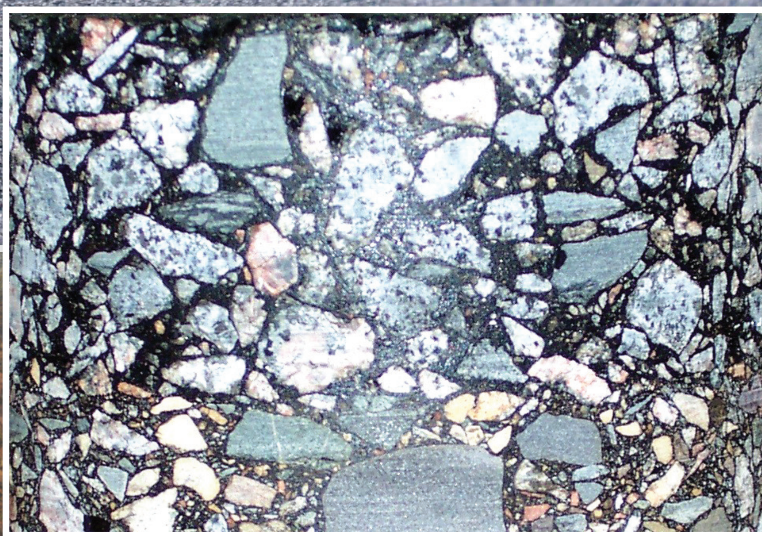
Special Report 223

NATIONAL ASPHALT PAVEMENT ASSOCIATION

Advances in the Design, Production, & Construction of Stone Matrix (Mastic) Asphalt



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NATIONAL ASPHALT
PAVEMENT ASSOCIATION

NAPA Building ▪ 5100 Forbes Blvd. ▪ Lanham, MD 20706-4407
Tel: 301-731-4748 ▪ Fax: 301-731-4621 ▪ Toll free 1-888-468-6499
www.AsphaltPavement.org ▪ NAPA@AsphaltPavement.org

Mike Acott
President

Audrey Copeland, Ph.D., P.E.
Vice President for Engineering, Research & Technology

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1ST INTERNATIONAL CONFERENCE ON

Stone Matrix Asphalt

NAPA

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Tel: 301-731-4748 ■ Fax: 301-731-4621 ■ Toll free 1-888-468-6499

www.AsphaltPavement.org ■ NAPA@AsphaltPavement.org

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1st International Conference on Stone Matrix Asphalt

Atlanta, Georgia — November 5–7

1st INTERNATIONAL CONFERENCE ON STONE MATRIX ASPHALT: OPENING REMARKS

November 5, 2018 — Atlanta Georgia

MIKE ACOTT
National Asphalt Pavement Association
5100 Forbes Blvd., Lanham, MD 20706
Tel: +1-301-731-4748

SMA — otherwise known as Split Mastic Asphalt/Stone Mastic Asphalt/Stone Matrix Asphalt — was developed in Germany in the late 1960s. The original impetus was to develop a mix with better resistance to studded tires. Simply stated, SMA is a gap-graded mix designed to achieve direct stone on stone contact. The mix includes a binder rich mortar of the desired quantity and consistency/characteristics. Later in the program, we will hear more details on the development and use of SMA in Germany, as well as experiences in other countries.

I will provide a short perspective on its development in the U.S. NAPA introduced the concept of SMA into the U.S. in the mid-'80s. Our interest in SMA was precipitated by the presentation of a paper by Mr. Otto Kast at the Eurobitume symposium at the Hague on SMA experience in Germany in 1985. Meanwhile, the American Association of State Highway and Transportation Officials (AASHTO) and the Federal Highway Administration (FHWA) were in the final planning stages of the Strategic Highway Research Program, which ultimately led to the introduction of Superpave and performance-graded PG Binders. The prevailing distress in the 1980s was rutting. The approach had been to move to 75 blow Marshall, increase aggregate top size, coarsen the mixtures, and reduce asphalt content. The inevitable trade-offs were a reduced fatigue life and an increased permeability.

Deployment of SMA in the U.S. gained momentum following the FHWA/AASHTO/NAPA European asphalt study tour in 1990. With very few exceptions, the U.S. was wedded to dense-graded asphalt concrete, so the interest in a gap graded mixture was a departure from practices at that time. In 2002, SMA was discussed at the "SMA in the USA" workshop in Maryland. This workshop was co-sponsored by the Asphalt Pavement Alliance (APA), Maryland State Highway Administration, Virginia Department of Transportation, FHWA, and the International Society for Asphalt Pavements. More than 400 attended from 11 countries and 35 states. At that conference, several DOTs had become lead states in the use of this technology. There was considerable activity and energy focused on evaluating the product and its implementation, including field trials and completion of numerous research initiatives. It seemed in 2002 that SMA was off and running.

The expectation expressed at that time by FHWA, state DOTs, the National Center for Asphalt Technology at Auburn University (NCAT), the State Asphalt Pavement Associations, and NAPA was that SMA would become the preferred premium surface for heavy duty, high-traffic volume pavements such as interstates.

For example, Larry Michael, who was with the Maryland State Highway Administration, said: “It is without a doubt the most tenacious mix I have seen. It is almost impossible to make it rut, and will outlast any other mix.” Larry was not alone in his support of this technology. Former NCAT Director Ray Brown advised it is now reasonable to expect SMA to perform satisfactorily over a 20 to 40 percent longer service life than dense-graded asphalt concrete type mixes. John Bukowski, formerly with FHWA, stated, “It has become the preferred premium pavement surface-it lasts a long time without maintenance.” Richard Schreck, former Executive Vice President of the Virginia Asphalt Association, commented, “I believe SMA raises the bar for the industry. It is truly the 20-year mix that can be combined with perpetual pavement design to produce an outstanding system.” And yours truly opined that “SMA is a product that incorporates the many outstanding developments in asphalt technology.”

It appeared that the promise of SMA — a longer lasting, durable high-performance surface – was exactly what the customer wanted because SMA was specifically designed to address rut resistance and durability — a truly “balanced” mixture.

Over 25 years have passed since that 1990 study and, apart from a handful of states that have successfully implemented SMA, SMA has not become the widespread, national surface mix of choice for high-traffic volume applications.

So what may have stalled the nationwide use of SMA? Did we lose interest in SMA, or did the high cost of liquid asphalt cement and polymers in 2008/2009 and shrinking highway budgets relegate SMA to being just an expensive tool in the toolbox? Maybe the national advocates such as Larry, Ray, and Richard moved on to other topics. Some would argue that we changed the formulation/ingredients and we did not always get the performance benefits. Also, could the aggregate industry provide the gap-graded fractionated aggregate? Perhaps the introduction of warm-mix asphalt technology into U.S. markets in 2004 consumed all the oxygen in the room, shifting the focus away from SMA.

What about those key states that use and have used SMA for over a quarter of a century? What lessons can we learn from states that have successfully adopted the technology? Some report SMA pavement service lives greater than 25 years. And, how about our partners in Europe, especially Germany? Does SMA continue to be the material of choice for the Autobahns, and has the technology changed?

The answer to these and many other questions will be presented at this, the 1st International Conference on SMA. Agencies and industry will share case studies and best practices that highlight producing and constructing a mixture with high value and performance capabilities. SMA is not only a premium asphalt pavement, but it is an engineered mixture worthy of a fresh look and an intensive discourse, especially given that dedicated freight corridors in the U.S., i.e., Critical Commerce Corridors, is a fast approaching reality. Academia will provide insight into how SMA can continue to evolve through material choices such as reclaimed asphalt pavement, polymer modifiers, recycled tire rubber, fibers, and careful aggregate selection. Specifiers and producers will discover enhanced mixture characteristics in terms of life-cycle cost, performance, and sustainability. Those who have never used SMA will gain a new understanding of what it means to specify, produce, and construct SMA using today’s technology.

And NAPA is here to learn alongside you. We are using information shared at this conference to revise our 2002 SMA state-of-the-practice publication, refreshing it to include new innovations, modern technology, and best practices.

I am excited to see what unfolds and am confident that we will rediscover new ways to deliver a truly balanced mix that exceeds expectations.

Performance and Life-Cycle Cost Benefits of Stone Matrix Asphalt

Fan Yin
Assistant Research Professor (Corresponding Author)
National Center for Asphalt Technology
277 Technology Parkway
Auburn, Alabama 36830
Tel: (334) 844-6288
F-yin@auburn.edu

Randy West
Director & Research Professor
National Center for Asphalt Technology
277 Technology Parkway
Auburn, Alabama 36830
Tel: (334) 844-6228
Westran@auburn.edu

ABSTRACT

Since its introduction into the United States, stone matrix asphalt (SMA) has gained great popularity among state highway agencies as a premium asphalt mixture to enhance field performance and extend life expectancy of asphalt pavements and overlays. Considering that SMA is generally more expensive than the conventional Superpave dense-graded mixture, mainly due to higher asphalt contents, requirements for more durable aggregates, and inclusion of fibers as stabilizers, it remains unclear whether the higher cost of SMA can be justified by the increase in life expectancy of the mixture. This study was undertaken to quantify the performance and life-cycle cost benefits of SMA versus those of polymer-modified Superpave dense-graded mixtures used on similar trafficked highways. Field performance data was collected from nine state highway agencies that used SMA on a routine basis and analyzed to determine the performance benefits of SMA. Analysis results indicated that SMA generally had equivalent or better field performance than the comparable Superpave dense-graded mixtures. For cases where SMA had better performance, the life extension of SMA varied from one to 13 years among the states and varied for different pavement types (i.e., flexible or composite pavements). It should be noted that the predicted service lives of SMA were based on extrapolation of limited performance data; thus, longer-term performance data is needed to verify the performance benefits of SMA. Additionally, case studies on both deterministic and probabilistic life-cycle cost analysis (LCCA) were conducted, but no consistent conclusion was obtained for comparing the life-cycle cost benefits of SMA versus polymer-modified Superpave dense-graded mixtures. Whether or not SMA is more cost-effective depends on the relative level of significance from increased cost versus extended life expectance. Therefore, state highway agencies are recommended to conduct their own analyses to determine the cost-effectiveness of SMA.

INTRODUCTION

Stone matrix asphalt (SMA), also called stone mastic asphalt, is a durable and rut-resistant gap-graded asphalt mixture that relies on stone-on-stone contact to offer strength and a rich mortar binder to provide durability (Hughes, 1999). SMA was first introduced into the United States in the early 1990s through the efforts of a Technical Working Group established by the Federal Highway Administration. Ever since, SMA has gained popularity among state and local highway agencies as a premium asphalt mixture to enhance field performance and extend life expectancy of asphalt pavements and overlays. Typically, SMA is comprised of 70 to 80 percent coarse aggregate, 8 to 12 percent filler, and 6 to 7 percent asphalt binder. Small amounts of cellulose or mineral fibers are usually added into the mixture to prevent asphalt draindown during silo storage and transportation. SMA is generally more expensive than conventional Superpave dense-graded mixtures containing polymer-modified asphalt binders, mainly due to higher asphalt contents, requirements for more durable aggregates, and inclusion of fibers as stabilizers. Despite that, it is oftentimes believed that the higher cost of SMA can be offset by the increase in life expectancy because of improved rutting resistance and durability.

Over the past few decades, a number of research studies were conducted to characterize the engineering property of SMA through laboratory testing. Most of these studies included a control dense-graded mixture, which consisted of the same asphalt binder, but not necessarily a polymer-modified binder, as the SMA. In general, these studies indicated that SMA had better rutting resistance than conventional Superpave dense-graded mixtures, which was likely attributed to the stone-on-stone aggregate structure (Johnson, 2005; Qiu and Lum, 2006; Asfaw, 2011; NCAT, 2017). In addition, SMA showed better results in moisture susceptibility tests. It was interpreted that the improved resistance to moisture damage of SMA was due to a thicker asphalt film between the aggregate particles (Mogawer and Stuart, 1994; Asi, 2005; Prowell et al., 2009; Haghshenas et al., 2015). However, no consistent trend was reported for the comparisons on mixture stiffness and cracking resistance results. Some studies indicated that SMA had lower stiffness than Superpave mixtures, and oftentimes, these studies reported better resistance to fatigue cracking or low-temperature cracking for SMA due to greater flexibility (Mogawer and Stuart, 1994; Prowell et al., 2009; Saboo and Kumar, 2016). Other studies, however, showed an opposite trend that SMA was more brittle and more susceptible to cracking and fatigue damage compared to Superpave dense-graded mixtures (Asi, 2005; NCAT, 2017). A few studies evaluated the aging characteristics of SMA and they consistently found that SMA experienced a slower rate of aging in the field and laboratory (Mogawer and Stuart, 1994; Han et al., 2015; Wu et al., 2016).

Meanwhile, several studies were undertaken to evaluate the field performance of SMA. One of the most notable study was National Cooperative Highway Research Program (NCHRP) project D9-8, which monitored the performance of 85 SMA pavement sections in the United States (Brown et al., 1997). Most of these sections were constructed between 1992 and 1996 and were on high traffic volume routes. As of the time pavement survey was conducted, these sections were approximately two to six years old. Field performance data indicated that SMA sections showed outstanding performance in terms of rutting and cracking resistance. Most of these sections had rut depth of less than 4 mm and less cracking than the corresponding dense-graded sections. Similar findings had also been reported by others (Watson and Jared, 1996; EAPA, 1998). In addition to improved field performance, SMA pavements offer functional benefits such as improved visibility, reduced splash and spray, increased frictional resistance, and noise reduction (Hoppe, 1991; Polcak, 1994; Rockliff, 1996; Bellin, 1998; Hughes, 1999). Data collected at the National Center for Asphalt Technology (NCAT) Test Track showed that a SMA section provided a maximum of 2 dB(A) reduction in noise and approximately 15% increase in surface friction compared to the Superpave dense-graded section with the same granite aggregates and styrene-butadiene-styrene (SBS) modified asphalt binder (Yin and West, 2018).

The objective of this study was to quantify and compare the performance and life-cycle cost benefits of SMA versus those of polymer-modified Superpave dense-graded mixtures used on similar trafficked highways. To accomplish the objective, market analysis was first performed to determine the current usage of SMA through surveys of state Department of Transportation (DOTs) and state asphalt pavement associations (SAPAs). Pavement management system (PMS) data were then requested from states that routinely use SMA to conduct performance analysis. The goal of the analysis was to compare the long-term field performance of pavement sections with SMA and polymer-modified Superpave dense-graded mixtures. Information gathered from the market analysis and performance analysis was then used as inputs to compare the life-cycle cost between these two surface mixtures. Results obtained from this study provide highway agencies with additional guidance regarding the use of SMA as a premium asphalt mixture.

MARKET ANALYSIS

Survey of SAPAs showed that at least 18 states currently use SMA on a routine basis; these states are highlighted in the map in Figure 1. In July 2016, an email-based survey questionnaire was sent to representatives from 20 highway agencies to gather the following information regarding their use of SMA: mixture selection policy, mixture design specification, bid item numbers, cost and tonnage of SMA and comparable Superpave dense-graded mixtures, and publications that document the statewide performance of SMA pavements. Responses from 16 agencies were received with an 80% response rate.

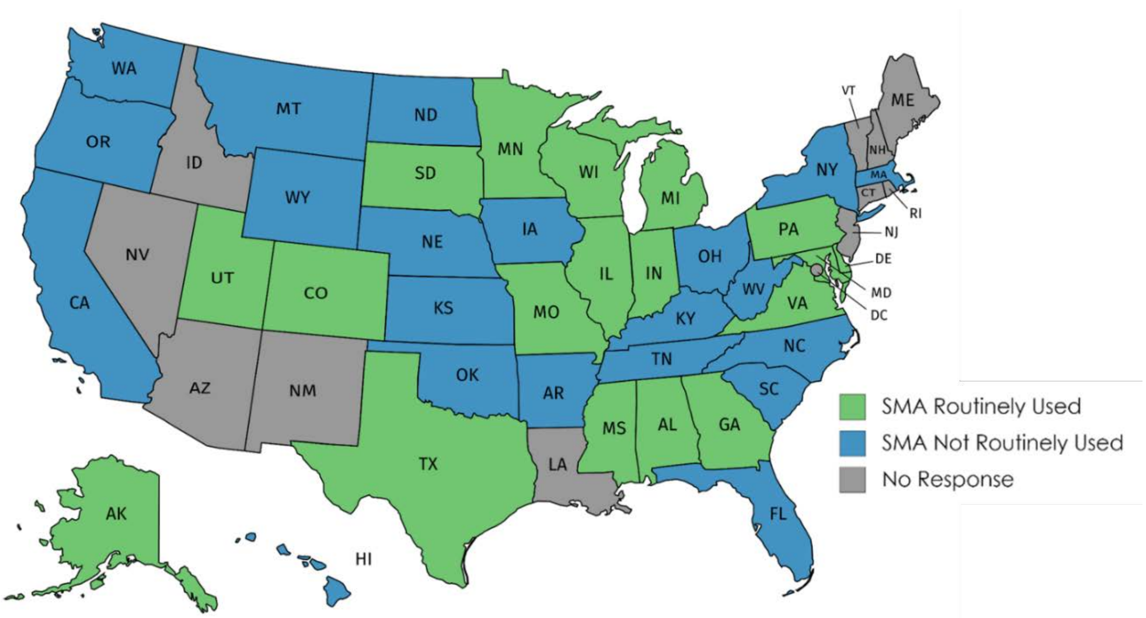


Figure 1. SMA Usage in the United States

Among the 16 agencies that responded to the survey, ten have specific mixture selection policy for using SMA, while the rest indicated that the use of SMA was a decision by the state or district pavement engineer. In general, SMA is used on state and interstate routes and projects with high traffic volumes. Additionally, SMA is considered on projects where frequent maintenance is costly and projects where the higher cost can be justified by the improved performance. Regarding SMA mix design procedure, nine agencies follow

AASHTO R 46-08, Standard Practice for Designing Stone Matrix Asphalt (SMA), or a modified version of it. Five agencies have their own specifications while the other two agencies follow AASHTO R 35, Standard Practice for Superpave Volumetric Design for Asphalt Mixtures. From 2011 to 2015, the total tonnage of SMA produced in various states ranged from approximately 68,000 to 1,872,000 tons. The three states with the highest SMA tonnage were Maryland, Alabama, and Utah, respectively. Over this five-year period, only Alabama, Illinois, and Maryland produced more SMA than polymer-modified Superpave dense-graded mixtures on similar trafficked highways.

Figure 2 compares the five-year average weighted bid price of SMA and polymer-modified Superpave mixtures, where the weighted bid price is calculated as the sum of project bid price times the project tonnage divided by the total tonnage for that mixture for the year (Equation 1). In the figure, the bars represent the average weighted bid price from 2011 and 2015, and the whiskers denote one standard deviation from the average weighted bid price. Over this five-year period, the cost of SMA was consistently higher than that of comparable Superpave mixtures. The difference in the bid price of these two mixtures ranged from \$6 to \$31 per ton. As discussed previously, the higher cost of SMA was likely due to higher asphalt contents, requirements for more cubical and durable aggregates, and inclusion of fibers as stabilizers. In addition, several agencies noted that recycled materials, including reclaimed asphalt pavements (RAP) and recycled asphalt shingles (RAS), are not permitted in SMA but are allowed in Superpave mixtures. Additional factors that could also contribute to the higher cost of SMA include reduced plant versatility and shortened paving windows.

$$Weighted\ Bid\ \mathrm{Pr}\,ice = \frac{\sum T_i P_i}{\sum T_i} \qquad \textbf{Equation 1}$$

Where:

T_i = tonnage of project *i*; and
P_i = Unit bid price of project *i*.

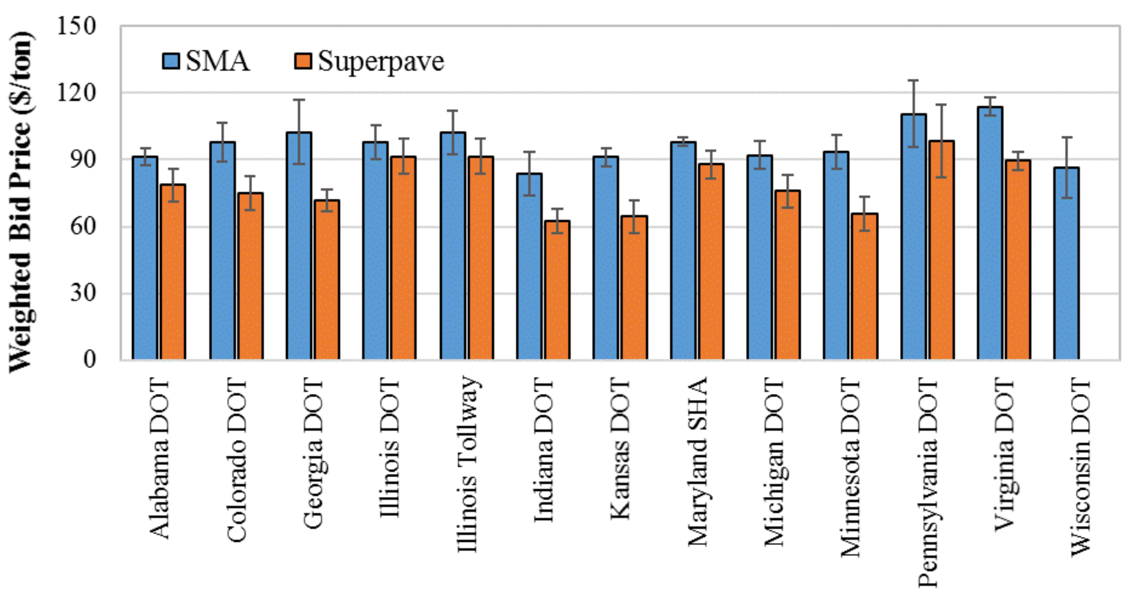


Figure 2. Weighted Bid Price of SMA versus the Superpave Dense-Graded Mixture

PERFORMANCE ANALYSIS

Pavement management system (PMS) data from nine highway agencies were received and analyzed to compare the long-term field performance of SMA versus polymer-modified Superpave dense-graded mixtures used for equivalent road categories and pavement types. Performance analyses were conducted using the network-level analysis approach to determine the life expectancy of these two mixtures. Note that most pavement sections included in the analyses were constructed within the past ten years, and thus, their longer-term performance data is not available and needs to be predicted using a performance deterioration model. In most cases, an s-shaped logistic model, as described in Equation 2, was used because it could simulate the general development trend of pavement condition, where it deteriorates slowly during the first few years, but afterwards, drops at a significantly faster rate, and finally shows a steady decrease to approach a low boundary (Jackson et al., 1996; Wang, 2016).

$$y(t) = a - \frac{b}{1 + ce^{-dt}} \qquad \textbf{Equation 2}$$

Where:

$y(t)$ = pavement condition at time t.
a, b, c, and d = model coefficients,
t = pavement age

For data analysis, the collected PMS data was first used to calibrate the selected performance models, which were then used to predict the service life of pavement sections with SMA and Superpave dense-graded mixtures. For agencies using individual pavement distresses (e.g., rutting, cracking, and roughness) for pavement maintenance and rehabilitation decisions, performance was evaluated with regard to each distress. Otherwise, PMS data was analyzed using composite pavement condition indexes. PMS data from a total of 407 SMA and 807 Superpave pavement sections were analyzed. Due to space limitations, only two examples of the performance analysis are presented below.

Example 1 – Michigan DOT Data

The Michigan DOT conducts pavement distress survey by videotaping the pavement surface (MDOT, 2016). The videos are analyzed to identify distress type, extent, and severity, which are then used to compute a composite pavement condition rating termed Distress Index (DI). The DI starts at zero for a distress-free condition and increases as the pavement deteriorates. A DI of 50 or higher indicates the need for rehabilitation or reconstruction. This DI threshold also corresponds to a remaining service life of zero. Since most of the SMA sections identified by Michigan DOT were composite pavements, a performance comparison between SMA and polymer-modified Superpave dense-graded mixtures for flexible pavements was not available.

Figure 3 presents the DI data of 113 composite pavement sections; the dots represent the average DI values of pavement sections with the same age, and the whiskers denote one standard deviation from the average DI values. Twenty-three of these sections had SMA as the surface layer and the rest used comparable Superpave dense-graded mixtures. The asphalt layer of both SMA and Superpave sections had similar thickness ranging from 3.5 to 5.0 inches. The average daily truck traffic (ADTT) of these sections was between 1,000 and 6,000. For data analysis, an s-shaped performance model (Equation 3) was first used to fit the measured DI data based on non-linear regression. This model has been used by the Michigan DOT to predict the development of pavement distresses since 1995 (Kuo, 1995). Once the model coefficients were determined, the pavement service life was then predicted using a DI threshold of 50. Based on the results in Figure 10, composite pavement sections with SMA and polymer-modified Superpave dense-graded mixtures showed similar performance and were predicted to last for 22.2 and 21.3 years, respectively.

$$DI = m\left(\frac{1}{1+ce^{-\gamma * t}} - \frac{1}{1+c}\right) \quad \textbf{Equation 3}$$

Where:

t = pavement age,
m, γ, c = model coefficients

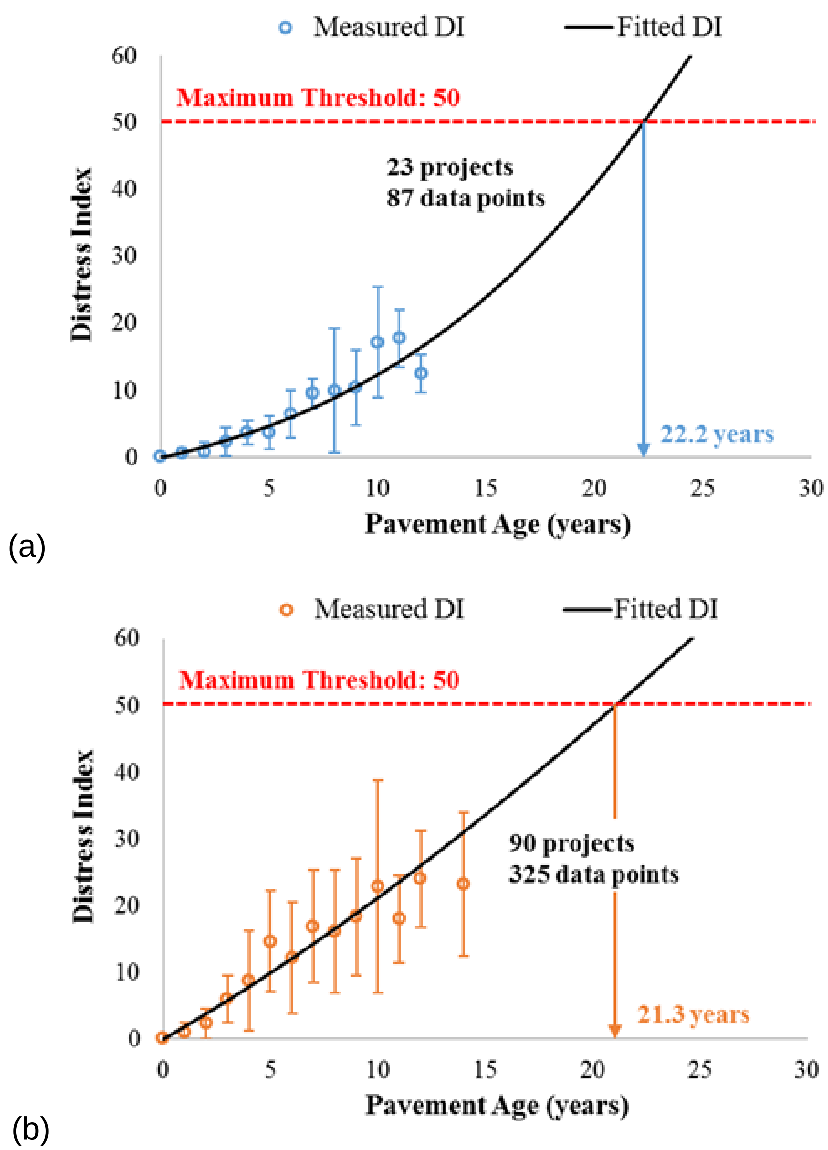


Figure 3. Michigan DOT Pavement DI Data; (a) SMA, (b) Polymer-Modified Superpave Mixture

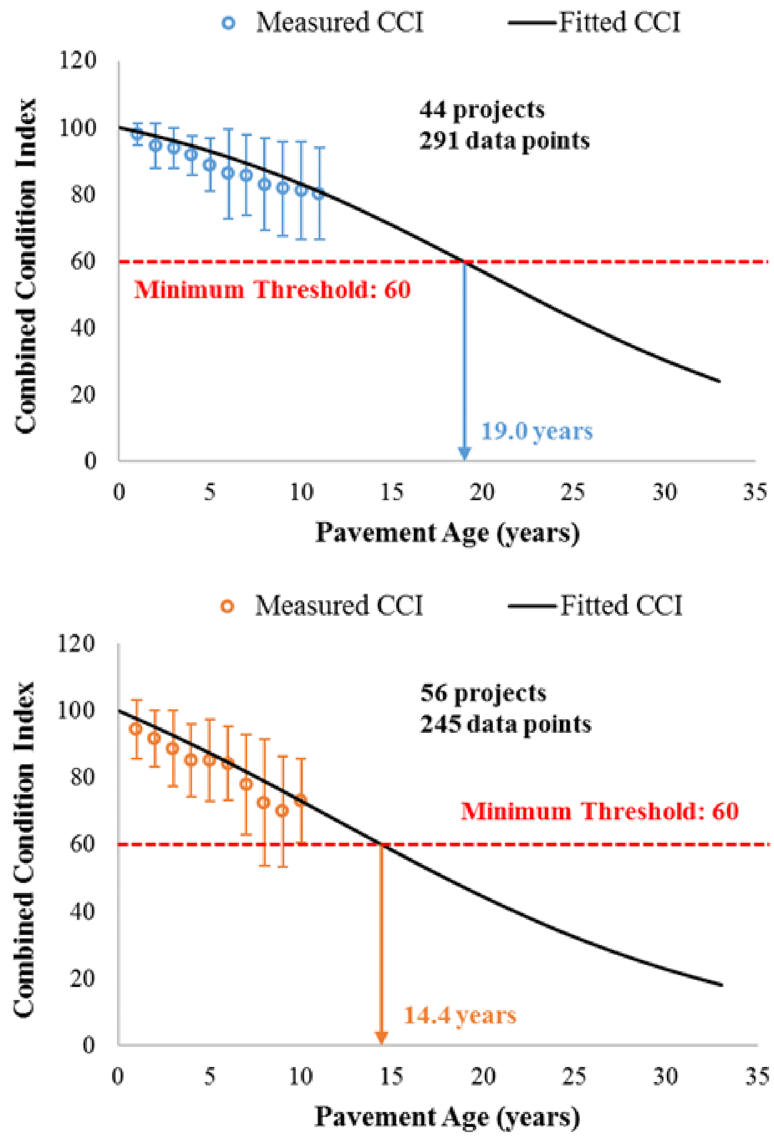
Example 2 – Virginia DOT Data

The Virginia DOT uses an Automated Road Analyzer (ARAN) van to collect pavement data based on digital images and automated crack detection methodology (VDOT, 2012). The ARAN is equipped with a distance measuring instrument, a laser rut measuring system, a laser longitudinal profiling system, a global positioning system, and downward facing cameras. The collected pavement distress data are then analyzed to calculate the Load Related Distress Rating (LDR) and Non-load Related Distress Rating (NDR). The LDR is determined based on alligator cracking, wheel path patching, and rutting, and the NDR considers longitudinal and transverse cracking, non-wheel path patching, and bleeding. The lower of the two ratings is defined as the Critical Condition Index (CCI). The CCI has a scale of zero to 100, with 100 indicating a distress-free condition and zero for a completed failed condition. Pavement sections with a CCI value of 60 or lower are considered "deficient" and need immediate rehabilitation and reconstruction.

Figure 4 and Figure 5 present the CCI results of 100 flexible pavement sections and 47 composite pavement sections, respectively; the dots represent the average CCI values of sections with the same age, and the whiskers refer to one standard deviation from the average CCI values. Both SMA and Superpave pavement sections had similar design traffic levels. The thickness of the surface layer ranged from 1.5 to 3 inches. For performance analysis, an s-shaped logistic model (Equation 2) was first used to fit the measured CCI results versus pavement age. The pavement service life was then predicted with a minimum CCI threshold of 60. As shown in Figure 4, flexible pavement sections with SMA had a life expectancy of 19.0 years, which was approximately five years longer than that of polymer-modified Superpave dense-graded mixtures (i.e., 14.4 years). A greater life extension of approximately ten years was observed for composite pavement sections in Figure 5, where SMA and Superpave mixtures were predicted to last for 23.1 and 12.8 years, respectively. It should be noted that the predicted service lives of SMA and comparable Superpave mixtures discussed above were determined based on extrapolation using a non-linear performance model, which might not necessarily represent the observed service lives in the field.

Summary

Table 1 and Table 2 summarize the performance analysis results of SMA versus polymer-modified Superpave dense-graded mixtures for flexible and composite pavements, respectively. The predicted service life was determined based on the agency's PMS data for pavement performance (either individual distresses or composite condition indexes) to reach a specific threshold. In most cases, SMA showed better performance and had a longer predicted service life than comparable Superpave dense-graded mixtures used on similar trafficked highways. The life extension of SMA compared to Superpave mixtures varied from five to eight years for flexible pavements and varied from one to 13 years for composite pavements. For the four exceptional cases where Superpave mixtures showed better performance than SMA, the difference in life expectancy between these two mixtures was less than two years. Note that performance analyses for several highway agencies were based on a limited number of pavement sections (i.e., less than 5); thus, those results should be interpreted with caution as different conclusions can be obtained as additional data become available. In addition, most pavement sections included in the analyses only have performance data available for ten years or less, but their predicted service lives to failure fall into a longer time period of at least 15 years. Therefore, the predicted life expectancy of SMA and comparable Superpave mixtures based on extrapolation of a performance model may not necessarily represent the observed service lives in the field. There is a need to continue monitoring the long-term performance of these mixtures to validate the performance benefits of SMA.



