University Transportation Research Center - Region 2

Final Report

Determining Binder Flushing Causes in New York State

Performing Organization: Rutgers Unviersity

December 2014

Sponsor(s):
New York State Department of Transportation (NYSDOT)
University Transportation Research Center - Region 2

University Transportation Research Center - Region 2

The Region 2 University Transportation Research Center (UTRC) is one of ten original University Transportation Centers established in 1987 by the U.S. Congress. These Centers were established with the recognition that transportation plays a key role in the nation's economy and the quality of life of its citizens. University faculty members provide a critical link in resolving our national and regional transportation problems while training the professionals who address our transportation systems and their customers on a daily basis.

The UTRC was established in order to support research, education and the transfer of technology in the field of transportation. The theme of the Center is "Planning and Managing Regional Transportation Systems in a Changing World." Presently, under the direction of Dr. Camille Kamga, the UTRC represents USDOT Region II, including New York, New Jersey, Puerto Rico and the U.S. Virgin Islands. Functioning as a consortium of twelve major Universities throughout the region, UTRC is located at the CUNY Institute for Transportation Systems at The City College of New York, the lead institution of the consortium. The Center, through its consortium, an Agency-Industry Council and its Director and Staff, supports research, education, and technology transfer under its theme. UTRC's three main goals are:

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NYSDOT
SPR Project # C-08-15

Determining Binder Flushing Causes in New York State

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In 2007, a number of asphalt pavements in New York State flushed. An extensive forensic and laboratory investigation was conducted to determine why particular New York State asphalt pavements constructed in 2007 had undergone “atypical” flushing. Analysis of quality control records, laboratory characterization of field cores, and a laboratory mixture evaluation component were conducted to help best determine the potential reasoning for unexpected pavement flushing. At the conclusion of this study, there were no definitive reasons as to why these pavements had flushed. For every task evaluated where a potential reason was identified that may have caused the flushing issue, there were always exceptions that prohibited a conclusive answer. Therefore, although the findings in the study outline how material testing and specification can be improved in New York State to help reduce the potential for rutting/flushing in the future, the exact reasoning for the flushing in 2007 is still unknown.</td></tr>
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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

EXECUTIVE SUMMARY

During the 2007 hot mix asphalt (HMA) paving season, several newly placed HMA pavements in New York State showed signs of pavement flushing. Initial investigations of the flushing showed it was not attributable to the factors associated with conventional flushing. While conventional flushing is typically attributable to mix factors such as excess fines, excess asphalt binder, low air voids or to construction factors, such as contamination or allowing traffic on the pavement before the HMA has had time to set up, NYSDOT defined the condition as "Atypical" flushing. "Atypical" flushing describes a pavement that has flushed without any known reason.

An extensive forensic testing program was conducted in an effort to determine how and why the asphalt pavement flushed. A field visit of the flushed locations occurred prior to any testing to get a better understanding of the pavement condition and location with respect to traffic. A thorough evaluation of quality control records and test data provided by the NYSDOT was also conducted. Field cores were taken from a number of pavement sections for laboratory characterization and chemical analysis. Additionally, raw aggregates were sampled from five different quarries/asphalt plants that had provided hot mix asphalt to the flushed locations. Asphalt binders were modified with two different dosage rates of polyphosphoric acid (PPA) to evaluate whether or not the PPA had an adverse reaction to the limestone aggregate sources in some of the asphalt mixtures evaluated.

The field visit showed that the flushing was only located in areas where traffic speeds were very slow/stopped due to intersections with stop signs and/or traffic lights. During the field visit, flushing was not observed further in front, or behind, the intersection areas where traffic would have been moving at faster speeds (i.e. > 40 mph). The analysis of quality control records indicated that some of the mixtures produced and placed on the flushed pavement sections were significantly finer than their respective job mix formula (JMF). This would have resulted in the field mixtures with lower air voids and higher voids filled with asphalt (VFA), which would promote the potential for flushing. It was also determined that the high temperature PG specified for the asphalt mixtures used in some of the flushed areas may not have been appropriate based on the respective traffic conditions for the roadway. In some of the flushed pavement areas where traffic levels were of 10 million ESAL's or greater, a high temperature PG grade of 70 or higher should have been specified based on the recommendations of LTPPBind 3.1.

Solvent extraction and recovery of the asphalt binder from the field cores indicated that tri-chlorethylene (TCE) source may play a role in some of the variability noticed in the high temperature PG grade. Non-research grade TCE sources generally contain "acid-scavengers" to help reduce corrosion of metal shipping containers. These "acid-scavengers" may pull out PPA from the modified asphalt binder during the recovery process, thereby making the asphalt binder appear softer. Solvent extractions and recoveries with research grade TCE, as well as Toluene, showed acceptable high temperature PG grades for the cores evaluated. It was also noted that PG grading of asphalt binder extracted from field cores currently does not have a precision-bias statement attached to it. However, information gathered from the Federal Highway

Administration (FHWA) noted that a 10% range in test results for high temperature PG grading appeared appropriate based on AMRL test results. Using this 10% range, it was found that high temperature PG graded conducted by NYSDOT in 2007 and 2008, were statistically equal to the Certificate of Analysis (COA) results from the asphalt suppliers' storage tanks.

Further chemical analysis of the recovered asphalt binders indicated that perhaps not all of the assumed PPA modified asphalt binders actually contained PPA. If PPA exists in the asphalt binder, phosphorus should clearly be observed in the asphalt binder. Yet, one of the PG64-28 asphalt binders recovered and tested, supposedly containing PPA, had no traceable evidence of phosphorus. Therefore, this may indicate that not all of the flushed asphalt mixtures contained an asphalt binder modified with PPA.

The laboratory evaluation showed that asphalt binder "softening", due the interaction of PPA modified asphalt binder and limestone aggregate, could not be simulated under laboratory mixing and conditioning procedures. Dynamic modulus testing of the asphalt mixtures under various conditioning times clearly stiffened (aged). This would indicate that "softening" of the mixture at elevated temperatures was not occurring. Flow Number testing in the Asphalt Mixture Performance Tester indicated that a majority of the asphalt mixtures evaluated may not be suitable for intersection traffic conditions.

Follow-up interviews with NYSDOT Regional Engineers that have been involved in the flushed sections for the past 7 years indicated that in most of the areas that were affected and left in place, flushing is still occurring in hot weather. However, areas that were replaced with identical asphalt mixtures containing neat PG64-22 asphalt binder instead of a PPA modified PG64-28 are not undergoing the same flushing issues.

At the conclusion of this study, there were no definitive reasons as to why these pavements had flushed. For every task evaluated where a potential reason was identified that may have caused the flushing issue, there was always an exception to this reason that prohibited a conclusive answer. Therefore, although the findings in the study outline how material testing and specification can be improved in New York State to help reduce the potential for rutting/flushing in the future, the exact reasoning for the flushing in 2007 is still unknown.

CHAPTER 1 - INTRODUCTION

During the 2007 hot mix asphalt (HMA) paving season, several newly placed HMA pavements in New York State showed signs of pavement flushing. Initial investigations of the flushing by the NYSDOT indicated it was not attributable to the factors commonly associated with conventional flushing. While conventional flushing is typically attributable to mix factors such as excess fines, excess asphalt binder, low air voids or to construction factors, such as contamination or allowing traffic on the pavement before the HMA has had time to set up, NYSDOT defined the condition as "Atypical" flushing. "Atypical" flushing describes a pavement that has flushed without any know reason. Field investigations conducted by NYSDOT showed that there were several key characteristics of the "Atypical" flushing:

1. Flushing began to appear a couple of weeks after placement;
2. The pavement continued to be soft and pliable weeks after placement; and
3. The compacted mix moved, rutted, and/or shoved under traffic.

The flushed pavements fitting this "Atypical" category were limited to Central New York (Syracuse, Utica, Watertown, and the surrounding areas), and were placed from May to August 2007. Additional facts regarding the "Atypical" flushed pavements were:

- Different contractors constructed the pavements;
- Different HMA producers made the HMA mixtures;
- Different HMA mix designs were affected;
- Almost all of the HMA mixtures were produced with limestone aggregates;
- Almost all of the HMA mixtures were produced with PG64-28 binder that was modified with polyphosphoric acid (PPA);
- The PG64-28 binder was supplied by three (3) different Primary Sources;
- All retained binders were verified as a PG64 high temperature. Meanwhile, binder recovered from field cores generally tested 1 to 2 grades lower; and
- Gradations of cores taken in the wheelpath, where the "Atypical" flushing occurred, showed gradations finer than the gradations performed during production. Meanwhile, gradations from cores taken outside the wheelpath, where flushing did not occur, showed to be relatively comparable to gradations performed during production.

After the initial recognition of the flushing problem, NYSDOT directed the asphalt supplier to stop using PPA modified PG64-28 and substitute it with a "neat" PG64-22 asphalt binder without making adjustments to the mix design. The "Atypical" flushing did not appear in any of the pavements placed after the substitution. Unfortunately, the root cause of the "Atypical" flushing had yet to be determined as of 2009.

Research Need Statement

As previously described, the NYSDOT witnessed what was termed "Atypical" flushing in a number of HMA pavement sections in Central New York State, in particular the

Syracuse, Utica, and Watertown areas (Regions 2, 3, and 7 shown in Figure 1). According to NYDSOT Engineering Bulletin, EB07036;

> "In mid-August 2007, the Department began to observe a number of projects that exhibited HMA pavement flushing. On many of these projects, the flushing did not appear until several weeks after paving was completed. In addition, the pavements continued to be soft and pliable. This specific flushing was not attributable to the factors normally associated with conventional flushing. This type of flushing has been termed Atypical Flushing. Projects exhibiting Atypical Flushing were found in Regions 2, 3, and 7. At this time, the common factors associated with projects exhibiting Atypical Flushing appear to be HMA material containing limestone, either as an aggregate blend component or as a constituent in crushed gravel aggregate, and polyphosphoric acid (PPA) modified PG Binder (liquid asphalt)."

In an effort to determine the root cause of the "Atypical" flushing, the Consultant has proposed a Research Plan to help identify the main causes of the flushing issues and evaluate "flushing susceptibility" of laboratory mixed and compacted materials, using asphalt binder, aggregates, and job mix formulas mirroring the flushed sections in Central New York State. The research proposed involves several components, as listed in the RFP for Project C-08-15, "Determining Binder Flushing Causes in New York State".

CHAPTER 2 – STATE OF PRACTICE

Introduction

In the project statement for NYSDOT RFP C-08-15, "Determining Binder Flushing Causes in New York State", a Literature Review is required to assess the current state of the practice with respect to asphalt binding flushing. Along with asphalt binder flushing, a review of PPA modified asphalt mixture performance was also conducted and summarized in this chapter.

To help the reader, subsequent sections of this chapter are organized by topic. The first section discusses asphalt binder flushing, the mechanisms causing flushing, and studies attempting to evaluate asphalt binder flushing. The second section discusses PPA modified asphalt binder and mixtures and their general performance. The third and final section is a summary of the literature search. Individual summaries are provided for each reference.

Flushing of Asphalt Pavements

It is interesting to note that there does not exist much literature pertaining the flushing of asphalt pavements, especially literature pertaining to causes and forensic studies to determine causes. A majority of the information pertaining to flushing is the pure definition of flushing and what is hypothesized to be the main factors contributing to flushing. However, the below publications did contain more specific reasoning for how this distress does take place.

1. **Button, J.W., J. Epps, D. Little, and B. Gallaway, 1984, "Influence of Asphalt Temperature Susceptibility on Pavement Construction and Performance",** ***NCHRP Report 268*****, National Cooperative Highway Research Program, Transportation Research Board.**

A product of NCHRP Project 1-20, which was responsible for evaluating some of the rutting issues in the United States in the 1980's, evaluated similar phenomena to what had occurred on NYSDOT's flushed pavements. Although an exact reason could not be identified, the researchers did note that the flushing phenomena can be aggravated by the following;

- Highly temperature susceptible asphalt liquid;
- Asphalt with low asphaltene content (less than 10% or so); and
- Lesser degrees of asphalt binder hardening during the mixing process.

The work in NCHRP Project 1-20 also indicated that a low strain stiffness test (resilient modulus used in the 1980's) and indirect tensile strength would be sensitive enough to evaluate tender mixes and provides general criteria for tough and tender mixes. With this in mind, the dynamic modulus test may be a good surrogate test to utilize instead of the resilient modulus test.

2. Curtis, C., K. Ensley, and J. Epps, 1993, *Fundamental Properties of Asphalt-Aggregate Interactions Including Adhesion and Absorption*, Strategic Highway Research Report (SHRP) SHRP-A-341, National Research Council, Washington, D.C., 614 pp.

Chemical and physical properties of asphalt in the structured interphase region and their influence on asphalt mixture properties were evaluated under this part of the SHRP research program. Within this context, the ability of different asphalt binders to bond to different aggregate sources was researched. Bonding energy measurements obtained from the mixing of asphalt-aggregate pairs showed a high specificity among the various pairs. Testing indicated that aggregate composition and morphology were highly influential in determining the bonding energy of the asphalt/aggregate. Elemental components in aggregates that promote bonding include; calcium, iron, magnesium, and aluminum. Those elemental components that are detrimental to bonding include; sodium and potassium. It was also noted that from the SHRP aggregates evaluated, the RC- and RD-limestones bonding energies varied with asphalt sources and were at the low end of the ranking (i.e. – poorer bonding properties with asphalt binder).

The study also reported that not all limestone aggregates react with asphaltic compounds the same way and variability among limestone aggregates using the identical asphalt binder can be expected.

3. Krishnan, J. and C. Rao, 2001, "Permeability and Bleeding of Asphalt Concrete Using Mixture Theory", International Journal of Engineering Science, Vol. 39, p. 611 – 627.

The authors described the phenomena of asphalt mixture bleeding or binder flushing as the migration of asphalt binder to the surface of the pavement caused by the two major mechanisms:

> "The first mechanism is diffusion of binder into air voids when subjected to a temperature exceeding the binder softening point. The second mechanism is movement of the binder due to a pressure gradient developed within the pavement. Development of pressure gradient results from the reduction of air voids under traffic loading. Both mechanisms can occur simultaneously. The contribution of each depends on the temperature-stiffness relationship of the asphalt binder, the distribution of air voids in the asphalt mix, and the traffic loads on the pavement."

4. Abdelrahman, M., 2005, *Investigating Binder Flushing of SP-2 Mixes*, NDOR Research Project P559, Nebraska Department of Roads, 84 pp.

Abdelrahman (2005) investigated asphalt binder flushing that occurred on the LTPP pavement sections located in Nebraska. Some of the major findings of the study were:

1. All flushed sections had high asphalt content and insufficient dust (i.e. – low dust to binder ratio contents);
2. Extracted asphalt binder from field cores were found to be significantly softer than was expected/specified in the flushed sections as compared to the unflushed sections.

> "This indicates that a stiffer binder would have been less likely to flush." (Abdelrahman, 2005)

3. It was concluded that there may have been a problem with the asphalt source, mostly related to compatibility, caused by the separation of the light fractions of the binder.
4. Flushing occurred in spite of the fact that an acceptable percentage of air voids were present. However, it was noted that visual observation of the field cores showed that the voids were mostly filled with higher levels of asphalt and not mastic (asphalt and dust).

Two of the final recommendations from the study that the researchers were not able to quantify were:

1. Evaluating the nature, structure, and/or distribution of air voids in the asphalt mix as related to the binder flushing.
2. Examining this nature during the mix design process and compare the lab compacted air voids to the field compacted samples.

5. Abdelrahman, M., W. Jensen, and H. Salem, 2008, "Binder Flushing in Low Traffic Volume Superpave Mixes", *International Journal of Pavement Research and Technology*, Chinese Society of Pavement Engineering, Vol. 4, p. 121 – 128.

The authors provided an early look at the flushing that occurred on low volume Superpave mixes in Nebraska. The authors noted that:

> "Flushing occurs only under specific combinations of conditions, which are directly related to temperature-stiffness of the binder, air voids distribution, and traffic loading."

The authors also noted that:

- A significant percentage of voids in the flushed sections were filled with high asphalt content and binder-dust paste. However, most of the pavement sections were found to be within an acceptable percentage of air voids.

- Extracted binder samples from the flushed sections varied in physical and chemical properties but all were significantly softer than binders extracted from the non-flushed sections.
- Material properties of the binders, related to compatibility, are believed to have caused at least partial separation of the light fractions of the binder within the top pavement layer.

Performance of Polyphosphoric Acid (PPA) Modified Asphalt Mixtures

Patents pertaining to the use of Polyphosphoric Acid (PPA) modified asphalt can be found dating back to 1948. In a presentation provided by Baumgardner (2009) at the FHWA Symposium on the Use of PPA Modification, the following patents were quickly discussed.

Dr. Hoiberg (1948): Used Phosphorus Pentoxide and other Stable Acids of Phosphorus

- Lowered the temperature range of processing air blown asphalt binder from typical range of 490 to 500F to a range of 44o to 450F.
- Typical straight blown air properties at a softening point temperature of 212F had a penetration of 12 to 15mm, while at the same temperature, the acid modified asphalt had a penetration of 35 to 40mm.

Dr. Hoiberg (1962): Used stable acids as a catalyst to improve the adhesion properties of the asphalt in conjunction with an organic amine

Dr. Alexander (1973): First patent pertaining to paving grade asphalts. Used PPA to modify non air-blown paving grade asphalt

- Improved the viscosity range of the asphalt binder (i.e. – widened the PG temperature range)

It appears that the asphalt refining industry has long been aware, and used, stables acids, predominantly PPA, during the refining process to either help improve the base properties of the asphalt binder, or the PPA was used as a catalyst in conjunction with other modifiers. However, peer reviewed/journal articles and research-related publications regarding PPA modified asphalt mixtures were not readily published until recently. Below were the most relevant published technical articles found pertaining to the performance of PPA modified binders and mixtures.

1. Baumgardner, G., J-F Masson, J. Hardee, A. Menapace, and A. Williams, 2005, "Polyphosphoric Acid Modified Asphalt: Proposed Mechanisms", Journal of the Association of Asphalt Paving Technologists, Vol., pp.

The authors looked at how PPA influenced the asphalt binder/mixture from a chemical standpoint. The authors utilized different asphalt binder crude sources, neat and modified with PPA, and analyzed these binders for chemical composition by asphaltene precipitation, thin-layer chromatography (TLC), and nuclear magnetic resonance (NMR), by gel-permeation chromatography (GPC) and atomic force microscopy (AFM). The test results showed that the magnitude of the modification that PPA has is dependent on the crude asphalt source and it's respective chemical composition. For example, two different crude sources both showed a very similar increase in the PG grade after similar PPA modification. However, when evaluated chemically, one asphalt source showed that the PPA affected the dispersed phase, while in the second crude source, the PPA affected the matrix.

Figure 2 shows a perfect example of this phenomena, where 5 different crude sources were modified with the same percentage of PPA and had different effects. The data in Figure 2 was provided by Baumgardner (2009).

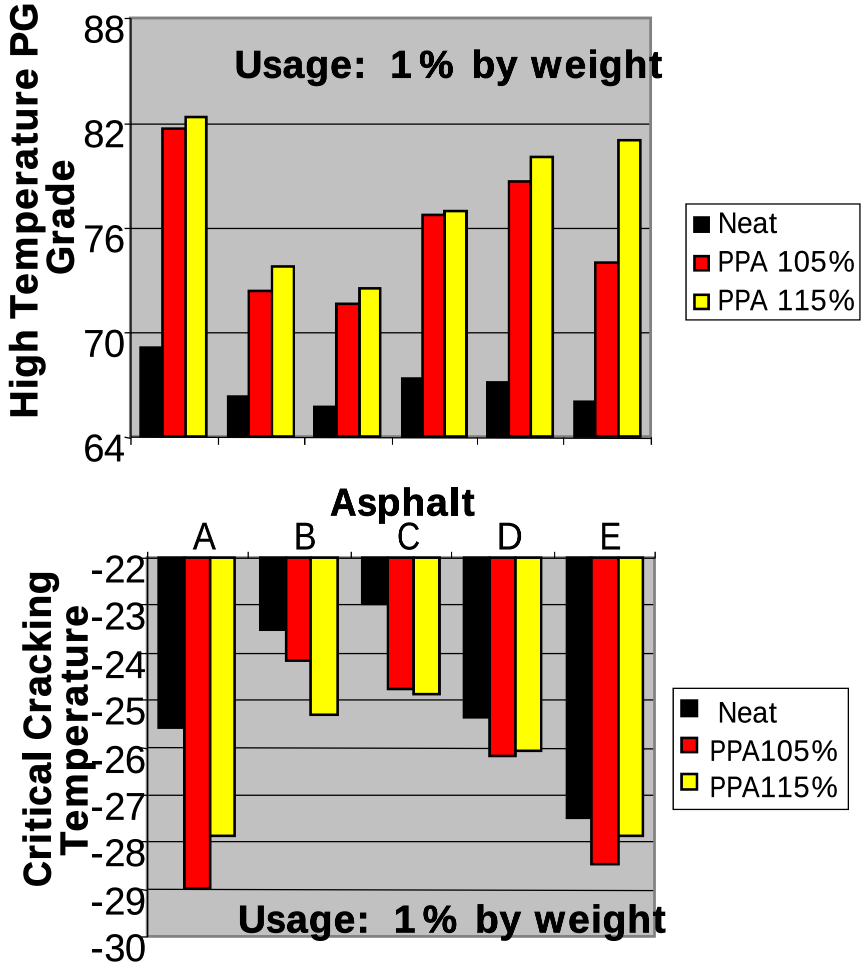


Figure 1 - Change in Asphalt Binder PG Properties Due to PPA Modification – 5 Different Crude Sources

2. Daranga, C., C. Clopotel, A. Mofolasayo, and H. Bahia, 2009, "Storage Stability and Effect of Mineral Surface on Polyphosphoric Acid (PPA) Modified Asphalt

Binders", Submitted for Publication to the 88th Annual Meeting of the Transportation Board (TRB), January 11th to 15th, 2009.

The authors conducted a study to assess how PPA modified binders aged under storage and if different mineral fillers influenced their properties. The authors concluded from their study that:

- The presence of PPA does not promote oxidation. In fact, it seemed from the study that for some binders, the oxidation aging slowed down. The authors hypothesized this may be due to the blocking of the reactive sites in the binder by reaction with PPA molecules and hydrogen bond formation. The found trends in oxidation aging were similar for both unmodified and modified binders, but the presence of PPA seemed to diminish oxidation aging for some of the binders tested.
- PPA modified binders, one with high Asphaltene content and one with low Asphaltene, showed different levels of aging. The high Asphaltene binder oxidized less than the low Asphaltene. This was hypothesized as the PPA molecules blocking the reactive sites of the asphaltenes that would otherwise take part in an oxidation reaction. This may indicate that crude binder sources with higher asphaltene contents may actually oxidize less and appear to be softer after plant production.

3. D'Angleo, J., 2009, "Effect of Polyphosphoric Acid on Asphalt Binder Properties", ***Journal of the Association of Asphalt Paving Technologists,*** **Volume 70, p. 679 – 694.**

The author discussed the recent work by the FHWA pertaining to PPA modified asphalt research, as well as present findings from other researchers to help summarize the overall impact of PPA on asphalt binder and mixture performance. The author concluded that:

- PPA does increase the stiffness of asphalt binders, although the magnitude of the stiffening is crude source dependent. Based on research at Turner-Fairbanks;
    - For high asphaltene Venezuelan crude, 0.5% PPA will increase the high temperature PG grade by one grade
    - For lower asphaltene Saudi crude, 1.2% PPA is required to increase the high temperature PG grade one grade
- The addition of PPA improves the properties of SBS modified binders. It is hypothesized that the PPA aids in the cross-linking of the SBS and the asphalt binder
- The addition of hydrated lime to PPA modified binders appeared to reduce some of the stiffening effect of the PPA. The amount of reduction in stiffness varied from 30% to 50%. Hydrated lime did not appear to reduce the % recovery of the binder as measured in the MSCR test. Once the cross-linking takes effect, the addition of lime does not reduce it.

4. Reinke, G. and S. Glidden, 2010, "Analytical Procedures for Determining Phosphorus Content in Asphalt Binders and Impact of Aggregate on Quantitative

Recovery of Phosphorus from Asphalt Binders", *Journal of the Association of Asphalt Paving Technologists*, Volume 70, p 695 – 718.

The authors presented information pertaining to methods of characterizing PPA modified asphalt binders through extraction and recovery procedures and discussed these procedures through case studies conducted across the United States. Of particular interest to this study, Reinke and Glidden (2010) conducted testing on cores extracted from the flushed pavements in New York State – the same pavement locations of concern for this study. The authors discussed how initial extraction and recovery data showed a decrease in the PG grade which was originally believed to be due to the interaction of the PPA and limestone aggregate used in the hot mix asphalt. However, upon further review of additional extractions conducted with different solvents, it was concluded that the original extraction and recoveries produced erroneous results due to the acid scavenger associated with the n-propyl bromide used as the solvent. According to the authors, the "neutralization of acid by extraction solvent" may be problematic with most industrial grade solvents.

> "Industrial grades of extraction solvents such as trichloroethylene and n-propyl bromide contain acid scavengers to stabilize the solvents. These typically used extraction solvents will over time form hydrochloric acid (HCl) or hydrobromic acid (HBr) unless a chemical is added to the solvent to scavenge the acid. Without the acid scavenger, the acid concentration can increase in the solvent to the extent that there can be corrosion in vacuum and centrifugal extraction equipment. The acid concentration, if high enough, will also cause hardening of the asphalt during Abson recovery." … "If there is acid present in the binder in the form of PPA, the 1.2 epoxy butane acid scavenger should also react with and neutralize those acidic ions, which would result in a reduction in the stiffness of the extracted binder."

Table 1, taken from Reinke and Glidden (2010), shows how acid scavengers may affect the asphalt binder results. The data in Table 1 show that the acid scavenger in the virgin n-propyl bromide had reduced the recovered asphalt binder stiffness by 50%, where as the reclaimed n-propyl bromide has reduced the recovered asphalt binder stiffness 27%. It is hypothesized that additional extraction and recoveries using the same sample of n-propyl bromide would provide even closer comparisons as the acid scavenger in the solvent is further used up. Meanwhile, the recovered asphalt binder from the toluene, which does not contain an acid scavenger, had an 11% reduction in asphalt binder stiffness. This slight reduction may be due to testing variability or the result of a small amount of solvent left due to the higher boiling point of toluene compared to that of the n-propyl bromide.

Table 1 - Results of Recovery of PPA Modified Binder from Different Solvents

Binder Source	Binder type	G*/sin(δ) @ 64°C of binder, kPa	G*/sin(δ) @ 64°C of recovered binder, kPa	Comments, solvent type
Lab Sample	PG 64-28 (0.75% PPA)	1.76	1.28	Reclaimed n-propyl bromide
Lab Sample	PG 64-28 (0.75% PPA)	1.67	0.825	Virgin n-propyl bromide
Lab Sample	PG 64-28 (0.75% PPA)	1.67	1.49	Toluene

Although Table 1 from Reinke and Glidden show data from laboratory prepared samples, the data shown in Table 2, again taken from Reinke and Glidden, shows a similar reaction with the field cores taken from the flushed New York State pavements. Once again, it can be seen that the acid scavenger in the virgin n-propyl bromide reduces the asphalt binder stiffness while the reclaimed n-propyl bromide provides more reasonable, although still lower than expected, asphalt binder stiffness. It is not until the Toluene solvent was used that expected asphalt binder stiffness properties were measured.

Table 2 - Recovered Asphalt Binder Properties from New York State Flushed Pavement Sections

Source, cores taken in Oct 2007, from mix placed in summer 2007	Binder type	G*/sin(δ) @ 64°C, kPa	Solvent
New York 2007 mix, Core 12B	PG 64-28 produced using 0.8% PPA	0.977	Reclaimed n-propyl bromide
New York 2007 mix, Core 12C	PG 64-28 produced using 0.8% PPA	0.455	Virgin n-propyl bromide
New York 2007 mix, Core 12A	PG 64-28 produced using 0.8% PPA	1.93	Toluene
New York 2007 mix, Core 3C	PG 64-28 produced using 0.8% PPA	2.66	Toluene
New York 2007 mix, Core 15C	PG 64-28 produced using 0.8% PPA	1.15	Toluene
New York 2007 mix, Core 15B	PG 64-28 produced using 0.8% PPA	2.32	Toluene
New York 2007 mix, Core 15A	PG 64-28 produced using 0.8% PPA	1.96	Toluene

5. Reinke, G., S. Glidden, D. Herlitzka, and S. Veglahn, 2010, "PPA Modified Binders and Mixtures: Aggregate and Binder Interactions, Rutting and Moisture Sensitivity of Mixtures", ***Journal of the Association of Asphalt Paving Technologists*****, Volume 70, p. 719 – 742.**

The authors presented an extensive mixture study on the rutting, fatigue cracking and moisture damage potential of asphalt mixtures containing asphalt binder modified with PPA. An interesting component of the study was that the researchers evaluated these properties with three different aggregate minerologies; Limestone, Gravel, and Granite. The authors noted that:

- Hydrated lime improved the moisture sensitivity of all mixes regardless of the use of PPA

- For any binder, there is a level of PPA that is too high that will result in decreased moisture resistance performance, mainly due to the PPA having reacted with all components and then there being free PPA in the binder volume.
    - The authors noted that some binders are just not suitable for PPA modification and this needs to be assess by the refinery
- PPA modification by itself is not equivalent to polymer modification alone, in terms of rutting and moisture sensitivity performance. However, PPA used in conjunction with polymer results in mixture performance that is generally equal to or better than polymer alone.

The authors also made another interesting note on the historical use of PPA modified asphalt binder. According to the authors, PPA modification has been used in Louisiana for about 20 years to make AC-30 and AC-40 with some asphalt binders. And since 1992/1993, the use of PPA only and polymer + PPA modification has grown exponentially across the United States. Based on their estimates, approximately 3,000, 000 tons of asphalt binder has been produced containing PPA. This equals to approximately 51,000,000 tons of hot mix asphalt. Given these quantities, if there are large problems with PPA being used as an additive, there should be significant field evidence – however, there is not.

6. Bennert, T. and J.V. Martin, 2010, "Phosphoric Acid in Combination with Styrene-Butadiene-Styrene Block Copolymer – Laboratory Mixture Evaluation", *Journal of the Association of Asphalt Paving Technologists*, Volume 70, p. 773 – 792.

The authors evaluated the mixture performance with asphalt binders modified with and without PPA. Three mixtures were tested; 1) Neat binder (base binder to the modified binders), 2) Binder solely modified with SBS, and 3) Binder modified with both SBS and PPA. The authors research indicated that the mixture modified with SBS and PPA performed equal to the solely SBS modified mixture in rutting, fatigue cracking, and moisture damage resistance. The study also showed that the PPA modified mixture was less sensitive to long term aging in the laboratory, resulting in better fatigue resistance. This finding of lesser aging with PPA modified asphalt binder appears to be consistent with other data reported.

7. Bennert, T. and J.V. Martin, 2011, "Recyclability of Polyphosphoric Acid-Modified Asphalt – A Laboratory Study", *Transportation Research Record: Journal of the Transportation Research Board, No. 2207*, Transportation Research Board of the National Academies, Washington, D.C., p. 79 – 88.

The authors conducted a research study to evaluate the performance of asphalt mixtures modified with various percentages of laboratory produced RAP. Three different RAP types were produced; 1) SBS polymer modified only, 2) SBS + PPA modified, and 3) PPA only modified. The RAP types were produced to originally meet the same PG grade prior to the long term aging protocol followed to produce RAP-like material. The RAP types were then mixed at varying percentages in a standard asphalt mixture and the resultant mixture performance assessed using permanent deformation, fatigue cracking, and moisture sensitivity testing. Laboratory testing indicated that PPA only modified and SBS + PPA modified asphalt, aged to produce a RAP-like material, are just as recyclable as the SBS only modified RAP material. Long term aging was not detrimental to the PPA modified mixtures and the PPA modified binder aged equally as the non-PPA modified mixtures.

