Long term performance of a highly modified asphalt pavement and application to perpetual pavement design

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Digital Object Identifier (DOI): dx.doi.org/10.14311/EE.2016.212

ABSTRACT

This paper reports on the final surface characterization and full forensic analysis of a highly modified asphalt pavement versus conventional Hot Mix Asphalt (HMA) at the National Center for Asphalt Technology (NCAT). The highly modified asphalt pavement is 20% thinner than a series of companion sections. The sections were subjected to 20 million Equivalent Single Axle Loads (ESALs) over a period of 5 years. At the end of two full track cycles, the thinner highly modified section has outperformed the others in permanent deformation and bottom up fatigue cracking. Modeling results give quite reasonable agreement with observed performance.

The performance results validate pavement modeling reported at the International Conference on Perpetual Pavements (ICPP) in October and demonstrate that substantial thickness reduction is possible while retaining and even improving long term performance. Equally important, material properties of highly modified mixes may be used to adjust the damage model calibration factors in the AASHTOWare/r/ Pavement ME Design software so that appropriate pavement thickness can be determined through rational design. This methodology has been put into practice and so far the performance results on commercial projects have validated the design predictions.

Keywords: Design of pavement, Fatigue Cracking, Modified Binders, Permanent Deformation, Polymers

1. INTRODUCTION

As described in a 2001 Transportation Research Board circular, "The concept of perpetual asphalt pavements is rapidly gaining acceptance in the United States. The idea is to extend the 20-year life expectancies of hot-mix asphalt pavement to greater than 50 years. To do so requires combining a rut-resistant, impermeable, and wear-resistant top structural layer with a rut-resistant and durable intermediate layer and a fatigue-resistant and durable base layer *(1)*."

In the United States, hot mix asphalt pavement thickness varies quite a lot depending on substructure, but a perpetual pavement often means a very thick hot mix asphalt structure which can be up to 300 mm or more. In the conceptual structure, a high modulus rut resistant layer bears the high compression from the surface of the pavement while a flexible, fatigue-resistant layer bears the maximum strain at the bottom of the pavement. A surface layer of sufficient quality will then only require periodic maintenance and surface restoration while the body of the pavement is expected to last indefinitely. The total pavement is thick to ensure sufficiently low strain at the bottom of the pavement to preclude bottom up cracking.

Functionally this approach works well, but may be cost prohibitive. Raw material costs steadily increase while agency construction funds steadily decrease. The current drive toward more spending on timely preservation and maintenance is a noble strategy to maintain pavements in fair to good condition, but it funnels even more money away from badly needed rehabilitation and new construction. Part of the issue is lack of reliable pavement design tools that have sufficient predictive accuracy to simultaneously provide designs with minimal risk without overdesign. The current AASHTOWare® Pavement ME Design software, in the process of "local calibration" by many states, is a potential solution to this problem.

In addition to improved design methodology, novel, high performance materials and designs have the potential to reduce overall raw material usage, initial construction cost and overall life cycle cost of pavements. In particular, Highly Modified Asphalt (HiMA) has been used in recent years in a variety of applications and shown promise as a high performance binder that can increase pavement life even with reduced pavement thickness *(2-7)*. Figure 1 shows the change in phase structure going from standard polymer loading to high polymer loading. The schematics on the left show the relative proportions of swollen polymer to bitumen while the images on the right are micrographs of such blends. The light regions are swollen polymer while the dark regions are bitumen. As the polymer content increases, the structure changes from bitumen with a dispersed swollen polymer phase (top micrograph) to swollen polymer with a dispersed bitumen phase (bottom micrograph). In effect, the binder behaves more like bitumen-modified rubber than rubber-modified bitumen.

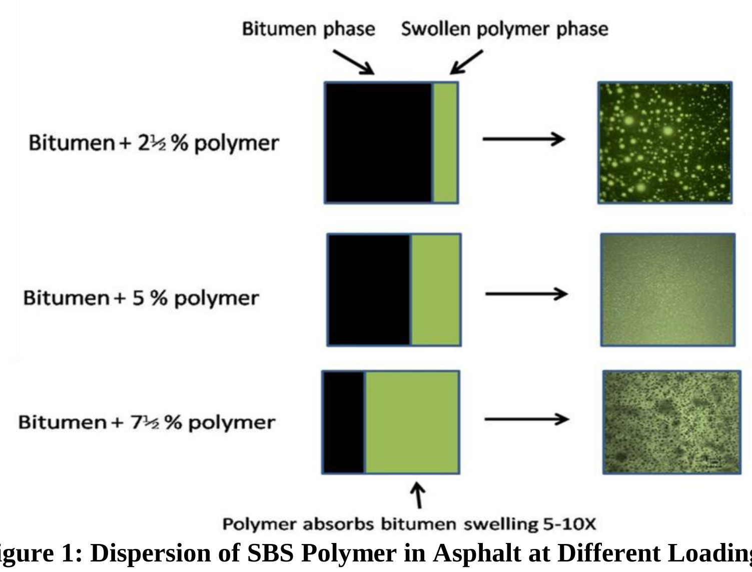

Figure 1: Dispersion of SBS Polymer in Asphalt at Different Loadings.

Modeling and material property testing reported at the 2009 ICPP conference *(8)* predicted that such highly modified binder could show substantial performance improvement in both permanent deformation and cracking resistance. Since that time, numerous projects around the world *(9)* have borne out these predictions in varied applications including structural pavements, thin overlays, micro surfacing and extreme load pavements. One example is the highly modified N7 section at the NCAT Test Track. It was constructed in 2009 as a 20% thinner companion to six sections in the Performance Group. The original design of this section, its construction and field performance through early 2014 were reported at the 2014 ICPP conference *(10)*. Also reported were the material properties of the HiMA section and the control section along with a preliminary demonstration of how the data can be used for rational pavement design and performance prediction using Pavement ME Design. In this paper, previous work is summarized and the final field performance of the sections is compared to the Pavement ME Design predictions.

