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RESEARCH REPORT

DESIGN AND EVALUATION OF FOAMED ASPHALT BASE MATERIALS

Charles W. Schwartz and Sadaf Khosravifar

UNIVERSITY OF MARYLAND
COLLEGE PARK

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16. Abstract

Foamed asphalt stabilized base (FASB) combines reclaimed asphalt pavement (RAP), recycled concrete (RC), and/or graded aggregate base (GAB) with a foamed asphalt binder to produce a partially stabilized base material. The objectives of this study are to determine for Maryland conditions: (a) FASB mix design procedures; (b) typical engineering and structural design properties; (c) guidelines for FASB production and placement; and (d) best practices for construction quality assurance. The study consisted of three major components: (a) laboratory evaluation of FASB mix design procedures; (b) field evaluation of in-place FASB stiffness, and especially the increase in this stiffness with time due to curing; and (c) direct laboratory measurement of the performance related properties of FASB, specifically dynamic modulus and permanent deformation resistance. Recommended default design values for resilient modulus and structural layer coefficient are 300 – 400 ksi and 0.30 – 0.35, respectively. The study findings also provided guidance for the development of draft specifications.

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EXECUTIVE SUMMARY

Foamed asphalt stabilized base (FASB) combines combinations of reclaimed asphalt pavement (RAP), recycled concrete (RC), and/or graded aggregate base (GAB) with a foamed asphalt binder to produce a partially stabilized base material. The foamed asphalt content is usually too low to fully coat all of the aggregate particles, as is the case in hot mix asphalt (HMA). Instead, the foamed asphalt coats only the fine aggregates which then form localized FASB "spotweld" bonds between the coarser aggregate particles (Wirtgen, 2010) that lead to increased cohesion and stiffness of the aggregate assemblage.

As compared to other recycled road materials improvement methods such as asphalt emulsion and Portland cement stabilization, foamed asphalt treatment has shown significantly better performance as reported by researchers. FASB holds the potential to incorporate significant quantities of recycled materials into Maryland State Highway Administration (SHA) projects. FASB can be used either as a replacement for base HMA layers or for GAB. FASB may reduce the cost of conventional flexible pavements.

A thorough understanding of all technical issues related to design, production, and construction of FASB is required to ensure proper use and performance of the material on SHA projects. Specific questions that must be addressed include the following:

- What are appropriate FASB mix designs for Maryland materials and conditions?
- What are typical engineering/design properties of FASB mixes for Maryland conditions?
- What are appropriate guidelines for FASB production and placement?
- What are best practices for construction quality assurance (QA)?
- What are the economic advantages of FASB?

All of these questions are addressed in this report.

FASB mix designs were developed for eight different mixtures of RAP/RC/GAB of interest in Maryland. An SHA provision specification (Appendix A) for FASB course was developed by the investigators early in this study to provide a starting point for the mix design study and to guide construction of the field evaluation sections. Five different aggregates, three PG 64-22 binders, and Portland cement additives were investigated. The moist aggregates were mixed with varying percentages of foamed asphalt in the laboratory, compacted into 4 inch Marshall specimens, oven cured, and then tested for indirect tensile strength (IDT strength) in both dry and soaked conditions. Several ancillary studies of details of the mix design procedure were also performed. Principal findings from the mix design study are as follows:

- The Maryland SHA provisional specification for FASB mix designs generally agrees with most of the requirements from other agencies.
- The gradation requirements should be revised to require 100% passing the 1.5 inch sieve. This reduction of the coarsest aggregates will have no impact on the mix designs from the current Maryland producers.

- Adding Portland cement adds cementitious bonds and increases unsoaked and soaked IDT strength values. This increase was greater for the 100% RAP (Mix G) than for the 40%RAP+60%RC (Mix A) material.
- For many of the mixes, the IDT strength vs. asphalt content curves did not exhibit the expected concave downward shape. This makes it difficult to determine an optimum binder content. In these cases, it is suggested that the lowest foamed asphalt content that passes all specification requirements be used, but in no case should the foamed asphalt content be less than 2%. An upper bound of 3.5% for foamed asphalt content is also suggested to eliminate any potential for mix instability and rutting potential.
- The soaked IDT strength requirement of 50 psi in the Maryland provisional specification is at the upper limit of other states' practices and is substantially higher than the new FHWA-approved specification in Virginia (minimum dry strength of 45 psi and minimum TSR of 70% corresponding to a minimum soaked strength of 31.5 psi). It is recommended that the Maryland soaked IDT strength requirement be reduced slightly to 45 or 40 psi and that the minimum TSR be kept at 70%. This will still conform to practice in most other states and be achievable by Maryland producers. It will also still produce FASB mixtures with satisfactory structural properties.

Other findings from the mix design studies include:

- 24 hour soaking was more effective than low vacuum saturation but slightly less effective than 72 hour soaking. 24 hour soaking also has practical advantages over both vacuum saturation and 72 hour soaking. For all of these reasons, 24 hour soaking is recommended for mix design testing.
- The mixing moisture content was not found to have a significant impact on asphalt dispersion in the low fines mixtures typical for Maryland producers. The tensile strength is mainly affected by the conventional moisture-density behavior of the granular material. Consequently it is recommended that the mixing moisture content be near the optimum moisture content for compaction.
- A limited laboratory evaluation suggests that stockpiling FASB for even just a few days after production may cause a reduction of soaked IDT strength by up to 30%. More work should be done to confirm this preliminary finding. In the interim, stockpiling should therefore either be strictly curtailed in the specification or, alternatively, mixtures that will be stockpiled should be required to satisfy a higher soaked IDT strength requirement during mix design.

A major objective of this study was to evaluate the in-place properties of FASB materials, and in particular the increase in in-place stiffness gain during curing. Several attempts to do this during the early stages of the project failed for a variety of reasons. However, a successful field evaluation at the MD 295 lane addition project near BWI airport provided a wealth of high quality data. Field tests included GeoGauge, lightweight deflectometer (LWD), falling weight deflectometer (FWD), nuclear density gauge, and field moisture content testing. Principal findings regarding the stiffness gains of FASB during curing are as follows:

- The Zorn LWD and GeoGauge devices gave significantly different values for the apparent *in-situ* FASB stiffness, with the Zorn LWD systematically reporting values approximately 0.5 times those from the GeoGauge at the same locations. The reasons for these differences include different load levels, loading rates, depth of zones of influence, analysis assumptions, and other factors. Given these issues, neither of the devices can be considered to give the "true" in place stiffness. The more useful measures are the percentage increase in stiffness with time and the relative stiffness of the FASB sections versus the conventional GAB section.
- Curing of the FASB in the Control Strip and mainline Segment B placement produced stiffness increases of 188 to 234% within one week after placement as measured by the GeoGauge and Zorn LWD respectively. The stiffness increases measured using the Zorn LWD tended to be slightly higher than those measured using the GeoGauge. The initial stiffness of the FASB sections (excluding Segment A) was on average 1.4 (GeoGauge) to 2 (Zorn LWD) times the equivalent GAB sections. The gain in stiffness after one week of drying and curing of the FASB sections was also greater than the corresponding stiffness gains for the GAB; the FASB sections (excluding Segment A) increased in stiffness by a factor of 2.9 (GeoGauge) to 3.3 (Zorn LWD) while the GAB increased by a factor of 2.1 (GeoGauge) to 2.9 (Zorn LWD).
- FWD testing conducted 4 to 6 months after paving and just prior to opening to traffic found that the FASB became significantly stiffer than the final GeoGauge and LWD measurements seven days after placement. The long-term stiffness of the field-cured FASB as measured using the FWD was about 295 ksi as compared to 35 to 70 ksi as measured using the LWD and GeoGauge, respectively. The long-term stiffness of the GAB backcalculated from the FWD results was about 24 ksi; the corresponding stiffness of the FASB was 12.3 times that of the GAB. Placement of the HMA layer may have improved the curing of the underlying FASB by applying additional heat and enhancing moisture evaporation.

Other findings during the field study, especially with regard to appropriate construction and QA practices include:

- In situ stiffness test devices are not yet sufficiently mature for use in construction QA of FASB (or GAB) layers. The effects of load level, loading rate, depth of zone of influence, analysis assumptions, and other factors must be better understood. These factors affect both the magnitude of the measured stiffness and the trends of stiffness increase with time. Consequently, nuclear density gauge testing remains the best practical approach for construction QA at the present time.
- FASB materials are inherently variable. Consequently, nuclear density gauge results are also variable. Sufficient Proctor compaction testing should be performed in the laboratory prior to construction to enable one-point compaction testing in the field at the beginning of each day of construction as a check on nuclear density gauge readings.
- The bitumen in the RAP and foamed asphalt causes bias in the moisture content reported by nuclear density gauges. Field moisture content should be evaluated

independently in order to determine the appropriate moisture offset for input to the nuclear gauge. Sending specimens to the laboratory for moisture content measurement is usually not practical during construction. Moisture content can be determined in the field using microwave drying (ASTM D 4643), but care must be taken to avoid high temperatures in the mixture that may burn off some of the bitumen; appropriate microwave power settings and heating time must be determined via trial and error.

- Breaking the installation of FASB layers into two separate days affected the final stiffness of the material even with rewetting of the surface before placement of the second lift. All lifts should be placed in a single day.
- FWD testing on subgrades and granular base layers is pointless unless the standard 12 inch diameter load plate is replaced with a larger plate and/or the loads are substantially reduced. The high stresses under the 12 inch diameter plate cause excessive plastic deformations in the unbound materials.
- Laboratory and field permeability tests found that the permeability of FASB is comparable to and in some cases slightly higher than that of GAB.

Post-construction laboratory testing was conducted on field cores of FASB material from two sites. The emphasis of this testing was on the material properties relevant to pavement structural design, specifically the stiffness and permanent deformation characteristics of the material. Dynamic modulus tests (AASHTO TP 62-07) and repeated load permanent deformation tests (NCHRP 9-30A protocols) were conducted on 7 field cores of FASB taken from the MD 295 test site (M cores), 6 field cores taken from the P. Flanigan and Sons demonstration site (F cores), and 3 laboratory compacted and cured specimens of the 100%RAP FASB used in the I-81 reconstruction (I cores). In addition, 4 sets of tests were conducted on HMA cores taken from MD 295.

At a 77 °F temperature and a 10 Hz loading rate typical for base layer conditions in high volume highway pavements, the mean value of dynamic modulus ± one standard deviation for the M and F cores was 629 ±134 and 462 ±145, respectively. The influence of confining stress on dynamic modulus was slight compared to the influence of loading rate and temperature. The lower dynamic modulus limit measured for the FASB is substantially greater than the typical 25 ksi design modulus for GAB material and the upper limit of the FASB dynamic modulus is close to the lower bound of HMA at this temperature and loading rate (e.g., a mean modulus of 851 ksi for the HMA cores from MD 295). The effects of improper construction were observed in the measured dynamic modulus from field cores from the MD 295 site. Field cores from Segment A where the material was placed in two lifts at different times (4 days apart) exhibited lower dynamic moduli and higher permanent deformation as compared to Segment B and the Control Strip where the FASB material was placed on the same day.

None of the M or F cores entered the tertiary stage of permanent deformation. The laboratory prepared I specimens gave significantly higher permanent deformations, but it is believed that tests on laboratory prepared specimens significantly underestimate the rutting resistance of FASB. The permanent deformation resistance of FASB cores from both mixtures was found satisfactory as compared to HMA. This is especially true given that FASB in thick pavement sections will experience lower stress levels in the field as it is placed deeper in the pavement structure than the HMA layer. However, it is important that the mix designs and field placement conform to specifications and that the foamed asphalt content be limited to avoid mix instability.

Appropriate structural property values for FASB must be defined if these materials are to be used rationally in pavement design. The required structural properties vary by pavement design methodology. For the MEPDG, the relevant property for asphaltic materials like FASB is the dynamic modulus E* as a function of temperature and loading rate. For the empirical 1993 AASHTO design procedure, the relevant property is the structural layer coefficient, which in turn is a function of material stiffness. Although not explicitly required by either design procedure, the permanent deformation (rutting) resistance is also important.

The present study has compiled a wealth of information that is directly relevant to the estimation of structural material properties for FASB materials relevant to Maryland. Sources of information on the stiffness characteristics of FASB include the following:

- Laboratory E* testing performed by the University of Maryland as part of this study
- Laboratory E* testing performed by others on the same materials considered in the present study
- Laboratory resilient modulus M_R testing performed by others on materials similar to those in the present study
- Field modulus estimates backcalculated from FWD data
- Modulus estimates from standard empirical correlations with mix design test values (e.g., IDT strength)

The empirical 1993 AASHTO design procedure requires that stiffness be converted to an equivalent structural layer coefficient value. Because FASB exhibits behavior sharing characteristics of both HMA and GAB, there are consequently several approaches for estimating the structural layer coefficient values:

- The 1993 AASHTO relationship for a_1 vs. M_R for HMA
- The 1993 AASHTO relationship for a_2 vs. M_R for non-stabilized base materials
- The 1993 AASHTO relationship for a_2 vs. M_R for bituminous stabilized base materials
- The Wirtgen (2012) empirical relationships for structural layer coefficient

Some of these approaches will provide higher quality estimates than others. In all cases, though, there will be a range of resilient modulus and layer coefficient values and thus some judgment will be required to determine appropriate values for design. Given these caveats, the recommended default structural properties are as follows:

- 50 psi soaked IDT strength specification limit: M_R = 400 ksi, a_2 = 0.35
- 40 psi soaked IDT stregnth specification limit: M_R = 300 ksi, a_2 = 0.30

Design values for an intermediate 45 psi IDT strength specification limit can be reasonably interpolated from the above. It is clear from these evaluations that, even with a lowered IDT strength specification limit, FASB has structural properties approaching those of an HMA base mixture.

The findings from this study clearly confirm the suitability of FASB material for high volume pavement applications if designed and installed properly and cured under favorable climatic condition. The final in place structural capacity of this flexible, partially bound material is substantially higher than unbound GAB and approaches that of base HMA mixtures. Proper use of FASB can reduce pavement cost and help Maryland SHA meet its recycled materials goals.

CHAPTER 1. STUDY OBJECTIVES

BACKGROUND

Foamed asphalt stabilized base (FASB) combines combinations of reclaimed asphalt pavement (RAP), recycled concrete (RC), and/or graded aggregate base (GAB) with a foamed asphalt binder to produce a partially stabilized base material. The foamed asphalt content is usually too low to fully coat all of the aggregate particles, as is the case in hot mix asphalt (HMA). Instead, the foamed asphalt coats only the fine aggregates which then form localized FASB "spotweld" bonds between the coarser aggregate particles (Wirtgen, 2010) that lead to increased cohesion and stiffness of the aggregate assemblage.

The original foaming process developed by Csanyi (1957) for full-depth reclamation projects injected steam into hot asphalt through a specially designed nozzle. This reduced the viscosity and surface energy in the foamed asphalt to enable intimate coating when mixed with wet aggregate at its ambient temperature. In 1980s, Mobil Oil Australia made the process more practical for field applications by replacing steam with pressurized cold water. A controlled flow of cold water and pressurized air is introduced into a hot asphalt stream in a mixing chamber and then delivered through a nozzle as asphalt foam.

Foamed asphalt stabilization can be performed either through cold in-place recycling (CIR) of HMA or via cold central plant recycling (CCPR). FASB holds the potential to incorporate significant quantities of recycled materials into paving projects. Its structural properties are expected to fall somewhere between conventional untreated GAB and HMA. As an added benefit, FASB has the potential for reducing the cost of conventional flexible paving.

Foamed asphalt stabilization of recycled materials (mostly RAP) with/without virgin aggregate has gained great attention worldwide. It has been implemented over the past several decades in South Africa (Jenkins *et al.*, 2000; Asphalt Academy, 2002; Long and Theyse, 2004; Saleh, 2004; Jenkins *et al.*, 2007), Australia (Ramanujam and Jones, 2007), Europe (Schimmoller *et al.*, 2000; Nunn and Thom, 2002; Loizos *et al.*, 2004; Loizos, 2007; Khweir, 2007), and more recently and to a lesser extent in the U.S. (Marquis *et al.*, 2003; Mohammad *et al.*, 2003, 2006; Romanaschi *et al.*, 2004; Kim and Lee, 2006; Kim *et al.*, 2007; and Fu *et al.*, 2008).

As compared to other recycled road materials improvement methods such as asphalt emulsion and Portland cement stabilization, foamed asphalt treatment has shown significantly better performance as reported by researchers. Ramanujam and Jones (2007) reported a direct comparison between foamed asphalt (with lime) treatment and emulsion treatment (with Portland cement) in which the foamed asphalt section showed significantly better performance in terms of handling early traffic and also superior rain resistance before placement of the wearing course. Compared to recycled road base materials treated with Portland cement or other cementitious agents, foamed asphalt mixes (which may include small amounts of Portland cement as well in the form of active fines) have the additional benefit of improved flexibility and reduced brittleness. Jenkins *et al.* (2000), on the other hand, found that foamed asphalt and asphalt emulsion stabilized mixes have comparable strength, stiffness, and moisture susceptibility. However, foamed asphalt stabilization is often preferred because the asphalt emulsion treatment introduces extra moisture into the mix and requires considerably longer curing periods before the road can be opened to traffic. Muthen (1999) demonstrated that foamed asphalt treated materials exhibit higher stiffness in comparison to emulsion treated materials at ambient temperature and the can resist higher strains before failure.

STUDY OBJECTIVES

FASB holds the potential to incorporate significant quantities of recycled materials into Maryland State Highway Administration (SHA) projects. FASB can be used either as a replacement for base HMA layers or for GAB. FASB may reduce the cost of conventional flexible pavements. However, much of current experience with FASB materials is from regions that have quite different native materials, design standards, and traffic conditions than in Maryland. In addition, the climate conditions in these regions are often significantly different from those in Maryland with respect to high/low temperatures, precipitation/moisture, and freeze/thaw cycles. Consequently, SHA is interested in evaluating the suitability of FASB for Maryland paving conditions.

Pavement material producers in Maryland experienced in the production of FASB for commercial clients are promoting its application to SHA projects. A thorough understanding of all technical issues related to design, production, and construction of FASB is required to ensure proper use and performance of the material on SHA projects. Specific questions that must be addressed include the following:

- What are appropriate FASB mix designs for Maryland materials and conditions?
- What are typical engineering/design properties of FASB mixes for Maryland conditions?
- What are appropriate guidelines for FASB production and placement?
- What are best practices for construction quality assurance (QA)?
- What are the economic advantages of FASB?

All of these questions are addressed in this report.

REPORT ORGANIZATION

Chapter 2 of this report summarizes recommended mix design practices for FASB. These include aggregates and binders material requirements, mixing and curing procedures, and mixture design tests. Chapter 3 discusses field evaluation, including QA practices during placement, lightweight deflectometer and GeoGauge testing to assess stiffness increases during short-term curing immediately after placement, and falling weight deflectometer testing to quantify long-term stiffness gains. Field test data collected during the MD 295 lane addition project are included here. Chapter 4 summarizes post-construction laboratory dynamic modulus and flow number testing of the FASB and HMA materials at the MD 295 project. Chapter 5 proposes appropriate FASB properties for use in pavement structural design. Chapter 6 summarizes the overall conclusions from this study. Appendices A through F provide extensive technical detail supporting the findings and conclusions from the study.

CHAPTER 2. MIX DESIGNS

INTRODUCTION

Foamed asphalt stabilization provides a potentially fast, cost-effective, and environmentally friendly flexible pavement construction strategy if designed and produced effectively. Several FASB mix design procedures already exist, e.g., ARRA (2001), Asphalt Academy (2002), Mohammad *et al.* (2003), Kim and Lee (2006), Wirtgen (2010), and others. Most of the methods are based on Marshall compaction and a combination of Marshall stability and indirect tensile (IDT) strength under wet vs. dry conditions. It is important to note that several of these design procedures were developed in geographic regions that have quite different climate conditions than in Maryland with respect to high/low temperatures, precipitation/moisture, and freeze/thaw cycles. In addition, the native materials, design standards, and trafficking are much different in Maryland than they are in South Africa and much of Europe and even much different from U.S. locations like Louisiana and Iowa where earlier evaluations were conducted. The suitability of existing design procedures for Maryland conditions must therefore be very carefully evaluated.

FASB mix designs were developed for eight different mixtures of RAP/RC/GAB of interest in Maryland. The important factors in mix design procedure from relevant studies in the past provided a starting point for the present study. These include the role and characteristic of each component, e.g., aggregate, active additives, binder, and water in the foamed asphalt stabilization process, and the complexities and considerations with respect to mixing, compaction, curing, soaking, and stockpiling in order to attain a proper FASB mixture.

An SHA provision specification for foamed asphalt stabilized base course was developed by the investigators early in this study to provide a starting point for the mix design study and to guide construction of the field evaluation sections. This provisional specification was a merger of existing specifications from the Federal Highway Administration’s Federal Lands office and from state transportation agencies (Table 1). The full provisional specification is included as Appendix A. Key provisions from this specification governing aggregate gradation requirements and FASB mix requirements are reproduced below as Table 2 and Table 3, respectively. Note that the Maryland SHA provisional specification generally agrees with most of the requirements from other agencies, although the wet tensile strength requirement is at the upper limit of the commonly accepted values. It is also substantially higher than the new FHWA-approved specification in Virginia, which requires a minimum dry strength of 45 psi and minimum TSR of 70% corresponding to a minimum soaked strength of 31.5 psi.

Table 1. Summary of FASB specifications.