(a)

(b)

Figure 4. Virginia DOT Flexible Pavement CCI Data; (a) SMA, (b) Polymer-Modified Superpave Mixture

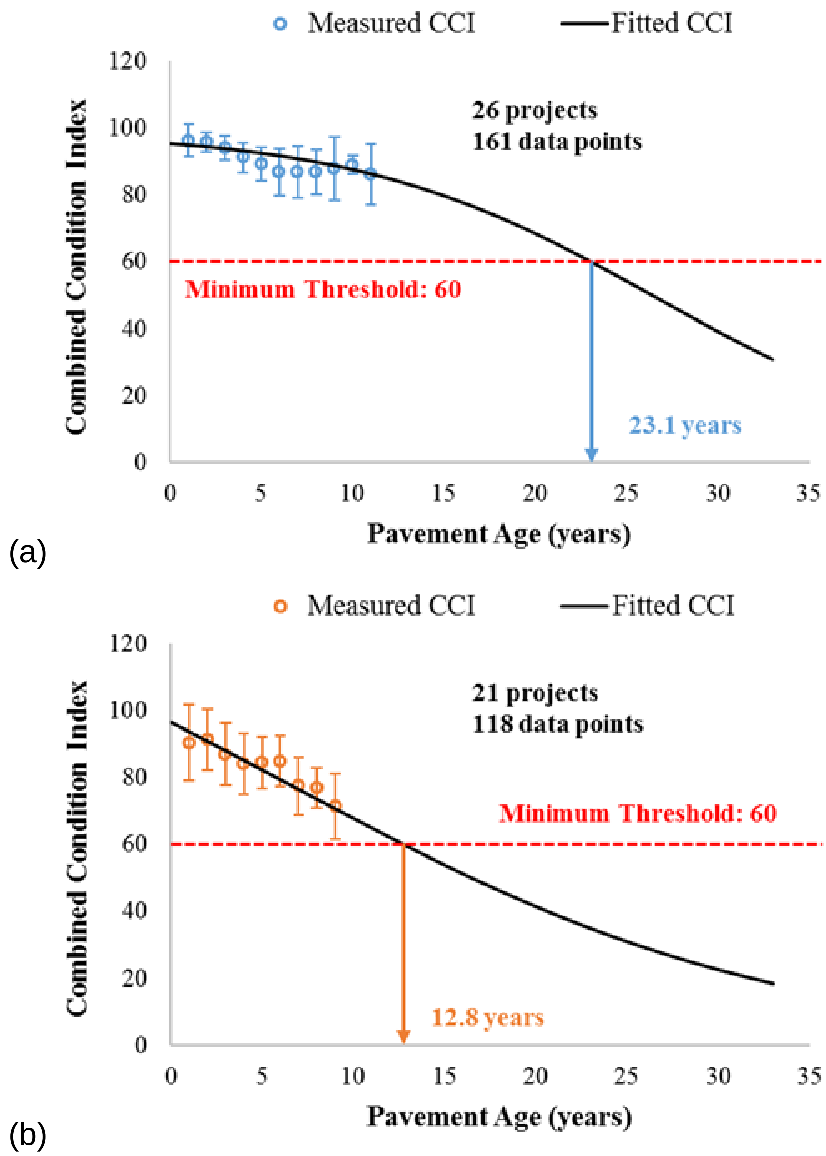


Figure 5. Virginia DOT Composite Pavement CCI Data; (a) SMA, (b) Polymer-Modified Superpave Mixture

Table 1. Summary of Performance Analysis Results for Flexible Pavements

Highway Agency	Performance Measure	Number and Max. Field Life of Pavement Sections		Predicted Service Life (Years)		SMA Life Extension	
		SMA	Superpave	SMA	Superpave	Years	Percentage
Alabama DOT	Pavement Condition Rating (PCR)	33 (12 years)	146 (11 years)	16.2	16.6	—	—
Colorado DOT	Rutting Fatigue Cracking Transverse Cracking Longitudinal Cracking	52 (9 years)	111 (9 years)	17.0	17.4	—	—
Georgia DOT	PACES Rating	4 (16 years)	4 (13 years)	16.0	11.0	5.0	45%
Maryland SHA (Interstate)	Rutting Cracking Index (CI)	103 (16 years)	31 (17 years)	24.8	26.9	—	—
Maryland SHA (Principal Arterial)	Rutting Cracking Index (CI)	60 (14 years)	158 (17 years)	32.2	24.0	8.2	34%
Minnesota DOT	Ride Quality Index (RQI) Surface Rating (SR)	5 (11 years)	4 (6 years)	16.6	11.3	5.3	47%
Virginia DOT	Critical Condition Index (CCI)	44 (11 years)	56 (10 years)	19.0	14.4	4.6	32%

Table 2. Summary of Performance Analysis Results for Composite Pavements

Highway Agency	Performance Measure	Number and Max. Field Life of Pavement Sections		Predicted Service Life (Years)		SMA Life Extension	
		SMA	Superpave	SMA	Superpave	Years	Percentage
Illinois Tollway	Overall Condition Rating Survey (CRS)	2 (5 years)	2 (10 years)	13.5	9	4.5	50%
Maryland SHA (Principal Arterial)	Rutting Cracking Index	43 (15 years)	56 (15 years)	21.8	19.6	2.2	11%
Michigan DOT	Overall Distress Index (DI)	23 (12 years)	90 (14 years)	22.2	21.3	0.9	4%
Pennsylvania DOT (Interstate)	Overall Pavement Index (OPI)	5 (12 years)	17 (13 years)	21.1	22.2	—	—
Pennsylvania DOT (Non-Interstate)	Overall Pavement Index (OPI)	5 (14 years)	108 (13 years)	24.5	11.0	13.5	123%
Virginia DOT	Critical Condition Index (CCI)	26 (11 years)	21 (9 years)	23.1	12.8	10.3	80%

CASE STUDIES ON LIFE-CYCLE COST ANALYSIS

Case studies on deterministic and probabilistic life-cycle cost analysis (LCCA) were conducted for three highway agencies to determine if the higher cost of SMA could be justified by the improved performance and extended life expectancy. For each case study, information gathered from the market analysis and performance analysis of that specific agency was used as inputs to determine and compare the net present value (NPV) and equivalent uniform annual cost (EUAC) of SMA versus polymer-modified Superpave dense-graded mixtures on similar trafficked highways. The assumption made in the LCCA was to construct a two-inch thick asphalt overlay with these two alternative mixtures using the recent five-year (i.e., 2011 to 2015) weighted bid prices and predicted service lives for the respective state. Discount rates were selected by following the agency's current practice. As shown in Equation 4 and Equation 5, the NPV and EUAC were determined based on the present value of the first overlay cost, future value of the replacement overlay cost, and salvage value at the end of the analysis period.

$$NPV = PV_0 + \sum FV_i * \left[\frac{1}{(1+r)^{n_i}} \right] + SV * \left[\frac{1}{(1+r)^{n_s}} \right]$$ **Equation 4**

Where:

PV_0 = present value of the first overlay cost;
FV_i = future value of the i^{th} overlay cost;
SV = salvage value at the end of analysis period;
r = discount rate;
n_i = time to apply the i^{th} overlay; and
n_s = analysis period.

$$EUAC = NPV * \left[\frac{r(1+r)^{n_s}}{(1+r)^{n_s} - 1} \right]$$ **Equation 5**

Although traditional LCCA requires an analysis period (35 to 40 years) that includes a minimum of one pavement rehabilitation activity, a shorter analysis period was used in the study to compare the life-cycle cost benefits of SMA versus polymer-modified Superpave mixtures. For each case study, the analysis period was selected using the predicted service life of SMA determined from the performance analysis (Table 1 and Table 2). Considering that SMA and polymer-modified Superpave mixtures were used on roadways with equivalent pavement types and similar traffic levels, user costs associated with these two mixtures were likely comparable, and thus, were not included in the analysis. In addition, costs of routine maintenance and traffic control were not considered because these costs would have limited effect on the EUAC when discounted to the present value. Due to space limitations, only two LCCA case studies are provided below.

Example 1 – Michigan DOT Data

According to Figure 2, the recent five-year average weighted bid price of SMA was $92 per ton, which was approximately 21% higher than that of polymer-modified Superpave dense-graded mixtures (i.e., $76 per ton). The performance analysis results showed that SMA and Superpave mixtures had predicted service lives of 22 years and 21 years, respectively. Thus, an analysis period of 22 years was selected in this case study. Figure 6 presents the deterministic LCCA models and the corresponding cost expenditure streams for the two alternatives mixtures.

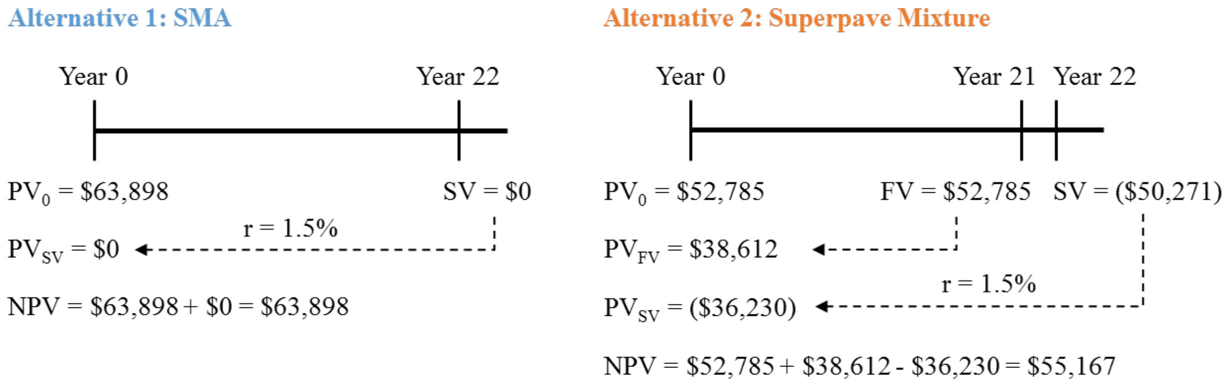


Figure 6. LCCA Models and Cost Expenditure Streams for Michigan DOT

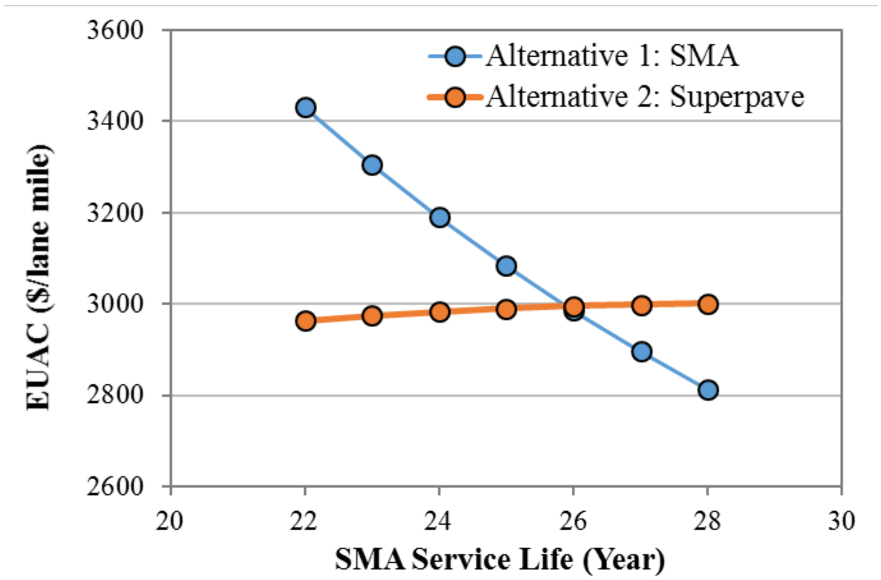


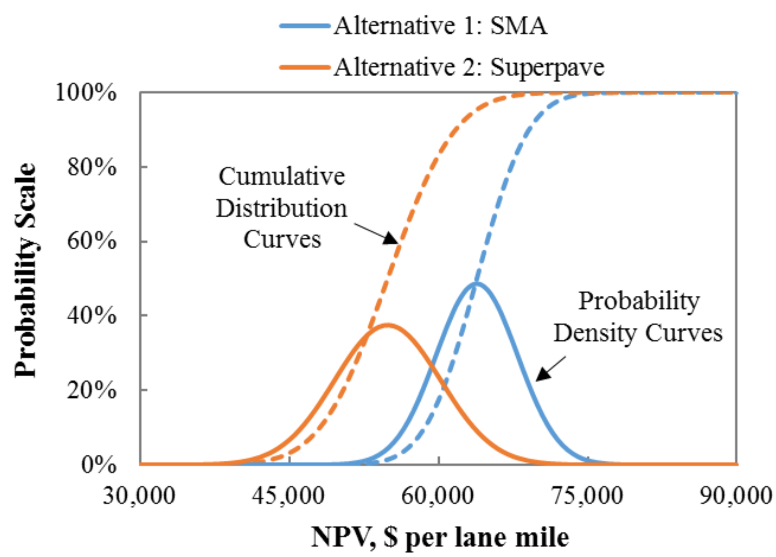
Figure 7. Additional Deterministic LCCA Results for Michigan DOT

In Alternative 1, the SMA overlay was expected to last 22 years. The agency cost for the initial construction (i.e., present value at year 0) was $63,898 per lane mile. At year 22, the overlay would be replaced; thus, the salvage value at the end of the analysis period (i.e., year 22) would be $0. The NPV for Alternative 1 was $63,898 per lane mile.

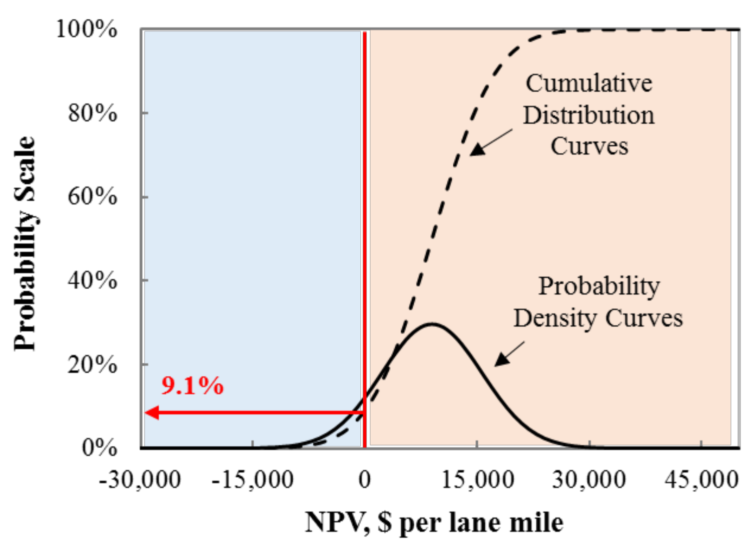
In Alternative 2, the Superpave overlay was expected to last 21 years with a cost of $52,785 per lane mile. At year 21, the overlay would be replaced with a new one. The future value of the new overlay was assumed identical to the cost of the first overlay (i.e., $52,785). The new overlay was also expected to last 21 years.

At the end of the analysis period (i.e., year 22), the overlay would have a remaining life of 20 years with a salvage value of $−50,271. The salvage value of the new overlay was calculated as a prorated portion of its cost. Using the 2016 real discount rate of 1.5% (MDOT, 2017), the new overlay cost (at year 21) and its salvage value (at year 22) were then discounted back to year 0 as $38,612 and $−36,230, respectively. The NPV for Alternative 2 was $55,167 per lane mile.

Based on the deterministic LCCA results, SMA was not as cost-effective as polymer-modified Superpave dense-graded mixtures, as indicated by a higher NPV. Therefore, in this example, the higher cost of SMA was not justified by the extended life expectance. Additional analysis showed that a minimum service life of approximately 26 years was needed for SMA to be more cost-effective than Superpave mixtures with a predicted life of 21 years (Figure 7).



(a)



(b)

Figure 8. Probabilistic LCCA Results for Michigan DOT; (a) NPV Distribution Curves, (b) NVP Difference Distribution Curves

In addition to deterministic LCCA, the probabilistic analysis was also conducted to compare the cost benefits of SMA and polymer-modified Superpave dense-graded mixtures by considering the variability of the inputs. For the probabilistic analysis, the two inputs of weighted bid price and discount rate were specified following a normal distribution. The standard deviations of the weighted bid price of SMA and Superpave mixes were $6.1 and $7.2 per ton, respectively. The standard deviation of the discount rate was assumed 0.15%, which yielded a coefficient of variance of 10%. One thousand Monte Carlo simulations were carried out to generate a probability distribution of NPV for the two alternative mixtures. As shown in Figure 8(a), SMA had a higher mean NPV than Superpave mixtures, indicating a higher life-cycle cost. In addition, the NPV of SMA was considerably higher at other major probability levels (e.g., 25%, 75%, and 90%). Figure 8(b) presents the distribution curve of cost difference between SMA and the Superpave mixture, where the majority of Monte Carlo simulations yielded a higher NPV for SMA than Superpave mixtures. The probability level for SMA being more cost-effective was only about 9.1%.

Example 2 – Virginia DOT Data

Cost information gathered in the market analysis showed that the recent five-year average weighted bid prices of SMA and comparable Superpave dense-graded mixtures were $114 and $89 per ton, respectively. According to the performance analysis results, composite pavement sections with SMA had a predicted service life of 23 years, which was ten years longer than that of polymer-modified Superpave mixtures (i.e., 13 years). An analysis period of 23 years was used in the case study. Figure 9 presents the deterministic LCCA models and the corresponding cost expenditure streams for SMA and Superpave mixtures.

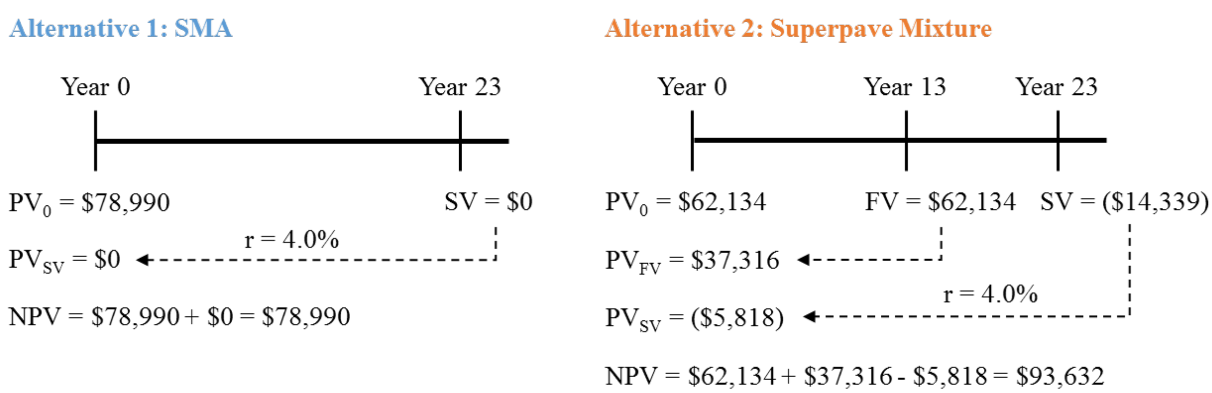


Figure 9. LCCA Models and Cost Expenditure Streams for Virginia DOT

In Alternative 1, the SMA overlay was expected to last 23 years. The agency cost for the initial construction (i.e., present value at year 0) was $78,990 per lane mile. At year 23, the overlay would be replaced; thus, the salvage value at the end of analysis period (i.e., year 23) would be $0. The NPV for Alternative 1 was $78,990 per lane mile.

In Alternative 2, the Superpave overlay was expected to last 13 years with a cost of $62,134 per lane mile. At year 13, the overlay would be replaced with a new one. The future value of the new overlay was assumed identical to the cost of the first overlay (i.e., $62,134). The new overlay was also expected to last 13 years. At the end of the analysis period (i.e., year 23), the overlay would have a remaining life of 3 years with a salvage value of $−14,339. The salvage value of the new overlay was calculated as a prorated portion of its cost. Using an agency specified discount rate of 4.0%, the new overlay cost (at year 13) and its salvage value (at year 23) were then discounted back to year 0 as $37,316 and $−5,818, respectively. The NPV for Alternative 2 was $93,632 per lane mile.

The deterministic analysis showed that SMA was more cost-effective than Superpave dense-graded mixtures with polymer-modified asphalt binders, with an approximately 16% savings in NPV over a 23-year analysis period. Therefore, in this case study, the higher cost of SMA was justified by the improved pavement performance and extended life expectance. A similar conclusion was also obtained from the probabilistic LCCA. As shown in Figure 10, approximately 99.8% of the Monte Carlo simulations yielded a lower NPV for SMA relative to Superpave mixtures.

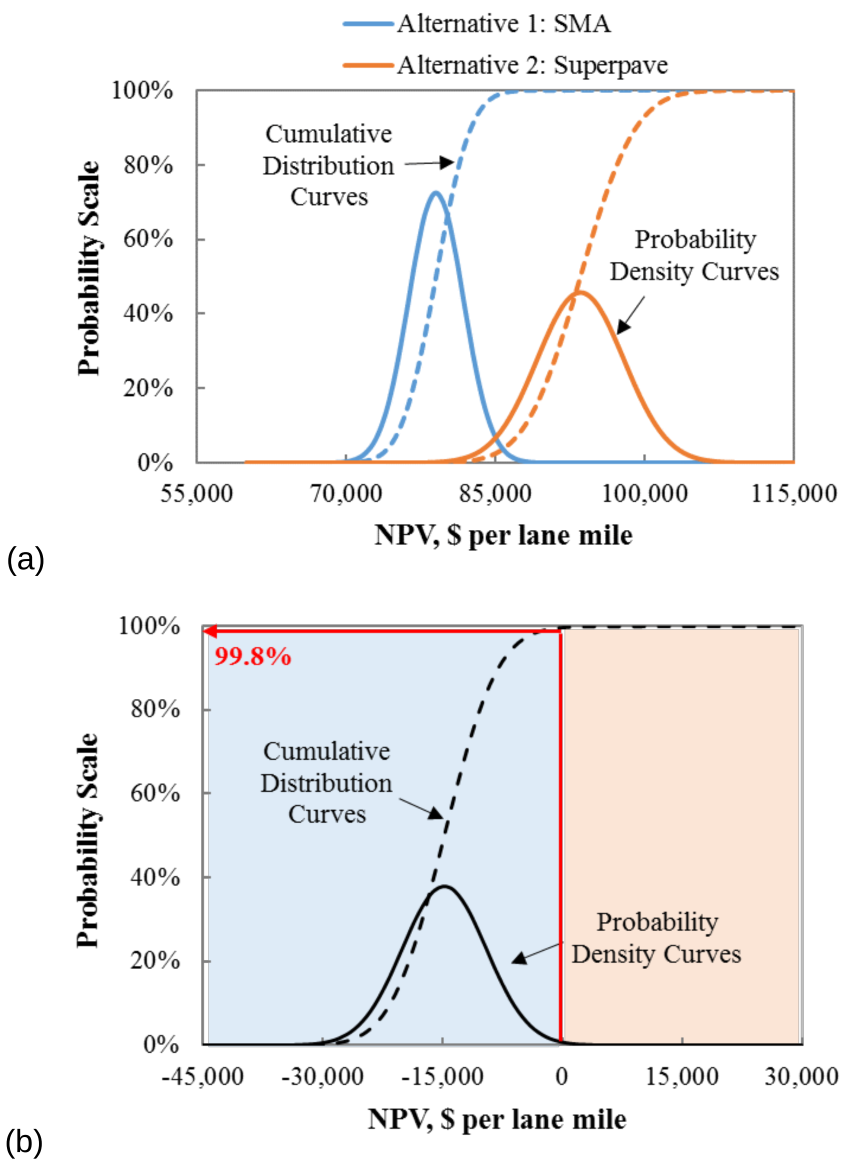


Figure 10. Probabilistic LCCA Results for Virginia DOT; (a) NPV Distribution Curves, (b) NVP Difference Distribution Curves

Summary

Figure 11 summarizes the deterministic LCCA results for comparing the EUAC of SMA versus polymer-modified Superpave dense-graded mixtures used on similar trafficked highways. For the Michigan DOT results, SMA had higher a EUAC than Superpave dense-graded mixtures, which indicated that SMA was not as cost-effective as Superpave mixtures and that a greater extension in pavement life was needed for SMA to justify its higher cost. However, the Virginia DOT and Maryland SHA data showed a different trend, where SMA was more cost-effective than polymer-modified Superpave mixtures as indicated by lower EUACs. The same conclusions were obtained in the probabilistic analysis results in terms of the percentage of simulation outcomes with a lower NPV for SMA. Specifically, agencies that showed higher EUAC for SMA in the deterministic approach had probability levels lower than 50% in the probabilistic analysis, and vice versa. Overall, there was no consistent conclusion among the states for comparing the life-cycle cost of SMA versus polymer-modified Superpave dense-graded mixtures. Whether or not SMA is more cost-effective depends on the relative level of significance from increased cost versus extended life expectance. Therefore, state highway agencies should conduct their own analyses to determine the cost-effectiveness of SMA within their states.

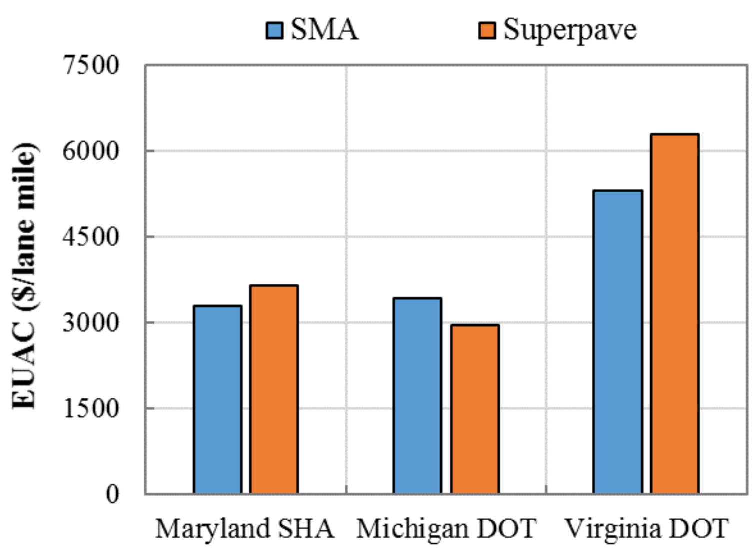


Figure 11. Summary of Deterministic LCCA EUAC Results

SUMMARY AND CONCLUSIONS

This study was undertaken to quantify and compare the performance and life-cycle cost benefits of SMA versus those of polymer-modified Superpave dense-graded mixtures. Market analysis was first conducted to determine the usage of SMA through surveys of state DOTs and SAPAs. Field performance data was then collected from nine highway agencies to determine if SMA outperformed comparable Superpave mixtures used for equivalent roadway categories and pavement types. Finally, case studies on deterministic and probabilistic LCCA were performed to compare the life-cycle cost of these two mixtures. The following conclusions were made based on this study:

- Currently, SMA is used on a routine basis by at least 18 state highway agencies on state and interstate routes with high traffic volumes and on projects where frequent maintenance is costly and disruptive to high traffic volumes.
- The most recent five-year average weighted bid price of SMA was 7% to 43% higher than that of Superpave dense-graded mixtures with polymer-modified asphalt binders. The difference ranged from $6 to $31 per ton among the states.
- SMA generally had equivalent or better field performance than conventional Superpave dense-graded mixtures used on similar trafficked highways. For cases where SMA had better performance, the life extension of SMA varied from one to 13 years among the states and varied for different pavement types. It is worth noting that the predicted service lives of SMA and Superpave mixtures were based on extrapolation of limited field performance data; thus, longer-term performance data is needed to verify the performance benefits of SMA.
- There was no consistent conclusion for comparing the life-cycle cost of SMA versus conventional Superpave dense-graded mixtures. Whether or not SMA is more cost-effective depends on the relative level of significance from increased cost versus extended life expectance. State highway agencies should conduct their own analyses to determine the cost-effectiveness of SMA within their states.

ACKNOWLEDGEMENTS

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DISCLAIMER

The opinions and conclusions expressed or implied in the paper represent a consensus of the authors but do not necessarily represent that of NAPA, FHWA, and the abovementioned highway agencies.

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Test Method Analysis for Different Types of Fillers Used in the SMA Mix Through Semi-Circular Bending (SCB) Fracture Energy Test

Pedro Limón-Covarrubias (Corresponding Author)
Department of Civil and Topography Engineering, Guadalajara University
1421 Blvd. Marcelino García Barragán, Guadalajara, Jal., México, C.P. 44430
Tel: (52) 1378 5900 Ext: 27699
Fax: (52) 3619 8054
Ingenieria_limon@hotmail.com

Gonzalo Valdés-Vidal
Department of Civil Engineering, Universidad de La Frontera, Temuco, Chile
01145 Ave. Francisco Salazar, Temuco - Chile - Casilla 54-D
Tel: (56) 45 2325680
Gonzalo.valdes@ufrontera.cl

Oscar Javier Reyes-Ortiz
Department of Civil Engineering, Nueva Granada Military University
Bogotá, Colombia
Tel: (57) 3208655665
Oscar.reyes@unimilitar.edu.co

Rey Omar Adame-Hernández
LASFALTO, Guadalajara, México
3042 Agua Marina Street, Fracc. Agua Blanca, Zapopan, Jal., México, C.P.45236
Tel: (52) 3684 7304
Fax: (52) 3283 4758
Omar.adame@lasfalto.com.mx

David Avalos-Cueva
Department of Civil and Topography Engineering, Guadalajara University
1421 Blvd. Marcelino García Barragán, Guadalajara, Jal., México, C.P. 44430
Tel: (52) 1378 5900 Ext: 27699
Fax: (52) 3619 8054
David.avalos@academicos.udg.mx

ABSTRACT

In most cases, Stone Mastic Asphalt (SMA)-type mixtures placed in thin layers and submitted to stress may generate early cracks (e.g., improperly linked to the underlying layer, placed over previously cracked asphalt pavement, placed over Portland cement concrete slabs). However, the filler used in SMA production is very influential the performance of the mix. Fillers used in this type of mix have a low plastic index or are inert (calcium carbonate or lime), so it is important to know the effect that each material can have on the possible fissuring and cracking process of the SMA mixture. The objective of this study is to present an evaluation of SMA asphalt mixture behavior using different types of filler and temperatures by fracture energy test semi-circular bending (SCB). This research compares results between fracture energy and different types of filler in SMA asphalt mixes performed at temperatures ranging from −5 to 25°C.

INTRODUCTION

Stone Mastic Asphalt Mixtures (SMAs) were developed in Germany at the end of the 1960s by STRABAG and J. Rettenmaier. SMAs were intended to improve deterioration, increase useful life and reduce maintenance costs compared to conventional pavements. However, despite their advantages, their use was not normalized in Germany until 1984, when it began to develop in other countries in Europe, America and Asia (1, 2).

SMA mixtures are asphalt mixtures characterized by a large amount of coarse aggregate, a high proportion of binder and mineral powder, a low amount of intermediate size aggregate and a small amount of stabilizing additive. In this way, a good mineral structure and a high proportion of filler-based mastic are generated, which enable a high carrying capacity without affecting the flexibility of the mixture (3, 4).

The SMA mixtures were conceived with clear and well-defined objectives: to increase the durability, safety and stability of the communication routes and to generate savings in their construction. SMAs are hot-prepared mixtures characterized by being impermeable and resistant to the formation of ruts. Arabani and Ferdowsi (5), Erdlen and Yu (1), NAPA (4), Hainin, Reshi and Niroumand (2), and others have described the advantages of using SMA-type mixtures (Figure 1).

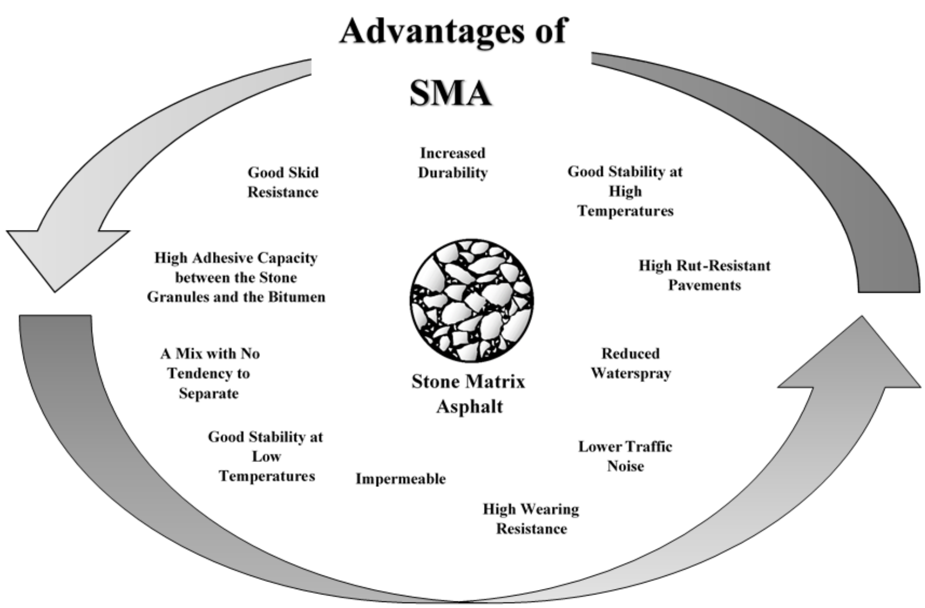

Figure 1. Advantage SMA

Knowing the propagation and time in which deterioration will occur in flexible pavement is the greatest challenge for researchers. This is why several modeling schemes exist to discover the type and time in which failure will occur, which is one of the purposes of this study.

It has been stated that damage due to cracking in an asphalt layer causes the greatest damage, since this is a reflection that some part of the pavement structure has failed (6). In addition, the passage of vehicles across deteriorated asphalt pavement is uncomfortable for the driver, and its repair in some cases is quite expensive. As a result, different laboratory tests have been proposed to mitigate this problem and suggest a more representative model of asphalt layer deterioration in the field (7–9).

Currently, there are methods to prevent this type of failure, such as the stability and Marshall flow and dynamic and static tests, in which it is intended to predict the deterioration and durability of an asphalt layer (6, 10–12). Moreover, there are relatively new tests that study fracture energy, which can provide information on the behavior and durability of an asphalt layer, such as the semicircular bending test (5, 13–16), which is the test used to evaluate the SMA mixtures in this investigation.

Although there are several methods for the study of asphalt pavement or bearing surface cracking, not all of them satisfy the requirements or can accurately measure the different problems they face. Thus, researchers have crafted methods to develop satisfactory theories and have standardized the problems in laboratory settings (6, 17).

The semicircular bending test determines the fracture energy necessary to cause an asphaltic mixture to crack by measuring various physical and/or chemical characteristics and external factors such as temperature or induced damage to the specimen.

The SCB test was presented by Kuruppu, Obara (18) as a quick 3-point bending test. It is used to evaluate resistance to fracture at different temperatures and with different characteristics such as different stony materials, asphalt types, asphalt content and filler types (as in the case of this investigation). In general, it is standard procedure to perform the test at three different temperatures to obtain a record and comparison between materials. This is possible because it uses the finite element method to determine the variation of the load intensity factor with respect to the crack length.

Despite the use of the finite element method to calculate the load with respect to the crack, the fracture energy is still calculated through experimental results of different tests on the specimens, simply because there are factors that make the test difficult to carry out accurately (19). For example: a rock particle is in the direction of the crack and will result in resistance at that point. Thus, it is recommended to test at least six different samples at three different temperatures and loading speeds. The results of are directly affected by the type of mixture, which includes the type of stony material, asphalt, and filler. Moreover, another factor that affects the results is the void ratio in each specimen.

It is important to note that the SCB test is based on the assumption that the fracture energy is absorbed only by the affected area where the fracture occurs and does not affect the rest of the sample, although recent studies have shown that the entire sample is affected (19, 20). However, it is valid to depreciate this theory because the purpose of conducting a test using the SCB method is to measure fracture energy, which means that attention is focused only on knowing the energy required at different temperatures and loading speeds to produce the fracture.

EXPERIMENTAL ANALYSIS

The purpose of the experiments carried out in the present study is to evaluate the performance of the different fillers used in an SMA mixture by means of the SCB fracture energy test. The tests in the SCB were carried out at temperatures of −10, 5, 15 and 25°C. The load was controlled by displacement at a speed of 1 mm/min.

Materials

The stony materials were of basaltic origin and complied with regulation N·CMT·4·04/17 of the Secretariat of Communications and Transportation of Mexico (Secretaria de Comunicaciones y Transporte de México, SCT), and the results are shown in Table 1. The asphalt used for the test was PG 70-16. Under the norms of SCT-N·CMT·4·05·003/16, the values are presented in Table 2.