2. NCAT PAVEMENT DESIGN

Figure 2 shows the 2009 pavement design for section S9 at the Test Track, the control section for the Performance Group and section N7, the highly modified binder section. All of the six Performance Group sections were basically the same standard dense mix design as previously documented *(11,12)*. The only design differences between S9 and N7 were the lift thicknesses and the binder. The base course bitumen for the Performance Group sections was an unmodified PG 67-22 while the bitumen for the binder and wearing courses was a PG 76-22 modified with approximately 3% SBS. The Performance Group design was based on the experience from previous cycles with the intent of seeing significant distress during the 10 million ESAL loading cycle. An appropriate thickness for the N7 section, however, was not obvious. With no previous field experience and only material property data, the section was simulated in the MEPDG. However, since the default damage model calibration coefficients were used, the same 178 mm thickness as the control was determined. To make the section economically viable, it was decided to use the minimum lift thicknesses based on Alabama construction protocol which limits lift thickness to 3× Nominal Maximum Aggregate Size (NMAS). That gave a total of 3×(9.5+19+19) mm or 145 mm. In hindsight, based on performance, this thickness has worked well, but it is certainly not the optimum way to design a perpetual pavement.

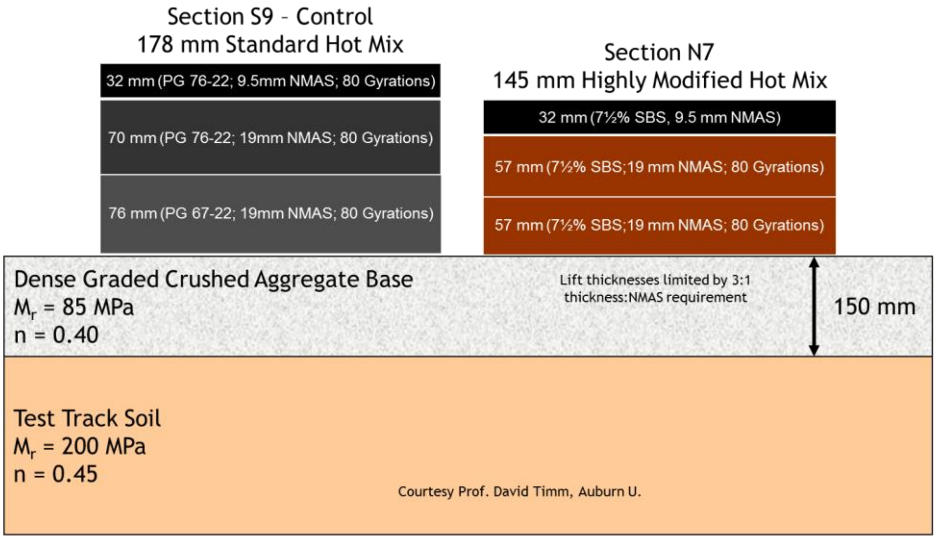

Figure 2: Pavement Design for NCAT Sections N7 and S9.

3. BINDER PROPERTIES AND SPECIFICATION

The binder for Section N7 was a blend of 7.5% of low viscosity SBS polymer in a PG 67-22 base. Standard AASHTO M320 PG grading gave a true grade for the binder of PG 93.5-26.4. Multiple Stress Creep and Recovery (MSCR) testing was also conducted according the AASHTO TP70 (now T350) and MP19 (now M332) at 64 °C. The results are in Table 1.

Table 1: MSCR Properties of HiMA Binder

Parameter	$J_{nr0.1}$ (kPa^{-1})	$J_{nr3.2}$ (kPa^{-1})	J_{nrdiff} (%)	R0.1 (%)	R3.2 (%)
Value	0.004	0.013	200.7	98.6	96.4

For the standard PG grading, one has to question how meaningful such numbers are and what a “PG 94” really means for performance. Conventional PG grading also gives no value to resilience which is one of the key characteristics for HiMA binders. For these reasons the sponsor has suggested using M332 grading wherever MSCR testing is being adopted by specifying agencies. As is clear from the table, at the environmental PG temperature the values of J_{nr} are so low that the precision is questionable. Consequently, it has also been suggested to “bump” the test temperature even though that is not strictly according to protocol. Testing at two grades above the environmental temperature brings J_{nr} up to a more reasonable level of ~0.1 kPa^{-1} which is much more suitable for specification purposes.

There is still no widely accepted binder parameter that correlates with fatigue performance. In lieu of a true performance-related parameter, a high value for MSCR recovery has been suggested as a surrogate. This at least ensures a high density elastomer network. Thus, a typical specification for the Southeast would be PG 76E-22 or PG 76E-28 with $J_{nr} \leq 0.1$ kPa^{-1} and MSCR recovery ≥ 90%. This type of specification has been used for numerous commercial bid projects and has been adopted by several states *(13)*. The HiMA binder is designated as “PG 76E-22” in this paper.

4. FINAL FIELD PERFORMANCE

The original expectation for the 2009-12 Performance Group experiment was that most if not all of the sections would show distress by the end of the cycle according to predictions made by the MEPDG using national calibration factors. However, all of the pavements proved to be a bit more durable than expected and none showed any cracking by the end of the cycle. Thus, the sponsors collectively decided to continue trafficking for another cycle with agreement that preservation treatments would be applied at 20% surface cracking. In about the middle of the 2012-15 cycle, clear performance differences began to emerge. The 2012-15 trafficking was completed in October 2014 and the detailed performance analyses for Sections S9 and N7 have been reported *(14)*.
The early 2014 data for the control, S9, and the thinner PG 76E-22, N7, are shown in Figure 3. Final rutting and cracking data for the 2012-15 cycle are shown in Figures 4 and 5, respectively. Note that the last S9 cracking data was from April 2014 and that the measured S9 rut depths decreased at the same time. That section reached its 20% cracking limit and was resurfaced at about 17 million ESALs.