				Marshall Compaction					IDT Minimums			Compaction (Modified		
State	Expansion Ratio	Half-Life (sec)	Gradation %<0.075mm	Blows	Stability (lbs)	Gyratory N	Cure	Soak	Dry (psi)	Wet (psi)	TSR (%)	Density (%)	Moisture	Weather
Alaska														Air≥40°F for 24 hrs
Arizona	10	8	5-20	75	1625		104°F to constant mass	77°F for 24 hrs		45	50			Air≥10°C (2°C for 24 hrs), Surface≥2°C
FHWA	15	12		75						50	70	97		
Hawaii				75			104°F to constant mass					100 avg, none <98		≥50°F
Iowa	10	10		75		25	T283	T283		44	50	97		Air≥10°C, Surface≥4°C
Maine		12				25	40°C for 72 hours	77°F for 20 min, 50mm Hg for 45 min, 77°F for 10 min	43			92		
Ohio							140°F for 48 hours	77°F for 20 min, 50mm Hg for 45 min, 77°F for 10 min	43	30	70	100 avg, none <95		Air ≥60°F
Ontario			7-15				60°C for 72 hours			22	50	97		
Minnesota			7-15											
New Mexico	10	8	4-20	75	1625		104°F to constant mass	77°F for 24 hrs		45	50	97		Air>50°F, Surface>40°F, no rain, no temp <36°F expected for 24 hrs
Maryland	10	8	5-15	75			104°F to constant mass	77°F for 20 min, 50mm Hg for 50 min, 77°F for 10 min		50	70	95	OMC±2%	Air>50°F, Surface>40°F, no rain, no temp <36°F expected for 24 hrs
Virginia				75		30	40°C for 72 hours	25°C for 24 hours	45		70	98		Air≥50°F; no freezing temp for 48 hrs

Table 2. Gradation requirements for FASB.

SIEVE SIZE	PERCENT PASSING
2”	100
1 ½”	90-100
¾”	60-100
No. 4	30-70
No. 200	5-15

Table 3. FASB mix requirements.

Design Parameters	Value
Specimen compaction – either: (1) Marshall compaction (AASHTO T 245), number of blows (2) Gyratory compaction (AASHTO T 312), number of gyrations	 75 25
Indirect Tensile Strength (AASHTO T 283; no freeze-thaw cycle) (1) Minimum Wet Tensile Strength, psi (2) Minimum Tensile Strength Ratio (TSR), %	 50 70
Foamed Asphalt Expansion Characteristics @ 160, 170, & 180°C (1) Minimum Half-Life of Foamed Expansion, sec.[(1)] (2) Minimum Expansion Ratio [(2)]	 8 10

(1) Total time for foamed asphalt to settle to half of the maximum foamed volume.
(2) Maximum foamed asphalt volume divided by non-foamed asphalt volume.

MATERIALS

Aggregates

Five different aggregates were used in this study; their general characteristics are described in Table 4. Graded aggregate base (GAB), recycled Portland cement concrete (RC), and recycled asphalt pavement (RAP-1) were provided by P. Flanigan and Sons, Inc. RAP-2 was provided by Global Resource Recyclers, Inc. (GRR) and RAP-3 was provided by the Virginia Center for Transportation Innovation and Research (VCTIR) of the Virginia Department of Transportation. RAP-2 was a processed (crushed at plant) RAP; the others were not. As shown in Table 4, the RC aggregate had a relatively high 4% absorption. Absorption was not a concern for the rest of the test materials.

Figure 1 shows the gradation of the aggregates. The gradation was obtained according to AASHTO T-11 and was monitored during the mix design and testing process to ensure uniformity.

Table 4. Aggregate types for study.

Material	Nominal maximum size	% Fine[1]	Absorption
GAB	1”	7	-
RC	1”	5	4%
RAP-1	¾”	1	-
RAP-2	½”	8	-
RAP-3	½”	1	-

(1) Fine= Particles passing sieve #200

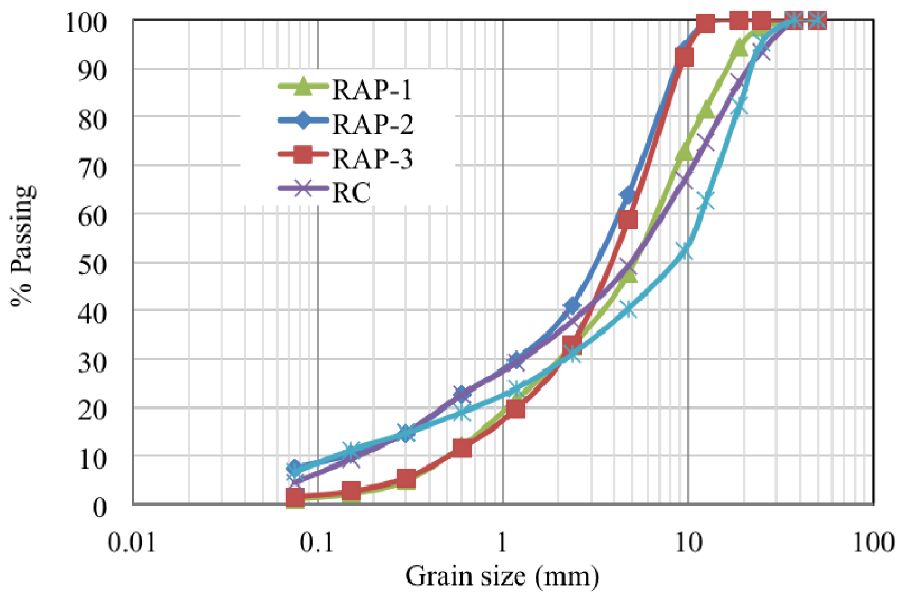


Figure 1. Gradations of aggregates used in the study.

Small amounts of Portland cement were included as active fines in some of the mix designs. Adding 1% of Portland cement is a common practice in FASB design (Wirtgen, 2010). Cement serves several important roles in FASB mixtures.

- Improved foamed asphalt dispersion in the mix. Foamed asphalt coats the fines and makes asphalt mastic. The asphalt mastic forms partial bonds with larger aggregates (Ruckel *et al*., 1983).
- Increased adhesion of the asphalt mastic to the aggregate (Wirtgen, 2010).
- Increased initial rate of strength gain (curing) and the stiffness of the mix. Strong but brittle cementitious bonds usually form faster than the weaker but ductile bonds of foamed asphalt (Fu *et al*., 2008).
- Reduction of moisture susceptibility of FASB (Fu *et al*., 2008)

However, excessive use of cement should be avoided to avoid rigidity and shrinkage cracking of the brittle cementitious bonds (Fu *et al*., 2008). The effect of added cement is evaluated in Chapter 3. Whenever cement was used, it was blended together with the aggregates in the mixer just before including the foamed asphalt.

Binders

Three PG 64-22 binders were used in different parts of the study. The binders included:

- B-1 provided by P. Flanigan and Sons, Inc. This binder was refined by NuStar GP Holdings LLC.
- B-2 provided by GRR. This binder was also refined by NuStar GP Holdings LLC.
- B-3 provided by the VCTIR.

The binder used in FASB mixtures must have adequate foaming characteristics to insure proper foamed asphalt dispersion in the mixture. The best binder for foaming purposes is the one that expands the most and stays foamed as long as possible. These characteristics are quantified in terms of the expansion ratio (ER) and half-life ($t_{1/2}$).

- ER is defined as the ratio of maximum foamed volume to original liquid asphalt volume. Values for ER typically range between 10 to 20.
- $t_{1/2}$ is defined as the time in seconds for the foam volume to dissipate to half of its initial maximum value. Typical values for $t_{1/2}$ range from 6 to 15 seconds.

A minimum ER of 8 and a minimum $t_{1/2}$ of 6 seconds are typical foaming requirements provided in literature (Wirtgen, 2010).

Foaming temperatures typically range from 300ºF to 360ºF and foaming water contents typically range from 2 to 3%. For a given binder, increasing the asphalt temperature and foaming water content generally increases the ER but decreases the $t_{1/2}$ (Wirtgen, 2010). The objective of the binder foaming tests is to determine the temperature and foaming water content that optimizes the foamed asphalt ER and $t_{1/2}$.

Three replicate tests were performed on each binder at different temperatures and foaming water contents to measure the ER and $t_{1/2}$ values according to the Wirtgen Cold Recycling Technology manual (Wirtgen, 2010). The optimum foaming water content was obtained as the average of the foaming water content that met the minimum ER of 8 and the foaming water content meeting the minimum $t_{1/2}$ of 6 seconds. In cases where both the ER and $t_{1/2}$ requirements can be met at multiple temperatures, the lowest temperature is chosen. Figure 2 shows the foaming test results for binder B-1 at 320ºF (160ºC).

The foaming parameters for the three binders in this study are tabulated in Table 5. Binder B-2-a was found to have high expansion with a relatively low half-life that barely met the minimum criterion of 6 seconds. However the expansion ratio was high enough to compensate for the short half-life. This means that by assuming a linear trend for the foamed asphalt collapse the retained volume after 6 seconds is still well beyond the minimum ER of 8 suggested by researchers and design procedures. Fu *et al.* (2011) found that optimizing the foaming parameters (temperature and foaming water content) significantly affected the expansion ratio and half-life; however, the asphalt dispersion and indirect tensile strength of the FASB mixture were not significantly sensitive to small changes in temperature or foaming water content. They suggested that in design practice more effort should be devoted to sourcing a binder with the best foaming characteristics from refineries close to the project site rather than over-emphasizing identification of the "best" foaming parameters for a given asphalt binder (Fu *et al*., 2011).

Additional details of the characteristics of the materials used in this study are provided in Appendix B.

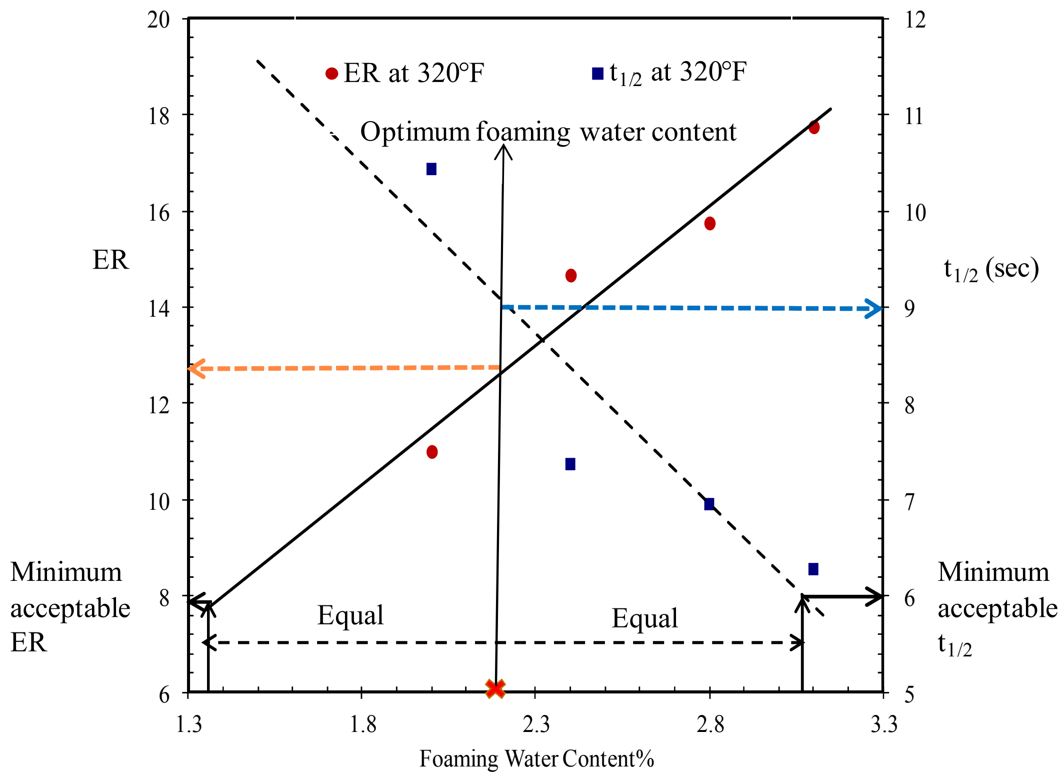


Figure 2. Foaming characteristics of binder B-1 at 320ºF (160ºC).

Table 5. Foaming characteristics of binders in the study.

Binder	Foaming water content (%)	Temperature ºF (ºC)	ER	$t_{1/2}$ (sec)
B-1	2.2%	320ºF (160˚C)	12.8	9.0
B-2	3%	320ºF (160˚C)	18.5	6
B-3	2%	302ºF (150˚C)	26	7.5

MIX DESIGNS

The FASB mix design process consists of the following steps:

- Blending of the aggregates
- Mixing, compaction, and curing at varying foamed asphalt contents

- Indirect Tension (IDT) Strength testing under soaked and unsoaked conditions
- Determination of optimum foamed asphalt content

These steps are described in the following subsections.

Aggregate Blends

Eight different combinations of RAP, RC and GAB were evaluated in this study. Their key properties are summarized in Table 6. The blends were treated with different amounts of foamed asphalt ranging from 2% to 3.5% by weight, the common range for FASB design. The foamed asphalt contents evaluated for each mixture are also listed in Table 6. Binder B-1 was used for Mix A to F, B-2 was used for Mix G, and B-3 was used for Mix H.

Figure 3 shows the gradations of the eight mixtures. All of the mixtures had small percentages of fines, ranging from 1% to 7.5%. Fines are necessary for satisfactory dispersion of the foamed asphalt in the mixture. However, excess fines reduce the permeability and drainage capacity, which is another important criterion for a base course. A limited study on the permeability of a selected FASB mixture (Mix A with 2.8% foamed asphalt and 3.5% passing sieve #200) showed that its coefficient of permeability (3.2E-3 in/sec) is similar to that of GAB (1.1E-3 in/sec).

Table 6. Mix design aggregate and foamed binder content combinations.

Mix Group	Mix Description	Foamed Asphalt Contents (%)	Percentage Passing Sieve			
			¾”	#4	#8	#200
A	40%RAP-1+60%RC	2, 2.5, 3, 3.5 %	100	54	40	3.5
B	60%RAP-1+40%RC	2, 2.5, 3, 3.5 %	100	53	38	2.7
C	80%RAP-1+20%RC	2, 2.5, 3, 3.5 %	100	52	36	1.9
D	40%RAP-1+60%GAB	2, 2.25, 2.5, 2.75 %	100	50	36	5.3
E	60%RAP-1+40%GAB	2, 2.25, 2.5, 2.75 %	100	50	36	3.9
F	80%RAP-1+20%GAB	2, 2.25, 2.5, 2.75 %	100	50	35	2.5
G	100%RAP-2	2, 2.3, 2.6%	100	64	41	7.5
H	100%RAP-3	1.75, 2, 2.25, 2.5 %	100	59	33	1.6

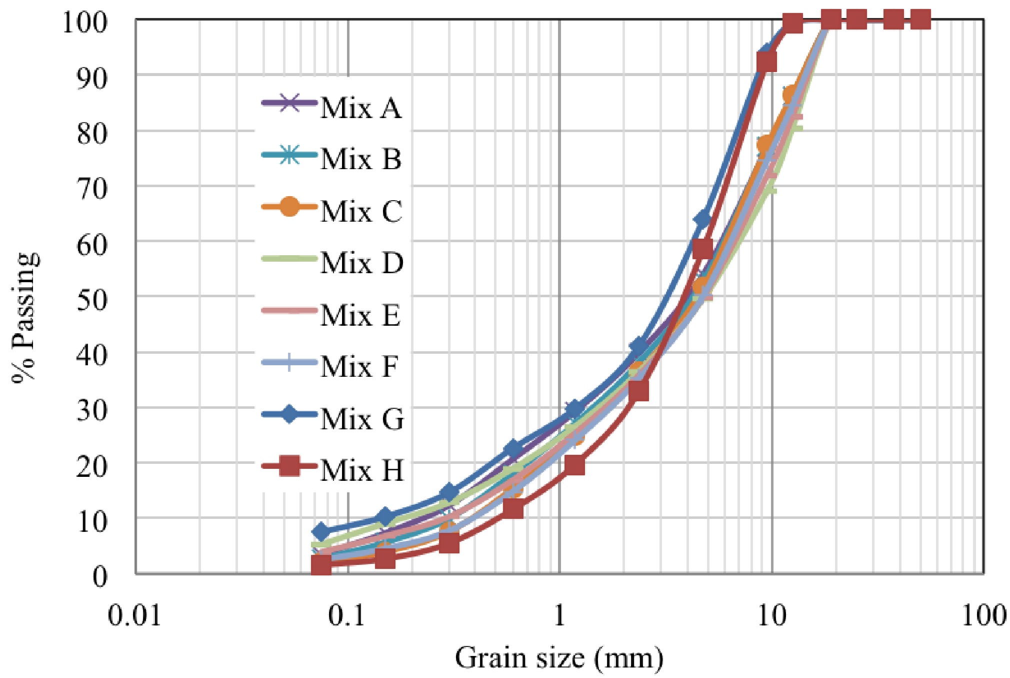


Figure 3. Gradations of aggregate blends.

Mixing, Compacting, and Curing

Figure 4a shows the Wirtgen WLB 10S laboratory-scale foamed asphalt plant that was loaned to the UMD Pavement Materials Laboratory by GRR for this research study. The foamed asphalt is mixed together with aggregates (RAP/RC/GAB) in the companion Wirtgen WLM 30 laboratory-scale twin-shaft pug-mill mixer (Figure 4b). This mixer has the capacity of 50 lb.

The foamed asphalt is mixed together with the moist aggregate at ambient temperature in the mixer, as shown in Figure 4b. During the mixing process, the foamed asphalt droplets first coat some of the fine aggregates producing an asphalt mastic phase. This asphalt mastic is then dispersed in the aggregate blend where it partially bonds with larger aggregates (Fu *et al.*, 2010). There are also some fine particles in the mineral filler phase that are not coated by foamed asphalt. These can provide additional apparent cohesion bonds under unsoaked/partially saturated conditions.

After the FASB mixture is prepared it must be compacted to a density representative of field conditions. The compaction moisture content (CMC) of the FASB mixture influences workability and compactability, as is the case with any granular material. Several studies have suggested mixing moisture contents (MMC) on the dry side of the optimum moisture content (OMC). Lee (1981) recommended using a MMC equal to 65 to 85 percent of OMC as determined by the standard Proctor test. Wirtgen (2010) suggests mixing at 65 % – 95 % of OMC. Fu *et al*. (2010a) suggested 75% to 90% of the modified Proctor OMC as appropriate with respect to both compactability and asphalt distribution.

In this study, all the mixtures were mixed and compacted at a moisture content of 90±6% of OMC as determined by a modified Proctor test (AASHTO T 180) on the non-stabilized aggregate, unless otherwise noted. The OMC and maximum dry density for the evaluated aggregate blends are summarized in Table 7. Mixtures containing recycled concrete had a relatively higher OMC because of the high absorption of the RC aggregate (Table 4). During mix production, the aggregate proportions were mixed together with the prescribed amount of mixing water a day prior to foaming treatment to let the aggregates absorb water and reach equilibrium.

Six indirect tension specimens were compacted for each trial mix design. Marshall compaction (AASHTO T 245-97) with 75 blows on each face was employed to simulate the initial compaction after construction. Standard Marshall molds were used to compact samples 4 inches in diameter and approximately 2.5 inches high. All the aggregates retained on the ¼" sieve were scalped (removed) prior to compaction. All the specimens were prepared within 45 minutes of mixing unless otherwise noted. Test specimens were then placed in a forced-draft oven at 104°C (40°C) for 72 hours to reach a constant mass condition as suggested by Wirtgen (2010). The dry density of the specimens was controlled and specimens with inconsistent density were disregarded.

Figure 4.a) Wirtgen WLB 10S laboratory-scale foamed asphalt plant, b) WLM 30 laboratory-scale pugmill mixer.

Table 7. Optimum moisture content and maximum dry density of mixtures. Tests were performed on the non-stabilized aggregate.

Mix Group	MDD (pcf)[1]	OMC (%)[2]	MMC (%)[3]	CMC (%)[4]
A	124.8	9.1	8.2	8.2
B	125.5	8.8	7.9	7.9
C	121.5	9.3	8.4	8.4
D	137.7	6.6	5.9	5.9
E	132.2	8.8	7.9	7.9
F	126.4	7.8	7.0	7.0
G	130.9	7.6	6.8	6.8
H	113.5	8	7.2	7.2

(1) MDD= Maximum Dry Density, AASHTO T 180 (modified Proctor)
(2) OMC= Optimum Moisture Content, AASHTO T 180 (modified Proctor)
(3) MMC= 90% of OMC
(4) CMC= 90% of OMC

Indirect Tensile Strength Testing

The indirect tensile (IDT) strength, originally designed for evaluating the moisture susceptibility of HMA, is an accepted method for evaluating FASB mixtures for mix design purposes. The South African TG2 (Collings *et al.*, 2002) and Wirtgen (Wirtgen, 2004) design procedures both employ IDT and uniaxial compressive strength (UCS) tests in the dry condition for mix design optimization purposes and advise a minimum requirement for moisture susceptibility. Moisture susceptibly in terms of a tensile strength ratio (TSR) is defined as the ratio of the IDT strength in the soaked condition to the IDT strength in the unsoaked condition. TSR minimum criteria typically vary from 50% to 75% depending on climatic condition. Some researcher (Muthen, 1999; Romanoschi *et al.*, 2004; Marquis *et al.*, 2003; Kim and Lee, 2006, and Fu *et al.* 2008) have proposed using soaked IDT strength for determining optimum foamed asphalt content. Mohammad *et al.* (2003) suggested maximizing the TSR value to define the optimum asphalt content for a project in Louisiana. The Maryland provisional specification for FASB design (Appendix A) requires a minimum TSR of 70% along with a minimum soaked IDT strength of 50 psi.