Table 1. Aggregate analysis

Aggregate Type	Test	Result
Coarse	L.A. abrasion, (%)	13
	Accelerated Weathering, (%)	5
	Crushed particles (%)	100
	Elongated particles (%)	24
	Flat particles (%)	16
	Density (T/m^3)	2.72
	Absorption (%)	0.75
Fine	Sand equivalent (%)	61
	Methylene blue (mg/g)	10
	Density (T/m^3)	2.55

Table 2. Asphalt analysis

	Test	Result
Original binder	Penetration to 25°C 100gr 5 sec (1/10 mm)	69
	Elastic recovery by torsion to 25°C (%)	5
	Softening point 5°C/min. (°C)	49
	Performance grade PG	70
	Cleveland Flashpoint	>260
	Brookfield Viscosity to 135°C SC4-27 12 rpm (cP)	530
	Fail temperature [G*/sinδ=1.0 kPa] (°C)	71.1
	Module DSR to PG [G*/sinδ] (kPa)	1.21
	Phase angle (δ) to PG (°)	82.36
Aged binder RTFO	Loss mass to 163°C (%)	0.57
	Performance grade PG	70
	Fail temperature [G*/sinδ=2.2 kPa] (°C)	70.1
	Module DSR to PG [G*/sinδ] (kPa)	2.23
	Phase angle (δ) to PG (°)	81.11
Aged binder PAV	Module DSR to 34 °C [G*sinδ] (kPa)	2102
	slope (m) BBR test to -6 °C	0.312
	module stiffness BBR test to -6 °C (MPa)	287

The design was carried out in accordance with the AASHTO MP-8 standard, in which a glanulometry adjusted to the limits for a nominal maximal size of 0.5 in (Figure 2) was established.

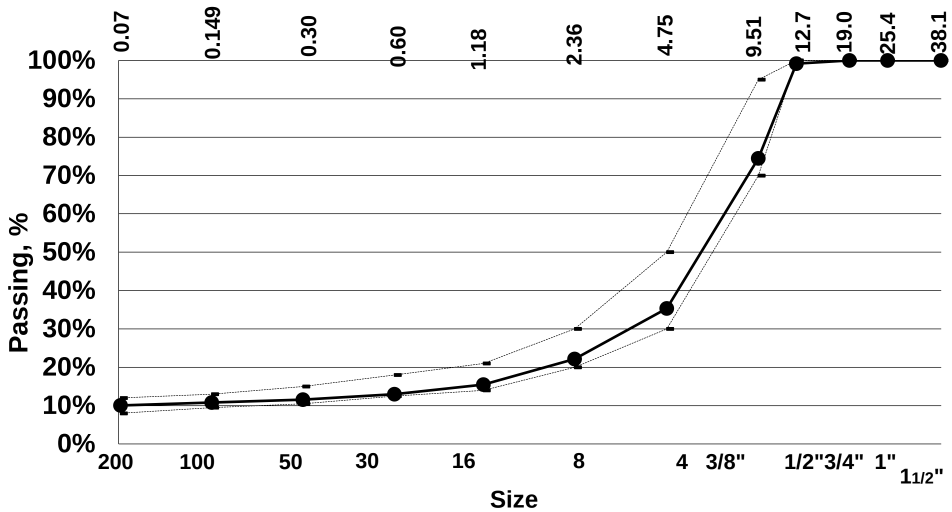

Figure 2. Gradation curve in millimeters (AASHTO MP-8)

The optimal asphalt content was calculated by using a gyratory compactor at 100 turns, with an internal angle of 1.16° and a stress of 600 KPa (Table 3).

Table 3. Volumetric properties of SMA mixtures

AC (%)	G_{mm} (Kg/m^3)	G_{mb} (Kg/m^3)	Air voids (%)	VMA (%)	VFA (%)
6.5	2,393	2,299	3.9	18.3	78.5

Five types of fillers were used for the formation of the SMA mixture. The composition of the fillers was analyzed at the Laboratorio de Microscopia de la Universidad Nacional Autónoma de México (Figure 3). Therefore, it was observed that the industrial-product fillers were finer and had discontinuous granulometry, particularly the lime filler (Figure 3a). The material with a more continuous granulometry is filler #1 (Figure 3c). Figure 3 shows industrial-product fillers (lime and calcium carbonate) that have a rounded shape (Figure 3a-b). Figure 3c-d shows that filler #1 and #2 are cubic in shape, and filler #3 tends to resemble a needle or sheet form (Figure 3e).

The chemical compositions of the industrial-origin fillers mostly have oxygen and calcium elements. In contrast, rock crushing fillers have more chemical elements; specifically, filler #3 presents a high combination of aluminum and silicon that are characteristic of clay materials (Figure 3).

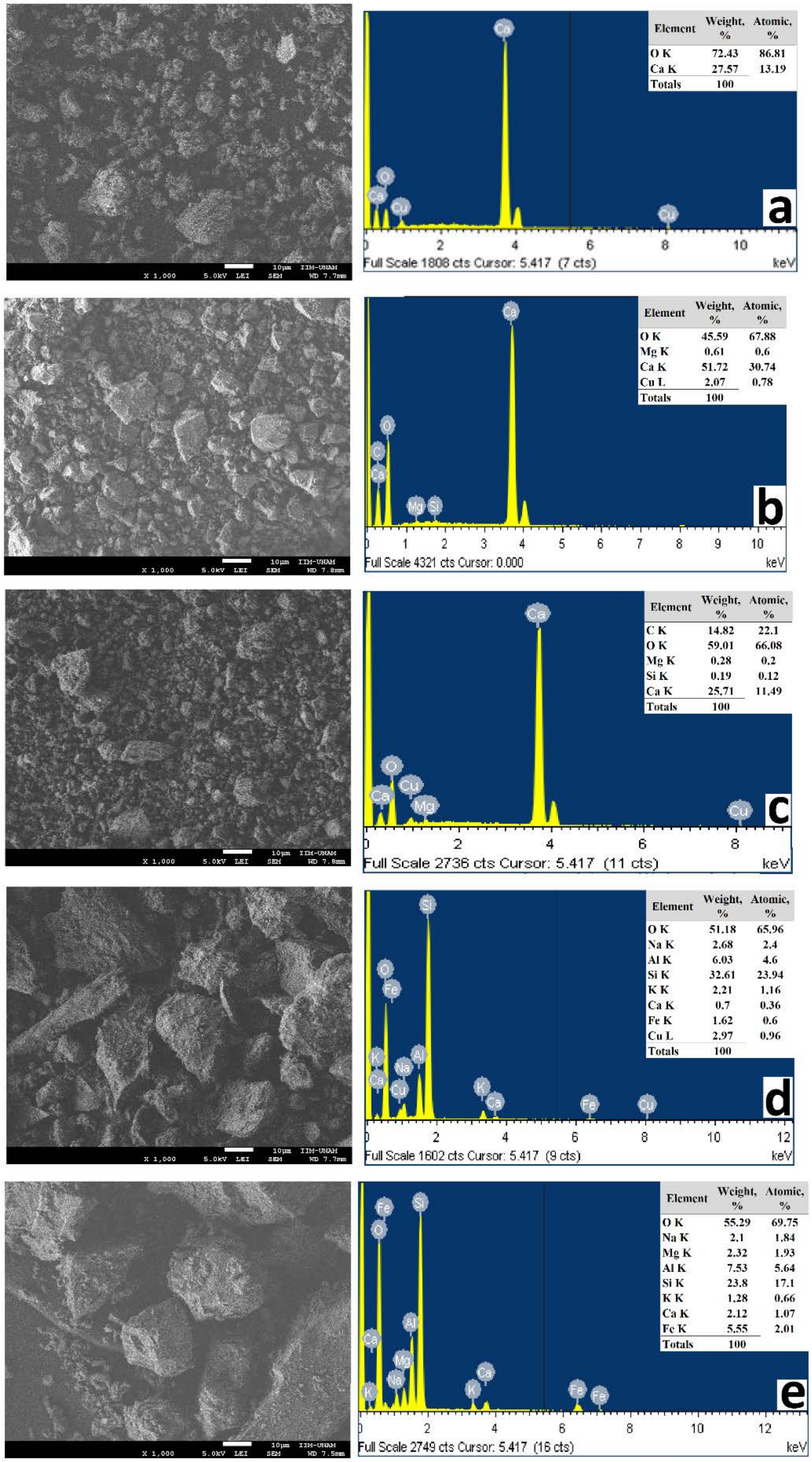

Figure 3. Scanning electron microscopy images of a) Cal, b) $CaCO_3$, c) filler #1, d) filler #2 and e) filler #3.

SCB Tests

The specimens are made with the same proportions of asphalt content, granulometry, and type of stone aggregate and have different types of filler as described in Figure 3. The SCB tests were performed in specimens of 4 in diameter at 1 cm groove depth, a loading speed of 1 mm/min and at four different temperatures. The SCB tests taken in the present study correspond to the average of four specimens.

Temperature of −10°C

The SCB test at −10°C will allow determination of which SMA mixture requires greater fracture energy, since deterioration due to thermal cracking can occur at these temperature conditions.

The results of the SCB test at −10°C are shown in Table 4 and Figure 4. It is evident that the SMA mixture with filler # 1 has the highest values, with 1,215 and 1,540 J/m^2 in test temperature-deformation energy (U) and test temperature-fracture energy (GD), respectively, which implies that this mixture presents the best behavior. The lowest records in the test were presented in the SMA mixture with filler #2; consequently, this mixture is more vulnerable to being fragile (Table 4).

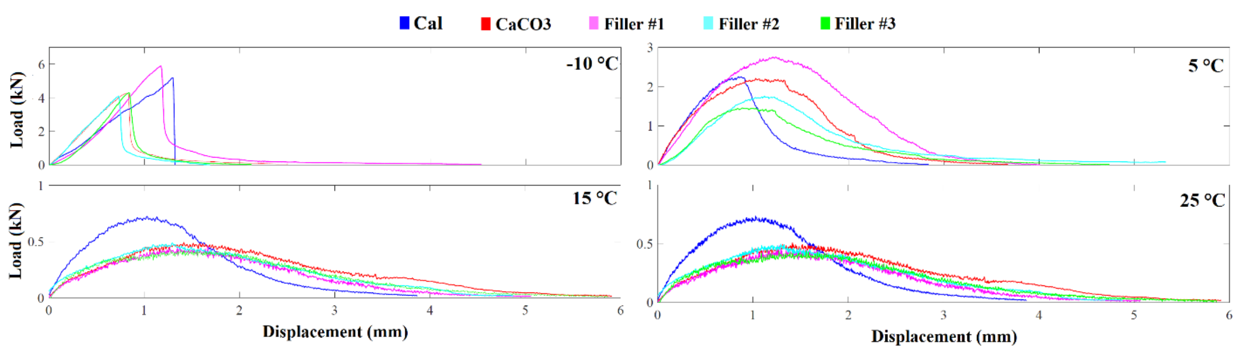

Figure 4. SCB load-displacement graph of all fillers to −10°C, 5°C, 15°C and 25°C.

Temperature of 5°C

In the SCB test at 5°C, a transition point can occur between the fragility and elastic behavior of the mixture. The best behavior of the mixture was presented with filler #1, registering the highest values of U, GD and IT, corresponding to 1,099, 2,379 and 979 J/m^2, respectively. The lowest results were recorded in the SMA mixture with lime filler. Therefore, filler # 1 is the one with the greatest flexibility and tenacity at 5°C, which means that thermal fissures and transition to fatigue cracking are less likely to occur (Table 4).

Temperature of 15°C

At 15°C, the SMA mixture tends to exhibit an elastic behavior due to the rheological properties of the asphalt. In this test, the best behavior is recorded with the mixture containing filler #1, with values of 241, 494 and 182 J/m^2 for U, GD and IT, respectively (Figure 4 and Table 4). Therefore, filler #1 has greater resistance to fatigue cracking that occurs under these temperature conditions. The minimal recorded values correspond to calcium carbonate and lime.

Temperature of 25°C

The SCB test at 25°C presents a transition point between the elastic behavior and deterioration with permanent deformation (rutting); this occurs if soft asphalts, fine granulometries or high asphalt content is used. In Figure 4 and Table 4, the results of the SCB test are presented. The highest records are recorded in the mixtures with filler #1 (values of 309, 762, 484 J/m^2 for U, GD and IT, respectively) and calcium carbonate (values of 367, 832, and 305 J/m^2 for U, GD and IT, respectively). The lowest results are for filler #2, filler #3 and lime. Therefore, filler #1 and calcium carbonate have the highest flexibility and tenacity at 25°C, which makes fatigue cracking less likely to occur in these fillers.

Table 4. Results of the fillers analyzed in the SCB test at −10°C, 5°C, 15°C and 25°C.

Filler type	−10°C							5°C						
	F_{max}	ΔF_{max}	ΔR	IRT	U	GD	IT	F_{max}	ΔF_{max}	ΔR	IRT	U	GD	IT
Cal	4.72	0.95	0.95	5.59	1146	1176	n.a.	2.30	0.83	2.17	3.93	588	1109	190
$CaCO_3$	4.27	0.69	1.35	7.13	753	914	n.a.	2.28	1.14	2.89	3.11	860	1699	487
Filler #1	4.91	0.99	2.46	5.70	1215	1540	n.a.	2.85	1.28	3.46	2.95	1099	2379	979
Filler #2	4.44	0.74	0.94	6.13	738	855	n.a.	2.39	1.06	2.66	3.20	782	1559	344
Filler #3	4.63	0.85	0.78	5.97	847	949	n.a.	1.83	1.04	2.61	3.06	605	1249	403

Filler type	15°C							25°C						
	F_{max}	ΔF_{max}	ΔR	IRT	U	GD	IT	F_{max}	ΔF_{max}	ΔR	IRT	U	GD	IT
Cal	0.74	0.58	1.18	2.33	146	286	56	0.76	0.87	2.40	1.43	204	526	159
$CaCO_3$	0.31	0.78	1.49	1.55	81	170	58	0.62	1.80	4.37	0.48	367	832	305
Filler #1	0.66	1.05	2.37	1.65	241	494	182	0.60	1.63	3.91	0.64	309	762	484
Filler #2	0.62	0.71	1.76	2.16	153	363	130	0.60	1.25	3.17	0.74	258	650	287
Filler #3	0.58	0.85	1.69	1.63	170	328	87	0.60	1.67	3.81	0.60	285	668	323

Note: F_{max} = maximum load, ΔF_{max}= maximum load displacement, ΔR= break displacement, ITR = stiffness index to F_{max}, IT= tenacity index, GD= fracture energy y U= deformation energy.

In Figures 5 and 6, it can be seen how filler #1 has greater deformation and fracture energy at all testing temperatures, which shows that the SMA mixtures made with this filler are more resistant to a cracking process. It is also observed that lime and filler #3 show a tendency to present lower deformation and fracture energy at the different temperatures of the SCB test. On the other hand, the calcium carbonate filler and filler #2 present greater variability in deformation and fracture energy; which confirms their instability in the thermal cracking and fatigue cracking processes.

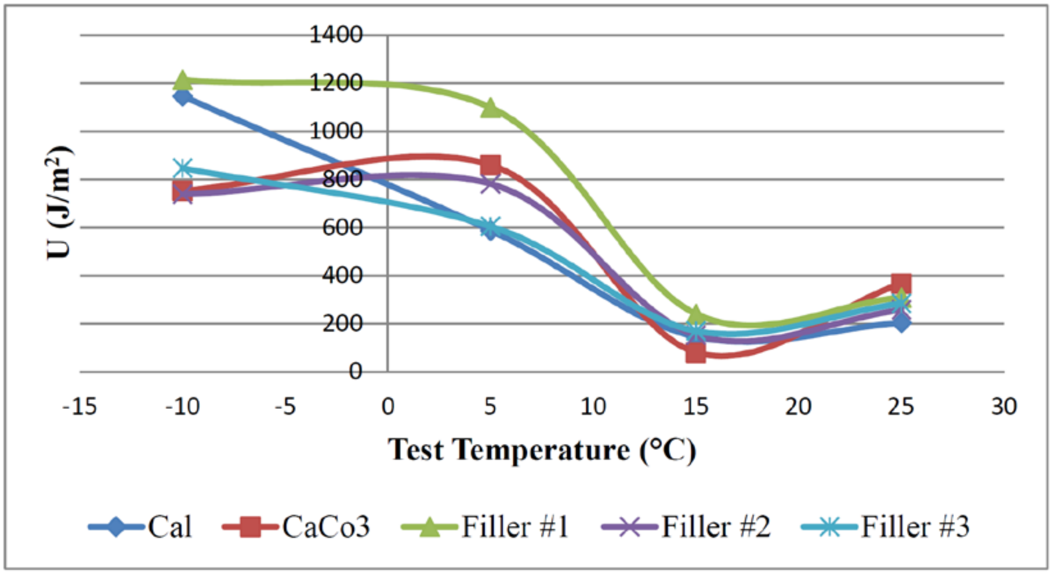

Figure 5. Test temperature-deformation energy (U)

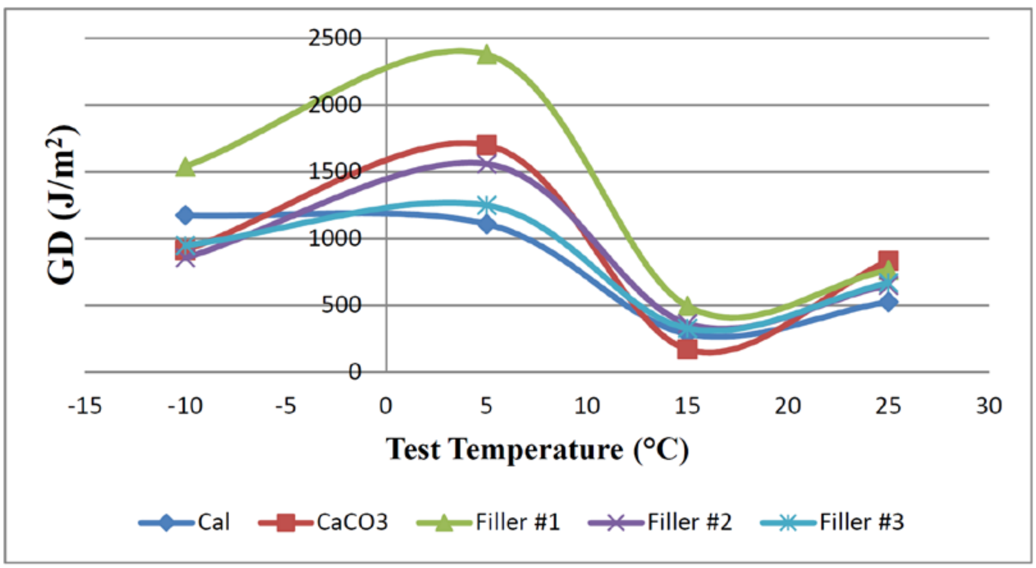

Figure 6. Test temperature-fracture energy (GD)

Consequently, it was determined that the filler from rock crushing filler #1 has the best behavior in the energy fracture parameters measured through the SCB test, guaranteeing greater resistance to possible thermal or fatigue cracking than the other fillers. In addition, it was demonstrated that the industrial products (particularly lime) are more susceptible to cracking.

CONCLUSIONS

From the results of the SCB test, it was determined that the fillers that present the best performance are those with the lowest proportion of aluminum and silicon and thus require more fracture energy. In the present study, filler #1 performed the best, while the filler with lime showed lower performance with respect to fracture energy.

It is important to note that the use of industrial materials such as CaCO3 and lime do not guarantee good performance of the SMA mixtures in cracking processes, as in the case of the fillers evaluated in this study through the SCB test.

SMA mixtures are often placed in thin layers (<4 cm), which are very susceptible to wearing out and cracking. Moreover, these layers are placed on underlaying surfaces with possible existing crack propagation. Thus, the proper use of materials is important to ensure the proper operation and durability of these layers. Properly considering the filler, as an important material in an SMA mixture, can help slow the process of this type of deterioration relative to the norm. Therefore, the SCB test proved to be a good alternative due to its ease of implementation and congruent results. Furthermore, the test can be carried out for both laboratory-made and field-extracted specimens. Consequently, it can serve as either a verification or quality control test, which can measure the fracture energy property and try to predict possible cracking.

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Forensic Investigation on Premature Rutting in Pavements and Remedial Treatment With Stone Matrix Asphalt

Lt.Col. Girish Kumar, VSM
M.Tech (Transportation Engineering)
Project Manager

Dr. A. Veeraragavan, Ph.D.
Professor
Transportation Engineering Division
Department of Civil Engineering
Indian Institute of Technology Madras
Chennai, India

ABSTRACT

Several thousand kilometres of highways are being constructed and large numbers of airports are being revived each year with the growth in transport sector in India. To achieve low vehicle operating cost it is essential to construct roadways runways and taxiways that enable safe and smooth ride and the constructed pavement should last long. Good construction material, proper mix design and good construction practices are factors that determine the quality and performance of the pavement. Numerous pavements are suffering premature failure, primarily in terms of rutting, cracking and ravelling. The causes for such premature failure must be identified at the earliest and rectified to reduce operating cost and carry a lesson for future construction.

National Highway-7 was widened from two-lane to four-lane highway in 2003 and further strengthened in 2008, however, a stretch of 7 kilometres suffered premature rutting by 2009. Forensic investigation has been carried out to identify the cause of premature rutting in this stretch. The rut profile on the highway was studied using Network Survey vehicle (Hawkeye 2000).

Core samples of bituminous mix were extracted from site and analysed for density, gradation, bitumen content and binder properties to check whether they conform to the standards. Gap graded mix such as Stone Matrix Asphalt was prepared and tested for rut resistance. The mixes for wearing and binder course were prepared using modified binder and studied for their performance under Cooper Rut Wheel Tracker for rut resistance. The mixes were also tested under Simple Performance Test Machine for dynamic modulus and flow number. The dynamic modulus values have been incorporated in Kenlayer program to determine the damage sustained. It was found that remedial treatment by constructing a Stone Matrix Asphalt overlay is the best alternative for the long lasting highway pavement.

The use of better materials provides improved performance of expensive facilities like highways and runways. SMA due to the stone-stone contact, though marginally expensive, can be a formidable solution for the desired lowest life cycle cost. Use of modified binders for hotter or extreme climatic zones will result in better performance of pavements. Runway pavements may not be subjected to high traffic volume, however, the quality of such pavements have to be of high standards to meet unforeseen circumstances where such failure is unacceptable and Stone Matrix Asphalt will be a viable alternative for improved performance and long lasting highway and airfield pavements.

INTRODUCTION

A large number of highways and airfields are being constructed or upgraded each year for infrastructure development to meet rapidly increasing demand of transport sector in India. With the government spending 10% of its annual GDP on infrastructure development, of which nearly 30% is towards highways with additional 4-5% towards airports.

Rutting may occur due to failure of one or more layer as a result of densification of bituminous layers or deformation of lower layers. Shoving and rutting may occur simultaneously. Overlay on rutted pavements despite adequate structural strength of lower layers does not solve the problem.

Gap graded mixes such as Stone Matrix Asphalt (SMA) are known to be rut resistant due to stone-stone contact for better load dispersion and reduce the effect of mastic available.

Rutted pavements prevent cross-drainage, cause accidents, and increase vehicle operating cost. Factors contributing towards premature rutting must be investigated to understand the causes of failure better. Road construction records are poorly maintained in the country, thereby demanding forensic investigation to identify material used and their performance Identifying cause of failure will pave towards economical solutions and low vehicle operating cost.

Scope

Field and laboratory studies were done on a section of Highway in the state of Tamil Nadu to identify cause of extensive rutting within a year of resurfacing in 2008. Causes of rutting were investigated and Stone Matrix Asphalt mix was designed to evaluate its performance in comparison to existing materials using a Rut Wheel Tester and the same were confirmed with Simple Performance Test (SPT).

Methodology

Non-availability of construction details is a major constraint of this study. Core samples extracted from site were studied for density. Indirect tensile strength was tested. Gradation of the aggregate was checked for identifying type of mix and conformity to standards. Aggregate quality in southern part of the country is generally good and therefore, tests to confirm quality were not conducted in this study. Binder was tested for its properties and performance under Dynamic Shear Rheometer (DSR).

Specimen of similar gradation, binder content and type were fabricated at laying density for testing under rut wheel tracker for comparison to rut-resistant mix (SMA). Since virgin binder used was not available, nor was the type of binder known, hence, PMB was used to fabricate specimen with similar properties for comparison with SMA.

SMA was designed based on Marshall Design and samples at laying density for testing under rut wheel tracker. Dynamic Modulus and strain under cyclic loading were determined using SPT to evolve the most suitable treatment.

COLLECTION OF DATA AND FIELD SAMPLES

Rut data was collected using Network Survey Vehicle (NSV) in hilly terrain of Thopur Ghat near Dharmapuri in Tamil Nadu from Chainage 156.00 to 163.40 on NH-7. Rutted and unaffected stretches were identified for extracting cores. The study stretch from chainage 156.00 to 163.40 on National Highway-7 (NH-7) has pavement composition of 50 mm Bituminous Concrete (BC) over 160 mm Dense Bituminous Macadam (DBM) constructed in two lifts. Base layer composes of 300 mm Wet Mix Macadam (WMM) constructed in two lifts over 200 mm granular sub-base. Distress in the form of rutting and shoving developed on the stretch mainly at the up-gradient and near humps on the down gradient.

Collection of Rut Data

Pavement condition survey was done using Network Survey Vehicle (NSV) to collect rut data. Indian Road Survey & Management Pvt. Ltd. (IRSM) undertook road condition survey using "Hawkeye 2000 Network Survey Vehicle" for NH-7. The NSV consists of digital laser profiler (eleven lasers) capable of measuring rut depth for width of 3 m at every 50 mm of longitudinal travel, digital imaging system and a Gipsi-Trac unit. Rut data was processed at 5 m interval due to limiting criteria of rut depth more than 10 mm and 5 m long in this case. High rut depth up to 80.83 mm at chainage 159.455 on outer lane in Thopur-Krishnagiri direction was identified. The rut data acquired is depicted graphically in Figure 1.

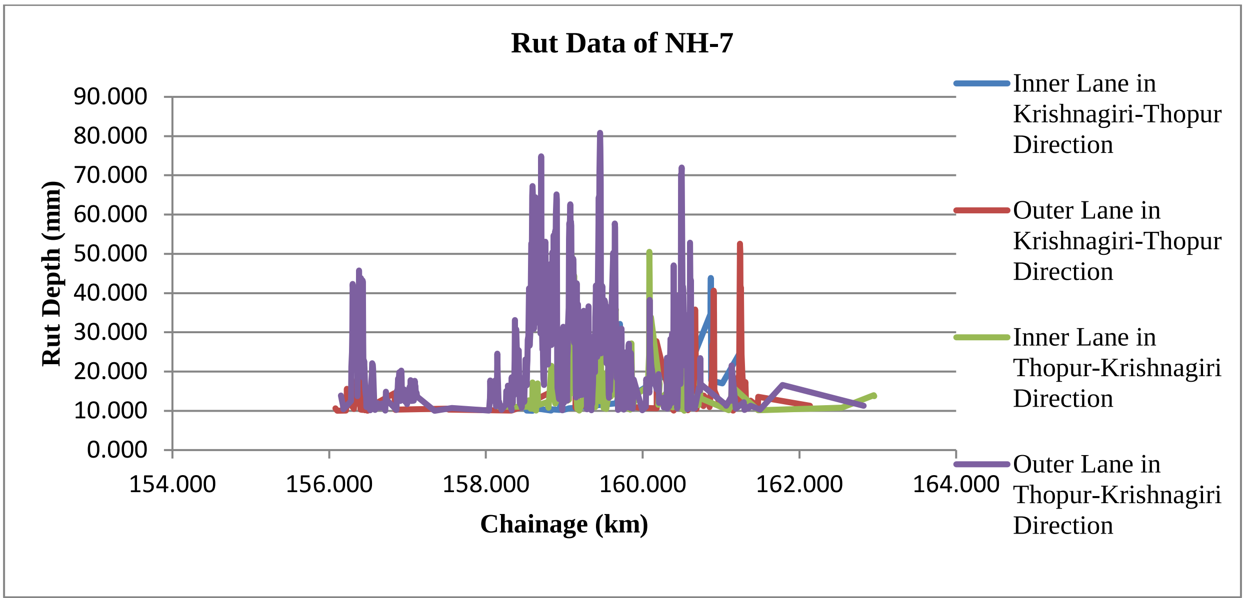

Figure 1. Rut Depth on NH-7

Core Samples Collected from Site

Samples of 100 mm diameter were extracted using a core cutting machine from bituminous layers on the pavement in locations that had suffered rutting as well as one location which was unaffected. The samples were from Bituminous Concrete (BC) as well as Dense Bituminous Macadam (DBM) layers. DBM samples were taken at three different depths due to its construction having been in different lifts, top 75 mm being laid in 2008 after milling. Samples were also taken from rutted shoulder. The location of the various core samples with the condition at that chainage is tabulated in Table 1. Figure 2 shows the activity of extracting core from rutted section on the shoulder.

Table 1. List of Core Samples Extracted from Site

Sample No.	Chainage	Location	Layer	Condition	Material Type
1	159.85	2.5 m from edge	Single	Rutted	BC
2	159.85	2.5 m from edge	Top	Rutted	DBM
3	159.85	2.5 m from edge	Middle	Rutted	DBM
4	159.85	2.5 m from edge	Bottom	Rutted	DBM
5	159.85	Shoulder	Single	Rutted	BC
6	159.85	Shoulder	Top	Rutted	DBM
7	159.85	Shoulder	Middle	Rutted	DBM
8	159.85	Shoulder	Bottom	Rutted	DBM
9	157.4	2.5 m from edge	Single	Good	BC
10	157.4	2.5 m from edge	Top	Good	DBM
11	157.4	2.5 m from edge	Middle	Good	DBM
12	157.4	2.5 m from edge	Bottom	Good	DBM

Figure 2. Core Extraction from Rutted Shoulder

FORENSIC INVESTIGATIONS OF FIELD SAMPLES

The dimensions and weight of the samples collected from site were measured in accordance with ASTM standard D3549-03.

Bulk Specific Gravity (G_{mb})

Bulk Specific Gravity determined in accordance with D6752-03 is lower than those determined by test method specified in ASTM standard D2726. However, test method as per D2726 is not applicable to porous mix like SMA. Hence, method under D6752 was adopted throughout the entire project to avoid errors due to different methods used in determining such critical data.

Indirect Tensile Strength (IDT)

The specimens were tested according to procedure specified in ASTM standard D6931-07 wherein vertical compressive ramp load at a deformation rate of 50 mm per minute was applied using a loading strip of 12.7 mm width. The maximum load subjected on to the specimen before it failed was recorded. IDT was calculated and averaged to 1333 KPa for BC.

IDT values for unmodified mixtures at 250C should generally be in the range of 680 to 920 KPa, increasing with binder content for specimens with air void ratio at 4%. BC layer as investigated subsequently comprises of modified binder. Hence, its IDT would be higher upto 1000 KPa approximately. Both low air voids and high bitumen content contribute towards high IDT strength. Thus, it can be inferred from this test that either binder content is high or air voids is low or both in all the samples that experienced rutting.

Theoretical Maximum Specific Gravity (G_{mm})

G_{mm} was determined in accordance with test procedure in ASTM standard D2041-03a using Corelok apparatus. G_{mm} was calculated and found ranging between 2.42 and 2.47.

Air Void Ratio (V_a)

Air void ratio was calculated to identify compaction level using individual values of G_{mb} and G_{mm}. V_a in case of BC was determined to be 2.5% approximately for the samples from rutted sections while for the unaffected stretch it was found to be approximately 4.75%. The rutted section of DBM was found to have V_a ranging from 4.7% to 6.5 %.

Bitumen Content

Bitumen content has been determined using a combination of methods specified in ASTM standards D2172-05 and D6847-02. Bitumen was extracted from the mix and weight of aggregate measured to arrive at the weight of bitumen. The solution comprising of solvent and bitumen was placed in rotary evaporator to distil the solvent, and the bitumen be left as residue. The bitumen content as calculated by measuring the residue in rotary evaporator flask is more accurate. The binder content of the BC samples from rutted location are 6.19% and 6.07% whereas that at unaffected section is 5.23%, clearly indicating high binder content in distressed locations. High binder content though having lower air voids but containing higher viscous component will undergo larger deformation with similar loading.

Gradation

Aggregate collected after bitumen extraction was analysed for its gradation taking the fine material filtered from the solution of bitumen and solvent after extraction into account. The gradation was found conforming to coarser gradation of BC and finer gradation of DBM on analysing graphically. Graph plotting the gradation of BC is shown in Figures 3. Conforming to Ministry of Road Transport and Highways (MORTH) specifications tabulated in Table 2.

Table 2. Gradation of BC and DBM as per MORTH Specifications

Sieve Size (mm)	BC 1 (Coarse)		BC 2 (Fine)		DBM 1 (Coarse)		DBM 2 (Fine)	
37.5	100	100	100	100	100	95	100	100
26.5	100	100	100	100	93	63	100	90
19	100	79	100	100	84	59	95	71
13.2	79	59	100	79	75	55	80	56
9.5	72	52	88	70	—	—	—	—
4.75	55	35	71	53	54	38	54	38
2.36	44	28	58	42	42	28	42	28
1.18	34	20	58	42	—	—	—	—
0.6	27	15	38	26	—	—	—	—
0.3	20	10	28	18	21	7	21	7
0.15	13	5	20	12	—	—	—	—
0.075	8	2	10	4	8	2	8	2

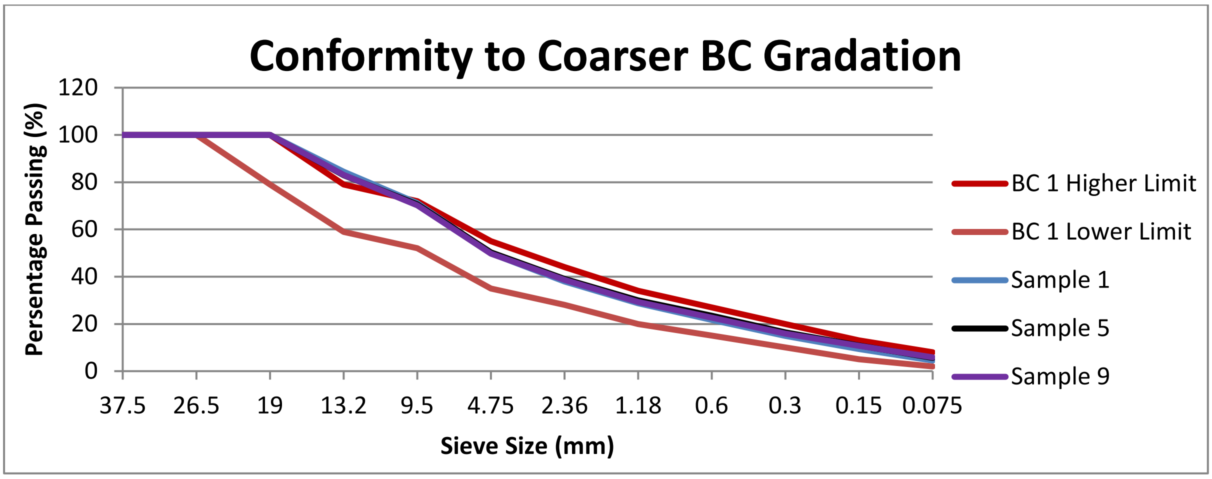

Figure 3. Conformity of Samples to Coarser Gradation of BC

Binder Properties

Virgin sample of binder used was not available; hence, binder extracted from the core samples was tested for binder properties. Since the binder was used in the mix, laid and compacted, it had already undergone short-term aging. In addition the sample was extracted one year after pavement was in service; hence, it was considered more than short term aged. The penetration, viscosity, Complex Shear Modulus (G*) would definitely have undergone change. The same was confirmed when the G* values of the binder were compared to data pertaining to virgin and short term aged bitumen samples. The complex modulus values of the bitumen extracted from core samples was much higher than standard VG-30 and modified binders. The G*/sin(δ) value was higher than the specification for short term aged binder and G*sin(δ) lesser than long term aged binder as per specifications in ASTM standard D6373-07 when tested under Dynamic Shear Rheometer (DSR) in accordance with ASTM D7175-08.

Phase angle (δ) being the lag between shear and strain of the mix when subjected to a load at a given temperature will generally be larger at higher temperature and smaller at lower temperature. At sub-zero temperatures phase lag may cease to exist, causing brittle nature of the binder. As seen in Figure 4, phase angle for binder used is limited at lower frequency, thereby, indicating use of modified binder in BC. Modified binder does not seem to have been used for DBM mix since the phase angle continues to rise at lower frequency, while it continues to decrease at high frequency.