8. Arnold, T., S. Needham, N. Gibson, A. Shastry, S. Parobeck, S. Li, and S. Stokowski, 2011, *Investigation of Oriskany Falls and Morrisville Road Projects in New York State*, Report Conducted by the FHWA Research, Development and Technology Division at the Turner-Fairbank Highway Research Center, US Department of Transportation.

After NYSDOT conducted an initial study of the flushing failures in NY State, material was supplied to the Turner-Fairbank Highway Research Center (TFHRC) to conduct an extensive material characterization to help determine reasons or mechanisms for the flushing issue. TFHRC received asphalt, aggregate and core samples from two road projects in New York State. Standard volumetric analysis, chemical composition and physical property tests were performed at the TFHRC. Findings from the TFHRC report showed that:

- The asphalt tests confirmed that the asphalt binders were modified with phosphoric acid. They met the PG specifications required by NYSDOT. XRF analysis showed that several binders were modified with 0.5-1.0% PPA. Some were not PPA modified at all. The results from one recovered binder suggest the PPA modification could have been as high as 2%. Elemental Analysis for Carbon, Nitrogen, Hydrogen, Sulfur and Oxygen were normal. The Scanning Electron Microscope/ Energy Dispersive Spectroscopy (EDAX) confirmed the aggregate was limestone.
- Analysis of the asphalt content and the gradation results obtained from the extracted aggregate shows a high dust/effective asphalt binder ratio. The voids in the mineral aggregate were high for both projects. Volumetric analysis of the mix primarily indicated that the mix was produced with a significantly finer blend of aggregate than designed (absence of #9 stone); in both the dust region and the primary control sieve region. This is likely to have resulted in a less than optimally stable mix.
- Less than optimal binder, low air voids, high VMA and VFA values, a large dust to binder ratio, and a larger-than-optimal asphalt film thickness will cause a mix to be prone to rutting, flushing and bleeding. This combination of factors was the most likely cause of the premature failure of the asphalt cement projects in New York State.
- No evidence was found to support the theory that phosphoric acid in the binder had reacted with the limestone aggregate thereby reducing the binder performance grade.

9. Arnold, T., S. Needham, and A. Shastry, 2011, *Effect of Wet Aggregate on PPA Modified HMA* and *Effect of Water on the Performance Grade of PPA Modified Asphalt*, Reports Conducted by the FHWA Research, Development and Technology Division at the Turner-Fairbank Highway Research Center, US Department of Transportation.

Work on the NYSDOT investigation prompted some additional studies at TFHRC to try to reproduce the problems experienced by NYSDOT. It was theorized that the phosphoric acid may have reacted with water to form an emulsion that softened the binder. To investigate the water susceptibility of a PPA modified HMA, the introduction of moisture into the asphalt binder and mixture containing PPA modified asphalt binder was investigated. The researchers at TFHRC concluded from the studies on that:

- Using different aggregate sources (limestone and diabase), the researchers at the TFHRC could not replicate the flushing/bleeding condition observed by NYSDOT on their pavements. The researchers did observe some raveling and soft asphalt mixtures. However, this was explained by the lower than normal mixing temperatures that resulted in the highly wet aggregates.
- The TFHRC used a modified procedure to introduce moisture in various asphalt binder sources, heat the material until boiling of the water was noticeable, and then sampled the material and tested for the original DSR PG grade. This procedure was conducted on samples before and after this "wetting" procedure. A table from the report summarizing the data is reproduced and shown as Table 3 for review. The test results indicate:
    - PPA Modified Asphalts: Fifteen PPA modified asphalts were boiled and tested on the DSR and for the presence of phosphorous. All of the boiled PPA modified asphalts and the water used to boil the asphalts tested positive for phosphorous. The difference in continuous PG after boiling varied from -2.2°C to -10.8°C. Five of the asphalts dropped one PG (6°C) and 10 of the asphalts did not. Only one of the NYSDOT samples lost one grade while all the laboratory prepared asphalts lost one performance grade.
    - Control and SBS Modified Asphalts: Seven control asphalts were tested. All of the control samples were negative for phosphorous after boiling. None of the control asphalts had greater than a 6°C drop (one PG) after boiling.

The researchers' final conclusion with respect to the addition of moisture to solely the asphalt binder was that when neat and SBS modified asphalts are boiled in distilled water, no measurable loss of stiffness was noted. One third of the PPA modified asphalt samples lost one performance grade when boiled and the remaining two thirds had some loss of stiffness. The presence of water in a PPA Modified Asphalt can affect the Performance Grade of the binder.

Table 3 - DSR Results for Boiled Asphalt/Water Samples

Original Asphalt				After Boil			Difference °C
NYSDOT #	Continuous PG	Supplier	FHWA #	Continuous PG	PPA Asphalt	PPA DIW	
220	66.6	Petro-Canada	08-0504	64.1	yes	yes	-2.5
455	66.6	Petro-Canada	08-0506	58.5	yes	yes	-8.4
737	65.9	Hanson Lima	08-0502	62.0	yes	yes	-3.9
852	65.4	Hanson Lima	08-0501	61.9	yes	yes	-3.5
232	70.7	Suitkote	08-0508	65.0	yes	yes	-5.7
587	72.9	CITGO	08-0513	70.7	yes	yes	-2.2
957	66.9	Petro-Canada	08-0512	63.7	yes	yes	-3.2
958	67.1	Petro-Canada	08-0510	64.0	yes	yes	-3.1
959	68.4	Petro-Canada	08-0511	63.5	yes	yes	-4.9
961	64.7	Suitkote	08-0503	59.4	yes	yes	-5.3
962	70.2	Suitkote	08-0509	65.7	yes	yes	-4.5
	76.6	CITGO	6362 1%PPA	68.7	yes	yes	-7.9
	76.6	CITGO	6362 1%PPA	68.6	yes	yes	-8.0
	76.6	CITGO	6362 1%PPA	65.8	yes	yes	-10.8
	76.6	CITGO	6362 1%PPA	66.6	yes	yes	-10.0
Control Samples							
	67.5	CITGO	6362-control	65.6	no	no	-1.9
	67.5	CITGO	6362-control	65.6	no	no	-1.9
	67.5	CITGO	6362-control	65.5	no	no	-2.0
	67.5	CITGO	6362-control	65.5	no	no	-2.0
	74.3	CITGO	6362 + SBS	72.9	no	no	-1.4
323	67.6	Suitkote	08-0505	63.5	no	no	-4.1
960	69.0	Suitkote	08-0507	65.9	no	no	-3.1

9. Arnold, T. and S. Needham, 2011, *Effect of Limestone Aggregate on PPA Modified Asphalt*, Reports Conducted by the FHWA Research, Development and Technology Division at the Turner-Fairbank Highway Research Center, US Department of Transportation.

In the last series of experiments conducted at the TFHRC pertaining to the flushing/bleeding found in NYSDOT, the researchers evaluated the compatibility of PPA modified binders with limestone aggregate. Three different asphalt binder sources and three different limestone sources (2 from NY State) were used in the study. All three asphalt binders were modified with 1% PPA by weight of the asphalt binder. All three asphalt binders had a before modification high temperature PG grade of 64^{o}C. After PPA modification, all three asphalt binders resulted in a high temperature PG grade of 70^{o}C.

Loose mix samples were produced with the different aggregate and asphalt binder sources and then aged for 4 hours at 135^{o}C. After mixture aging took place, the asphalt binder was extracted with TCE and evaluated for high temperature PG grade. Prior to any PG testing on the extracted binders, the asphalts were evaluated for residual solvent using Fourier Transform Infrared Spectroscopy (FTIR).

A summary of the resultant data reproduced from the report is shown in Table 4. The test results clearly indicate that after mixing and short term aging at 135^{o}C for 4 hours, no reduction of the asphalt binder stiffness occurs, and thereby, no loss in the Performance Grade. This and the fact that the recovered binder did contain phosphorous, suggests that the PPA did not react with any of the limestone aggregates. Table 2 shows that the recovered PPA modified asphalt was as stiff as or stiffer than the control (unmodified) asphalt for each aggregate type. For all three binders, any reaction that might have occurred between the phosphoric acid and the limestone aggregate did not adversely affect the performance grade of the binder. The addition of PPA to a limestone HMA did not adversely affect the predicted performance of the asphalt binder in any of the mixes tested.

Table 4 - Performance Grade of Asphalt Binders with and without PPA Modification

		RTFOT Binder PG	Recovered Binder		
			Aggregate type		
			NY4	NY3	MD
CITGO	PG	64	70	70	70
	Continuous PG	66.2	70.7	70.0	72.0
CITGO & 1% PPA	PG	70	70	70	70
	Continuous PG	74.8	71.0	70.9	71.3
BP Whiting	PG	64	64	64	70
	Continuous PG	66.0	69.2	69.6	71.9
BP Whiting & 1% PPA	PG	70	70	70	76
	Continuous PG	73.0	72.2	71.1	77.1
Lion Oil	PG	64	64	64	70
	Continuous PG	66.9	68.5	68.6	71
Lion Oil & 1% PPA	PG	70	70	70	70
	Continuous PG	70.8	70.4	69.7	73.1

Summary of Literature Review Findings

The Literature Review provided an interesting look at the possible mechanisms that cause flushing/bleeding of asphalt binder in pavements, as well as the general performance of Polyphosphoric Acid (PPA) modified asphalt binders. The information collected during the Literature Review showed that:

- Two major mechanisms are believed to cause flushing. The first mechanism is diffusion of asphalt binder into air voids when subjected to a temperature exceeding the binder softening point. The second mechanism is movement of the asphalt binder due to a pressure gradient developed with the pavement. Development of a pressure gradient result from the reduction of air voids under traffic loading. Both mechanisms can occur simultaneously. The contribution of each depends on the temperature-stiffness relationship of the asphalt binder, the distribution of air voids in the asphalt mix, and the traffic loads on the pavement. Other less common causes found in the literature include diesel fuel and solvent contamination of the asphalt binder.
- A final determination of the reasons for the flushing/bleeding that had occurred on Nebraska's LTPP pavement sections were never identified with 100% confidence. However, the authors did conclude that the flushing was most likely a result of the combination of high asphalt content, low fines content, lower stiffness asphalt

binder and compacted densities that were low, although still within specification tolerances. The authors also stated that the asphalt binder may have had a compatibility issue with the aggregates used in the hot mix asphalt, although scientific testing was never done to confirm this hypothesis.

- Asphalt binders modified with Polyphosphoric Acid (PPA) show less sensitivity to oxidation aging that asphalt binders modified without PPA. This was concluded through both asphalt binder and asphalt mixture tests. However, this reduction in oxidation aging due to PPA modification appears to the asphalt binder crude source dependent. Limited testing has indicated that the amount of Asphaltenes present in the asphalt binder may be an indicator of whether or not the PPA will aid to the resistance to oxidation aging; the higher the Asphaltene content, the lesser the oxidation stiffening.
- The research conducted by Reinke and Glidden (2010) indicated that the type of solvent used during the extraction and recovery of PPA modified asphalt binders is extremely important. Solvents containing acid scavengers, such as n-Propyl Bromide and industrial grade TCE contain acid scavengers to help reduce corrosion of metals commonly used around these acids. The researchers showed that these acid scavengers will have the tendency to draw the PPA out of the asphalt binder, thereby reducing the high temperature stiffness of the PPA modified asphalt binder and reducing the overall high temperature PG grade. The authors recommended to use a laboratory, research grade TCE or Toluene to help reduce the possibility of the PPA loss to the acid scavengers.
- Data generated at the Turner-Fairbanks Highway Research Center (TFHRC) on cores sent to them from New York State showed that five of the six recovered binders from NYSDOT cores showed a drop in the PG after extraction/recovery. However, the continuous PG shows that the difference between the original grade and the recovered grade of the samples is less than 5°C in all but one of the cores. The last 6 rounds of proficiency sample testing results for Hot Mix Asphalt Solvent Extraction by Abson Method, published by the AASHTO Material Reference Laboratory (AMRL), shows the coefficient of variation for single operator precision is over 11%. All of the binders are within 11% of the original binder value of 64°C. On this basis the binder grade did not change. It should be noted that the material evaluated by TFHRC were from Oriskany Falls and Morrisville Area.
- Further testing conducted by the TFHRC on the New York State flushed pavements indicated that analysis of the asphalt content and the gradation results obtained from the extracted aggregate showed a high dust/effective asphalt binder ratio. The voids in the mineral aggregate were high for both projects. Volumetric analysis of the mix primarily indicated that the mix was produced with a significantly finer blend of aggregate than designed (absence of #9 stone); in both the dust region and the primary control sieve region. This is likely to have resulted in a less than optimally stable mix.

CHAPTER 3 – NYSDOT SPECIFICATIONS AND DOCUMENTS REVIEW

Introduction

As part of an extension of the Literature Review, New York State specifications and construction documents were reviewed to help provide a better understanding of whether or not the asphalt mixtures produced met NYSDOT material requirements. It was hopeful that a more in-depth look at the mixture design and production data may lead to specific areas that may have caused the flushing condition to take place – whether this was asphalt mixture, aggregate, mixture, construction or pavement condition related.

PG Grade Selection

As indicated in the Literature Review, one of the potential reasons for flushing/bleeding is the use of a soft asphalt binder that is mobilized due to traffic loading conditions during warmer climate conditions. For all of the flushed pavements in NY State, the high temperature PG grade of the asphalt binders was a PG64.

When pavement designers select appropriate PG grades for asphalt binder selection, it is recommended that the pavement designer utilizes the FHWA's LTPPBind software. This software was developed using weather stations across the country to accurately determine the high and low temperatures of a particular area to provide guidance as to the most appropriate asphalt binder grade. The software contains options for the designers to also select traffic level, traffic speed (fast or slow/standing) and reliability levels. These options allow a pavement designer to select the correct asphalt binder grade for local climatic and traffic conditions.

Six pavement locations, well documented by the NYSDOT during the initial flushing problems, were reviewed to assess the mixture design ESAL level for the asphalt mixtures placed on those pavements. These sections and their design ESAL level were;

- Rt 12B, Region 2 Oneida County: < 3 million ESAL's
- Rt 365, Region 2 Rome to Barnevard: < 3 million ESAL's
- Rt 315, Region 2 Oneida County: < 3 million ESAL's
- Rt 20, Region 2 Madison County: < 30 million ESAL's
- Rt 921C, Region 2 Utica: < 30 million ESAL's
- Rt 5, Region 3 Auburn: < 30 million ESAL's

In each one of the pavement sections above, flushing/bleeding in the asphalt pavement overlay was observed by the Consultant at the intersection areas of the pavements. Therefore, this would be classified as "Slow/Standing" traffic.

The LTPPBind 3.1 software was used to determine the “most appropriate” asphalt binder for the climate and traffic conditions in the area of the pavement sections (primarily in Region 2) noted above, as well as Region 3 (Syracuse area) and Region 7 (Watertown area). The resultant recommended high temperature PG grades, and also the continuous high temperature PG grade, is shown in Table 3.1. The data generated in Table 3.1 indicates that 3 of the 6 pavement sections noted earlier, which had a design ESAL level of < 30 million ESAL’s, should have utilized a PG binder grade of at least 70°C. And if one decides to utilize the pavement surface (0 mm from the surface) as the pavement temperature location since the asphalt binder flushed at the surface, the Utica area would require a PG76 high temperature for the climate and traffic conditions associated with this area and the < 30 million ESAL level.

Table 5 - Recommended High Temperature PG Grade for Region 2, Region 3, and Region 7 Area

Location in NY State	Design ESAL's (millions)	High Temperature Recommended PG Grade at a Specific Depth in Pavement			
		0 mm from Pavement Surface		20 mm from Pavement Surface	
		PG Grade	Continuous PG	PG Grade	Continuous PG
Syracuse	< 3	58	54.5	58	52.1
	3 to 10	64	62	64	59.6
	10 to 30	70	67.2	70	64.8
	> 30	70	69.4	70	67
Watertown	< 3	58	54	52	51.6
	3 to 10	64	61.5	64	59.1
	10 to 30	70	66.7	70	64.3
	> 30	70	68.9	70	66.5
Utica	< 3	58	55.7	58	53.3
	3 to 10	64	63.2	64	64
	10 to 30	70	68.4	70	66
	> 30	76	70.6	70	68.2

Meanwhile, for the pavement sections containing ESAL levels of the < 3 million ESAL range, the appropriate PG grade had been recommended.

This quick exercise demonstrated that in some of the pavement locations that flushed (Figures 2 to 4), due to the higher traffic level and slow/standing condition of the traffic at the intersections, a stiffer asphalt binder should have been specified for these areas. Although this PG asphalt binder grade selection issue was not a problem with the < 3 million ESAL asphalt mixes, it does indicate that issues pertaining to asphalt binder grade selection may have contributed to some of the flushing problems witnessed.

It should be noted that the data generated in Table 3.1 was done so using a 98% Reliability factor for the PG grade selection in the LTPPBind 3.1 software. This type of analysis is further shown in detail later in the report.

Figure 2 - Flushing at Intersection of Rt 20 (< 30 Million ESAL’s)

Figure 3 - Flushing at Intersection of Rt 921C (< 30 Million ESAL's)

Figure 4 - Flushing at Intersection of Rt 5 (< 30 Million ESAL's)

Review of NYSDOT Collected Data and Documents

Immediately after the NYSDOT discovered the flushing issues in New York State, efforts were taken to determine the issues surrounding the flushing. A number of cores were taken from the various projects and evaluated at the NYSDOT Materials Laboratory. Material properties include, but not limited to mix composition, asphalt binder grade, and mix volumetrics.

Mr. Prithvi Kandhal, Director Emeritus of the National Center for Asphalt Technology (NCAT) was subcontracted to evaluate the documents and test data provided by the NYSDOT. Mr. Kandhal is known as an expert in the field of asphalt mixture design, production, and quality control and has over 30 years of experience working for the Pennsylvania Department of Transportation (PennDOT), as well as another 15 years at NCAT. The title of the provided document/data is noted in the following section, along with the summarized NYSDOT information and the analysis and comments from Mr. Kandhal.

<u>Route 12B</u>

(a) Document, "9-2008 Cores for AC Grade" dated 15 September 2008

Eight cores were obtained: 4 from places where PPA modified PG 64-28 binder sections were replaced with neat PG 64-22 and 4 from places where PPA 64-28 material was left in place.

Recovered binder from neat PG 64-22 section was classified as PG 61.5 to PG 65.3 (almost the same PG grade as used in reconstruction). Recovered binder from PPA 64-28 section was classified as PG 52 or PG 58 (mostly PG 52), that is two grades softer than the PG grade used in construction.

Comments: Standard solvent used by the NYSDOT in extractions and recoveries needs to be documented. If there are any cores left with NYSDOT, extractions and recoveries need to be made using toluene, which is believed not to react with PPA. If not, cores need to be taken again and binder recovered using standard solvent (used by NYSDOT) as well as toluene for comparison. It has been reported that the mix produced was much finer than the job-mix formula (JMF). Therefore, there is a need to check the mix volumetrics by putting together the mix as produced. This replicated field mix is likely to have low VMA, low air voids, and low stability. Although the binder may have been a problem (due to PPA or other causes), a finer mix probably accentuated the problem of flushing.

(b) Document, "Pre-warranty_Pavement_Distress_Evaluation"

Rut depth was measured on both Route 12 B and Route 26 on October 5, 2007. Rut depth was about 1/8 inch at intersections and 1/16 inch at other places.

Comment: The reported rut depths are believed to be normal; these are not ruts but usual consolidation of mix in wheel path under traffic. In such a case, testing these mixes from these two projects for rutting as in work plan needs to be reconsidered because rutting is not a problem.

Route 20

(a) Document, "D 260103_CCC"
This document contains correspondence between NYSDOT and Contractor CCI about the flushing problem and replacements of affected sections.

Comment: It appears replacements were done after September 11, 2007.

(b) Document, "Pre-Warranty Pavement distress Evaluation"
This document gives rut depths as measured on October 4, 2007. Rut depths range from 2 to 3 (no unit is given).

Comment: Is the unit for rut measurement in mm? If so, it is negligible and is not of any concern. Again, the need for testing mix from Route 20 for permanent deformation as in work plan needs to be reconsidered.

(c) Document, "Rte_20_Memo_doc", dated September 18, 2007
This document gives the following test data:

- Binder was recovered from one core; it was classified as PG 52 (similar to route 12B) and not PG 64 as used in construction.
- Retained PPA PG 64-28 binder samples were retested and were found to meet the specification.
- Two cores were tested for mix gradation and asphalt content. The gradation in the cores was finer than the JMF gradation especially the material passing 0.075 mm sieve, which was 8.1 percent.

Comment: Again, whatever standard solvent was used by NYSDOT for extraction and recovery must be used along with toluene for comparison similar to Rte 12 B.

If a sample of PPA modified PG 64-28 retained by NYSDOT is available it should be obtained for further testing for PPA as well as presence of re-refined oil in the binder. There is a patent on using re-refined oil in paving asphalt binders in which most common metals are taken out in re-refining of oil.

Since the produced gradation is very fine and has no resemblance to JMF, the produced mix should be replicated in the laboratory to determine its volumetrics similar to Rte 12 B as mentioned earlier. This is important because most likely the finer mix accentuated the flushing problem.

Route 921C

This route has the most test data obtained by the NYSDOT. Therefore, this route should be investigated in more detail in this project.

(a) Document, "Bleeding Issue – Aggregate Data"

This document gives some mix design data obtained by Barrett Paving on July 18, 2007. The mix contains #1A Stone (40.3% carbonate and 19.7% noncarbonate) plus 34% fine aggregate. The mix had fines/effective AC ratio of 1.80 (normally it should be 0.6 to 1.2), which is too high.

Comments; The complete mix design (gradation and asphalt content) needs to be obtained. It also needs to be determined whether the fine aggregate used was limestone or non-carbonate aggregate.

(b) Document, "Bleeding Issue- Core Data pdf"

According to this document, paving was done on June 18, 2007 and nine cores were obtained on July 14, 2007 (about a month later). Cores were obtained in pairs, one on wheel path and another in center of lane. Core density values indicate 96.3% compaction (3.7% air voids) on wheel path and 94.7% compaction (5.3% air voids) in center of lane. Normally, such low air voids are obtained in the field after 2-3 years of traffic densification. Average gradation of mix from the cores indicates a finer mix compared to JMF. The material passing 0.075 mm was determined to be 8.9% compared to 4.8% in the JMF. This partly explains low air voids in the mat.

Comments: As mentioned above, field air voids after about a month are very low. This can happen from excessive asphalt content, excessive fines and/or soft asphalt binder grade. The mix design needs to be rechecked using the finer field gradation to determine mix volumetrics.
The core test data does not indicate any significant difference between wheel path and center of lane mat density values; it appears to be a normal variation considering some densification took place under traffic during the first month. This is contrary to what has been pointed out in the document.

(c) Document, "Bleeding Issues – Plant Data"

The JMF of the mix has been given in this document along with gradation chart after conversion to 3.5% air voids done on June 18, 2007.

Comments: It is not clear whether the fine aggregate is limestone or a noncarbonated aggregate. The job-mix formula gradation is very open graded touching the lower limit on 2.36 mm sieve. There is a need to compare the JMF given in this document with document (b), especially both gradations should be plotted for comparison.

(d) Document, "Core Density – N. Genesse St"
This document is similar to document (b), "Bleeding Issues – Core Data"

(e) Document, "D 260235, Rte 921 C, N. Genesse St Bleeding Issues"
This document shows pictures of 8 cores with and without flushing. Cores in wheel path (Cores # 1, 3, 5, 7) show flushing on surface.

(f) Document, "D 260235 Barrett pdf"
This document gives correspondence between Barrett and NYSDOT concerning Rte 12, 46, and 921 C. Barrett stated that their paving got a bonus of 5%, so it was more than satisfactory. NYSDOT disallowed the use of PPA modified PG binder on August 24, 2007. Removal and replacement of the mix placed on June 18-19, 2007 was performed on October 2007. According to Barrett, NYSDOT should pay for the removal and replacement.

(g) Document, "Petro Canada Report – pdf"
Petro Canada submitted report to NYSDOT on September 5, 2007. Barrett was Petro Canada's customer for supply of asphalt binder for these projects. Barrett requested Petro Canada to investigate Rte 921 (N. Genesse St., Utica, NY) paved on June 26, 2007.

Petro Canada visited the site. Rutting was reported to be minimal, 1-2 mm only. Twelve cores were received by Petro Canada on August 16, 2007 and tested by them with the following tests and their own interpretations:

- Core densities showed 96.7% compaction (3.3% air voids) indicative of flushing problem.
- Abson recovery (TCE extraction, centrifugation and rotovapor distillation) was performed. Recovered binder was rated PG 57.2-30.7 (supplied was PG 64-28). Metal analysis of recovered binder (ICP) showed a decrease in P content and corresponding increase in CA confirming that the recovered asphalt binder suffered a depletion of PPA from 0.46 to 0.12% by weight, that is, 75% depletion of PPA (Table 6).
- Test data on SHRP evaluation of original and recovered asphalt binder is given in Table 4 of Petro Canada report. Ideally, the recovered asphalt binder should be comparable to the RTFOT residue of the original binder. However, the recovered binder is significantly softer than the RTFOT residue as shown in the following table.

Table 6 - Extracted and Recovered Binder Data from Petro Canada Report

Test	RTFOT Residue of Original binder	Recovered binder
G*/sin delta at 58 C, kPa	6.78	1.95
G*/sin delta at 64 C, kPa	3.08	0.95
Softening point, C	54.4	46.9
Penetration at 25 C	55	103
Rotational viscosity at 135 C, cp	767	375
PG Grade	PG 64-28	PG 57.2 – 30.7

The recovered asphalt binder is also soft at 25 C with a high penetration of 103. It is equivalent to an original binder of 120-150 penetration grade. Obviously, the mix could be scrapped off with a knife.

- The gradation of mix obtained from cores is very fine compared to the JMF, especially the material passing 0.075 mm sieve.
- Chemical reactivity of 3 aggregates with pure and diluted PPA was determined. There was no reaction with granite; some reaction with a Montréal limestone; and high reaction with the job limestone.
- According to report, asphalt film thickness was too low due to high amounts of 0.075 mm (P 200) material (larger surface area) and therefore it has been postulated that the reaction between aggregate and PPA was enhanced.

[Comment: All P 200 material does not necessarily provides a high surface area because some may simply be embedded in asphalt binder making the fines/asphalt mortar.]

- Recovered aggregate had a DP (dust proportion) of 3.0.

[Comment: It is not understood as to how DP was calculated. Normally, F/A ratio should range between 0.6 and 1.2.]

Comment: The reaction of limestone P 200 with PPA 64-28 should be investigated. It should be examined whether a wet P 200 (using a pressure cooker) can be used to simulate drum mix plant. The report states such an environment exists in the front end of the mix drum of the HMA plant.

(h) MISCELLANEOUS DATA

Document, "General Flushing Spread Sheet for all Projects"

This document gives the following data for Rte 20, Rte 921C, and rte 12 B & 26: HMA placing dates, tonnage, % flushed, contractor, producer, binder supplier, JMF asphalt content, extracted asphalt content, and appearance of mat in the field. The data is summarized in the following table (Table 7).

Table 7 – Summary of Construction Data from NYSDOT Reports

Information	Rte 20	Rte 921 C	Rte 12 B & 26
Placing dates	7/12-26, 2007	6/18/2007	7/21-8/6, 2007
Total tonnage	4,500	1,019	2,493
Contractor	CCI	Barrett	Hanson
Binder supplier	Suit-Kote	Petro Canada	Suit-Kote
Percent flushed	50	80	25
JMF AC	6.0-6.2	6.1-6.2	6.6
Extracted AC	6.1-6.4	5.6-6.6	6.7-7.0
Core gradation	All screens fail – too fine	Most screens fail – Too fine	Most screens fail – Too fine
Appearance of mat	Intersections poor; lanes flushing	Intersections poor; lanes flushing	Intersections poor; lanes flushing

Summary of NYSDOT Document Review by Mr. Kandhal

- Recovered binder from PPA 64-28 sections was generally classified as PG 52 or PG 58 (mostly PG 52), that is two grades softer than the PG grade used in construction. Extractions and recoveries were made by NYSDOT with a standard solvent which may have reacted with PPA. Therefore, there is a need to reconfirm the grade of the recovered binder using toluene which has been reported not to react with PPA. If there are no cores left, additional cores need to be taken to do it.

- On most projects that flushed, it has been reported that the mix produced in the field during construction was finer than the job-mix formula (JMF). Therefore, there is a need to check the mix volumetrics by putting together the mix as produced. This replicated field mix is likely to have low VMA, low air voids, and low stability. Although the binder may have been a problem (due to PPA or other causes), a finer mix probably accentuated the problem of flushing.

- On some projects the reported rut depths are believed to be normal; these are not ruts but usual consolidation of mix in wheel path under traffic. Testing the mixes from these projects for rutting needs to be reconsidered.

The findings of Mr. Kandhal confirm the information reported by FHWA's TFHRC findings in their reports, which was summarized during the Literature Review.

CHAPTER 4 – FIELD CORES LABORATORY INVESTIGATION

An extensive laboratory investigation was conducted to evaluate a number of potential reasons why the NYSDOT asphalt mixtures placed on the various pavement sections in New York exhibited flushing and premature cracking. First, as previously noted in the Literature Review and even in the comments by Mr. Kandhal, cores taken from various "flushed pavements" underwent solvent extraction using two different solvents; TCE and Toluene. TCE was used for all of the original recoveries where the NYSDOT observed a reduction in high temperature PG grade. However, as the Literature Review identified, commercial grade TCE may contain "acid scavengers", which are commonly used to provide stable storage of TCE in metal containers. Unfortunately, it is hypothesized that these "acid scavengers" may have taken PPA away from the asphalt binder, thereby softening it. Therefore, a comparison study between TCE and Toluene solvents was conducted to determine if differences could be found.

Second, a chemical analysis of the recovered asphalt binder was conducted to evaluate if phosphorus was detectable in the asphalt binder, meaning that PPA was present. Calibration procedures conducted in the laboratory allowed for an estimate of the percentages of PPA used. Along with phosphorus, the chemical component breakdown of the asphalt binder was also determined using a SARA analysis procedure.

Third, X-ray Tomography of field cores taken from pavement sections with and without flushing was conducted to try to evaluate the air void and aggregate distribution. As noted in the Literature Review, early testing conducted by a research laboratory in 2010 noted that some of the cut cores appeared to show signs of "segregation". Therefore, the X-ray work conducted to hopefully quantify what was observed. It should be noted that at the time of the testing, the methods utilized were highly experimental.

Asphalt Binder Properties of Extracted Field Cores

Asphalt Binder Grade "Appropriateness"

The asphalt binder was extracted and recovered from various pavement sections that had exhibited flushing. During a field investigation conducted by the Consultant and accompanied by a NYSDOT Materials Engineer, it was noted that the flushed pavement sections were primarily located at intersection areas, where slow to stopped traffic exists. The FHWA's LTPPBind 3.1 software allows for the determination of the appropriate asphalt binder grade based on the climatic and traffic conditions on the specific location. Included in the traffic conditions is the traffic speed, which has a significant impact on rutting performance. Due to the creep properties of asphalt materials, asphalt binders/mixtures are time dependent. Asphalt mixtures loaded at typical traffic speeds (> 55 mph) perform in a stiffer manner than at slow traffic speeds (< 10 mph). Therefore, asphalt mixtures placed at intersections may be rut susceptible due to the slow to stopping traffic. Based on this general concept, a number of the flushed pavements were evaluated for the asphalt binder grade “appropriateness” and are summarized in the following section.

