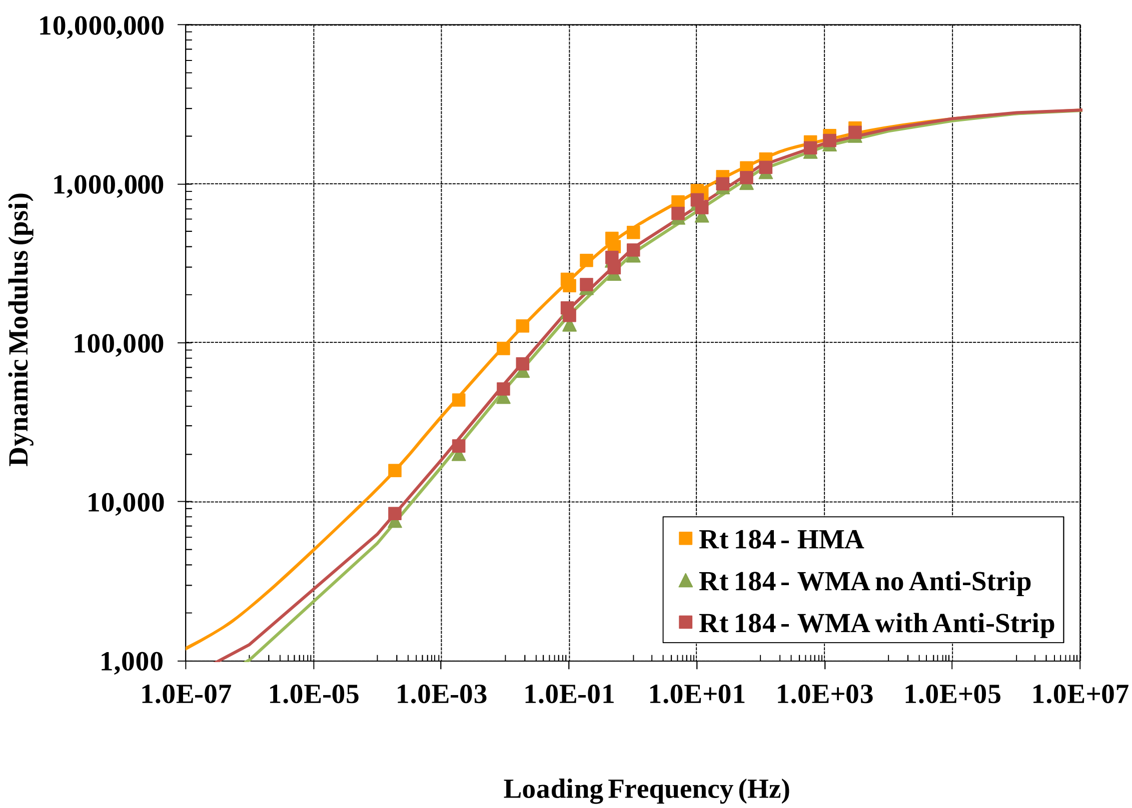

Figure 2 – Master Stiffness Curves of HMA and WMA Produced on Rt 184

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).

Asphalt Pavement Analyzer (APA)

Compacted asphalt mixtures were testing were their rutting potential using the Asphalt Pavement Analyzer (APA) in accordance with AASHTO TP63, *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°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 test results for the APA testing are shown as Figure 3. The results show that the HMA material had a lower APA rutting than both of the WMA mixtures.

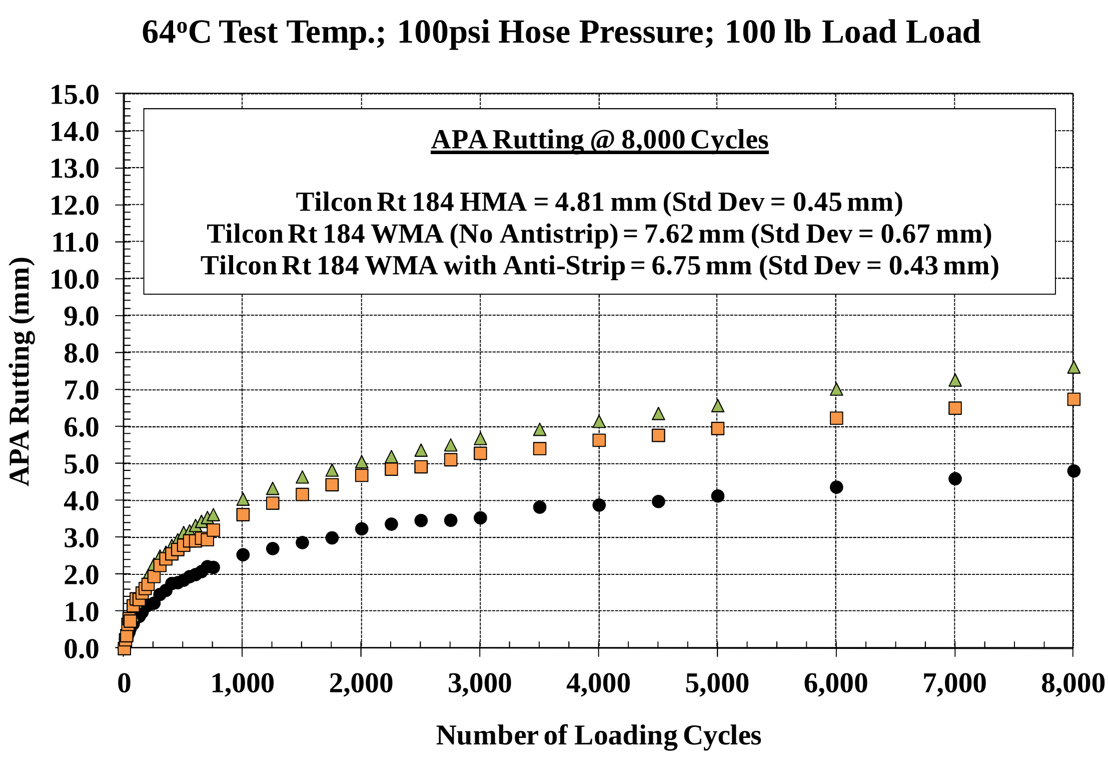

Figure 3 – Asphalt Pavement Analyzer (APA) Test Results for Rt 184 HMA and WMA

Repeated Load – Flow Number Test

Repeated Load permanent deformation testing was measured and collected in uniaxial compression using the Asphalt Mixture Performance Tester (AMPT) 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 9-43. Testing was conducted until a permanent vertical strain of 5% or 10,000 cycles was obtained.

The test results from the Flow Number test show the same trend as both the APA tests, as well as the high temperature stiffness measured during the Dynamic Modulus test.

Table 2 – Repeated Load (Flow Number) Testing Summary

AMPT Flow Number (AASHTO TP79) 54^{o}C, 600 kPa Deviatoric Stress (NCHRP 9-43 Specs)			
Mix Type	**Sample ID**	**Flow Number (cycles)**	**Cycles to Achieve 5% Strain**
HMA	#1	131	409
	#2	168	509
	Average	***150***	***459***
WMA - No Antistrip	#1	66	202
	#2	66	216
	Average	***66***	***209***
WMA + Antistrip	#1	95	355
	#2	56	190
	Average	***76***	***273***

Under NCHRP Projects 9-33 and 9-43, tentative criteria were established that recommended minimum Flow Number values for minimum ESAL levels. Tables 3 and 4 contain these values, respectively. Based on the proposed criteria from the NCHRP research, both mixtures would be appropriate for pavements of less than 10 million ESAL's.