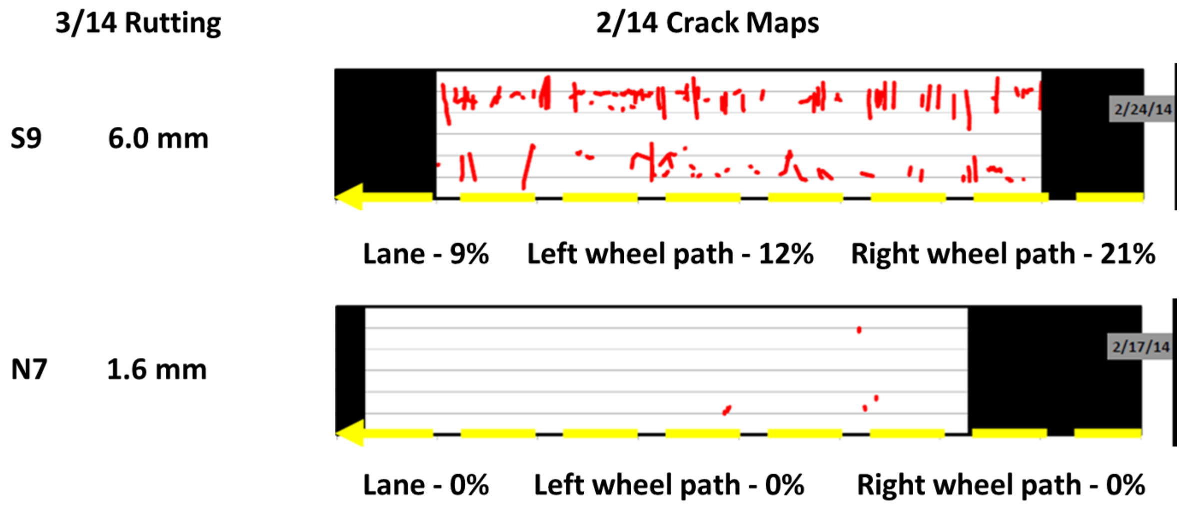

Figure 3: Early 2014 Permanent Deformation and Cracking for NCAT Sections N7 and S9.

As of early 2014, despite being 20% thinner, the highly modified binder section showed only about 1/3 as much rutting and much less cracking. S9 had significant, mostly transverse, cracking in the wheel paths which is a typical manifestation of bottom-up fatigue cracking at the track. N7 had a few hairline transverse cracks in the wheel path. These cracks first appeared over the winter, but curiously had not grown beyond the width of the wheel path. Top down cracking usually manifests as longitudinal cracks just outside the wheel path, so the exact nature of the distress was not clear at the time.
Section S9 was resurfaced shortly thereafter. As for Section N7, fine wheel path cracks continued to develop over the summer. Forensic coring was performed on both sections. As expected, the cracking in S9 is bottom up indicating significant structural distress. Unexpectedly, the cracking in N7, though similar in appearance, proved to be superficial top down cracking. For modeling comparison, the working assumption based on this forensic work is that N7 experienced little or no bottom up cracking.

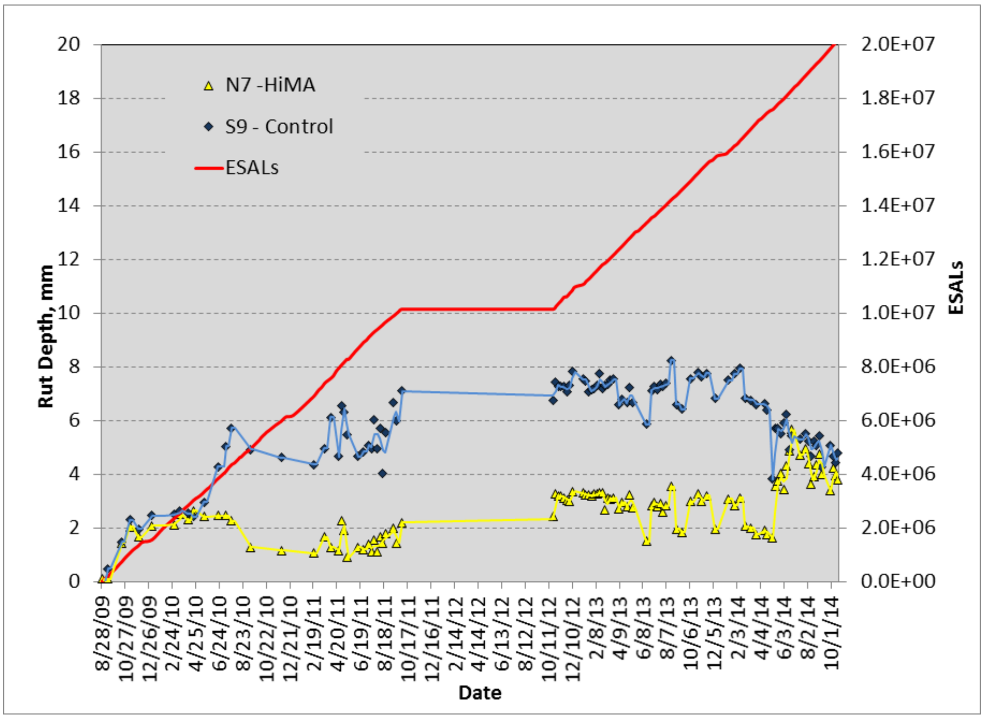

Figure 4: Comparative Rutting Performance for Sections N7 and S9.

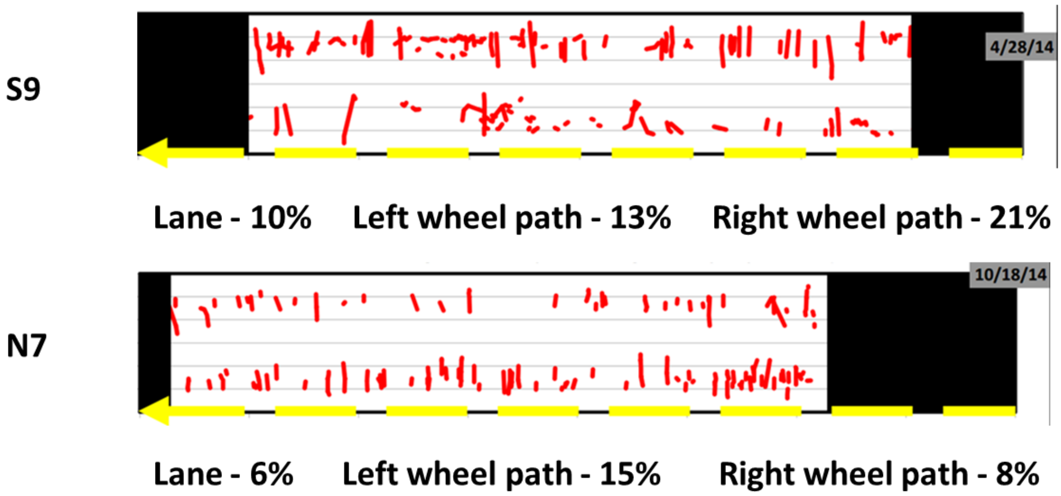

Figure 5: Final Crack Maps for NCAT Sections N7 (end of cycle) and S9 (pre-resurfacing).