Rt 921C – Litchfield, NY (Genesee, Street)

Rt 921C was reported to have an ESAL level of 10 to 30 million ESAL’s. The pavement was located in the immediate area of an on-ramp for an interstate highway. Pavement flushing, and in some areas, excessive rutting was observed for this area (Figure 5).

Figure 5 – Flushing and Rutting Located on Rt 921C, Utica, NY

The LTPPBind 3.1 software was used to determine the appropriate high temperature binder grade for the area. The “Slow Traffic/Intersection” traffic speed was used due to the location and observed traffic pattern in the area. Table 8 shows the recommended high temperature PG grade for the pavement location and traffic condition. For both the surface and 20 mm of depth from the pavement surface, LTPPBind 3.1 recommends a high temperature PG Grade of PG70. It should be noted that the NYSDOT was using a PG64-28 asphalt binder in that area.

Table 8 – LTPPBind 3.1 High Temperature PG Grade Recommendation for Rt 921C, Utica, NY

Location in NY State	Design ESAL's (millions)	High Temperature Recommended PG Grade at a Specific Depth in Pavement ("Slow" Traffic/Intersections)			
		0 mm from Pavement Surface		20 mm from Pavement Surface	
		PG Grade	Continuous PG	PG Grade	Continuous PG
Rt 921C Litchfield, NY Genesee St.	< 3	58	55.7	58	53.3
	3 to 10	64	63.2	64	60.8
	10 to 30	70	68.4	70	66
	> 30	76	70.6	70	68.2

Rt 20, Morrisville, NY

Rt 20 in Morrisville, NY was another pavement section that had traffic levels of 10 to 30 million ESAL's. The areas with extensive flushing were located at an incline/decline area (Figure 6).

Figure 6 – Flushing Located on Rt 20 in Morrisville, NY

Table 9 shows the LTPPBind 3.1 high temperature PG grade for the area and traffic conditions. Again, due to the slow moving traffic, the "Slow Traffic/Intersection" condition was chosen in the software. The LTPPBind 3.1 software recommends a high temperature PG grade of PG70. As with Rt. 921, the asphalt binder originally used at the location was a PG64-28.

Table 9 - LTPPBind 3.1 High Temperature PG Grade Recommendation for Rt 20, Morrisville, NY

Location in NY State	Design ESAL's (millions)	High Temperature Recommended PG Grade at a Specific Depth in Pavement ("Slow" Traffic/Intersections)			
		0 mm from Pavement Surface		20 mm from Pavement Surface	
		PG Grade	Continuous PG	PG Grade	Continuous PG
Rt. 20 Morrisville (Region 2)	< 3	58	55.7	58	53.3
	3 to 10	64	63.2	64	60.8
	10 to 30	70	68.4	70	66
	> 30	76	70.6	70	68.2

Rt 5, Auburn

Another 10 to 30 million ESAL pavement section, the flushing and slight rutting on Rt 5 again occurred at an intersection (Figure 7). LTPPBind 3.1 was used to determine the correct PG grade used for the climatic and traffic conditions located in this area. Table 10 shows the results for this pavement section. Similar to the previous 10 to 30 million ESAL areas, the appropriate PG grade for this location should have been a high temperature grade of PG70. However, a PG64-28 asphalt binder was specified and utilized.

Figure 7 – Flushing at Intersection on Rt 5

Table 10 - LTPPBind 3.1 High Temperature PG Grade Recommendation for Rt 5, Auburn, NY

Location in NY State	Design ESAL's (millions)	High Temperature Recommended PG Grade at a Specific Depth in Pavement ("Slow" Traffic/Intersections)			
		0 mm from Pavement Surface		20 mm from Pavement Surface	
		PG Grade	Continuous PG	PG Grade	Continuous PG
Rte. 5, Auburn	< 3	58	55.6	58	53.2
	3 to 10	64	63.1	64	60.7
	10 to 30	70	68.3	70	65.9
	> 30	76	70.5	70	68.1

Rt 315, Deansboro, NY

Rt 315 in Deansboro, NY (Region 2) is a low volume pavement section carrying < 3 million ESAL's. Again, flushing, although minor compared to the previously mentioned sections, was observed in an intersection area. And although the traffic was determined to be < 3 million ESAL's, construction vehicles do frequent the area, as shown in the photo below (Figure 8).

Figure 8 – Flushing at the Intersection of Rt 315 in Deansboro, NY (Region 2)

The LTPPBind 3.1 software was used to determine the appropriate PG binder grade for the climatic and traffic conditions of the area. Table 11 shows the results of the LTPPBind 3.1 analysis. According to LTPPBind 3.1, for the climate and traffic level selected, a high temperature grade of PG58 should have been sufficient for the intersection.

Table 11 - LTPPBind 3.1 High Temperature PG Grade Recommendation for Rt 315, Deansboro, NY

Location in NY State	Design ESAL's (millions)	High Temperature Recommended PG Grade at a Specific Depth in Pavement ("Slow" Traffic/Intersections)			
		0 mm from Pavement Surface		20 mm from Pavement Surface	
		PG Grade	Continuous PG	PG Grade	Continuous PG
Rt 315 Deansboro (Region 2)	< 3	58	55.7	58	53.3
	3 to 10	64	63.2	64	60.8
	10 to 30	70	68.4	70	66
	> 30	76	70.6	70	68.2

Rt 12B, Oriskany Falls, NY

Rt 12B in Oriskany Falls, NY was identified as having minor flushing at the intersection area shown in the figure below (Figure 9). The pavement section in the immediate area was determined to be carrying < 3 million ESAL's.

Figure 9 – Flushing at the Rt 12B Intersection in Oriskany Falls, NY

Table 12 shows the results of the LTPPBind 3.1 software recommendation for the appropriate high temperature binder grade for the climatic and traffic conditions in the immediate area. The table shows that a high temperature grade of PG58 should have been sufficient for the intersection area.

Table 12 - LTPPBind 3.1 High Temperature PG Grade Recommendation for Rt 12B, Oriskany Falls, NY

Location in NY State	Design ESAL's (millions)	High Temperature Recommended PG Grade at a Specific Depth in Pavement ("Slow" Traffic/Intersections)			
		0 mm from Pavement Surface		20 mm from Pavement Surface	
		PG Grade	Continuous PG	PG Grade	Continuous PG
Rt 12B Oriskany Falls (Region 2)	< 3	58	55.7	58	53.3
	3 to 10	64	63.2	64	60.8
	10 to 30	70	68.4	70	66
	> 30	76	70.6	70	68.2

Rt 365, Rome to Barneveld, NY

Another lower volume road, Rt 365 Rome to Barneveld, NY was estimated at carrying < 3 million ESAL's. Similar to the other lower volume pavement sections, minor flushing was observed at the intersection area of this pavement section (no picture available).

The LTPPBind3.1 software was used to determine the appropriate high temperature PG grade for the climate and traffic conditions in the area. Table 13 shows the results of the LTPPBind 3.1 software. The results show that LTPPBind 3.1 recommends a high temperature PG grade of PG58. It should be noted that the NYSDOT had specified a PG64-28 on this section.

Table 13 - LTPPBind 3.1 High Temperature PG Grade Recommendation for Rt 365, Rome to Barneveld, NY

Location in NY State	Design ESAL's (millions)	High Temperature Recommended PG Grade at a Specific Depth in Pavement ("Slow" Traffic/Intersections)			
		0 mm from Pavement Surface		20 mm from Pavement Surface	
		PG Grade	Continuous PG	PG Grade	Continuous PG
Rte. 365, Rome to Barneveld	< 3	58	55.7	58	53.3
	3 to 10	64	63.2	64	60.8
	10 to 30	70	68.4	70	66
	> 30	76	70.6	70	68.2

<u>PG Grade "Appropriateness" Conclusions</u>

Six (6) different pavement sections, where field cores were available for testing, were first analyzed using the LTPPBind 3.1 software, which is recommended by the FHWA to determine the appropriate PG grade for specified climatic and traffic conditions. Three of the pavement sections in NY were determined to be carrying 10 to 30 million ESAL's while the other three pavement sections were determined to be carrying < 3 million ESAL's.

The LTPPBind 3.1 software showed that for the three pavement sections of 10 to 30 million ESAL's, a stiffer asphalt binder should have been specified due to the Slow/Intersection traffic associated with the pavement sections. All three pavement sections had utilized a PG64-28 and all three exhibited severe flushing and in some cases severe rutting. Meanwhile, for the three lower volume pavement sections (< 3 million ESAL's), the LTPPBind 3.1 software recommended a high temperature PG grade of PG58. Therefore, the PG64-28 specified for these pavement sections should have been sufficient, assuming that the traffic level allocated for the pavement section was correct.

PG Grade of Extracted Binder and Precision/Bias

Whenever test data is analyzed and compared to the results of other laboratories or even identical test samples, it is important to consider that each test procedure has an inherit "precision and bias" that will result in test result deviations when comparing two or more test specimens. Precision and Bias statements typically accompany most AASHTO and ASTM specifications to let the user know how much deviation between test results are expected due to the inevitable equipment and user error/bias associated with the test procedure itself. Procedures with multiple steps and material handling will generally result in higher allowable deviations than test procedures with fewer steps.

It is important to consider that when conducting the solvent extraction/recovery and PG grading of asphalt binder from field cores, multiple steps are required that could ultimately influence the test results. For example, below are just the MAJOR steps necessary to obtain asphalt binder properties from a field core. In each of the MAJOR steps below, a number of smaller steps could influence the test results:

- Handling and storage of field cores
- Breaking down reheated field cores
- Type of solvent and extraction process used
- Type of recovery process used
    - Is all of the solvent out of the asphalt binder sample which would have resulted in a softer asphalt binder?
- Dynamic Shear Rheometer (DSR) testing
    - Just for the high temperature PG grade – low temperature requires further laboratory aging and Bending Beam Rheometer testing

Unfortunately, due to the multiple processes involved in the extraction/recovery and PG grading of asphalt binders from field cores, neither AASHTO nor ASTM provide a precision and bias statement. Therefore, test data obtained from the AASHTO Material Reference Laboratory (AMRL) was used to help establish what the expected repeatability of this process should be. Based on averaging the repeatability results of six (6) consecutive AMRL Proficiency Sample Rounds, the FHWA determined that the coefficient of variation for a single operator precision is slightly over 11% (Arnold et al, 2011). This means that if the high temperature PG grade of two test specimens is within 11%, then the test results are statistically equal when considering the precision and bias of the multiple step test procedure.

The FHWA repeatability results were used to compare extracted/recovered high temperature PG grade results to the quality control/quality assurance test results submitted to or conducted by NYSDOT. Generally, three sets of test data are shown in the following figures; 1)Test Results from "Retains" sampled from the asphalt plant's tank; 2) Field cores extracted/recovered and graded by NYSDOT; and 3) Field cores extracted/recovered and PG graded by Paragon Technical Services (PTS) for this study. It should be noted that there was approximately a 3 year difference in the age of the field cores between NYSDOT and PTS field cores tested, with the PTS cores taken after the NYSDOT cores.

Figures 10 through 14 show comparisons between the Retains and the extracted/recovered asphalt binders tested by NYSDOT and PTS. The high temperature PG grade of the "Retains" was used as the "Target" asphalt binder grade and the allowable range of 11% was calculated based on that value. The allowable range is indicated with the red lines in the figures. The following observations are made from the collected test data;

1. For asphalt binder supposedly specified to be a PG64-28, the high temperature PG grade of the asphalt binder from the suppliers' binder tanks (Retains) are extremely close to failing 64° C high temperature grade. It is the general practice by asphalt suppliers to ensure the performance grade temperatures well exceed the required temperature to ensure the performance grades are met even if slightly contaminated in the tanker truck or storage tank at the asphalt plant. However, in the case of the five (5) samples shown in Figures 10 through 14, 4 out of 5 Retains were very close to falling the 64° C temperature. Only the Rt 365 pavement section Retain had a high temperature PG grade comfortably above the minimum 64° C temperature (Figure 14). The other four locations (Rt 921C, Rt 20, Rt 315, Rt 12B) averaged 0.7° C higher than the 64° C required.
2. The figures clearly show that the resultant range of acceptable test results, based on the 11% coefficient of variation of the Retain values, surrounds the high temperature PG grade results for the extracted/recovered asphalt binders. All but 2 test results (two from Rt 365) from the NYSDOT and PTS extracted/recovered are found to be within the acceptable range. This would indicate that even though the NYSDOT resulted in extracted/recovered high temperature PG grades that fell below 64° C, the test results were within the expected range of repeatability.

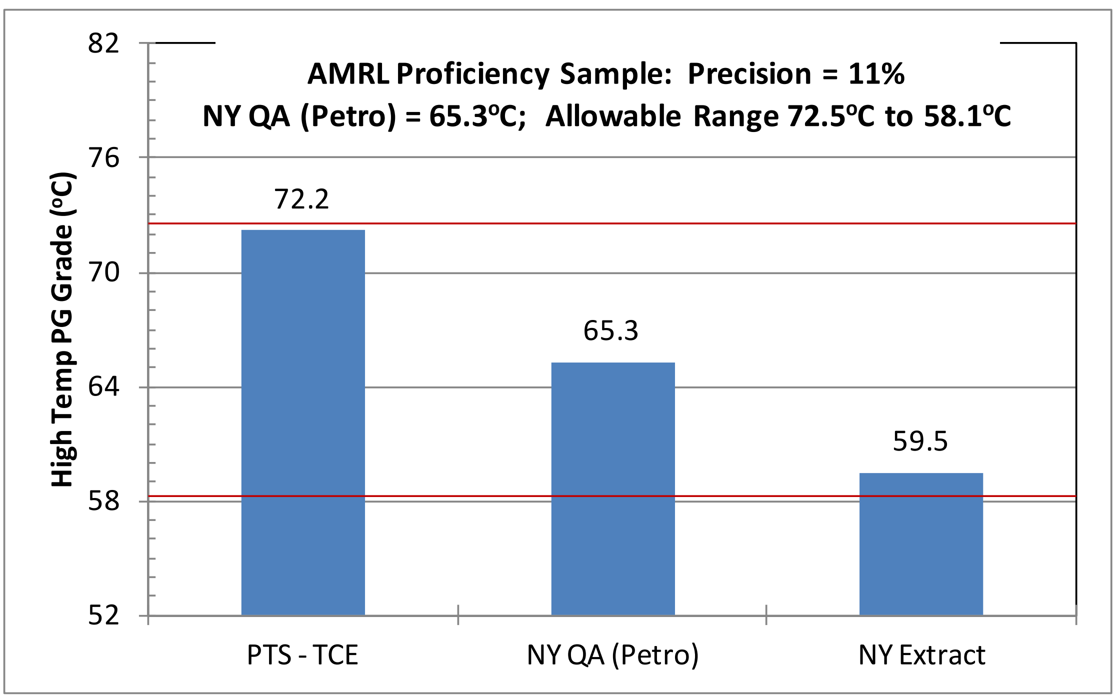


Figure 10 – Comparison of High Temperature PG Grade for Rt 921C, Utica, NY (Genesee Street)

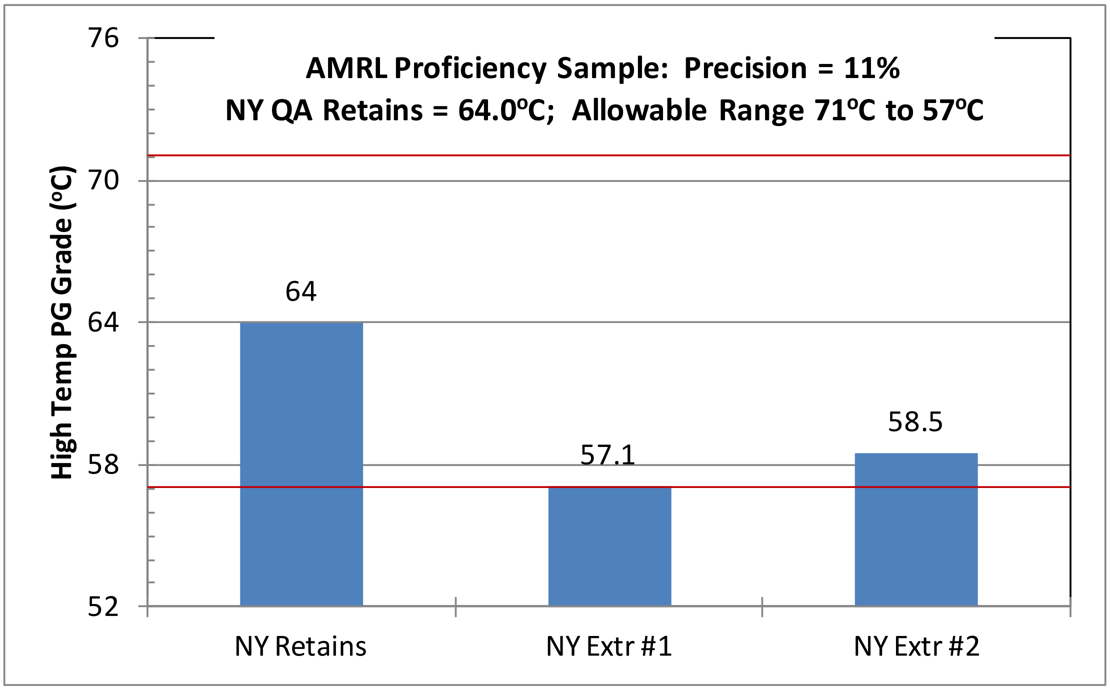


Figure 11 – Comparison of High Temperature PG Grade for Rt 20, Morrisville, NY

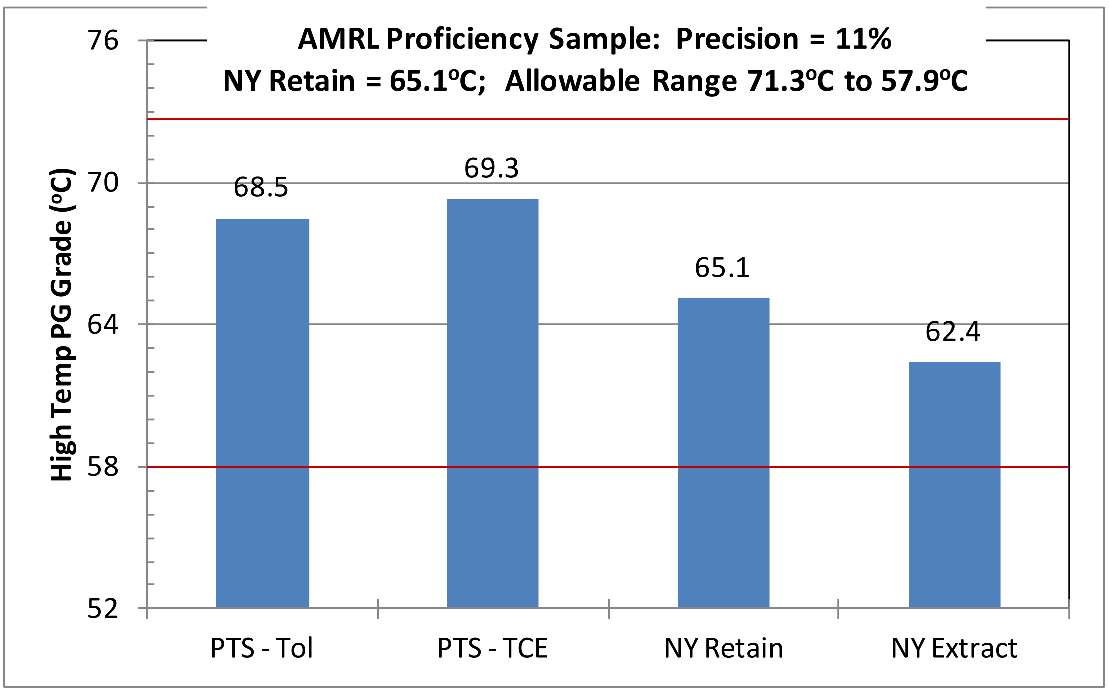


Figure 12 – Comparisons of High Temperature PG Grade for Rt 315, Deansboro, NY

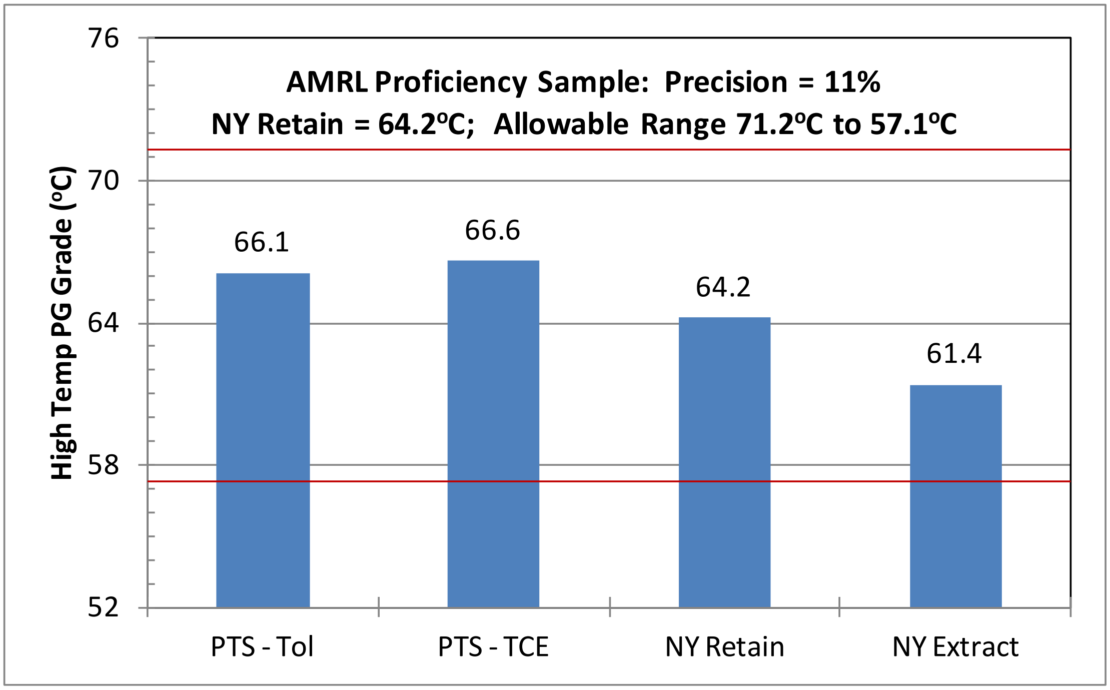


Figure 13 – Comparison of High Temperature PG Grade for Rt 12B, Oriskany Falls, NY

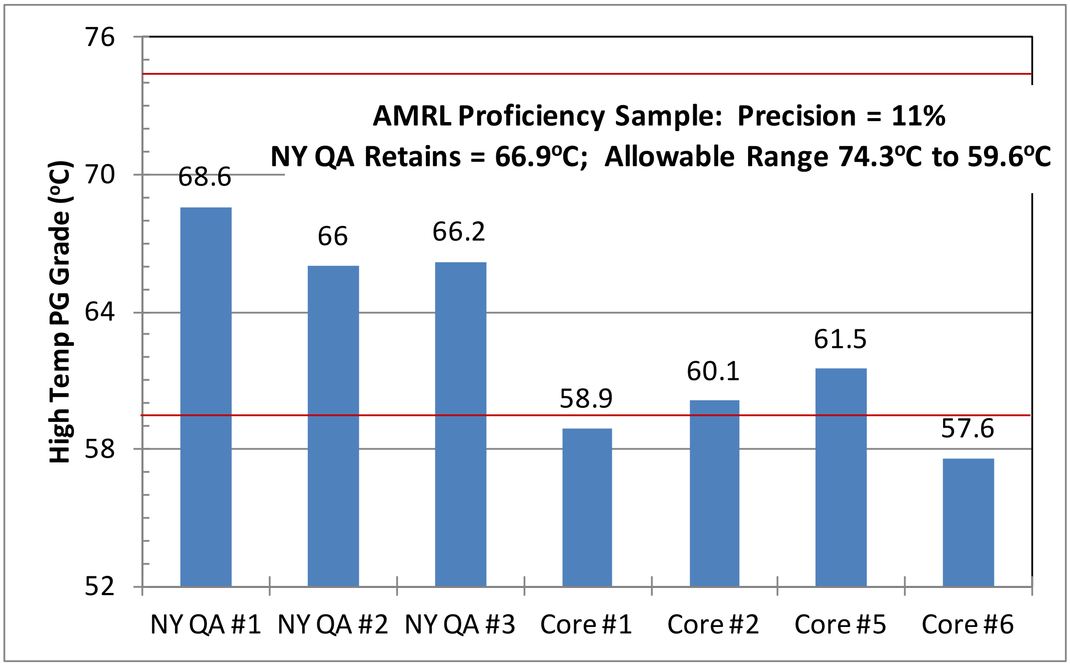


Figure 14 – Comparison of High Temperature PG Grade for Rt 365, Rome to Barneveld, NY

Effect of Extraction Solvent

As noted earlier in the Literature Review, industrial grade TCE and n-Propyl Bromide typically contain acid scavengers to help prevent corrosion in the metallic containers commonly used to store and ship with the solvents. It should be noted that discussions with NYSDOT asphalt binder laboratory technicians indicated that the NYSDOT laboratory uses TCE for all of their extraction and recovery work.

Two different solvents were used during this phase; 1) TCE and 2) Toluene. Previous work by Reinke and Glidden (2010) had suggested that TCE may act as an "acid scavenger", pulling PPA out of the asphalt binder thus reducing the asphalt binder stiffness. To evaluate this, a number of field cores were extracted using both solvents and then PG graded. The following tables (Tables 14 to 19) contain the data generated during the solvent extraction experiment. Please notes: PTS – Paragon Technical Services; QA – NYSDOT Quality Assurance (sampled from suppliers tanks during production); and Extrd – NYSDOT Extracted and recovered binders.

Table 14 - Extracted and Recovered Asphalt Binder Data from Rt 812

RT 812 Lowville				
	PTS - Toluene	**PTS - TCE**	**NYSDOT - QA**	**NYSDOT - Extrd**
Target PG Grade	**PG64-28**	**PG64-28**	**PG64-28**	**PG64-28**
True Grade	69.5-26.57	70.8-25.18	65.8	60.3
High Temp PG	PG64	PG70	PG64	PG58
Asphaltenes	19.41	20.60		
Resins	21.49	24.44		
Cyclics	51.30	49.17		
Saturates	7.80	5.79		

Table 15 - Extracted and Recovered Asphalt Binder Data from Rt 12E

RT 12E Clayton			
	PTS - TCE	**NYSDOT - QA**	**NYSDOT - Extrd**
Target PG Grade	**PG64-28**	**PG64-28**	**PG64-28**
True Grade	74.1-20.95	66.3	61.5
High Temp PG	PG70	PG64	PG58
Asphaltenes	21.69		
Resins	25.65		
Cyclics	47.90		
Saturates	4.76		

Table 16 - Extracted and Recovered Asphalt Binder Data from Rt 12B

RT 12B						
	PTS - Toluene	**PTS - TCE**	**NYSDOT - QA**	**NYSDOT - Extrd**	**TOLUENE**	**TCE**
Target PG Grade	**PG64-28**	**PG64-28**	**PG64-28**	**PG64-28**	**PG64-22**	**PG64-22**
True Grade	66.1-27.35	66.6-27.7	> 64	61.4	75.3-21.9	76.2-20.87
High Temp PG	PG64	PG64	PG64	PG58	PG70	PG76
Asphaltenes	17.79	18.10			19.53	20.10
Resins	22.84	22.19			26.67	25.44
Cyclics	54.07	53.89			48.83	50.32
Saturates	5.30	5.82			4.97	4.14

Table 17 - Extracted and Recovered Asphalt Binder Data from Rt. 315

RT 315						
	PTS - Toluene	**PTS - TCE**	**NYSDOT - QA**	**NYSDOT - Extrd**	**PTS - Toluene**	**PTS - TCE**
Target PG Grade	**PG64-28**	**PG64-28**	**PG64-28**	**PG64-28**	**PG64-22**	**PG64-22**
True Grade	68.5-27.09	69.3-25.85	> 64	62.4	77.4-20.3	77.3-19.64
High Temp PG	PG64	PG64	PG64	PG58	PG76	PG76
Asphaltenes	18.62	17.87			19.90	22.08
Resins	20.56	24.68			26.42	25.42
Cyclics	49.88	50.02			48.77	47.00
Saturates	10.94	7.43			4.91	5.50

Table 18 - Extracted and Recovered Asphalt Binder Data from Rt 921C

RT 921C				
	PTS - TCE	**NYSDOT - QA**	**NYSDOT - Extrd**	**PTS - TCE**
Target PG Grade	**PG64-28**	**PG64-28**	**PG64-28**	**PG64-22**
True Grade	72.2-25.49	---	59.5	77.5-23.37
High Temp PG	PG70		PG58	
Asphaltenes	20.07			22.39
Resins	23.29			22.15
Cyclics	46.98			48.44
Saturates	9.66			7.02

Table 19 - Extracted and Recovered Asphalt Binder Data from Rt 96B

RT 96B				
	PTS - Toluene	**PTS - TCE**	**NYSDOT - QA**	**NYSDOT - Extrd**
Target PG Grade	**PG64-28**	**PG64-28**	**PG64-28**	**PG64-28**
True Grade	70.8-25.5	75.3-21.05	64.5	---
High Temp PG	PG70	PG70	PG64-28	
Asphaltenes	18.68	20.39		
Resins	24.15	27.95		
Cyclics	50.74	44.66		
Saturates	6.43	7.00		

The test results from the extracted binders indicate that the in-place high temperature PG grade ranges between a PG64 and PG70, as determined on cores extracted during the Fall 2009. In all cases, the PG grades of the extracted binders evaluated by the Consultant were higher than those determined by the NYSDOT; on average, 9.3^{o}C higher than what NYSDOT reported when comparing the high temperature grading of the extracted asphalt binders.

Data compiled from similar work of FHWA and Mathy Technical Services (MTE) indicate that the test results of these laboratories compare more favorably to the findings of the Consultant, as opposed to the original data generated by NYSDOT. As noted in the Summary of Chapter 3, a majority of the asphalt binder high temperature PG grade was a PG 52. However, testing conducted by FHWA and Mathy Technology resulted in binder grading that was equivalent to a high temperature PG grade of 64^{o}C, not the 52^{o}C. When comparing the 2007 extracted and recovered binder properties to the Consultant's 2009 extracted and recovered binder properties, the data generated by the Consultant makes more sense with respect to what would be expected after 2 years of field aging. For most cores evaluated by the Consultant, the resultant PG grade on the extracted and recovered asphalt binder was a PG 64 for a majority of the cores with a few grading out to a high temperature of 70^{o}C.

With respect to the use of different solvents, the Consultant evaluated both Toluene, which does not contain an acid scavenger, and a laboratory research grade (RG) TCE, which contained a minimal if any amount of an acid scavenger. Using either Toluene or RG-TCE showed to have minimal effect when comparing the test results. On average, the Toluene extracted binders had a high temperature PG grade of approximately 1.3^{o}C lower than the TCE extracted asphalt binders. This is well within the expected repeatability of the test procedure and therefore these results could be considered statistically equal. Therefore, it is recommended that for future work with PPA modified binders, NYSDOT should look to utilize a non-acid scavenger solvent, such as Toluene or RG-TCE, which should have minimal detrimental effect on the PPA modified asphalt binder.

Chemical Analysis of Extracted Asphalt Binders – Asphaltene Content

General Asphaltene contents of asphalt binder typically range from around 10 (Low) to 20 – 25 (High), depending on the crude source from where the asphalt binder was refined. The Literature Review presented earlier found interesting facts pertaining to the general Asphaltene content of asphalt binder and their relative performance.