Table 3 – 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

Table 4 – Recommended Flow Number vs ESAL Level for WMA

Traffic Level, Million ESAL's	Minimum Flow Number
<3	N.A.
3 to < 10	30
10 to < 30	105
≥ 30	415

Overlay Tester (TxDOT Tex-248-F) – Fatigue Cracking Evaluation

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). 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 are shown in Figure 4. The test results clearly show that the HMA mixture had a significantly lower fatigue life than the two WMA mixtures. Rutgers University has found that this is typical of WMA mixtures as the reduction in production temperature lowers the oxidation of the virgin binder, as well as reducing the absorption of the asphalt binder, increasing the effective asphalt content of the mix.

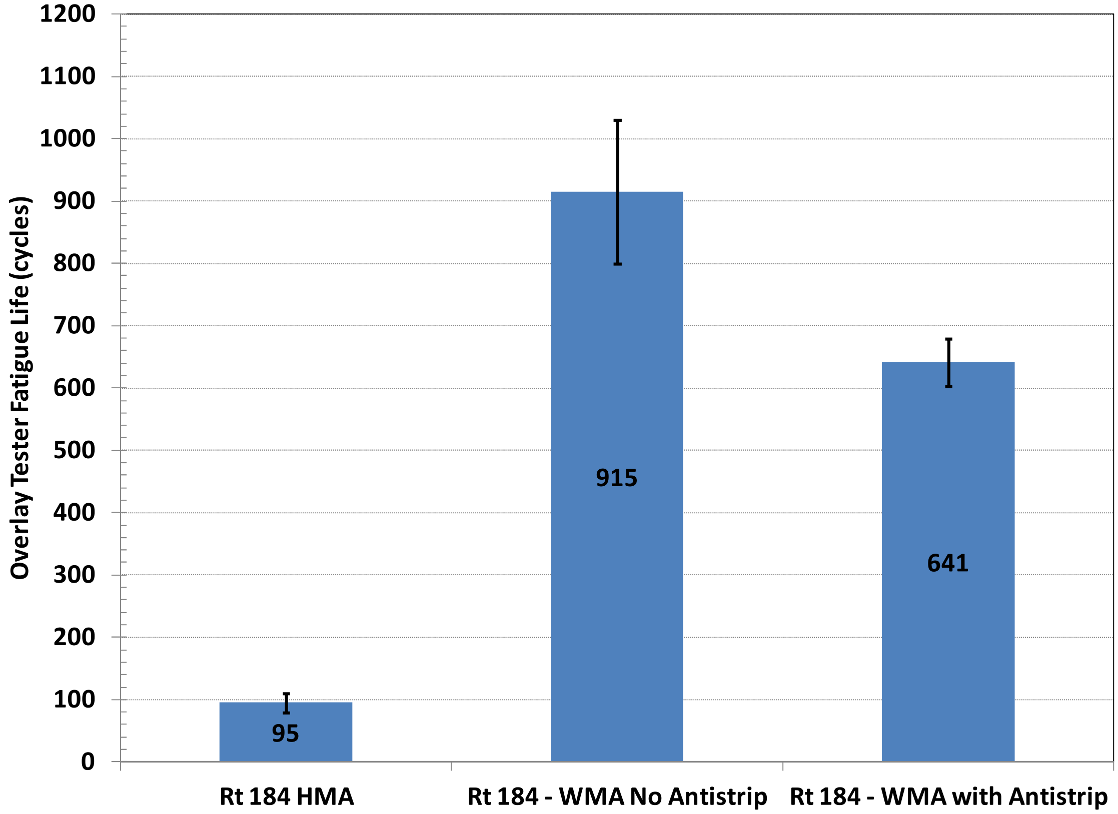

Figure 4 – Overlay Tester Results for NJ Rt 184 HMA and WMA Mixes

Resistance to Moisture-Induced Damage

The resistance to moisture damage was evaluated using both the tensile strength ratio (TSR) test procedure and the wet Hamburg Wheel Tracking Test (AASHTO T324). The test procedures and results are discussed below.

Tensile Strength Ratio, TSR (AASHTO T283)

Tensile strengths of dry and conditioned asphalt samples were measured in accordance with AASHTO T283, *Resistance of Compacted Asphalt Mixtures to Moisture Induced Damage*. The TSR values and IDT strengths are shown in Table 5. The results show that all three mixtures resulted in very similar TSR values with the WMA with anti-strip mixture having a slightly higher TSR value.

Table 5 – Tensile Strength Ratio (TSR) Values for HMA and WMA

Tilcon Keasby, Rt 184 - HMA			
Specimen Type	Indirect Tensile Strength		Average TSR (%)
	Dry	Conditioned	
AASHTO T283 Conditioned	69.0	55.5	***86.6%***
	57.6	54.1	
	63.3	***54.8***	

Tilcon Keasby, Rt 184 - WMA without Anti-Strip			
Specimen Type	Indirect Tensile Strength		Average TSR (%)
	Dry	Conditioned	
AASHTO T283 Conditioned	59.4	51.0	***87.1%***
	60.5	53.4	
	59.9	***52.2***	

Tilcon Keasby, Rt 184 - WMA with Anti-Strip			
Specimen Type	Indirect Tensile Strength		Average TSR (%)
	Dry	Conditioned	
AASHTO T283 Conditioned	49.8	52.4	***93.8%***
	52.5	43.5	
	51.1	***47.9***	

Wet Hamburg Wheel Track Test (AASHTO T324)

Hamburg Wheel Track tests were conducted in accordance with AASHTO T324, *Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA).* Test specimens were tested at a test temperature (water) of 50^{o}C. For comparison purposes, the number of cycles to reach 0.5 inches (12.5 mm) of rutting is commonly used for comparison purposes and for some state agency pass/fail specifications. For a PG64-22 asphalt binder, the mixtures must achieve a minimum of 10,000 cycles before achieving 0.5 inches of rutting.

The test results (Figures 5 to 7) show that the Hamburg data for the HMA would clearly pass the 10,000 passes before 12.5mm rutting. However, both WMA mixtures, even with the anti-strip added, would have failed the minimum criteria.