5. DESIGN CONSIDERATIONS

As previously reported *(10)*, initial modeling work was done with the Delft Asphalt Concrete Response (ACRe) model *(15)* which predicted performance results similar to those seen at NCAT and elsewhere, but it is a complex 3D finite element model that requires a high level of expertise and extensive computer time to run. It is not a practical design tool. A commonly used design protocol, the DARWin 3.1 or AASHTO 93 design guide is not suitable for materials with non-standard distress versus modulus relationships so the current AASHTOWare Pavement ME Design is the tool of choice.

Pavement ME Design shows much more promise as a design tool for non-standard materials as the damage models can be adjusted based on material property data. That process works nearly the same as the process for local calibration. The Pavement ME models are based on average performance of several thousand pavements from around the country and thus it cannot account for all local conditions such as aggregates, moisture, special traffic conditions, etc. With local calibration, Pavement ME design predictions are compared to local performance experience with existing designs. The local calibration coefficients, the β parameters for the cracking:

- $N_{f\text{-}HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}}$

where:

- $N_{f\text{-}HMA}$ = Allowable axle load applications
- ε_t = Tensile strain
- E_{HMA} = Dynamic modulus measured in compression
- $k_{f1,f2,f3}$ = Global field calibration parameters
- $\beta_{f1, f2, f3}$ = local or mixture field calibration factors
- C = volumetrics parameter (asphalt content and air voids)
- C_H = Thickness correction term (depends on type of cracking)

and rutting:

- $\Delta_{p(HMA)} = \varepsilon_{p(HMA)} h_{HMA} = \beta_{r1} k_z \varepsilon_{r(HMA)} 10^{kr1} \eta^{kr2\beta r2} T^{kr3\beta r3}$

where:

- $\Delta_{p(HMA)}$ = Accumulated vertical plastic (permanent) deformation
- $\varepsilon_{p(HMA)}$ = Accumulated axial plastic strain
- $\varepsilon_{r(HMA)}$ = Calculated mid-depth resilient strain
- h_{HMA} = Thickness
- η = number of axle load repetitions
- T = pavement temperature
- k_z = depth confinement factor
- $k_{r1,r2,r3}$ = global field calibration parameters
- $\beta_{r1,r2,r3}$ = local or mixture field calibration factors

The models are then adjusted to bring the Pavement ME Design predictions in line with actual field experience.
A similar process can be used with the global calibration coefficients, the k parameters, to adjust the damage models based on material properties of the mixes. The general procedure for permanent deformation parameters is described in detail in the report on NCHRP Project 9-30A *(16)*. Similarly, standard fatigue testing can be used for the parameters for bottom up and top down fatigue cracking.
For this work, Level 1 inputs for the mixture and binder were used. The low temperature indirect tensile test (IDT) strength and creep compliance data were not generated. Instead, Level 3 default Pavement ME values were used. Note that the Level 3 resilient modulus (Mr) data for subbase and base unbound layers used in this analysis were obtained in a separate analysis *(17)* and are slightly higher than those suggested in Figure 2. Average Annual Daily Truck Traffic (AADTT) input was determined in the same analysis. All other traffic details were Pavement ME defaults with traffic growth factor of zero as the Test Track truck traffic level is constant. The highly modified binder skews the low frequency (high temperature) end of the mastercurve. So, for better accuracy for the sigmoidal model fit data at four temperatures 4.4, 20.0, 37.8 & 54.4 °C were required. The rutting damage model adjustment came from Asphalt Mixture Performance Tester (AMPT) flow number determination and the cracking model adjustment came from either AMPT or four point bending beam fatigue testing. The material properties used in the analysis are shown in the next section.

6. MATERIAL PROPERTIES

Material property testing for all three lifts of Sections S9 and N7 was conducted as reported in detail elsewhere *(10,18)*. Input data for Pavement ME calibration included full mastercurves and flow number (F_n) data generated by AMPT and fatigue curves generated by four point bending beam testing. The control and highly modified mixtures have similar low temperature modulus, but the highly modified mix is about twice as stiff at high temperature. In damage testing, however, the highly modified mixtures show order of magnitude increased resistance to both rutting and fatigue. Thus it is clear that design methodology based solely on modulus could significantly under predict Section N7 performance.

7. PAVEMENT ME DAMAGE MODEL CALIBRATIONS

The protocol in NCHRP Report 719 *(16)* details the procedure to determine rutting calibration coefficients “ab initio,” from first principles. In concept, this process incorporates global calibration and local calibration into a single package. This procedure is complex and there is additional uncertainly built in at each step. For the purposes of this work, it is more sensible to do a relative calibration to a structure with known performance. According to the 719 protocol, section N7 would be analyzed in the absence of other data and have no basis for judging the accuracy of the predicted performance. With relative calibration, a “known” pavement, S9, is analyzed using the existing global calibrations. The global calibration coefficients are adjusted using the relative performance of S9 versus N7 materials. Assuming the S9 calculation is close to reality, only the material properties are variables in the N7 analysis.