- In data presented by Daranga et al (2009), it was mentioned that asphalt liquid with higher Asphaltene contents and modified with PPA may actually age less than asphalt liquid with low Asphaltene contents. Data presented indicated an asphalt binder with an Asphaltene content around 20% resulted in lower oxidation aging than a second asphalt binder with an Asphaltene content of around 10%. Lower levels of oxidation aging may result in softer asphalt binders immediately after placement. However, lower oxidation aging is also beneficial with respect to fatigue and low temperature cracking.
- In NCHRP Report 269, it was noted that asphalt binders with Asphaltene levels lower than 10% were more susceptible to being "tender" and not setting up, which may describe some of the issues observed in the NYSDOT roads.
- Discussions with Gerald Reinke at MTE indicated that asphalt binders with higher Asphaltene contents require lesser amounts of PPA to react and modify the asphalt binder. For example, an asphalt binder with an Asphaltene content of 20% may only require 0.5% of PPA to increase one PG grade. Meanwhile, an asphalt binder with an Asphaltene content of around 10% would require around 1.5% PPA to get the same PG grade increase.

All of the asphalt binders extracted and recovered show asphaltene levels between 18 and 22% (Tables 14 through 19 shown earlier), indicating that the asphalt binder should not have had "tenderness" or set-up issues, as per the conclusions from NCHRP Report 269. However, as per what was found by Daranga et al. (2009), PPA modification to asphalt binders with higher Asphaltene contents may have reduced the oxidative aging of the asphalt binders during plant production, thereby resulting in lower high temperature PG grades in the extracted/recovered binders from the field cores. Other researchers have also noted that PPA modified asphalt binders and mixtures are less prone to oxidative aging and this can be found in the Literature Review section.

The NYSDOT also noted that one or two of the flushed sections were placed with the LEA warm mix asphalt process. Considering the production of WMA at lower temperature drastically reduces the oxidation aging during production, WMA used in conjunction with PPA modification may have resulted in lower anticipated asphalt binder high temperature stiffness. However, as noted earlier, only the NYSDOT extraction and recovery test results indicated the drastic reduction in PG grade. Both the FHWA and MTE test results, tested from cores extracted in the same time frame as the NYSDOT, contradicted the failing PG grades noted by the NYSDOT.

Chemical Analysis of Extracted Asphalt Binders – Phosphorus Presence

Another facet of the chemical analysis of the extracted and recovered asphalt binders was the determination of phosphorus present in the extracted asphalt binder. A procedure developed by Mathy Technical Services (MTE) to determine the presence of phosphorus in asphalt binder was conducted to determine if PPA could be detected in the recovered asphalt binders. The test procedure allows for;

1. The determination if phosphorus is present. If no phosphorus is present in the asphalt binder, then PPA can not be present either.
2. If phosphorus is present, the methodology allows for the estimation of the amount of 115% PPA present in the asphalt binder.

This methodology was published in the *Journal of the Association of Asphalt Paving Technologists* by Reinke and Glidden (2010).

Since the testing was decided upon after the other binder testing was conducted, the Consultant utilized any of the remaining field cores available. The laboratory testing was generally conducted on two sets of samples; 1) Extracted asphalt binder from cores containing a neat PG64-22 (no PPA was used), and 2) Extracted asphalt binder from PG64-28 cores supposedly modified with PPA. The two sets of cores (PG64-22 and PG64-28) were taken from pavement sections at the identical locations. In fact, the same mixture design and aggregates were used in both mixes – the PG64-22 mixture was used to replace the PG64-28 flushed areas. In some cases, only the PG64-22 asphalt samples were remaining and tested. The results of the testing are shown in the Table 20.

Table 20 - Results of Phosphorus Content Testing of Extracted Asphalt Binders from Flushed Pavement Sections

Pavement Section	Requested PG Grade (Solvent Used)	% Phosphorus	% of 115% PPA
Rt 921C	PG 64-28 (TCE)	0.00	0.00
Rt 921C	PG 64-22 (TCE)	0.00	0.00
Rt 315	PG 64-28 (TCE)	0.20	0.55
Rt 315	PG 64-28 (Toluene)	0.02	0.06
Rt 315	PG 64-28 (Toluene), PAV Residue	0.03	0.09
Rt 315	PG 64-22 (TCE)	0.00	0.00
Rt 315	PG 64-22 (Toluene)	0.00	0.00
Rt 96B	PG 64-28 (Toluene)	0.07	0.20
Rt 96B	PG 64-22 (TCE)	0.00	0.00
Rt 12B	PG 64-22 (Toluene)	0.00	0.00
Rt 12B	PG 64-22 (TCE)	0.00	0.00

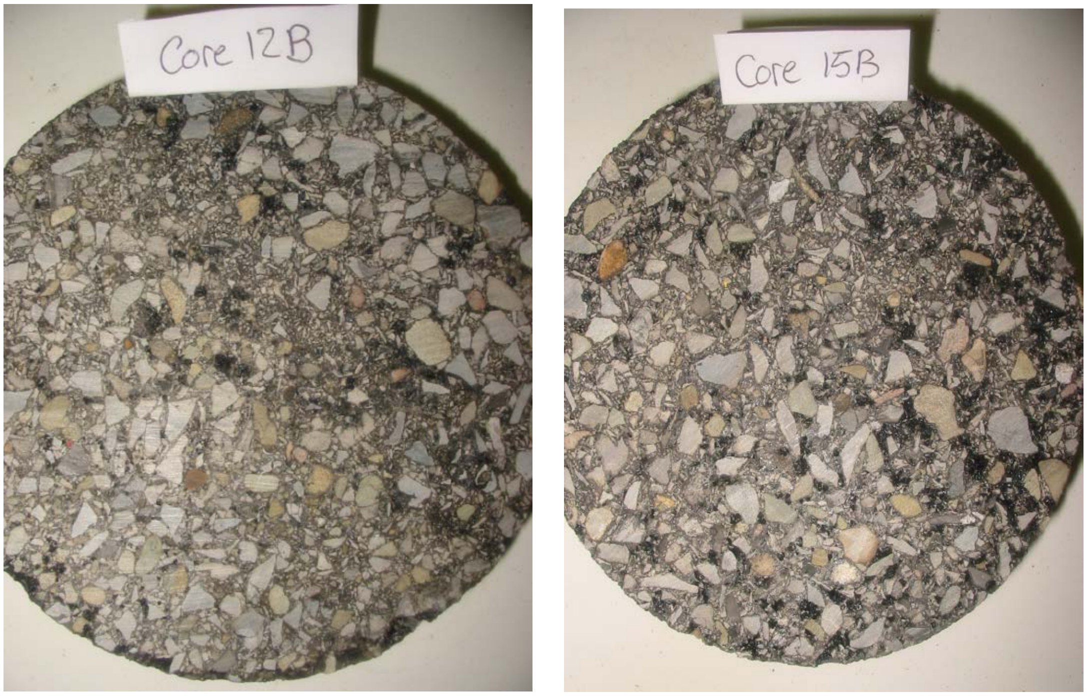
The phosphorus content testing shown in Table 20 clearly shows that for all PG64-22 asphalt binders tested, the phosphorus content was 0.00%. This would be expected as the PG64-22 was an unmodified asphalt binder. It also shows that the aggregates used in the mixture did not contain levels of phosphorus that could possibly skew the test results.

Meanwhile, the test procedure did indicate that 2 of the 3 PG64-28 asphalt binders did contain phosphorus, and therefore, most likely modified with PPA. Based on the predictive relationship developed by MTE, the %PPA ranged from approximately 0.1% to 0.55%. This would be consistent with the higher Asphaltene contents measured earlier as asphalt binders with high Asphaltene contents require lesser amounts of PPA for modification.

What is interesting to note is that for Rt 921C, 0.00% phosphorus was detected in the PG64-28. This would indicate that this asphalt binder did not contain PPA. In fact, it would be virtually impossible for an asphalt binder to contain PPA and there be no trace of phosphorus. Unfortunately, due to lack of materials, this testing could not be conducted for all cores extracted. The limited data suggests that perhaps not all of the asphalt binder in the flushed pavement sections was modified with PPA. Therefore, PPA modification may not have been the main culprit in the flushing/bleeding problem.

X-ray Tomography of Field Cores

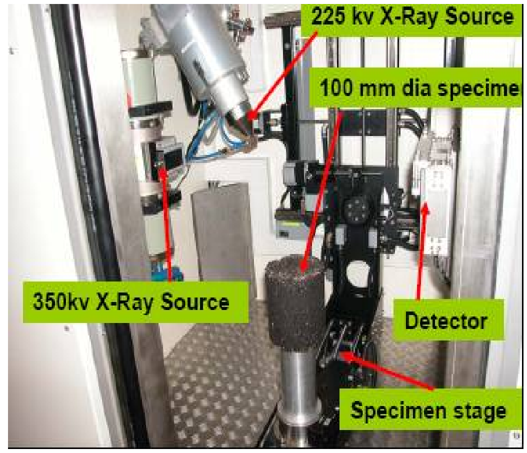
Additional cores were evaluated for void distribution and aggregate orientation using X-ray Tomography techniques. As recommended in the Nebraska Department of Roads report, *Investigating Binder Flushing of SP-2 Mixes* (Abdelraham, 2005), it is important to quantify the nature of the void distribution and aggregate structure of flushed and unflushed pavement sections. Unfortunately, traditional methods of identifying segregation require the visual observation of core slices, as shown in Figures 15a and b. Visual segregation can be seen in both photos, along with zones of higher air voids. This is a clear example of improper void distribution and poor aggregate structure. What is interesting to note is that both cores were taken from the “Atypical” flushed area (Reinke, 2009), although their exact locations are unknown. It should also be noted that according to the extracted binder results from Reinke (2009), Core 15B actually achieved the highest binder stiffness while still exhibiting the “Atypical” flushing.



(a) **(b)**

Figure 15 - Photos of Cores Taken from "Atypical" Flushed Area in Central NY State (Reinke, 2009)

In an attempt to efficiently quantify the entire core, and not just a "slice", X-ray Computed Tomography (CT) testing was conducted on selected cores from both flushed and unflushed sections of the pavement (in and outside the wheelpath). X-ray CT is a nondestructive test to capture the internal structure of materials. The X-ray CT setup at Texas A&M University is shown in Figures 16a and b. This setup includes two separate systems placed in the same shielding cabinet. The mini-focus system has a 350 kV X-ray source and a linear detector, while the micro-focus system has a 225 kV X-ray source and an area detector. The mini-focus source can penetrate thicker and denser specimens than the micro-focus source. The micro-focus system, however, has a better resolution than the mini-focus system. All the experimental measurements in this study shall be conducted using the mini-focus 350 kV X-ray source system. This system has the necessary power and resolution to penetrate the asphalt mix specimens and provide good quality images for air void/structural analysis.



(a) (b)

Figure 16 - a) Overall X-ray CT System and Shielding Cabinet; b) Mini- and Micro-focus System

In X-ray CT a test specimen is placed between an X-ray source and a detector. The intensity of X-rays change from (I_o) before entering the specimen to (I) after penetrating the specimen due to the absorption and scattering of radiation. The relationship between I_o and I is related to the linear attenuation coefficients of the materials that constitute the specimen, which are related to the densities of these materials. As such determining the attenuation coefficients allows calculating the density distribution within a specimen.

X-ray Tomography Results of Flushed Pavement Areas

The X-ray tomography result for pavement areas where flushing was identified (wheelpath areas) are shown through Figures 17 to 19. What is consistent in all of these sections is that the air void distribution is lowest at the top of the core and increases with depth. This would indicate a decrease in air voids at the top part of the pavement core, either due to consolidation or asphalt binder migration to the top area of the pavement surface/core. The plots of air voids vs height also indicates that a majority of the consolidation/binder migration took place in the upper 10 to 15 mm (0.4 to 0.6 inches) of the pavement. Below this depth, the air void distribution returns to expected levels (greater than 4% air voids).

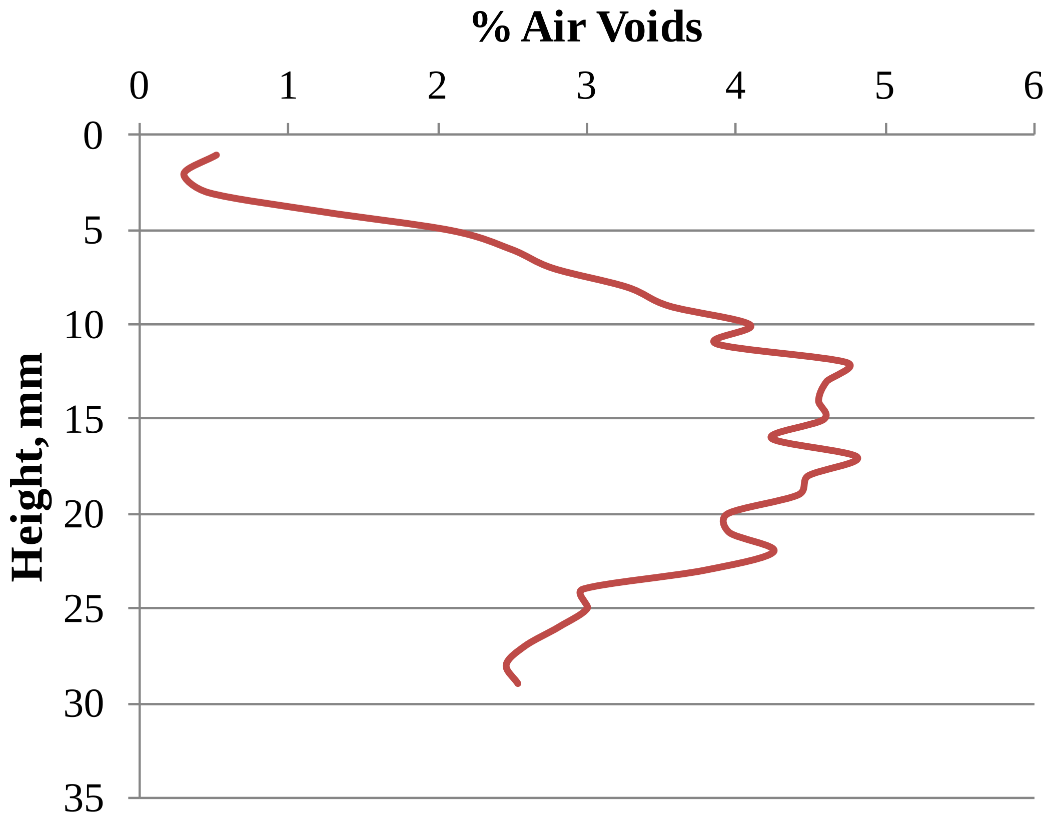


Figure 17 - Air Void Distribution of Core from the Wheelpath of Rt 315

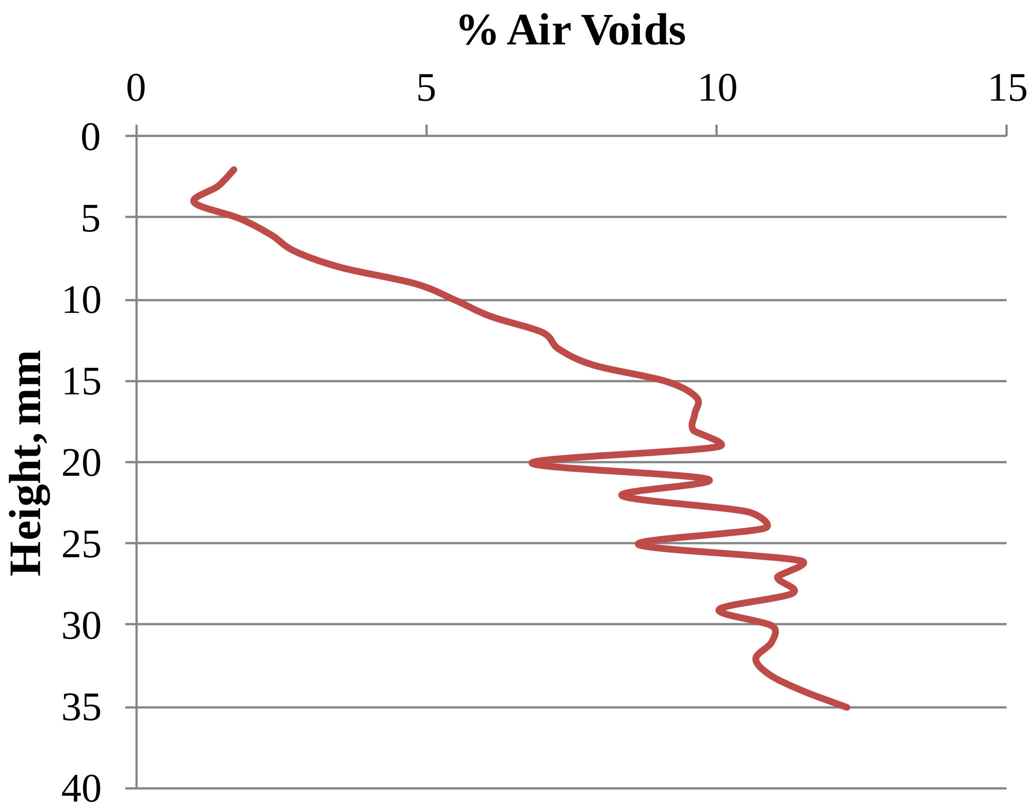


Figure 18 - Air Void Distribution of Core from the Wheelpath of Rt 96B 1095-15

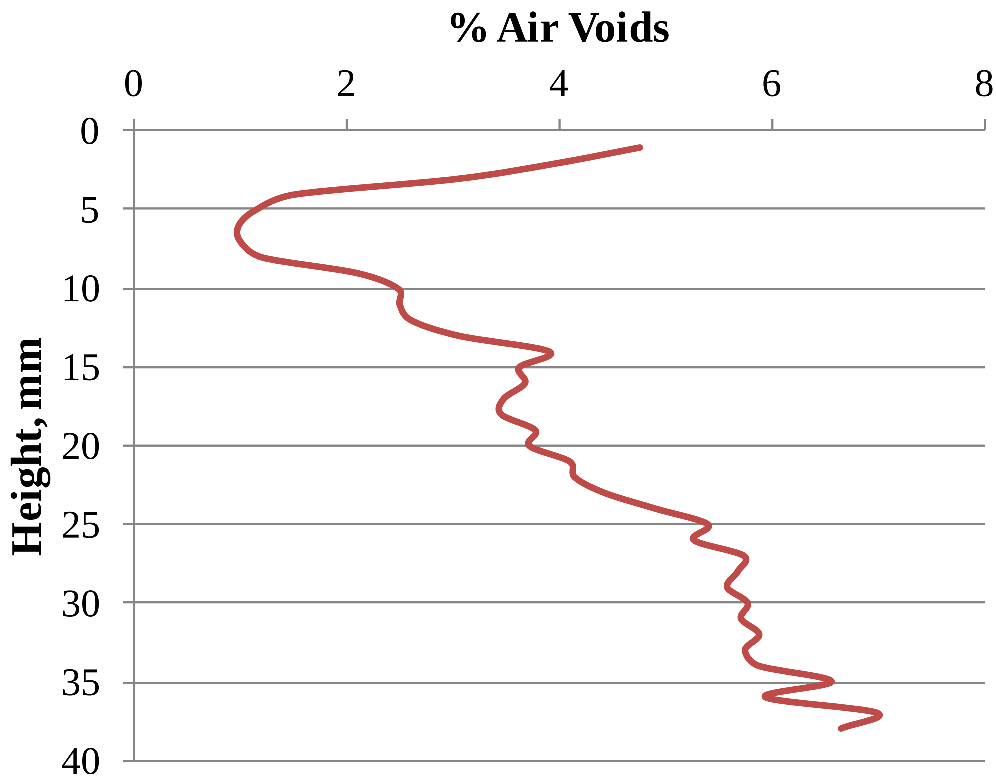


Figure 19 - Air Void Distribution of Core from the Wheelpath of Rt 96B 1095-40

<u>X-ray Tomography Results of Unflushed Pavement Areas</u>

The comparative X-ray tomography results from the unflushed areas (PG64-22 section and outside the wheelpath area) are shown in Figures 20 to 25. The air void distribution from the unflushed areas have a very different trend with a majority of the plots showing higher levels of air voids within the top 10 to 20 mm, a contrast from the air void distribution of the flushed cores.

What is interesting in Figures 24 and 25 is that one can begin to observe consolidation/densification in the PG64-22 cores, even though these were not areas where flushing was observed. Although not as dramatic as the flushed PG64-28 sections, there is a slight reduction of air voids the closer the air void distribution profile gets to the surface. Air voids as low as 2% can be seen close to the pavement surface, possibly indicating that the PG64-22 cores where in the initial process of undergoing the same flushing witnessed by the PG64-28 cores, but then stabilized and did not bleed the asphalt binder to the surface. Another more likely reason would have been that these areas on Rt 12B were compacted very tight during construction.

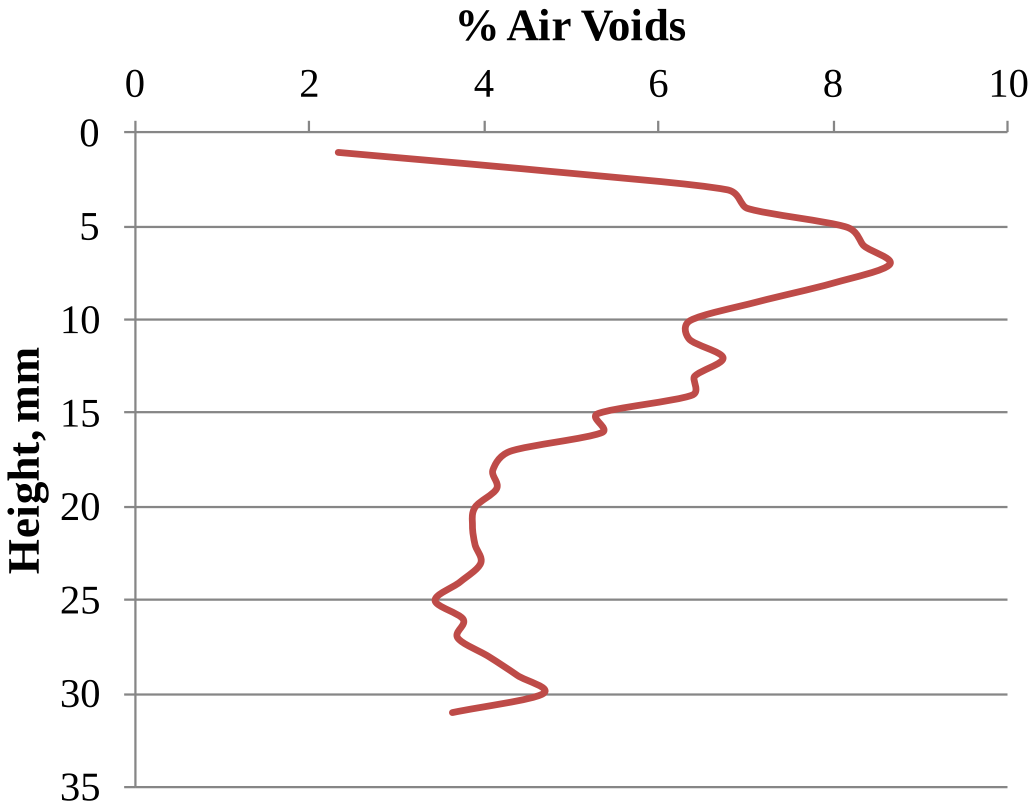


Figure 20 - Air Void Distribution of Core from Outside the Wheelpath of Rt 315

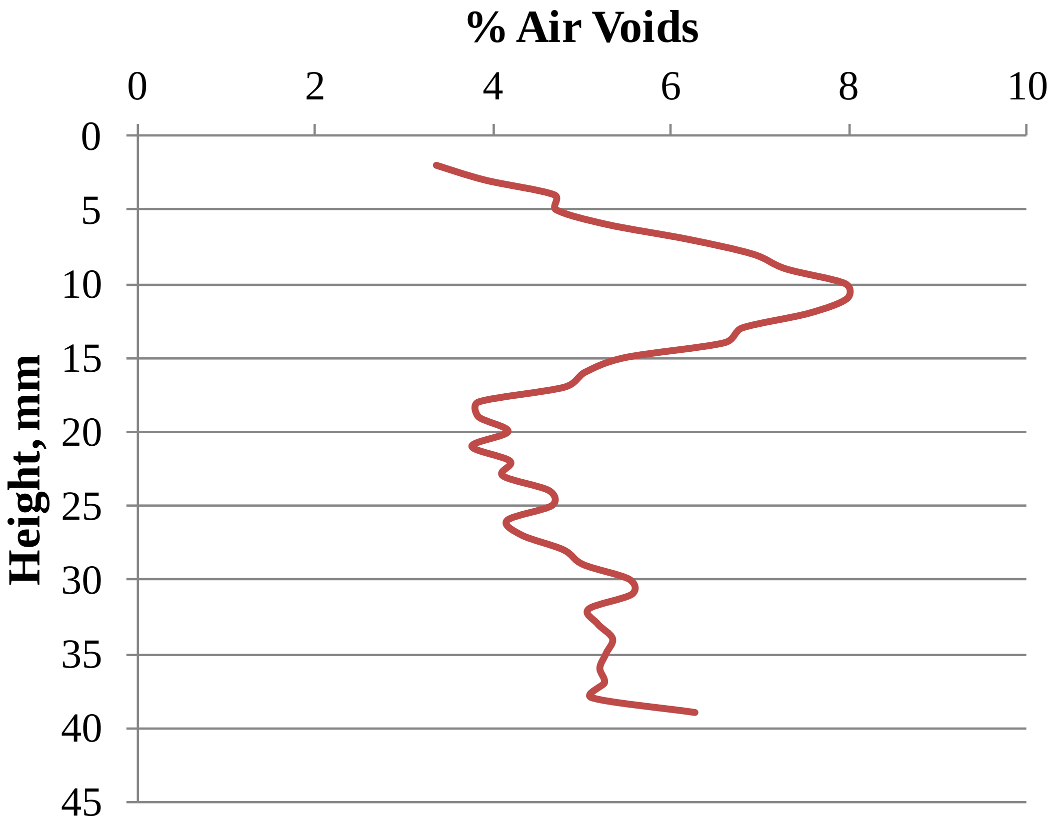


Figure 21 - Air Void Distribution of Core from Outside the Wheelpath of Rt 315

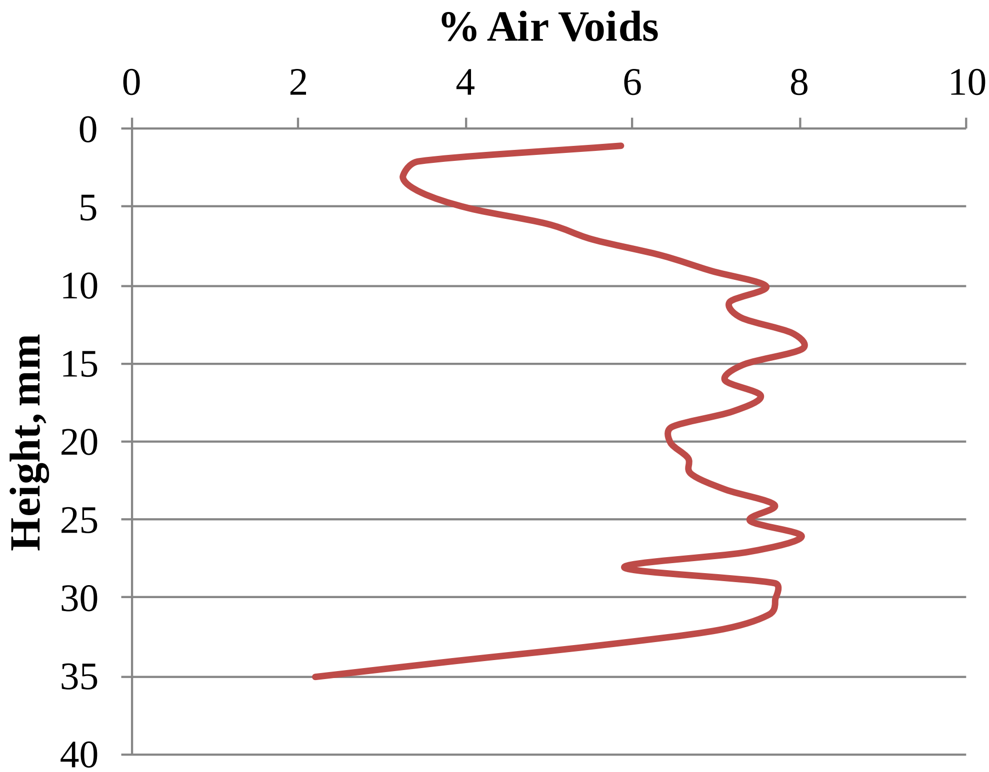


Figure 22 - Air Void Distribution from Outside of Wheelpath of Rt 96B NB 1095-15

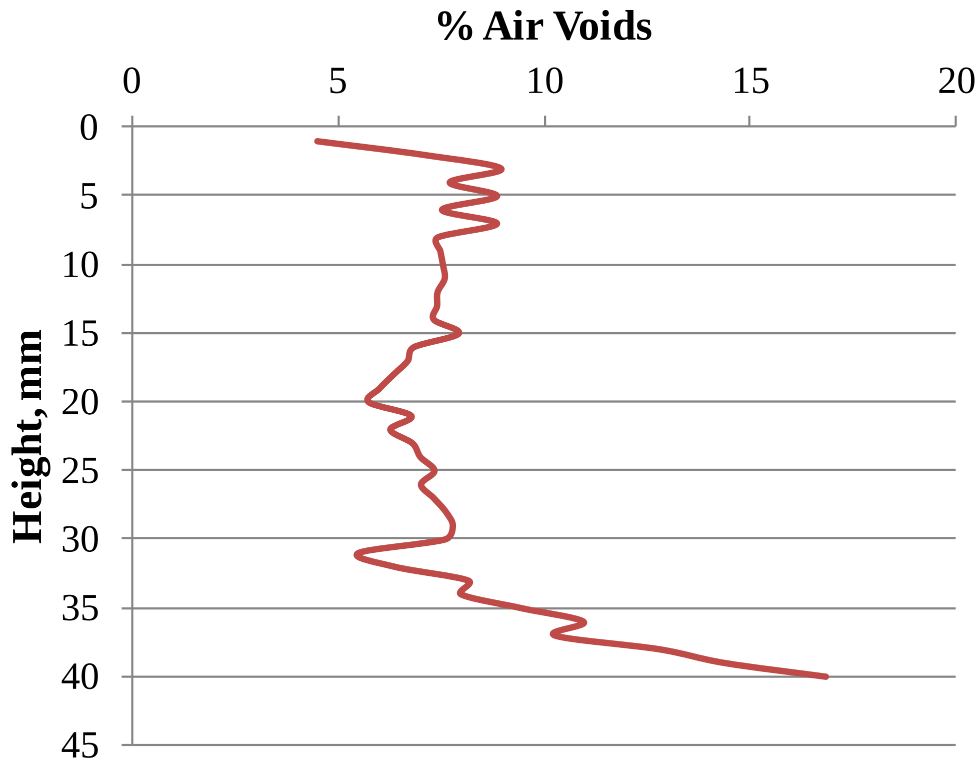


Figure 23 - Air Void Distribution from Outside of Wheelpath of Rt 96B NB 1095-40

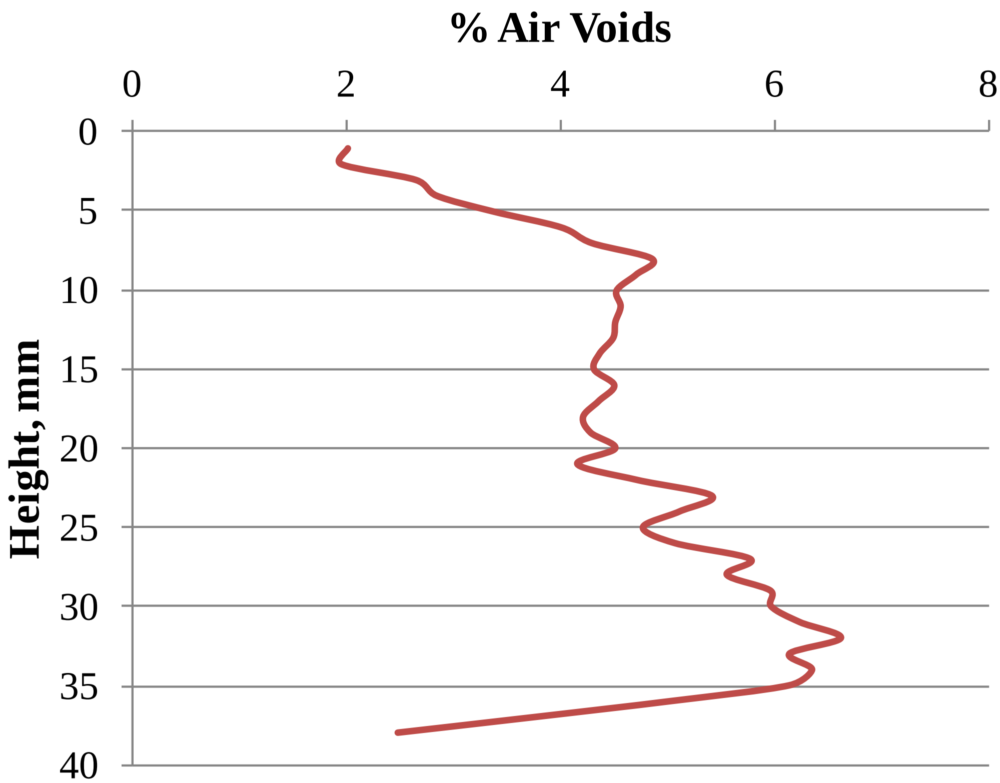


Figure 24 - Air Void Distribution from PG64-22 Section on Rt 12B – Core #38 (in the Wheelpath)

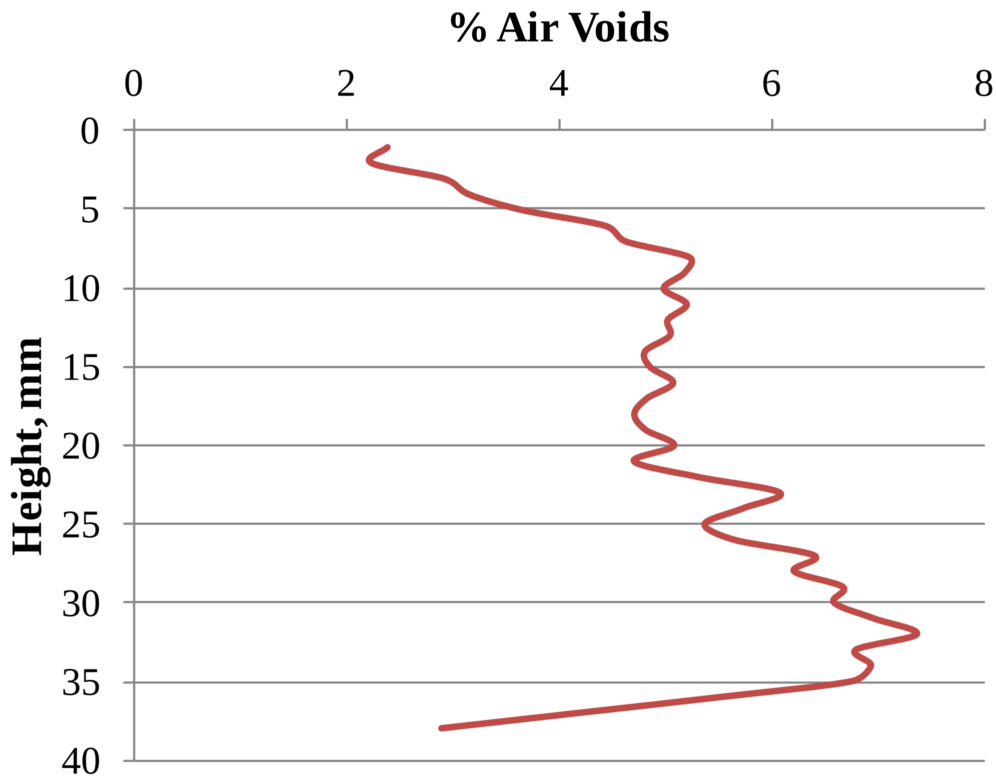


Figure 25 - Air Void Distribution from PG64-22 Section on Rt 12B – Core #42 (in the Wheelpath)

A 3-D representation of two field cores from Rt 96B, from inside and outside of the wheelpath, are shown in Figures 26 and 27. Both cores show a well distributed amount of aggregate within the core matrix. Although only one pavement location was evaluated, there is evidence suggesting that the visual observation made by Reinke (2009) may have been to just looking at a single saw cut and not represent the 3-D perspective of the field core.