The IDT strength of FASB in the unsoaked condition is due not only to the foamed asphalt bonds but also to the matric suction from partial saturation, cohesive bonds from partially oxidized residual binder in the RAP (Fu *et al.*, 2008), cementitious bonds from residual non-hydrated cement in RC or newly introduced cement, weak chemical bonds in the mineral phase of the aggregate (Fu *et al.*, 2008), and contacts within the aggregate skeleton. The effect of foamed asphalt stabilization is largely masked in the unsoaked condition because of the combined effects of these various bonding mechanisms.

Soaking reduces or eliminates several of the bonding mechanisms. Matric suction from residual water, weak chemical bonds in the mineral phase, and adhesion from partially oxidized residual binder in RAP are especially vulnerable to induced moisture, but cementitious bonds and interlocking in aggregate skeleton are negligibly affected by soaking. Foamed asphalt bonds after curing are only moderately sensitive to moisture content (Fu *et* *al.*, 2008). Consequently, the effects of foamed asphalt bonds are better isolated in the soaked rather than unsoaked condition.

In this study, IDT strength was measured for both soaked and unsoaked conditions at foamed asphalt contents ranging from 2% to 3.5% for all mixtures. More attention was given to the soaked strength because base courses in Maryland could have high moisture conditions over the course of a year. Several case studies have reported moisture damage as the primary cause of distresses in FASB layers (Chen *et al.*, 2006; Ramanujam and Jones, 2007; Fu *et al.*, 2008). Therefore, performance of FASB materials in the soaked condition is critical and must be properly considered in mix design.

The IDT strength test procedure for FASB material was derived from AASHTO T 283-07, with a slight deviation in the soaking process. Instead of the high vacuum saturation procedure recommended by AASHTO T 283, three cured specimens were soaked via immersion in a water tub for 24 hours at 77˚F (25˚C). This protocol was demonstrated to be adequate in an ancillary investigation of soaking procedures, which is described in more detail in a subsequent section. Three cured unsoaked specimens were kept in an environmental chamber at 77˚F (25˚C) to equilibrate to room temperature.

Optimum Foamed Asphalt Contents

Table 8 summarizes the dry and soaked IDT strengths and the TSR values for all mixtures at all foamed asphalt contents. Table 9 gives the of the mixture properties at the optimum foamed bitumen content, where optimum foamed binder content is defined as maximizing both the soaked IDT strength and TSR. Insights from these data include the following:

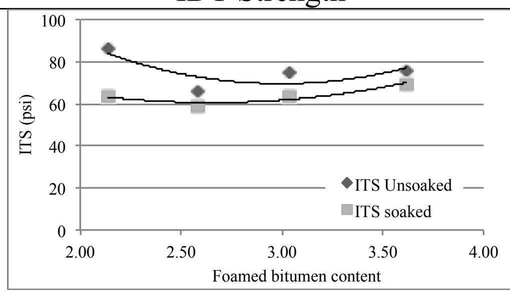
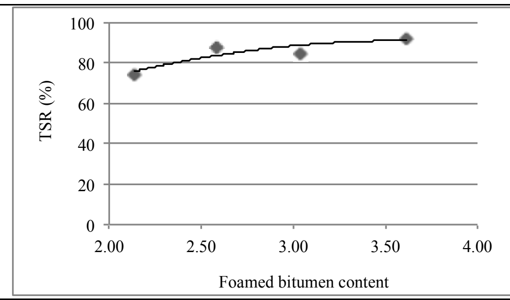
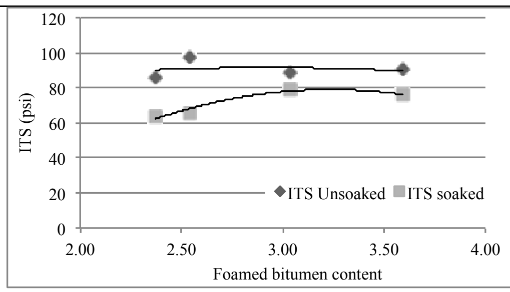
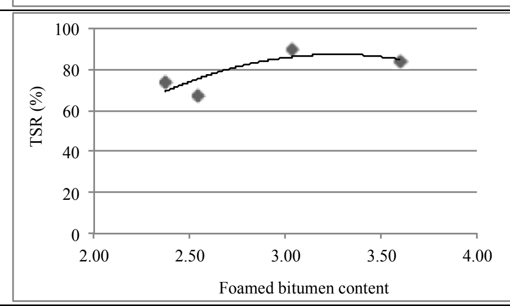
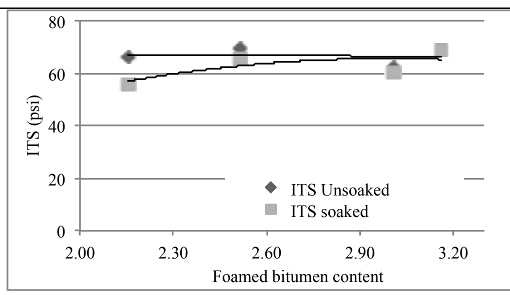
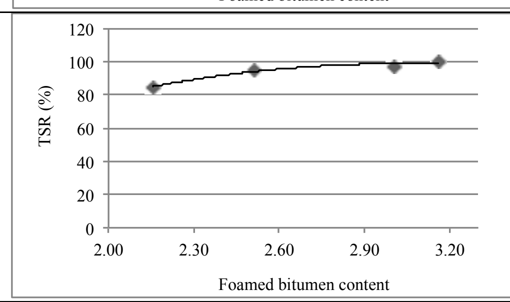
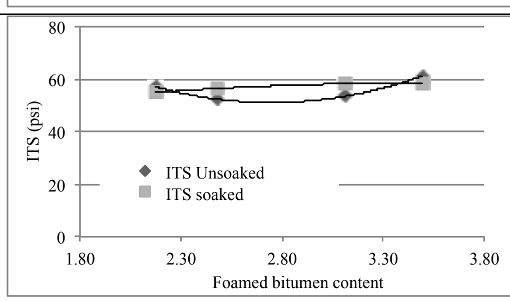
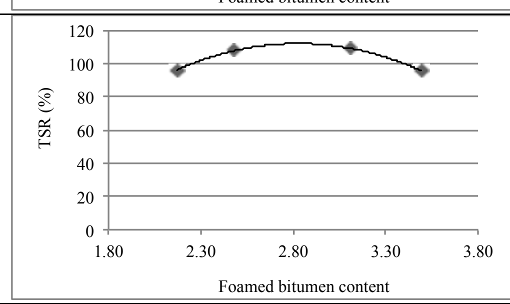
- For many of the mixes, the IDT strength vs. asphalt content curves did not exhibit the expected concave downward shape. This makes it difficult to determine an optimum binder content. When the IDT strength curve did not have a well-defined maximum, he optimum binder content was set at the value that maximized TSR, although in some instances this was also difficult to determine. In cases where there are no well-defined IDT strength or TSR values, it is suggested that the lowest foamed asphalt content that passes all specification requirements be used, but in no case should the foamed asphalt content be lower than 2%. An upper bound of 3.5% for foamed asphalt content is also suggested to eliminate any potential for mix instability and rutting potential.
- Adding Portland cement adds cementitious bonds and increases unsoaked and soaked IDT strength values. This increase was greater for the 100% RAP-2 Mix G than for the 60% RC + 40% RAP-1 Mix A.
- Increasing the ratio of RAP to RC tends to decrease unsoaked and soaked IDT strength values. This could be due to a number of cementitious bonds from residual non-hydrated cement in recycled concrete (RC) and stronger aggregate skeleton in RC as compared to RAP.
- Increasing the ratio of RAP to GAB tends to decrease unsoaked IDT strength and increase soaked IDT strength. The increase in soaked IDT strength is due to more

effective foamed asphalt bonds to RAP aggregate because of the adhesion in the partially oxidized binder in RAP.

- Replacing RC with GAB increases the dry density, has little effect on unsoaked IDT strength, but dramatically decreases the soaked IDT strength.

Detailed mix design and test data are provided in Appendix B.

Table 8. Soaked and unsoaked IDT strength and TSR vs. foamed asphalt content for all mixtures. (Note: In charts, ITS = indirect tensile strength.)

Mix	IDT Strength	TSR
A 40% RAP-1 60% RC		
A* 40% RAP-1 60% RC 1% Cement		
B 60% RAP-1 40% RC		
C 80% RAP-1 20% RC		

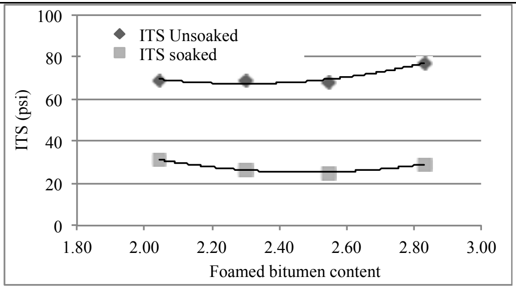
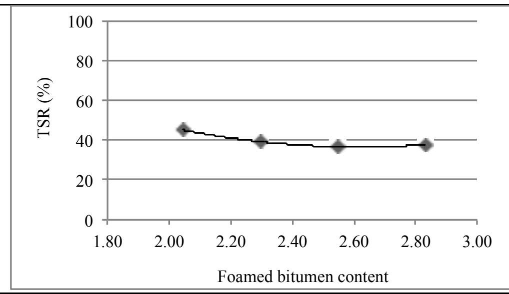
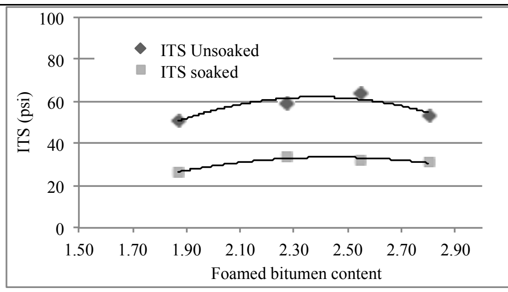
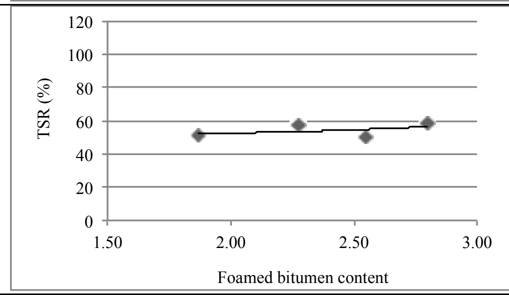
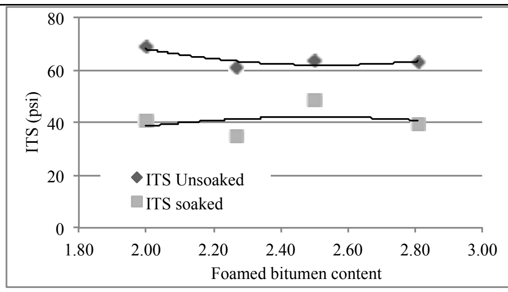
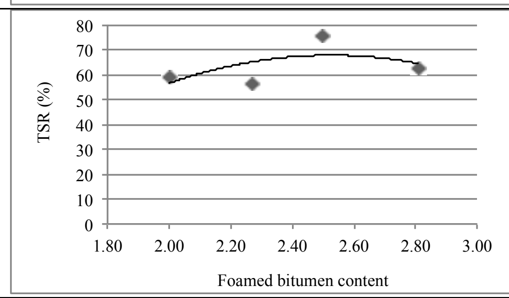
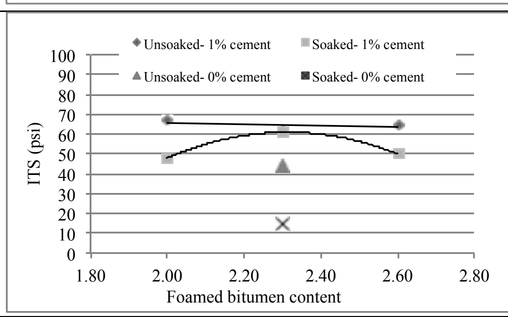
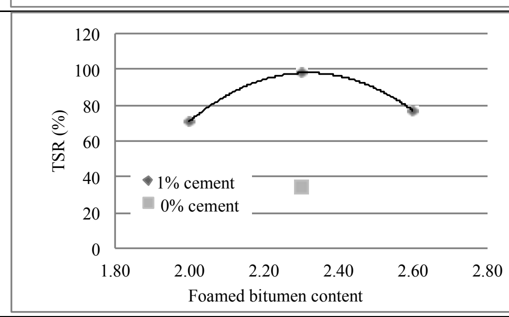
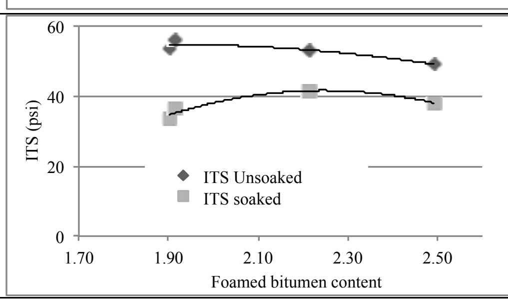
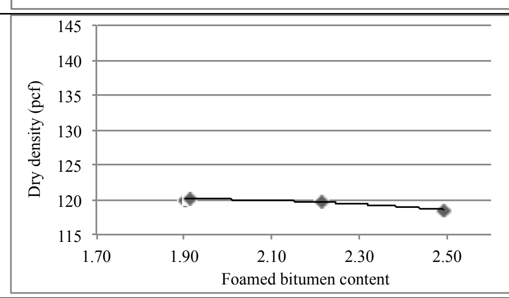
D 40% RAP-1 60% GAB		
E 60% RAP-1 40% GAB		
F 80% RAP-1 20% GAB		
G* 100% RAP-2 1% Cement		
H 100% RAP-3		

Table 9. Summary of mixture properties at optimum binder content.

Mix No.	Mix description	Optimum Binder%	Cement%	Unsoaked IDT Strength (psi)	Soaked IDT Strength (psi)	TSR %	Dry density (pcf)	Soaked MC at break %
A	40% RAP-1 + 60% RC	3.0	0	75	64	85	127	4.9
A*	40% RAP-1 + 60% RC	3.0	1	89	79	90	126	5.0
B	60% RAP-1 + 40% RC	3.0	0	65	64	98	127	3.6
C	80% RAP-1 + 20% RC	3.5	0	61	58	96	125	2.6
D	40% RAP-1 + 60% GAB	2.8	0	77	29	38	140	3.1
E	60% RAP-1 + 40% GAB	2.5	0	64	32	50	133	4.2
F	80% RAP-1 + 20% GAB	2.5	0	64	48	76	131	2.6
G	100% RAP-2	2.3	0	44	15	34	133.5	3.6
G*	100% RAP-2	2.3	1	62	61	98	133.4	3.3
H	100% RAP-3	2.2	0	53	41	78	120	4.5

INFLUENCE OF DETAILS OF MIX DESIGN PROCEDURE

Several ancillary studies were conducted to evaluate the influence of various details of the mix design procedure on mix performance in terms of IDT strength. The investigated details included:

- The soaking procedure
- Active additives (i.e., Portland cement)
- Mixing moisture content
- Stockpiling

Details of these ancillary studies are provided in Appendix B. Key findings from these ancillary studies include the following:

Soaking Procedure. Three soaking procedures were evaluated using Mix A* (40% RAP-1+ 60% RC + 1% PCC): 72 hour soaking, 24 hour soaking, and low vacuum saturation. 72 hour soaking produced the highest water contents and the lowest soaked IDT strength values; however, it also produced the highest coefficient of variation in IDT strength values. Low vacuum saturation was not found to be as effective as 24 hour soaking. 24 hour soaking produced more consistent IDT strength values with lower variability. It also has the practical advantage of convenience.

Active Additives. Adding cement to the mixtures not only increase the fines percentage of the mixture, which generally produces better stabilization (Ruckel et al., 1983), but also promotes early strength gain of the material (Fu et al., 2008). The hydration process of the active filler is faster than the curing process of foamed asphalt, therefore strong and brittle bonds of cement are formed earlier than the weaker and ductile bonds of foamed asphalt (Fu et al., 2008).

The effect of added cement on tensile strength was evaluated both for the GRR standard mixture (Mix G: 100% RAP-2) and the Flanigan standard mixture (Mix A: 40% RAP-1+ 60% RC). Adding 1% PCC to the GRR material produced a 40% increase in unsoaked IDT strength, a 300+% increase in soaked IDT strength, and a 190% increase in TSR values at the optimum design foamed bitumen content. The results for the Flanigan FASB were less pronounced. Adding 1% PCC to the Flanigan produced a 20% average increase in unsoaked IDT strength, a 10% average increase in soaked IDT strength, and about a 5% average decrease in TSR.

The Flanigan mixture without added cement exhibits comparatively good performance under both soaked and unsoaked conditions, and adding cement does not make a significant change in the IDT strength results. It is believed that the hydrated cement in the recycled concrete may contribute to a small amount of cementitious bonds in the mixture even in the absence of added cement.

Mixing Moisture Content (MMC). Fu *et al.* (2010) found previously that foamed asphalt dispersion and correspondingly IDT strength in mixes with high fines contents (e.g. higher than 12% passing No. 200 sieve) are sensitive to MMC. Since all of the mixtures in the present study had much lower percentages passing the No. 200 sieve, it was important to evaluate the influence of MMC on blends with low fines content. The effect of MMC on foamed asphalt dispersion and consequent IDT strength results was studied for Mix F with 2% fines at 2% foaming asphalt content. This mixture was selected because its soaked IDT strength had the highest sensitivity to changes in foamed asphalt content and thus, it is hypothesized, to foamed asphalt dispersion and foamed asphalt bonds.

The aggregate blend was mixed with the same foamed asphalt percentage (2%) at two different MMC targets: 90% of modified Proctor optimum moisture content (OMC) and 104% of OMC. Marshall specimens were made for each of two mixtures at different compaction moisture contents (CMC). The low MMC mixture (90% OMC) was compacted at target CMC values of 90% and 73% OMC. The high MMC mixture (104% OMC) was compacted at CMC values of 104% and 73% OMC. The high MMC (104% OMC) specimens were 29% less strong in the unsoaked condition and 9% less strong in the soaked condition than the blend mixed and compacted at 90% OMC. The specimens compacted at 73% OMC showed very similar IDT strength values both in the soaked and unsoaked conditions regardless of the MMC. This suggests that MMC does not have a significant impact on the asphalt dispersion of these low fines mixtures and that the tensile strength is mainly affected by the conventional moisture-density behavior of the granular material (Khosravifar *et al*., 2012).

Stockpiling. The ability to stockpile FASB is an important practical consideration for production. Jenkins *et al*. (2000) has reported that FASB mixtures can be stockpiled for several months. To evaluate this claim in the laboratory, loose material for Mixes C (80% RAP + 20% RC), D (40% RAP + 60% GAB), and a mixture from GRR (100% RAP + 1%

PCC at 2.5% foamed asphalt) were kept in sealed buckets for 15 days to investigate the effect of stockpiling. These three mixtures are representative of RAP+RC, RAP+GAB and RAP mixtures. In order to eliminate any moisture content influence, only mixtures having the smallest CMC variations (less than 6%) were evaluated.

Figure 5 depicts the unsoaked and soaked IDT strength variations versus time. After 14 days of stockpiling, Mix C (RAP+RC) showed 9% and 27% average decreases in unsoaked and soaked IDT strength, respectively, as compared to values immediately after mixing. Mix D (RAP+GAB) showed 14% and 29% decreases in unsoaked and soaked IDT strength after 14 days of stockpiling. The third mixture (RAP) showed 26% and 24% decreases in unsoaked and soaked IDT strength after 15 days of stockpiling.

The conclusion from these results is that stockpiling the FASB can reduce both the unsoaked and soaked strength of all of the materials. The third mixture containing 100% RAP plus PCC additive showed a particularly high vulnerability to stockpiling. This may be the result of breakage of the early cementitious bonds that develop during stockpiling. Based on these limited laboratory evaluations, stockpiling of FASB should be avoided when possible; if this is not possible, then the material structural properties should be appropriately reduced.