7.1 Fatigue Calibration

The basics of the fatigue model are shown in Figure 6 and Equation 1. The k_{f1} and k_{f2} factors come directly from the cycles to failure versus strain. The k_{f3} factor is then determined from the other known parameters and the initial modulus at the test temperature. In this analysis with three beams at three strain rates, the modulus was taken as the average of all nine beams. The data were then regressed with the traditional strain fatigue model shown in Equation 1 below where N_f is cycles to failure, ε_0 is strain and E_{init} is initial modulus:

$$N_f = k_{f1}\left(\frac{1}{\varepsilon_0}\right)^{k_{f2}}\left(\frac{1}{E_{init}}\right)^{k_{f3}} \quad (1)$$

Calculations were done using the Simplified Viscoelastic Continuum Damage (S-VECD) protocol *(19)*.

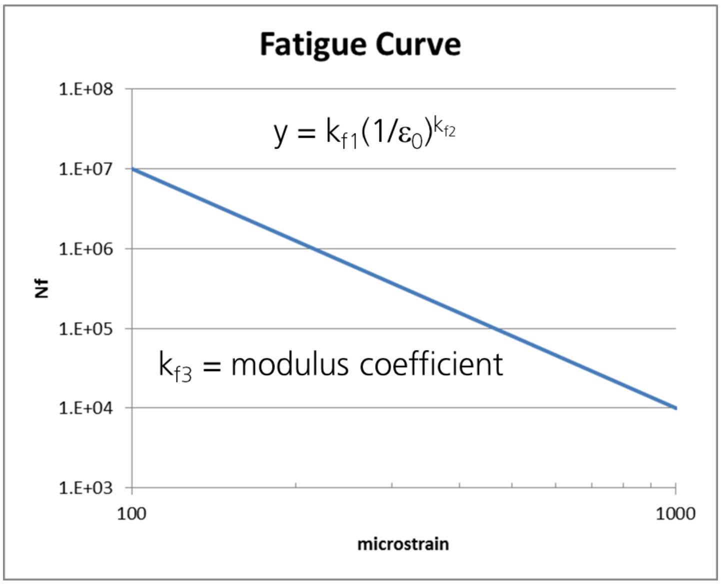

Figure 6: Conceptual Fatigue Analysis

Table 2 shows the calculated fatigue coefficients. MEPDG standard values are the "built in" values. S9 calculated values are the coefficients adjusted for PMA as recommended in the Asphalt Institute Engineering Reports ER-215 and ER-235 *(21,22)*. These are only included for reference. N7 calculated values are the coefficients as calculated with the S-VECD analysis of the beam fatigue data. Ratios are the log ratios of the N7 to S9 coefficients. Finally the N7 adjusted values are the MEPDG standard values adjusted by the determined ratios. MEPDG standard values versus N7 adjusted values were used for the comparative Pavement ME analyses.

Table 2: Fatigue Calibration Factors for Section N7

	k_{f1}	k_{f2}	k_{f3}
MEPDG Standard Values	7.566E-3	3.9492	1.2810
S9 Calculated Values	1.4964E-2	3.9492	1.2810
N7 Calculated Values	7.5721E-5	7.3135	2.3655
Ratios	0.9762	0.7595	0.0491
N7 Adjusted Values	7.386E-3	2.9994	0.0630

7.2 Rutting Calibration

As noted above, the rutting calibration process is greatly simplified by relative calibration. The k_{r1} and k_{r2} parameters are the intercept and slope of the secondary damage regime in the AMPT flow number test as seen in Figure 7. When using relative calibration, this process could, in principle, be accomplished using any deformation test that exhibits primary, secondary and tertiary flow. The k_{r3} parameter is the temperature dependence. This is determined by running flow number at, ideally, three temperatures. In this case flow number data only at a single temperature 59.5 °C, so k_{r3} was left constant.

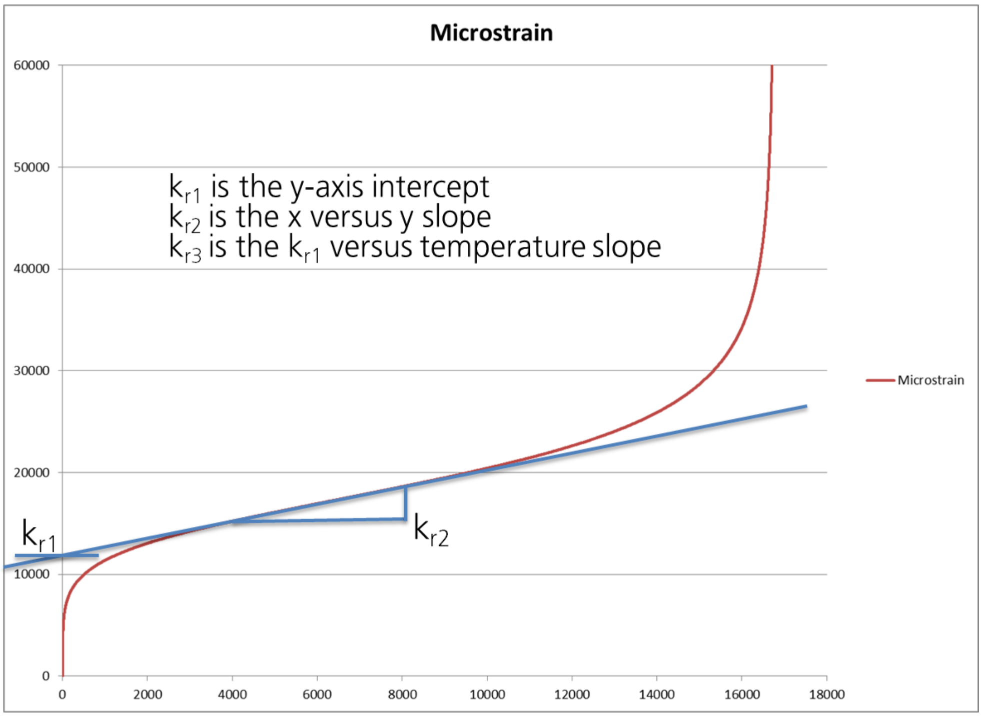

Figure 7: Conceptual Rutting Analysis

Table 3 gives the rutting coefficients. The ratios for kr1 and kr2 are the ratios of m and Is in the flow number data as described in NCHRP Report 719 *(16)*. MEPDG standard values versus N7 adjusted values were used for the comparative Pavement ME analyses.