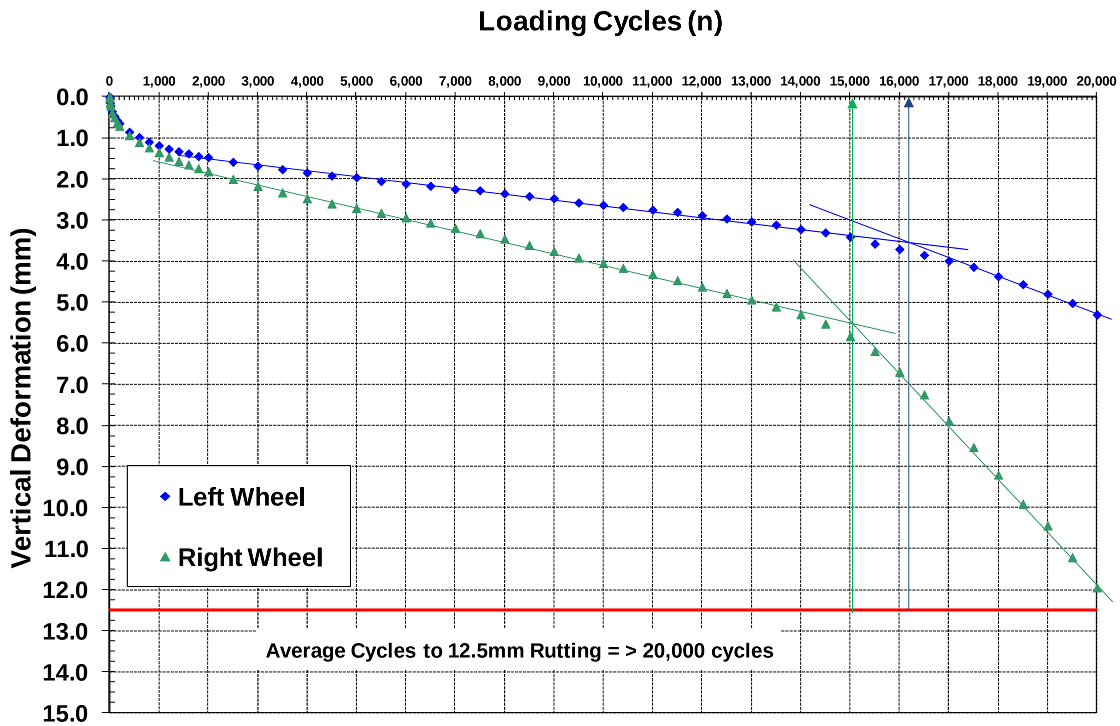

Figure 5 – Hamburg Wheel Tracking Results for Rt 184 HMA Mixture

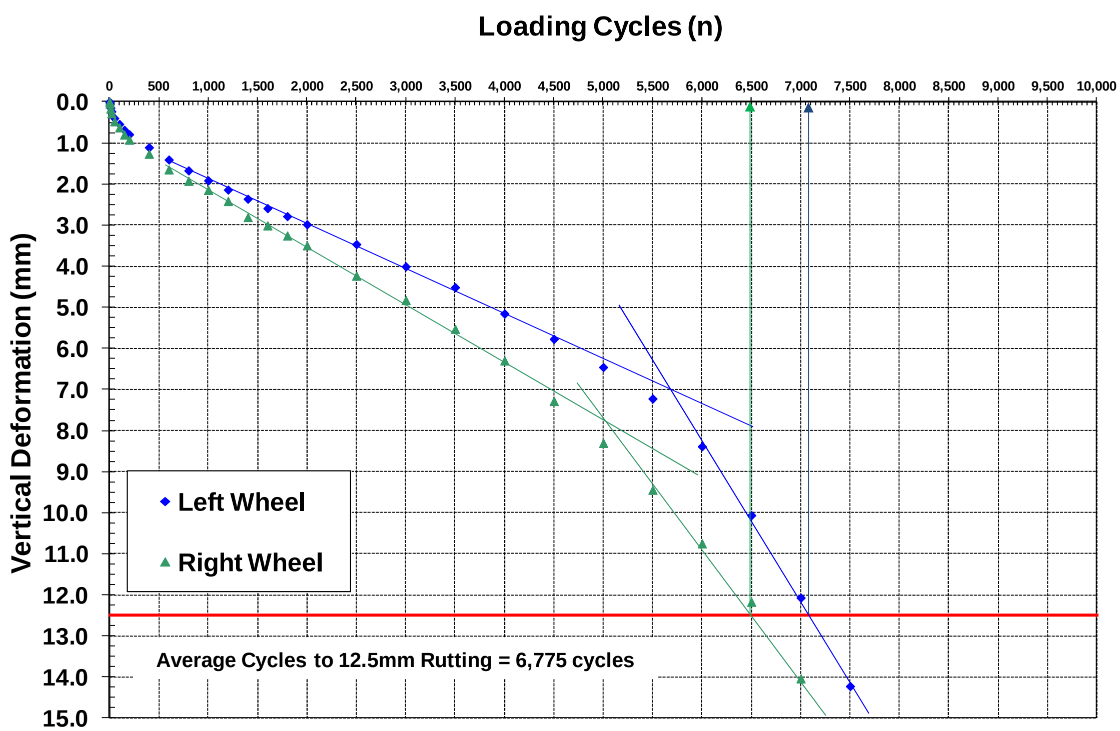

Figure 6 – Hamburg Wheel Tracking Results for Rt 184 WMA with No Anti-Strip

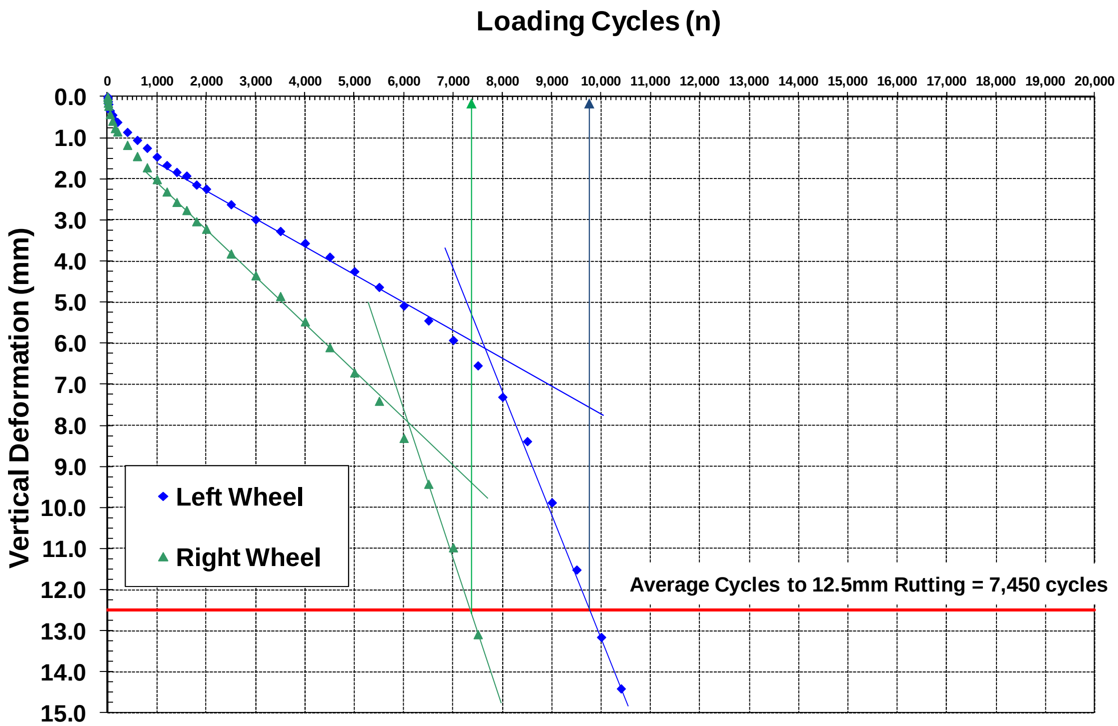

Figure 7 – Hamburg Wheel Tracking Results for Rt 184 with Anti-Strip

Conclusions

The test results from the HMA and WMA materials placed on New Jersey Rt 184 indicates:

- The WMA mixtures are not as stiff as the HMA mixture at elevated temperatures. It is hypothesized that this is due to lower levels of oxidation aging for the asphalt binder during production, as well as lower amounts of asphalt binder absorption increasing the effective asphalt content in the mix.
- However, this resulted in a slightly greater potential for rutting when comparing the mixtures in the Asphalt Pavement Analyzer and the Flow Number. Using the criteria established under NCHRP projects 9-33 and 9-43, both the HMA and WMA mixtures should be suitable for pavements with ESAL levels under 10 million ESAL's.
- Although the WMA mixtures appeared to be more susceptible to rut, they clearly achieved a significantly greater resistance to fatigue cracking when evaluated in the Overlay Tester.
- When assessing the mixtures' resistance to moisture damage potential, all three mixtures achieved tensile strength (TSR) values greater than 80%. However, the test results for the Hamburg Wheel Tracking showed a significant difference between the HMA and WMA mixtures, with the HMA mixtures performing significantly better.