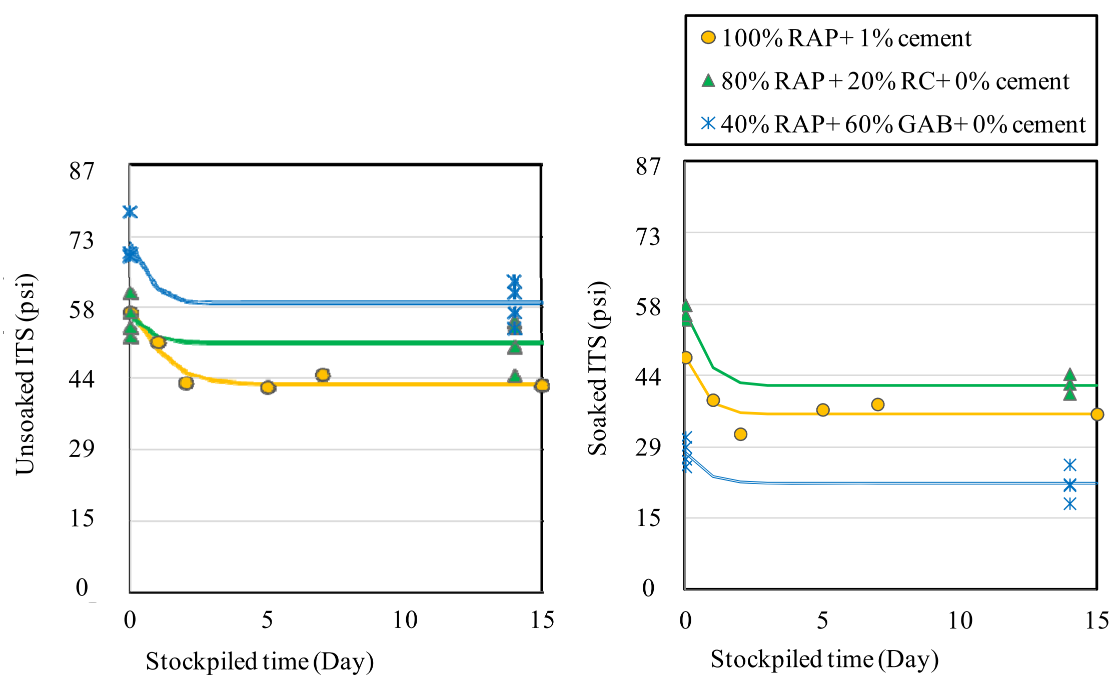


Figure 5. Unsoaked and soaked IDT strength vs. stockpiling time.

CHAPTER 3. MD 295 FIELD EVALUATION

INTRODUCTION

A major objective of this project was to evaluate the in-place properties, and in particular the increase in in-place stiffness gain during curing, of FASB materials. The first several attempts to do this failed for a variety of reasons. These unsuccessful attempts included:

- Widening of York Road north of Baltimore. Construction took place just as the research project was getting underway. Due to inadequate preparation time, less-than-ideal material placement, and heavy rain the night after placement, little useful quantitative information was obtained from this site. Details of the York Road evaluation are provided in Appendix C.
- Reconstruction of the Harry Truman Park and Ride parking lot in Annapolis. Schedule delays pushed construction so late in the season that it was too cold to place FASB. The planned FASB layer was consequently replaced by conventional GAB.
- A full-depth patch at the entrance to the Glenn Dale maintenance facility in Prince George's County. High groundwater conditions made for unstable subgrade conditions. Extensive undercutting and backfill was required, making interpretation of any field stiffness measurements difficult. Details of the Glenn Dale evaluation are provided in Appendix D.
- A full-depth patch at the Oxon Hill fire station in Prince George's County. Heavy thunderstorms for the entire week after FASB placement prevented the material from setting up and curing properly. The FASB eventually had to be removed and replaced. Details of the Oxon Hill evaluation are provided in Appendix E.

The only successful field evaluation was at the MD 295 lane widening project near BWI airport. A wide variety of very good field and laboratory data were collected at this project. This chapter summarizes the major results and findings from the MD 295 field evaluation; details are provided in Appendix F.

CONSTRUCTION

The field evaluation site was the northbound lane addition on MD Route 295 near BWI Airport. The pavement design for this project consisted of an 8 inch thick HMA layer over an 8 inch thick base layer of either FASB or GAB over selected portions of the alignment. The subgrade construction included several remedial undercuts and backfills with GAB along the 3600 ft construction site. The base and HMA layers were placed using the paver and compacted using a Bomag intelligent compaction roller.

The project layout is presented in Figure 6. The FASB Control Strip was placed in May 2011 in one 8 inch thick lift. Around 200 tons of FASB were placed and compacted along the 200 ft strip. Curing conditions were ideal with daily average temperatures in the mid-to-upper 70ºF range and little precipitation for the entire week after placement.

The main FASB placement started on July 7, 2011, and included two segments. Because of problems maintaining target grade during placement of the single 8 inch thick lift in the Control Strip, the main FASB segments were placed in two 4 inch thick lifts. The mainline construction included:

1. Segment A: The first 4 inch layer was placed on July 7 and the second 4 inch lift was placed four days later on July 11.
2. Segment B: FASB was placed in two 4 inch lifts on the same day (July 11).

A companion GAB section was constructed on July 13 in two 4 inch lifts.

Weather conditions for the mainline segments were mostly favorable for curing, with daily average temperatures in the low 80ºF range and no significant rain except for a local thunderstorm on site on the night of July 7 shortly after the placement of the first layer of Segment A. On-site rain gauges recorded 3 inches of rain. All FASB and GAB base layers were covered by 8 inch HMA layer on July 18.

Figure 6. Project layout. Numbers in parentheses indicate placement dates; numbers in white squares indicate stations.

MATERIAL PROPERTIES

The FASB test mixture in this study consisted of a blend of 40% RAP and 60% RC mixed with 2.8% foamed asphalt at ambient temperature. The PG 64-22 binder used in the mix was foamed at a 2.2% foaming water content at 320ºF. The detailed information about the material characteristics and the mix design is provided under “Mix Group A” in Appendix B. Figure 7 shows the gradation of the GAB and FASB used in the study. The optimum moisture content (OMC), maximum dry density (MDD), and the *in-situ* permeability (k) of GAB and FASB tabulated in Table 10 show that FASB has a lower MDD, lower percentage of fines, and a competitive permeability as compared to GAB. This suggests that the drainage function of FASB may be comparable to that of GAB.

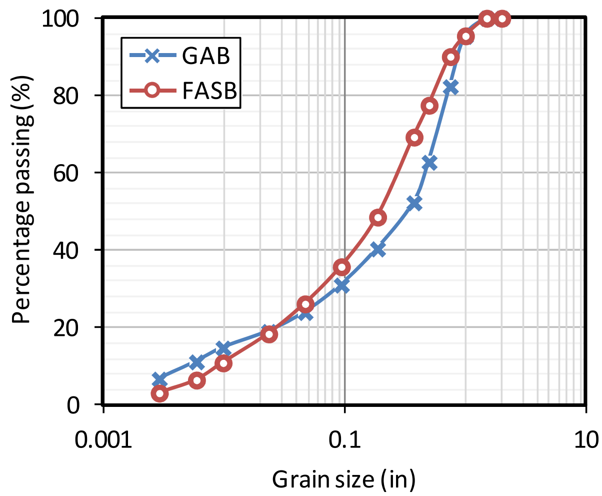


Figure 7. Gradation of the base material, GAB and FASB.

Table 10. GAB and FASB Properties.

Material	Percentage passing sieve #200 (%)	Optimum moisture content (%)	Maximum Dry density (pcf)	*In-situ* Permeability (in/sec)	
	AASHTO T 27	AASHTO T 180- method D, AASHTO T 224		Borehole test	Falling head test
GAB	6.7	5.2	148.8	1.11E-03	3.98E-03
FASB	3.1	10.2	122.4	3.17E-03	1.13E-02

FIELD TESTING

Construction/Immediate Post-Construction

Control Strip

The project team performed *in situ* stiffness tests using the Humboldt GeoGauge 4140 and Zorn ZFG 3000 LWD at multiple locations every 25 to 30 ft along the 200 ft FASB Control Strip on May 24, 25, 26, 27, and 31. Two replicate measurements were performed at each location for the GeoGauge. For the LWD, 3 seating drops were first applied followed by 3 replicate test measurements. A thin layer of moist fine-grained sand was placed beneath both the GeoGauge and LWD to ensure intimate contact with the compacted base layer.

Field density and moisture content were measured after FASB compaction in accordance with AASHTO T 310 using a 3430 nuclear gauge. Because the foamed asphalt and the RAP binder contain hydrogen, the raw moisture content readings from the nuclear gauge are inaccurate. This was corrected by using a moisture offset factor *k* of -2.9% as determined using the Troxler 3430 manual (Troxler, 2006):

$$k = \frac{\%MC_{LAB} - \%MC_{GAUGE}}{100 + \%MC_{GAUGE}} \times 100 \tag{1}$$

in which MC_{LAB} is the moisture content measured via laboratory oven drying and MC_{GAUGE} is the raw moisture content measured by the gauge. For the control strip, the values for MC_{LAB} and MC_{GAUGE} were 10.2% and 13.4%, respectively.

Mainline Segments

Similar to the Control Strip, GeoGauge and Zorn LWD stiffness measurements were obtained on the mainline Segment A and Segment B immediately after placement of the FASB and on subsequent days. Tests were performed approximately every 100 feet along the 1600 foot total length of the test segments.

The stiffness readings for GAB sections were performed on the day of its placement (7/13/11) and on the following day. Tests were performed approximately every 100 feet along the 1180 foot test segment. The measurements were interrupted because of a thin leveling layer placed on top of the existing layer on July 14. This additional layer made it impossible to track accurately the stiffness increase with time at this location. However, the potential equilibrium stiffness of the GAB material was obtained from a nearby undercut and fill section (822+50 to 823+10) on July 8.

The equilibrium stiffness is obtained when the material dries in the field until the moisture content equilibrates with the surrounding environment and the rate of evaporation approaches zero. This equilibrium can be reached within seven days for uncovered coarse sand under favorable weather conditions (Yanful and Choo, 1997).

Dynatest FWD measurements were performed on the surface of the GAB and FASB layers. Since the FWD had a full schedule of production testing at other project locations around the state, the standard 12 inch diameter plate used for FWD testing of paved sections was used for these drops. Unfortunately, this induced excessive contact pressures and plastic deformations. The FWD data directly on the GAB and FASB surfaces were therefore ignored.

FIELD TEST RESULTS

Construction/Immediate Post-Construction

General

Nuclear moisture and density measurements on the Control Strip immediately after compaction are summarized in Table 11. Satisfactory compaction was achieved throughout the FASB construction site.

The increase in stiffness with time during the first week after placement was measured using the GeoGauge 4140 and Zorn ZFG 3000 LWD. The GeoGauge and Zorn LWD devices are designed to measure stiffness values within the range typical for unbound materials. According to the GeoGauge user's manual, the device is capable of measuring stiffness values from 4 to 80 ksi although its practical upper limit was found to be about 65 ksi. Overload errors were observed at several FASB testing locations on the 7th day, indicating that the material was too stiff to be measured accurately. According to the Zorn LWD user's manual, this device is capable of measuring stiffness values up to 19.1 ksi although the actual upper limit was found to be about 30 ksi at deflections as low as about 0.13 mm. Error messages were observed at several FASB testing locations on the 4th day, indicating the material was too stiff to be measured accurately.

Table 11. Nuclear Gauge Measurements after FASB Placement.

Test Segment	Date	Corrected Moisture Content (%)	Wet Density (pcf)	Dry Density (pcf)	Compaction (%)
Control Strip	5/24/2011	10.9	134.1	120.8	98.7%
Segment A- 1st lift	7/7/2011	10.2	133.2	120.8	98.7%
Segment B- 2nd lift	7/11/2011	10.9	134.8	121.5	99.3%

Overall, the Zorn LWD underpredicted the stiffness of FASB and GAB material by a factor of about 0.5 in comparison to the GeoGauge, as illustrated in Figure 8. Reasons for this systematic difference include differences in the loading rate, induced stress levels, and the zone of influence for each device. The latter factor, in particular, is noteworthy. The GeoGauge user manual reports the depth of the zone of influence to be 9 to 12 inches. The depth of the zone of influence for the Zorn LWD as determined by finite element simulations (Khosravifar *et al.*, 2013a) is about twice the diameter of loading plate or about 24 inches. Both influence zones extend beyond the bottom of the FASB and GAB layers. The *in-situ* stiffness values ($E_{\textit{in-situ}\ \mathrm{Device}}$) reported by the GeoGauge and LWD are thus the average modulus of all materials within their respective zones of influence.

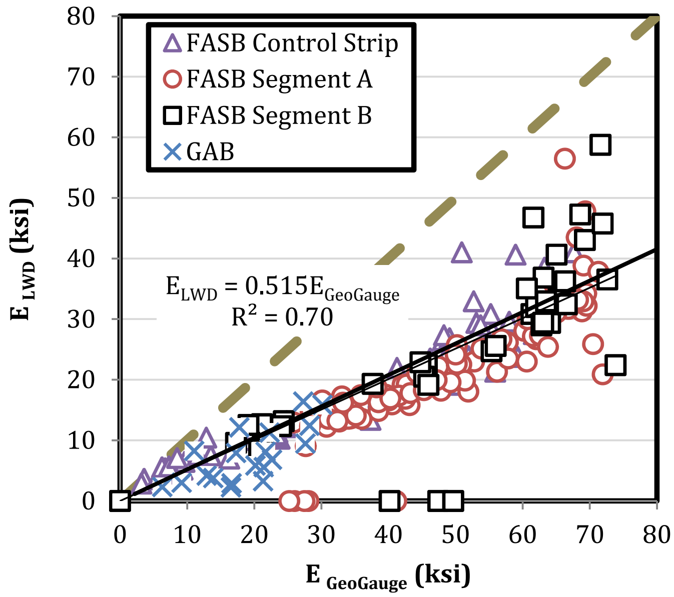


Figure 8. Correlation between Zorn LWD and GeoGauge stiffness measurements for different FASB and GAB sections over 7 days of monitoring.

Stiffening of GAB Versus FASB

A conventional GAB material will gain stiffness as it dries from the compacted moisture content near the optimum to a lower long-term equilibrium moisture condition. During this process, aggregate particles are drawn closer as the moisture in the pores evaporates and matric suction pressures develop.

FASB material has an additional stiffening mechanism after placement in the field. The moisture evaporation not only brings together the aggregate particles but also promotes the bonding of the foamed asphalt droplets to the aggregate particles. As the moisture disappears and foamed asphalt bonds become stronger, the material gets stiffer. This is the process that is often termed field curing.

Precipitation can adversely affect the stiffening process. Figure 9 shows the stiffness gains as measured using the GeoGauge and Zorn LWD devices on the first 4-inch thick lift of Segment A. Trends are presented both in terms of absolute stiffness and percentage stiffness change. As is clear from the figures, both the GeoGauge and Zorn LWD captured the rain-induced decrease in stiffness following a thunderstorm the evening after FASB placement, as highlighted by the red circle in Figure 9b. Stiffness then resumed increasing after this drop, but the net gain at 4 days after placement was relatively small. On possible explanation for this is that the 4-inch thick lift is too thin relative to the zone of influence of the GeoGauge and LWD devices and therefore the measurements are unduly affected by the underlying subgrade. It could also be that the heavy rain irrevocably hampered curing on the night after the FASB was placed.

Figure 10 demonstrates the increase in *in situ* stiffness over time for the GAB versus FASB sections as measured by the Zorn LWD and GeoGauge. The average initial stiffness values (E_i) measured using the Zorn LWD on the FASB sections were 11.4, 10.9, and 14.2 ksi for the Control Strip, Segment B, and Segment A-2nd lift, respectively. The corresponding initial stiffness for the GAB section was 5.6 ksi. The average initial stiffness values measured using the GeoGauge were 23.0, 21.2, and 30.3 ksi for the respective FASB sections and 15.7 ksi for the GAB section. The reason for a higher initial stiffness of the second lift of Segment A is that the first 4 inch lift of FASB material had already cured and stiffened for a few days before the second lift was placed and the measurements were influenced by the underlying first lift. This is also the reason for the relatively small percentage increase in $\Delta E/E_i$ for the second lift of Segment A (Figure 11); the first lift had already partially cured and increased in stiffness.

The average stiffnesses of the FASB sections after 7 days of curing and drying in the field were 35.1 ksi, and 63.8 ksi as measured by the LWD and GeoGauge, respectively (Figure 10). The corresponding stiffnesses of the GAB section were 16.1 ksi and 32.8 ksi. As shown in Figure 11a, the LWD measured a stiffness increase of 243% and 224% for Segment B and Control Strip, respectively. The corresponding trends measured using the GeoGauge are depicted in Figure 11b. The percentage increases in the stiffness of the GAB are also shown in Figure 11. The GAB stiffness increase was substantially lower than in the FASB sections (except for Segment A). This is consistent with the strengthening of the foamed asphalt bonds due to curing in addition to the partial saturation effects from drying.

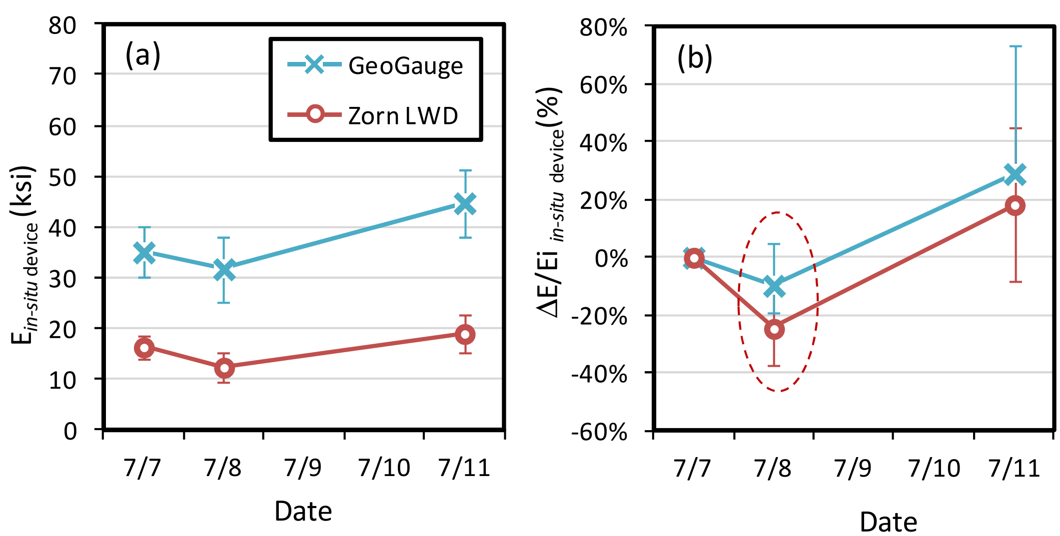


Figure 9. (a) Average stiffness and (b) percentage increase in stiffness with time as measured using the Zorn LWD and GeoGauge on the first 4 inch lift of Segment A.

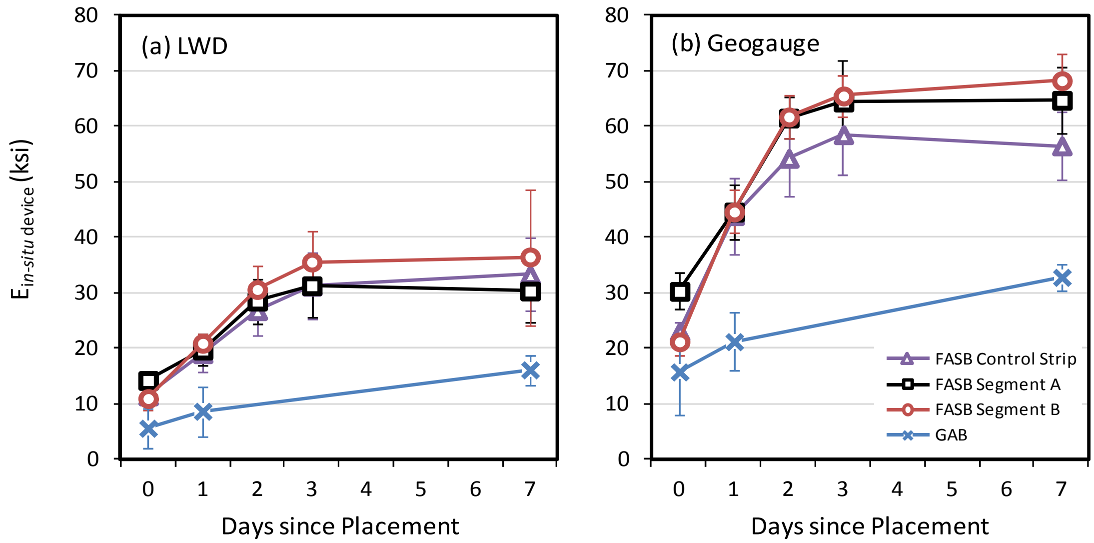


Figure 10. The average stiffness increase with time as measured using the (a) Zorn LWD and (b) GeoGauge on FASB Control Strip, Segment A (second 4 inch lift), Segment B, and GAB. Error bars show one standard deviation.

Both the Zorn LWD and GeoGauge stiffness measurements exhibited higher variability as the material stiffened and the devices approached their measurement limits after 3 to 4 days of drying and curing in the field. Therefore, it is not certain that the stiffness of the FASB material had stabilized similar to what was observed in the less-stiff GAB; the curing process was likely still ongoing for the FASB. This was confirmed by falling weight deflectometer (FWD) measurements performed on the paved sections four to six months after HMA placement, as described in the next section.