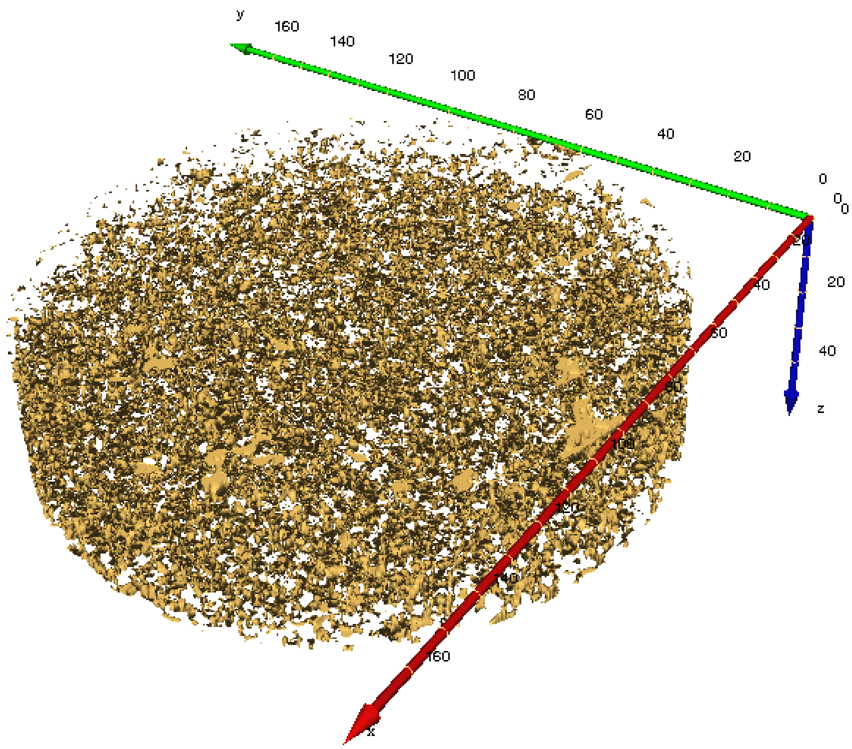


Figure 26 – 3-D Representation of Rt 96B Field Core Taken from the Wheelpath

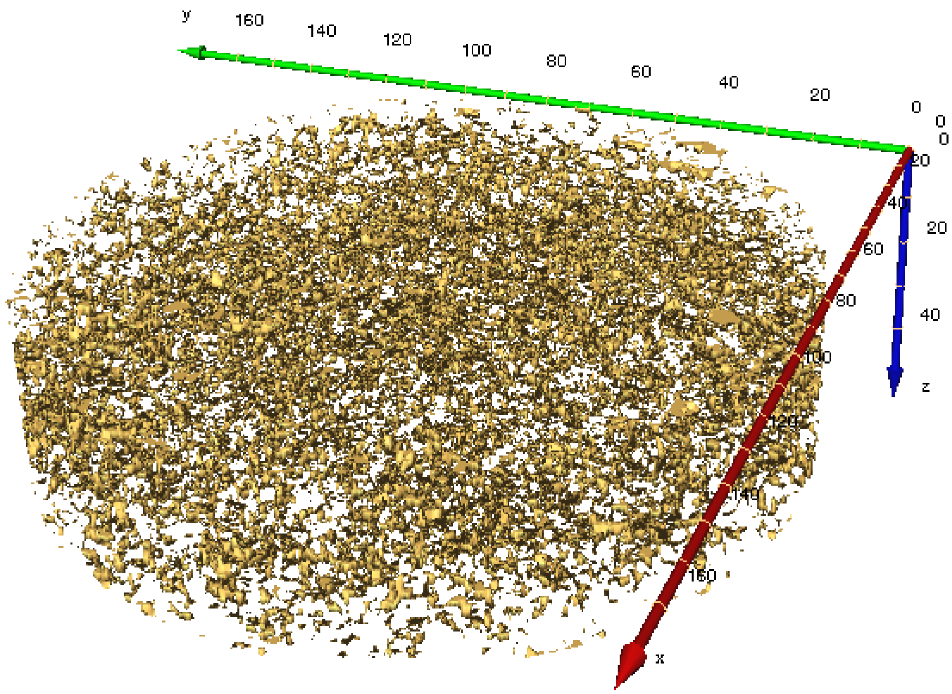


Figure 27 – 3-D Representation of Rt 96B Field Core Taken Outside of Wheelpath

CHAPTER 5 – LABORATORY MIXTURE EVALAUTION PROGRAM

Introduction

A laboratory workplan was conducted to try and replicate the “softening” or lack of curing that took place on the NYSDOT roadways. As identified in the review of SHRP A003-B Report, *Fundamental Properties of Asphalt-Aggregate Interactions Including Adhesion and Absorption*, the aging of the asphalt binder in HMA is affected by the asphalt-aggregate interaction. In an effort to evaluate this interaction, dynamic modulus testing and repeated load permanent deformation testing were conducted on asphalt mixtures at various levels of short-term aging. As noted in NCHRP Report 269, *Paving with Asphalt Produced in the 1980’s*, the resilient modulus test was found to be sensitive in evaluating stiffness tender mixes. The following levels of mixture conditioning were conducted during the laboratory testing program;

- 2 hours of aging at 135C;
- 4 hours of aging at 135C (recommended procedure of AASHTO R30, *Mixture Conditioning of Hot Mix Asphalt*); and
- 8 hours of aging at 135C.

The purpose of differing the conditioning times was to evaluate different degrees of asphalt-aggregate interaction at elevated temperatures. The dynamic modulus (E*) was measured using the Asphalt Mixture Performance Tester, AMPT (AASHTO TP79) at a high test temperature of 35^{o}C and the stiffness compared at the different conditioning times. Mixture stiffness values that achieve greater magnitudes as aging time increases would indicate higher degrees of asphalt binder aging. Mixes that show minimal changes, or even reduced stiffness, at the different aging times may indicate resistance to aging due to asphalt-aggregate interaction.

RFP NYSDOT C-08-15 indicated that rutting was noticed in association with the “Atypical” flushing. To evaluate if certain combinations of PPA concentrations and asphalt binder crude sources are prone to rutting, the Flow Number test (AASHTO TP79) was also conducted. The Flow Number test was performed in accordance with AASHTO TP79. Test specimens were cored and trimmed from large gyratory specimens to final nominal dimensions of 100 mm diameter by 150 mm-high. An applied, cyclic deviator stress of 600 kPa was applied to the test specimen. All specimens were tested at a temperature of 50^{o}C, which is approximately the average, 7-day maximum pavement temperature 20 mm from the surface, at 50 % reliability as determined using LTPPBind Version 3.1 for the locations of the original pavement sections where the flushing had occurred.

Table 21 provides Recommended Minimum Flow Number Values as a function of design traffic level based on the work in NCHRP Project 9-33. The recommended values were used for comparison purposes to determine if the designed mixtures were capable of meeting the minimum requirements for the pavement section. An average traffic speed of 40 mph was assumed when generating the minimum values shown in Table 1.

Table 21 - Minimum Flow Number Requirements (adapted from NCHRP Project 9-33)

Traffic Level *Million ESALs*	**Minimum Flow Number** *Cycles*	**General Rut Resistance**
< 3	---	Poor to Fair
3 to < 10	200	Good
10 to < 30	320	Very Good
≥ 30	580	Excellent

Materials

Asphalt Binder

A total of seven (7) different asphalt binders were produced for evaluation during the study. Three different asphalt binder suppliers provided two binders each; one binder that was a PG58-28 and modified with PPA to achieve a PG64-28 and then a second binder that contained an additional 1.0% of PPA above what was required to achieve the PG64-28.

In addition to the two binders from each binder supplier, a third asphalt binder was produced using the base asphalt from Suncor. During the initial part of the study, it was determined that the Suncor base asphalt was highly sensitive to PPA. In fact, the addition of 1.0% above what was required to produce a PG64-28 resulted in an asphalt binder PG grade of 82-10. To bring the asphalt binder back into specification for a PG64-28, a heavy Vacuum Tower Oil (VTO) from the refinery was used to soften the asphalt binder. Based on conversations with industry officials, this is one of the general procedures utilized when asphalt binders need to be softened to achieve lower PG grades. It should be noted that there was no evidence to indicate that this situation actually occurred in New York State. However, the VTO modified binder was added to simply evaluate if the addition of VTO had a detrimental impact on aggregate and asphalt binder interaction.

Table 22 shows the asphalt binders used in the laboratory experiment, along with the base PG58-28 that was blended with the different dosage rates of PPA. It should be noted that all of these asphalt binders were blended and PG graded by Paragon LLC of Richland, MS. The Non-recoverable Creep Compliance (Jnr) at 64^{o}C was conducted at the asphalt laboratory of Rutgers University.

Table 22 - Asphalt Binder Properties in Laboratory Study

Base Binder Supplier	% of PPA Used	Continuous PG Grade		Final PG Grade	J_{nr} @ 64°C
		High Temp	Low Temp		
Marathon	0% (Base)	60.0	-30.3	58-28	---
	0.9%	66.9	-29.3	64-28	3.24
	1.9%	75.2	-30.2	70-28	0.568
United	0% (Base)	60.5	-31.0	58-28	---
	0.9%	65.5	-30.6	64-28	4.29
	1.9%	74.0	-31.4	70-28	1.046
Suncor (Previously Petro Canada)	0% (Base)	60.9	-29.7	58-28	---
	0.35%	69.3	-28.5	64-28	1.146
	1.35%	85.2	-14.3	82-10	0.0286
	1.35% + VTO	65.3	-25.6	64-22	1.101

The asphalt binder Continuous High Temperature PG grade and the Non-recoverable Creep Compliance (Jnr) from the Multiple Stress Creep Recovery test play a significant role in the rutting resistance of asphalt mixtures. The High Temperature PG grade is determined in the linear-elastic range of the asphalt binder while the MSCR Jnr parameter is measured in the non-linear range of the asphalt binder. It would be assumed that although both parameters are related to one another, there may not exist an excellent correlation to one another due to the differences in strain ranges the binders are measured in. Figure 28 shows the relationship between the High Temperature PG grade and MSCR Jnr parameter for the asphalt binders used in this study. A relatively good correlation exists between these high temperature indices.

Mixture Designs and Aggregates

Five (5) different NYSDOT mixture designs were evaluated in the laboratory study. Each of the mixture designs were selected by the NYSDOT Technical Working Group (TWG). The designs and their respective aggregate components are shown in Table 23. Table 24 consists of the job mix formula gradations and asphalt content that were reproduced in the laboratory for evaluation.

Prior to compaction, the loose mix was conditioned at different times in an attempt to allow the aggregates and asphalt binders to "comingle" with one another. As noted in NCHRP Report 269, the interaction between the asphalt binder and aggregate has a significant impact on the aging of the asphalt binder itself. Since the NYSDOT engineers noted that some of the asphalt mixture placed did not "cure", or appeared to be softer than expected, extended conditioning times were imposed in an attempt to assess this interaction. The loose mix was conditioned using the guidance of AASHTO R30, *Mixture Conditioning of Hot Mix Asphalt (HMA).* Immediately after mixing, the specimens were conditioned for 2 hours at compaction temperature. Then, the asphalt mixtures were aged at three different time intervals (2, 4, and 8 hours) at a temperature of

135ºC. Immediately after the conditioning time was completed, the loose mix was brought up to compaction temperature and then compacted in the gyratory compactor. The final test specimens were compacted to a target air void level of 6 to 7% air voids.

The combination of job mix formula, asphalt binder, and mixture conditioning resulted in a total of 105 different mixture types evaluated for mixture stiffness and rutting resistance. With three test specimens required for the Dynamic Modulus and Flow Number respectively, over 600 test specimens were produced during this phase.

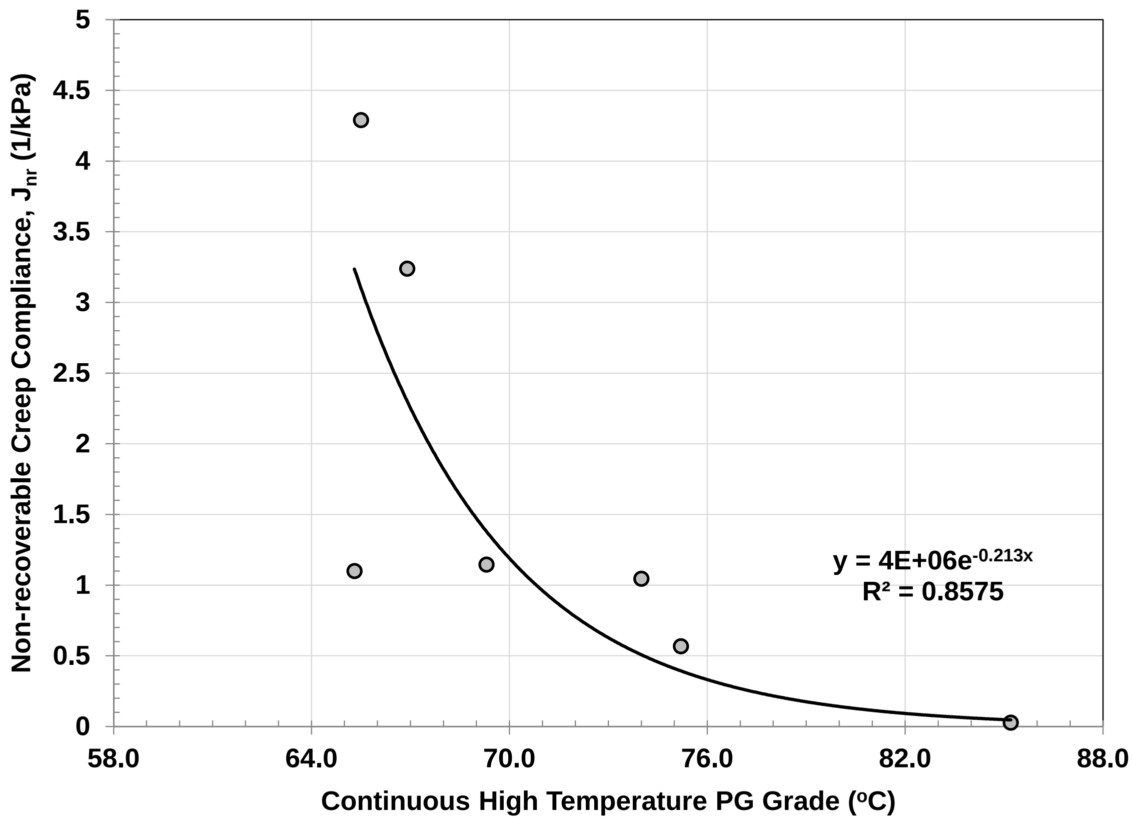


Figure 28 – Relationship Between High Temperature PG Grade and Multiple Stress Creep Recovery Jnr at 64ºC

Table 23 - Aggregate Type, Source and Percentages for Laboratory Study

Producer: Morlyn Asphalt				
Aggregates		Source Number	Aggregate Type	Blend %
Coarse	No. 1 Non-Carb	9-38R	Greywacke	11
	No. 1A Non Carb	9-38R	Greywacke	29
Fine	Man Sand	9-38R	Greywacke	60
Producer: Hanson Aggregates				
Aggregates		Source Number	Aggregate Type	Blend %
Coarse	No. 1A Stone	2-9R	Limestone	25
	No. 1A Non Carb	2-16R	Granite	25
Fine	Man Sand	2-9R	Limestone	15
	No. 1B	2-9R	Limestone	35
Producer: Suit Kote (Cortland Asphalt)				
Aggregates		Source Number	Aggregate Type	Blend %
Coarse	No. 1A Non Carb	3-20G	35% Limestone	38
Fine	Natural	3-20FB	35% Limestone	4
	No. 1B	3-20GFM	35% Limestone	58
Producer: Rochester Asphalt Materials (Dolomite)				
Aggregates		Source Number	Aggregate Type	Blend %
Coarse	No. 1 Stone	3-8RS	Dolomite	31
	No. 1A Stone	3-8RS	Dolomite	25
Fine	No. 1B	3-8RS	Dolomite	24
RAP				20
Producer: Hanson (Watertown)				
Aggregates		Source Number	Aggregate Type	Blend %
Coarse	No. 1A Stone	7-5R	Limestone	22
	No. 1A Non-Carb	7-76G	Non-carbonate	22
Fine	Manufactured	7-5R	Limestone	56

Table 24 - Job Mix Formula Reproduced for Laboratory Study

Sieve Size (mm)	Percent Passing (%)				
	Hanson (2-9R)	Dolomite (3-8R)	Suit Kote (3-20R)	Hanson (7-5R)	Morlyn (9-38R)
19	100.0	100.0	100.0	100.0	100.0
12.5	100.0	99.0	100.0	100.0	100.0
9.5	100.0	87.0	100.0	100.0	98.0
4.75	87.0	53.0	75.0	77.0	75.0
2.36	42.0	31.0	46.0	50.0	48.0
1.18	23.0	20.0	28.0	33.0	33.0
0.6	14.0	13.0	19.0	17.0	22.0
0.3	8.0	10.0	11.0	10.0	13.0
0.15	5.0	7.0	5.9	6.9	5.0
0.075	3.0	4.0	4.4	5.5	4.0
AC %	6.0	6.5	6.7	6.5	6.7
RAP %	---	20	---	---	---

Test Results

Dynamic Modulus Test (E*)

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 29). 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.

The dynamic modulus of asphalt mixtures provides an assessment of the overall stiffness properties of the asphalt mixture. Asphalt mixtures with higher stiffness' at elevated temperatures will be more rut resistant at higher temperatures. Meanwhile, asphalt mixtures with lower stiffness properties at intermediate and lower temperatures will generally be less likely to result in intermediate and low temperature cracking.

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.

Figure 29 – Photo of the Asphalt Mixture Performance Tester (AMPT) at Rutgers University

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

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)

Dynamic Modulus Test Results

The Dynamic Modulus test results, at a test temperature of 35°C and loading frequency of 0.1 Hz, are shown in Figures 30 through 34. The test results indicate that as the conditioning time increased, the small strain stiffness measured in the Dynamic Modulus increased. There did not appear to be any softening due to aggregate-asphalt interaction due to the conditioning times used in this study.

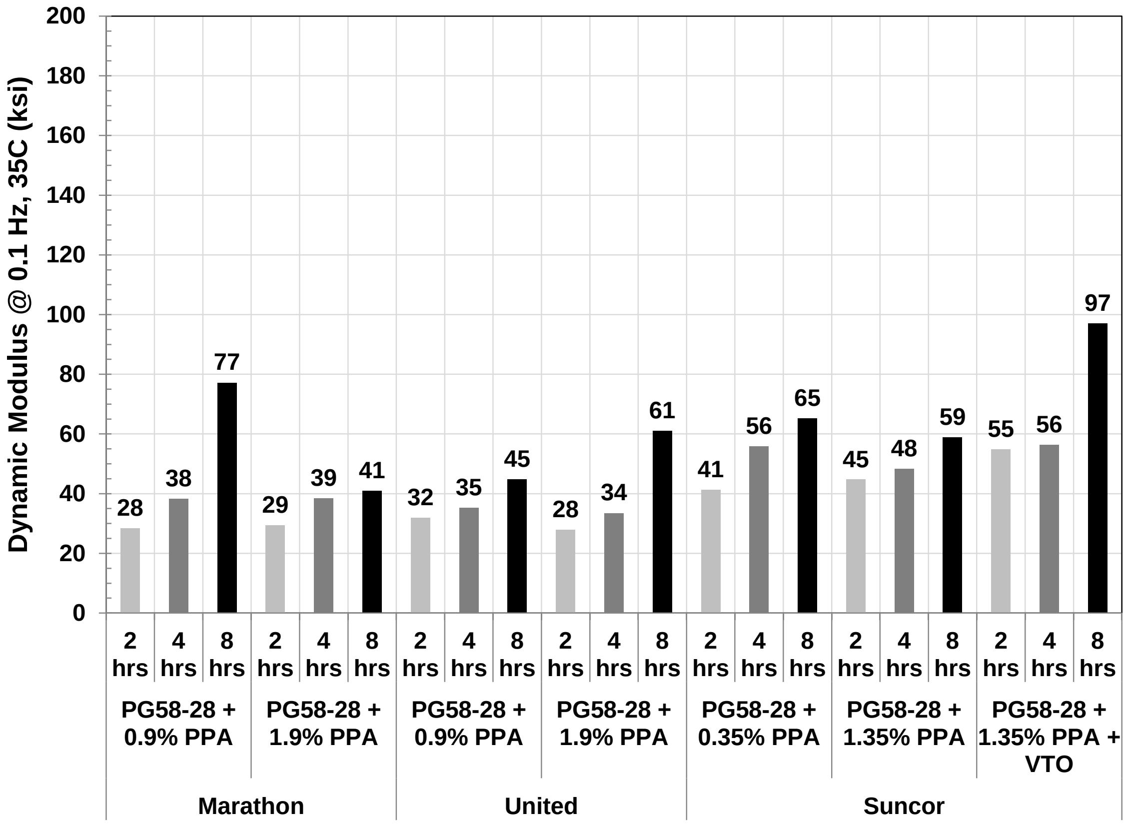


Figure 30 – Dynamic Modulus Test Results at 0.1 Hz, 35°C for Hanson Aggregates – Region 2

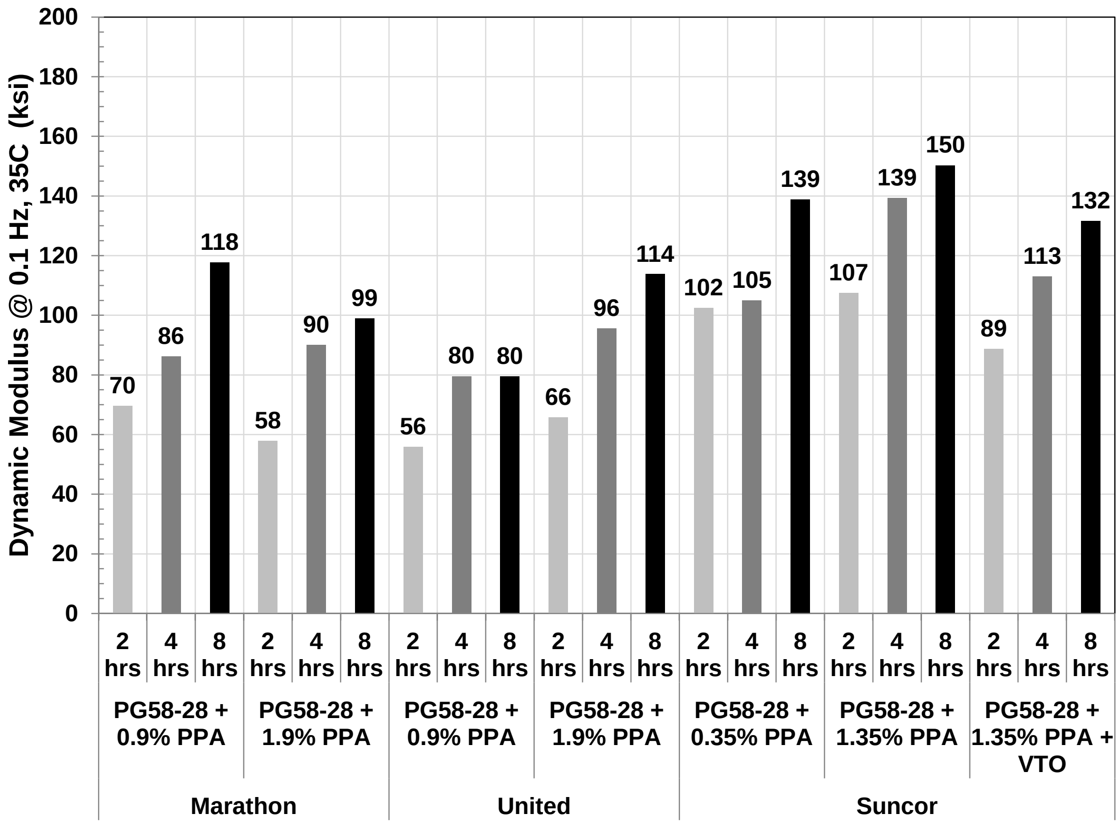


Figure 31 – Dynamic Modulus Test Results at 0.1 Hz, 35°C for Rochester (Dolomite) – Region 4

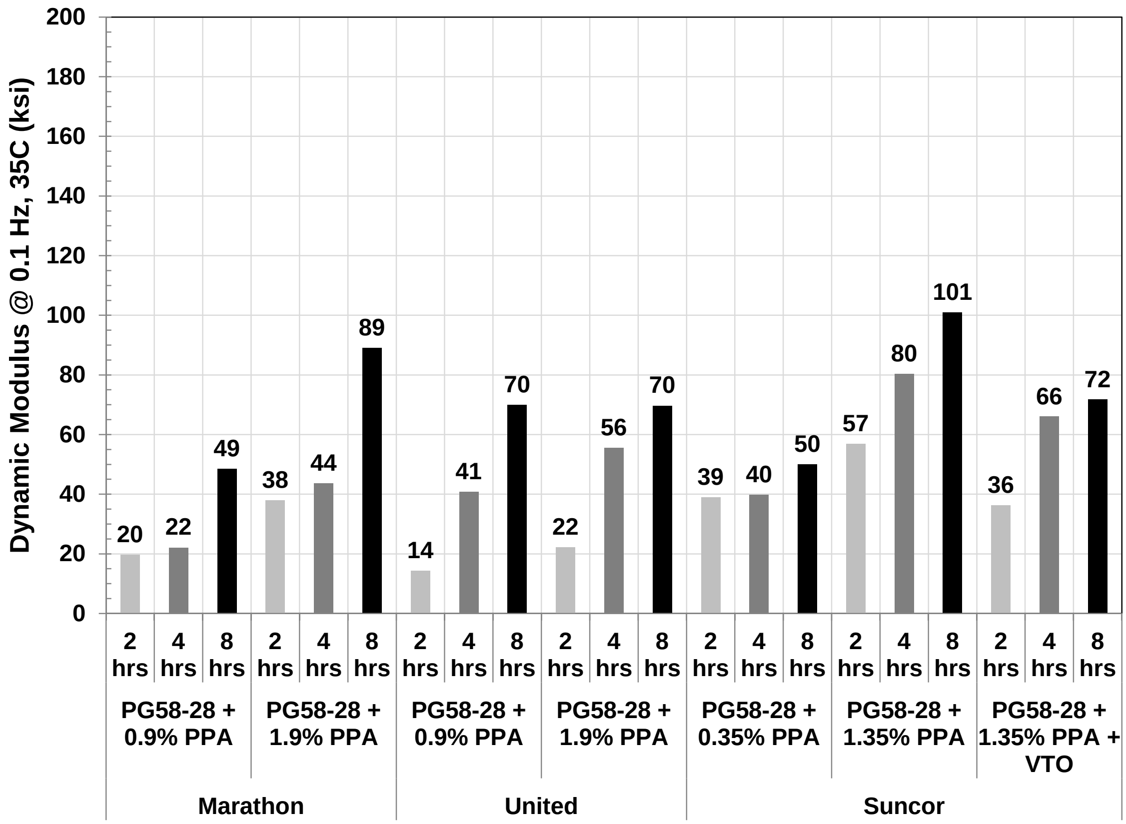


Figure 32 – Dynamic Modulus Test Results at 0.1 Hz, 35°C for Suit Kote – Region 3