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

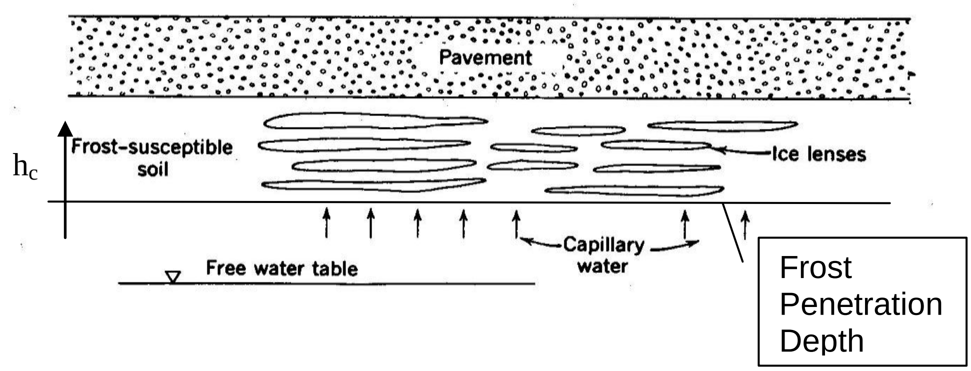


Figure 1. Mechanics of Frost Damage

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.

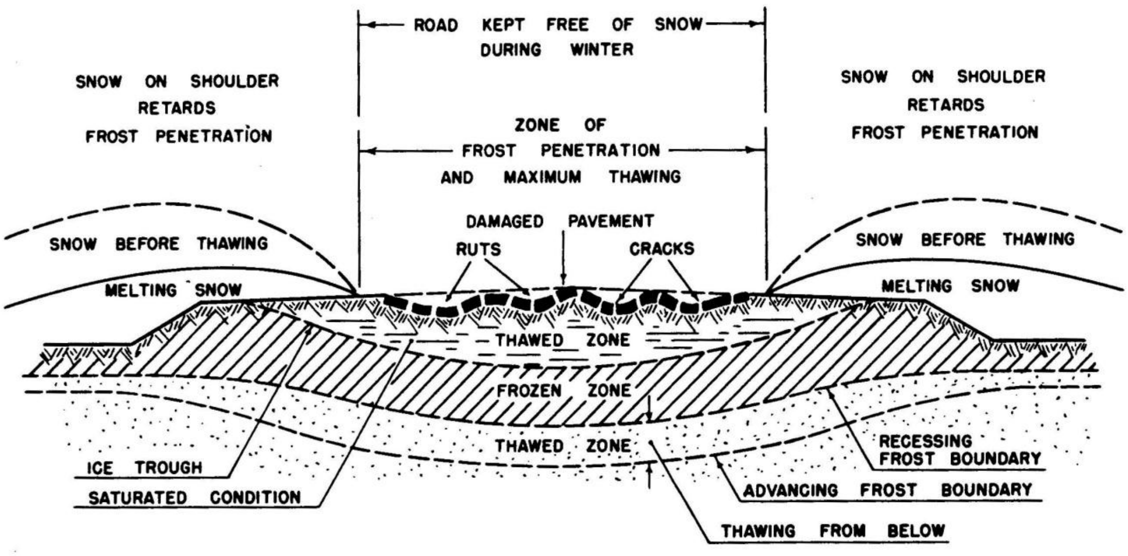


Figure 2. Frozen and Thawed Zones within and below the Pavement Structure

<u>Sources of Water</u>

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).

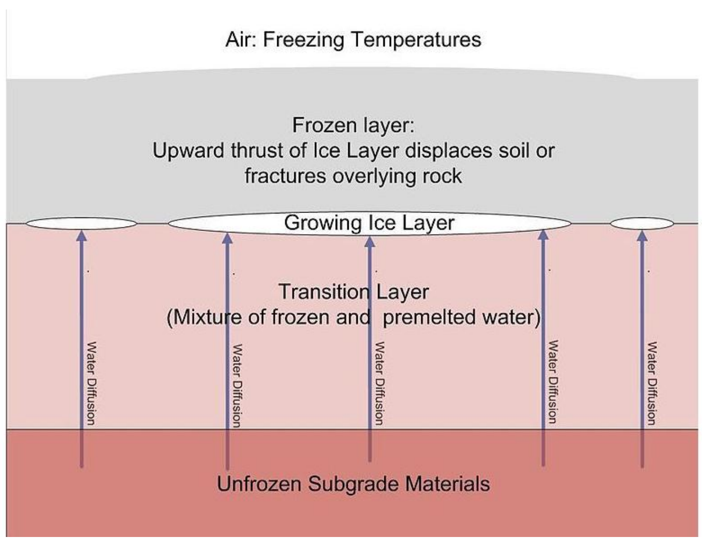


Figure 3. Frozen, Transition and Unfrozen Layers

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}.

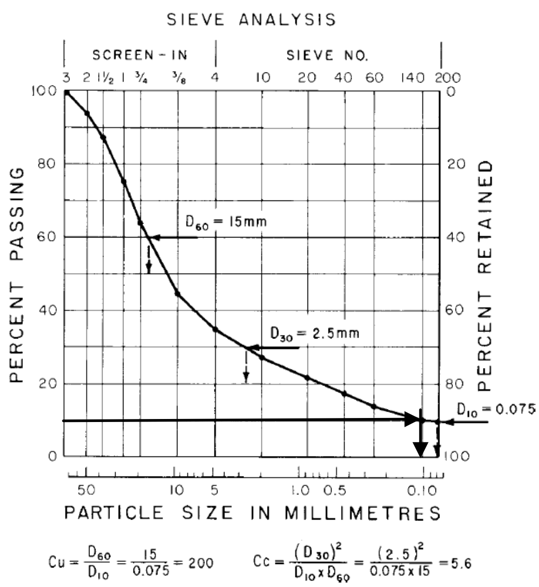


Figure 4. Particle Size Distribution

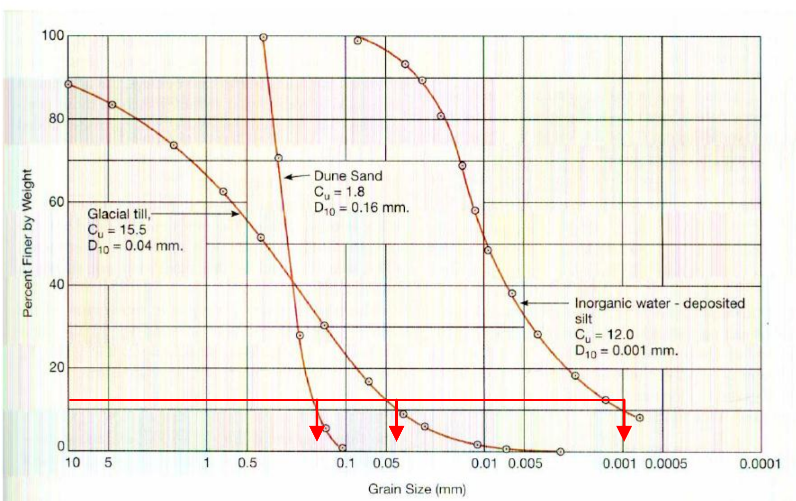


Figure 5. Example of D_{10} size for Gravel, Sand and Silt Soil Aggregates

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.