Loss modulus values of binder used in BC reach an asymptote at higher temperatures beyond which it does not increase and also an asymptote at lower temperature below which it does not decrease on plotting, confirming it to be modified.

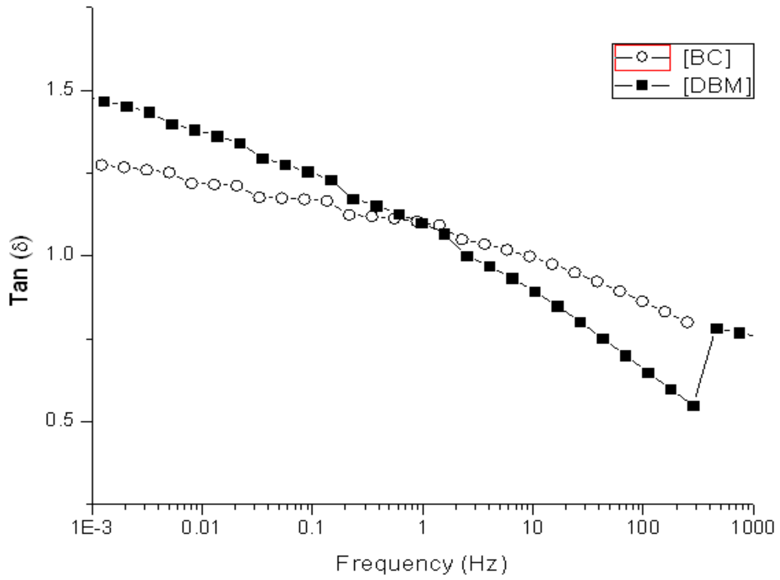

Figure 4. Phase Angle Determined by DSR Plotted Against a Range of Frequency

CONCLUSION FROM FORENSIC INVESTIGATIONS

a) IDT results indicate high strength which may be rendered due to high binder content or low air voids or both.
b) Air voids are low in the case of BC samples possibly due to high binder content or high percentage of fines or excessive compaction.
c) Binder content for BC samples is high and could be a contributory factor towards rutting.
d) Gradation is within specifications especially the particle size passing 75 µm sieve, indicating that gradation has not contributed towards rutting.
e) Binder properties show that modified binder was used for BC, while unmodified binder for DBM. The value of unmodified binder conforms to performance grading specifications. Therefore, binder quality is satisfactory.
f) High binder content and over-compaction seem to be the cause of rutting in the pavement under heavy loading conditions.

DESIGN OF ALTERNATE RUT-RESISTANT MIX

Gap-graded mix such as SMA is considered to be rut resistant because of stone-stone contact formed. SMA consists of 75 percent coarse aggregate which assists in forming skeletal structure. Particle size passing 2.36 mm sieve to 600 µm is least causing the gap in gradation. Percentage of fine aggregate passing 75 µm sieve is larger and forms mastic in the mix as seen in Figure 5. When such mixes are subjected to load, the larger particles transfer the load to each other through the medium of mastic. There is very high inter-particle friction due to angularity of the aggregate, which prevents significant movement within the mix, causing it to be rut resistant to the extent that, secondary compaction due to traffic is minimal after construction of the layer. SMA has two gradations, with 19 mm and 13.2 mm Nominal Maximum Aggregate Size (NMAS). Generally, the finer is used for wearing course, while the coarser as binder course.

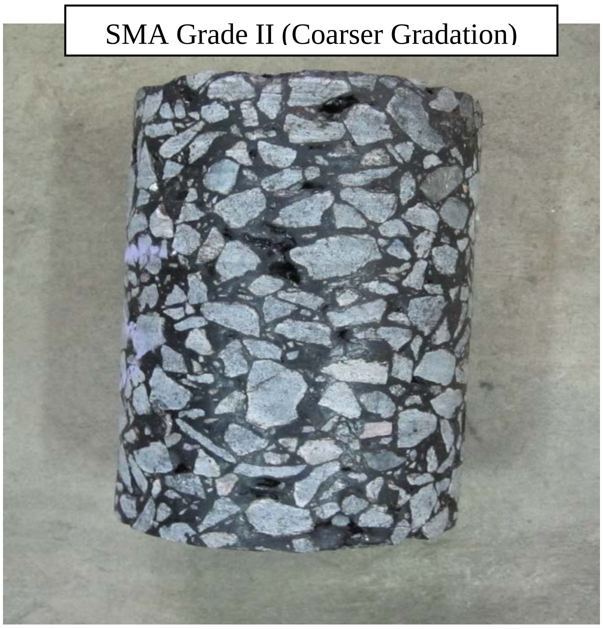

Figure 5. Stone-Stone Contact in SMA Mix

BC and DBM were designed using Marshall Design method. BC I (coarse grade) and DBM II (fine grade) as found at site were designed using Polymer Modified Bitumen Grade 40 (PMB-40 Cariphalte) binder for comparison to SMA. The target air voids was set at 4%. The binder content trials were carried out at 4.5%, 5% and 5.5% for DBM and 5%, 5.5% and 6% for BC. Binder content was determined as 5% for DBM and 5.5% for BC. The G_{mm} values for BC and DBM were determined to be 2.537 and 2.535 respectively.

SMA Specifications

Specifications for Stone Matrix Asphalt (SMA) as per Indian Roads Congress Special Publication: 79 (IRC SP: 79 – 2008) is listed in Tables 3 and 4. The AASHTO specifications vary regarding the bitumen content at 6% minimum as specified in AASHTO MP 08 and PP 41 (2004).

Table 3. SMA Specifications as per IRC SP:79 – 2008

Parameter	Specification
Air Void Content V_a	4.0 %
Bitumen Content Pb	5.8 % min
Fiber Content (by weight of total mix)	0.3 %
Voids in Mineral Aggregate VMA	17 % min
Draindown Characteristics	0.3 % max
Tensile strength ratio	85 % min

Table 4. Gradation of Fine and Coarse Grades of SMA

Sieve Size	Percentage by Weight of aggregate passing			
	SMA Grade I		SMA Grade II	
mm	Min	Max	Min	Max
37.5	100	100	100	100
26.5	100	100	100	100
19	100	100	90	100
13.2	90	100	45	70
9.5	50	75	25	60
4.75	20	28	20	28
2.36	16	24	16	24
1.18	13	21	13	21
0.6	12	18	12	18
0.3	10	20	10	20
0.075	8	12	8	12

Material Properties

Modified binder PMB-40 (Cariphalte) has been used for mix design. Tests on aggregate, filler as well as binder were conducted to determine their properties. Lime was used as filler in the entire project. Specific Gravity of various materials determined prior to use in the study are tabulated in Table 5.

Table 5. Specific Gravity of Materials

Material	Specific Gravity
Binder PMB 40 (Cariphalte)	1.0226
Coarse Aggregate (Bin A)	2.7392
Coarse Aggregate (Bin B)	2.7569
Coarse Aggregate (Bin C)	2.7200
Fine Aggregate (Bin D)	2.7314
Filler (Lime)	2.1918

Three stockpiles of aggregate were available; Bin B and C contained coarse aggregate while Bin D contained fine aggregate. Certain aggregate sizes that were not available were used from a previously sieved stockpile (Bin A). Sieve analysis was carried out for the three stockpiles and the results are tabulated in Table 6.

Table 6. Sieve Analysis of Aggregate Stockpile

Sieve Size	Percentage Passing as Per Sieve Analysis		
mm	B	C	D
37.5	100	100.00	100.00
26.5	100.00	100.00	100.00
19	59.50	99.66	100.00
13.2	14.52	97.27	100.00
9.5	6.60	62.58	95.51
4.75	2.91	1.66	72.92
2.36	2.65	1.22	59.83
1.18	2.24	1.08	46.73
0.6	1.76	0.97	32.93
0.3	1.21	0.88	23.86
0.075	0.64	0.63	7.76

Design Trials without Fiber

Initial trial was carried out without using fiber in SMA Mix. However, drain-down was to the extent of 7%. SMA II (Coarse) specimens were fabricated using three types of aggregates with addition of lime as filler and specific size of aggregate passing 19 mm and retained on 13.2 mm. Six samples were prepared at 5.5%, 6.0%, and 6.5% binder content by Marshall method. The mix design was done using Marshall Method of compaction with 50 blows on either side as per IRC. V_a achieved was in the range of 7.09 to 11.94%.

Compaction using Superpave Gyratory Compactor

An attempt was made at using Superpave Gyratory Compactor for Mix Design as per AASHTO standard with 6% binder content. However, most literature brings out the attempts to formulate the level of compaction desired using gyratory compactor. Due to very high air voids attained in the range of 11% and above on applying 100 gyrations according to AASHTO specifications, the design method was not continued further during the study.

SMA II (Coarse Grade) Design

SMA II (coarser grade) was planned as binder course. Gradation adopted for SMA II design is as given in Table 7, with Bin A having aggregate passing 19 mm and retained on 13.2 mm sieve. Bulk Specific Gravity G_{sb} was determined as 2.681. Average G_{mm} value determined was 2.495. Drain-down characteristics obtained were 0.17%. VMA went down to 16.3% as compared to minimum requirement of 17 %. V_a below 4.33% could not be obtained as compared to 4% desired. Higher percentage of bitumen in the mix did not reduce air voids, however, VMA increased to permissible level. Volumetrics of the Mix designed are given at Table 8. The results of design were accepted for further fabrication of samples for comparative testing.

Table 7. Gradation Adopted for SMA II

Sieve Size	Max	Min	Proportion of Aggregate Stockpile (%)					Total Proportion
mm	(%)	(%)	A	B	C	D	E Filler	
37.5	100	100	0.25	0.15	0.31	0.21	0.08	100.00
26.5	100	100	0.25	0.15	0.31	0.21	0.08	100.00
19	90	100	0.25	0.15	0.31	0.21	0.08	93.82
13.2	45	70	0.25	0.15	0.31	0.21	0.08	61.33
9.5	25	60	0.25	0.15	0.31	0.21	0.08	48.45
4.75	20	28	0.25	0.15	0.31	0.21	0.08	24.26
2.36	16	24	0.25	0.15	0.31	0.21	0.08	21.34
1.18	13	21	0.25	0.15	0.31	0.21	0.08	18.48
0.6	12	18	0.25	0.15	0.31	0.21	0.08	15.48
0.3	10	20	0.25	0.15	0.31	0.21	0.08	13.46
0.075	8	12	0.25	0.15	0.31	0.21	0.08	9.92

Table 8. Calculations of Volumetrics for SMA II Samples

Sample No.	Bitumen Content	G_{mm}	G_{mb}	G_{se}	G_{sb}	Pb_a	Pb_e	VMA	V_a
Initial									
1	6	2.495	2.361	2.751	2.692	0.981	5.153	17.27	5.37
2	6	2.495	2.309	2.748	2.692	0.932	5.125	19.03	7.45
3	6	2.495	2.282	2.754	2.692	1.020	5.175	20.09	8.54
Set 2									
4	6	2.495	2.364	2.752	2.686	0.989	5.157	17.18	4.72
5	6	2.495	2.366	2.752	2.686	0.994	5.160	17.12	4.64
6	6	2.495	2.357	2.753	2.686	1.000	5.164	17.44	5.00
Set 3									
9	6	2.495	2.357	2.750	2.683	0.968	5.145	17.40	5.53
10	6	2.495	2.359	2.751	2.683	0.974	5.149	17.34	5.45
11	6	2.495	2.37	2.752	2.683	0.985	5.155	16.96	5.01
Set 4									
12	6	2.495	2.387	2.750	2.681	0.958	5.140	16.33	4.33
13	6	2.495	2.381	2.757	2.681	1.049	5.192	16.66	4.57
14	6	2.495	2.369	2.751	2.681	0.971	5.147	16.98	5.05
15	6	2.495	2.382	2.750	2.681	0.957	5.139	16.51	4.53

SMA I (Finer Grade) Design

Design procedure for this mix is similar to SMA II. The gradation is finer for use as wearing course as tabulated in Table 9 and bitumen content required is generally higher due to smaller aggregate size. Bin A consisted of previously sieved aggregate passing 13.2 mm and retained on 9.5 mm sieve. G_{sb} was determined as 2.674 for the gradation adopted while G_{mm} was determined as 2.476 for the mix. Drain down was contained to 0.15% using cellulose fiber. Table 10 gives out volumetrics for the mix adopted for the study.

Table 9. Gradation Adopted for SMA I

Sieve Size	Max	Min	Proportion of Aggregate Stockpile (%)					Total Proportion
mm	(%)	(%)	A	B	C	D	E Filler	
37.5	100	100	0.105	0.05	0.565	0.2	0.08	100
26.5	100	100	0.105	0.05	0.565	0.2	0.08	100
19	90	100	0.105	0.05	0.565	0.2	0.08	100.00
13.2	45	70	0.105	0.05	0.565	0.2	0.08	94.18
9.5	25	60	0.105	0.05	0.565	0.2	0.08	62.79
4.75	20	28	0.105	0.05	0.565	0.2	0.08	23.67
2.36	16	24	0.105	0.05	0.565	0.2	0.08	20.79
1.18	13	21	0.105	0.05	0.565	0.2	0.08	18.07
0.6	12	18	0.105	0.05	0.565	0.2	0.08	15.22
0.3	10	20	0.105	0.05	0.565	0.2	0.08	13.33
0.075	8	12	0.105	0.05	0.565	0.2	0.08	9.94

Table 10. Calculations of Volumetrics for SMA I Samples

Sample No.	Bitumen Content	G_{mm}	G_{mb}	G_{se}	G_{sb}	Pb_a	Pb_e	VMA	V_a
Set 1									
1	6	2.476	2.31	2.726	2.674	0.72	5.37	18.85	6.70
2	6	2.476	2.268	2.724	2.674	0.69	5.36	20.29	8.40
3	6	2.476	2.283	2.725	2.674	0.71	5.37	19.78	7.79
Set 2									
4	6.2	2.471	2.31	2.725	2.674	0.72	5.37	18.84	6.52
5	6.2	2.471	2.329	2.726	2.674	0.73	5.38	18.19	5.75
Set 3									
6	6.5	2.459	2.359	2.729	2.674	0.76	5.40	17.18	4.07
7	6.5	2.459	2.341	2.726	2.674	0.72	5.37	17.76	4.80

PERFORMANCE-BASED LABORATORY TESTS ON RUT-RESISTANT BITUMINOUS MIXES

Laboratory tests are economical to study performance of materials; however, they must be used in models based on field data for co-relation. Tests carried out to study performance and distress were indirect tensile strength, dynamic modulus of asphalt mix, creep and recovery tests and subjecting specimen to wheel track load to compare performance for rutting.

Cooper Wheel Tracker

Standard load of 700 N at a frequency of 36.5 cycles per minute up to 10000 cycles is specified as test criteria in Cooper Wheel Tracker manual. Sample of size 300 × 300 mm is catered in the apparatus; sample of 150 mm diameter and 50 mm thick prepared using Gyratory Compactor was placed in a wooden mould that engulfs the 150 mm diameter sample. Rut data for centre stretch of 80 mm was considered since each time it loads the specimen there is an impact, enhancing the rut at the edges. Each sample was prepared and checked for air voids and tested at 35°C and 55°C.

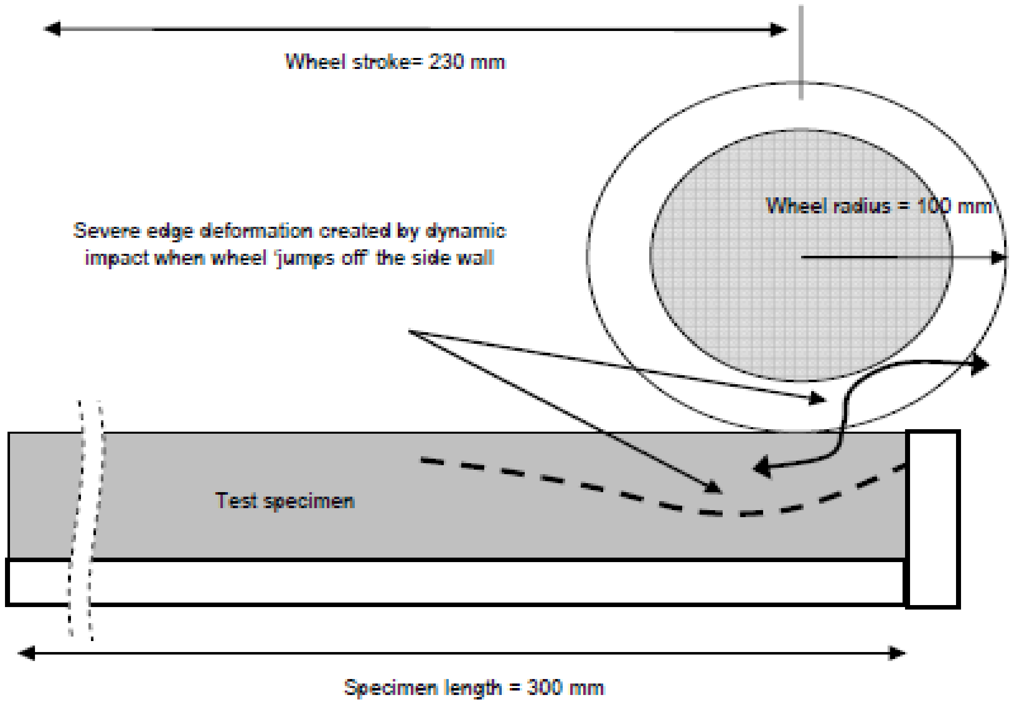

Figure 6. Arrangement of Wheel Load in Cooper Wheel Tracker

The rut test at temperatures of 35°C and 55°C were carried out at comparable densities of specimens. Specimens tested at 55°C have 7.5% and 7.8% V_a for SMA-I and BC respectively. SMA I at 6.3% V_a have better rut resistance to that at 7.5% V_a as well as to BC at 4.5% V_a as seen in Figure 7. The air voids of specimen tested at 35°C vary widely and may not be justified comparable.

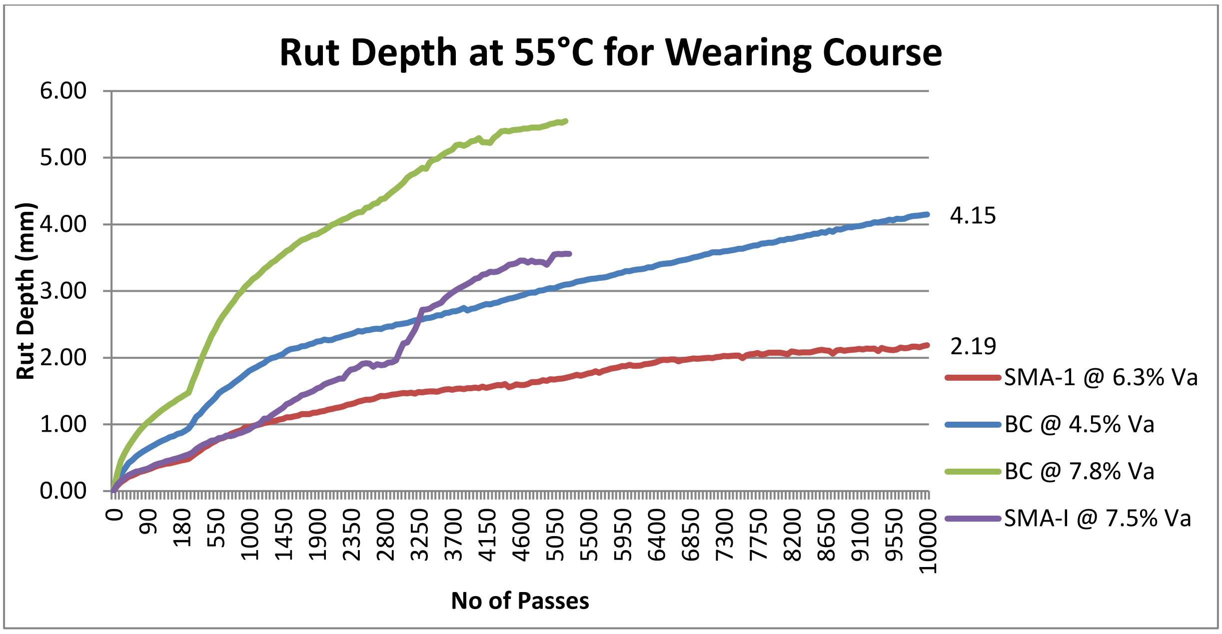

Figure 7. Rut Depth at 55°C for Wearing Course

Rut tests conducted for SMA II and DBM at 55°C show that SMA II at 11.9% V_a and 9.4% V_a rut lesser than DBM at 11.1% V_a and 6.7% V_a respectively. The same is compared in graph plotted in Figure 8.

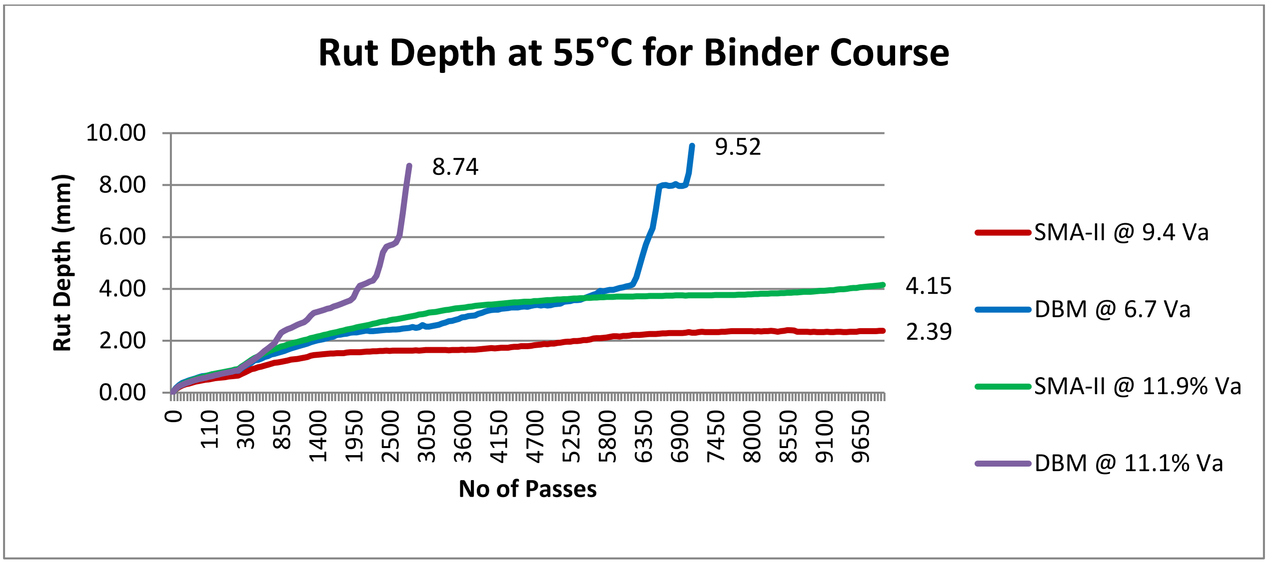

Figure 8. Rut Depth at 55°C for Binder Course

Graph for rut tests on SMA II and DBM at 35°C plotted in Figure 9 shows that SMA II at 11.9% V_a and 9.6 % V_a is more rut resistant to DBM at 8.7% V_a and 7.2 % V_a respectively. SMA II at 9.6% V_a has rutted lesser than DBM at 8.7% V_a.

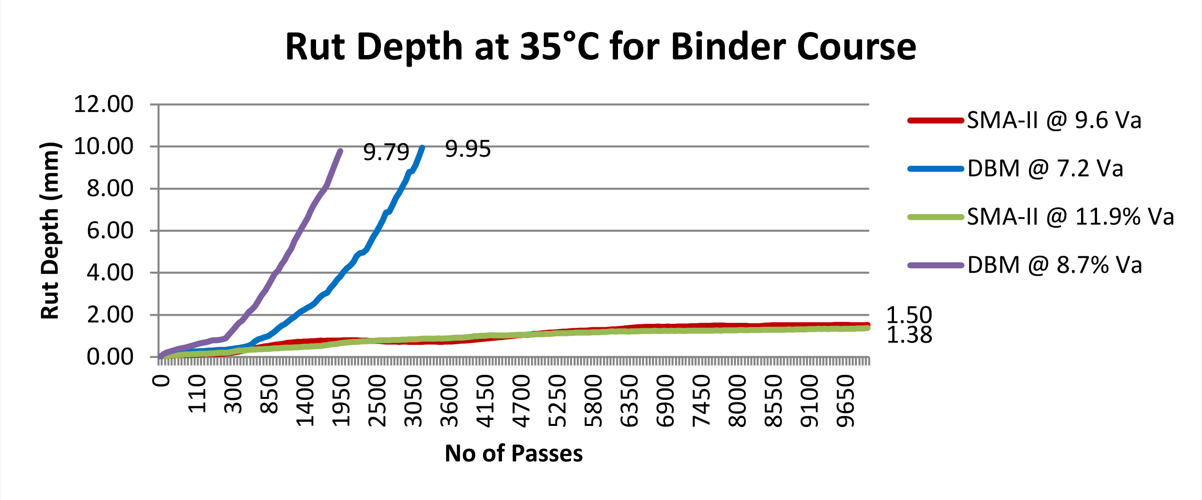

Figure 9. Rut Depth at 35°C for Binder Course

Dynamic Modulus

Dynamic modulus test was conducted complying with AASHTO Provisional Standard TP62, Standard Method of Test for Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures. The dynamic modulus was conducted at frequencies ranging from 0.01 Hz to 25 Hz at 30°C and 55°C to determine the Dynamic modulus and Phase angle at a single effective temperature (T_{eff}) and design loading frequency.

After the test specimen allowed equilibrating to the specified temperatures of 30°C and 55°C, three axial LVDTs were mounted on test samples at an angle of 120°. A standard contact load of 5 KPa is applied on the specimen to keep the deformation in the linear range. Although deformation in bituminous mixes is non-linear after initial strain occurs, the analysis has complications. Haversine loading was applied on the specimen without impact in cyclic manner. The Dynamic load was adjusted to obtain axial strain between 75 and 125 micro-strains. The dynamic modulus and phase angle data for various temperature and frequencies were collected by the data acquisition system.

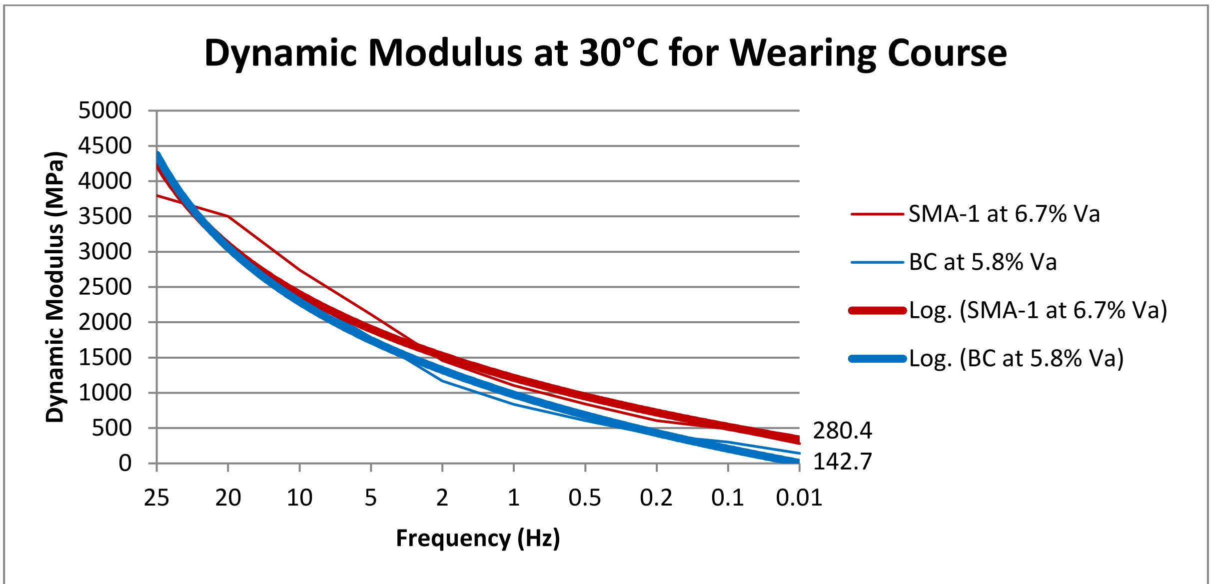

Figure 10. Dynamic Modulus at 30°C for Wearing Course

Dynamic Modulus values acquired from the system were plotted for different temperatures after determining air voids of individual specimens. Dynamic modulus of SMA samples is higher in comparison with BC and DBM samples at low frequencies (i.e. heavier loading conditions). This shows that, SMA is more rut resistant under heavy load conditions. SMA I at 6.7% V_a has higher dynamic modulus compared to BC at 5.8% V_a at 30° Celsius temperature as shown in Figure 10.

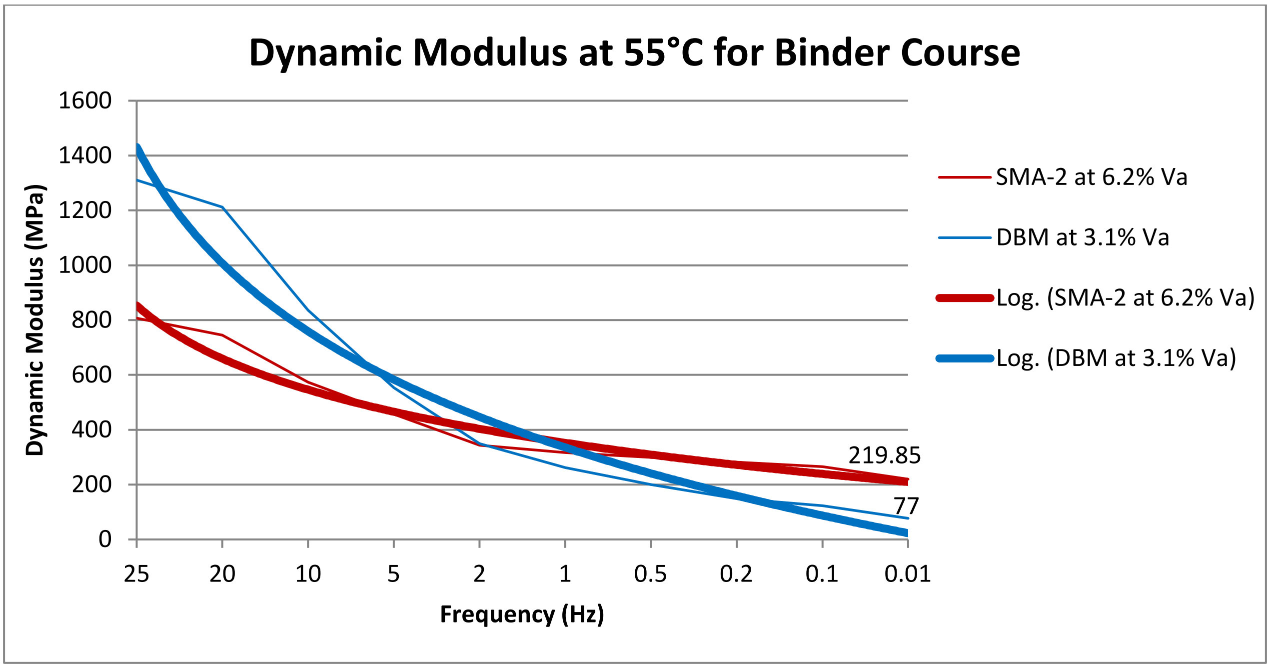

Figure 11. Dynamic Modulus at 55°C for Binder Course

Dynamic modulus measured at 55°C for SMA II at 6.2% V_a and BC at 3.1% V_a is plotted in Figure 11, wherein due to high air voids, SMA II has low dynamic modulus values for higher frequencies. However, for lower frequencies, SMA II has better modulus values than DBM.

Phase Angle (δ)

Phase angle acquired from the system when tested under the same apparatus is plotted for 30°C and 55°C test temperatures. At 30°C, Phase angle of SMA I with 6.7% V_a is substantially lower than BC with 5.8% V_a as plotted in Figure 12.

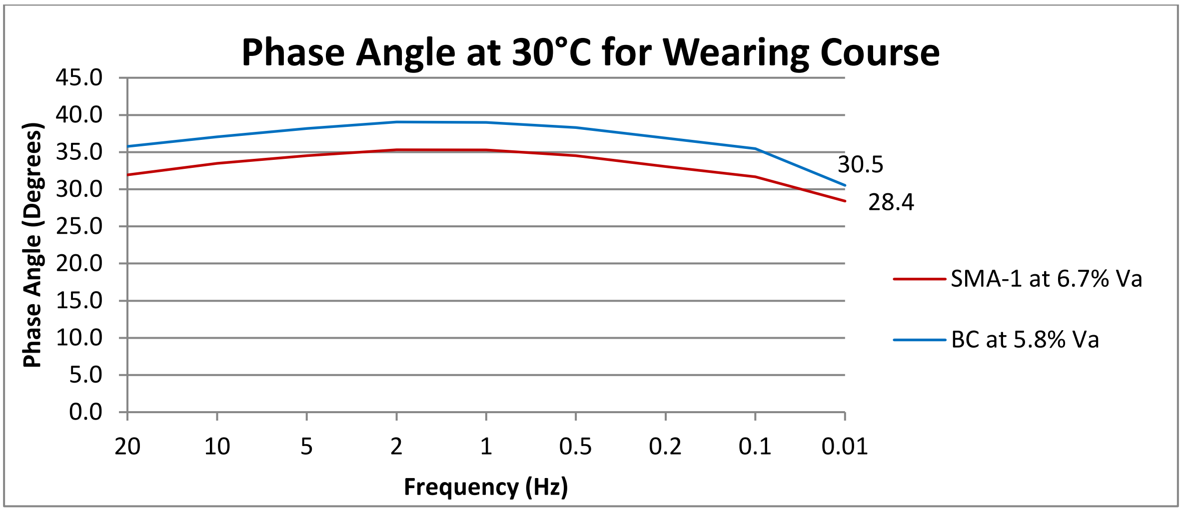

Figure 12. Phase Angle at 30°C for Wearing Course

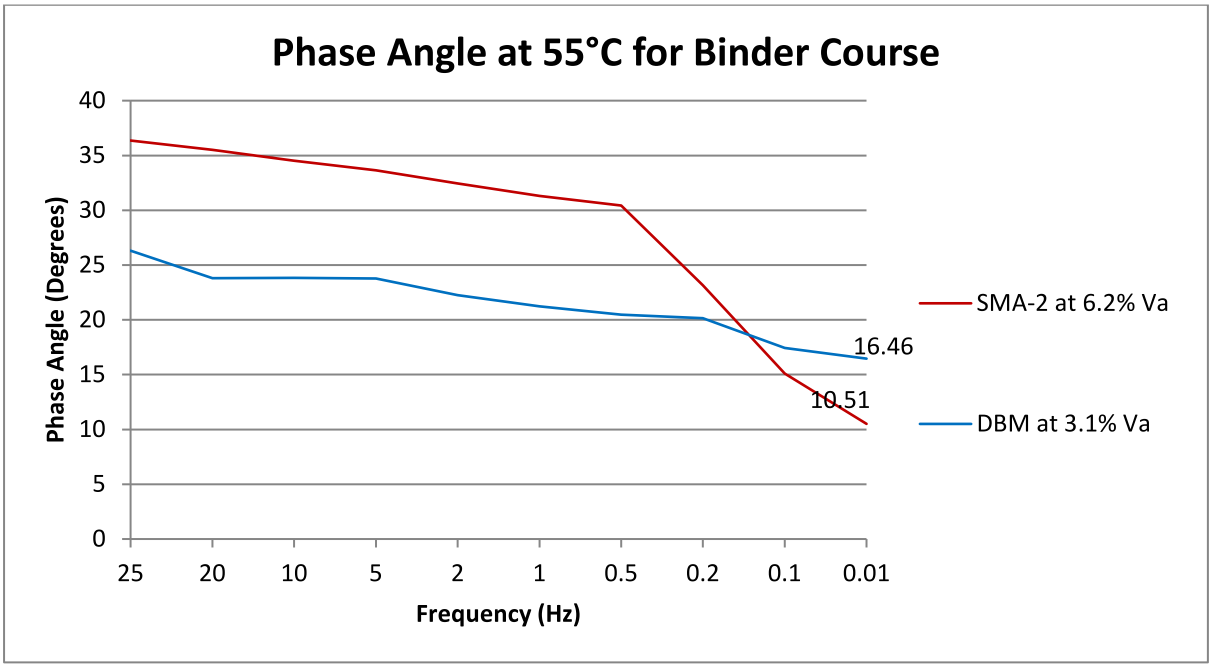

Figure 13. Phase Angle at 55°C for Binder Course

At test temperature of 55°C, SMA II sample having 6.2% V_a has lower phase angle than DBM with 3.1% V_a when subjected to loading at low frequencies as shown in Figure 13. Similarly, at 30°C SMA II at 6.1% V_a has lower phase angle than DBM sample at 1.4% when subjected to loading at low frequencies.