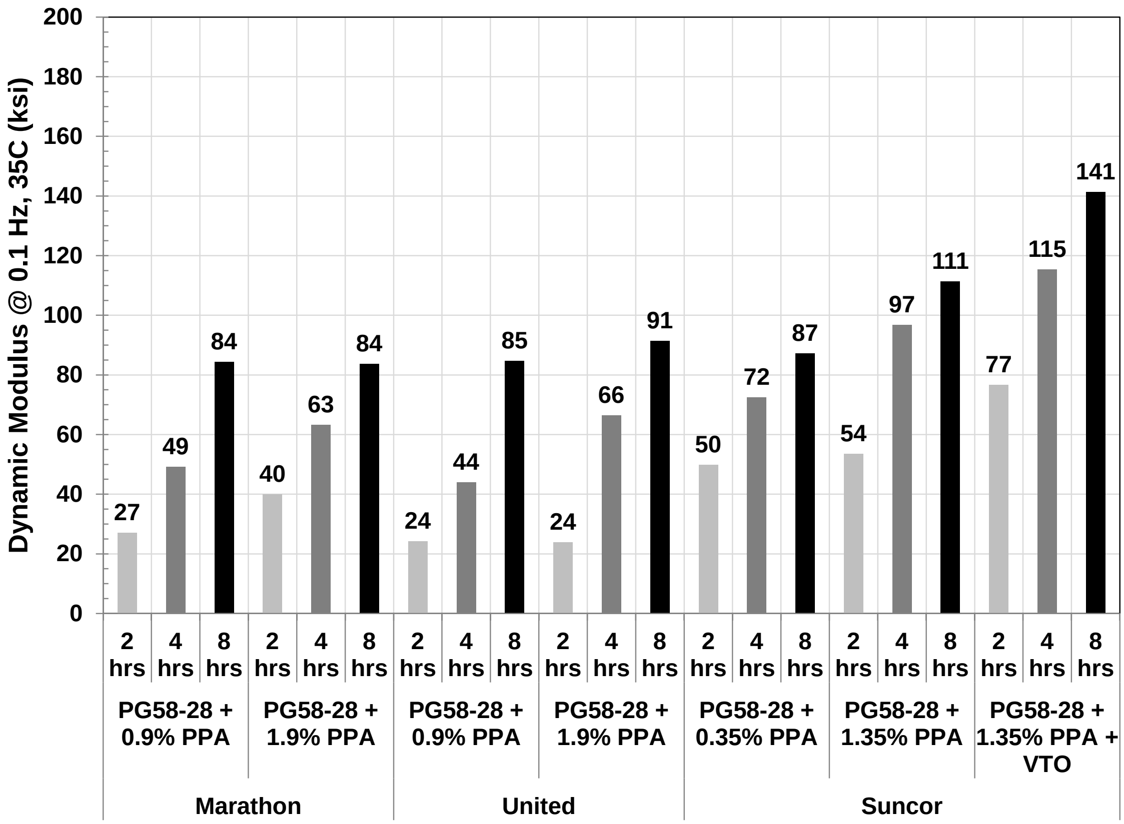


Figure 33 – Dynamic Modulus Test Results at 0.1 Hz, 35°C for Hanson Aggregates – Region 7

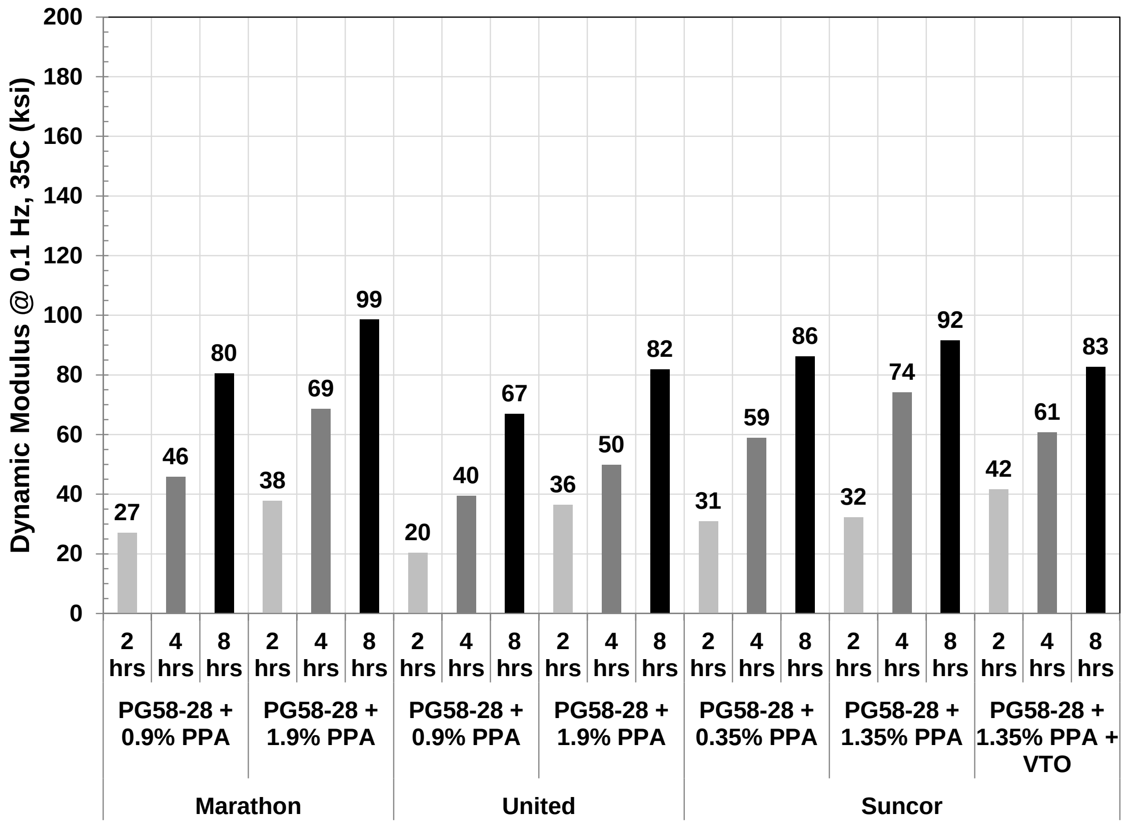


Figure 34 – Dynamic Modulus Test Results at 0.1 Hz, 35°C for Morlyn Asphalt – Region 9

Flow Number Test

The Flow Number test was conducted using an Asphalt Mixture Performance Tester in accordance with AASHTO TP79, *Determining the Dynamic Modulus and Flow Number for Hot Mix Asphalt 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 50^{o}C, which corresponds to approximately New York's average 50% reliability high pavement temperature at a depth of 20 mm according the LTPPBind 3.1 software. These testing parameters (temperature and applied stress) conform to the recommendations currently proposed in NCHRP Project 9-33, *A Mix Design Manual for Hot Mix Asphalt*. Testing was conducted until a permanent vertical strain of 5% or 10,000 cycles was obtained.

Flow Number Test Results – General Results

The overall test results for the Flow Number testing are shown in Figures 35 through 39. The test results clearly indicates that the Flow Number test results range from as low as 59 cycles to as high as over 10,000 (without showing Tertiary Flow). This was expected as a number of factors play a role in an asphalt mixtures resistance to rutting; aggregate structure, aggregate angularity, effective asphalt content, dust content, RAP content and asphalt binder high temperature properties. What is also clear from the test data is that as the conditioning time increases for the different mixtures, there is an increase in the resistance to permanent deformation (i.e. – the Flow Number increases). In only a few cases is there a slight deviation from this trend, but this is most likely the influence of the repeatability of the test procedure itself. This would indicate that the interaction between the aggregate and asphalt binders used in the study did not result in a condition where the asphalt binder did not age or soften, due to extended exposure to the aggregates at elevated temperatures.

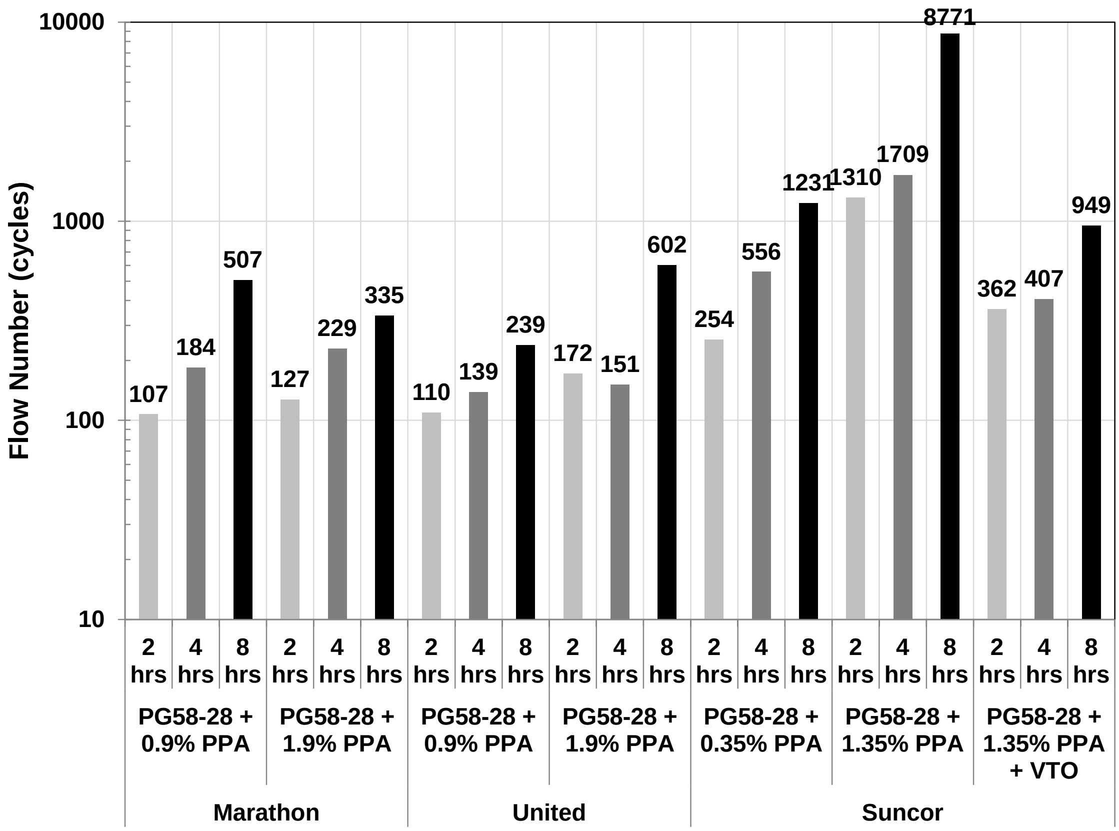


Figure 35 – Flow Number Test Results for Hanson Aggregates – Region 2

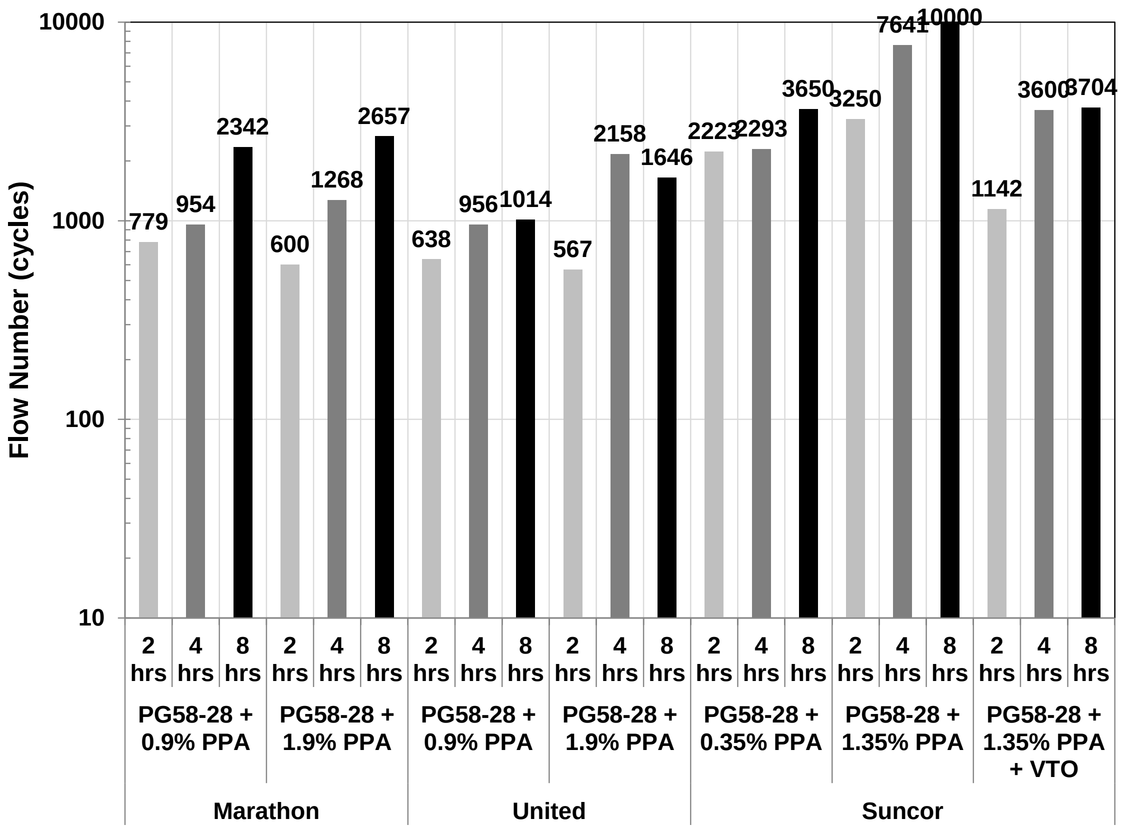


Figure 36 – Flow Number Test Results for Rochester (Dolomite) – Region 4

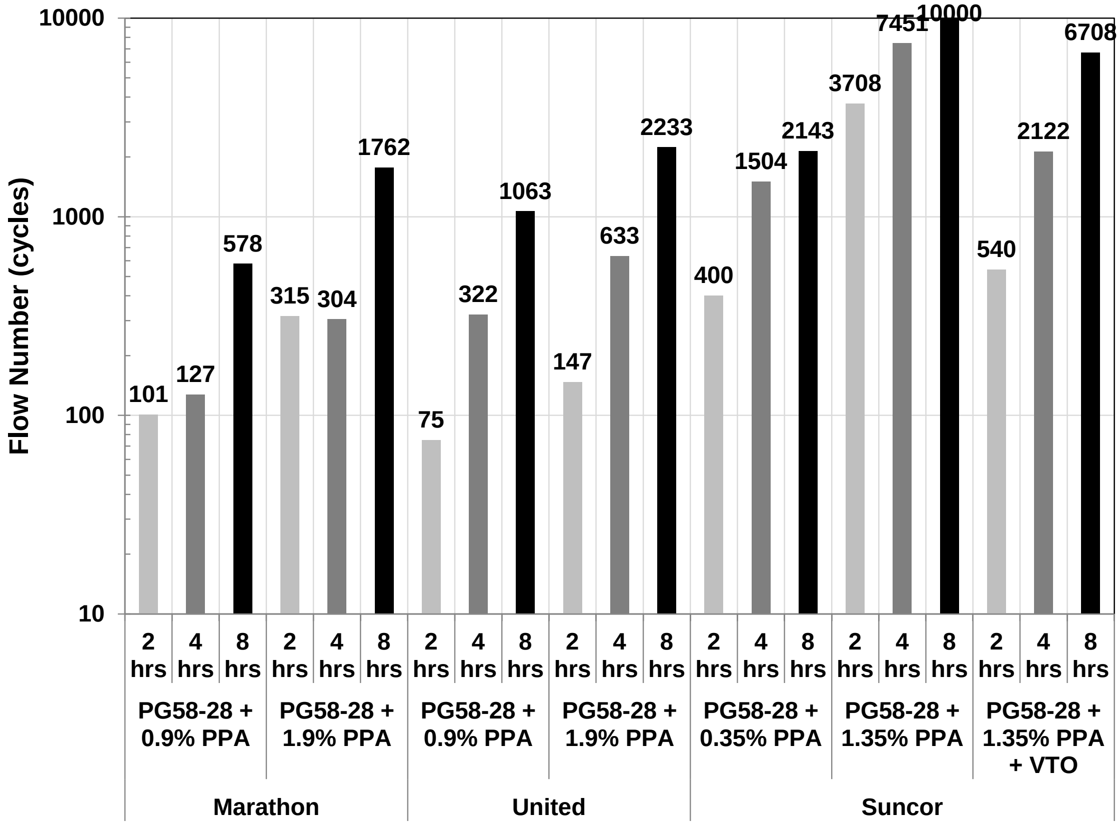


Figure 37 – Flow Number Test Results for Suit Kote – Region 3

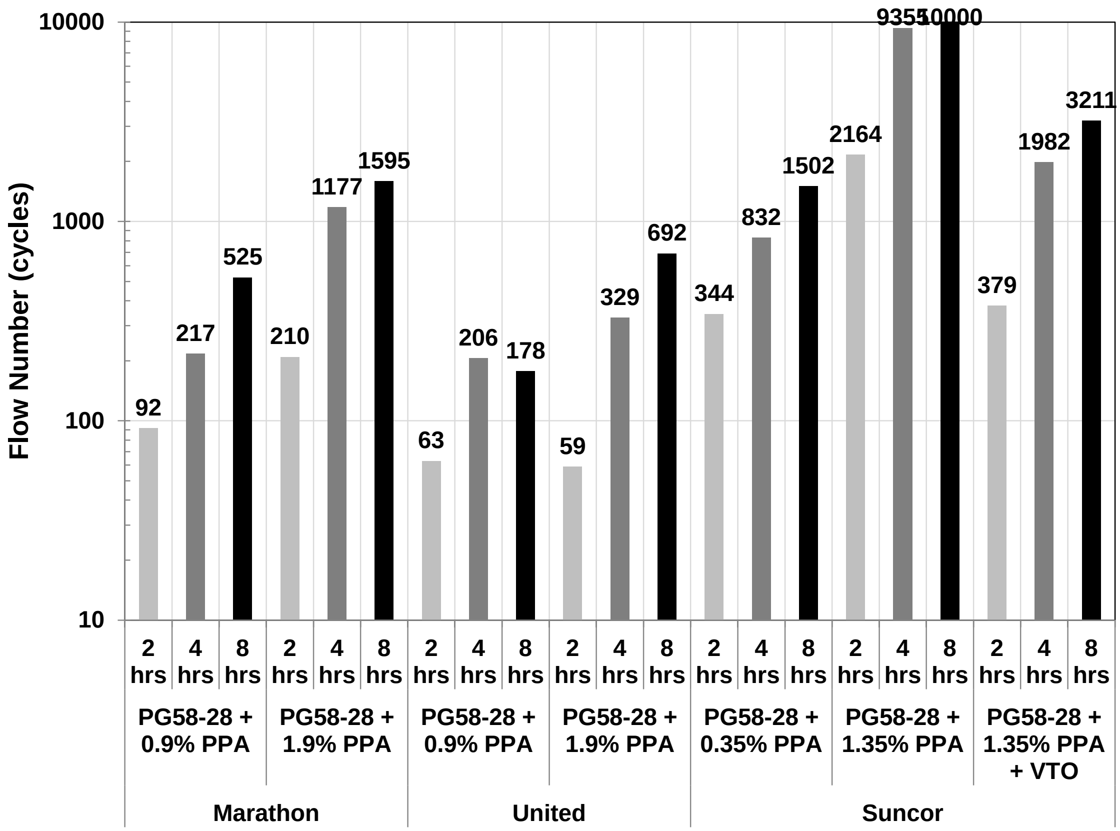


Figure 38 – Flow Number Test Results for Hanson Aggregates – Region 7

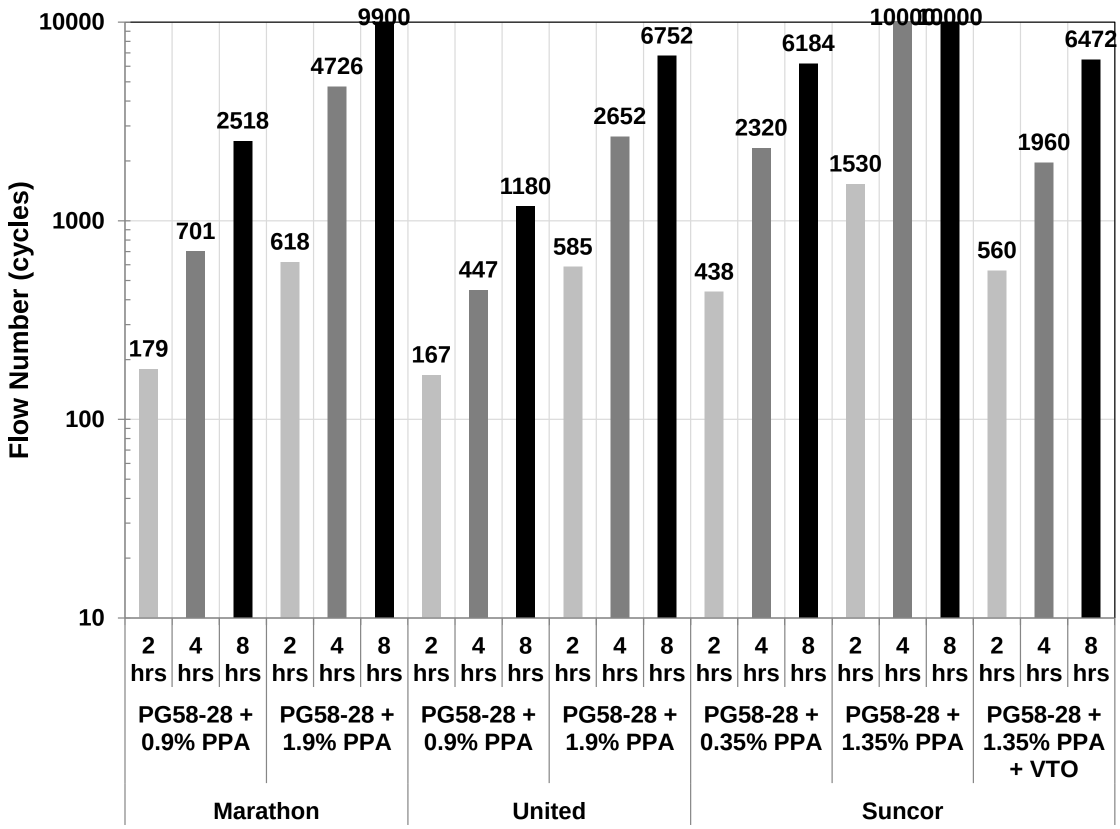


Figure 39 – Flow Number Test Results for Morlyn Asphalt – Region 9

The AMPT Flow Number results were plotted against the Non-recoverable Creep Compliance (J_{nr}) of the different asphalt binders. The test data was presented in this form to determine if the measured Flow Number values trended with the MSCR J_{nr} value. It would be expected that if there was a "softening" of the asphalt binder due to the interaction of some of the asphalt binder sources, then there would not be a trend between the asphalt binder J_{nr} value and the Flow Number value of the mixtures.

The results of the Flow Number vs MSCR J_{nr} for the different conditioning times are shown in Figures 40 through 44. The data in the figures show a trend of an increasing Flow Number with a decreasing MSCR J_{nr} value, which would be expected if the MSCR J_{nr} parameter influenced permanent deformation performance. The data in the figures also verifies that as the conditioning time increases, the Flow Number of the mixtures increases as well. This further demonstrates that the interaction between the aggregates and asphalt binder sources selected in this study do not result in a softening of mixture performance at high temperatures.