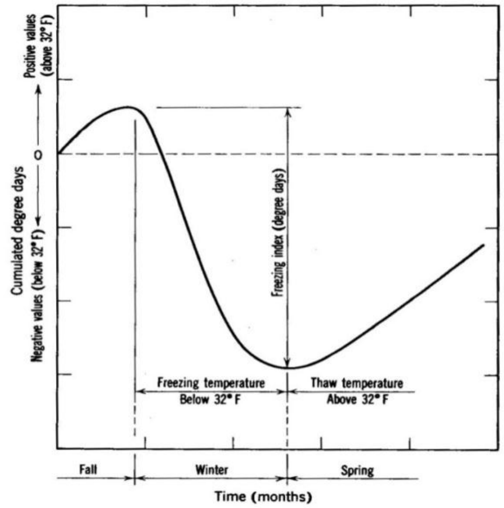


Figure 6. Cumulative Plot of Degree-Days and Frost Index

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.

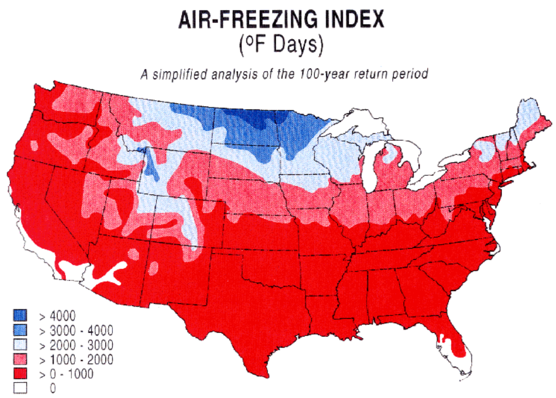


Figure 7. Map of the cumulative freezing index for the United States
Using Air freezing index to estimate Pavement Freezing index

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.

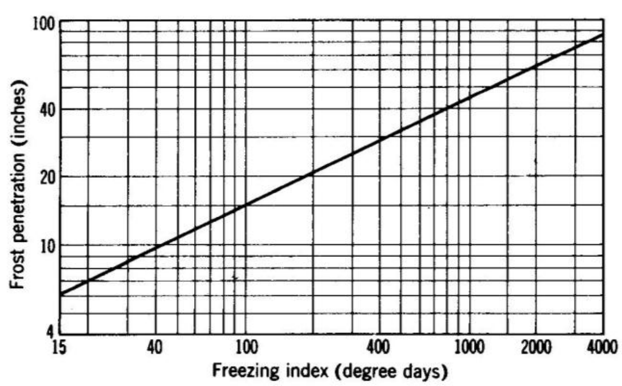


Figure 8. Estimation of Frost Penetration based on cumulative freezing index for the United States (Corps of Engineers)

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

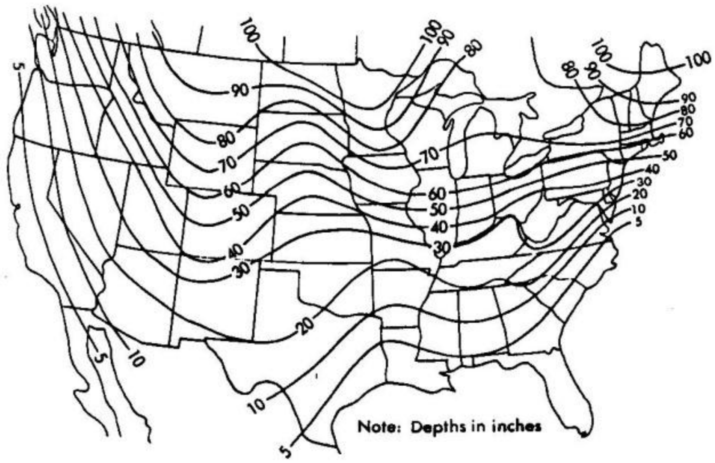


Figure 9. Estimate of Frost Penetration Depths in the United States.

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.).

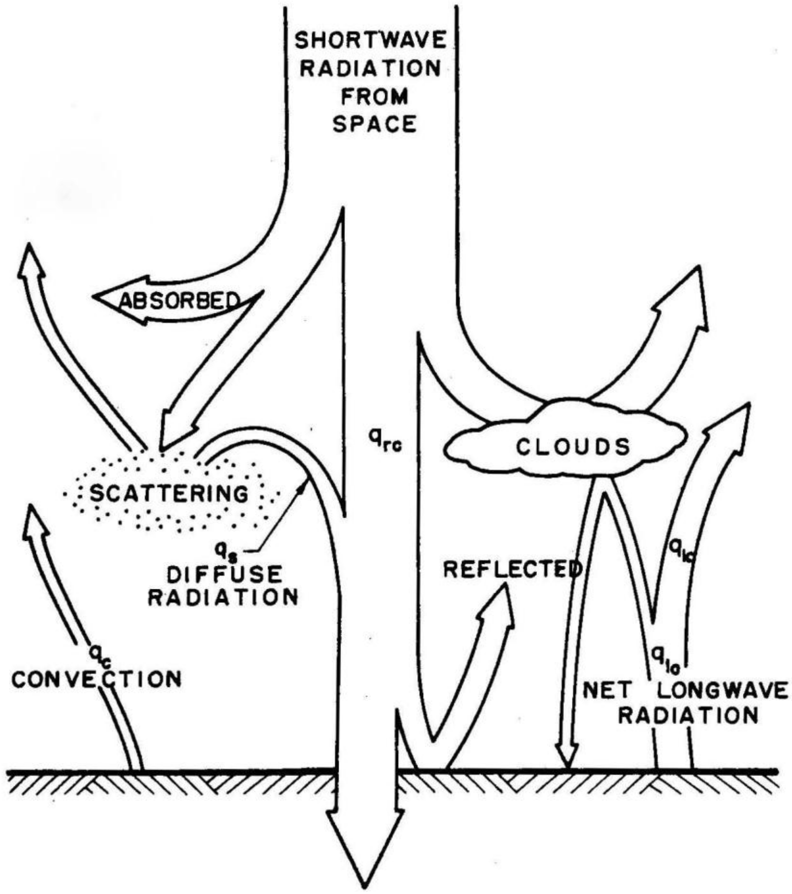


Figure 10. Illustration of the net heat flow at the pavement surface

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.

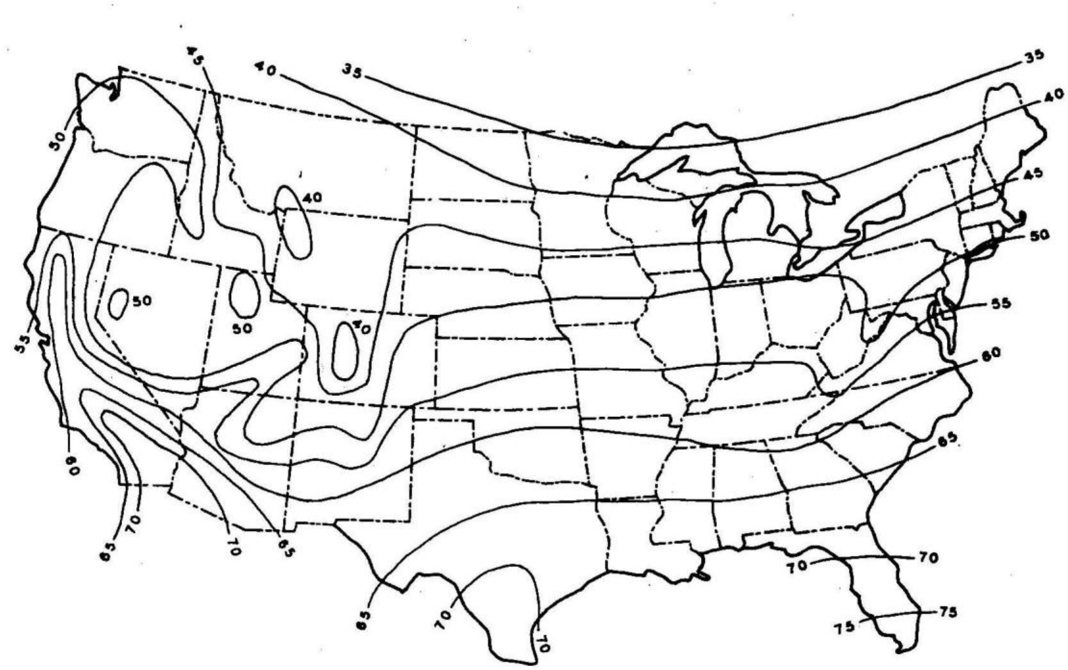


Figure 11. provides an illustration of the mean annual air temperature for NJ.