Creep and Recovery

The testing apparatus is capable of applying haversine loads up to 25 KN onto test specimen at desired temperature ranging from 25°C to 60°C to an accuracy of 0.5°C at constant pressure in a control chamber. Platens at top and bottom transfer the load from testing machine onto the specimen. The design stress level covers the range between 69 and 207 KPa for the unconfined tests. The repeated load flow number (F_n) test is a dynamic creep test where haversine axial load is applied for duration of 0.1 second with a rest period of 0.9 second. The rest period has a load equivalent to the seating load. The test can be performed under confinement, or unconfined. Confined test simulates the field condition better. Cumulative permanent axial and radial strains are recorded throughout the test. This test can be used as a performance criteria indicator for permanent deformation resistance of the asphalt concrete mixture, or to compare the shear resistance properties of various bituminous paving mixtures.

The permanent strain can be divided into three major zones. In the primary phase, the strain rate decreases; in the secondary phase, the permanent strain rate is constant; and in the tertiary phase the permanent strain rate dramatically increases. At low stress levels, the material may mainly exhibit primary and/or secondary permanent strain.

The permanent strain rate may approach a value equal to zero as the total strain reaches a certain value suggesting that, at this very low stress level the tertiary flow region may never appear within a reasonable amount of time. At higher stress levels, constant secondary permanent strain rate phase depends on the stress level applied. Large increase in permanent strain generally occurs at a constant volume within the tertiary zone.

The flow number is defined as the postulated cycle when shear deformation, under constant volume, starts. This is the point when tertiary flow starts in the mixture i.e. the point where the rate of change of permanent strain reaches the minimum value. So, the flow number can be viewed as the minimum point in the relationship of rate of change of permanent strain versus load.

The deformations occurring in samples of different mixes have been plotted from the micro-strain data acquired from SPT on carrying out creep and recovery test. Air voids of each sample before testing was determined. Higher deformation has occurred in samples having higher air void ratio. Figures 14 and 15 show the deformation in BC and SMA I at 30°C and DBM and SMA II at 55°C. The air voids are comparable and the magnitude of deformation is higher in BC and DBM as compared to SMA I and SMA II respectively. The deformation has been calculated using strain value and original height of the specimen.

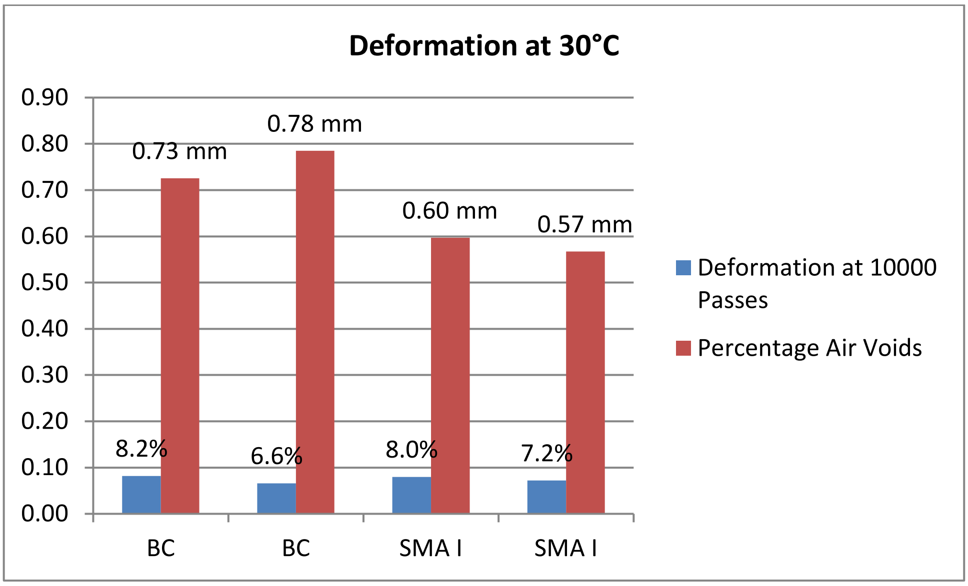

Figure 14. Deformation for Wearing Course at 30°C

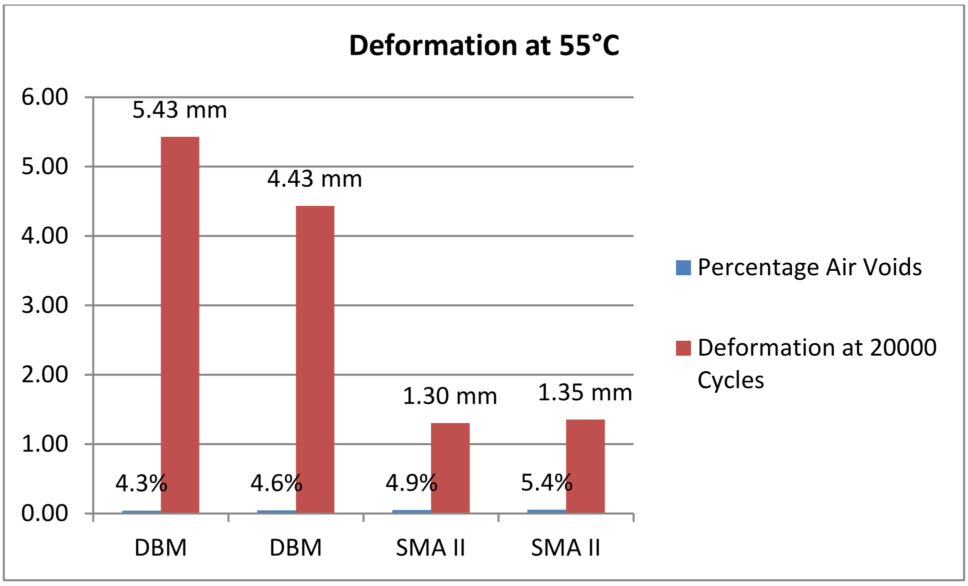

Figure 15. Deformation for Binder Course at 55°C

Strain vs. loading cycle graph has been plotted. The strain values for BC and DBM are higher at 30°C and 55°C as compared to SMA I and SMA II respectively at comparable air voids as shown in Figures 16 and 17 respectively. Figure 18 shows SMA II at 6.7% V_a and 7.1% V_a undergo lesser strain as compared to DBM at 2.9% V_a and 2.7% V_a at 30°C.

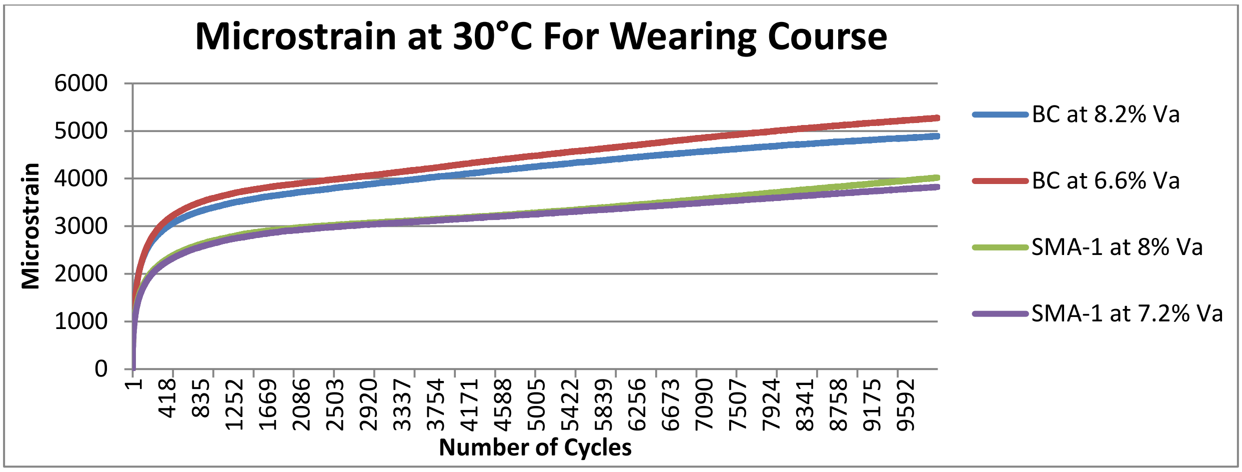

Figure 16. Microstrain for Wearing Course at 30°C

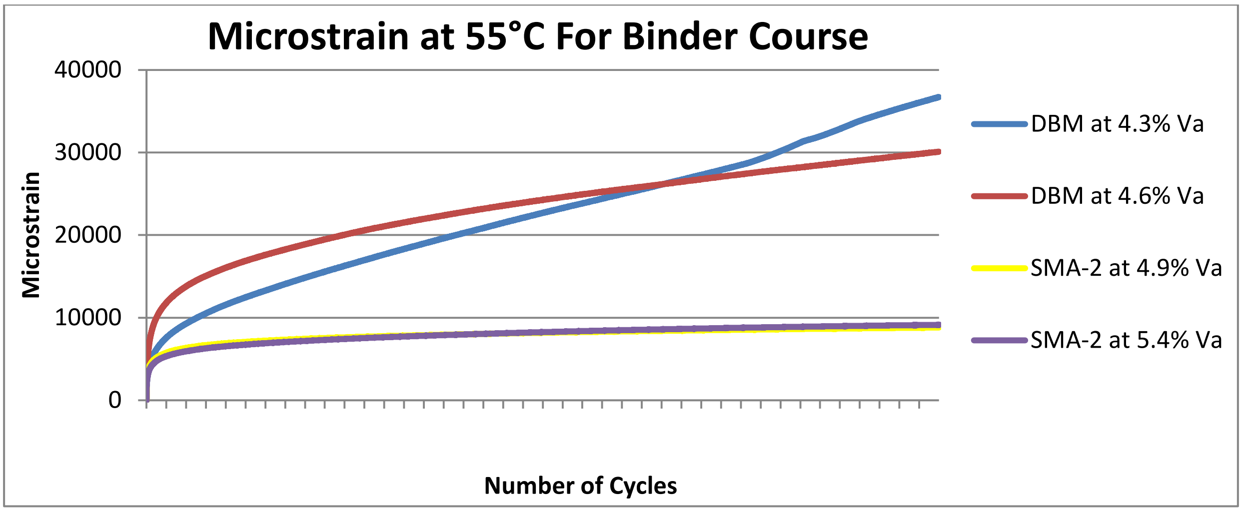

Figure 17. Microstrain for Binder Course at 55°C

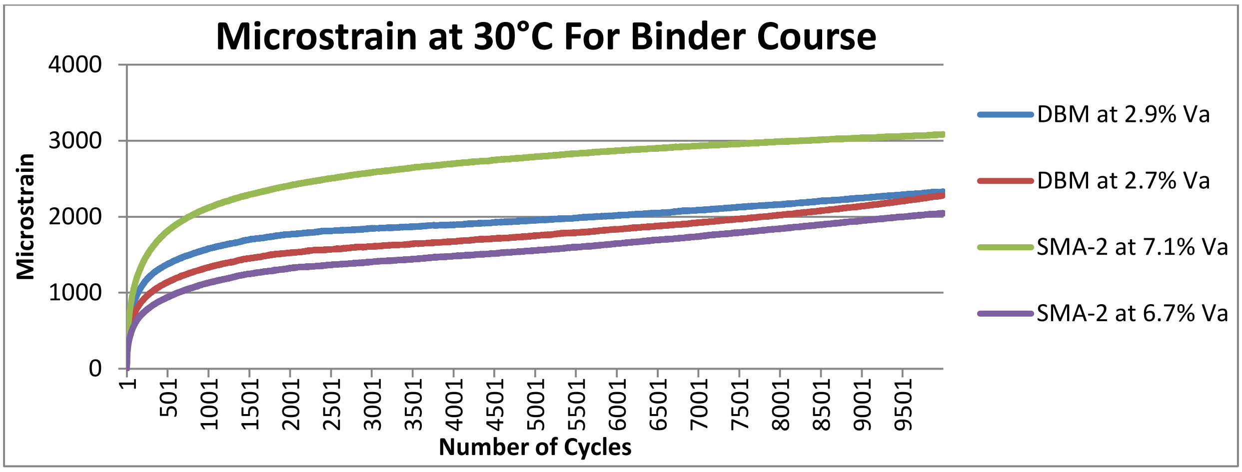

Figure 18. Microstrain for Binder Course at 30°C

CONCLUSION

Focus of this study was forensic investigation of failed pavement and alternate remedy using gap graded mix such as SMA. Forensic investigation was inevitable due to non-availability of construction history, thereby emphasising requirement of maintaining proper construction records. Focus of alternate remedy was to evolve a rut resistant mix, for longer service life of the pavement. Use of modified binder gives the advantage of better modulus at high temperatures while having the ability to maintain viscous properties during extreme low temperatures. Laboratory tests were carried out and found that, performance of SMA in comparison to conventional mixes like BC and DBM is better in terms of permanent deformation.

Conclusions

(a) Higher compaction level can reduce the life of pavement drastically, leading to premature rutting and cracking.
(b) Higher binder content does not ensure better performance of pavement. It improves adhesion to an extent, but causes mastic flow in viscous conditions (at higher temperatures). High binder content yielded high IDT strength in the mix, but failed due to rutting.

(c) The behaviour of binder needs to be understood well to get good performance from it. Modification of a binder must be done in accordance with the required specifications of the site to, get the best performance.
(d) Gradation plays an important role and aggregate quality must be ensured for better performance of the mix.
(e) Fiber is an essential component of SMA Mix, and must be included.
(f) Gap-graded mix has better resistance to deformation due to stone-stone contact.
(g) SMA experiences negligible reduction in air voids due to secondary compaction caused by traffic. Hence, it must be placed and compacted to 4% air voids during construction.
(h) Use of SMA for airfields (runways and taxiways) may prove prudent since, certain portions of airfield pavements (such as edges of runway/taxiway) do not undergo loading due to regular traffic while certain other portions (centre of runway/taxiway, apron, etc.) undergo loading due to heavy traffic.
(i) Gap graded mix like SMA can also fail when construction quality is poor. Excessive compaction may lead to crushing and eventually ravelling. Low compaction level will make the mix susceptible to rutting despite the stone-stone contact.
(j) Due to the gradation of SMA having high percentage of coarse aggregate, mixing becomes very critical, and needs to be closely monitored to prevent stripping. Use of anti-stripping agent may be helpful.
(k) SMA mixes are compressible to an extent. Beyond recommended lift thickness; the compaction effort will increase drastically and may lead to aggregate crushing.
(l) One lift each of SMA II as binder and SMA I as wearing course after milling the BC layer, may prove rut resistant in the test stretch.

Limitations

Cost of SMA is high due to high binder content and requirement of cellulose fiber. Binder required to coat coarse aggregate should be low. Hence, reduction in binder content needs to be researched, thereby also reducing dosage of fiber.

Compaction of SMA Mix beyond 100 mm lift is difficult. Methodology to fabricate specimen of 150 mm height for test in SPT needs to be devised.

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Road Trials of Low-Noise High-Performance Asphalt Surfacings

Iswandaru Widyatmoko
(Corresponding Author)
Technical Director, AECOM
12 Regan Way, Nottingham NG9 6RZ
Tel: 44-115-907-7000
Fax: 44-115-907-7001
Daru.Widyatmoko@aecom.com

Chibuzor Ojum
Senior Engineer, AECOM
12 Regan Way, Nottingham NG9 6RZ
Tel: 44-115-907-7000
Fax: 44-115-907-7001
Chibuzor.Ojum@aecom.com

Arash Khojinian
Team Leader — Pavements,
Safety Engineering and Standards
Highways England
Woodlands, Manton Lane, Bedford MK41 7LW
Tel: 44-300-470-4892
Arash.Khojinian@highwaysengland.co.uk

Robin Hudson-Griffiths
Senior Advisor — Pavements,
Highways England
The Cube, 199 Wharfside Street
Birmingham B1 1RN
Tel: 44-778-612-6545
Robin.Hudson-Griffiths@highwaysengland.co.uk

David Giles
General Manager, Eurobitume UK
Suite 2, Wulfrun House, Lawton Road
Stoke-on-Trent ST7 2AA
Tel: 44-1270-879-205
David.Giles@eurobitume.eu

Ian Michael Lancaster
Senior Chemist and Development
Lab Manager, Nynas UK AB
North Road, Ellesmere Port,
Cheshire CH65 1AJ
Tel: 44-151-327-3171
Mike.Lancaster@nynas.com

Gary Schofield
Technical Manager, Total UK Ltd.
Chain Caul Way, Preston Riversway,
Preston PR2 2TZ
Tel: 44-1772-729-302
Gary.Schofield@total.com

Chris Southwell
Eurobitume UK
Suite 2, Wulfrun House, Lawton Road,
Stoke-on-Trent ST7 2AA
Tel: 44-7774-415-783
Chris.Southwell@ukrba.com

David Markham
Senior Manager
Asphalt Technology, Tarmac
Millfields Road, Ettingshall, Wolverhampton
West Midlands WV4 6JP
Tel: 44-7702-632-636
David.Markham@tarmac.com

Malcolm Simms
Director, Mineral Products Association
38-44 Gillingham Street, London SW1V 1HU
Tel: 44-207-963-8000
Malcolm.Simms@mineralproducts.org

Tim Smith
Regional Materials Manager, Tarmac
Holborough House, Holborough Road,
Snodland, Kent, ME6 5PJ
Tel: 44-7740-065-1178
Tim.Smith@tarmac.com

ABSTRACT

This paper presents findings and recommendations from trials of a new low noise high performance asphalt surfacing on a section of Highways England's all-purpose trunk road network. AECOM facilitated and coordinated delivery of a collaborative research program for Highways England, Eurobitume UK and the Mineral Products Association.

This research started in 2015 by defining a concept of a next generation asphalt surfacing material with enhanced durability properties without compromising on safety requirements and noise performance properties, under the working title of "Premium Asphalt Surfacing System (PASS)." Since then, a series of events comprising an international workshop, detailed laboratory development of the material, the construction of off-network demonstration trials and the road trial installation of the new material were coordinated by AECOM with support from U.K. industry stakeholders. Performance related mix design principles were adopted, and were targeted to produce a dense, low voided, durable asphalt material with good surface characteristics. The key challenge was having a balanced performance between a dense asphalt mixture and good surface characteristics (specifically, low noise and appropriate macrotexture/skid resistance).

During the laboratory and field assessments, the test results were benchmarked against the UK Specification for Highway Works (SHW) Clause 942 for Thin Surface Course Systems. The laboratory assessments included mixture volumetrics, void structure analysis (using X-Ray Computed Tomography scan), deformation resistance and sensitivity to moisture. The field assessments comprised in situ density, macrotexture, surface regularity and road noise. Findings to date have been encouraging and further trials are being organized during summer 2018 which will involve different aggregate sources and a different asphalt producer and installer.

INTRODUCTION

A series of collaborative research projects were set up to accommodate the aspirations of the main stakeholders, Highways England (HE), Eurobitume UK and the Mineral Products Association (MPA), which were seeking to improve the performance of asphalt surfacings currently being used on the motorways and all-purpose trunk road network. The primary objectives of these projects were to ensure that asphalt surfacings continue to deliver value for money on the road network and to maximize the benefit from innovation. The main criteria for the new surfacing were an asphalt material which has significantly enhanced durability, without compromising on safety requirements and noise performance properties. This material was developed under a working title of “Premium Asphalt Surfacing System (PASS).”

AECOM facilitated and coordinated delivery of the research program which successfully completed an international workshop, detailed laboratory development of the material, the construction of demonstration trials and the installation of the PASS on a section of HE’s all-purpose trunk road network. Timeline of the work completed during this research is illustrated in Figure 1.

Figure 1. Timeline of the Collaborative Research Projects

LITERATURE REVIEW

The process used in developing the PASS started with a review of published literature and case studies, as well as using surveys to assess the views of stakeholders from the U.K. and overseas. Details are presented in the published project report (1); summary of findings are presented below.

The initial literature review and worldwide surveys identified complex interactions between parameters which influence durability of asphalt surfacing. These interactions are illustrated in Figure 2. Key parameters are presented in bold black boxes, specifically safety, durability, noise and constructability. The words within lighter orange boxes denote the main factors contributing to each of these key parameters (workmanship, air voids, specification, mix design, PMB/additives and wet friction) and which also acted as the main hubs for more complex interactions between individual components of these factors.

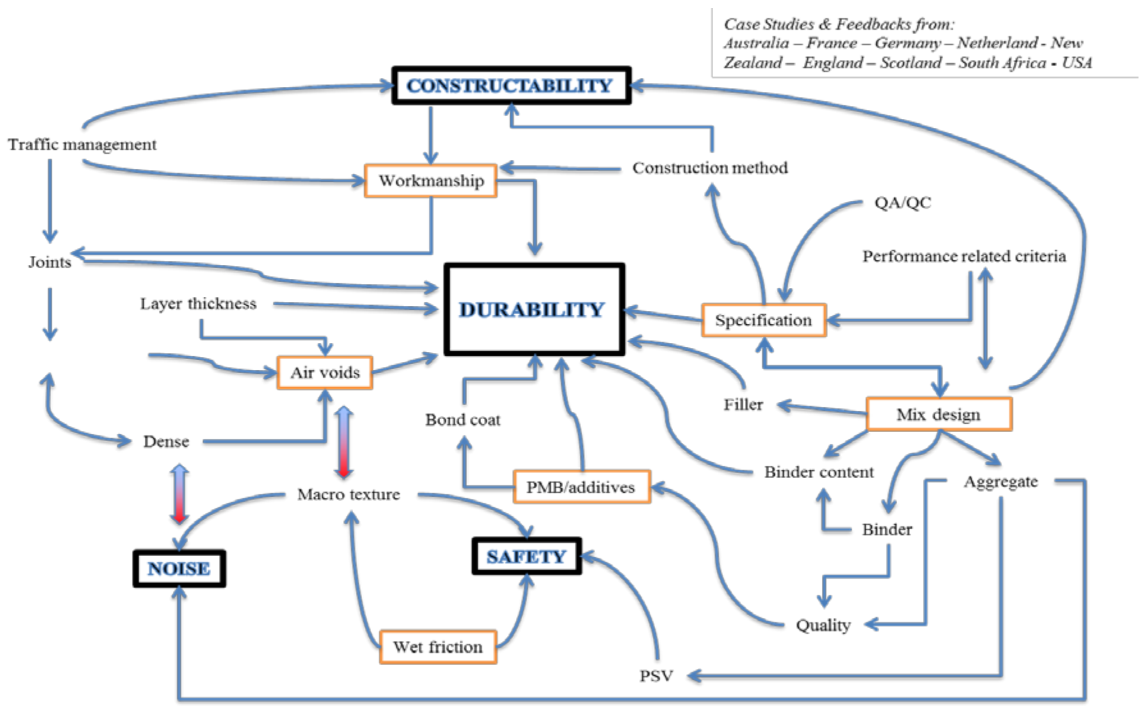

Figure 2. Mapping the complex interactions between parameters which influence durability

INTERNATIONAL WORKSHOP

The workshop took place on 2nd June 2015 and was attended by over 40 experts and professionals from the U.K., France and Germany with various backgrounds; including consultants, contractors, road authorities, asset managers, academics and industry associations. In this workshop, the participants were challenged with the following question: "What are your ideas for the next generation of asphalt surfacing for use on Highways England's Network that will increase durability without compromising the current performance?"

This workshop was an integral part of the project as participants mapped the complex interactions and parameters that influence the durability of asphalt materials and help offer best practice guidelines for future development of improved durable asphalt materials. Amongst the key concepts gathered during the workshop, the “dual layer” approach which was associated with a dense asphalt mixture with good macrotexture was considered practical for trials within this project. In addition, it was agreed that these concepts must be supported by ‘good practice’. The key themes discussed under the broad banner of ‘good practice’ included: improvements to mix design process; better understanding of aggregate packing; improving workmanship (e.g. operational upskilling, training); procurement and ‘risk sharing’ (e.g. investment in plant and equipment, visibility of program); better feedback loop on performance; substrate condition; bond between layers; traffic management (e.g. full road closure, improved safety and joint construction, enable echelon paving); relationship with supply chain and temperature control (use of shuttle buggies).

LABORATORY MIX DESIGN

The mix design principle for the PASS was targeted on a dense, low voided, durable asphalt material with good surface characteristics (low noise and good macrotexture). The following tests were adopted during the mix design stage: mixture workability, resistance to deformation, and sensitivity to moisture. The test results were benchmarked against those expected for UK Specification for Highway Works (SHW) Clause 942 Thin Surface Course Systems (TSCS, which are the preferred surfacing materials for the motorway and trunk road network in England due to their low noise properties) (2).

The mix design explored aggregate packing theories including Bailey’s method (3) to produce the PASS. Bailey’s method provided a good starting point for the mix design of the PASS in order to obtain the required air voids and workability of the mix. The desired in-service performance properties for the PASS include long-term durability, skid resistance and noise reducing properties.

The focus of Bailey’s method is aggregate packing, and the mix design was based on 10 mm nominal aggregate size. The packing characteristics are determined by several factors that include the shape, strength and texture of the aggregates. A slight adjustment to the grading was subsequently done to open up the surface texture by including around 30% oversize aggregate fraction. The optimized mix design incorporated polymer modified binder (class 45/80-60, i.e. penetration 45-80 and softening point >60°C), added at 5.4% by weight of the total mixture. Details have been presented elsewhere (1) and a summary is illustrated in Figure 3. Note that the lower and upper limits shown in the chart were associated with PASS1 mix design whilst CA, FA_c and FA_f denotes coarse aggregate, fine aggregate coarse and fine aggregate fine fractions, respectively, as defined by Varvrik et al (3). The right hand photographs in Figure 3 illustrate good open texture surface course with relatively dense body (see the photos of top and side views). These were considered to meet the description of a “dual layer” material, which was the main target of this study. Further evidence is presented later in this paper.

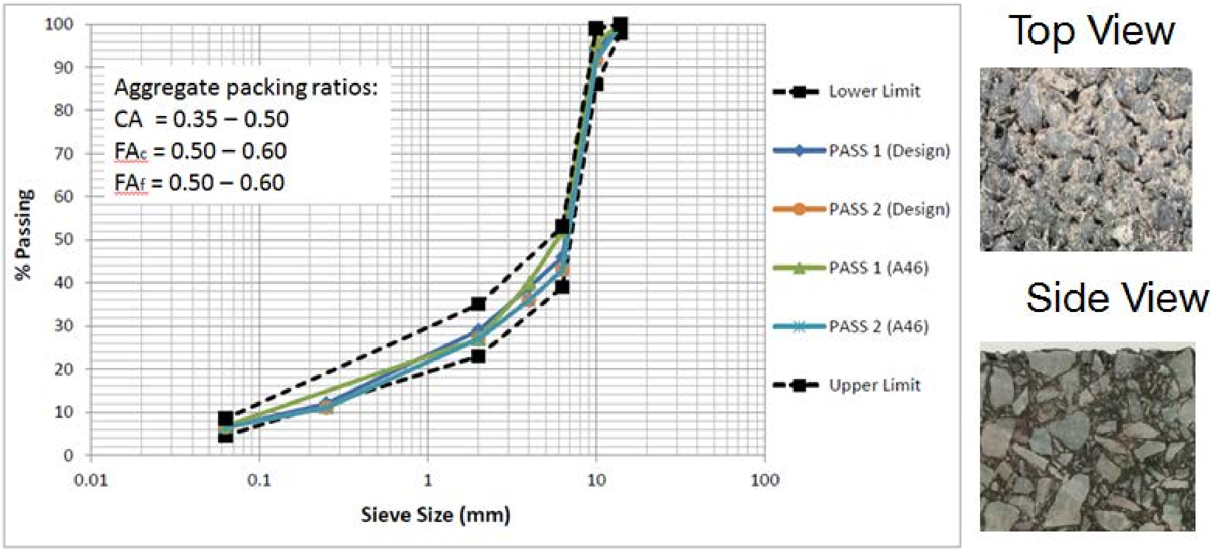

Figure 3. Optimized mix designs

DEMONSTRATION TRIAL

The main objective of the trial was to validate the optimized laboratory mix design. The demonstration trial was conducted on 27th June 2016 at Tarmac's Alrewas Quarry in Staffordshire. The trial section was installed on the main access road of the quarry, where it would be subjected to approximately 100 heavy goods vehicles each day.

The results on cores recovered from the PASS trial section are summarized below:

- The materials were reported as relatively easy to produce and install.
- Air voids of the recovered cores were around 4%, which satisfied the target values (2–6%).
- The mean texture depth to BS EN 13036-1 (4) was 0.8 mm, which was lower than the target (1 mm). It was therefore recommended that further adjustment to the mix composition should be done prior to the main road trials.
- Skid resistance by British pendulum to BS EN 13036-4 (5) demonstrated good surface characteristics with pendulum test value (PTV) being greater than 70.
- There was no indication of moisture sensitivity (BS EN 12697-12 (6)) on the recovered cores, with the value obtained being greater than 70%.
- The resistance to deformation, determined by wheeltrack testing to BS EN 12697-22 (7) of the recovered cores, was considered excellent and better than the expected. At 10000 cycles (in air at 60ºC): wheel track slope less than 0.07 mm per 1000 cycles, proportional rut depth less than 5% and rut depths less than 2.5 mm.
- Impedance tube test to BS EN ISO 10534-1 (8) was used to measure the sound absorption on core samples obtained from the demonstration trial. However, the test method was unable to adequately differentiate between the different asphalt types. It was recommended that the Statistical Pass-By (SPB) test method in accordance with ISO 11819-1 (9) should be conducted for the road trials.

The demonstration trial site was revisited on 6th February 2018 for a visual condition survey and to record surface macrotexture depth measurements. The aim was to assess whether there was any pavement defect such as cracking, fretting, loss of aggregates and/or rutting on the installed asphalt surface course. The results showed no visible defects and this indicated that the surfacing had been able to withstand the relatively heavy traffic levels over the preceding 1 year 8 months since the installation.

HE ROAD NETWORK TRIAL

The project progressed from concept, through to a road network trial carried out over 9th and 10th August 2017. PASS was installed on part of HE's all-purpose trunk road network, a dual carriageway; A46 Hykeham to Carholme (Souhtbound) near Lincoln in England. For this trial, two PASS materials having slightly modified grading envelopes (PASS1 and PASS2), as illustrated in Figure 3, were manufactured and installed, each to the full width of lane 1 to a length of 100 m and a depth of nominally 50 mm.

Following the installation of the PASS materials, cores were collected from the site for further tests to ascertain the mechanical and performance properties of the materials:

- The target in situ design air void content was between 2% and 6%.
- The target in-situ macrotexture was > 1 mm with a provision that the material will be replaced if macrotexture fell below 0.7 mm.
- Figure 4 presents the test layout, test samples and core dimensions for the demonstration trials.

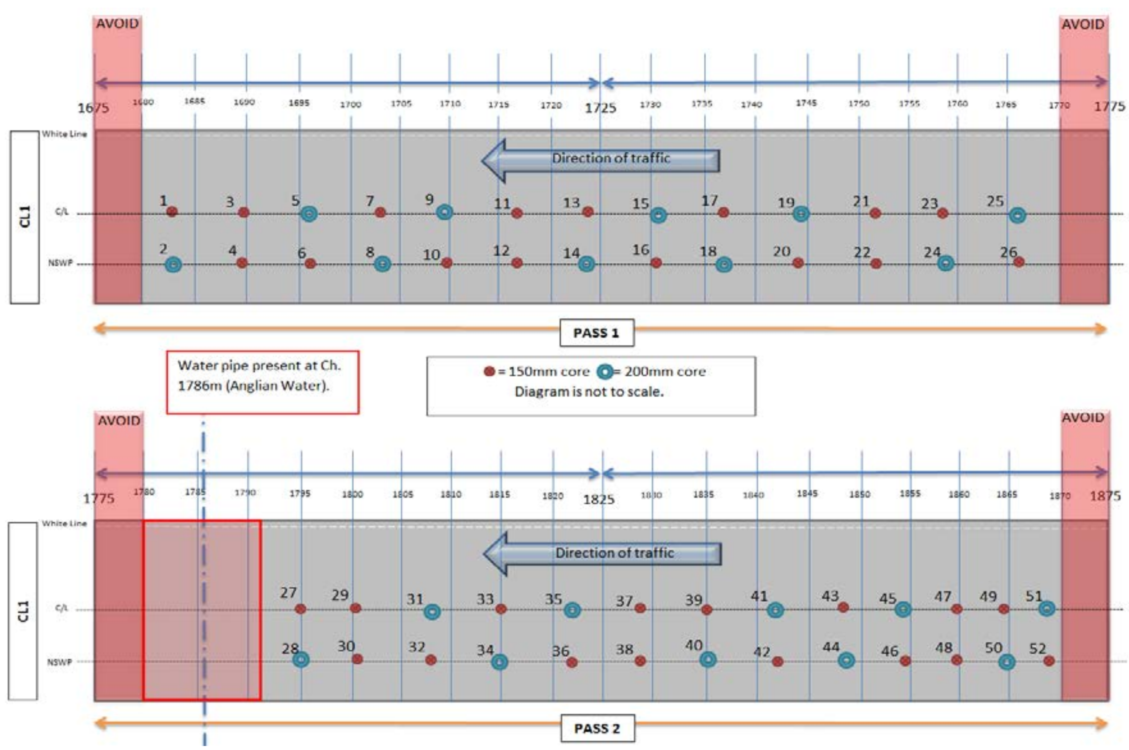

Figure 4. Layout of the trial sections and sampling locations

Summary of the initial findings from the trial sections are presented below:

- Materials in both sections were reported as relatively easy to produce and install.
- Air voids of the recovered cores were between 7 and 9%, which was greater than the design target.
- The mean texture depth (4) was 1.2 mm (PASS1) and 1.3 mm (PASS2); these met the minimum target.
- Skid resistance by British pendulum (5) demonstrated good surface characteristics; comparable to that obtained during the Alrewas demonstration trial.
- There was no indication of moisture sensitivity (6) on the recovered cores. The results were similar to those found during the previous demonstration trial.
- The resistance to deformation, determined by wheeltrack testing (7) of the recovered cores, was similar to the Alrewas demonstration trial and is considered excellent.
- The noise assessment using Statistical Pass-By method (9) suggested that both surface course can be classified as 'very quiet surfacing material', giving an average RSI (Road Surface Influence) of −5.6 dB(A), which is a substantial reduction in noise level when compared against a standard hot rolled asphalt surface course. Figure 5 illustrate the arrangement of the SPB testing, together with the noise levels currently specified in the U.K. (1).