Table 3: Rutting Calibration Factors for Section N7

	k_{r1}	k_{r2}	k_{r3}
MEPDG Standard Values	-3.3541	0.4719	1.5606
S9 Calculated Values	-3.7902	0.4719	1.5606
Ratios	0.8045	0.4791	1.0000
N7 Adjusted Values	-2.6985	0.2261	1.5606

8. PAVEMENT ME ANALYSES

Pavement ME Level 1 analysis was run on the S9 and N7 structures with the following inputs:

- Climate data for Montgomery, AL
- AADTT = 1465; Speed 72 kph; No growth
- Subbase Modulus = 85 MPa
- Subgrade Modulus = 200 MPa

As expected, no thermal cracking was predicted nor observed on the sections. Also, IRI was not considered for these sections due to the short length, though the analyses produced values. Figures 8 – 11 show the 50% reliability and 90% (specified) reliability damage prediction charts for bottom-up cracking in percent area and the 50% reliability rut depth in mm, respectively. Table 4 gives a summary of the 90% reliability analysis predictions at the end of 20 million ESALs' loading.
The reliability values assume that the standard errors of the estimates in the model are not affected by the changed calibration coefficients and this is not necessarily valid. Much more data will be needed to address this concern so for now the modeling data is reported with the existing standard error input.

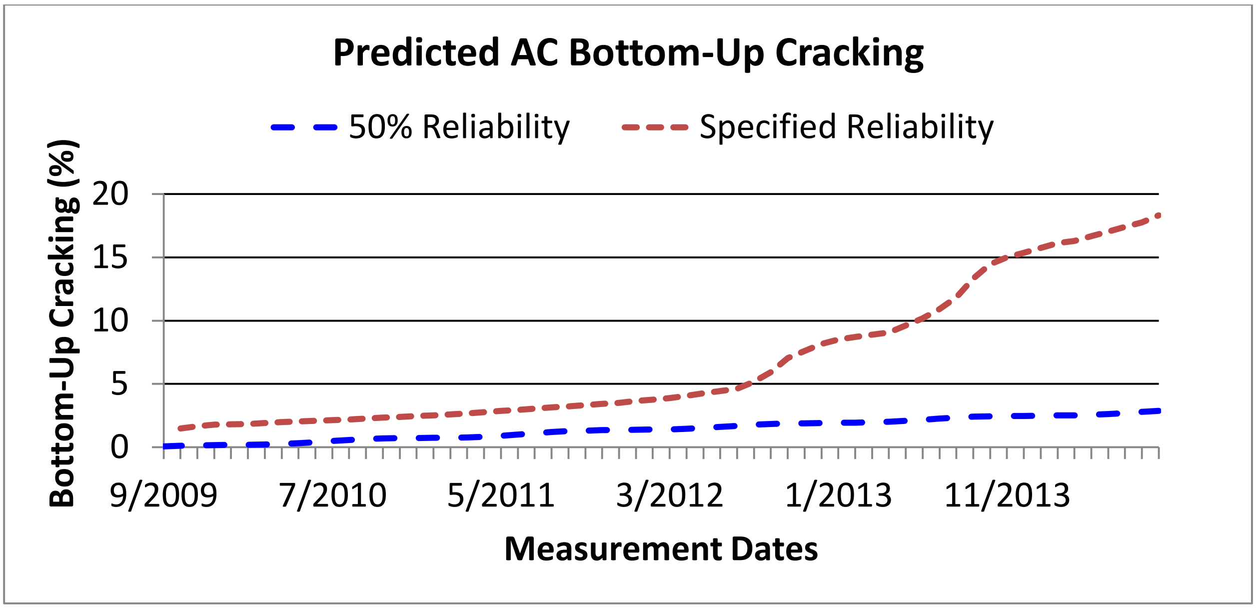

Figure 8: S9 Predicted Bottom-Up Cracking

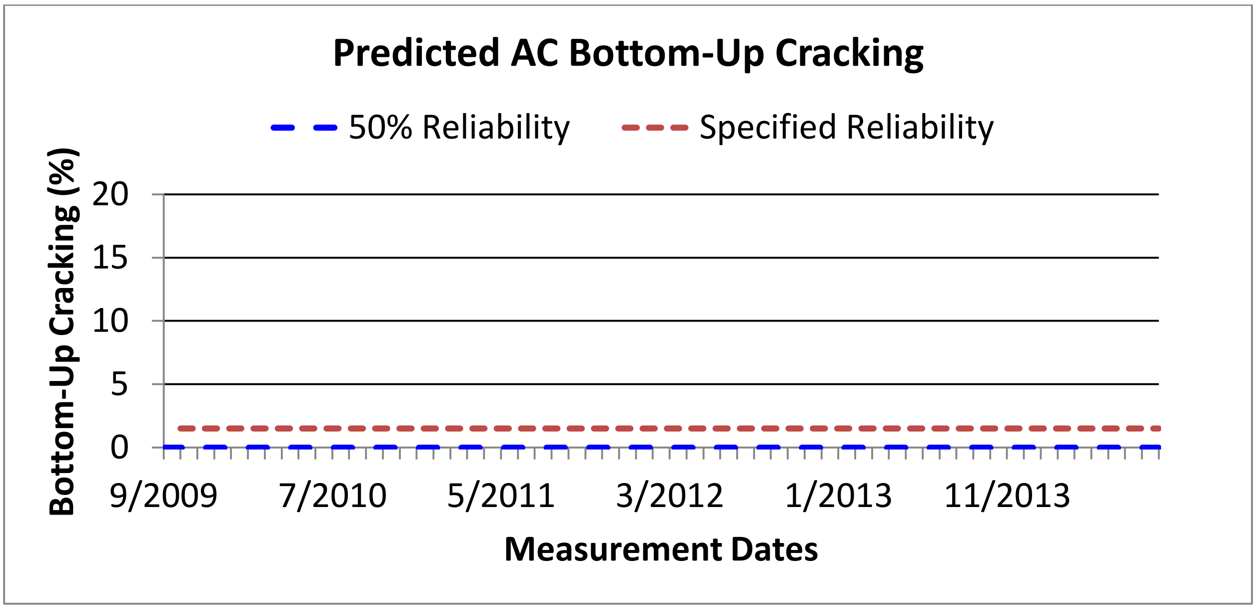

Figure 9: N7 Predicted Bottom-Up Cracking

Table 4: Predicted Damage Summary – 90% Reliability

Pavement Distress	S9	N7
Total Permanent Deformation, mm	10.2	8.4
AC Permanent Deformation, mm	6.4	1.5
Bottom-Up Cracking, % Area	18	1.5