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.

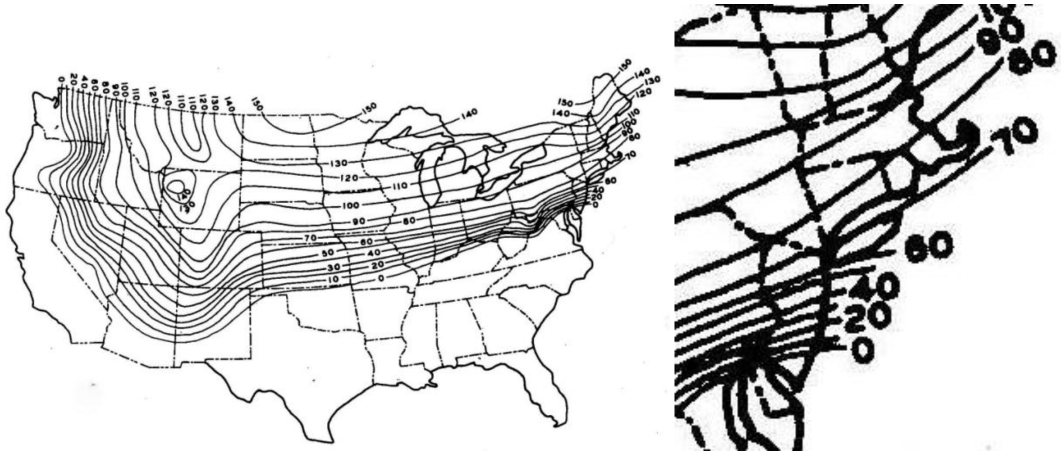


Figure 12. Map of the freezing period, t, in days for NJ.

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.

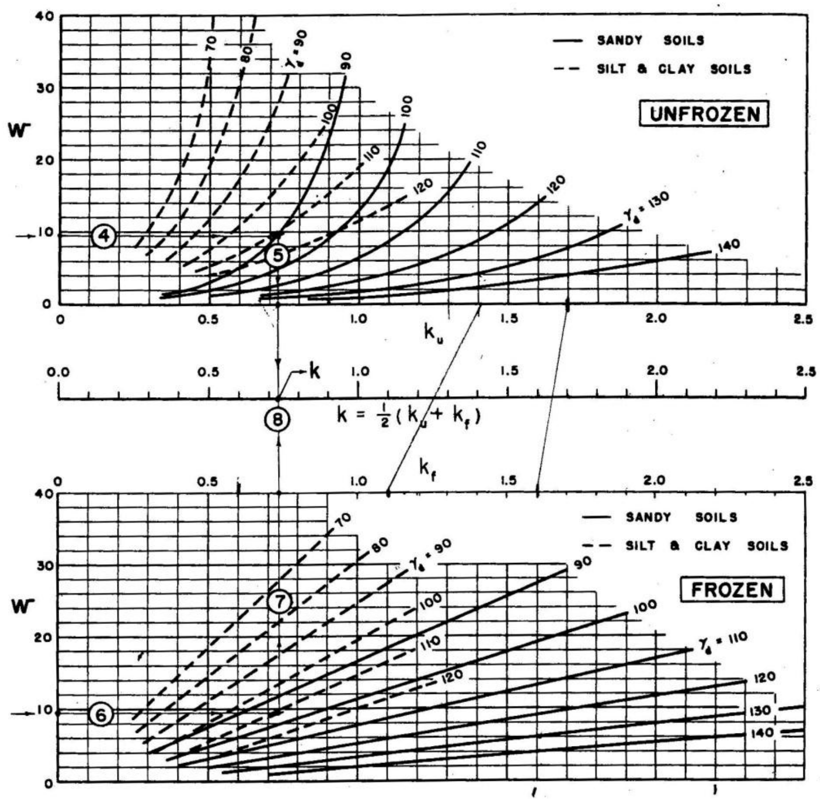


Figure 13. Thermal Conductivity of Soil Materials

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

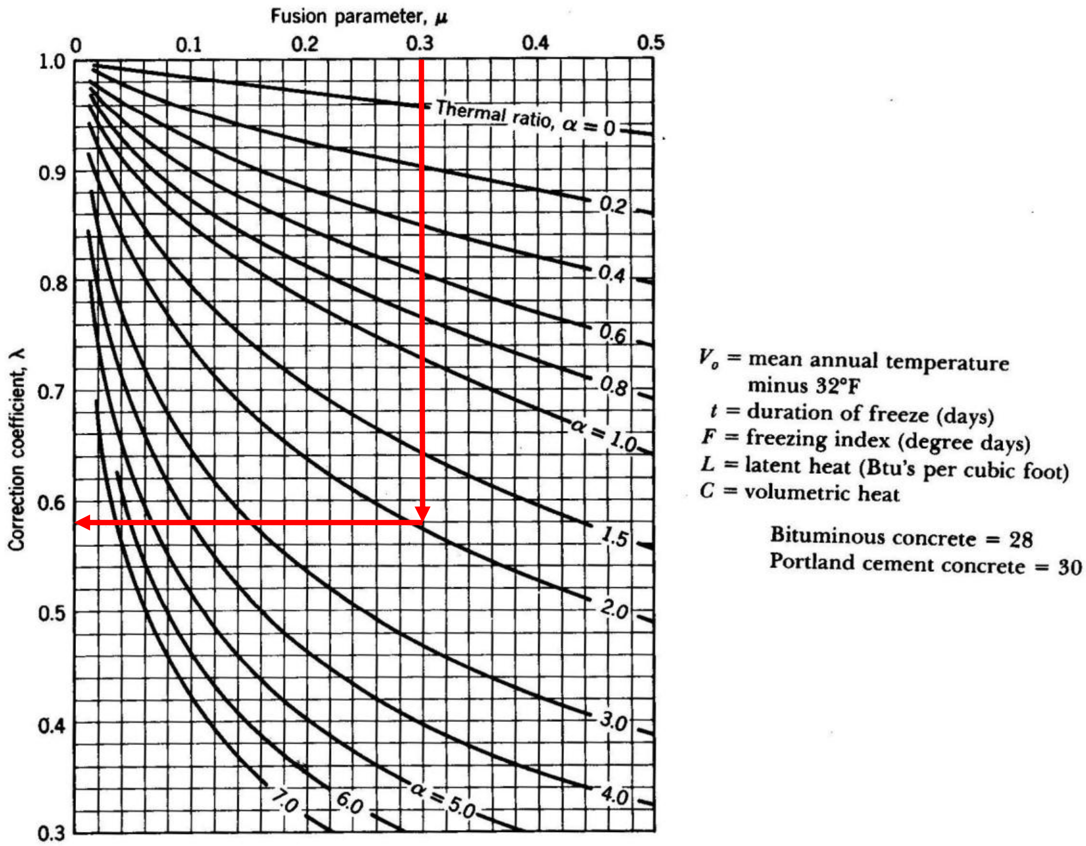


Figure 14. Correction Coefficient for the modified Berggren formula (from Aldrich, Highway Research Board Bulletin 135 Frost Penetration Below Highway and Airfield Pavements