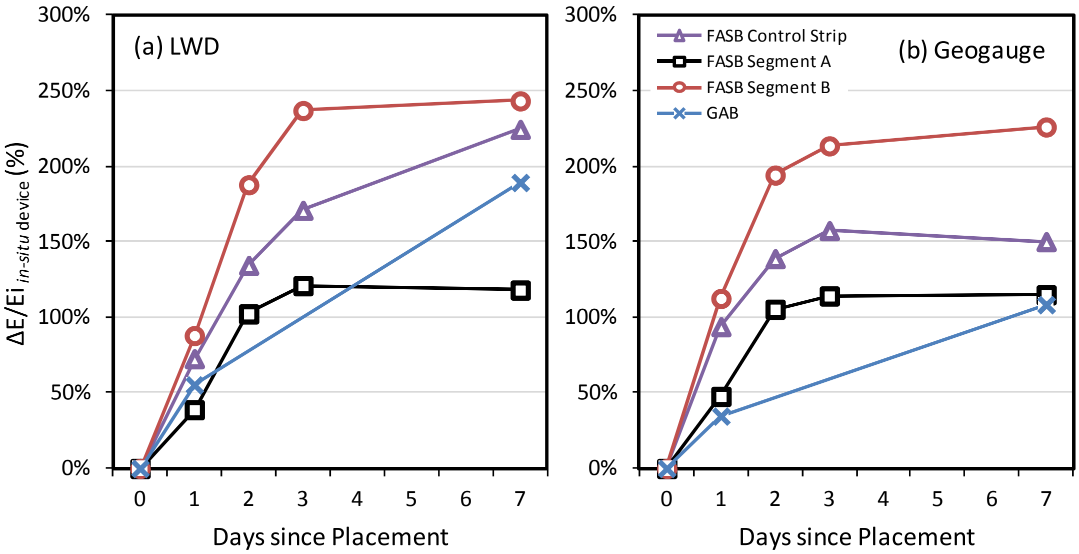


Figure 11. The average percentage increase in stiffness with time as measured using (a) the Zorn LWD, and (b) GeoGauge on FASB Control Strip, Segment A (second 4 inch lift), Segment B, and GAB. Error bars show one standard deviation.

Long-Term Post-Construction

Long-term post-construction stiffness was measured using the SHA Dynatest FWD machine on the final pavement structure in November 2011 before opening the site to traffic, 4 to 6 months after placement of the FASB and GAB layers. Three to four data measurements were obtained for each test segment. Backcalculation results using ModTag V4.3.0 are plotted in Figure 12 for the HMA, Base, and Subgrade. The RMS backcalculation errors were all less than 6.9% and averaged 3.9%, indicating a good fit. The backcalculated subgrade moduli averaged 41.6 ksi and were relatively uniform, Segment B showing slightly stiffer values. The subgrade moduli of the FASB sections were significantly higher than in the GAB section.

The stiffness of the GAB measured using the FWD was in the same range as the ultimate stiffness measured by GeoGauge and Zorn LWD. The FWD results showed that the FASB became significantly stiffer than the final GeoGauge and LWD values seven days after placement. The long-term stiffness of the GAB backcalculated from the FWD results was about 24 ksi. The corresponding long-term stiffness of the field-cured FASB was about 295 ksi, 12.3 times that of the GAB. Placement of the HMA layer may have improved the curing of the underlying FASB by applying additional heat and enhancing moisture evaporation.

The lower stiffness GAB layer appears also to have reduced the modulus of HMA layer. It is usually more difficult to achieve the same HMA compaction over a softer underlying layer than over a stiffer layer.

Within the FASB sections, Segment A had the lowest moduli, which could be due to the four day delay in placing the second 4 inch lift. This is important from a construction point of view. Breaking the installation of FASB layers into two separate days affected the final stiffness of the material even with rewetting of the surface before placement of the second lift.

Dynamic modulus laboratory tests were performed on FASB cores obtained from the FWD test locations. The results suggested an average dynamic modulus of 635.0 ksi at 5 Hz loading rate and 77°F, which is greater than but generally consistent with the high FWD backcalculated moduli. Details of the dynamic modulus testing are provided in Chapter 4.

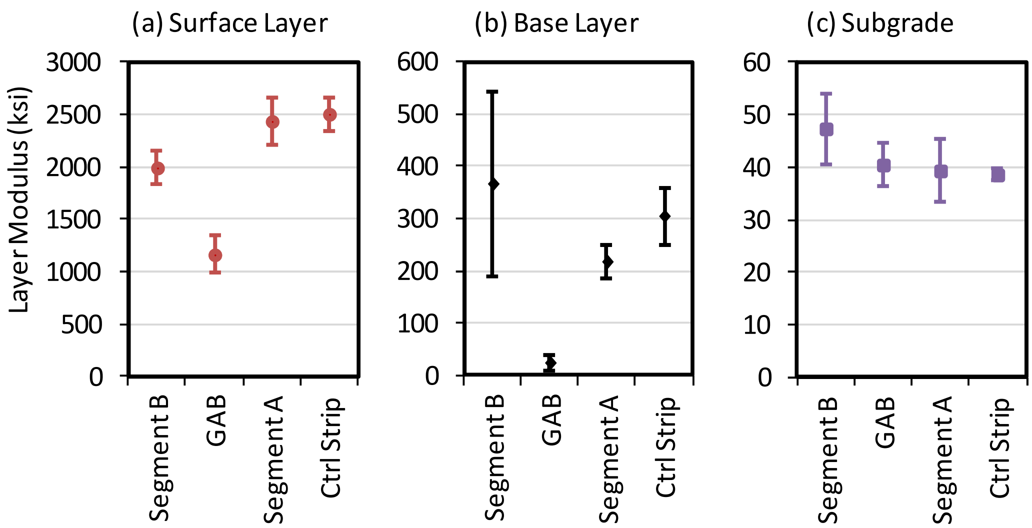


Figure 12. Backcalculated moduli for the (a) HMA layer, (b) Base layer, and (c) Subgrade. Error bars indicate one standard deviation.

CONCLUSIONS FROM FIELD EVALUATION

The principal conclusions from construction/immediate post-construction and long term post-construction testing on this field study are as follows:

- The test sections were properly compacted and showed a satisfactory, uniform compaction as indicated by Nuclear Gauge moisture and density measurements.
- The Zorn LWD and GeoGauge devices gave significantly different values for the *in-situ* stiffness, with the Zorn LWD systematically reporting values approximately 0.5 times those from the GeoGauge at the same locations. The reasons for these differences include different load levels, loading rates, depth of zones of influence, analysis assumptions, and other factors. Given these issues, neither of the devices can be considered to give the "true" in place stiffness. The more useful measures are the percentage increase in stiffness with time and the relative stiffness of the FASB versus conventional GAB.

- Curing of the FASB in the Control Strip and mainline Segment B placement produced stiffness increases of 188 to 234% within one week after placement as measured by GeoGauge and Zorn LWD respectively. The stiffness increases measured using the Zorn LWD tended to be slightly higher than those measured using the GeoGauge.
- Comparing the Control Strip placed in May 2011 and the mainline placement in July 2011, the influence of weather on FASB curing is clear. The Control Strip placed in May 2011 under generally favorable curing conditions (no rain, average daily temperatures in mid-70^{o}F range) increased in stiffness by 150% (224%) over a one-week period as measured by the GeoGauge (Zorn LWD). The mainline segments placed in July 2011 under ideal curing conditions (average daily temperatures in the low-80^{o}F range, no rain except for one brief but intense thunderstorm) exhibited greater increases stiffness, reaching average values 7 days after placement of 67.0 ksi and 34.2 ksi as measured by the GeoGauge and Zorn LWD, respectively. The thunderstorm on the night after placement of the first lift of Segment A caused a pronounced decrease in in-place stiffness of the FASB, but this completely rebounded during the subsequent warm and dry days.
- The initial stiffness of the FASB sections (excluding Segment A) was on average 1.4 (GeoGauge) to 2 (Zorn LWD) times the equivalent GAB sections. The gain in stiffness after one week of drying and curing of the FASB sections was also greater than after one week of drying of the GAB; the FASB sections (excluding Segment A) increased in stiffness by a factor of 2.9 (GeoGauge) to 3.3 (Zorn LWD) while the GAB increased by a factor of 2.1 (GeoGauge) to 2.9 (Zorn LWD).
- FWD backcalculated moduli of the FASB layers averaged 295 ksi, which is significantly higher than for the GAB layer. Placement of the HMA layer may have improved the curing of the underlying FASB by applying additional heat and enhancing moisture evaporation.
- Within the FASB sections, Segment A showed the lowest moduli, which could be due to the 4 day delay in placing the second 4 inch lift for this segment. This is important from the construction point of view. Breaking the installation of FASB layers into two separate days affects the final stiffness of the material even with wetting the surface before placement of the second lift.
- Laboratory and field permeability tests found that the permeability of FASB is comparable to and in some cases slightly higher than that of GAB. This suggests that the drainage function of FASB should be comparable to that of GAB.

This field study clearly supports the suitability of FASB material for high volume pavement applications if designed and installed properly and cured under favorable climatic condition. The final in place stiffness of this flexible, partially bound material is substantially higher than unbound GAB. Therefore, its proper use can reduce the required thickness of pavement sections, resulting in cost savings in addition to recycling benefits.

Stiffness-based in place QA/QC devices are a necessity for tracking the gain in stiffness during field curing. The LWD and GeoGauge stiffness devices were both able to track the stiffness increase with time, at least until the material gets too stiff and beyond the limits of the devices. The two devices did not report the same absolute stiffness values; the Zorn LWD generally reported stiffness values 0.5 times those from the GeoGauge. The combination of initial stiffness values, rate of stiffening, and the final stiffness measurements can be used as a guide in QC/QA of FASB material. Nuclear moisture and density gauge can effectively be used to monitor the post-construction compaction level and the field moisture content but cannot capture the stiffening of FASB during curing. Moisture corrections on the gauge are required to obtain reliable measurements.

FWD measurements, at least when done using the 12 inch diameter loading plate and 00 to 13000 pound loads typical for paved section testing, is not suitable for construction/immediate post construction QC/QA on the unpaved sections as it induces excessive stress levels and plastic deformations. FWD measurements on the paved sections are appropriate for backcalculating the stiffness of the cured FASB and other layers.

CHAPTER 4: POST-CONSTRUCTION LABORATORY EVALUATION

INTRODUCTION

Post-construction laboratory testing was conducted on field cores of FASB material from two sites. The emphasis of this testing was on the material properties relevant to pavement structural design, specifically the stiffness and permanent deformation characteristics of the material.

Stiffness

Different researchers have employed various test methods to measure the stiffness of FASB materials. Indirect tension resilient modulus testing on cores or laboratory prepared FASB specimens was employed in the past by several researchers (Muthen, 1999; Nataatmadjia, 2001; Marquis et al. (2003); Collings et al. (2004); Ramanujam and Jones, 2007; Khweir, 2007). Most of the mixtures evaluated in these past studies contained RAP and/or GAB with or without Portland cement; the reported moduli ranged from 100 ksi to above 800 ksi for different mixtures. This large range of the resilient modulus values is due to the wide variety of aggregates and binders and differences in the mixing, curing, and compaction procedures in the various studies.

The Wirtgen cold recycling manual (Wirtgen, 2010) states that the indirect tension resilient moduli of cured, unsoaked Marshall briquettes are often significantly higher than those from dynamic triaxial and flexural beam tests as well as backcalculated moduli from falling weight deflectometer (FWD) measurements. Fu *et al.* (2009) found similar results. Indirect tension resilient modulus thus may not be the most suitable test procedure. The lack of moisture in the test specimens as well as the geometry and stress states in the indirect tension loading mode could explain the higher resilient moduli.

Jenkins *et al.* (2007) performed triaxial resilient modulus tests on laboratory compacted and cured foamed asphalt treated RAP with and without Portland cement. They reported relatively low modulus values in the range of 20 to 50 ksi for FASB mixtures at different percentages of foamed asphalt and cement. They found that adding cement to the FASB mixture both increased the resilient modulus and its dependency on bulk stress. Fu and Harvey (2007) performed triaxial resilient modulus tests at different loading rates, deviatoric stresses, and confining pressures on laboratory compacted and cured specimens. They reported modulus values of 100 to 350 ksi, with most of the data in the range of 150 to 300 ksi. They found that resilient modulus was a function of both load pulse duration, stress level and, at higher temperatures/slower loading rates, confining stress level.

As would be expected from the rheological characteristics of the asphalt binder, the stiffness of FASB mixtures is also temperature dependent. Nataatmadja (2001) reported a 30 to 44% reduction in indirect tension resilient modulus when the test temperature increased from 50°F to 104°F. Muthen (1999) found that increasing the temperature from 41°F to 104°F caused a

20% to 45% decrease in the flexural stiffness of foamed treated weathered granite and aeolian sand. Fu and Harvey (2007) also confirmed the temperature sensitivity of triaxial resilient modulus.

Kim *et al.* (2009) performed uniaxial dynamic modulus tests on gyratory compacted FASB from seven different sources of RAP in Iowa and examined the effect of foamed asphalt content on the modulus. The reported dynamic modulus values ranged between 460 and 670 ksi at a 10 Hz loading frequency and 70°F for the RAP mixtures with 2% foamed asphalt. They also observed slight changes in dynamic modulus with increasing foamed asphalt content.

Comparing the results from these previous studies with the typical 20-40 ksi range for GAB resilient modulus (Papagiannakis and Masad, 2007) and the 450-1000 ksi range for dynamic modulus of HMA at 70ºF and a 10 Hz loading frequency (Huang, 2004) suggests that the stiffness of FASB lies somewhere between GAB and HMA but closer to HMA. In addition, previous studies have shown that the stiffness of FASB material is loading rate, temperature, and, to a lesser extent, bulk stress dependent. Therefore, triaxial dynamic modulus testing was adopted in this study to characterize the stiffness of the FASB field cores.

Permanent Deformation Resistance

Permanent deformation or rutting is one of the most important distresses occurring in flexible pavement sections. It is the accumulated deformation from densification and/or shear failure under repeated loading. Resistance to permanent deformation of FASB mixtures can be enhanced by improving the aggregate skeleton (e.g., shape, hardness, and roughness), increasing the maximum aggregate size, improving the compaction and curing process, and limiting the foamed asphalt content to a maximum of 3%. Excess foamed asphalt will act as a lubricant between aggregates, leading to increased shear failure and consequent permanent deformation (Wirtgen, 2010).

Long and Ventura (2004) conducted triaxial dynamic repeated load permanent deformation (RLPD) tests at a 4 Hz loading frequency on a high quality granular material stabilized with 1% Portland cement and different percentages of foamed asphalt. They performed the tests at various combinations of confinement, deviatoric stresses, relative densities, and saturation levels. The test temperature was not stated in their report. The writers formulated a combined model to account for all the aforementioned variables. Their principal conclusions included: (1) for stress-to-strength ratios lower than 0.6, which is typical for well-designed pavement structures, the resistance to permanent deformation is mainly governed by the characteristics of the aggregate and not the applied stress states; (2) permanent deformations significantly increase at stress-to-strength ratios greater than 0.6; and (3) resistance to permanent deformation decreases slightly as the foam binder content increases.

Mohammad *et al.* (2006) tested the rutting susceptibility of non-stabilized 100% RAP, a foamed asphalt stabilized blend of 50% RAP with 50% soil cement, and foamed asphalt stabilized 100% RAP. The test was conducted for 10,000 cycles with a 0.1 second loading period and 0.9 second rest period. Loading consisted of a cyclic 15 psi deviatoric stress at a constant 5 psi confinement pressure. The testing temperature was not stated in the study. The three materials exhibited 3000, 5000, and 21,000 με permanent strain, respectively, after 10,000 loading cycles. In other words, foam stabilized 100% RAP had the highest and unstabilized RAP had the lowest susceptibility to rutting. The authors offer no explanation for these paradoxical results.

Gonzalez *et al.* (2011) evaluated the stabilization of a marginal aggregate using 1% Portland cement and various foamed asphalt contents. The test procedure consisted of 50,000 load cycles at 4 Hz and a 7.2 psi confining pressure. The deviatoric stress was increased in 7 steps from 11 psi to 76 psi. The testing temperature was not stated in the study. High variability in the final plastic strains between replicates was observed. However, the results suggested that increasing the foamed asphalt content significantly increases the rutting susceptibility. The final permanent strains varied between 1000 to 13000 με for foamed asphalt contents ranging from 0% to 4%.

Fu *et al.* (2010) performed a limited set of triaxial permanent deformation tests to investigate the role of curing and cement content on the rutting resistance of foamed treated RAP aggregate. Two curing conditions were investigated: Condition A, sealed at 20°C for 24 hours; and Condition B, unsealed at 40°C for 7 days. The test sequence consisted of 20,000 load cycles at a 43.5 psi deviator stress, followed by another 20,000 cycles at 72.5 psi, and ending with 210,000 cycles at 101.5 psi. A 10 psi confinement stress was maintained through the test. The deviator stress was applied as a haversine pulse with a 0.1 second loading duration and 0.2 second resting period. The testing temperature was not stated in the study. The results showed that the enhanced curing condition (curing condition B) and the addition of Portland cement significantly improved FASB resistance to permanent deformation even in the soaked condition.

None of these prior studies stated the testing temperature, which suggests that there was no control on the temperature during the test. However, the presence of foamed asphalt and oxidized binder in the RAP suggests that temperature effects should be expected. Similar to asphalt mixtures, FASB mixtures are expected to show higher permanent deformations at higher temperatures because of the reduced viscosity and lubrication effects of the softened binder. It is important to control temperature when evaluating permanent deformation resistance of FASB.

Kim *et al.* (2009) performed uniaxial RLPD tests on laboratory prepared foamed treated RAP from seven different sources in Iowa. The tests were performed at a 20 psi deviatoric stress for 10,000 cycles with a 0.1 second loading and 0.9 second rest period at 104°F. In agreement with previous studies, they found that increasing the foamed asphalt content increased the susceptibility to permanent deformation.

Permanent deformation resistance is strongly influenced by confining pressure. A realistic characterization methodology is triaxial testing with stress states representative of field condition. Therefore, triaxial RLPD testing at field stress levels and the critical temperature condition was employed for permanent deformation characterization of the FASB materials in this study.

TEST SPECIMENS

The test mixtures in this study corresponded to Mix A and Mix H described previously in Chapter 2. Mix A consisted of a blend of 40% RAP-1 and 60% RC mixed with 2.8% foamed asphalt while Mix H was a 100% RAP-3 mixed with 2.2% foamed asphalt.

All test specimens for Mix A were extracted from field cores. Four inch diameter cores having an average thickness of 4.5 inches were obtained from two sites. The limited height of the field cores made it impossible to fabricate test specimens having the recommended 1.5 height-to-diameter ratio. Nonetheless, it is believed that substandard height specimens of field compacted and cured material provide better insights into the true behavior of the material than is possible from standard height laboratory compacted and cured specimens. Approximate corrections can be made to the test results to compensate for the substandard height-to-diameter ratio.

The first site was a demonstration strip placed in May 2009 at the P. Flanigan and Sons, Inc. production plant in Baltimore, MD. Curing conditions were ideal with clear weather, no rain, and daily average temperatures in the upper 70°F range. The demonstration strip was never covered with HMA and was left exposed to the environment and truck traffic for over two years. The cores from this site, designated F cores, were obtained in August 2011. The bulk specific gravity (G_{mb}) of the cores ranged from 2.109 to 2.112.

The second site was a lane widening section on MD Route 295 near BWI Airport. This project is summarized in Chapter 3; additional details can be found in Appendix F. Construction included a control strip placed in May 2011 followed by main construction (segments A and B) in July 2011. Curing conditions were mostly favorable, with daily average temperatures ranging from the mid 70°F to the low 80°F range and no rain except for a local thunderstorm one night after the placement of the first layer of Segment A. All three segments were covered with 8 inches of HMA on July 18. Cores from this site are designated as M cores. The G_{mb} of M cores ranged from 2.071 to 2.112.

Cores M1, M2, and M3 were obtained from Segment B four months after construction. Core M5, the only intact core from Segment A, was also obtained four months after construction. Cores M4 and M6 from the same section sheared at the interface between the two FASB lifts during coring or during transport to the laboratory. Cores M7, M8, and M9 were obtained from the control strip six months after its construction. All of the M cores were obtained before the paved lane was opened to traffic.

For Mix H, 4 inch diameter by 4.5 high specimens were prepared in the laboratory. These are labeled as I specimens (I1, I2, I3, and I4). The specimens were compacted using a standard Proctor hammer with modifications to the number of blows (75 blows per layer) and number of layers (5 layers) to match the AASHTO T 180-10 modified Proctor compaction energy. This procedure was adopted because of limitations in the facilities available in the laboratory of University of Maryland. After sample preparation, specimens were kept in the mold and cured in an oven at 104°F for 72 hours. The specimens were then extruded from the mold and kept at ambient temperature in the laboratory prior to test. To assure parallel ends for loading, the specimens were capped with sulfur capping compound. The mean G_{mb} of 1.910 for these laboratory prepared specimens was significantly lower than the 2.160 value measured for field cores of the same mixture placed on the I-81 reconstruction project (Apeagyei and Diefenderfer, 2013).

Due to the nondestructive nature of the dynamic modulus test, the same specimen was also used for the subsequent RLPD test. The exceptions to this were specimens F4, F5, F6, and I4, which were tested only for RLPD.

TEST METHODS

Dynamic Modulus

Dynamic modulus testing followed AASHTO TP 62-07 with some modifications to temperatures, loading rates, and confining pressures to reflect the conditions that FASB materials will experience in field. Tests were performed at 41°F, 59°F, 77°F, and 95°F temperatures, 20, 10, 5, 1, 0.5, and 0.1 Hz loading rates, and 0, 7.3, and 14.5 psi confining pressures. Two linear variable differential transformers (LVDT) were used to measure the axial deflection on the sides of the specimen.

A universal test machine (IPC UTM-100) was initially used for the tests. Due to electronic failures in the data acquisition system of this machine, dynamic modulus testing was completed on an MTS test system graciously provided at the FHWA Turner-Fairbank Highway Research Center. The tests on four of the samples were repeated to assess any potential machine-to-machine differences in measured modulus values. In most cases, the agreement between the test results was quite close. Therefore, the test results from the two test machines were treated equally.