TABLE NG 9/30: (08/08) **Road/Tyre Noise Levels**

Level	Equivalence to Traditional Surfacing Materials	Road Surface Influence RSI
3	Very quiet surfacing material	- 3.5 dB(A)
2	Quieter than HRA surfacing materials	- 2.5 dB(A)
1	Equivalent to HRA surfacing materials	- 0.5 dB(A)
0	No requirement	No requirement

Figure 5. The adopted SPB testing and the associated U.K. Specification NG942

Following some concerns about the relatively high air voids, a more detailed assessment was carried out on the recovered cores. The distribution of air voids within the 40mm depth of the layer was determined by using X-Ray Computed Tomography (CT scan) imaging technique. This work was performed by The Warwick Manufacturing Group (WMG) of the University of Warwick. The method and results are illustrated in Figure 6.

Results from the imaging analysis demonstrated satisfactory findings confirming the presence of a 'dual layer' material, i.e. a material with relatively open texture and high voids at the upper layer but dense (low voids) at the lower part of the layer. This test was repeated on other samples and similar results were obtained, specifically air voids in the upper 15 mm thickness were found to be between 6 and 10% whilst those in the subsequent lower layers (15mm to 40mm from the surface) were within 2 to 6%. This has raised thoughts as to the suitability of the standard test method for air voids calculation, which generates an average result throughout a core, which may not equally represent all of the necessary properties of the material e.g. low voids with surface texture.

The road network trial site was revisited for a visual condition survey on 14th February 2018. The visual observations indicated that there were no observed defects on the sections installed with both PASS mixtures. The surface macrotexture remained in good condition and there was no evidence of rutting or permanent deformation of the installed materials. These results are as expected and satisfactory for good material after 6 months in service.

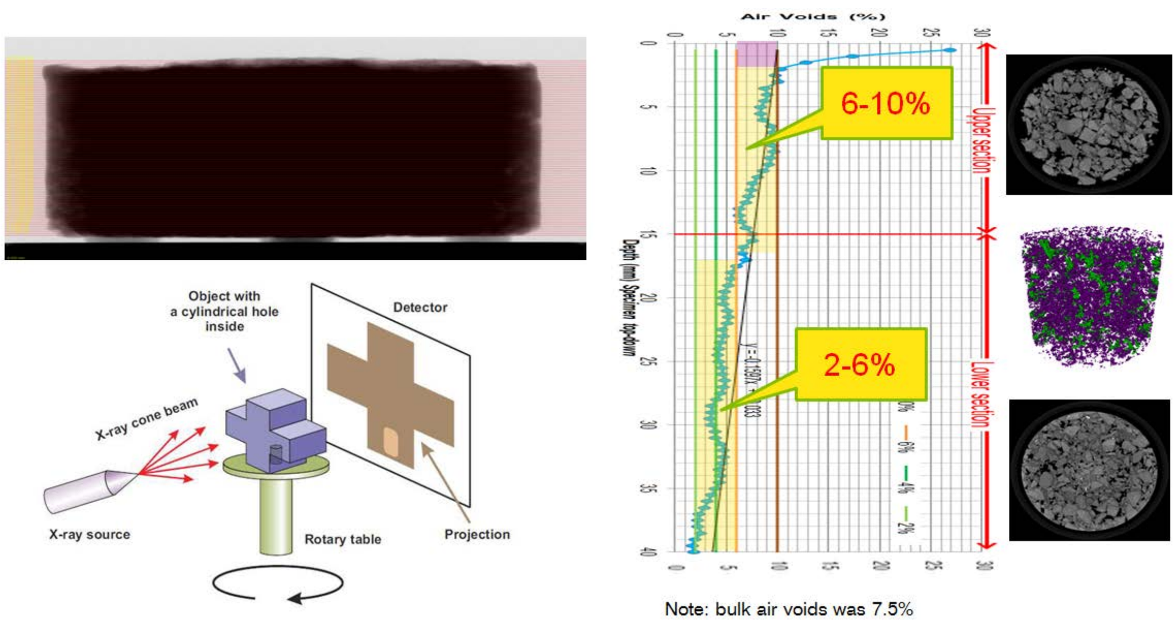

Figure 6. Distribution of air voids within a 40mm depth of asphalt core under CT scan

MANAGING RISK ASSOCIATED WITH INNOVATION

It is recognized that innovation (such as a new material or product) can bring new risks, and these risks must be managed. The PASS material developed under this study followed an assessment procedure for innovative techniques and materials called the Technology/Innovation Readiness Levels. The Readiness Levels, as detailed in Table 1, were used to provide a consistent approach, technique and procedures that are measured by Key Performance Indicators (KPIs) while developing the PASS. The procedure gives a rational approach for assessment of the PASS product at each stage for future developments.

Taking into account the Technology/Innovation Readiness Levels as detailed in Table 1, this project has now progressed from Level 4 — concept stages (laboratory development, investigation and validation), through to Level 5 — demonstration of concept by conducting a demonstration trial off-site at Alrewas

Quarry Access Road in Staffordshire and most recently, to Level 7 — the PASS was installed on HE's all-purpose trunk road — A46 Hykeham to Carholme (Southbound).

Table 1. Assessment Procedure for 'Innovative' Techniques and Materials, adapted from Sanders (10)

Readiness Level	Description
1	Basic principles observed and reported
2	Technology concept and/or application formulated
3	Analytical and experimental critical function and/or characteristic proof-of-concept
4	Technology validation in a laboratory environment
5	Technology basic validation in a relevant environment
6	Technology model or prototype demonstration in a relevant environment
7	Technology prototype demonstration in an operational environment
8	Actual technology completed and qualified through test and demonstration
9	Actual technology qualified through successful mission operations

CONCLUSIONS

This paper presented a new approach to innovation. This was mostly driven by the clients, to meet their requirement to do things better. In this context, working collaboratively with the supply chain is the best way to get results. This project provided an excellent example of managing risk and opportunity by means of collaboration to achieve successful delivery of the innovation. Reports from various stages of this research can be found at http://www.aecom.com/uk/pavement-design-publications/. The new 'next generation' materials were found to demonstrate 'dual layer' structure (dense layer with high surface texture) and good mechanical properties and low noise characteristics. This research has successfully installed Premium Asphalt Surfacing System (PASS) trial sections on the strategic road network demonstrating that PASS can be installed using current practices.

ON-GOING AND FURTHER WORK

The continuation of the development and assessment of the PASS material taking into account the key findings, lessons and recommendations is important. The road network trial on the A46 undertaken in August 2017 should be monitored at regular intervals for its visual condition, mechanical and performance properties, especially the noise characteristics of the PASS to assess suitability for its use on the road network. Further to this, Sideway-force Coefficient Routine Investigation Machine (SCRIM) measurements (11) should be conducted to measure the wet skidding resistance of the PASS mixtures.

The work presented in this paper has been limited to the use of single source each of aggregate and binder, produced by a single asphalt supplier and installed on a section of the strategic road network. Arrangement for further network trials of PASS, but using different aggregate sources and a different asphalt producer and installer, followed by a similar suite of assessment, is currently being currently scheduled to be conducted in the summer of 2018.

ACKNOWLEDGEMENT

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Development of a High-Performance SMA Suited to the Surface Course of National Highways in Japan's Cold, Snowy Regions

Shunsuke Tanaka, Ph.D.
Civil Engineering Research Institute for Cold Region
1-3 Hiragishi, Toyohira-ku, Sapporo, 062-8602, Japan
Tel: +81-11-841-1747
Fax: +81-11-841-9747
S-tanaka@ceri.go.jp

Kimio Maruyama, Ph.D., P.E.
Civil Engineering Research Institute for Cold Region
1-3 Hiragishi, Toyohira-ku, Sapporo, 062-8602, Japan
Tel: +81-11-841-1747
Fax: +81-11-841-9747
k.maruyama@ceri.go.jp

ABSTRACT

Japan's cold, snowy regions have snowfall on a scale that is rare compared with almost everywhere else in the world. In these regions, it is necessary to secure driving safety even under severe environmental conditions in winter. Because harsh environmental conditions in winter are detrimental to highway pavements, asphalt mixtures applied to the highway surface need to be durable enough to resist adverse conditions. The authors conducted research with the aim of developing an asphalt mixture for the surface course of highways that would be highly durable and would secure safe driving under severe winter conditions. That research led to the development of a mixture called high-performance stone matrix asphalt (HP-SMA). In this study, techniques for mixing and applying HP-SMA were examined, and laboratory tests and outdoor experiments were performed to assess the durability and effectiveness of SMA as a measure for improving the winter road surface. The test and experiment results were compared with those of porous asphalt (PA) and dense-graded asphalt 13F (DGA). HP-SMA was found to be obviously superior to PA in terms of durability, and as effective as PA and superior to DGA in terms of improving the winter road surface. These results suggest that HP-SMA is suitable for use as the surface course of national highways in Japan's cold, snowy regions. HP-SMA has been increasingly used for national highways managed by the Hokkaidō Regional Development Bureau of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT).

INTRODUCTION

Cold and snowy regions in Japan have snowfall on a scale that is rare compared with almost everywhere else in the world (Table 1). Because of these severe environmental conditions, it is required for the pavement to have sufficient driving safety functions to ensure safe road traffic in winter. Therefore, porous asphalt (PA) has been used for pavement for national highways managed by the Hokkaidō Regional Development Bureau of the Ministry of Land, Infrastructure, Transport and Tourism (MLIT) since around the year 2000. Indoor and outdoor tests and actual road surveys (1) (2) have confirmed that this PA achieves driving safety functions on freezing road surfaces. However, some issues have been identified regarding PA from the viewpoint of durability (Figure 1). The cause of these issues is considered to be environmental conditions special to cold and snowy regions in Japan, with their heavy snowfall.(3) That is, 1) the freezing and thawing effect of snowmelt water generated during snowmelt season and 2) wear caused by snow-removing graders and tire chains. In light of these issues, national highway pavement in the cold and snowy regions in Japan needs to be sufficiently durable as well as having sufficient driving safety functions.

Based on this background, the authors have carried out research aimed to develop an asphalt mixture for road surfaces that is suitable for national highway pavement in the cold and snowy regions in Japan. These research activities led to the development of asphalt mixture for road surfaces called High-Performance Stone Matrix Asphalt (HP-SMA).

In this research, the driving safety functions and durability of HP-SMA for freezing road surfaces were assessed by indoor and outdoor tests. It was verified that HP-SMA can be expected to fulfill the required driving safety functions and has higher durability than that of PA. Therefore, HP-SMA is found to be suitable pavement for cold and snowy regions in Japan.

Based on this result, HP-SMA has been fully applied for national highways managed by the Hokkaidō Regional Development Bureau since 2017.

Table 1. Population and annual snowfall of major cities in cold and snowy regions of the world

City (Country)	Population (Million)	Snowfall (m)
Sapporo (JPN)	1.9	5.82
Shenyang (CHN)	7.8	0.49
Montréal (CAN)	1.6	2.15
Vienna (AUT)	1.7	1.72
Munich (GER)	1.3	1.00
St. Petersburg (RUS)	4.6	2.97

Figure 1. Damaged PA

HP-SMA

HP-SMA is an asphalt mixture for road surfaces with superior features of SMA: it has driving safety functions due to its coarse surface texture and high durability. The surface texture and schematic drawing of a cross section of HP-SMA are shown in Figure 2. A portion of HP-SMA close to its surface has coarse texture like PA, and its inside portion is a dense structure with the space of aggregate filled with asphalt mortar which contains much filler and asphalt.

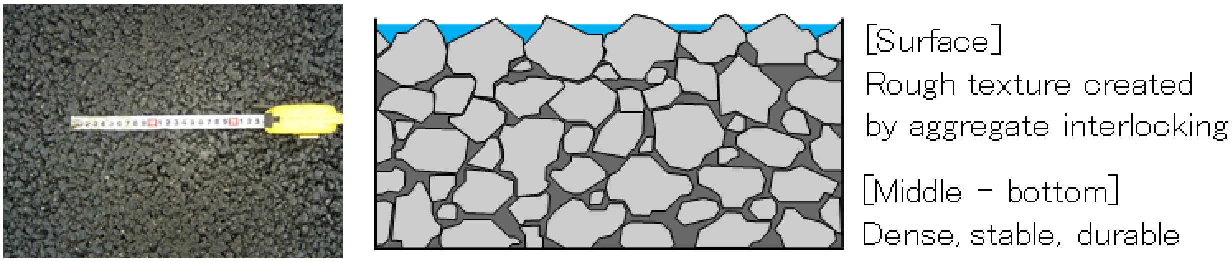

Figure 2. Surface texture and cross-section of HP-SMA

An example of the mix proportion of HP-SMA and that of PA and Dense-Graded Asphalt 13F (DGA) are shown in Table 2. HP-SMA specifications require an asphalt content of around 6.0%. This is to secure the standard surface roughness (mean profile depth (4) ≥ 0.9 mm), which greatly affects driving safety, and to maintain sufficient durability in Japan's cold, snowy environment.

When comparing HP-SMA with SMA 11S (5)—which is a standard SMA made in Germany as a representative of general SMA—, the quantity of asphalt is more than or equal to 6.6% for SMA 11S and 5 to 6% for HP-SMA, which shows that the quantity of asphalt may be normally slightly less for HP-SMA. Void content is 2.5–3.0% for SMA 11S and 3.0–7.0% for HP-SMA. Road surface texture cannot be compared because SMA 11S has no specified value, though HP-SMA does. However, judging from specified values of the quantity of asphalt and void content, the road surface texture of HP-SMA may be coarser.

Table 2. Example of Mixtures of HP-SMA, PA and DGA

Pavements (Target void content)	Asphalt type	Mix proportion (%) (Grain distribution (mm))							
		Asphalt Upper: applied Lower: standard range*	Filler (0 – 0.15)	Screenings (0 – 2.5)	Coarse sand (0 – 2.5)	Crusher run #7 (2.5 – 5)	Crusher run #6 (5 – 13)	Total	Added to the total
HP-SMA (3-7%)	Modified II-type	5.9 5.0 – 7.0	10.4	6.1	6.1	8.5	63.0	100	0.3 (cellulose fiber)
DGA	Straight asphalt	5.8 5.0 – 7.0	9.4	3.8	35.8	8.5	36.7	100	—
PA (17%)	Modified H-type	5.1 4.0 – 6.0	4.8	—	16.3	—	73.8	100	—

* The standard range of the Hokkaidō Regional Development Bureau.

Compaction

The compaction of HP-SMA can be done by using compaction machines that are ordinarily used in Japan, including three-wheel rollers, tandem rollers and tire rollers. An example of a compaction condition is shown in Table 3. Compaction work is shown in Figure 3. Three-wheel rollers that have large linear pressure are used for initial surface compaction at high temperature. For secondary compaction pressure, tandem rollers that have low compaction pressure are used to increase density and flatness while keeping the road surface texture. For finishing compaction, tire rollers are used. Due to this finishing compaction, an effect making it difficult for the aggregate located at the surface to separate is expected.(6)

For HP-SMA, sufficient compaction at high temperature is necessary because the optimal temperature for the compaction of HP-SMA in which modified asphalt is used is higher than that for DGA. In cold, snowy regions, it is necessary to investigate and decide the following before construction: the rolling temperature, the combination of rolling compaction machines, and the number of runs of the machine. This is because it may be necessary to conduct work under environmental conditions which may cause a rapid decrease in the temperature of the mixture.

Table 3. An example of compaction condition

	Breakdown roller compaction	**Secondary roller compaction**	**Finishing roller compaction**
Machine	Three-Wheel Roller	Tandem Roller	Tire Roller
Machine weight	9,300 kg	6,800 kg	12,600 kg
Number of runs	6 runs	6 runs	4 runs
Target temperature	150–170°C	120–140°C	60–80°C

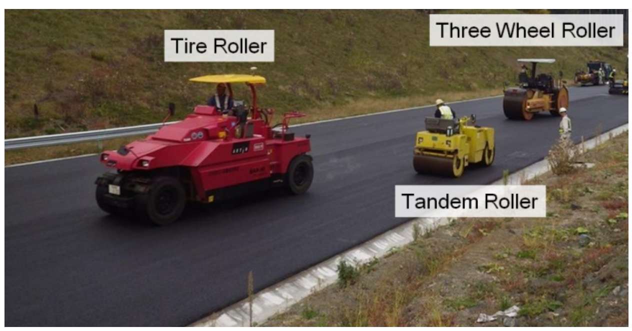

Figure 3. Compaction work

STUDY ABOUT DRIVING SAFETY ON THE ROAD SURFACE IN WINTER

Ensuring safe road traffic is required even under the specific heavy snow environment in the cold and snowy regions in Japan. For this reason, road surface control in winter is a major issue in this region. Therefore, pavement in this region is required to have sufficient driving safety functions for the road surface in winter. PA is already confirmed to have driving safety functions for the road surface in winter in Japan's cold and snowy regions due to its coarse road surface texture. The new asphalt mixture for road surfaces also requires sufficient driving safety functions.

Using the Cold Region Test Track in Hokkaidō (Figure 4), which lies in the snowy and cold region, the driving safety function of HP-SMA for road surfaces in winter was assessed. A similar test was carried out also for PA and DGA and compared their driving safety functions to each other.

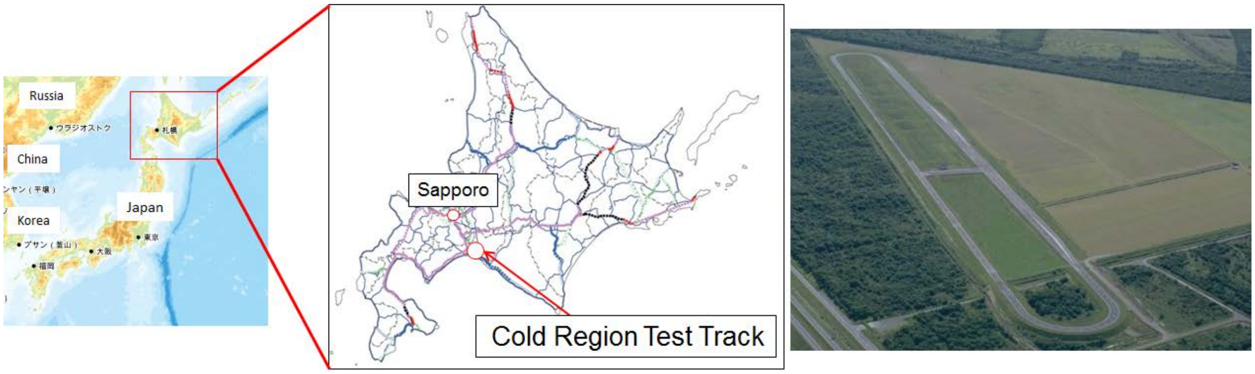

Figure 4. Cold Region Test Track in Hokkaidō

Driving safety assessment test on a freezing road surface

These tests were performed for HP-SMA, PA, and DGA installed in the Cold Region Test Track. A wet road surface, ice film road surface, and ice sheet road surface were artificially made and measured in terms of sliding friction. For the measurement of sliding friction, a locked-wheel friction tester (LWFT: Figure 5), which is standard equipment used in Japan for measuring slide resistance, is used. The running velocity of LWFT when the sliding friction was measured was 40km/h. Table 4 shows the summary of test conditions.

Table 4. Summary of test conditions

Ambient temperature during test	Test on wet road surface	+1°C
	Test on ice film and ice sheet road surface	−8°C
Road surface conditions	Wet road surface	
	Ice film road surface	Film thickness ≥1mm
	Ice sheet road surface	Sheet thickness ≤1mm
Measured item	Sliding friction factor	
Measuring equipment	Locked-wheel friction tester (LWFT)	

Figure 5. LWFT

Figure 6 shows the test results. The sliding friction of HP-SMA is similar to that of DGA and PA for wet or ice sheet surfaces. However, on the ice film surface, the sliding friction of HP-SMA is 0.26 greater than that of DGA, and is similar to that of PA. The results show that HP-SMA is as effective as PA, and better than DGA, at mitigating icy roads.

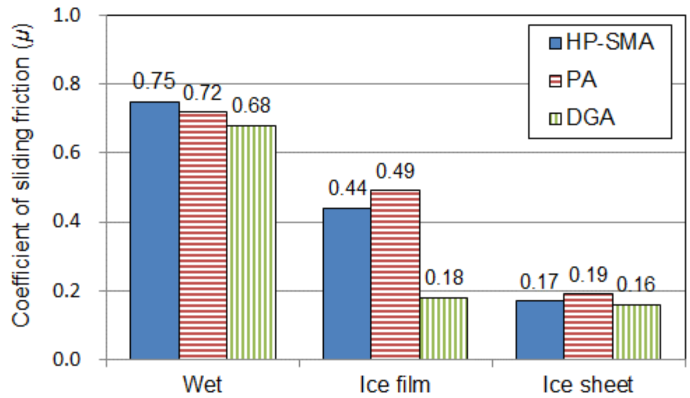

Figure 6. Sliding friction for wet, ice film and ice sheet surfaces on PA, HP-SMA and DGA as measured by LWFT.

Driving safety assessment test when an anti-freeze agent is spread

This test was carried out with HP-SMA, PA, and DGA used on a Cold Region Test Track in a similar way to that described above. At first, an ice film road surface was artificially made on each pavement and an anti-freeze agent was spread. Sodium chloride is used as an anti-freeze agent, and its application rate was 20g/m^2. This application rate is the most regular rate for road surface control in the winter season performed by the Hokkaidō Regional Development Bureau. Then, actual road conditions were simulated by having the vehicle run for a specified number of times, and the sliding friction was measured.

Skid resistance was measured with a Continuous Friction Tester (CFT: Figure 7). A CFT can measure skid resistance continuously at a regular running velocity. CFTs also have the merit of not largely disturbing an ice film road surface because, unlike an LWFT, it is not necessary to perform braking action for a CFT's test wheel.

The value of skid resistance is called the Halliday Friction Number (HFN). The HFN is indicated by an integer between 0 and 100. The lower the HFN value, the slipperier the pavement.

Table 5 shows a summary of test conditions.

Table 5. Summary of test conditions

Ambient temperature during the test	At the start of the test	−3.5°C
	At the end of the test	−6.0°C
Road surface condition	Ice film road surface	
Measured items	Skid resistance (HFN) Number of vehicle runs: 0, 50, 100, 150, 200, 250, 300. Measured after each number of runs shown.	
Measuring equipment	Continuous Friction Tester (CFT)	
Running velocity of CFT	40km/h	
Anti-freeze agent	Sodium chloride 20g/m^2	

Figure 7. CFT

The test result is shown in Figure 8. HFN values for HP-SMA and PA were greatly improved after 50 vehicle runs after salting, and the improved friction continued until 300 vehicle runs. In contrast, DGA did not show improved HFN values after salting. Moreover, even worse, the HFN values of DGA show a smaller value after salting than before. This is likely to occur because, for a DGA road surface, the ice film does not melt after salting until the HFN recovers and the surface of the ice film is covered with a water film.

Consequently, the anti-skid performance that was achieved after salting the ice film was greater on the HP-SMA and PA than on DGA.

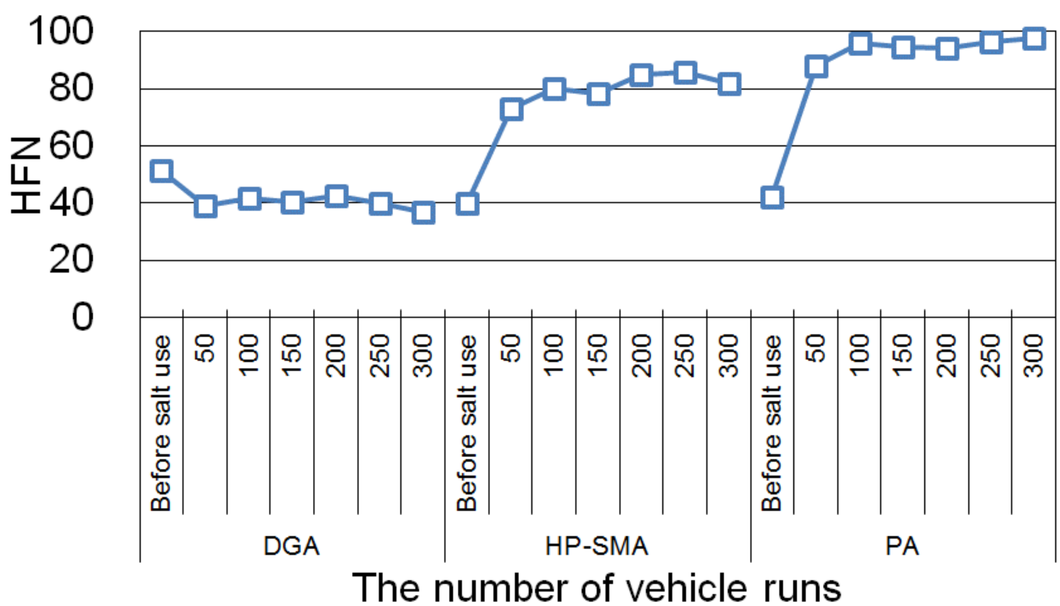

Figure 8. HFNs measured for DGA, HP-SMA, and PA after salting.

VERIFICATION OF DURABILITY

In addition to driving safety functions, sufficient durability even under heavy snow and the cold environment in the cold and snowy regions in Japan is required. PA does possess the driving safety functions for road surfaces in winter as shown in the results of the outdoor test described above; however, as the need for pavement with high durability is increasing in Japan, higher durability on pavement than that of PA is now required along with the maintenance of driving safety functions.

The durability of HP-SMA and PA were compared here with two indoor tests called the Low-temperature Cantabro test and Raveling test. The methods of both tests are specified in "Pavement Performance Evaluation Methods" of the Japan Road Association.(7) The Low-temperature Cantabro test is for assessing the aggregate fretting resistance of pavement and the Raveling test is for the assessment of resistance to raveling characteristics. Both tests aim at the assessment of durability under the cold environment by setting the test temperature under 0°C.

Low-temperature Cantabro test

The low-temperature Cantabro test was developed to assess the impact aggregate loss of PA at low temperature. In Japan’s cold, snowy regions, the impact forces of tire chains and the blades of snow graders cause aggregates to separate from the pavement body. The test is used in Japan to assess durability against aggregate fretting for pavements in snowy cold environments.

First, the test room and the HP-SMA and PA specimens are cured at room temperature. In this study, the temperature was set at −20°C. The well-cured specimens underwent 300 revolutions in a Los Angeles abrasion testing machine. Then the low-temperature Cantabro loss was calculated as a percentage of weight loss over the initial weight. Test specimens are the same size, and the same preparation method as that for the Marshall test is used. The Los Angeles abrasion testing machine and a view of the test specimens before and after the test are shown in Figure 9.

(a) Los Angeles abrasion testing machine

(b) View of test specimens before and after test

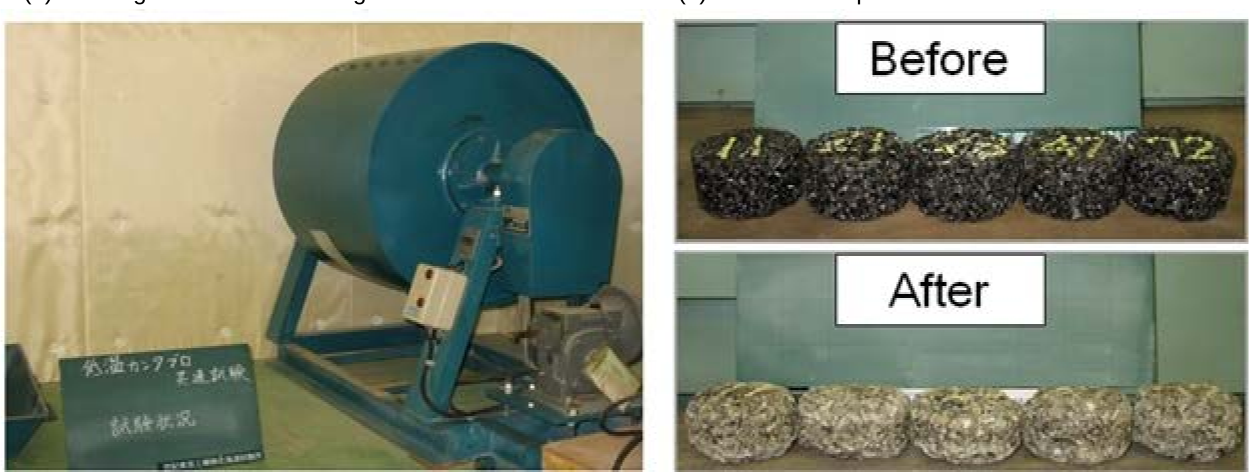

Figure 9. Los Angeles abrasion testing machine and test specimens before and after the test

Figure 10 shows the test results. The low-temperature Cantabro loss of HP-SMA is shown to be about half of that of PA. This result shows that the aggregate fretting resistance of HP-SMA is superior to that of PA. The low-temperature Cantabro loss of PA is specified as less than or equal to 20% for national highways managed by the Hokkaidō Regional Development Bureau. As HP-SMA has been confirmed to have higher aggregate fretting resistance than that of PA, the Hokkaidō Regional Development Bureau decided to put the specified value of low-temperature Cantabro loss in HP-SMA, less than or equal to 16% (a severer value than that of PA), into practice.

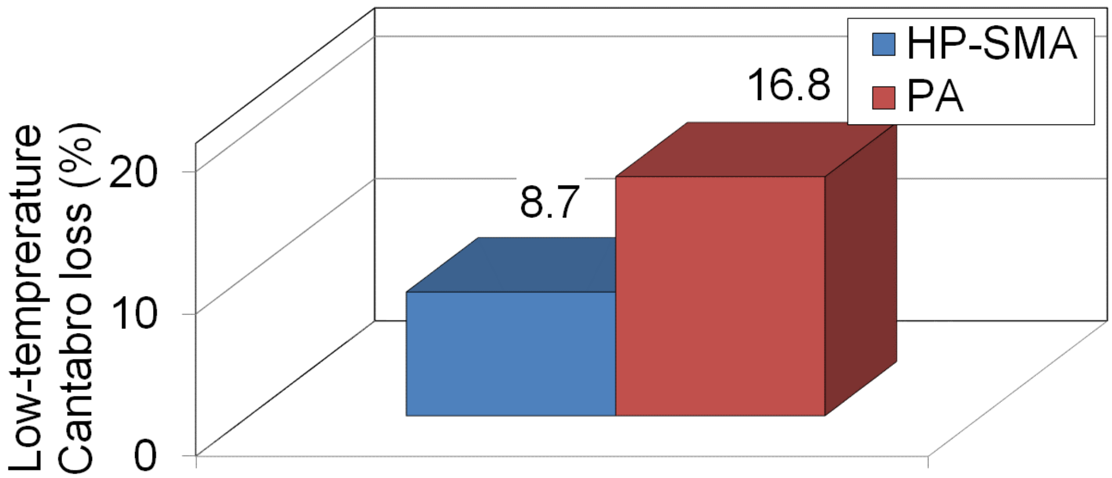

Figure 10. Low-temperature Cantabro losses of HP-SNA and PA

Raveling test

The test was made using CERI's unique wheel-tracking test machine (Figure 11). This test machine has a circular table for specimens and two wheels for attaching the chain. While the table and the wheel rotate, the chain hits the specimen. Then, the abraded areas of the specimen were measured to assess durability.

The room temperature was kept at the test temperature (−10°C). The specimen was cured at the test temperature. The machine simulates driving at 40 km/h for 10 minutes. The cross-sectional area of raveling or aggregate fretting on the specimen was measured at three cross-sections with a laser displacement sensor. Assessment of the aggregate interlocking capability of the specimen was made for the averaged areas of aggregate loss in the three cross-sections.

Figure 11 shows the wheel-tracking test machine and the raveled areas for HP-SMA and PA, as determined by the raveling test. HP-SMA has a smaller raveling area than PA has. HP-SMA has a 150 mm2 smaller raveling area than PA has when the wheel travels 1,000 times, and a 250 mm2 smaller raveling area than PA when the wheel travels 2,000 times. From these results, the raveling resistance of HP-SMA is considered to be higher than that of PA.

Figure 11. Wheel-tracking test machine and raveled area as determined by raveling test.

CONCLUSION

The research findings are as follows.

1. The driving safety function of HP-SMA at the road surface in winter was assessed by the test at the Cold Region Test Track. From the test results, it is found that the driving safety function of HP-SMA is almost equivalent to that of PA and superior to that of DGA.
2. The durability of HP-SMA was assessed by the indoor test. From the results of the test, it is found that the durability of HP-SMA is higher than that of PA.
3. From these findings, it can be said that HP-SMA is a suitable asphalt mixture for national highways in the cold and snowy regions in Japan.

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Effect of Flat and Elongated Aggregate on Stone Matrix Asphalt Performance

Donald E. Watson
Corresponding Author
Watsode@auburn.edu

Grant Julian
Juliagg@auburn.edu

National Center for Asphalt Technology at Auburn University
277 Technology Parkway
Auburn, AL 36830

ABSTRACT

The in-place cost of stone matrix asphalt (SMA) mixtures ranges from 20 to 80% higher than conventional dense-graded mixes. This increased cost has been a factor in the slow implementation of SMA despite its benefits. The production of SMA aggregate alone is believed to cost approximately twice that of conventional aggregate production. Therefore, it is essential to determine whether the need for such high quality aggregate products is necessary for satisfactory SMA performance.

European specifications were considered in developing the requirements of AASHTO M325 and typically require SMA aggregates to have no more than 30 percent Los Angeles abrasion loss and no more than 20 percent flat and elongated (F&E) particles when measured at a 3:1 ratio of length to maximum thickness. However, these strict aggregate requirements were primarily developed for use in Europe, where studded tires are used for winter travel, and may not be necessary for other countries or in areas where studded tire use is prohibited or not needed.

The objective of this research is to evaluate the performance of SMA mixes designed with different percentages of flat and elongated aggregate to determine how critical this aggregate property is relative to performance. Quarries that produce both SMA and non-SMA stone and two quarries that do not produce SMA stone (due to high F&E values) were evaluated.

Based on this research, there is generally no significant adverse effect on performance from using high F&E aggregate if that aggregate has low abrasion loss values. This study was conducted with lab-produced, lab-compacted specimens, and plant-produced, field compacted mixture may produce different results. However, the same F&E requirements are recommended for SMA mixes as is used for Superpave mixes.

INTRODUCTION

Problem Statement

The importance of this study sponsored by the Georgia Department of Transportation (GDOT) was driven by asphalt mixture economics. The cost of using stone matrix asphalt (SMA) mixtures ranges from 20 to 80% higher than conventional dense-graded mixes. Part of the increase can be explained in that SMA is typically used on high traffic volume routes that require night time construction with limited work hours. A portion of the higher cost is also due to a higher asphalt binder demand for SMA mixes which provides increased durability. But much of the increased cost is due to the additional effort and special crushing equipment needed for quarries to produce stone that meets the special requirements for SMA. The production of SMA aggregate alone is estimated to cost approximately twice that of conventional aggregate production. Therefore, it is essential to determine whether such high quality aggregate products are necessary for satisfactory SMA performance.

Since the introduction of SMA from Europe in 1990, there have been questions about aggregate quality requirements needed for these high performance mixtures. European specifications in 1990 required SMA aggregates to have no more than 30 percent Los Angeles (L.A.) abrasion loss and no more than 20 percent flat and elongated (F&E) particles when measured at a 3:1 ratio of length to maximum thickness. These values were adopted as guidelines by a Technical Working Group selected by the Federal Highway Administration (FHWA) in 1991 (*1, 2*). However, as pointed out by Barksdale, these strict aggregate requirements were developed for use in Europe due to degradation resulting from the use of studded tires for winter travel and may not be necessary for other countries (*3*).