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	

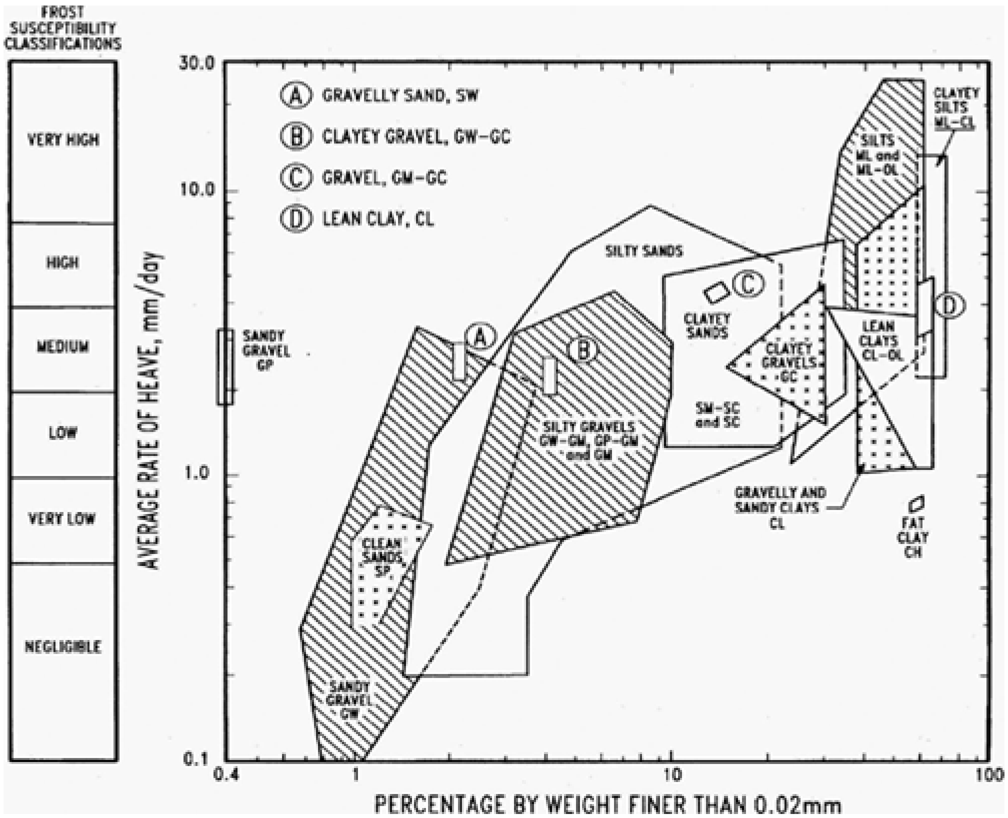


Figure 15. Graphical Identification of Frost Susceptible Soils

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.

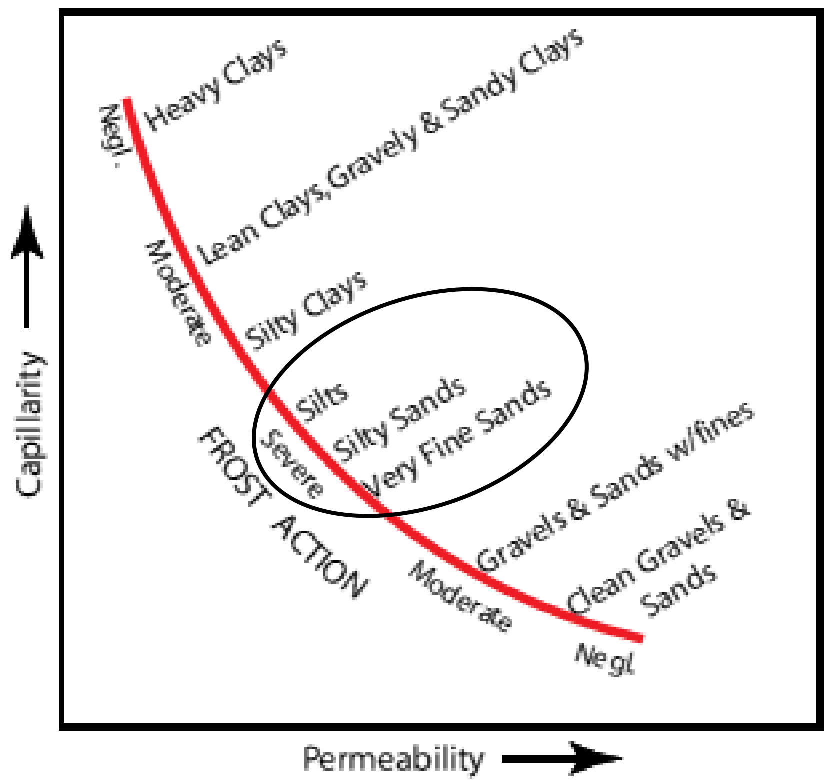


Figure 16. Relationship between Frost Action and Hydraulic Soil Properties

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 high frost hazard potential 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 low frost hazard potential 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

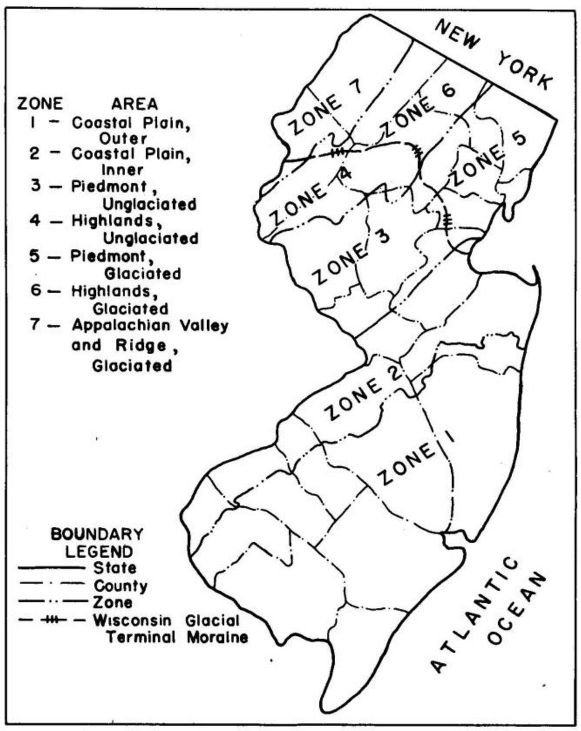
Rutgers Engineering Soil Survey Series

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.