At each confinement level, the measured dynamic moduli at different temperatures were shifted with respect to a reference temperature of 77ºF to construct a master curve using standard time-temperature superposition techniques. Equation (2) defines the sigmoidal function (AASHTO TP 62-10) used to describe the loading rate dependency of the modulus:

$$\log|E^*| = \delta + \frac{\alpha}{1+e^{\beta+\gamma \log t_R}} \qquad (2)$$

in which $|E^*|$ = dynamic modulus (ksi)
t_R = reduced time at reference temperature (77ºF)
δ = minimum value of $|E^*|$
$\delta+\alpha$ = maximum value of $|E^*|$
β, γ = shape parameters

The temperature dependency of the modulus is incorporated in the reduced time parameter (t_R) in Eq. (2). Equation (3) defines the reduced time as the actual loading time divided by the time-temperature shift factor, a(T):

$$t_R = \frac{t}{a(T)} \quad (3)$$

in which t = loading time (sec)
$a(T)$ = shift factor as a function of temperature
T = temperature

The shift process is illustrated schematically in Figure 13a. Figure 13b shows the final master curve along with its shift factors. A quadratic polynomial is fit to the log a(T) versus temperature (T) data points:

$$\log a(T) = aT^2 + bT + c \quad (4)$$

in which a, b, and c are the polynomial coefficients.

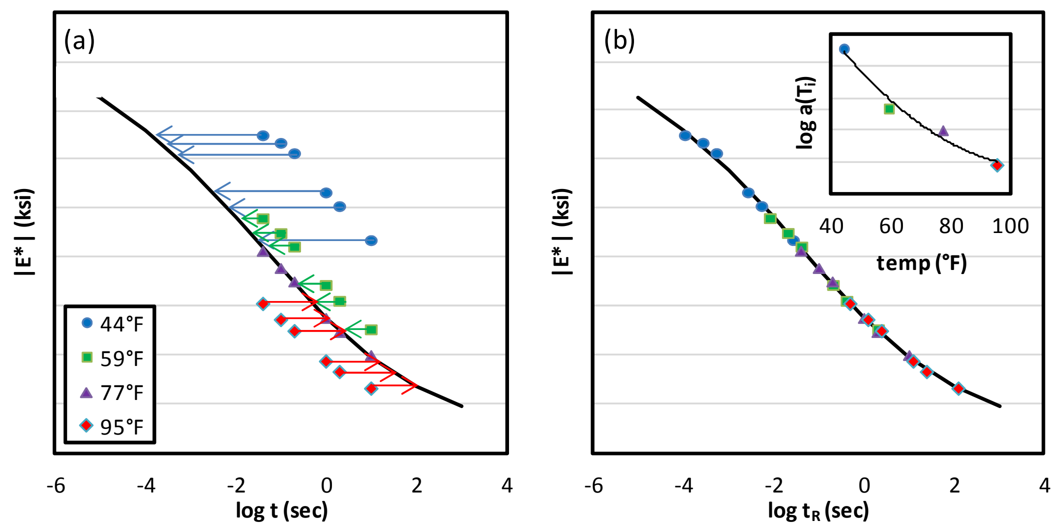


Figure 13. (a) Temperature shifting process for constructing master curve; (b) Master curve and temperature shift factors.

Repeated Load Permanent Deformation

The triaxial RLPD test was originally developed to identify the rutting susceptibility of asphalt mixtures via the flow number. This test has been adapted for FASB mixtures. For the present study, the cumulative permanent strain under repeated load cycles is measured and plotted versus the number of cycles.

The behavior of asphalt mixtures with respect to permanent deformation is influenced by loading rate, stress levels, and test temperature. In this study, an Asphalt Mixture Performance Tester (AMPT) at the FHWA Turner-Fairbank Highway Research Center was utilized and the recommendations from NCHRP 9-30A for HMA were adopted for test stress levels and temperature (Von Quintus *et al.*, 2011).

A temperature of 102.2°F was selected as the intermediate test condition, defined as the average of 68°F and the performance graded (PG) high temperature at 50% reliability obtained from LTPPBind software (Von Quintus *et al.*, 2011). A 0.1 second load time followed by a 0.9 second rest period was employed. A 10 psi confining pressure and a 70 psi repeated deviator stress were applied according to NCHRP 9-30A recommendations for HMA. The test termination points were selected as 10,000 cycles or a cumulative strain of 80,000με, whichever occurred first.

The test stress states used here are expected to be high for a FASB layer below a thick HMA layer and thus can be considered conservative for assessing its permanent deformation susceptibility. However, there are other cases in which FASB may be used beneath a thin layer of HMA, and for these cases the test stress states are considered reasonable. An added benefit of using the NCHRP 9-30A recommended stress states and temperatures is that the rutting susceptibility of FASB can be compared directly to that of HMA.

The accumulated plastic strain curve is generally defined over three regions: primary, secondary, and tertiary (Figure 14). The primary stage is defined as the initial rapid permanent deformation increase with a high but decreasing permanent strain per cycle and is due mainly to densification and rearrangement of the aggregate structure. In the secondary stage, the plastic strain continues to increase but at a decreasing amount per cycle; the secondary stage response typically plots as a straight line on a log-log plot. The tertiary stage, if present, develops when the rate of permanent deformation per cycle increases, leading to a shear failure or flow in the material. Flow number is defined as the number of loading cycles at the beginning of the tertiary stage. The Franken Model captures the primary, secondary, and tertiary stages:

$$\varepsilon_P = AN^B + C(e^{DN} - 1) \tag{5}$$

in which ε_p = permanent strain
N = number of loading cycles
A, B, C, and D= regression constants

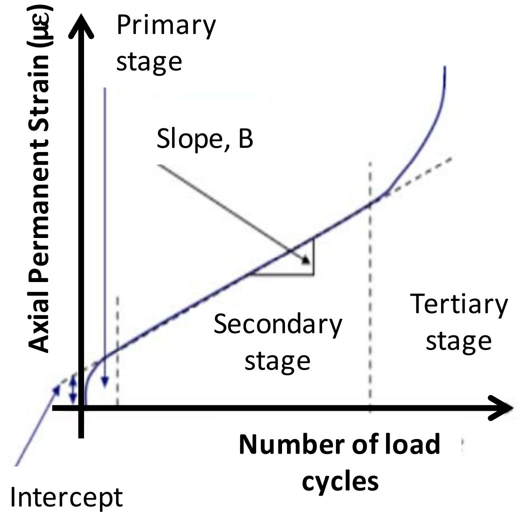


Figure 14. Schematic of the three main stages of the RLPD behavior and the slope and intercept of the secondary stage.

RESULTS

Dynamic Modulus

Triaxial dynamic modulus tests were performed at 3 confining pressures on cores M1, M2, M3, M8, M9, F2, and F3 to assess the effects of confining pressure on dynamic modulus. These tests were all performed using the UTM-100 test system in the University of Maryland Pavement Materials Laboratory.

The extent of loading rate, temperature, and confining pressure dependency varied for the different types of specimens. Sample F2 showed the greatest influence of confining stress. Master curves for F2 at the different confining stresses are shown in Figure 15a. The results suggest a low to moderate sensitivity to confining pressure that is most pronounced at slower loading rates/higher temperatures. At higher temperatures, the foamed asphalt bonds become softer and stress is primarily carried by aggregate skeleton. Therefore, FASB stiffness becomes more like granular soil behavior (i.e. confining stress dependent). The other specimens showed negligible effect of confining pressure on stiffness, even at higher temperatures/slower loading rates. An example of this is the master curves for M1 shown in Figure 15c. Overall, these results suggest that the effect of confining pressure is of secondary importance for FASB in real pavements.

Some deviatoric stress softening effect on measured dynamic modulus was also observed during the calibration stage of the experiment during which the axial load was varied to achieve the target strain of 100 με. However, this effect was even less significant than the confining stress influence.

Since the effect of confining pressure on dynamic modulus was found to be less significant than the effects of loading rate and temperature in these early tests, subsequent tests were performed in a uniaxial mode using the MTS test machine at the FHWA Turner-Fairbank Center. Details of the test results are provided in Khosravifar (2012).

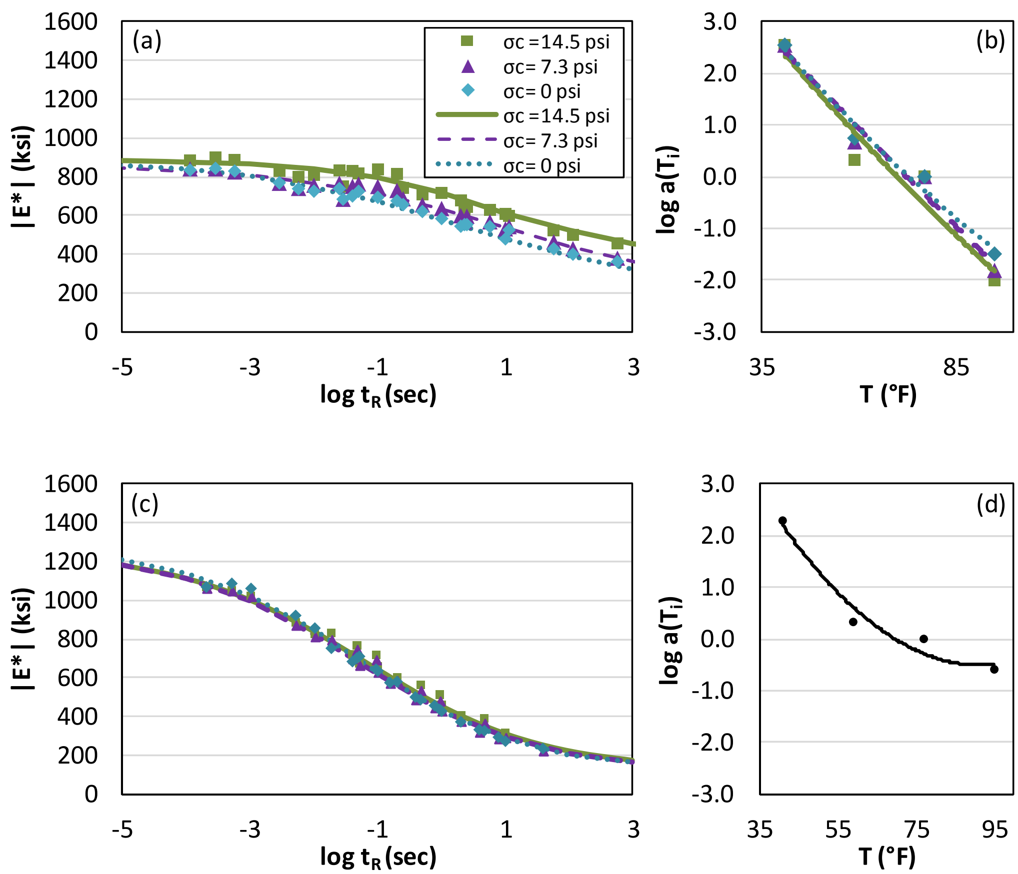


Figure 15. (a) Master curves at different confining pressures for core F2, (b) temperature shift factors for core F2; (c) Master curves at different confining pressures for core M1, (d) temperature shift factors for core M1.

Figure 16a depicts the master curves and temperature shift factors for the M, F and I specimens. The M and F specimens were taken from field cores, while the I specimens were laboratory compacted and cured. At a 77°F temperature and a 10 Hz loading rate (log t_R = -1) typical for base layer conditions in highway pavements, the mean dynamic modulus value for the M, F, and I cores was 740 ksi, 543 ksi and 262 ksi, respectively. The error bars for the I specimens are comparatively small, confirming the lower variability in the laboratory made specimens versus field cores. More significantly, though, the dynamic moduli measured from the I specimens are much lower than those of field cores of Mix A. It is believed that the lower dynamic moduli of I specimens are due primarily to inadequate compaction and curing in the laboratory as compared to field conditions. Apeagyei and Diefenderfer (2013) compared dynamic moduli from laboratory prepared specimens to the indirect resilient modulus (M_r) of field cores for FASB Mix H. Xiao (2009) also investigated the correlation of these two tests. Apeagyei and Diefenderfer found that the laboratory prepared specimens underestimated the stiffness of field compacted and cured cores by an average factor of 2.4. The master curve for the I specimens after correction by this factor are plotted as "corrected I" in Figure 16a, boosting the average modulus at 77°F and 10 Hz loading rate to 630 ksi.

These measured modulus values are likely higher than the true dynamic modulus values because of the substandard height-to-diameter ratios of the specimens obtained from field cores. Witczak *et al.* (1999) found that the dynamic modulus values from test specimens with H/D ratios less than 1.5 may be artificially elevated at low temperatures and high frequencies but little affected at high temperatures and low frequencies. Based on Witczak *et al.*'s results, correction factors of 0.71, 0.85, 0.85 and 1.00 are appropriate for correcting the short-specimen FASB dynamic modulus values at 41, 59, 77, and 95°F, respectively. However, all data presented in the graphs and tables here are the uncorrected actual test results unless specifically stated otherwise. Regardless of these specimen geometry issues, all of the FASB dynamic modulus values are substantially greater than the typical 25 ksi design resilient modulus for GAB material. The upper bound of the FASB dynamic modulus is nearly as high as the typical lower bound for HMA at comparable temperatures and loading rates.

Comparing the F and M cores from the same FASB Mix A, the F cores were less stiff and exhibited less viscoelasticity (i.e., loading rate/temperature dependency) than the M cores (Figure 16a). The lower stiffness could be due to the material at this site being more distressed as it was trafficked by plant trucks for more than two years by the time the cores were obtained. In addition, it was never covered with an HMA layer. This could have caused some oxidization of the foamed asphalt and therefore less loading rate dependency. In contrast, the M cores were obtained after placing the overlying HMA layer but before trafficking. Placement of the HMA layer may have also improved the curing of the underlying FASB by applying additional heat and enhancing moisture evaporation.

Figure 16b shows the variability of the master curves for three different test segments at the MD 295 site (M cores). This variability indicates the effects of construction method and curing conditions on the stiffness of the material. All samples were tested at zero confining pressure and the master curves are constructed based on the average dynamic modulus test results at each temperature and loading rate for each segment. The lowest master curve corresponded to Segment A (only core M5 could be tested) and the highest corresponded to the samples from the Control Strip (cores M7, M8, and M9). Recall that Segment A, which had the lowest stiffness, was placed in two 4-inch lifts at two different times (4 days apart). Rain the night after placing the first lift may also have adversely affected its curing. In addition to the lower dynamic modulus for Segment A, it was significantly more difficult to extract cores from this segment, as the cores would often split on the lift interface. Segments A and B were covered by HMA approximately one week after FASB placement. The Control Strip, which had the highest stiffness, was placed in one 8-inch lift under ideal weather conditions and was covered by HMA two months later, allowing ample time for curing.

Figure 17a shows the master curves and temperature shift factors for all M and F cores combined. The high variability in the results is due to: (1) a substandard height to diameter ratio—a greater height to diameter ratio of 1.5 will reduce the variability by half (Witczak *et* *al.*, 1999); (2) air voids varying between 14.7 to 16.4 for the M cores and between 13.7 to 14.8 for the F cores (G_{mm}= 2.467; Flanigan mix design report) — a one percent change in air voids produces a 2 to 10% change in dynamic modulus and up to a 20% change in the flow number for HMA (Bonaquist *et al.* 2010); and (3) inherent variability of recycled materials; (4) spatial variability in the field cores. Nonetheless, these results provide a rough but practical estimation of what should be expected for the stiffness of this material in the field.

Figure 17b plots the average phase angle (ϕ) for the evaluated cores versus reduced time. The phase angle increased as the temperature and loading time increased. The greatest phase angle measured for the FASB was 22.1° at 77°F, which is lower than what would be expected from HMA under these conditions (typically about 35°). This is due to the fact that FASB contains less new binder than HMA and is therefore less viscoelastic.

Table 12 summarizes the sigmoidal master curve parameters, temperature shift factors and the quadratic temperature shift coefficients for the FASB cores. The sum of the squared errors in log space (Σe^2) provides a goodness-of-fit measure for the sigmoidal function fit.

Dynamic modulus tests were also performed on HMA cores from the MD 295 project. HMA cores were cut to 5.9 inches (H:D 1.5). Specimens M1H, M3H, M5H, M6H, M9H survived coring and were tested using the AMPT machine at the FHWA Turner-Fairbank Highway Research Center according to AASHTO TP 79. The average master curves for the MD 295 FASB cores after applying the appropriate geometrical correction factors (Witczak *et al.*, 1999) are compared to those from MD 295 HMA cores in Figure 18 at the reference temperature of 77^{o}F. The FASB cores exhibit less loading rate/temperature dependence as compared to the HMA cores. The FASB cores are more stiff than the HMA cores at slow loading rates/high temperatures and less stiff at fast loading rates/low temperatures. The FASB and HMA stiffness values are roughly comparable at intermediate loading rates/temperature (t_R=0.1 sec). Table 13 summarizes the sigmoidal master curve parameters, temperature shift factors and the quadratic temperature shift coefficients for the HMA cores. Figure 18b compares the phase angles for FASB and HMA cores from the MD 295 project. FASB cores exhibited lower phase angle than HMA cores as would have expected from the lower viscosity of FASB materials.

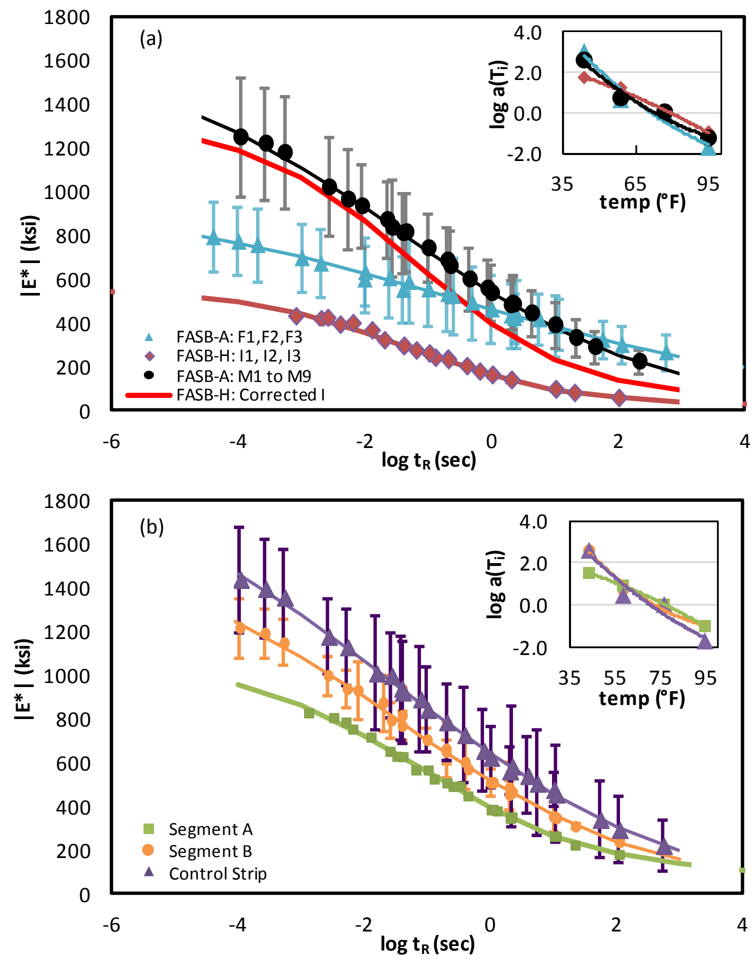


Figure 16. (a) Master curves and temperature shift factors for F and M field cores at zero confining pressure. (b) Master curves and temperature shift factors for M cores from three different test segments at zero confining pressure; error bars demonstrate one standard deviation of |E*|.

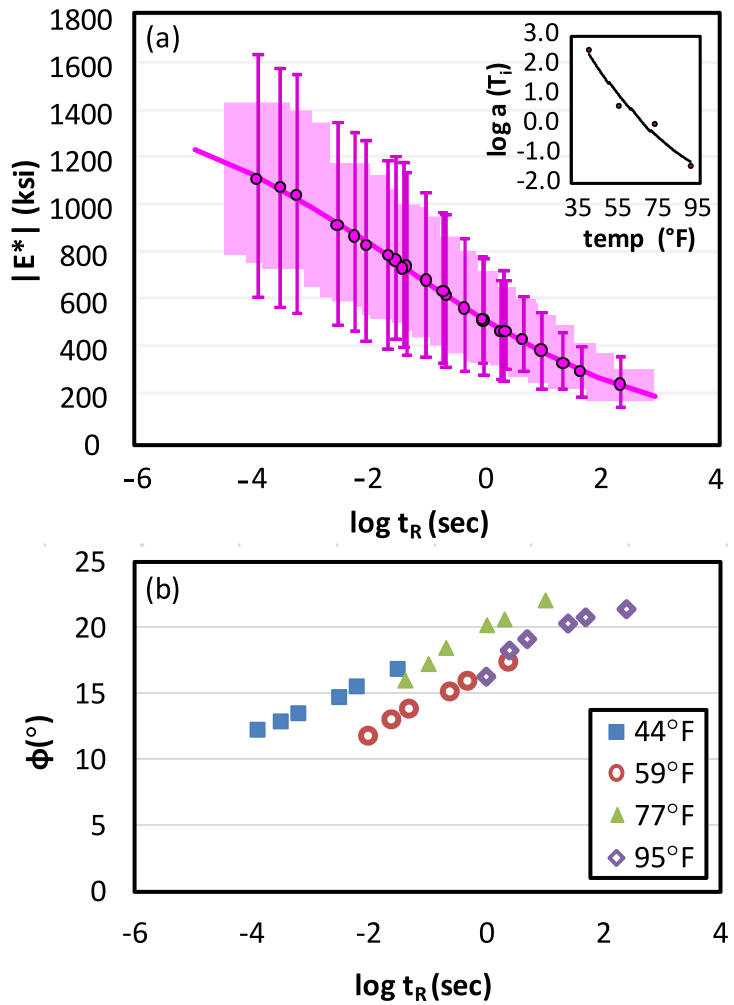


Figure 17. (a) Master curves for all the M and F cores; error bars show the maximum and minimum |E*|, and the shaded area shows +/- one standard deviation. (b) Phase angles for each temperature and loading rate.