A limited study of the effect of F&E aggregate particles on hot mix asphalt performance conducted by the National Center for Asphalt Technology (NCAT) found that the aggregate abrasion value was influenced to some degree by particle shape (*4*). Beam fatigue tests of Superpave mixtures using two aggregate types showed that fatigue resistance characterized by AASHTO T 321 actually improved as the percent 3:1 F&E aggregate particles increased. No significant difference in test results for moisture susceptibility or in aggregate breakdown was observed for the No. 200 (0.075 mm) sieve size. The study did show significant differences in rutting resistance and breakdown on the No. 4 (4.75 mm) sieve size related to percent F&E aggregate particles at the 3:1 ratio. The study concluded that there might be an upper limiting value for F&E aggregate particles at the 3:1 ratio somewhere between 30 to 50 percent. A study by Oduroh found that increases of up to 40% F&E aggregate particles at the 3:1 ratio did not adversely affect performance of Superpave mixes (*5*). The NCAT study further recommended that the upper limiting value for percent F&E aggregate particles should be dependent on L.A. abrasion loss requirements rather than using one threshold for all aggregate and mix types. This recommendation is consistent with Barksdale's comment that the use of a single property as an indication of aggregate degradation is not realistic (*3*). Barksdale related particle breakdown to both particle shape and L.A. abrasion loss.

Project Objective

The objective of this study was to evaluate the performance of SMA mixes in Georgia designed with different percentages of F&E aggregate particles to determine how critical this aggregate property is and whether aggregates that meet quality standards for conventional asphalt mixtures would also perform well for SMA mixtures. The work consisted of a laboratory study to evaluate the effects of various F&E aggregate particles on compactibility, rutting resistance, and cohesiveness of SMA mixtures.

Work Plan/Scope

Aggregates were obtained from three sources that produce specially crushed aggregate just for SMA production. A comparison was also made for SMA crushed aggregate versus the typical production stone used for conventional Superpave asphalt mixes from the same sources. Two additional sources of aggregate with high percentages of F&E aggregate particles that did not meet the current quality standards for SMA were also used for comparing mix design and performance testing.

For each of the aggregates selected, SMA mix designs were conducted and Cantabro abrasion loss (for cohesion and resistance to raveling), moisture susceptibility, and rutting performance were evaluated. Mixtures were compacted using the 50-blow Marshall procedure since this is the mix design compaction method used by GDOT.

A comparative degradation of samples from each mixture caused by the compactive effort was determined by placing the combined aggregate without liquid binder into a gyratory mold and gyrating for 100 gyrations. A comparison of before and after gradations was used to determine the effect F&E aggregate particles may have on the amount of aggregate breakdown that occurs during compaction.

Samples from each SMA mix design were prepared for laboratory performance testing. Cantabro abrasion testing was used to evaluate the durability and raveling potential of SMA mixtures with high percentages of F&E aggregate. Samples were also prepared for Asphalt Pavement Analyzer (APA) rut testing using current GDOT test requirements (GDT 115). GDT 115 is similar to AASHTO T340 except that 5.0 ±1.0% air voids are targeted during sample preparation. GDOT chose to evaluate rutting susceptibility at the 5.0% air void criteria instead of the higher level of 7.0% recommended in AASHTO T340 because it is more typical for pavements to rut due to low air voids rather than at higher air void levels. The test is conducted at 64°C with a 100 lb. vertical load and 100 psi hose pressure for 8,000 cycles. The APA results were compared to the relative rutting susceptibility of the mix designs as related to the percent F&E aggregate.

Moisture susceptibility testing was conducted according to the GDT 66 test procedure. This GDOT procedure is similar to AASHTO T283 with four exceptions:

1. Samples for SMA mixture are prepared at 6.0 ±1.0% air voids.
2. The vacuum saturation period is for 30 minutes and a certain saturation level is not required.
3. After saturation, samples are frozen for a period of at least 15 hours and then put in a hot water bath at 140°F ±3.6° for 24 hours. The freeze-thaw conditioned samples, along with the control samples, are then placed in a refrigerator at 55°F ±3.6° for three hours before testing. The conditioned samples are kept submerged in water.
4. The test loading rate is 0.065 in/min.

Since GDOT targets 5.0% air voids during roadway compaction, and accepts up to 7.0% air voids, it is believed that most in-place pavements have an average of about 6.0% air voids. For that reason, GDOT chose to use 6.0% air voids for moisture susceptibility testing. The remaining GDT 66 requirements are a result of GDOT participation in the original Lottman research in the 1970s and early 1980s. GDOT used criteria from that early research to implement the GDT 66 moisture susceptibility testing prior to the AASHTO T283 procedure being developed. Due to its success with the GDT 66 procedure, GDOT decided to keep the test and related criteria after AASHTO T283 was implemented by other agencies as part of the Superpave technology.

LABORATORY TESTS

F&E Tests

A 1994 study conducted by NCAT included a survey of the state of practice at that time, which revealed that 81% of agencies reported using the 5:1 ratio to determine F&E aggregate particles. The study also found that very few states measured F&E separately as required in ASTM D4791 Method A, but instead used Method B, which is quicker and simpler to perform (*6*). Method A is used to separate particles based on whether they are flat (comparing maximum particle width to maximum thickness), elongated (comparing maximum particle length to maximum width), both flat and elongated, or neither flat nor elongated. Method B is used to separate particles into two groups: flat and elongated (comparing maximum length to maximum thickness), or not flat and elongated.

All aggregates in this study were evaluated for flat and elongated properties using GDT 129, which compares maximum particle length to average thickness. The GDT procedure differs from the ASTM procedure which measures maximum length to maximum thickness. Tests were performed at both 3:1 ratio and 5:1 ratio of length to average thickness for aggregate particles retained on the No. 4 sieve. For comparison, samples were also tested at a 3:1 ratio using the ASTM D4791 Method B procedure for determining flat and elongated particles. All test results were based on a percent of sample mass. The data for comparison, as well as the results of the 5:1 ratio using GDT 129, are given in Table 1.

Table 1 shows the percent F&E aggregate particles for each of the coarse aggregates used in the mixture blend for each quarry source. F&E values ranged from 0 to 6.5% when measured at the 5:1 ratio using GDT 129. For the 3:1 ratio, F&E values ranged from 15.5 to 43.6%. Based on these results, the 5:1 ratio is not able to discriminate the significant differences in aggregate particle dimensions evaluated in this study.

Table 1. Flat and Elongated Aggregate Particles by Source and Stone Size

Quarry	Aggregate	F&E 5:1 (GDT 129), %	F&E 3:1 (GDT 129), %	F&E 3:1 (ASTM D4791), %
A	SMA 7	0.5	19.7	8.4
	7	1.4	25.5	17.3
	89	2.2	23.9	13.1
B	SMA 7	0.3	17.0	6.8
	7	0.1	19.9	9.5
	SMA 89	0.0	18.2	7.0
	89	0.0	19.2	10.2
C	SMA 7	0.0	15.5	9.1
	7	0.0	23.3	15.7
	89	3.0	30.4	17.8
D	7	6.5	38.9	26.5
	89	3.8	20.7	20.9
E	7	6.2	43.6	31.5
	89	1.9	31.6	16.8

As expected, the results in Table 1 show that all aggregates in this study will meet the maximum standard of 10% F&E based on the 5:1 ratio used for conventional stone. The 3:1 ratio appears to be much more useful for evaluating F&E aggregate particles (*7*). Although the ASTM results were almost always lower, there was a good relationship ($R^2 = 0.82$) between the F&E results from GDT and ASTM procedures at the 3:1 ratio as shown in Figure 1. Generally, the GDT procedure results in F&E values approximately 10% higher than the ASTM method.

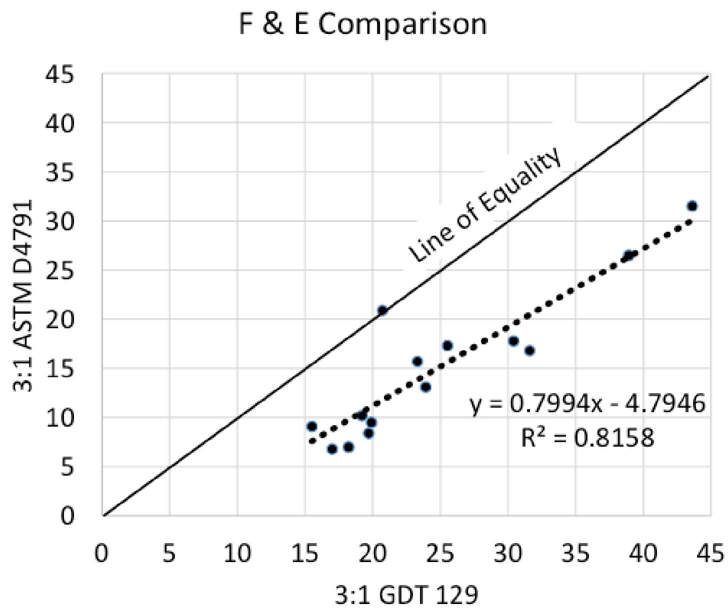

Figure 1. Comparison of GDT 129 versus ASTM D4791 for 3:1 Ratio

One concern regarding aggregates with high F&E aggregate particles is that they may be more prone to fracture during mix production, placement, and compaction than those with low F&E values (*7*). For this reason, the potential for aggregate breakdown was evaluated by placing samples of virgin aggregate in a gyratory compactor mold and gyrated for 100 revolutions. Previous work indicated that 100 gyrations resulted in approximately the same amount of aggregate breakdown as could be expected in the field (*8, 9*). The resulting aggregate breakdown does provide a relative comparison between aggregate sources and physical properties. The results reported in Table 2 are the average difference in percent passing each sieve in the gyrated samples minus the average percent passing the same sieve of the control samples. The comparison is for SMA and non-SMA aggregates from the same quarry source, except that quarries D and E do not have SMA stone.

Table 2. Differences in Percent Passing for Gyrated vs Control Samples

Sieve Size	Agg. A SMA	Agg. A Non-SMA	Agg. B SMA	Agg. B Non-SMA	Agg. C SMA	Agg. C Non-SMA	Agg. D Non-SMA	Agg. E Non-SMA
No. 4	4.0	6.1	4.5	4.5	9.3	9.6	3.0	1.8
No. 8	2.1	3.6	2.9	3.3	6.7	6.4	1.5	2.2
No. 200	0.0	0.6	0.0	0.3	0.6	0.5	0.1	0.3

A comparison of the amount of aggregate breakdown for SMA versus non-SMA aggregates was determined based on three samples of SMA and non-SMA stone from each of the three quarries that provide both SMA and non-SMA stone (3 samples x 3 sources = 9 comparisons). Differences in control samples prepared without gyration were compared to samples after 100 gyrations. Test results for the No. 4 (4.75 mm) sieve are compared in Figure 2. This sieve size was chosen because previous work by NCAT showed that the No. 4 (4.75 mm) sieve was critical in the formation of stone-on-stone contact in SMA mixtures (*7*). For this study there was also a clear breakpoint in gradation on the No. 4 (4.75 mm) sieve.

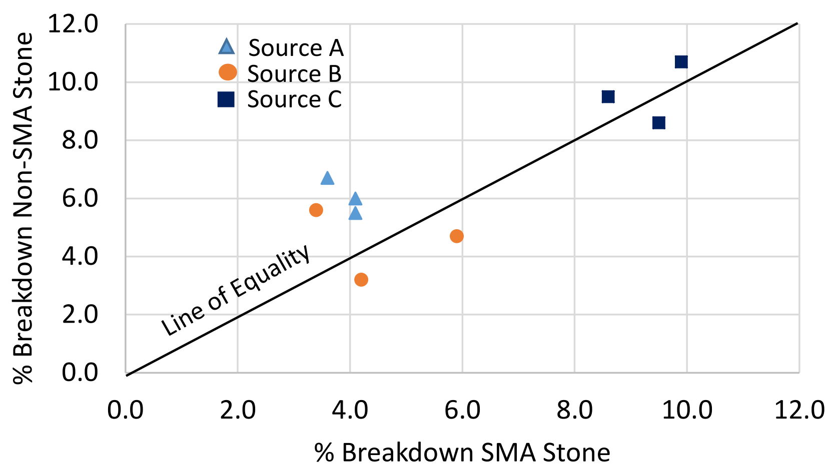

Figure 2. Comparison of Aggregate Breakdown on the No. 4 Sieve

Studies have also shown a direct relationship between percent F&E aggregate particles and the amount of breakdown on the No. 4 (4.75 mm) sieve (*9, 10, 11*). At low levels of breakdown, the non-SMA stone had a slightly greater difference from the control samples than the specially produced SMA stone; but as the amount of breakdown increased, the differences between aggregates approached the line of equality (Figure 2). Statistical results from an Analysis of Variance (ANOVA) indicates that the differences between breakdown of SMA and non-SMA stone are not significant at the 95% confidence interval (p-value = 0.105).

An analysis of aggregate breakdown on the No. 8 (2.36 mm) and No. 200 (0.075 mm) sieve was also performed, but as shown previously in Table 2, there was insignificant difference in percent passing either of the sieves after gyratory compaction. These results seem to verify earlier research findings that aggregate breakdown on the No. 200 (0.075 mm) sieve was not dependent on the percent 3:1 F&E aggregate particles (*4*).

An ANOVA of the breakdown on the No. 4 (4.75 mm) sieve was performed for all eight aggregate sources (three sources with and without SMA stone, and two additional sources without SMA stone). A comparison of aggregate breakdown to the percent flat and elongated particles was conducted. A p-value of 0.000 was obtained, which indicates the breakdown was significantly affected by the flat and elongated property of the aggregate. However, it was anticipated that breakdown would be greater for higher F&E aggregates, as has been the case in other studies (*7, 9, 10, 11*). In this study, the opposite trend was observed; aggregate with higher F&E values actually had less breakdown. For that reason, a comparison was also made of percent F&E aggregate particles to L.A. abrasion loss. The results show that as percent F&E increased, the L.A. abrasion loss decreased (Figure 3). These results show that the toughness of the aggregate in resistance to abrasion may explain why breakdown did not increase as the percent F&E increased. The values emphasize the importance of F&E properties and abrasion values being considered together when specifying aggregate properties for SMA performance.

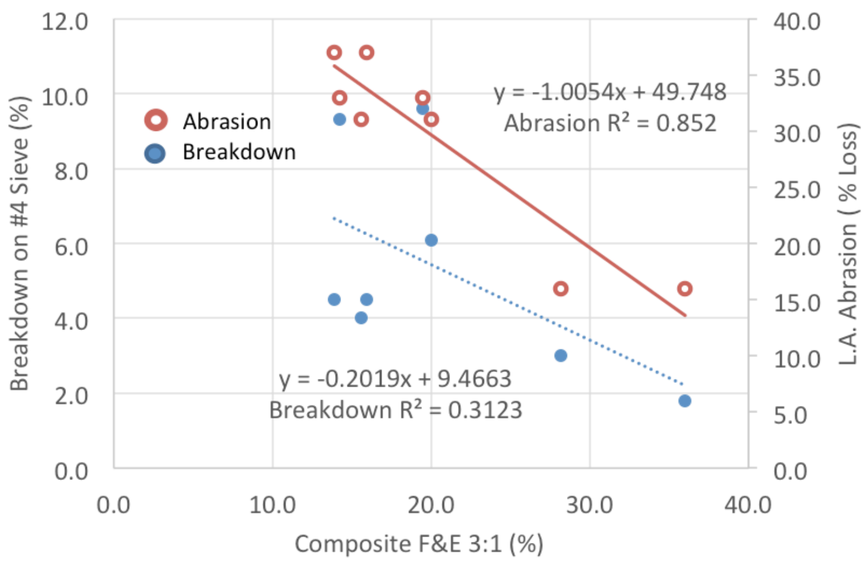

Figure 3. Comparison of the Effect of F&E on Aggregate Breakdown and L.A. Abrasion

Barksdale identified that aggregate properties of abrasion resistance and percent F&E must be considered together to identify aggregates that will perform well in an SMA pavement (*3*). As resistance to abrasion loss increases and the aggregate becomes tougher, it will have better resistance to aggregate degradation, and the amount of F&E aggregate particles can be increased. Likewise, as the percent of abrasion loss increases, the proportion of F&E must be reduced accordingly. Table 3 provides a summary of Barksdale's recommendation in relation to L.A. abrasion and F&E properties of aggregate at the 3:1 ratio for use in SMA mixtures. This approach will guard against the possibility of having a source with both high F&E and high abrasion while allowing more economical aggregates that may not meet the 20% maximum F&E generally specified.

Table 3. Limits of L.A. Abrasion and Percent F&E at the 3:1 Ratio for SMA Aggregate (*3*)

L.A. Abrasion, % Loss	F&E Limit, (3:1 Ratio)
≤ 45	≤ 20
≤ 40	≤ 25
≤ 35	≤ 35
≤ 30	≤ 40
≤ 25	≤ 45

Mix Designs

Approved SMA mix designs from three aggregate sources used on previous projects were provided by GDOT. All mixes were compacted using the 50-blow Marshall procedure. A summary of the mix blends, gradations, and volumetric properties is provided in Table 4.

SMA mix designs were also prepared using non-SMA aggregates from the same three sources. In addition, aggregate from sources D and E that did not meet the current requirements for SMA stone of a maximum of 20% F&E at the 3:1 ratio based on GDT 129 were used. Six trial blends were made for aggregate E material in order to find a combination that would meet the gradation specification as well as the voids in coarse aggregate (VCA) requirements. The most promising blend was used, although it had an optimum asphalt content of 8.3%, which was above the current maximum binder amount allowed by GDOT.

Other research has shown that mixtures with high F&E aggregate particles resist densification and result in higher air voids and increased asphalt demand (*4, 12*). The same trend was observed during the volumetric analysis. Sources D and E, which had the highest percent F&E aggregate particles, also had the highest voids in mineral aggregate (VMA) properties and highest optimum asphalt content as shown in Table 4. It is important to note that Georgia uses the effective specific gravity to calculate VMA. Figure 4 shows that VMA increases as the percent F&E increases. These results are consistent with findings during NCHRP 9-8 research (*8*). All mixes met the recommended minimum VMA threshold of 17% and were within the specification range of 70-90% for voids filled with asphalt (VFA) (*1, 2, 8*). It is possible that the high asphalt demand for mixes with high F&E values may limit their use even if satisfactory mixtures can be designed.

Table 4. Mix Design Blend and Volumetric Properties

Aggregate Source	Agg. A SMA	Agg. A Non-SMA	Agg. B SMA	Agg. B Non-SMA	Agg. C SMA	Agg. C Non-SMA	Agg. D Non-SMA	Agg. E Non-SMA
Blend:	79% SMA #7	69% #7	70% SMA #7	75% #7	68% SMA #7	68% #7	66.7% #7	79% #7
	14% M10	10% #89	11% #89	5% #89	12% #89	12% #89	11% #89	5% #89
		13% M10	10% 810	10% 810	11% 810	11% 810	11% M10	6% M10
	5.75% 50/50 Filler	6.75% 50/50 Filler	8.0% 50/50 Filler	9% 50/50 Filler	8% 50/50 Filler	8% 50/50 Filler	10.3% 50/50 Filler	9% 200W Filler
	1.25% H. Lime	1.25% H. Lime	1% H. Lime	1% H. Lime	1% H. Lime	1% H. Lime	1% H. Lime	1% H. Lime
Gradation (Spec):	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing	% Passing
3/4" (100)	100	100	100	100	100	100	100	100
1/2" (85-100)	98	94	87	91	96	97	95	96
3/8" (50-75)	57	54	54	61	66	70	64	69
No. 4 (20-28)	25	25	24	25	23	23	28	24
No. 8 (16-24)	21	20	18	19	16	16	20	16
No. 16	18	17	16	16	14	14	16	14
No. 30	17	16	14	15	13	13	14	13
No. 50	15	15	13	14	12	12	13	12
No. 100	13	13	12	12	11	11	12	11
No. 200 (8-12)	10.1	10.2	10.1	10.6	9.5	9.5	10.8	8.4
Composite F & E, 3:1	15.6	20.0	13.9	15.9	14.2	19.5	28.2	36.0
L.A.Abrasion Loss, %	31.0	31.0	37.0	37.0	33.0	33.0	16.0	16.0
Opt. AC, % (5.8-7.5)	6.4	6.2	6.5	6.2	6.6	6.6	7.1	8.3
Gmm	2.403	2.410	2.377	2.382	2.378	2.379	2.362	2.405
Gmb	2.306	2.313	2.282	2.287	2.283	2.283	2.268	2.308
Gse	2.644	2.647	2.614	2.615	2.622	2.621	2.623	2.738
Gsb	2.622	2.620	2.590	2.581	2.589	2.592	2.597	2.712
Va,% (3.5 ± 0.5)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
VMA, %	18.4	18.0	18.4	18.0	18.7	18.6	19.7	22.7
VFA, % (70-90)	78.2	77.8	78.2	77.7	78.6	78.5	79.7	82.4
Pbe	6.09	5.82	6.16	5.71	6.13	6.19	6.74	7.97
Dust/Pbe	1.66	1.75	1.64	1.86	1.55	1.53	1.60	1.05
VCAdrc	40.1	39.3	39.4	40.1	40.2	40.6	42.7	42.4
VCAmix	38.6	38.2	37.7	38.3	36.7	36.6	42.1	40.9
VCAmix < VCAdrc	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

Note: Georgia uses G_{se} rather than G_{sb} to calculate VMA.

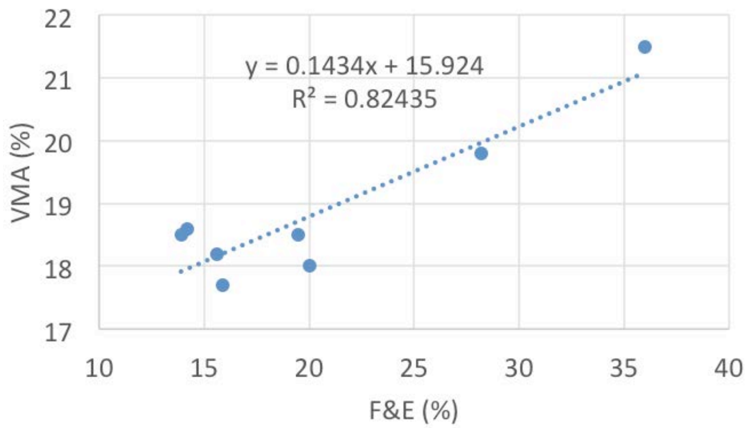

Figure 4. Effect of F&E on VMA Values

It is generally possible to reduce VMA and corresponding asphalt demand by changing the gradation. The result may vary depending on the mix, but generally the VMA for SMA mixtures can be reduced by increasing the percent passing the No. 4 sieve or the No. 200 sieve, or both. If the gradation is changed, VCA tests will need to be conducted to ensure stone-on-stone contact still exists for the coarse aggregate portion of the mixture.

PERFORMANCE TEST RESULTS

Flat and elongated aggregate particles are generally a concern in asphalt mixtures because elongated aggregate is prone to breaking during roadway compaction and the resulting uncoated aggregate faces may accelerate moisture damage and raveling. The elongated particles may also realign during service life and result in a potential for rutting. For these reasons, it was important to select a test procedure that would give an indication of performance under the distress conditions of raveling, rutting, and stripping. The Cantabro abrasion test was selected to evaluate the abrasion resistance while the Asphalt Pavement Analyzer was used to evaluate rutting. Moisture susceptibility was evaluated by the GDT 66 procedure which is a state-specific procedure similar to AASHTO T283.

Cantabro Results for Stone Loss

The Cantabro abrasion test, AASHTO TP 108-14, is used to evaluate the cohesiveness of asphalt mixes and raveling resistance. The procedure requires placing individual compacted samples into an L.A. abrasion machine at 77°F ± 2° and rotating for 300 revolutions at 30–33 rpm. The steel balls normally used in the L.A. abrasion procedure are omitted for the Cantabro test. After the test is completed, the amount of stone loss is determined by comparing the difference in original and final mass. The results from this study show that all of the samples had relatively little stone loss (Figure 5). There is no specific maximum value of Cantabro stone loss for SMA mixes, but the maximum value for open-graded friction courses is typically 20%. Noticeably, the two sources with the highest F&E values also had the lowest Cantabro loss. This may be due to those sources also having the highest asphalt content, which would tend to keep samples more intact. These results follow a trend reported by Aho et al. that stated degradation due to wearing will not occur if the aggregate has less than about 40% F&E at the 3:1 ratio (*9*).

A two-sample t-test was conducted to compare the nine compacted specimens made with SMA aggregate to the nine non-SMA aggregate specimens. A p-value of 0.951 was obtained, which indicates there is no significant difference in Cantabro performance for either SMA or non-SMA aggregates.

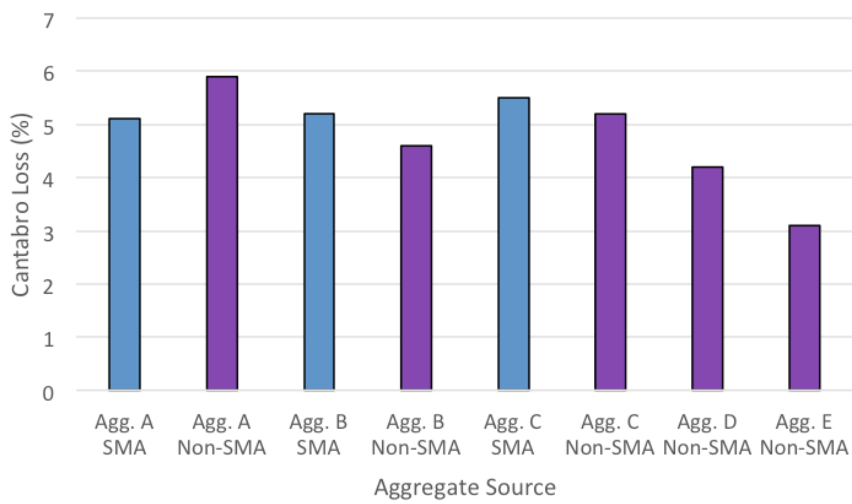

Figure 5. Cantabro Stone Loss Based on Aggregate Source

Rutting Susceptibility

Resistance to rutting was evaluated with the Asphalt Pavement Analyzer (APA) rutting test performed according to GDT 115 (which is similar to AASHTO T340 except it requires samples be targeted at 5% air voids). GDOT targets 5.0% air voids for the APA test because 5% to 7% air voids are typically targeted for roadway construction acceptance, and the concern is that a mix generally is more likely to rut with lower air voids than higher air voids. The test is conducted at 64°C and uses 100 lb. wheel load and 100 psi hose pressure. The test is conducted for 8,000 cycles and the maximum rut depth allowed is 5 mm. All test results were within the 5 mm maximum specification limit, and a coefficient of determination shown in Figure 6 indicates no correlation between rut depth and percent F&E (R^2 = 0.06).

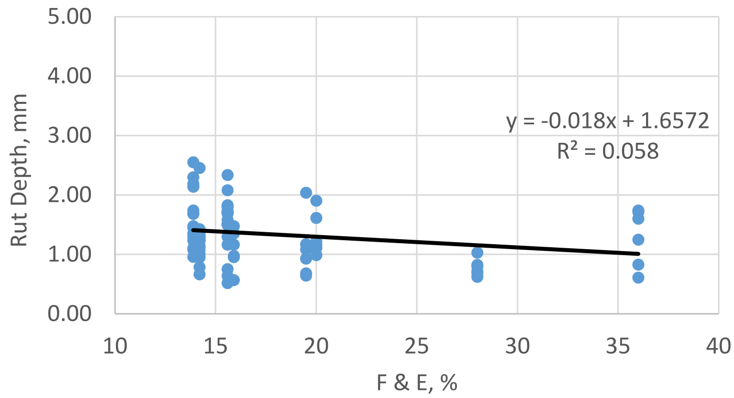

Figure 6. APA Rut Depth vs Percent F&E

Moisture Susceptibility

One concern regarding flat and elongated particles is that they may be detrimental to performance due to a potential for stripping. It has been suspected that F&E particles are more easily broken than cubical particles during production, placement, and compaction. Some agencies do not allow vibratory compactors to be used during the compaction process of SMA mixes to avoid fracturing aggregate. When aggregate particles are broken during construction, it not only changes the gradation but also exposes a face of two aggregate particles (*13*). The fractured, uncoated particles will make it easier for moisture to penetrate the particle and initiate stripping of the asphalt film.

The moisture susceptibility of mixes produced with low and high F&E properties was determined based on GDT 66. The test varies slightly from AASHTO T283 as discussed earlier in the report. GDOT requires a minimum tensile splitting ratio (TSR) of 80% after conditioned samples have been vacuum saturated, subjected to a freeze/thaw cycle, and conditioned in a hot water bath. The freeze-thaw cycle is an accelerated aging procedure to simulate several years of environmental conditioning. A provision is made that the TSR value may be as low as 70% so long as all six specimens used in the moisture susceptibility testing have a minimum tensile strength of at least 100 psi. GDOT also requires a minimum average tensile strength of 60 psi for both wet and dry subsets.

A statistical evaluation of results showed that there was a significant difference in control tensile strength but not for conditioned strength or TSR values. However, the control strengths were higher for the non-SMA stone mixes and explain why the three highest F&E non-SMA aggregate sources had the lowest TSR values. The results, shown in Table 5, indicate that the tensile strength of SMA mixes is not adversely affected by aggregate F&E values. Similar results were reported in NCHRP 425, which showed that F&E variations had no effect on moisture susceptibility (*8*). A study of the effect of F&E aggregate particles on asphalt mix performance also indicated that there was no clear trend or relationship in respect to fatigue resistance based on F&E values; however, it was not unusual for fatigue resistance of mixes to increase as the percent F&E particles at the 3:1 ratio increased (*4*).

For each of the SMA aggregate mixes and one of the non-SMA aggregate mixes the conditioned tensile strength was higher than the control strength. However, these results are not unusual when hydrated lime is used as an anti-strip additive with granite aggregate sources.

Table 5. Tensile Strength Results

Aggregate Source	Agg. A SMA	Agg. A Non-SMA	Agg. B SMA	Agg. B Non-SMA	Agg. C SMA	Agg. C Non-SMA	Agg. D Non-SMA	Agg. E Non-SMA
TS-Conditioned (psi)	88.3	89.9	78.3	92.6	85.1	84.7	76.4	77.1
TS-Control (psi)	79.4	104.8	72.5	93.7	78.8	77.6	85.2	86.4
TSR, % (≤ 80)	111.3	85.8	108.0	98.8	108.0	109.1	89.6	89.3

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

This study did not have a source of aggregate with a combination of high F&E aggregate particles and high L.A. abrasion loss. For the limited aggregate sources evaluated, several conclusions can be made from this research, many of which support previous findings.

1. The 5:1 ratio of F&E particles did not discriminate well between the differences in F&E properties of various aggregates. The 3:1 ratio was much more sensitive to such differences.

2. Previous recommendations of no more than 20% F&E aggregate particles based on a 3:1 ratio were taken from European requirements for use with studded tires during winter operations. Those limitations appear to be unnecessarily restrictive based on satisfactory performance of mixes with high F&E aggregate particles in this study if studded tires are not allowed.

3. F&E aggregate properties should be considered with other properties, such as L.A. abrasion, in order to evaluate aggregate acceptability. Aggregates with high F&E values may perform well if they have low abrasion loss.

4. Aggregate breakdown on the No. 4 (4.75 mm) and No. 200 (0.075 mm) sieves is not significantly different between SMA and non-SMA stone and is thus not dependent on the percent F&E particles.

5. Aggregates with high F&E aggregate particles generally have higher VMA properties and may require higher binder content. The economics of producing mix with high asphalt content may limit the use of sources that have high F&E aggregate particles. Changes in gradation may be used by designers to reduce VMA and asphalt demand.

6. There is no significant difference in Cantabro abrasion loss between SMA and non-SMA aggregate.

7. There is no correlation between rut depth and percent F&E particles. However, the differences in APA rut depth were statistically significant, with the non-SMA stone showing the greatest rutting resistance. All rutting values for both SMA and non-SMA stone were well within the 5 mm tolerance allowed by GDOT.

8. There was a statistically significant difference in control tensile strength values based on percent F&E. However, the control strengths were higher for non-SMA aggregates. Generally, the tensile strength of SMA mixes is not adversely affected by aggregate F&E values.

Recommendations

- The maximum F&E limit (≤ 20% F&E at a 3:1 ratio) that is a standard threshold used by most agencies for SMA aggregate should be reconsidered based on satisfactory performance results of high F&E aggregates in this study.

- It is recommended that Superpave F&E criteria required in AASHTO M323 at a 5:1 ratio be specified for SMA mixtures as well.

- It is recommended that field projects which use SMA stone with higher proportions of F&E aggregate be evaluated for performance during mix design.

Future Research

Similar research is needed for quarry sources that may have both high L.A. abrasion loss and a high proportion of F&E aggregate particles to determine if such sources can also provide satisfactory performance. The results of this study were determined from laboratory mixed and laboratory compacted samples. Performance test results from plant-produced, field-compacted specimens with high F&E may be different and should be evaluated.

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DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Georgia Department of Transportation, the National Center for Asphalt Technology, or Auburn University. This report does not constitute a standard, specification, or regulation. Comments contained in this report related to specific testing equipment and materials should not be considered an endorsement of any commercial product or service; no such endorsement is intended or implied.

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Stone Matrix Asphalt (SMA) Case Study, Thornton, Illinois: Analysis of 20-Year Stone Matrix Asphalt Material on Williams Street

Jay D. Miller
Abdul Z. Dahhan, P.E.
Chicago Testing Laboratory Inc.

INTRODUCTION

The intersection of Margaret and Williams Street in Thornton, Illinois, serves as the main gateway to one of the largest stone quarries, not only in the region but in the United States. The predominant share of the quarry's daily production is loaded on trucks that have to travel through this intersection. The average daily traffic for the intersection is not considered to be high, at a modest 9,500 ADT. However, a disproportionate percentage of the traffic is comprised of fully loaded trucks estimated at approximately 1,800 trucks per day during peak production months. Peak trucking during the season also coincides with the hottest days of the summer, further increasing the effects of the heavy loads.

Proven to be one of the most successful uses of Stone Matrix Asphalt (SMA) in the State of Illinois and the United States, the intersection of Williams and Margaret streets in Thornton, Illinois remained in place and in use for over two decades with minimal to no maintenance, until recently resurfaced in 2017. Chicago Testing Laboratory, in cooperation with Gallagher Asphalt and the Illinois Department of Transportation and with support from Levy Companies and Owens Corning Materials, performed a materials analysis in order to help determine just what made this pavement a 20-year success story.

BACKGROUND

SMA in Illinois was relatively new in the '90s, with one project being attempted by the Illinois Department of Transportation as the first SMA placement in Illinois. Gallagher Asphalt Corporation of Thornton, Illinois. was the contractor on this project that called for the installation of a 50 mm thick lift of a 12.5 mm SMA mixture. The overlay consisted of SMA surface, placed over a 25 mm thick leveling course of 9.5 mm conventional dense graded mix. The roadway to receive the overlay was a 20-year-old, bare, jointed Portland Cement Concrete pavement, five miles long of a four-lane divided highway locally known as IL 394. This first SMA placement on IL 394 by IDOT, was essentially the test strip for Williams and Margaret Streets.

Since the opening of the Thornton Quarry in Thornton, Illinois, there has been a history of significant traffic loading at the intersection of Williams St. and Margaret St. This loading is primarily due to a high volume of loaded truck traffic from the Thornton quarry, one of the largest aggregate quarries in the world and Gallagher Asphalt's primary hot mix asphalt production facility, both located near the intersection. Loads in this location approach one million equivalent single axle loads (ESALs) annually and have repeatedly caused significant deformation at or near the intersection.