GEOLOGY



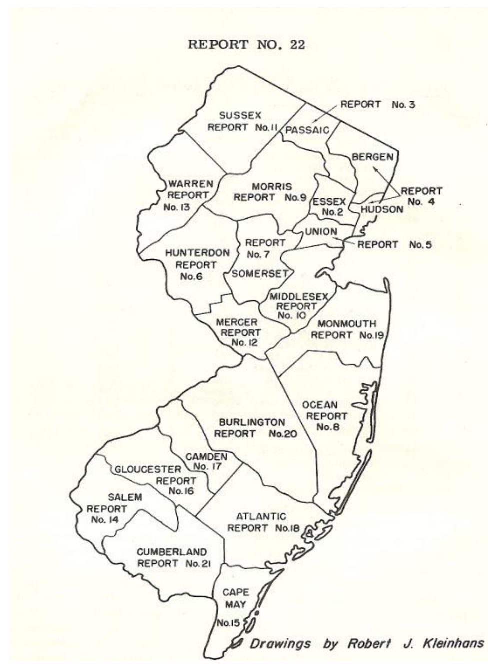


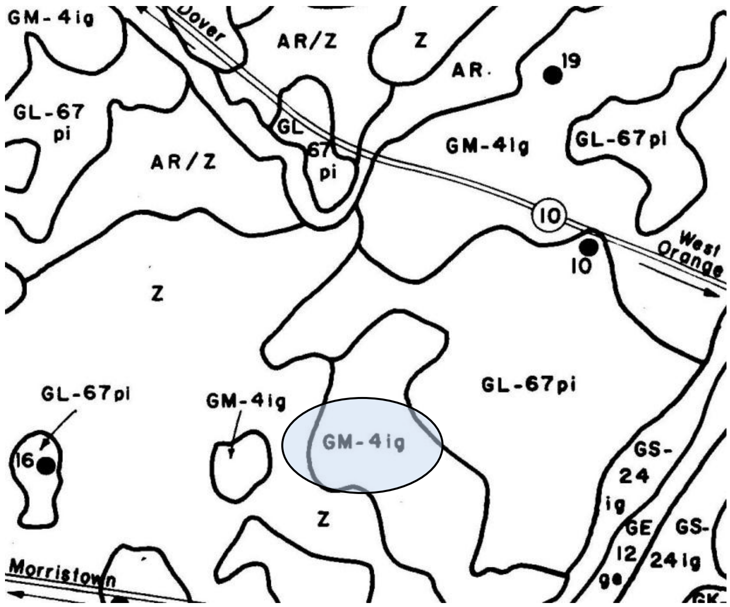
Figure 17. New Jersey's Geologic Zones and County Report Numbers

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.



Geologic – Textural (AASHTO Classification Range) DRAINAGE SPECIAL

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.

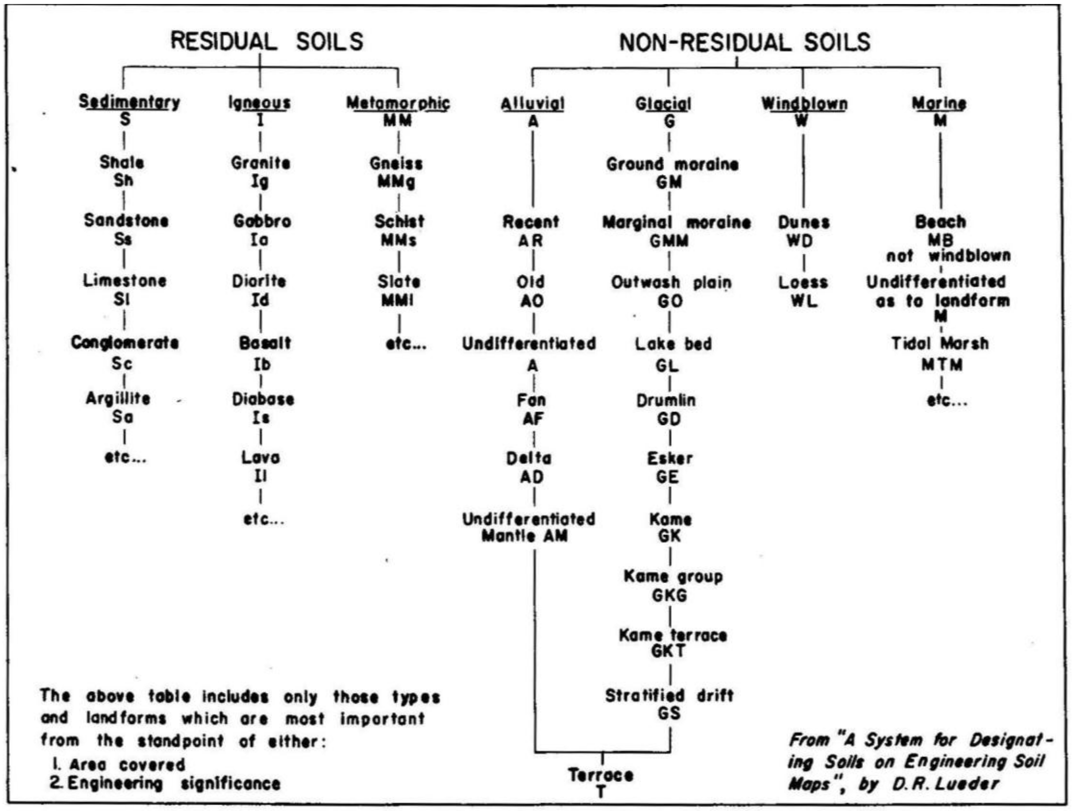


Figure 18. Geologic Notation

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

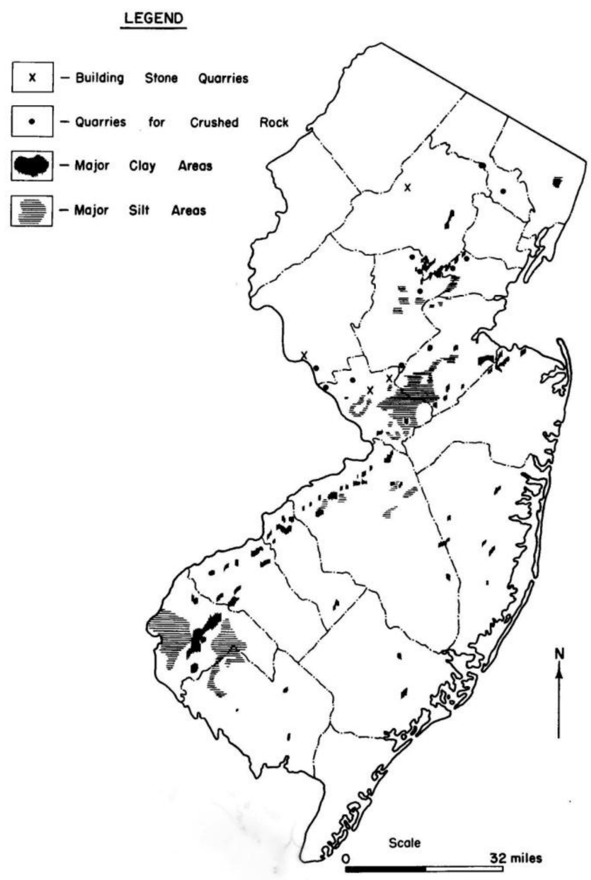


Figure 20. Illustration of locations that have large concentrations of Gravels, Sands, Silts and Clays

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).

	MORE THAN 25% PASSING 75µm			LESS THAN 25% PASSING 75µm		
Plasticity Index	PI ≤ 10	10 < PI <20	PI ≥ 20	PI ≤ 6 PI x % passing 75µm ≤ 60	PI ≤ 10	PI > 10
Form of Stabilisation						
Cement and Cementitious Blends	Usually suitable	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually not Suitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Bitumen/ Cement Blends	Usually suitable	Doubtful	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually not Suitable	Usually not Suitable	Usually suitable	Usually suitable	Doubtful
Miscellaneous Chemicals*	Usually not Suitable	Usually suitable	Usually suitable	Usually not Suitable	Doubtful	Usually suitable

Key: Usually suitable (black) — Doubtful (hatched) — Usually not Suitable (white)

* Should be taken as a broad guideline only. Refer to trade literature for further information.

Note: The above forms of stabilisation may be used in combination, e.g. lime stabilisation to dry out materials and reduce their plasticity, making them suitable for other methods of stabilisation.

Table 2.3 — Guide to Selecting a method of Stabilisation

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.