Table 12. Regression coefficient for master curve and its temperature shift factors for different mixtures.

	(M and F cores)	(M1 to M9)	(F1,F2,F3)
Σe^2	7.498E-04	7.856E-04	2.236E-03
δ	1.6544	1.3972	1.2345
α	1.5287	1.8405	1.8667
γ	0.3885	0.3936	0.2324
β	-0.8094	-0.9771	-1.1644
$a(T_1)$ 44°F	339.29	339.29	339.43
$a(T_2)$ 59°F	4.37	4.41	3.90
$a(T_3)$ 77°F	1	1	1
$a(T_4)$ 95°F	0.04	0.05	0.03
log $a(T_1)$	2.53	2.53	2.53
log $a(T_2)$	0.64	0.64	0.59
log $a(T_3)$	0	0	0
log $a(T_4)$	-1.38	-1.33	-1.56
a	0.0006	0.0006	0.0005
b	-0.1526	-0.1574	-0.1393
c	7.9836	8.1202	7.6059

Table 13. Regression coefficient for master curve and its temperature shift factors for HMA cores.

	HMA cores
Σe^2	1.384E-04
δ	0.2026
α	3.3589
γ	-0.4261
β	1.2575
$a(T_1)$ 39°F	189.64
$a(T_2)$ 70°F	1.00
$a(T_3)$ 77°F	1.00
$a(T_4)$ 104°F	0.01
log $a(T_1)$	2.28
log $a(T_2)$	0.00
log $a(T_3)$	0.00
log $a(T_4)$	-2.29
a	-0.0001
b	-0.0503
c	4.4216

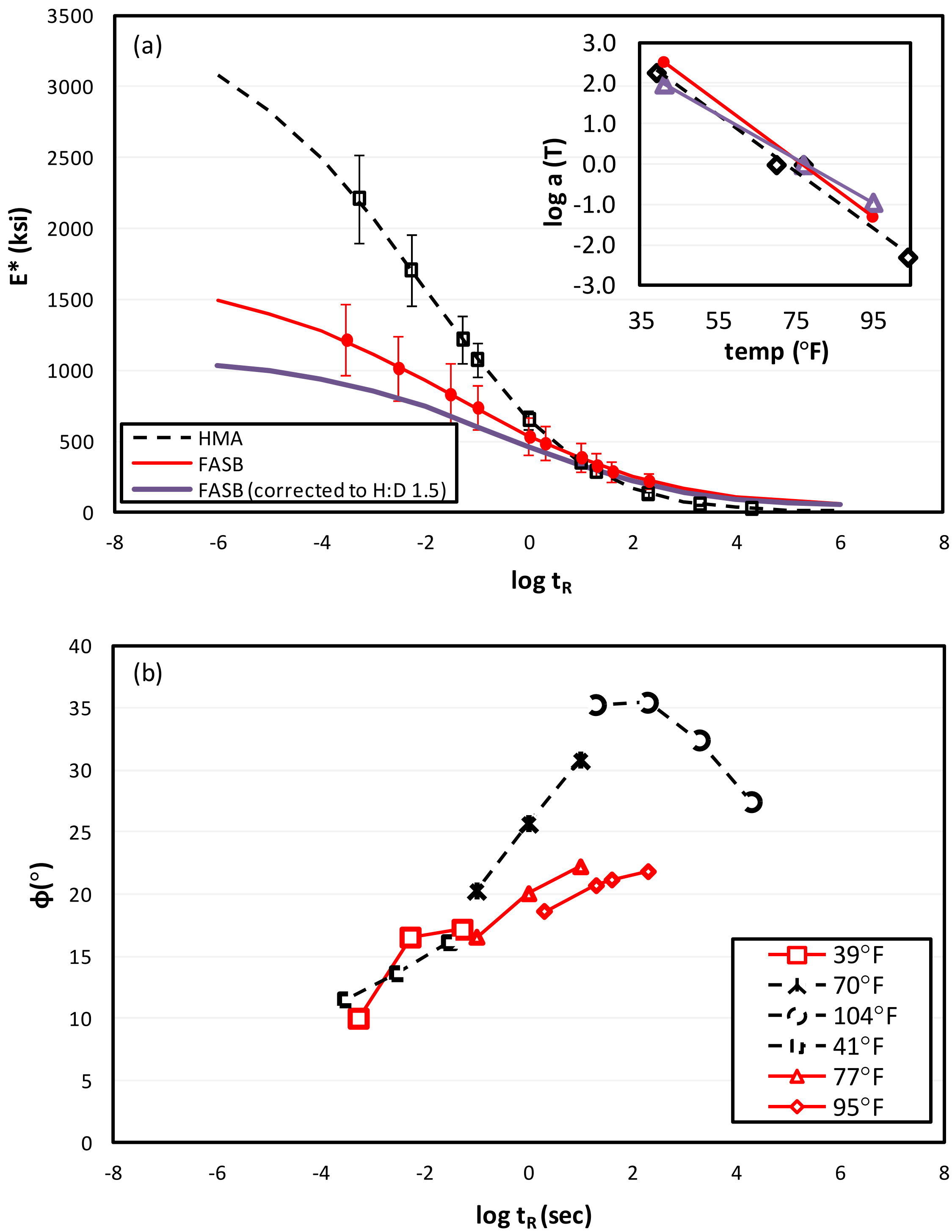


Figure 18. (a) Master curves and temperature shift factors for FASB and HMA cores from the MD 295 project. Error bars show +/- one standard deviation. (b) Phase angles for each temperature and loading rate.

Repeated Load Permanent Deformation

Repeated load permanent deformation (RLPD) tests were performed on the F, M, and I specimens to assess the rutting susceptibility of FASB materials. The substandard H:D ratio for these cores does not have a significant effect on the measured permanent strain in secondary stage according to Witczak *et al.* (1999). It is clear from Figure 19 that only the I3 and I4 laboratory samples of FASB-H reached the tertiary failure stage. The flow numbers as determined using the Franken model are summarized in Table 14. The low flow numbers and high strains at flow observed in these samples suggest a high susceptibility to rutting. However, comparing these results with RLPD tests performed on field cores of the same mixture from the I-81 project (A. Apeagyei, personal communication) reveals that the laboratory prepared samples significantly underestimate the rutting resistance of FASB-H. Even though the test temperature was higher in the RLPD tests on the field cores (129°F versus 102°F), a significantly higher rutting resistance (higher flow number) was observed as compared to the RLPD tests results on the laboratory samples. Table 15 summarizes the regression constants for all the evaluated specimens.

Some findings from the RLPD test results are summarized below:

1. Laboratory prepared I specimens significantly underestimate the rutting resistance of FASB Mix H.
2. None of the M and F cores entered the tertiary stage of permanent deformation.
3. Overall, the F cores exhibited lower permanent deformations after 10,000 cycles than did the M cores.
4. The slopes of secondary stage in log-log space (Table 15) ranged narrowly from 0.10 to 0.16 with an average of 0.12 for all of the evaluated F and M cores.
5. The intercepts of the secondary stage (Table 15) were lower for the F cores (average of 1458 $\mu\varepsilon$) than for the M cores (average of 2550 $\mu\varepsilon$). This may be because this site was older and not covered by HMA and thus was already more compacted under truck traffic.
6. Core M5 showed the highest permanent deformation among the FASB cores from Segment A. This could be partly due to its slightly higher actual test temperature and also because it was obtained from Segment A, where the two 4-inch lifts were placed at different times. Note that this specimen had the lowest dynamic moduli as well.

Overall, the permanent deformation resistance of the FASB was found satisfactory as compared to HMA. At 102.2°F and the same loading conditions (10 psi confining pressure and 70 psi deviatoric stress) the typical range of permanent deformation at 10,000 cycles for HMA is 0.7% to 2% (Von Quintus *et al.*, 2011) while it varied from 0.2% to 1% for the M and F cores. HMA cores from MD 295 (MH cores in Figure 19 and Table 15) showed a steeper secondary stage as compared to FASB M cores at a same test condition. The average permanent strain of HMA cores was 1.14% versus 0.80% for FASB M cores. Moreover, testing the rut susceptibility of FASB under 10 psi confining pressure and 70 psi deviatoric stress recommendations for HMA (Von Quintus *et al.*, 2011) overstates the permanent deformations in FASB for thick pavements, as the FASB is lower in the pavement structure and thus experiences lower stresses in the field than HMA. Given all of this, rutting is not expected to be a concern for FASB mixtures.

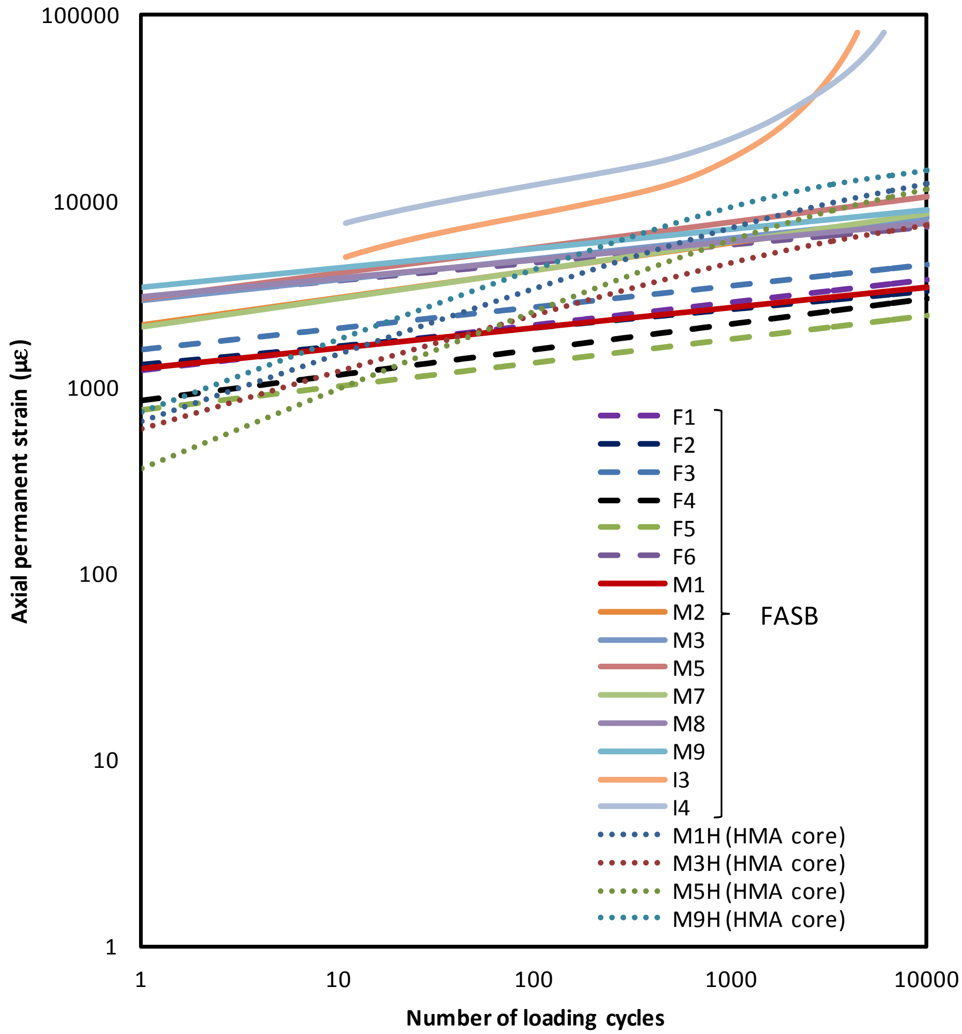


Figure 19. Triaxial RLPD test results FASB cores (M and F), FASB laboratory specimens (I), and HMA cores (MH) in log-log space.

Table 14. Flow number and strain at flow for laboratory made I specimens versus field cores (Mix FASB H)

Test sample	Test temperature (°F)	Flow number	Strain at flow (με)
I3	102	896	16278
I4	102	1342	24879
Laboratory- Average	129	1119	20578.5
Field cores- Average*	129	9013	21900

* Personal communication with A. Apeagyei.

Table 15. Regression constants for Eq. (5).

Regression Constants	A	B	C	D
M1	1258.3	0.1094		
M2	2128.8	0.1492		
M3	2918.4	0.1111		
M5	2985.3	0.1365		
M7	2081.5	0.1539		
M8	3025.4	0.1006		
M9	3450.4	0.1044		
F1	1239.2	0.1219		
F2	1316.7	0.0986		
F3	1576.3	0.116		
F4	854.95	0.1566		
F5	752.4	0.1267		
F6	3008.9	0.0954		
I3	422.7	0.7801	309423	-3E-04
I4	536.4	0.6884	170183.6	-3E-04
M1H	660.5	0.3599	10554.7	-8E-05
M3H	591.9	0.3108	5133.41	-8E-05
M5H	364.8	0.4296	16680.9	-6E-05
M9H	741.7	0.3887	21825.5	-8E-05

CONCLUSIONS FROM LABORATORY EVALUATION

Triaxial dynamic modulus and RLPD tests were performed on field cores of a FASB Mix A and laboratory compacted and cured specimens from Mix H. Comparing the performance test results from laboratory made specimens of Mix H (I specimens) to those reported by Apeagyei and Diefenderfer (2013) on field cores showed that compaction and curing of FASB cannot be well simulated in the lab. The laboratory compacted and cured specimens underestimated the modulus and rutting resistance of FASB Mix H significantly.

Influence of deviatoric stress and confining pressure was found to be negligible as compared to the effects of loading rate and temperature on the measured stiffness values. Cores from P. Flanigan and Sons demonstration strip (F cores) that were directly exposed to the environment and truck traffic for more than two years had lower dynamic moduli with less loading rate dependency and more confining pressure dependency as compared to cores from the MD 295 site (M cores) where the FASB layer was cured and covered with HMA and the cores were obtained before opening the site to traffic. At a 77 °F temperature and a 10 Hz loading rate typical for base layer conditions in high volume highway pavements, the mean value of dynamic modulus ± one standard deviation for the M and F cores was 629 ±134 and 462 ±145, respectively. The lower dynamic modulus limit is substantially greater than the typical 25 ksi design modulus for GAB material and the upper limit of the measured FASB dynamic modulus is close to the lower bound of HMA at this temperature and loading rate (e.g., a mean modulus of 851 ksi for the HMA cores from MD 295). The effects of improper construction were observed in the measured dynamic modulus from field cores from the MD 295 site. Field cores from Segment A where the material was placed in two lifts at different times (4 days apart) exhibited lower dynamic moduli and higher permanent deformation as compared to Segment B and the Control Strip where the FASB material was placed on the same day.

The permanent deformation resistance of FASB cores from both mixtures was found satisfactory as compared to HMA. This is especially true given that FASB in thick pavement sections will experience lower stress levels in the field as it is placed deeper in the pavement structure than the HMA layer.

CHAPTER 5. PROPERTIES FOR PAVEMENT STRUCTURAL DESIGN

INTRODUCTION

Appropriate structural property values for FASB must be defined if these materials are to be used rationally in pavement design. The required structural properties vary by pavement design methodology. For the MEPDG, the relevant property for asphaltic materials like FASB is the dynamic modulus E* as a function of temperature and loading rate. For the empirical 1993 AASHTO design procedure, the relevant property is the structural layer coefficient, which in turn is a function of material stiffness. Although not explicitly required by either design procedure, the permanent deformation (rutting) resistance is also important.

As described in Chapter 4, past studies from the literature provide little solid guidance on appropriate structural material properties for FASB. This is in part because of incomplete descriptions of the FASB materials, inconsistent testing procedures (e.g., axial E* vs. IDT resilient modulus), and uncontrolled or poorly defined test temperatures and loading rates.

The present study has compiled a wealth of information that is directly relevant to the estimation of structural material properties for FASB materials for Maryland conditions. Sources of information on the stiffness characteristics of FASB include the following:

- Laboratory E* testing performed by the University of Maryland as part of this study
- Laboratory E* testing performed by others on the same materials evaluated in the present study
- Laboratory resilient modulus M_R testing performed by others on materials similar to those in the present study
- Field modulus estimates backcalculated from FWD data
- Modulus estimates from standard empirical correlations with mix design test values (e.g., IDT strength)

The empirical 1993 AASHTO design procedure requires that stiffness be converted to an equivalent structural layer coefficient value. Because FASB exhibits behavior sharing characteristics of both HMA and GAB, there are consequently several approaches for estimating the structural layer coefficient values:

- The 1993 AASHTO relationship for a_1 vs. M_R for HMA
- The 1993 AASHTO relationship for a_2 vs. M_R for non-stabilized base materials
- The 1993 AASHTO relationship for a_2 vs. M_R for bituminous stabilized base materials
- The Wirtgen (2012) empirical relationships for structural layer coefficient

Some of these approaches will provide higher quality estimates than others. In all cases, though, there will be a range of moduli and layer coefficient values and thus some judgment will be required to determine appropriate values for design.

The amounts and types of information from this study that are suitable for estimating modulus and structural layer coefficient differ for the two major types of FASB evaluated. The largest amount of data is available for the 40%RAP+60%RC material (Mix A from Chapter 2) that was studied extensively in the field at the MD 295 project site. More limited data are available for the 100%RAP material (Mix G from Chapter 2), although the results for the 40%RAP+60%RC can be adjusted to provide estimates. Because of these issues, the two types of materials are evaluated separately in the following sections.

40%RAP+60%RC

Stiffness

The MEPDG ideally requires input of the dynamic modulus master curve (E* as a function of reduced loading time/frequency) and temperature shift relationship for asphaltic materials. For HMA, this information can be based on laboratory-measured values (Level 1 inputs) or on empirical relations that are functions of the binder stiffness/grade and mixture volumetrics and gradation (Levels 2 and 3). These empirical relationships, however, are not valid for FASB.

Although project-specific Level 1 E* inputs for FASB can be measured in the laboratory, it is unlikely that this will be done in practice. The E* master curve data reported in Chapter 5 provide some guidance for inputs, but only for materials that closely match the conditions in these tests.

A fallback position for FASB is to ignore the temperature and loading rate dependence of stiffness and to instead use a fixed value at a representative temperature and loading rate. This can be justified in part because the FASB layer is typically lower in the pavement structure where temperature and loading rate variations are less severe than at the surface. Reasonable temperature and loading time values for moderate to high speed roads in Maryland are 70^{o}F and 0.1 seconds (10 Hz), respectively. Dynamic modulus values at this temperature and loading rate can be treated as approximately equal to the resilient modulus.

The estimated range of modulus values for the 40%RAP+60%RC (Mix A) material at 70^{o}F and a 0.1 second load time are summarized in Figure 20. Explanations for the various sources of the modulus estimates are given below. Note that an attempt has been made to characterize qualitatively the various estimates as either high, medium, or low confidence.

UMD E*: These are the values measured in this study as reported in Chapter 4 of this report. The laboratory E* testing was performed on FASB cores extracted from the MD 295 test sections in November 2011 just prior to opening to traffic and after 4 to 6 months of field curing. The values thus represent the stiffness of field placed and field cured material. The measured dynamic modulus values at 70^{o}F and a 0.1 second loading rate ranged from 600 to 900 ksi. Note that the 4 inch diameter cores generally had less than the standard 1.5 height-to-diameter ratio, which results in an overestimate of dynamic modulus (Witczak *et al.*, 1999). Consequently, these measured stiffness values are categorized as medium confidence.

UMD E* (H:D Corrected): As described in Chapter 4, the UMD E* values can be corrected for the nonstandard height-to-diameter specimen geometry using the results from Witczak *et al.* (1999). This reduces the measured dynamic modulus range to 475 to 725 ksi. These corrected measured stiffness values are categorized as highest confidence.

Bonaquist E*: P. Flanigan and Sons, Inc. provided FASB samples to Advanced Asphalt Technologies LLC for characterization of dynamic modulus and estimation of structural layer coefficients. The results from this study were reported by Dr. Ramon Bonaquist, the Chief Operating Officer of AAT, in a 2 August 2011 letter report to Mr. Tom Norris of P. Flanigan and Sons, Inc. Mr. Norris provided a copy of this letter report to the UMD team. The test samples included field cores from the test strip at the Flanigan production facility in Baltimore and laboratory specimens prepared using a gyratory compactor. Dynamic modulus was measured by AAT under indirect tension loading. Only the results from the field cores corresponding to field placed and field cured conditions are included in Figure 20. The measured dynamic modulus values at 70^{o}F and a 0.1 second loading rate ranged from 660 to 790 ksi. These values are at the high range of dynamic moduli measured by UMD from uniaxial cylindrical specimens. As described earlier in Chapter 4, previous researchers have found that the indirect tension loading employed by AAT tends to give elevated values for dynamic modulus. Consequently, the AAT stiffness values are categorized as medium confidence.