In 1997, the Illinois Department of Transportation (IDOT) scheduled the intersection, as well as a significant section of Williams Street, to be resurfaced with Stone Matrix Asphalt (SMA). Gallagher Asphalt Corporation was the contractor on the project. The original pavement design on the project called for 2 inches of dolomitic binder course SMA (bottom lift) overlaid with 2 inches of steel slag surface SMA (wearing course).

Prior to construction of the intersection, Gallagher, in cooperation with IDOT, removed slabs of the existing pavement for analysis near the intersection of Williams and Margaret. The slabs were analyzed and it was found that approximately 8¼ inches of dense graded hot mix asphalt (HMA) was present over Portland cement concrete base. The analysis showed that significant permanent deformation was occurring at depth within the pavement structure. Although the bottom lift of pavement showed little to no deformation, it was decided by IDOT to remove the entire HMA pavement and replace with full depth SMA material. Using the same SMA mixes as on the mainline pavement at Williams Street, the intersection would be paved with three lifts of Binder SMA covered by a Surface SMA lift. The mainline pavement would be paved with 2¼ inches of Binder SMA covered by 2 inches of Surface SMA.

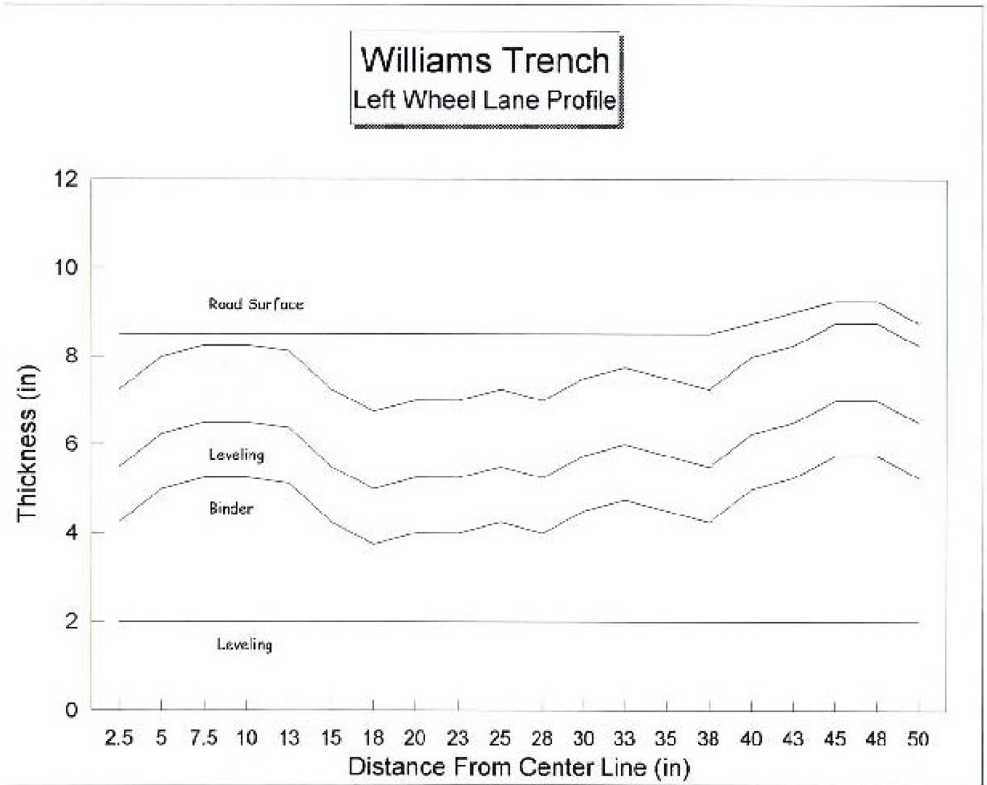

Figure 1. Original Trench Profile

The original mixture designs for this project were completed by IDOT Central Bureau of Materials, with material selection by Gallagher Asphalt. Materials used in the SMA mixture design included:

- Steel Slag (Levy Companies) [87% in surface SMA]
- Crushed Dolomite (Material Service) [85% in Binder SMA]
- Crushed Stone Sand (Material Service)
- Mineral Filler (Material Service)
- MAC 20 Polymer Modified Asphalt Cement (Seneca Petroleum)
- Cellulose Fiber (Kapejo Inc.)

OBJECTIVE

The SMA pavement at the intersection of Williams and Margaret has proven to be one of the most successful pavements placed in the State of Illinois, and although subjected to atypical and excessive stresses the pavement has remained in place, with little to no maintenance, for nearly two decades. The objective of this study is to analyze the pavement data during production, and after 20 years of aging, to determine what aspects have allowed this SMA to be so successful in the same hostile environment where other pavements have repeatedly failed.

Data Summary (Production 1997)

Not only was this one of the first SMA projects in Illinois, but this was also early in the move by the Illinois DOT to Quality Control/Quality Assurance. As such, control on this project was the responsibility of the contractor, Gallagher Asphalt Corporation. Below is a summary of the project production quality control testing results:

	Target	Binder SMA	Target	Surface SMA
G_{mm}	2.480	2.506	2.875	2.865
G_{mb}	2.381	2.365	2.760	2.792
Voids (%)	4.0	5.4	4.0	2.5
Density (%)	94.0	94.3	94.0	96.2
AC (%)	5.8	5.7	5.8	5.7

Gradation (%)	Formula	Binder	Formula	Surface
12.5 mm	76	70	86	86
4.75 mm	21	19	29	31
236 mm	17	15	17	18
750 mm	7.5	7.1	7.0	6.5

The original mixture design was modified by quality control, in cooperation with quality assurance, in order to maintain specified volumetric data and to minimize drain down of the material during transport to the project site which proved to be a significant issue.

Data Summary (2017)

SMA mixture at the intersection of Williams and Margaret Street was cored by CTL for evaluation after 20 years of service life. Testing was conducted in accordance with Illinois Test Procedure 405, Illinois Modified AASHTO T324, Illinois Modified AASTHO T164, Illinois Modified AASHTO T166, and Illinois Modified AASHTO T209. Testing was conducted in two phases.

For the first phase, two test sections were selected for testing. The first location was in an area which did not display visual rutting and the second in a location closer to the intersection which displayed some visual signs of rutting. Six-inch cores were extracted from each test section. Each core consisted of a surface lift on top of three additional binder lifts. Only the surface and first binder lifts were tested. A data set for each location was produced from I-FIT, Hamburg Wheel Track, AC Content determination, and density.

For the second phase, 4-inch cores were extracted for the following testing: Asphalt Content/Gradation (Illinois Modified AASHTO T164), Tensile Strength Ratio (Illinois Modified AASHTO T283), Bulk and Maximum Specific Gravity, with Stability and Flow (Illinois Modified AASHTO T166, Illinois Modified AASHTO T209, and Illinois Modified AASHTO T245). The surface and binder lift were separated prior to testing.

Table 1. Summary of Test Locations

CTL ID	Location Details
A	6-inch cores 100 feet from Williams St. and Margaret St. stop line. No significant visual
B	6-inch cores 50 feet from Williams St. and Margaret St. stop line. Some visual rutting
S-1	Surface lift of 4-inch cores taken from Williams St.
B-1	Binder lift of 4-inch cores taken from Williams St.

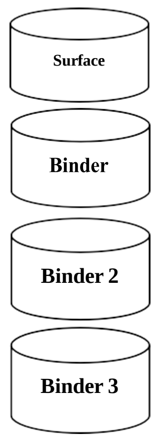

Figure 2. Lift Arrangement in Cores

The above figure shows the four lifts of SMA as originally constructed, including one lift of surface SMA and three lifts of Binder SMA. The SMA at these locations cored were full depth to existing concrete, and the cores were taken through the full SMA pavement profile. The surface lift as originally constructed was 2 inches, with each of the three Binder lifts constructed at approximately 2¼ inches.

Table 2 below, shows the summary of performance testing completed on the cored specimens, specifically the Illinois Flexibility Index test (I-FIT) and the Hamburg Wheel rut depth analysis.

Table 2. Summary of Performance Test Results

CTL ID	Flexibility Index	Rut Depth at 10,000 Cycles	Rut Depth at 20,000 Cycles
A Surface Lift	4.4	1.76 mm	1.98 mm
A Binder Lift	12.1	1.45 mm	1.83 mm
B Surface Lift	4.6	2.50 mm	2.85 mm
B Binder Lift	18.5	1.95 mm	3.13 mm

Table 3 below shows the volumetric analysis performed on each individual core and SMA pavement lift, including bulk specific gravity (G_{mb}), maximum specific gravity (G_{mm}), and percent density.

Table 3. Summary of Volumetric Test Results

CTL ID	Gmb	Gmm	Density
A Surface Lift	2.824	2.956	96%
A Binder Lift	2.421	2.533	96%
B Surface Lift	2.850	2.937	97%
B Binder Lift	2.484	2.530	98%

Table 4 below shows the asphalt analysis completed on the cored specimens, including asphalt content (AC) and Penetration of the recovered asphalt material.

Table 4. Summary of AC Test Results

CTL ID	AC Content	Penetration Value	
A Surface Lift	5.40	15	18
A Binder Lift	5.71	27	27

Table 5 below shows the summary of the gradation analysis performed on the cored specimens, including percent passing the critical sieves for each type and lift of pavement.

Table 5. Summary of Gradation Test Results[1]

Sieve Size		S-1 Surface Lift		B-1 Binder Lift	
		Percent Passing			
19 mm	3/4”	100	100	100	100
12.5 mm	1/2”	85	87	77	76
9.5 mm	3/8”	69	70	51	49
4.75 mm	No.4	37	36	26	26
2.36 mm	No.8	25	25	24	24
1.18 mm	No.16	20	20	20	20
0.6 mm	No.30	16	16	17	17
0.3 mm	No.50	14	13	14	14
0.15 mm	No.100	11	11	13	12
0.075 mm	No.200	8.3	8.1	9.7	9.8

Table 6 below shows the results of the analysis performed on each SMA lift for tensile strength ratio and average tensile strength.

Table 6. Summary of Tensile Strength Ratio Results

CTL ID	Average Unconditioned Strength	Average Conditioned Strength	Tensile Strength Ratio
S-1 Surface Lift	194.9	149.0	0.76
B-1 Binder Lift	184.5	122.6	0.66

[1] Relevant sieve sizes have been updated per IDOT Manual of Test Procedures Appendix B.22 procedure using correction factors for adjusting the gradation of cores to estimate the gradation of the in-place pavement.

Table 7 below shows the summary of stability and flow results on the individual lifts of the cored specimens.

Table 7. Summary of Flow and Stability Results

CTL ID	Average Corrected Stability (pounds-force)	Average Flow (inch)
S-1 Surface Lift	3469	23.50
B-1 Binder Lift	2479	33.67

Table 8 below shows additional material analysis performed on in place 20-year-old SMA pavement cores taken near the intersection of Williams and Margaret street.

Table 8. Asphalt and Steel Slag Testing (after 20 year aging)

	Binder	Surface
Penetration	34	23
PG Grade	70–26	76–23
ΔT_c	−2.1	−4.1
FI	15.3	4.5
%Metallic		84
% FeO		27

OBSERVATIONS

Although the data available is limited and not all-inclusive, we can still make observations and draw several conclusions based on the data that is available and presented herein:

- The Flexibility Index showed that the SMA Binder course provided a better flexibility index than the SMA Surface course.
- Additional rutting during the Hamburg wheel analysis showed that the in place mixture was not expected to have significant additional deformation. Analysis of the surface material closer to the intersection, which already showed more physical in place permanent deformation, showed higher rutting potential than the material tested further away from the intersection where

permanent deformation was not already present.

- Rutting by field observation, after two decades of severe loading, showed minimal to slight rutting (as compared to previous pavements at this location and typical expectations under these loading conditions) at the intersection (from 4 to 14 mm).

Figure 3: Rut Depth String Line, Williams St. Intersection

- Degradation of the aggregate structure was observed, likely from the compaction/loading that occurred during production and/or from the loading over the 20-year period, and included approximately 5% degradation from the original job mix formula.
- Surface SMA asphalt binder was oxidized more than the binder lift of SMA as shown by the lower penetration values and higher Performance Grade(s).
- The polymer modification remained effective, and the PG grade was consistent with a 70−22 polymer modified material, and showed marginal degradation of the low temperature grade, after 20 years of pavement service life.
- The steel slag in the surface SMA did not degrade over time and was still consistent with the steel slag aggregate material specified as measured chemically. The steel slag materials was shown in Table 8 as 84% Metallic and 27% ferrous oxide (FeO).

CONCLUSIONS

Several key factors contributed to the ultimate success of this SMA pavement, but a few stand out from the data obtained during production and the data obtained in the analysis of the pavement completed 20 years later:

- Density during production was above 94%, and with stone on stone contact achieved, little to no additional compaction was possible even with the degree of heavy loading seen throughout the lifecycle of the SMA. Demonstrated by the less than two percent increase in density achieved over the 20-year pavement life cycle.
- The stone matrix design, with stone on stone contact, also resulted in a high stability pavement which resists rutting of the mixture after initial compaction, as proven by the low Hamburg results of 2.5 (Surface SMA) and 2.5 (Binder SMA) shown after 20 years.

- The stone matrix design remained durable over the 20-year pavement life largely due to: a high asphalt binder content, high film thickness, and a high polymer content that remained effective over the pavement life as shown in the final pavement performance grade that remained consistent with specifications even when tested 20 years later. These properties allowed for a high durability pavement, as further validated by the I-FIT results of 4.5 (Surface SMA) and 15.3 (Binder SMA) in the 20-year-old pavement cores.
- Selection of proper ingredient materials and their quality properties, including: steel slag aggregate for friction wearing course, sound, clean dolomitic stone for the binder course, and high polymer modified asphalt cement (which was not common at the time of production), and cellulose fiber, along with the proper mixture design and production controls (both QC and QA), all contributed to the success of the final SMA.

RECOMMENDATIONS

Use of SMA, including full depth perpetual SMA, is recommended in any high loading condition pavement where long term rut resistance and durability are desired. From this pavement analysis one can conclude the use of polymer modified asphalt, coupled with steel slag and dolomitic limestone SMA mixtures will perform for decades when properly designed, produced, constructed and controlled. In order to validate durability and rutting resistance, performance testing (a rut test and cracking test), similar to the Hamburg wheel and Illinois Flexibility Index test, are recommended during production. In addition, as with any Hot Mix Asphalt (HMA) pavement, ensuring proper quality control, including monitoring and ensuring proper density during mixture laydown, is key to long term performance and ultimately to pavement success.

SUMMARY

The Williams and Margaret street intersection SMA and the SMA placed on Williams street has proven to be very successful, especially as compared to previous material placement at the same location. IDOT's decision to use full depth SMA at the intersection has proven successful, and the original SMA binder lifts remain in place as a perpetual pavement intersection, now with a new polymer 4.75 mm leveling course and 2 inches of new surface SMA. Unlike the original pavement replaced by the SMA over 20 years ago, there is no anticipated rutting potential at any depth throughout the pavement structure, and there is also increased confidence in the durability and ultimately the longevity of this SMA mix.

Material selection, mixture design, production modifications and quality control, construction techniques and the extra effort put forth by Gallagher Asphalt and the Illinois Department of Transportation all played a role in the impressive success of this Stone Matrix Asphalt pavement. The Williams and Margaret intersection project ultimately showed that an SMA pavement can withstand two decades of extreme loading, and even be considered perpetual, when it is designed, produced, and constructed properly.

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Introduction of Stone Matrix Asphalt to Australian Runways

Greg White
University of the Sunshine Coast
90 Sippy Downs Drive, Sippy Downs, QLD, 4561, Australia
Tel: +61 400 218 048
Fax: +61 5456 2882
Gwhite2@usc.edu.au

Sean Jamieson
University of the Sunshine Coast
90 Sippy Downs Drive, Sippy Downs, QLD, 4561, Australia
Tel: +61 431 181 823
Fax: +61 5456 2882
Sean.jamieson@research.usc.edu.au

Abstract

Flexible airport pavements in Australia, the United Kingdom and the United States have traditionally been surfaced with Marshall-designed dense graded asphalt mixtures. Grooves are generally sawn transversely across the runway to increase aircraft skid resistance. However, grooving is costly, time consuming, increases the complexity associated with surface preservation treatments and the grooves can irreversibly close under aircraft traffic. Consequently, Australian airports seek an alternate asphalt mixture that meets international regulatory requirements for surface texture and skid resistance without the need to groove. Such an alternate must also provide performance and durability comparable to, or better than, dense graded airport asphalt. This paper outlines the structural and functional performance requirements of airport asphalt, focusing on surface texture and friction, and presents the benefits of ungrooved stone mastic asphalt as an alternate to grooved dense graded asphalt. The current efforts to introduce stone mastic as an ungrooved runway surface in Australia is detailed and it is recommended the United States and the United Kingdom also consider this approach to runway surfacing in the future.

INTRODUCTION

Australia, the United States of America (USA) and the United Kingdom (UK) have traditionally surfaced their runways with Marshall designed, dense graded asphalt (DGA) (Figure 1). DGA for airports is not dissimilar to road DGA but the binder content is relatively high (typically 5.4-5.8% by mass) and polymer modified binders are normally used (AAA 2017). Because these mixtures generally only achieve 0.4–0.6 mm surface texture, grooves are commonly sawn into the surface (Figure 2) for wet weather aircraft skid resistance. Although grooves are effective at reducing the risk of aircraft skidding incidents, they are expensive and time consuming to install, increase the complexity of applying asphalt preservation treatments (White & Rodway 2014) and are prone to irreversible closure under traffic during hot weather (Figure 3). The risk of groove closure is greatest when aircraft are slow moving, aircraft have high tire pressure and wheel loads (White 2017) and the asphalt surface is hottest.

Figure 1. Typical dense graded asphalt

Figure 2. Runway grooves

Figure 3. Groove closure in an airport asphalt surface

In contrast to Australia, the USA and the UK, many countries in Europe and Asia do not surface runways with grooved DGA. Rather, alternate asphalt mixtures are used and these provide adequate surface texture and skid resistance without the need for grooving. Particularly in Germany and China, stone mastic asphalt (SMA), known in the USA as stone matrix asphalt, is a common (ungrooved) runway surfacing.

This paper presents current efforts to introduce SMA as an ungrooved alternate to grooved DGA for runway surfacing in Australia. The basis of current grooved DGA is presented, as well as alternates to grooved DGA, focusing on SMA. The previous use of SMA as a runway surface is summarized before the performance and specification requirements are outlined and the process for introducing SMA to Australian airports is detailed.

BACKGROUND

Airport asphalt

The design methods for airport asphalt surfaces, like airport pavement structures, was primarily developed in the 1940s and 1950s by the U.S. Army Corps of Engineers (the Corps) (White 1985). Many airports and aviation authorities retain the basis of the Corps methods in current airport asphalt design, specification and construction practice. As a result, the majority of airport asphalt surfaces are constructed from a 50–60 mm thick, 14 mm (nominal maximum size), densely graded and Marshall-designed asphalt mixture (Rodway 2016).

The Marshall mixture design method was developed in 1939 by Bruce Marshall for the Mississippi Highways Department (White 1985). During the 1940s and 1950s, it was adapted and adjusted for significant military aircraft loads. Many countries, including Australia, predominantly retain Marshall-designed asphalt for airport surfacing. The primary aims of the Marshall design method include (White 1985):

- A densely graded aggregate skeleton with 15% voids in the mineral aggregate.
- Filling the voids in the mineral aggregate with bituminous binder to retain 4% air voids.
- The Marshall Stability and Marshall Flow values are determined over a range of binder contents and the optimal binder content selected.

Grading, bitumen content, compacted density, Marshall Flow and Marshall Stability are the primary design criteria and quality assurance parameters. In practice, Marshall-designed dense graded airport asphalt mixtures generally contain 5.4–5.8% of bitumen (by mass) and 4–6% of aggregate passing the 75 µm sieve (by volume). The result is around 16% of airport asphalt volume being comprised of mastic (combination of bitumen and very fine aggregate). The high bitumen content results in the deformation resistance of Marshall-designed asphalt being highly influenced by the engineering properties of the binder (Emery 2005) as a critical element of the bituminous mastic that glues the coarse aggregate particles together.

Runway friction and texture

The physical characteristics of runways are specified by the International Civil Aviation Organization (ICAO) through a document commonly known as Annex 14 (ICAO 2013). According to Annex 14, an airport asphalt surface must be:

- Constructed and maintained without deviations or bumps that would adversely affect aircraft operation. In practice this requires asphalt that is resistant to deformation by shearing, shoving and rutting, so as to remain smooth and free-draining.

- Constructed and maintained to provide good friction characteristics when the surface is wet. In practice this requires either an asphalt surface with naturally not less than 1 mm surface texture or grooving of the asphalt surface.
- Maintained free of loose material on the surface to prevent damage to aircraft. In practice this requires airport asphalt to be durable and resistant to raveling.

Surface friction and texture are complex because skid resistance is provided by the aggregate properties and the asphalt mixture volumetrics, as well as any treatments, such as grooving. As the surface ages, the texture generally increases. However, the grooves can also be impacted by groove closure, rubber contamination from landing aircraft (Zuzelo 2014; Gagnon 2016) and preservation treatments applied to prevent surface raveling (Emery et al. 2011; White & Thompson 2016). Removal of rubber contamination by mechanical treatment also impacts skid resistance over the asphalt life by increasing surface texture.

Maintenance activities aside, an asphalt surface must provide an adequate level of surface friction and this must be maintained over the life of the surface. The surface friction must also remain above a minimum level when the surface is wet during rain events (Yager 2014). The use of fully crushed coarse aggregate generally provides adequate dry friction characteristics. The wet weather skid resistance is generally maintained by allowing paths for the water film to escape from under aircraft tires (Gagnon 2016). This can be achieved either by highly textured asphalt mixtures or grooving the tight surface of a dense graded mixture.

Alternates to grooved dense graded asphalt

Grooving of dense graded Marshall-designed airport asphalt is costly and introduces the risk of groove closure. Groove closure is likely the most difficult mode of airport asphalt distress to avoid in hot climates, due to the high reliance of airport asphalt on the deformation resistance of the mastic. Moreover, grooves cut through the coarse aggregate particles reduce the contribution of the coarse aggregate skeleton to deformation resistance. Furthermore, in wet climates, the delay between surfacing and grooving, when the surface has less skid resistance than after grooving, presents a skidding risk to aircraft safety. It follows that a range of alternate airport asphalt surfaces have been considered. These alternate mixtures aim to provide the surface texture and skid resistance levels recommended by ICAO and required by many regulators, without requiring grooving of the surface, and include (Figure 4):

- Open graded friction course (OGFC)
- Gap graded asphalt
- SMA

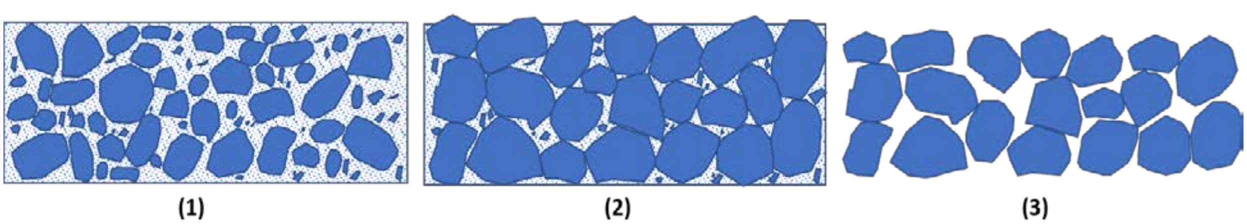

Figure 4. Representative mix drawing of (1) DGA, (2) SMA, and (3) OGFC

OGFC has a high air voids content and is internally free draining, but oxidizes rapidly, can become clogged with detritus and has a shorter life expectancy than other asphalt mixtures (White 2018). The benefit of OGFC for improved aircraft skid resistance on runways was first recognized in the 1970s (Jones 1973; White 1976) and the UK used OGFC extensively for runway surfacing for over 40 years (EAPA 2003). However, the shorter life expectancy has resulted in a general return to grooved DGA in the UK.

Of the gap graded mixtures, the French Béton Bitumeux Aéronautique (BBA) is most common and has been a normal runway surface in France for over 30 years (Widyatmoko et al. 2011). It has also been used on some UK runways in recent years. BBA is specified in different maximum aggregates sized and both a dense and a gap graded option. When a gap graded and 14 mm sized mixture is selected, the surface texture is generally 1.2–1.3 mm immediately following construction, and grooving of the surface is avoided (Hakim et al. 2014).

SMA was developed in Germany in the 1970s for resistance to damage from studded snow tires. Associated benefits regarding durability and resistance to shearing were quickly recognized (Nunn 1994). SMA is discussed in more detail below.

Stone Mastic Asphalt

The coarse aggregate in SMA is gap graded, but the voids are filled with mastic, rather than air. The result is a stone-on-stone matrix, with a bitumen-rich mastic mortar (Campbell 1999). The high bitumen content provides durability, while the stone-on-stone matrix provides a deformation resistant surface (Qiu & Lum 2006). SMA subsequently became a popular road surface in high traffic applications across Europe, the USA, Asia, South Africa and Australia (Campbell 1999; Nunn 1994). Large maximum aggregate size SMA mixtures routinely achieve 1.2–1.3 mm surface texture, making SMA attractive as an ungrooved runway surface for its combination of adequate high surface texture, as well as providing a durable and stress resistant surface.

USE OF STONE MASTIC ASPHALT IN AIRCRAFT PAVEMENTS

Detailed reviews of SMA as a runway surface have been undertaken by Campbell (1999) and Prowell et al. (2009). The latter determined that SMA performance, in comparison to a typical DGA airport mixtures, is either similar or better, based on laboratory testing, literary review and analysis of in-service airfields across Europe, China and the limited use in the USA.

Europe

Norway has used SMA as a runway surface extensively, with 15 runways resurfaced with SMA between 1992 and 1998 (Campbell 1999). SMA 11 (11 mm maximum aggregate size) and SMA 16 were used, with another runway surface constructed in 2002 using a smaller SMA 8 (Prowell et al. 2009). The practice of SMA as a runway surface in Norway continues with Oslo airport, Norway's main international airport, having its western runway overlaid with polymer modified SMA 11 in 2015 (Jacobsen 2015).

Germany uses DGA or heavy duty SMA on international runways. When SMA is used, SMA 11 is usually selected (Beer et al. 2012). Both Hamburg Airport and Spangdahlem USAF Base used the German SMA 11 for runway surfacing, in 2001 and 2007, respectively (Prowell et al. 2009).

Other European countries that have used SMA as a runway surface include Italy, Austria, Denmark, Belgium and Sweden. Belgium and Sweden have reported good performance of SMA runway surfaces, however, have had issues relating to the application of deicer, and have since reverted back to traditional DGA for airport runways (Campbell 1999; Prowell et al. 2009).

China

SMA was introduced into China in 1992, with the Civil Aviation Administration of China (CAAC) introducing specifications in 1997 (Xin 2015). Since then, China has become a leader in airport SMA with over 40 airports using the material as a runway surface, including Beijing International Airport (CACC 2016). China uses either an SMA 16 or SMA 13 mixtures, with each reporting minimum texture depths of 1.2 mm and 1.0 mm, respectively. China continues to use SMA for improved durability, low maintenance, improved skid resistance and lower life-cycle cost compared to DGA (Prowell et al. 2009).

United States

The USA conducted SMA field tests on a large taxiway at Indianapolis Airport in 2005, using a mix with a 12.5 mm maximum aggregate size. Friction results were reportedly better than other surfaces at the airport (Prowell et al. 2009). SMA was also used for the construction of a runway at a USAF Base near Aviano, Italy in 1999. As of 2006 no maintenance (except for rubber removal) had been undertaken and the runway was still demonstrating no issues related to performance Prowell et al. 2009. Despite these experiences, runways in the USA are primarily surfaced with grooved DGA or are concrete pavements (White 2018).

Australia

The use of SMA in Australia has been limited to roads and only two airports. Cairns and Sydney airports both used SMA on pavements, but neither used it on a runway. In 1998 and 1999, three trial sections on aprons at Cairns Airport were overlaid with SMA 10 and SMA 14 mixtures, generally based on the Queensland roads specification (Campbell 1999). The pavement required minimal maintenance up until 2007, excluding the first four to six weeks, when sweeping was required after each aircraft movement to remove loose stones that were generated from aircraft tires. Prowell et al. (2009) theorized that the loose stone generation could have been avoided if a stiffer binder was used. In 2005, the entire international apron was resurfaced with SMA 12, this time with stiffer acid-modified (multigrade) binder. The surface performed well and remains in service at the time of writing (2018).

Sydney airport performed an SMA trial on one taxiway in 1999, using a design similar to the Chinese SMA 13. The trial was unsuccessful with 20-30% of the surface exhibiting a coarse, uneven and poor finish (Campbell 1999). Raveling was evident on the taxiway and to prevent further generation of loose stones, a surface treatment was placed on the section in 2004. The poor surface finish is likely to be the results of construction issues, including plant difficulties, operational delays and low mix temperatures (Campbell 1999; Prowell et al. 2009).

PERFORMANCE AND SPECIFICATION

In 2017, Australia developed performance requirements for DGA used as an airport pavement surface (Table 1). A performance-based specification was subsequently developed and published in 2018 (AAPA 2018). The specification retains the traditional DGA mixture volumetrics and composition, but allows the mixture designer to select or develop a binder to achieve the asphalt performance properties (Table 2).

Table 1. Summary of airport asphalt performance requirements (White 2018)

Physical requirement	Protects against	Level of importance
Deformation resistance	Groove closure Rutting Shearing/shoving	High
Fracture resistance	Top down cracking Fatigue cracking	Moderate
Surface friction and texture	Skid resistance Compliance requirement	High
Durability	Pavement generated FOD Resistance to moisture damage	Moderate

Table 2. Performance-based airport asphalt requirements

Test Property	Test Method	Requirement
Indirect Tensile Strength Ratio (TSR)	AG:PT/T232	Not less than 80%
Wheel Tracking Test (Final rut depth after 10,000 cycles at 65°C)	AG:PT/T231	Not more than 2.0 mm
Fatigue life (at 20°C and 200 µm — three beams only)	AG:PT/T274	Not less than 500,000 cycles to 50% of initial flexural stiffness

For SMA to be used as an ungrooved runways surface, it must satisfy the Australian asphalt performance requirements, as well as reliably achieve a minimum 1 mm surface texture. This must be tested during the mixture design phase and verified during construction. Finally, as SMA has a higher binder content, it is prone to binder drain off during production, transport and paving. Consequently, an SMA specification must also have a requirement for minimum binder drain-off. SMA mixtures usually include stabilizers, or drainage inhibitors, commonly in the form of cellulose fibers, although other materials such as glass, polyester and mineral fibers can also be used (Wan et al. 2014). Although some jurisdictions require a minimum drainage inhibitor content, usually greater than 0.3%, others prefer a limit on a laboratory test for the amount of binder draining out during controlled conditions, commonly known as the Schellenberg test (Druschner & Schafer 2005). For a performance-based approach to asphalt mixture design, a drain off test is more appropriate than a prescriptive minimum stabilizing additive content.

INTRODUCTION TO AUSTRALIAN AIRPORTS

Australian airports clearly desire ungrooved runway surfaces, where compliance to international and local requirements are still otherwise achieved. The process for introducing SMA to Australian airports is based on a three-phase translation and validation of overseas airport practice, as well as Australian road pavement practice.

Firstly, a review of international literature identified that the most established SMA specifications for runway surfaces were Chinese and German. The specifications were compared to each other and to Australian road SMA specifications. In parallel, the Australian performance requirements and performance based DGA specification for airport asphalt were reviewed. From this, a draft specification requirement was prepared and included:

- Volumetrics consistent with German and Chinese airport SMA specifications.
- Performance test (deformation, fracture and moisture damage resistance) retained from the Australian performance based airport DGA specification (Table 2).
- Additional requirements added for surface texture and for binder drain-off, as outlined above.

The four main Australian airport asphalt contractors were each asked to prepare a mixture design to the proposed specification requirements, each using a different aggregate type, but all using the same source of typical Australian polymer modified binder. The surface texture results and the laboratory performance testing results were used to demonstrate the suitability of the draft specifications requirements.

The final phase of introduction included a field trial. A runway resurfacing project was identified and the contractor was required to perform an SMA mixture design to the specification requirements confirmed by the laboratory testing phase. Within the resurfacing project, one shift of the work was performed with the approved SMA mixture design. Importantly, the field trial was located in an area of pavement that was deemed non-critical, relatively easy to access for condition monitoring over time, but also allowing a 95 km/hr continuous friction measurement survey. The field trial confirmed the constructability and compactability of the SMA, as well as the achieved surface texture compared to the texture measured in the laboratory.

Based on the good performance of the field trial, the specification draft was finalized for industry feedback and preliminary publication. The performance of the field trial will be monitored, relative to the DGA runway surface constructed as part of the same project.

SUMMARY AND CONCLUSIONS

Avoiding grooving by providing an asphalt mixture for runway surfacing that provides adequate friction and surface texture is beneficial to airports in Australia, as well as in the UK and the USA. Although other mixtures are also viable, stone mastic is the preferred approach and Australia is currently validating its performance and will introduce a performance-based specification in 2019. The specification reflects the volumetric and compositional requirements of the German and Chinese airport SMA specifications and the Australian performance-based airport DGA performance requirements. It is important to Australian airports that SMA performs similarly or better than DGA and exceeds the 1 mm minimum surface texture required by regulators. Is it recommended that the USA and the UK also consider ungrooved SMA as an alternate to grooved DGA in the future.

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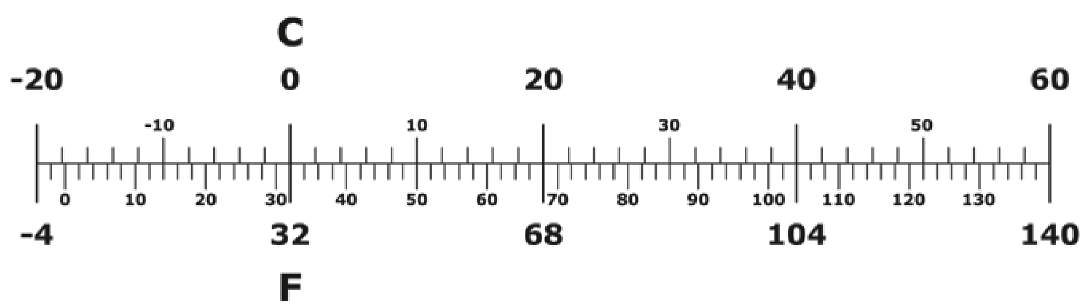
SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSION TO SI UNITS

Symbol	When You Know	Multiply by	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	645.2	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 L should be shown in m^3				
MASS				
oz	ounces	28.35	grams	g
lbs	pounds	0.454	kilograms	kg
T	short tons	0.907	megagrams	Mg
T	short tons	0.907	metric tonnes	t
NOTE: A short ton is equal to 2,000 lbs				
TEMPERATURE (exact)				
°F	Fahrenheit	$\frac{5(F-32)}{9}$	Celsius	°C

APPROXIMATE CONVERSION FROM SI UNITS

Symbol	When You Know	Multiply by	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.196	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.315	cubic feet	ft^3
m^3	cubic meters	1.308	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lbs
Mg	megagrams	1.102	short tons	T
t	metric tonnes	1.102	short tons	T
NOTE: A short ton is equal to 2,000 lbs				
TEMPERATURE (exact)				
°C	Celsius	(1.8×C)+32	Fahrenheit	°F



*SI is the symbol for the International System of Units

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National Asphalt Pavement Association
NAPA Building
5100 Forbes Blvd.
Lanham, Maryland 20706-4407
www.AsphaltPavement.org
NAPA@AsphaltPavement.org
Tel: 301-731-4748
Fax: 301-731-4621
Toll Free: 1-888-468-6499

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