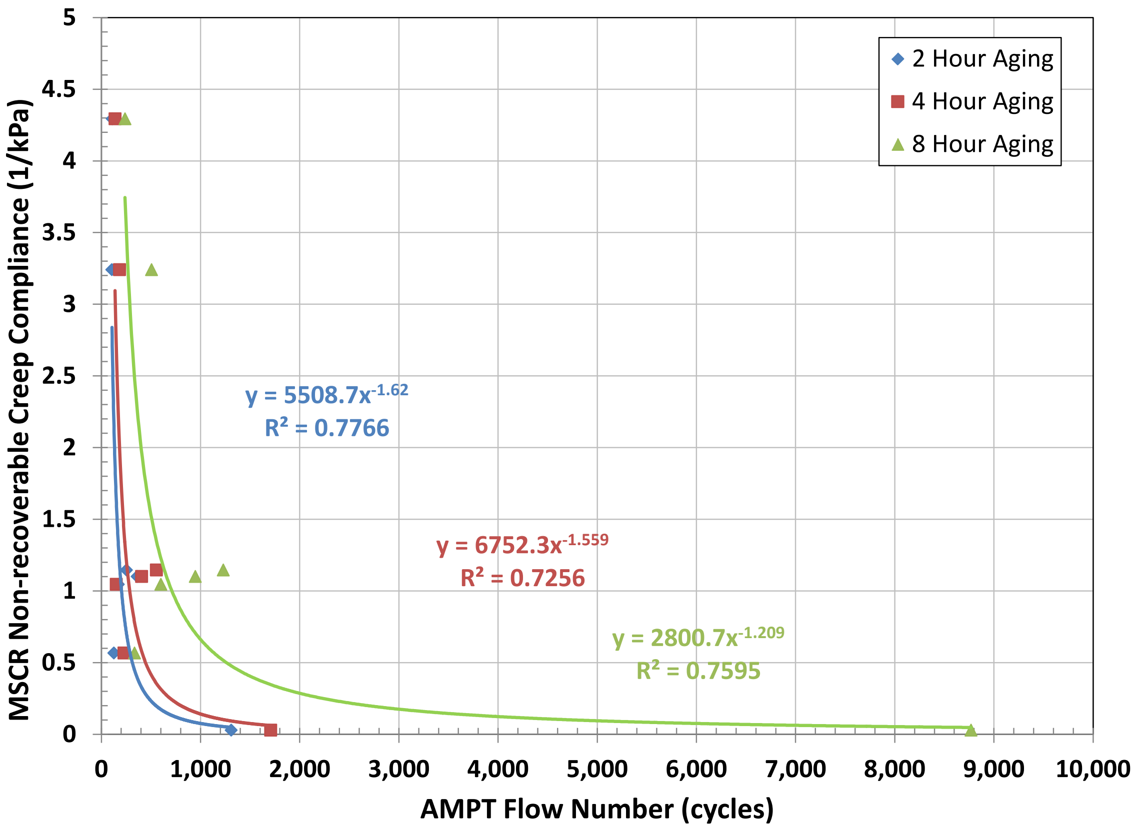


Figure 40 – AMPT Flow Number vs MSCR Jnr for Hanson Aggregates – Region 2

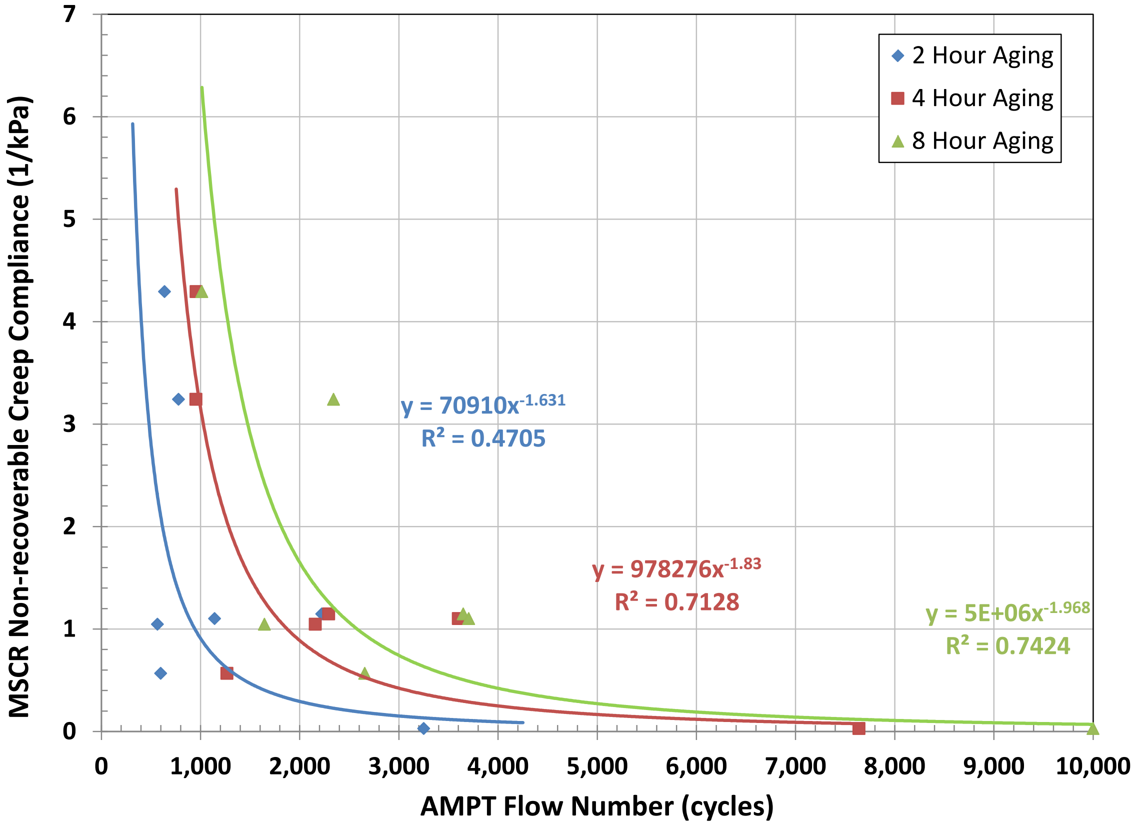


Figure 41 – AMPT Flow Number vs MSCR Jnr for Rochester (Dolomite) – Region 4

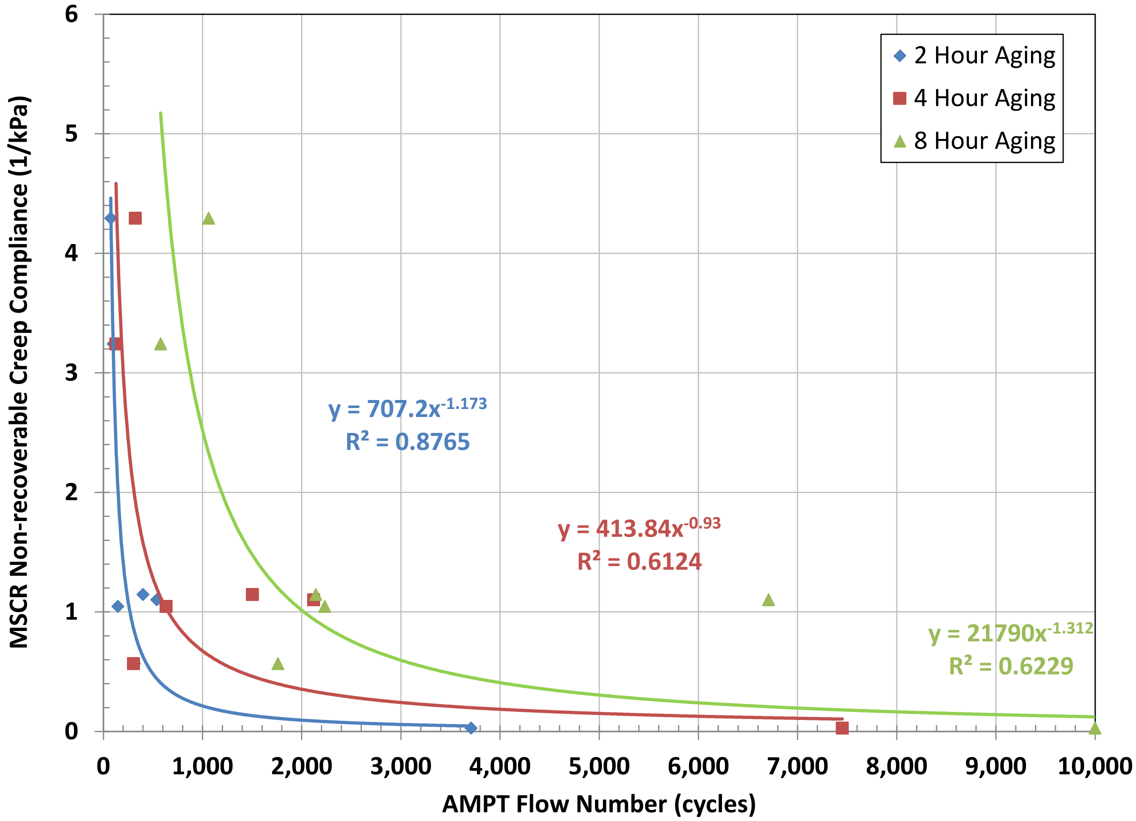


Figure 42 – AMPT Flow Number vs MSCR Jnr for Suit Kote – Region 3

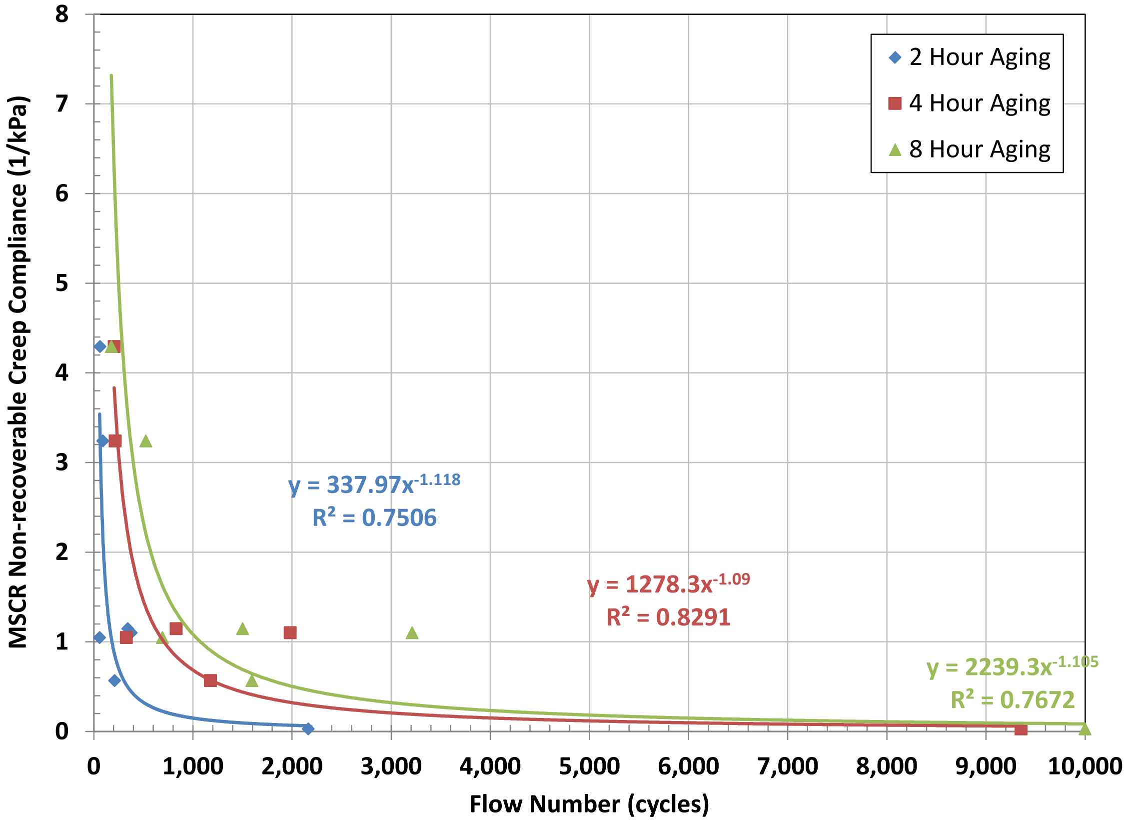


Figure 43 – AMPT Flow Numbers vs MSCR Jnr for Hanson Aggregates – Region 7

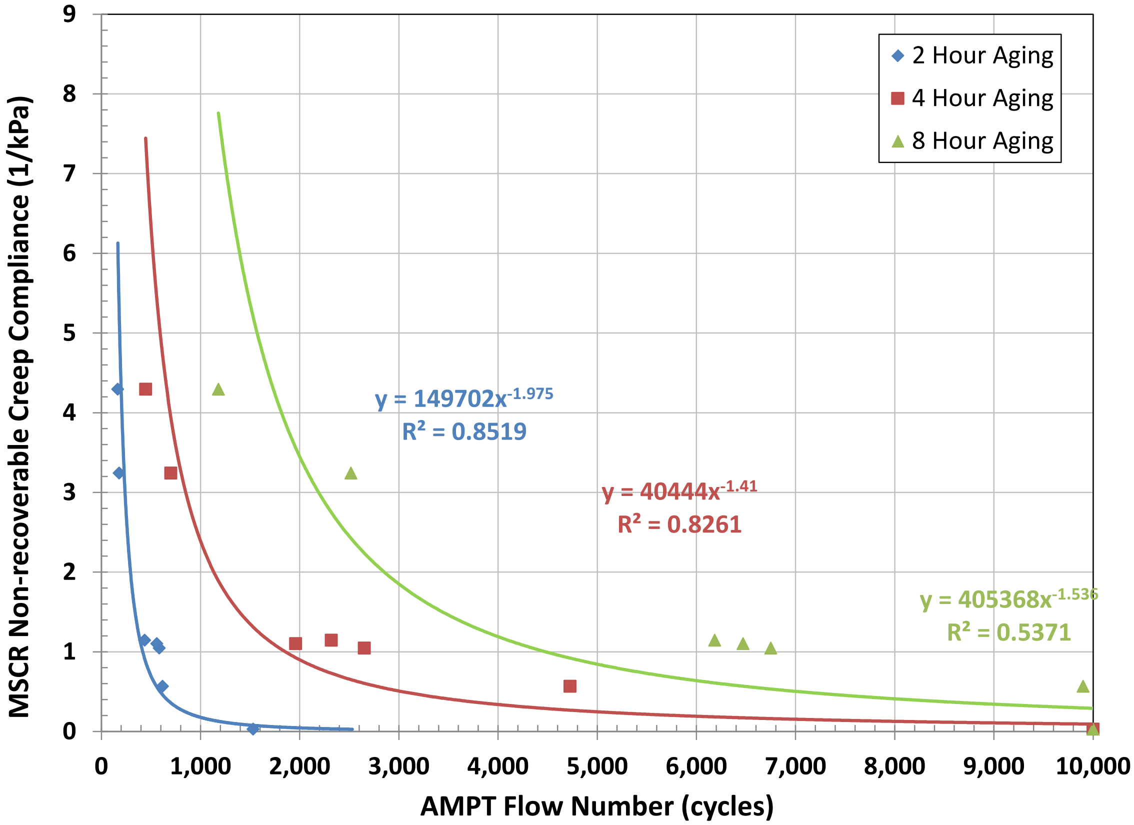


Figure 44 – AMPT Flow Numbers vs MSCR Jnr for Morlyn Asphalt – Region 9

Flow Number Results – Appropriate Rutting Resistance vs Traffic Levels

The recommended Flow Number values, shown earlier in Table 21, were used to compare to the measured Flow Numbers for the different mixtures evaluated in the study. According to the work conducted in the NCHRP 9-33 study;

> "AMPT specimens used to evaluate rutting resistance should be prepared to the expected average field air void content at the time of construction, not the design air void content. Mixtures should be short-term aged for 4 hours at 135°C in accordance with the procedure for Short-Term Conditioning for Mixture Mechanical Property Testing in AASHTO R30."

Therefore, to compare the AMPT Flow Number results from the study to the recommended values from NCHRP 9-33, the results for only the 4 hour conditioning will be shown.

The AMPT Flow Number results for the mixtures that had undergone the 4 hours of conditioning at 135°C are shown in Figures 45 through 49. The test results are further compared to the Recommended Minimum Flow Number values shown earlier in Table 21.

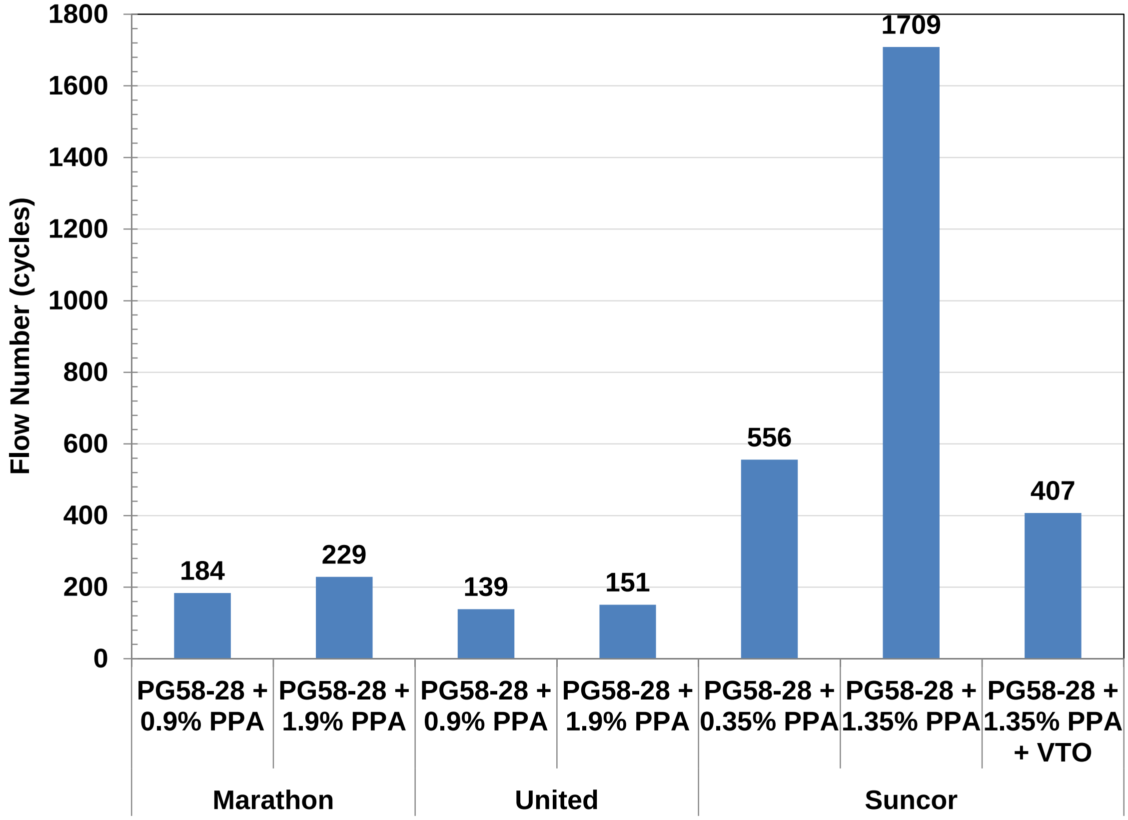


Figure 45 – AMPT Flow Number Results for Short-Term Conditioning for Mixture Mechanical Property Testing (AASHTO R30) – Hanson Aggregates Region 2

Flow Number vs Traffic – Hanson Aggregates Region 2

According to the Job Mix Formula information provided for the Hanson Aggregates Region 2 mixture, the traffic level the mixture was designed for was less than < 30 million ESAL's. For this traffic level, a minimum Flow Number value of 320 cycles is required to achieve good rutting performance in the field, as recommended by NCHRP 9-33 project. As shown in Figure 45, the asphalt mixtures designed and constructed with the Marathon and United asphalt binders would not have resulted in good rutting performance.

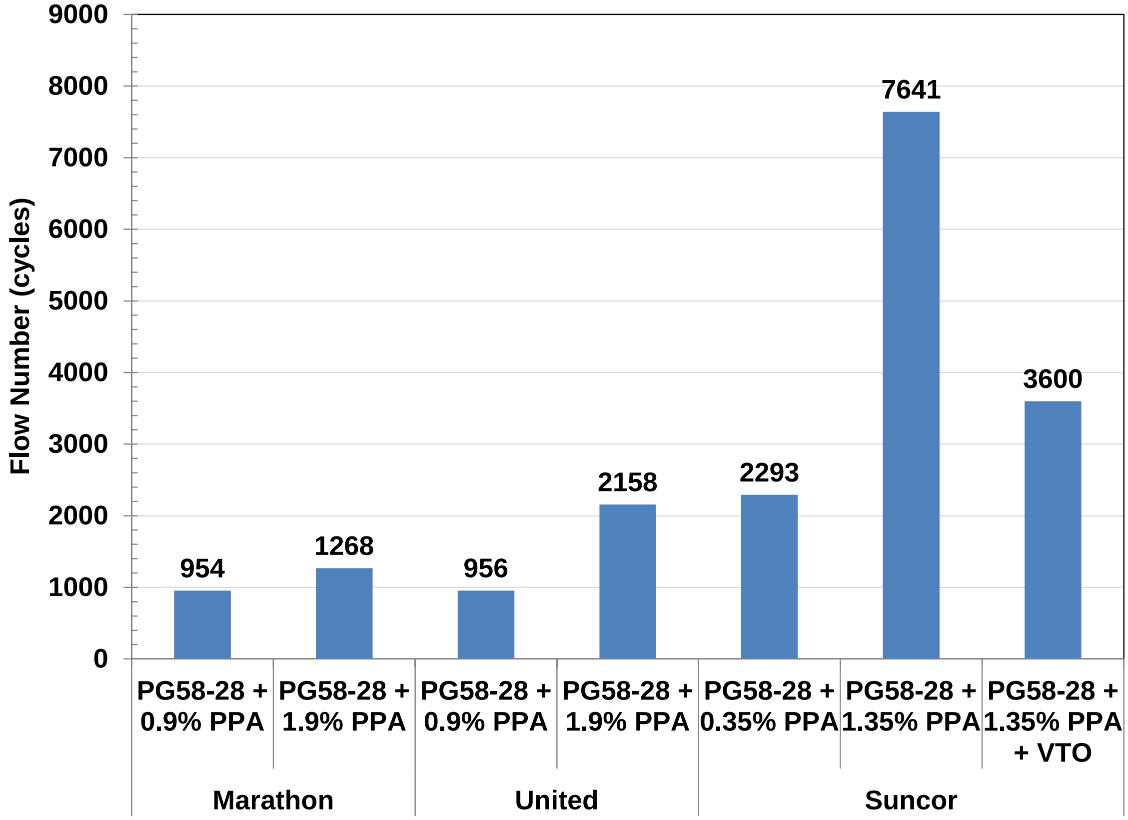


Figure 46 – AMPT Flow Number Results for Short-Term Conditioning for Mixture Mechanical Property Testing (AASHTO R30) – Rochester (Dolomite) Region 3

Flow Number vs Traffic – Rochester (Dolomite) Aggregates Region 3

According to the Job Mix Formula information provided for the Rochester (Dolomite) Region 3 mixture, the traffic level the mixture was designed for was less than < 30 million ESAL's. For this traffic level, a minimum Flow Number value of 320 cycles is required to achieve good rutting performance in the field, as recommended by NCHRP 9-33 project. As shown in Figure 46, all asphalt binders/mixtures evaluated should have achieved good field performance with respect to rutting. It should be noted that this mixture was the only mixture containing RAP in the study (20% by total weight of the mixture).

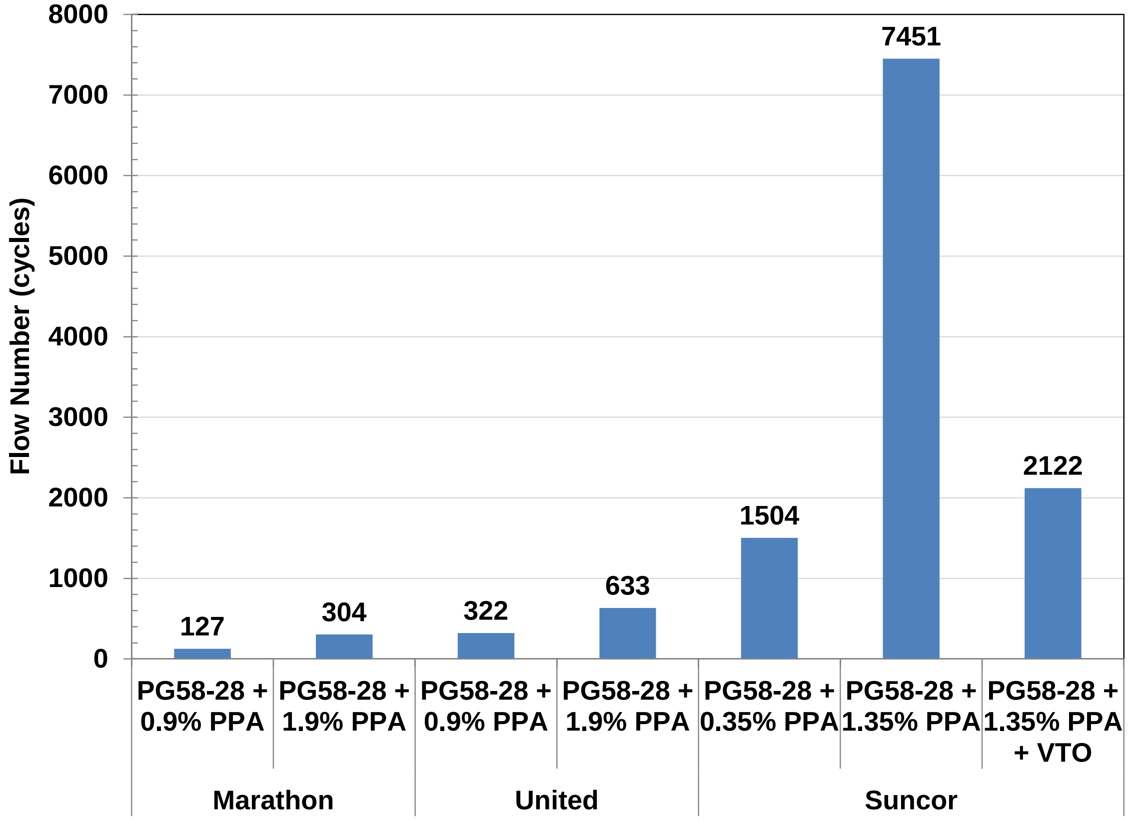


Figure 47 – AMPT Flow Number Results for Short-Term Conditioning for Mixture Mechanical Property Testing (AASHTO R30) – Suit Kote Region 3

Flow Number vs Traffic – Suit Kote Aggregates Region 3

According to the Job Mix Formula information provided for the Suit Kote Region 3 mixture, the traffic level the mixture was designed for was less than (<) 3 Million ESAL's. For this traffic level, there is no minimum Flow Number – with the assumption being that most asphalt mixtures, if designed using the Superpave principles, should perform under the low traffic conditions. A majority of the asphalt binders/mixtures evaluated for this mixture performed well, although the Marathon binder with 0.9% PPA did result in a rather low Flow Number value when compared to the other asphalt binders (Figure 47).

However, it should be noted that the concept of ESAL's does not include slow traffic speed/intersections areas, where a majority of rutting occurs. Therefore, even though Table 21 notes that there is no minimum Flow Number requirement for < 3 million ESAL's, pavement sections with intersections and slow moving traffic should be designed with some type of minimum requirement.

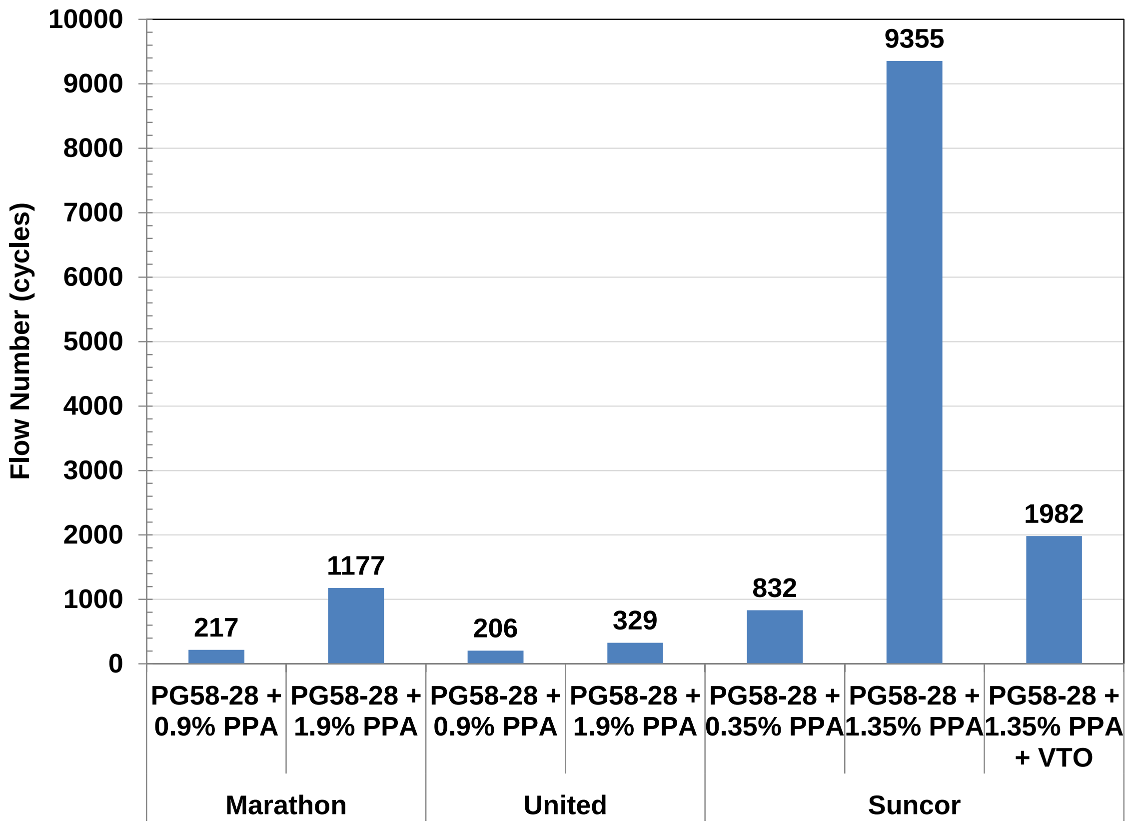


Figure 48 – AMPT Flow Number Results for Short-Term Conditioning for Mixture Mechanical Property Testing (AASHTO R30) – Hanson Aggregates Region 7

Flow Number vs Traffic – Hanson Aggregates Region 7

According to the Job Mix Formula information provided for the Hanson Aggregates Region 7 mixture, the design traffic level for the mixture was 3 to 10 million ESAL's. For this traffic level, NCHRP Project 9-33 recommends to have minimum Flow Number of 200 cycles. As shown in Figure 48, the Marathon and United PG64-28 asphalt binders just barely meets this requirement, while the Suncor PG64-28 results in a significantly higher Flow Number value than the other two binder sources.

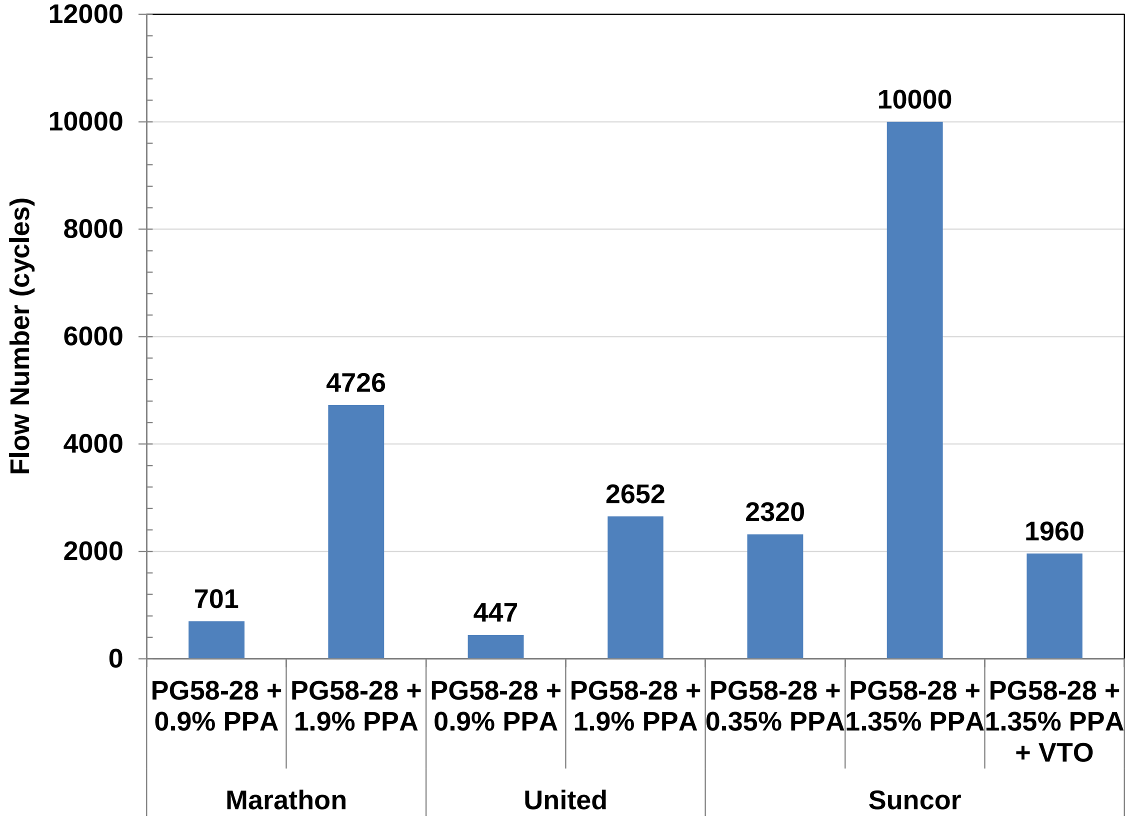


Figure 49 – AMPT Flow Number Results for Short-Term Conditioning for Mixture Mechanical Property Testing (AASHTO R30) – Morlyn Asphalt Region 9

Flow Number vs Traffic – Morlyn Asphalt Region 9

According to the Job Mix Formula information provided for the Morlyn Asphalt Region 9 mixture, the design traffic level for the mixture was < 3 million ESAL's. As noted earlier, there is no minimum Flow Number recommendation for this traffic level. However, with slow moving/intersection areas, a minimum Flow Number of some type should be required. Overall, good Flow Number values were measured for all binders tested (Figure 49).

Flow Number – Influence of Traffic Speed

As mentioned earlier, the minimum Flow Number recommendation based on traffic level was developed under NCHRP Project 9-33 to aid asphalt mixture designers/engineers in developing rut resistant asphalt mixtures. Unfortunately, not well publicized was that the traffic levels were based on a vehicle speed of 40 mph. It is well known that as vehicle speed decreases, there is a greater potential for rutting in the asphalt mixture. Since the majority of the flushing/rutting was observed in the intersection areas where the traffic levels are much slower than 40 mph, it can be assumed that the minimum Flow Number values noted earlier in Table 21 are too low for general intersection conditions.

In a 2012 research study conducted by Advanced Asphalt Technologies (AAT) for the Wisconsin DOT, the researchers attempted to adjust the minimum Flow Number requirements for traffic speed. Table 25 shows the resultant Flow Number "Correction Factors" developed using a rutting resistance model developed by the researchers. As Table 25 shows, there is a significant correction required to the minimum Flow Numbers noted earlier when traffic speeds are reduced to intersection-type speeds (i.e. – Less than 10 mph).

Table 25 - Traffic Speed Adjustment for Minimum Flow Number Requirements (Bonaquist, 2012)

Traffic Speed		Flow Number Speed Correction Factor
mi/h	km/h	
40.0	64.4	1.0
35.0	56.4	1.1
30.0	48.3	1.3
25.0	40.3	1.5
20.0	32.2	1.9
15.0	24.2	2.4
12.4	20.0	2.8
10.0	16.1	3.4
5.0	8.1	6.3
1.0	1.6	26.6

Figure 50 shows "Failure Envelopes" developed using the NCHRP 9-33 Recommended Minimum Flow Number values and the speed level correction factors shown in Table 25. The lines generated and shown in Figure 50 are the minimum Flow Number requirements based on traffic speed. For a mixture to "PASS", the resultant Flow Number for the respective traffic level and vehicle speed condition would need to fall above the respective condition lines shown in Figure 50. The 5 mph envelope line plateaus at 10 million ESAL's (MESAL) as it would be extremely difficult to assume that higher levels

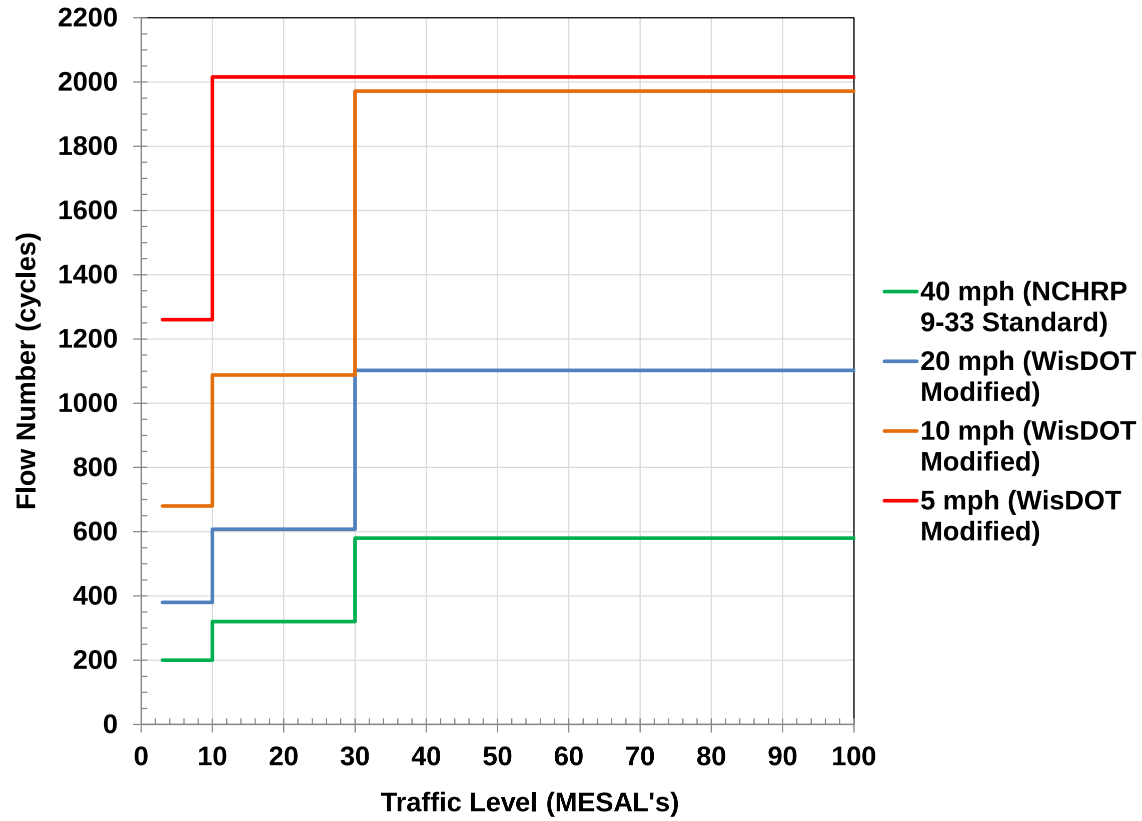


Figure 50 – Minimum AMPT Flow Number Results as a Function of Traffic Level and Vehicle Speed Adjustment

of traffic could pass through an intersection at 5 mph without causing significant traffic delays.