Starting with permanent deformation, both analyses over-predict total permanent deformation, but this over-prediction appears to be all in the subbase and subgrade. This over-prediction of deformation below the bound layer is well documented *(20)*. In the previous report from early 2014 results, the bound layer predicted deformation in both cases was remarkably close to the observed rutting with 6.4 mm calculated versus 6 mm measured for S9 and 1.5 mm calculated versus 1.6 mm measured for N7. The final results are less clear as S9 was resurfaced shortly thereafter and the rutting measurements on N7 became somewhat erratic with a final wire measurement of 3.8 mm. Nevertheless, the relative results, comparing Figures 4 (measured rutting) with Figures 10 and 11 (predicted rutting) are very encouraging.

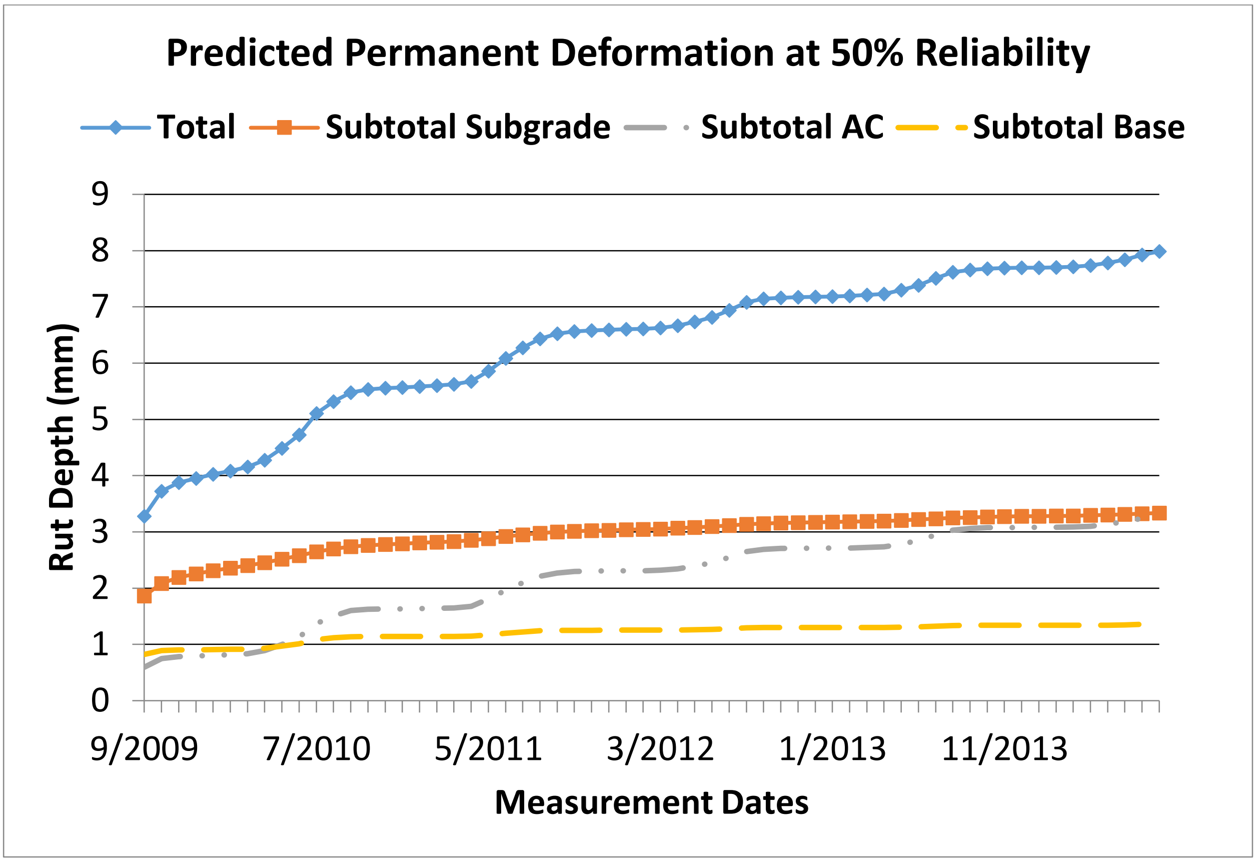

Figure 10: S9 Predicted Rutting

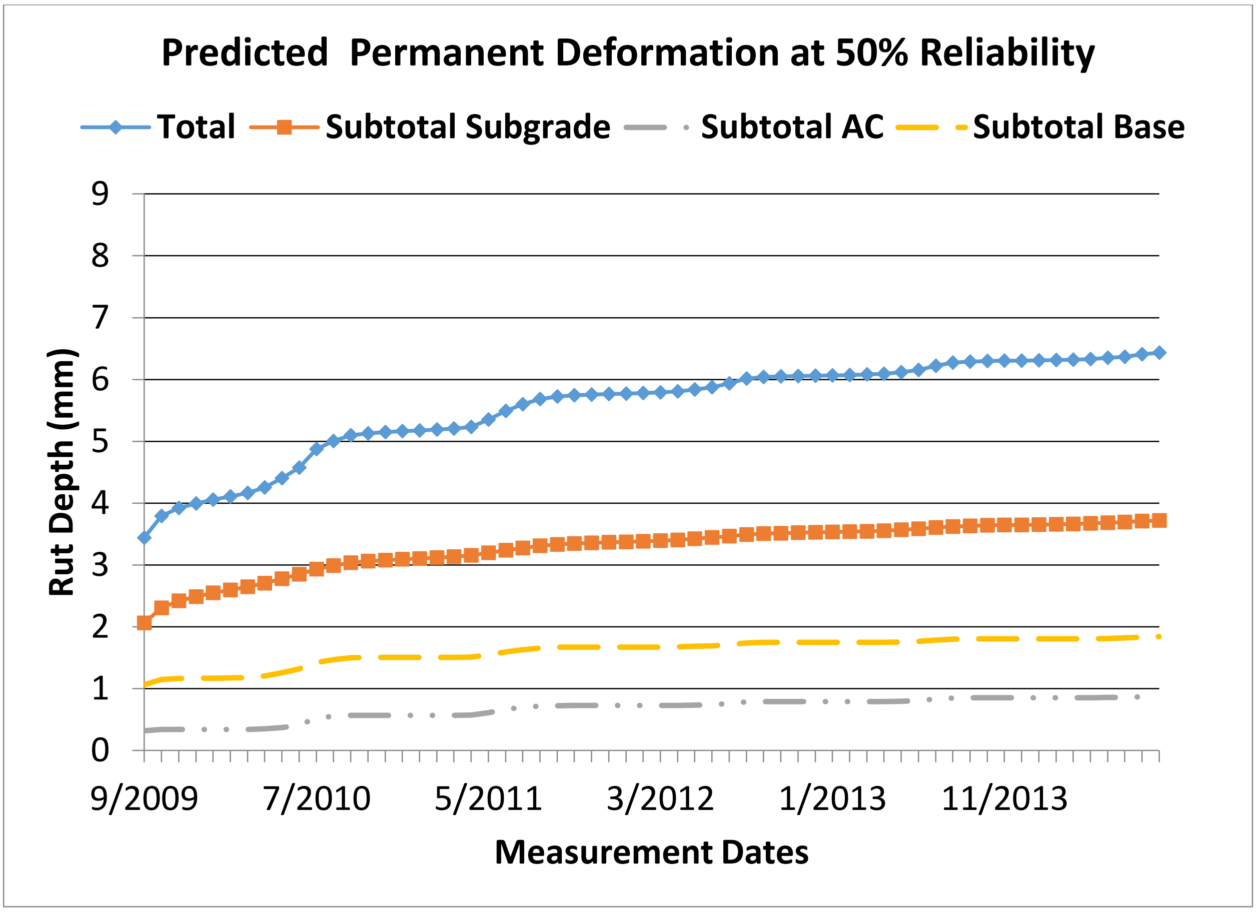

Figure 11: N7 Predicted Rutting

The results for bottom-up cracking are also quite encouraging. The observed cracking in S9 of 9%, 12% and 21% in the overall lane, left wheel path and right wheel path respectively before resurfacing was very close to the calculated 15% predicted at about 17 million ESALs. The very low 1.5% for N7 seems to be inherent from the start and is also present at the start for S9. As the observed surface cracking late in the life of Section N7 appears to be top down surface cracks, the prediction agrees well with the observed performance.
For S9, one issue not addressed is that the top two lifts were polymer-modified while the existing models were based dominantly on unmodified pavements. Studies by the Asphalt Institute *(21,22)* show a general improvement in fatigue cracking resistance with Polymer-Modified Bitumen (PMB) versus unmodified bitumen and they have a recommended adjustment for the rutting and fatigue calibrations:

$$k_{r1(PMA)} = 1.13 \cdot k_{r1(NeatHMA)} \qquad (2)$$

$$-\mathrm{Log}(k_{f1(PMA)}) = (-\mathrm{Log}(k_{f1(NeatHMA)}))^{0.8} \qquad (3)$$

Unfortunately, the current version of Pavement ME uses a single damage model for the entire bound layer. That is fine for the N7 section with the same binder throughout, but for S9 with modified top two lifts and unmodified base lift, it is not clear whether all HMA or all PMB is the better choice for modeling this pavement with two different binder types. That being said, Table 5 gives the results for S9 using the PMB k_{r1} and k_{f1}. The HMA model is closer to actual relative performance for rutting and bottom-up cracking.

Table 5: Predicted Damage Summary – 90% Reliability Including S9 PMA

Distress	S9 HMA	S9 PMB	N7
Total Permanent Deformation, mm	10.2	7.6	8.4
AC Permanent Deformation, mm	6.4	2.0	1.5
Bottom-Up Cracking, % Area	18	3.7	1.5

Overall, the Pavement ME analysis was not able to definitively predict the performance of these pavements but that was not the objective. The relative performance is clearly and accurately shown. The next step is local calibration to tune predicted to field performance but that is typically done with a large database of pavements.

9. CONCLUSIONS

This paper describes a single example of pavement analysis and performance, but numerous other projects around the world including several large projects in South America have shown good results to date *(9)*. While it is premature to draw broad conclusions, the indications so far are that the design analysis is reasonably robust. With that in mind, the following conclusions and recommendations are made.

- Highly modified asphalt may prove to be a useful tool in perpetual pavement design. Demonstrated performance up to 20 million ESALs shows that the thickness of pavement structures may be reduced while retaining or even improving long term performance. This may reduce initial construction cost and will certainly reduce construction time.
- Standard, though extreme, PG specifications using AASHTO M332 and T350 have been used to specify HiMA binders for commercial applications.
- Standardized test methods in increasingly common use are adequate to characterize HiMA mixtures for the purpose of pavement design.
- The current AASHTOWare Pavement ME Design protocol is suited to designing perpetual pavements with highly modified asphalts. Relative global calibration factor adjustment with Level 1 design gives performance predictions that agree well with actual field performance relative to known structures.
- NCAT section N7 developed fine surface cracking late in its life, but forensic analysis showed that the cracking was minor top down cracking not impacting the structural integrity of the pavement. Section N7 has been resurfaced and is currently undergoing a third loading cycle.

ACKNOWLEDGEMENTS

The authors wish to thank the control section sponsors that included the following state departments of transportation: Alabama, Florida, North Carolina, Oklahoma and Tennessee. The Federal Highway Administration also deserves special recognition for their support of the control section.

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