Figures 51 through 55 show the Flow Number results after incorporating the Speed Adjusted Minimum Flow Numbers. The Flow Number values shown are only for asphalt binders that were modified with an appropriate amount of PPA to modify a PG58-28 asphalt binder to meet a PG64-28 asphalt binder. This represents what would most likely have been placed in the field prior to the flushing issue.

From the test results shown in Figures 51 through 55, it is clear than many of the PG64-28 asphalt mixtures would not have meet the minimum requirements for slow moving/intersections, assumed to be “5 mph” in this study. In fact, most of the mixtures are not able to achieve the 10 mph Minimum Flow Number requirement. **Twelve (12) of the fifteen (15) mixtures evaluated in the laboratory did not meet the minimum Flow Number requirements for their respective traffic level at intersection locations where it was assumed traffic speeds were less than 5 mph. This would indicate that these mixtures may have rutting/stability issues when used for intersection applications.**

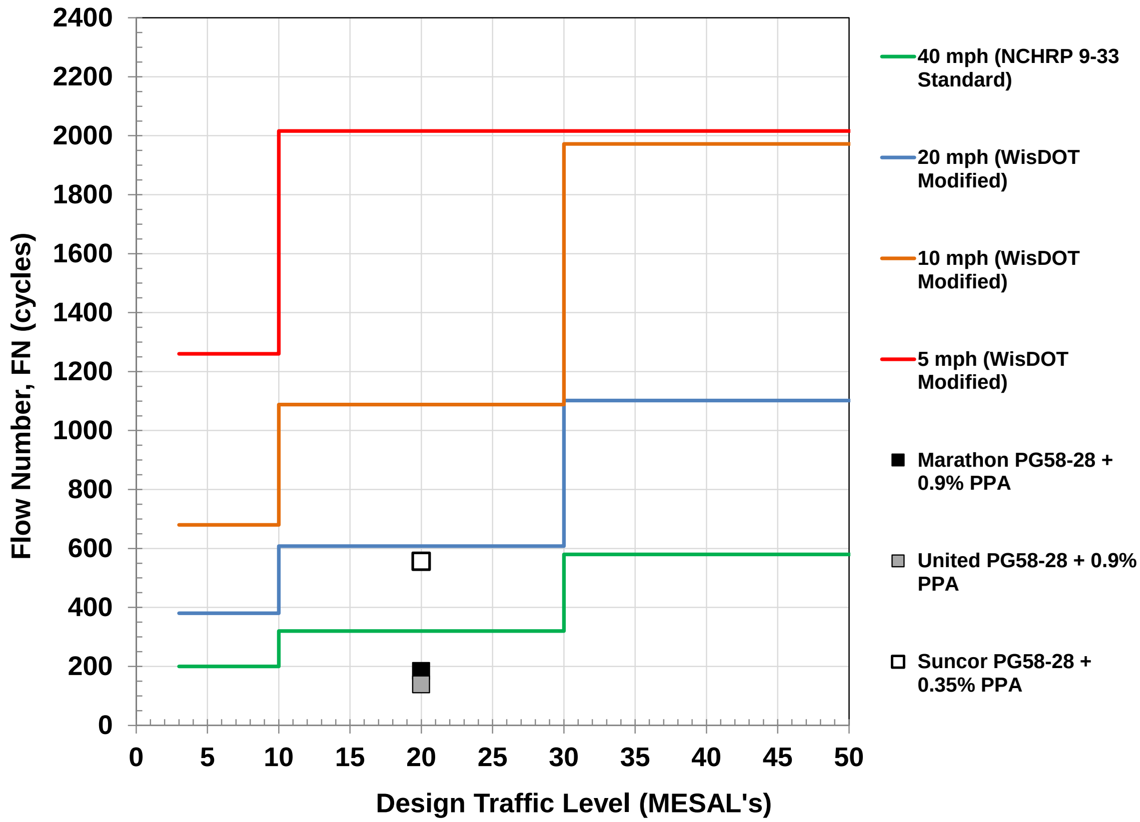


Figure 51 – Hanson Aggregates Region 2: Minimum AMPT Flow Number Results as a Function of Traffic Level and Vehicle Speed Adjustment with Laboratory Test Results

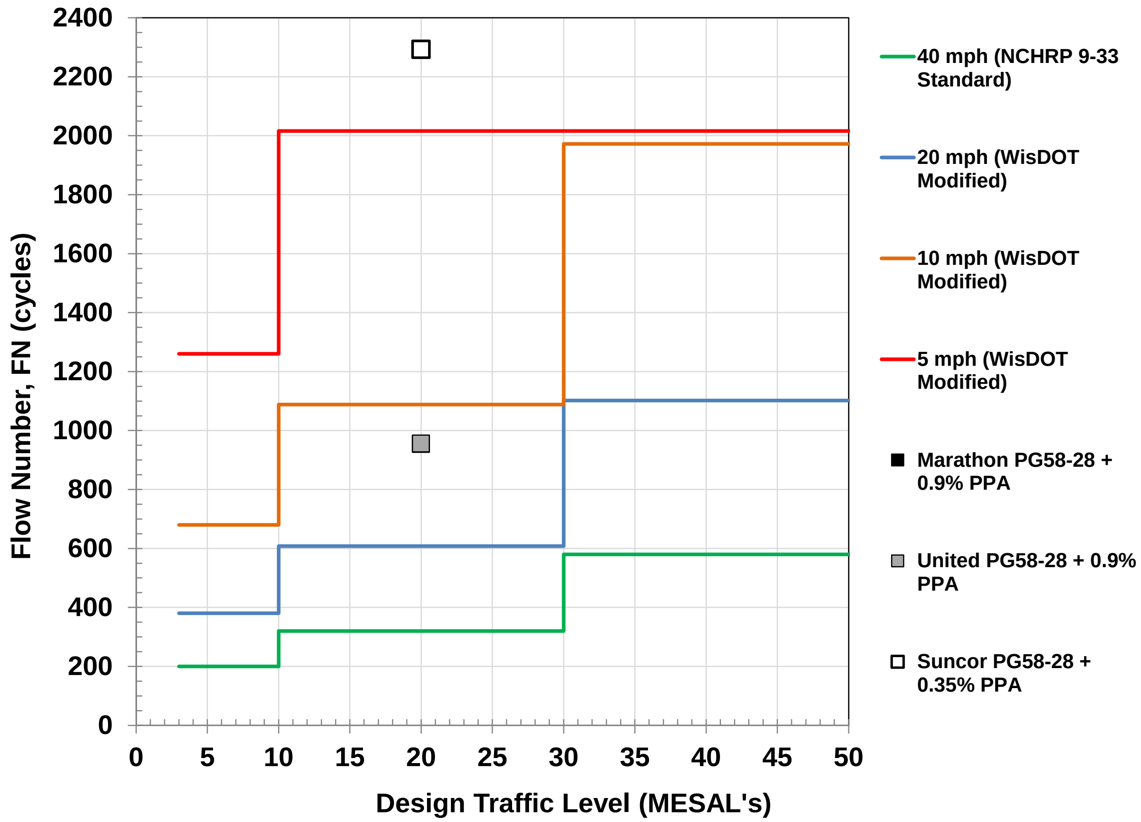


Figure 52 – Rochester (Dolomite) Region 3: Minimum AMPT Flow Number Results as a Function of Traffic Level and Vehicle Speed Adjustment with Laboratory Test Results

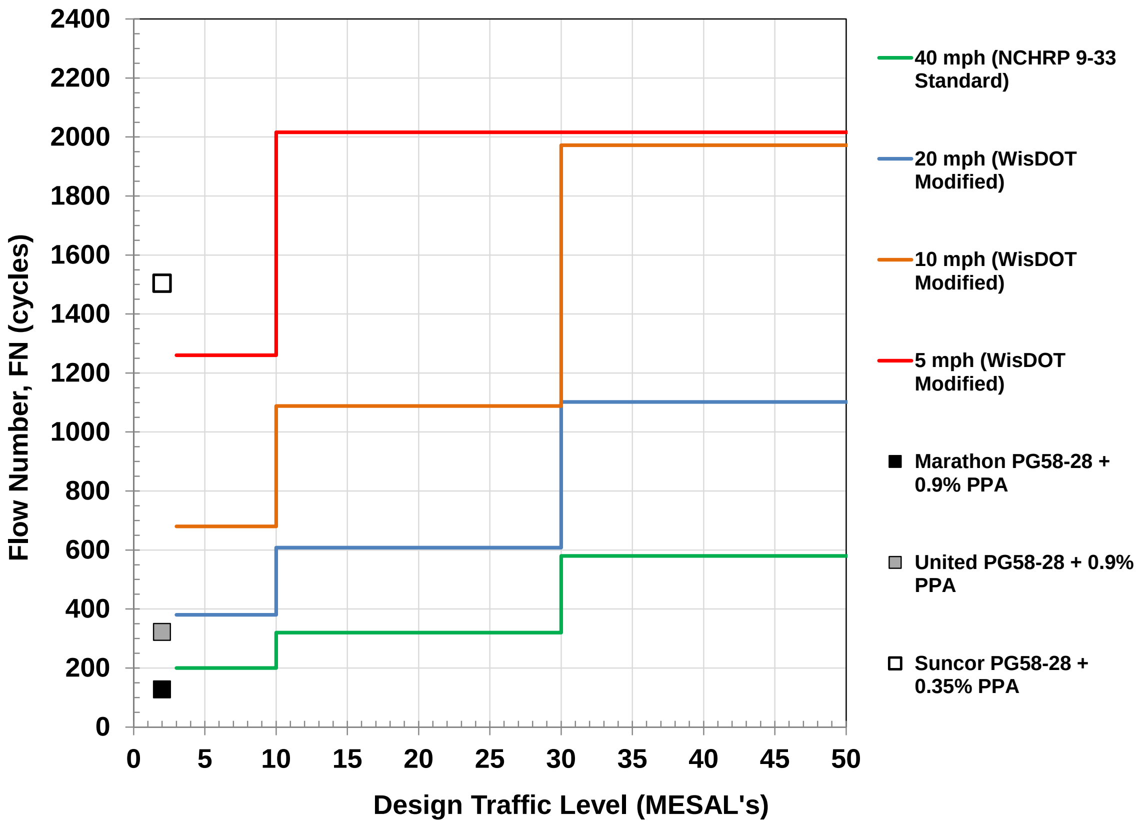


Figure 53 – Suit Kote Region 3: Minimum AMPT Flow Number Results as a Function of Traffic Level and Vehicle Speed Adjustment with Laboratory Test Results

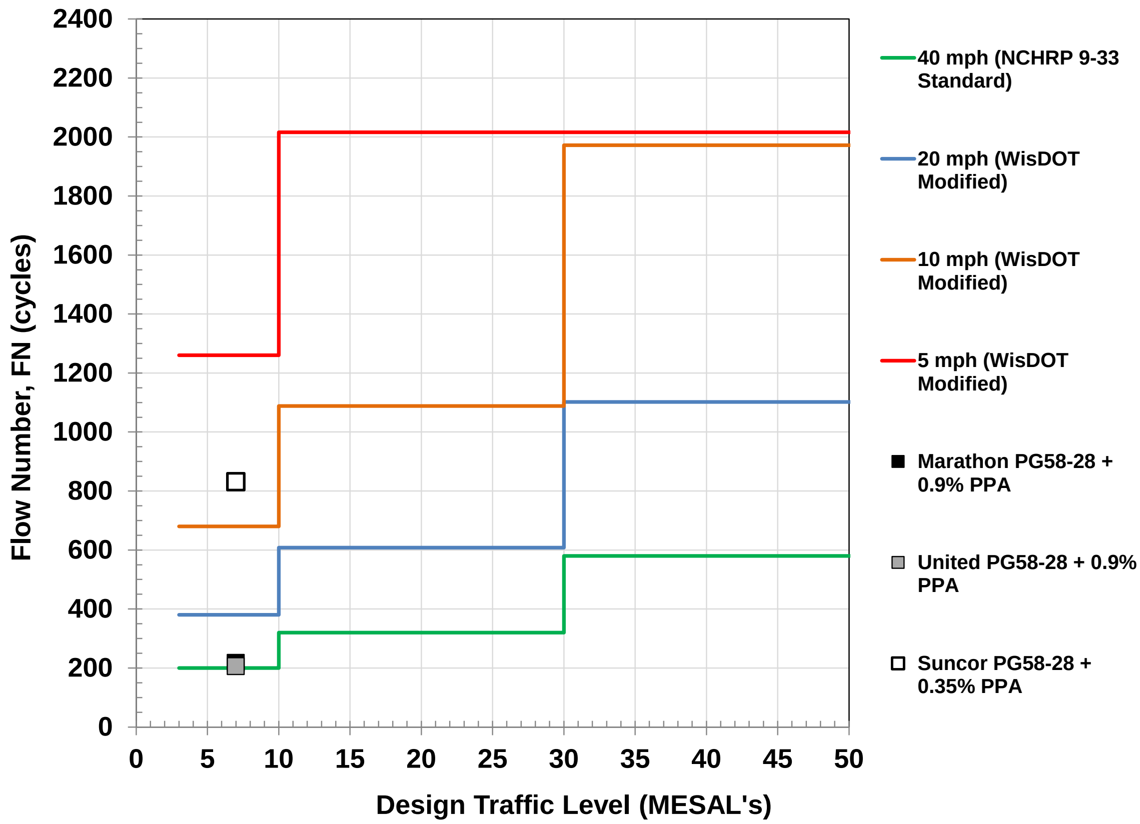


Figure 54 – Hanson Aggregates Region 7: Minimum AMPT Flow Number Results as a Function of Traffic Level and Vehicle Speed Adjustment with Laboratory Test Results

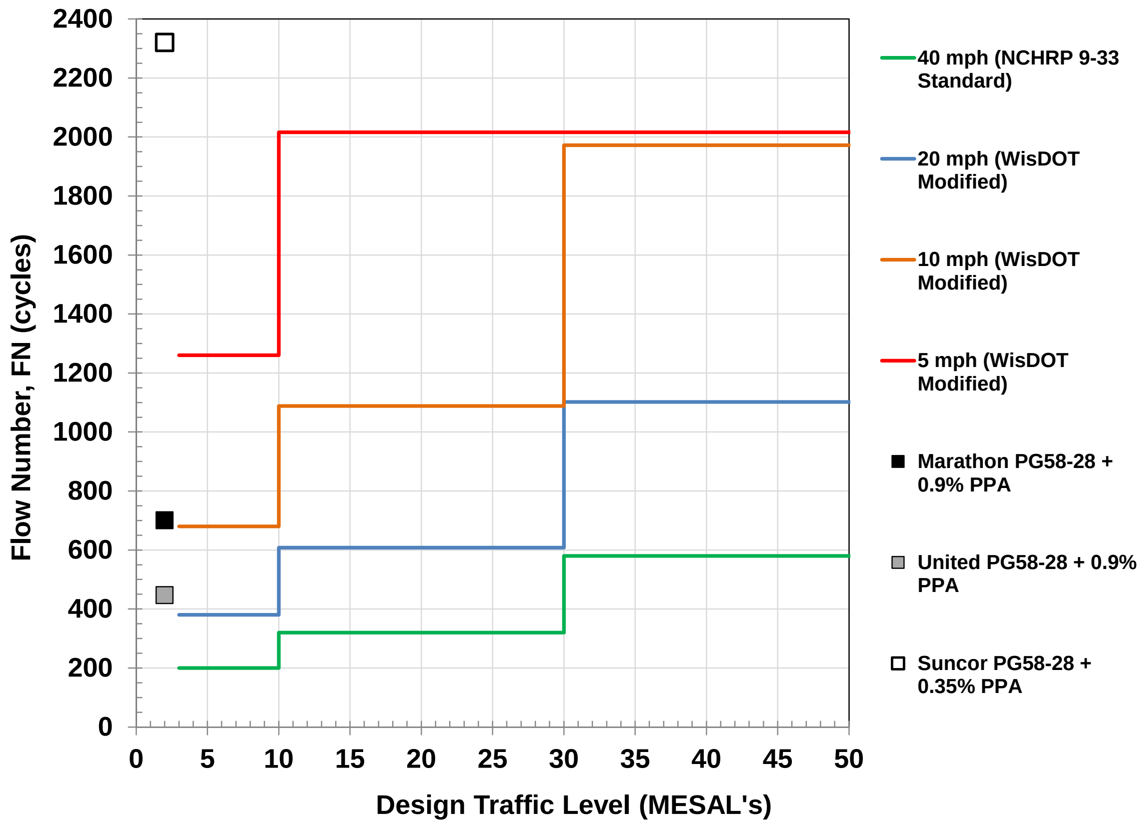


Figure 55 – Morlyn Asphalt Region 9: Minimum AMPT Flow Number Results as a Function of Traffic Level and Vehicle Speed Adjustment with Laboratory Test Results

Laboratory Mixture Evaluation Conclusions

An extensive laboratory study was conducted to evaluate the interaction between aggregate and asphalt binder for NYSDOT approved asphalt mixtures. The aggregates, asphalt binder sources, and job mix formulas represented asphalt materials that were found to have had flushing and stability issues on a number of NYSDOT pavement sections in 2007. In these sections, noticeable and sometimes excessive, flushing and rutting occurred at the intersection areas. It was hypothesized that one of the possible mechanisms for the flushing/rutting was the interaction between some of the aggregate sources (limestone) and PPA used to modify the asphalt binder may of "softened" or reduced the age stiffening required to help the stability of the asphalt mixtures. Therefore, a laboratory program was conducted to evaluate the influence of conditioning time and PPA dosage rate on the aggregate-asphalt interaction and resultant high temperature performance.

Based on the materials utilized in this study and the laboratory testing conducted, the following conclusions can be drawn:

- For the conditioning process and times utilized in the study, an increase in mixture stiffness was observed as conditioning time increased. In some cases, the measured dynamic modulus of two successive conditioning times may have been similar, but the average values always resulted in an increased in mixture stiffness. This would demonstrate that any asphalt-aggregate interaction taking place during the loose mix conditioning was not detrimental to the high temperature stiffness measured in the Asphalt Mixture Performance Tester (AMPT).
- The Flow Number test was conducted on the asphalt mixtures to evaluate their respective rutting resistance. Although the Dynamic Modulus provides a general sense of how mixtures performance at elevated temperatures (i.e. – mixtures with higher stiffness at higher temperatures will resist rutting better), the test measures the linear-elastic properties of the asphalt mixture and does not induce permanent straining commonly associated with rutting. Therefore, the Flow Number test provides a better indication of rutting resistance. The general testing in the Flow Number test demonstrated again that as the conditioning time of the asphalt mixtures increased, the rutting resistance during the Flow Number test increased as well (Figures 36 through 40). This is further indication that an asphalt-aggregate interaction is not taking place that is creating a stability issue in the asphalt mixtures.
- The Flow Number test results were compared with the Multiple Stress Creep Recovery (MSCR) Non-recoverable Creep Compliance (Jnr) parameter, which was determined at 64C. A strong correlation was typically found between the Flow Number and the Jnr parameter for the asphalt binder utilized in the asphalt mixture at each of the three conditioning times. If a change in asphalt binder high temperature performance had occurred in one or two of the mixtures, the relationships found between the Flow Number and Jnr would surely have suffered.

- When using the NCHRP Project 9-33 Recommended Minimum Flow Number values based on traffic level, it was found that one mixture did not achieve the minimum Flow Number while two other mixtures had Flow Number values that were borderline. However, it should be noted that the Recommended Minimum Flow Number values were developed based on a traffic speed of 40 mph. Since the flushing/rutting of the NYSDOT pavement sections occurred around intersection areas, where traffic speeds are much slower, an adjustment must be made to these minimum Flow Number values. Using a proposed Correction Factor methodology developed by Bonaquist (2012), the Recommended Minimum Flow Number values were corrected for traffic speeds more commonly associated with intersections (i.e. – speeds of 10 mph or less). Reanalyzing the Flow Number data under the Corrected Minimum Flow Number, it was found that a majority of the asphalt mixtures with the PG64-28 asphalt binder would not have met the minimum rutting requirements for intersection speeds (Figures 52 to 56). This would provide an indication that some of the flushing/rutting that occurred at the intersection areas of these pavement sections may have simply been a result of an asphalt mixture under-designed for rutting.

CHAPTER 6 – REGIONAL UPDATE (7 YEARS LATER)

After the completion of the forensic investigation and laboratory evaluation, Material Engineers from the three main regions affected by the flushing problem were interviewed and asked to provide an update on the flushed pavements. Two major topics were asked of the Material Engineers; 1) What is the current state of the flushed pavements; and 2) Have any other pavements in the region appeared to have flushed after the initial event in 2007. A summary from each region is found below:

Region 2 (Tim Roemer – Region Materials Engineer)

A number of pavement sections found in Region 2 had observed to have undergone flushing. Notable projects include:

- Route 315 showed significant additional flushing that was not present at the time of the original problem in 2007. This pavement had traffic levels of 2000 AADT with 8% trucks. A 9.5 mm NMAS mix design for < 30 MESAL's was originally placed on this pavement.
- Route 26 and 31 had very minor flushing in 2007 but in the last 2 years have become significantly worse throughout the project limits. Route 26 had traffic levels of 1560 AADT with 6% trucks, while Route 31 had traffic levels of 4000 AADT with 8% trucks. A 12.5 mm NMAS mix design for <30 MESAL's was originally placed on both pavement sections.
- I790 Ramp to Leland Avenue had minor flushing in 2007, but showed significant flushing in the 2013/2014 summers. This section has continued to appear "soft" in hot weather. A 12.5 mm NMAS mix design for <30 MESAL's was originally placed.
- Some pavement sections have not shown any additional flushing concerns since the original incidents occurred in 2007 (Routes 20, 12B, and 921C).

For areas where safety was a concern, the flushed asphalt was replaced with an identical mix design, but instead of using a PG64-28 asphalt binder, a neat PG64-22 asphalt binder was used.

Region 3 (Tom McPhilmy – Region Materials Engineer)

The main pavement section in Region 3 that showed extreme flushing characteristics was Rt 96B in Tompkins County, NY. Currently, the ride quality of the pavement is poor with constant "chatter" throughout due to the continued washboard shoving of the surface course (Figure 56), where a 9.5 mm using LEA warm mix technology was used. Several areas Northbound (in the downhill direction) have become safety concerns for vehicles breaking hard and turning in these washboard areas. In a dedicated left turn lane, slippage cracking are beginning to develop (Figure 57).

Figure 56 – “Wash-boarding” of 9.5 mm Surface Course in Region 3 (Rt 96B)

Figure 57 – Slippage Cracking in Dedicated Left Turn Lane in Region 3 (Rt 96B)

NYSDOT Maintenance reported 9 short locations where the existing flushing-related problems were intolerable and required to be milled out and overlaid in 2011. This totaled 700 tons of HMA with a material and installation cost of $65,000. In 2013, a fine-toothed, milling machine was used to level off several locations where wash-boarding and rutting were causing concerns.

Rutting is apparent in all of the areas where flushing was observed. Unmilled areas show rutting as deep as 5/8 inch in the wheel path area (Figure 58). Areas that underwent micro-milling still show slight rutting (1/4 inch). The pavement surface in these areas are beginning to show wheel path cracking (low to medium severity), slippage cracking (low to medium severity), and occasional raveling.

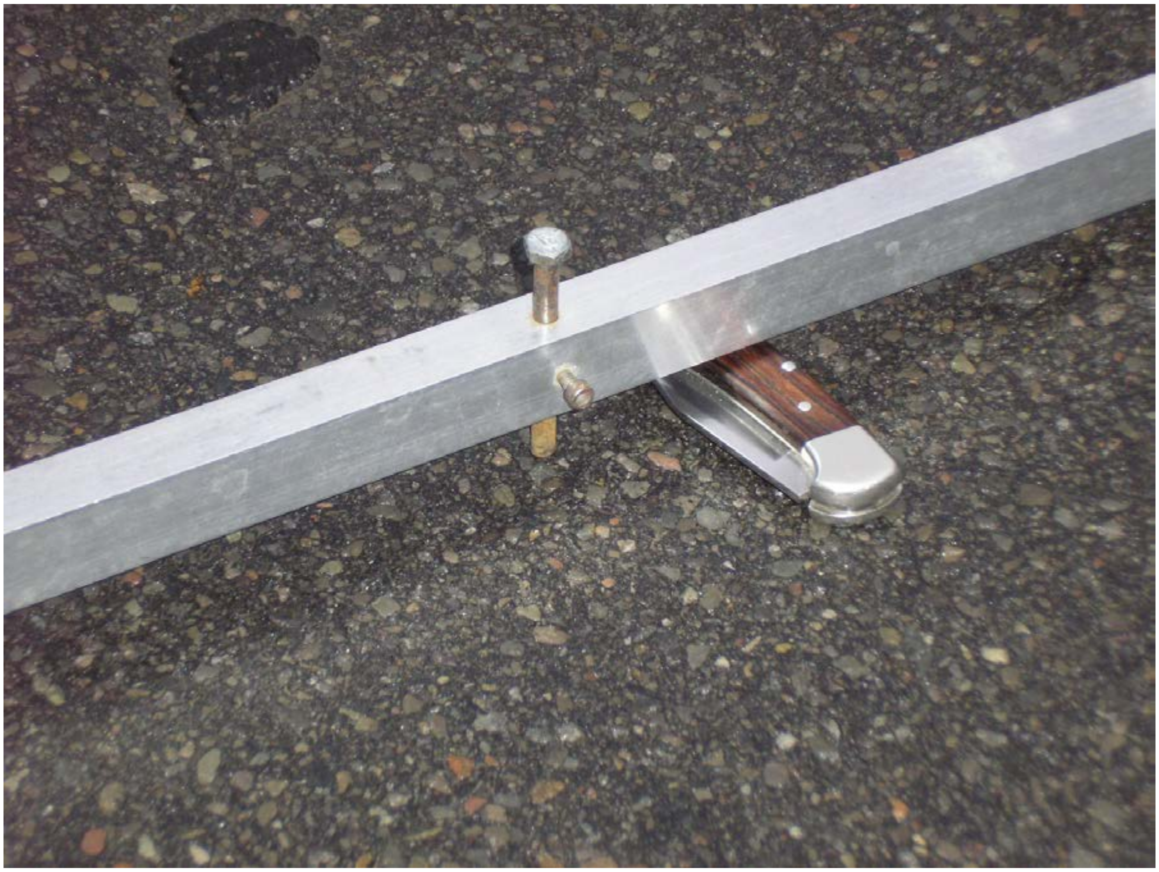


Figure 58 – Pavement Rutting in Flushed Pavement Areas in Region 3

Region 7 (Bill LaSage Region Materials Engineer)

The two pavements revisited in Region 7 were Route 812 and Route 12E locations. The areas of flushing in these areas were milled out and replaced with an unmodified PG64-22 asphalt mixture. This was conducted soon after the original flushing was observed. To date, no signs of flushing have occurred in those areas. The remainder of the pavement in these areas also show no signs of flushing/rutting. The Route 12E project looks “normal”, while the Route 812 section visually looks “tight” but flushing/rutting was not observed.

CHAPTER 7 – CONCLUSIONS

An extensive forensic and laboratory investigation was conducted to determine why particular New York State asphalt pavements constructed in 2007 had undergone "atypical" flushing. Analysis of quality control records, laboratory characterization of field cores, and a laboratory mixture evaluation component were conducted to help best determine the potential reasoning for unexpected pavement flushing. Based on the data analysis and laboratory evaluation conducted during this study, the following conclusions can be made:

- A review of quality control documents provided by the NYSDOT showed that some of the projects that resulted in flushing had aggregate gradations that were finer than the job mix formula. Field densities measured from field cores were generally low (between 3 to 5% air voids). Although lower air voids does not necessarily result in flushing/rutting by itself, low air voids combined with higher voids filled with asphalt (VFA) and asphalt binders softer than the temperature and traffic required, would certainly add to the flushing/rutting potential. Similar observations were made by the FHWA when conducting a forensic study on two of the flushed pavement sections in New York State.
- The characterization of asphalt binder extracted from field cores taken from flushed pavement sections indicated that;
    - Testing indicated that issues when conducting solvent extraction of PPA modified asphalt binder can occur when using TCE containing acid scavengers. Apparently, the acid scavengers will "pull out" some of the PPA from the asphalt binder, thereby reducing the high temperature stiffness of the asphalt binder. Follow-up extraction/recovery testing in 2009 using research grade TCE, as well as Toluene, resulted in high temperature PG grades that were closer to a PG70 high temperature. This would be expected as the pavement sections had undergone some oxidative aging within the 1 to 2 years of service life.
    - When comparing the NYSDOT field core extracted/recovered PG grade results to PG grade results conducted on samples retained from the asphalt binder storage tank, the test results were found to be within the acceptable range of results based on typical test data submitted to the AASHTO AMRL program. Even though the NYSDOT extracted/recovered results showed that asphalt binder to be classified as a PG58, the precision and bias of the test procedure itself indicated that these values were statistically equal to the PG64 results of the retained samples.
    - Using the FHWA's LTPPBind 3.1 software for asphalt binder PG grade selection, it was determined that some of the flushed pavement sections did not contain asphalt binders with the appropriate high temperature PG grade using a 98% reliability level. For the pavement sections carrying greater than 10 million ESAL's, the appropriate asphalt binder grade for the intersection areas should have been at least a PG70.

    - The presence of phosphorus, and estimated PPA contents, were determined using procedures developed by Reinke et al. (2009). The test results indicated that not all of flushed pavement sections contained asphalt binders that were PPA modified. This would indicate that the flushing/rutting observed may not be solely attributed to the PPA modification itself.
- Laboratory testing using limestone aggregate sources from the quarries that supplied the aggregates in the flushed and PPA modified asphalt binders was conducted to look at whether or not mixture softening occurs due to interactions between the limestone and PPA. The asphalt binders were formulated at 2 different PPA dosage rates; 1) Enough PPA to modify a PG58-28 into a PG64-28, and 2) 1% more PPA than what was required to achieve the PG64-28. The testing showed that;
    - The laboratory mixing and conditioning procedures utilized in the study did not result in an adverse reaction between the PPA and limestone aggregate measurable with mixture stiffness and permanent deformation testing. Dynamic Modulus and Flow Number testing, conducted in the Asphalt Mixture Performance Tester (AMPT), showed that as conditioning time increased, the high temperature mixture stiffness also increased, resulting the asphalt mixture to be more resistant to rutting – further verified with the Flow Number test. The addition of 1% PPA only resulted in a better resistance to permanent deformation.
    - Using the Flow Number test and criteria based on traffic level and vehicle speed, it was found that a majority of the mixtures evaluated would not have met the minimum Flow Number required for good field performance at slow moving traffic areas. This indicates that most of the mixtures evaluated containing PG64-28 asphalt binders may not be suitable for placement in the intersection areas of the roadways evaluated.
- Follow up interviews with NYDOT Materials Engineers, 7 years after the flushing had originally occurred, indicated that some of the flushed pavements left in place are still experiencing flushing-like performance during hot summer days. Pavement sections that were immediately replaced in 2007 with a neat PG64-22 asphalt binder have performed well over the past 7 years with no signs of flushing or rutting, even when air and pavement temperatures have exceeded those observed during the flushing issues in 2007. Field observations such as those expressed by the NYSDOT Materials Engineers would lead one to believe that the flushing problem was somehow related to the PG64-28 asphalt binders used on those pavement sections, even though the flushing/softening issue could not be replicated at the laboratories of Rutgers University and the FHWA's Turner-Fairbanks.
- At the conclusion of this study, there were no definitive reasons as to why these pavements had flushed. For every task evaluated where a potential reason was identified that may have caused the flushing issue, there were always exceptions that prohibited a conclusive answer. Therefore, although the findings in the study outline how material testing and specification can be improved in New

York State to help reduce the potential for rutting/flushing in the future, the exact reasoning for the flushing in 2007 is still unknown.

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