FWD M_R: These values are the raw resilient modulus values backcalculated from the FWD testing conducted on the MD 295 test sections in November 2011 just before opening to traffic and after 4 to 6 months of field curing. Details of the FWD backcalculation are given in Chapter 3. The backcalculated M_R values ranged from 190 to 540 ksi. Note that no field-to-laboratory correction has been applied to these values. As a consequence, these stiffness values are categorized as only medium confidence.

FWD M_R (Corrected): An attempt was made to apply a field-to-laboratory correction to the M_R values backcalculated from the FWD results. The 1993 AASHTO design guide recommends field-to-laboratory correction values for a variety of pavement scenarios. Unfortunately, none of these scenarios corresponds precisely to a stabilized aggregate base beneath an HMA layer. The closest matching scenario is a non-stabilized aggregate base beneath an HMA layer, for which the field-to-laboratory correction factor is 0.62. The reduced backcalculated M_R values range from 120 to 335 ksi. However, because of the uncertainty in the appropriate field-to-laboratory correction factor, these stiffness values are characterized as lowest confidence.

IDT Strength (Wirtgen, 2004): Figure 21 from the original version of the Wirtgen Cold Recycling Manual (Wirtgen, 2004) is a nomograph relating measured dry IDT strength for the mix design to the initial field stiffness after curing. Using the measured IDT strength range of 65 to 85 psi for Mix A from Chapter 2, the estimated field stiffness values from this nomograph range from 360 to 570 ksi. Although the basis for the Wirtgen nomograph is not stated, these results are nonetheless characterized as medium confidence because of the wide acceptance in practice of the Wirtgen approach.

As summarized in Figure 20, there is considerable divergence in the spread of measured and estimated stiffness values from these various approaches. Using engineering judgment with qualitative weighting for confidence level in the results, 250 to 800 ksi appears to be a reasonable range for the stiffness values for the 40%RAP+60%RC FASB under field cured conditions. This corresponds to a lower bound of 250 ksi, a mean value of 525 ksi, and a lower quartile break (assuming a uniform distribution) at about 385 ksi.

Note that these stiffness estimates are based on initial conditions after field placement and several months of field curing. The initial stiffness of all materials in the pavement may change over the service life of the pavement. Examples of this include stiffness deterioration caused by aging, stripping, and/or cracking of HMA and aggregate breakdown and/or fines contamination of aggregate bases. There is little information available on the stiffness trends of FASB over service life. The nomograph from Wirtgen (2004) implies a 40 to 50% loss in FASB stiffness in the long term (Figure 21). Long and Theyse (2004) observed decreasing stiffness with traffic in heavy vehicle simulator tests in South Africa, while Loizos (2007) observed increasing stiffness with traffic over the first year at field test sites in Greece. Halles *et al*. (2013) explained this apparent inconsistency in terms of the stress ratio, the ratio of applied stress to strength; the very lightly surfaced South African pavement sections had high stress ratios while the more heavily surfaced Greek pavements—closer to US practice—had low stress ratios.

Longer term data are provided by Lane and Kazmierowski (2012), who monitored annually for 10 years a full-depth reclamation project on the Trans Canada Highway in Ontario. They found no evidence of FASB degradation; IRI remained below 60 inches/mile, PCI remained above 85, and FWD normalized deflections decreased during the monitoring period. The backcalculated resilient modulus of the FASB was approximately 200 ksi at the end of 10 years. Overall, the FASB section delivered superior performance as compared to a companion control section.

In both the 1993 AASHTO and the MEPDG, the inputs to new pavement design are based on the initial material properties (e.g., initial dynamic modulus master curve, initial resilient modulus) and do not explicitly consider the effects of material degradation over time (except for overlay design). It is therefore consistent to estimate the FASB structural design properties using the initial material stiffness.

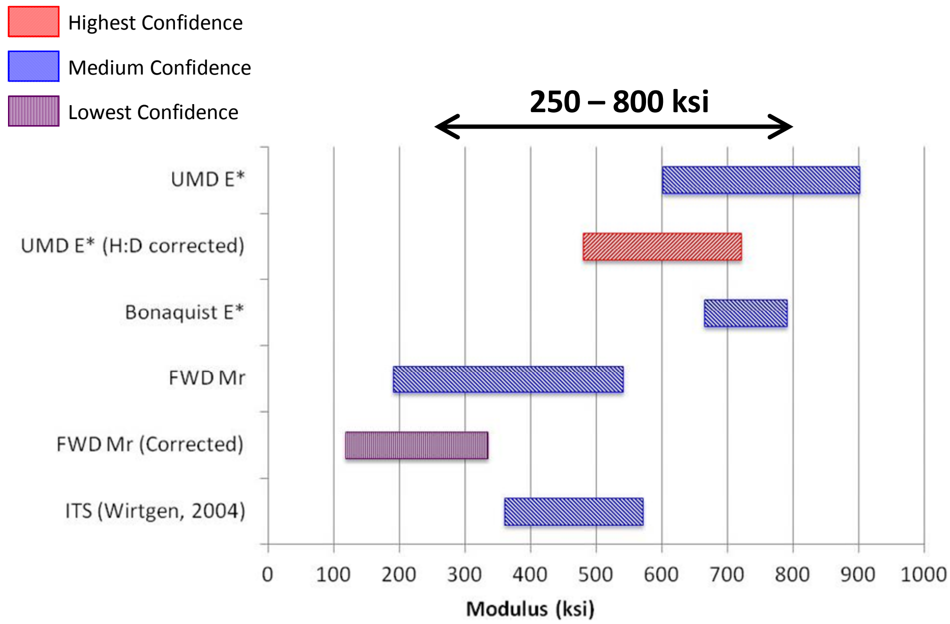


Figure 20. Estimated modulus ranges for 40%RAP+60%RC FASB (Mix A). Dynamic modulus E* values are at 70^{o}F and 0.1 Hz, approximately equivalent to resilient modulus M_R conditions.

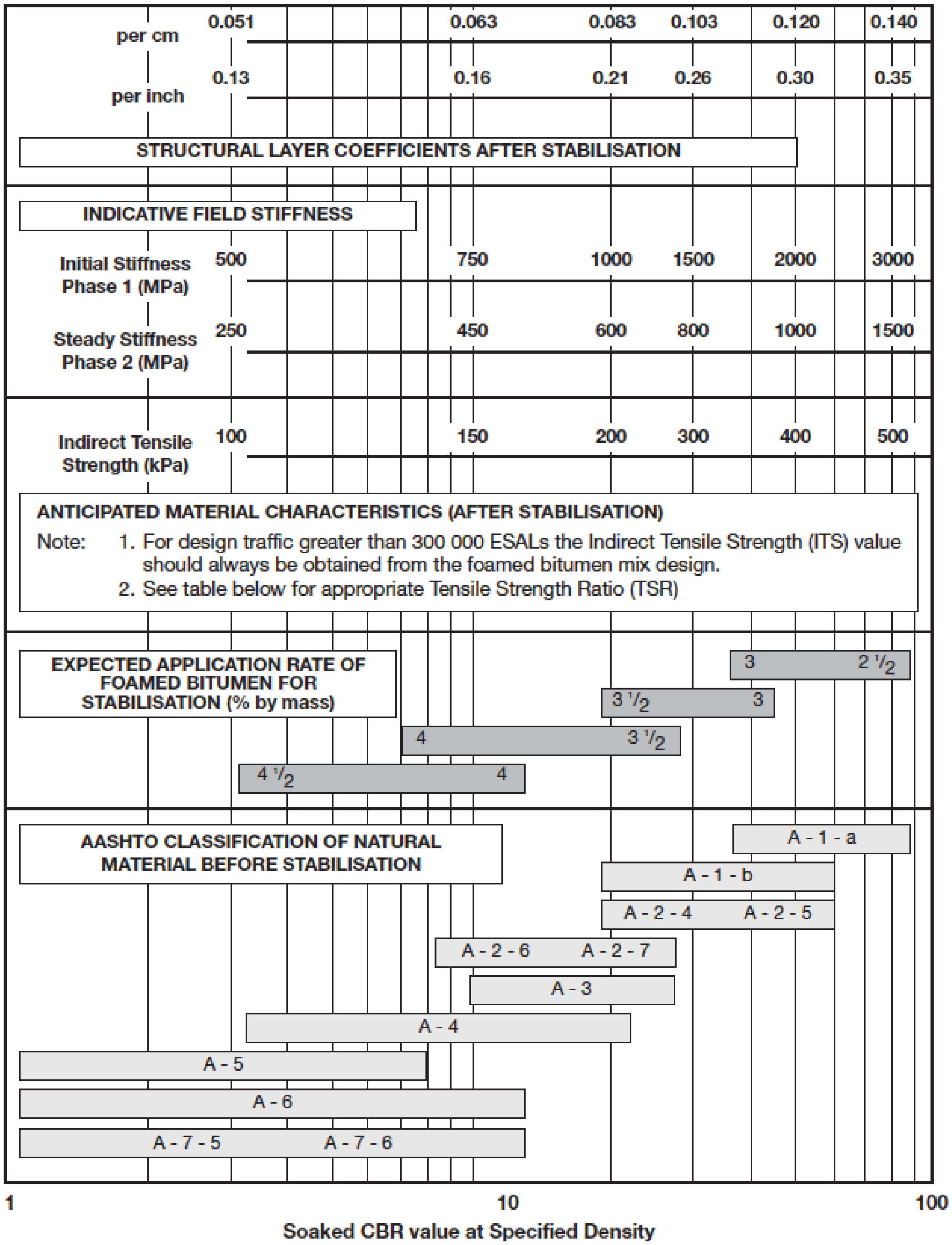


Figure 21. Suggested field stiffness and structural layer coefficient values for FASB (Wirtgen, 2004).

Structural Layer Coefficient

The empirical 1993 AASHTO pavement design guide characterizes the structural capacity of materials in terms of a structural layer coefficient. This layer coefficient in turn is a function of the material stiffness. The FASB stiffness range of 250 to 800 ksi inferred from the data in Figure 20 is therefore used to estimate the corresponding ranges for the structural layer coefficient using various approaches. These are summarized in Figure 22. Explanations for the various sources of the layer coefficient estimates are given below. Note that an attempt has been made to characterize qualitatively the various estimates as either high, medium, or low confidence.

AASHTO a_1 (no upper bound): The FASB here is considered to be a weak HMA-like material characterized by the layer coefficient a_1. Figure 23 shows the nomograph from the 1993 AASHTO pavement design guide that relates a_1 to HMA resilient modulus at 68°F. This relationship can be approximated by the following equation:

$$a_1 = 0.1665 * \ln(M_R) - 1.7309 \tag{6}$$

in which the resilient modulus M_R is in psi. Substituting the stiffness range of 250 to 800 ksi from Figure 20 into Eq. (6) yields a structural layer coefficient range of 0.35 to 0.53. The lower end of this range is typical for base course HMA and thus is considered a plausible value for FASB. The upper end of this range exceeds the highest values conventionally assigned to HMA. Although Davis and Timm (2011) suggest that the conventional upper bound may not be valid and that a_1 may in fact be as high as 0.58 for HMA, only medium confidence is assigned to the estimates from Eq. (6).

AASHTO a_1 (with upper bound): As indicated in Figure 23, there is uncertainty in the trends of a_1 vs. M_R for M_R values greater than about 450 ksi. As a consequence, an upper bound of 0.44 is typically used for a_1 in the 1993 AASHTO pavement design procedure. Applying this upper bound to the stiffness ranges from Figure 20 yields a structural layer coefficient range of 0.35 to 0.44. These more reasonable estimates are assigned a high confidence level.

AASHTO a_2 (unstabilized): The FASB here is considered to be a stiff granular base-like material characterized by the layer coefficient a_2. The 1993 AASHTO pavement design guide relates a_2 to granular base resilient modulus E_{BS} via the following relationship:

$$a_2 = 0.249 * \log_{10}(E_{BS}) - 0.977 \tag{7}$$

in which the resilient modulus E_{BS} is in psi. Equation (7) is a semi-analytical relationship based on a substitution ratio concept—i.e., the thickness of lower stiffness material needed to replace a given thickness of a higher stiffness material in order to produce the same pavement response (e.g., surface deflection) under load. Substituting the stiffness range of 250 to 800 ksi from Figure 20 into Eq. (7) yields a structural layer coefficient range of 0.37 to 0.49. These values are, as expected, much higher than the typical value of about 0.12 for unstabilized granular base materials. They also closely match the values from the HMA a_1

nomograph without the upper bound, and thus as before the higher values exceed the conventionally accepted upper bound for HMA. Despite this, these results are assigned a medium confidence level because of the non-empirical semi-analytical basis for Eq. (7).

AASHTO a_2 (stabilized): Figure 24 shows the nomograph from the 1993 AASHTO pavement design guide for estimating the layer coefficient a_2 for bituminous stabilized base layers. The estimated structural layer coefficient range for FASB based on this nomograph is 0.25 to 0.41 (after extrapolating the nomograph trendline). Although at first glance Figure 24 appears to be the most relevant approach for estimating the structural layer coefficient for FASB, the basis for this nomograph is unclear. Appendix X in the 1986 version of the AASHTO pavement design guide (which for this purpose is equivalent to the 1993 version) attempts to describe the basis for this nomograph, but the supporting studies cited in the appendix are short in technical depth and detail. Consequently, these results are assigned only a medium confidence level.

Bonaquist: Dr. Ray Bonaquist of Advanced Asphalt Technologies LLC (2 April 2011 letter report to P. Flanigan and Sons, Inc.) estimated the structural layer coefficients using their measured dynamic modulus values at 70^{o}F and 10 Hz loading rate (0.1 second loading time) and the 1993 AASHTO relationship for the structural layer coefficient of bituminous stabilized base layers (Figure 24). These estimates are assigned a medium confidence level, for the same reasons as described in the preceding paragraph.

Wirtgen (2004): Figure 21 from the original version of the Wirtgen Cold Recycling Manual (Wirtgen, 2004) relates the structural layer coefficient to the dry indirect tensile strength measured during mix design. Using the measured IDT strength range of 65 to 85 psi for Mix A from Chapter 2, the estimated structural layer coefficient values from this nomograph range from 0.34 to 0.36. These estimates are assigned a low confidence because the basis of the nomograph relations is unknown and because the structural layer coefficient approach is not the norm in Germany and many other locations in which Wirgten operates.

There is reasonable agreement in Figure 22 for the spread of measured and estimated structural layer coefficient values from the various approaches. Using engineering judgment with qualitative weighting for confidence level, a reasonable range for the structural layer coefficient values for the 40%RAP+60%RC FASB under field cured conditions is from 0.25 to 0.50. This corresponds to a lower bound of 0.25, a mean value of 0.375, and a lower quartile break (assuming a uniform distribution) at 0.31.

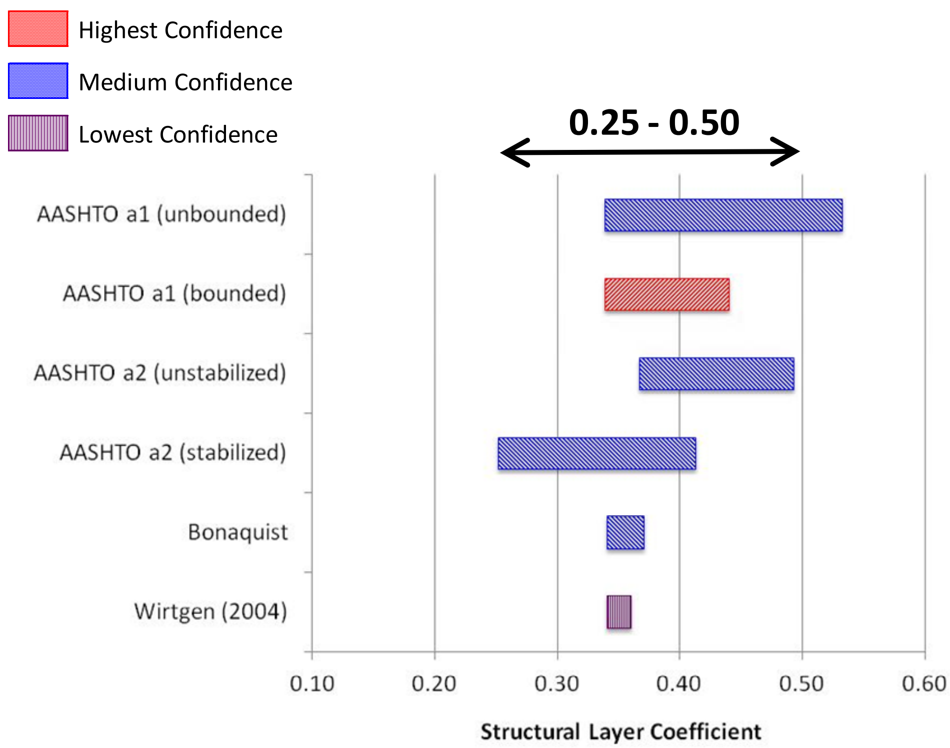


Figure 22. Estimated layer coefficient ranges for 40%RAP+60%RC FASB (Mix A).

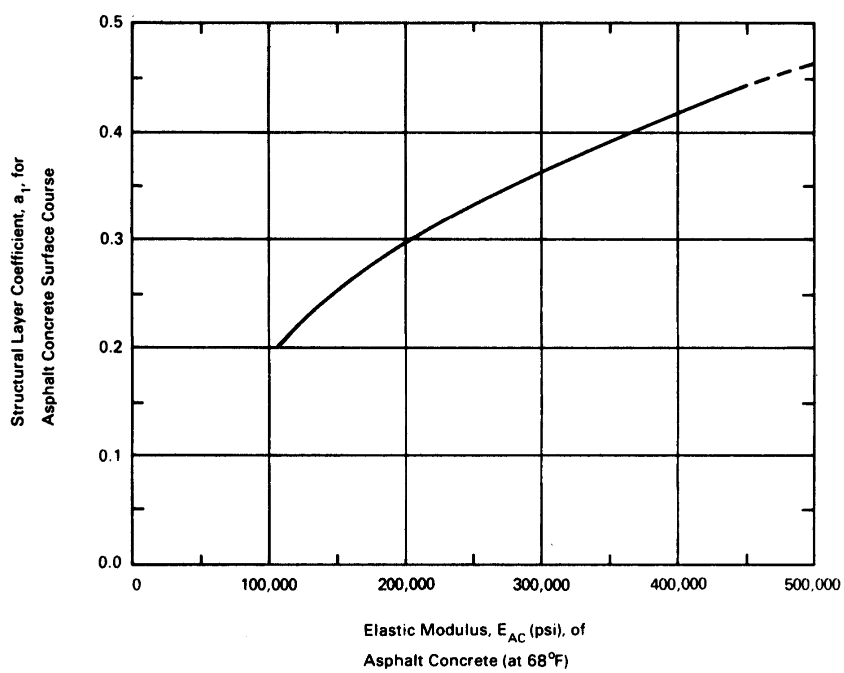


Figure 23. Chart for estimating the structural layer coefficient of dense-graded asphalt concrete based on the elastic (resilient) modulus (AASHTO, 1993).

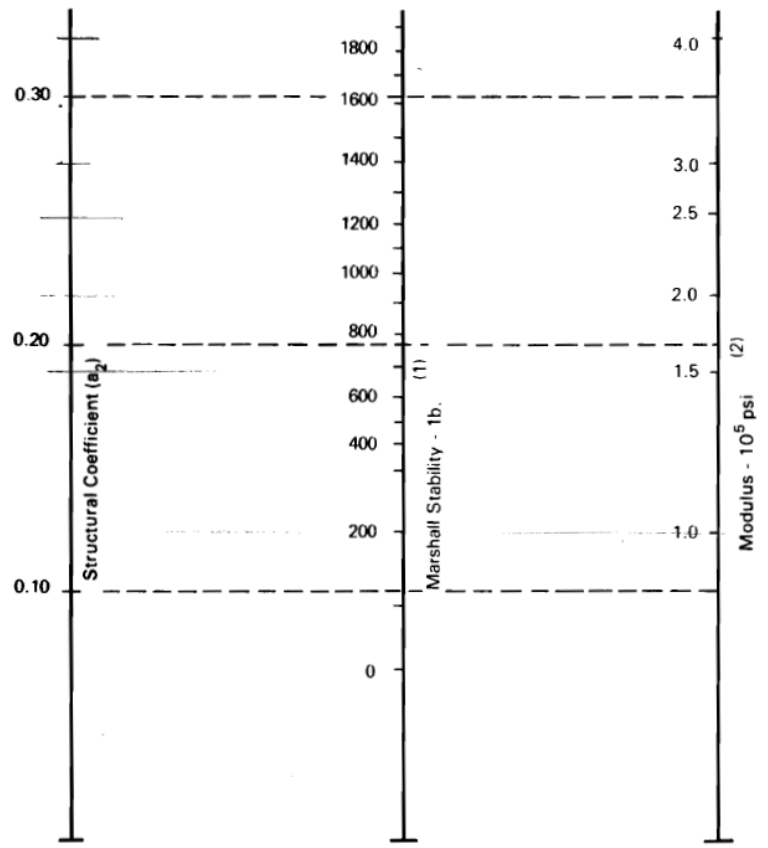


Figure 24. Chart for estimating the structural layer coefficient of bituminous stabilized base (AASHTO, 1993).

Permanent Deformations

Permanent deformation characteristics are not an explicit input in either the 1993 AASHTO empirical or the MEPDG design procedures. Consequently, there are no permanent deformation material properties to characterize. However, some assurance that FASB will not cause excessive rutting is still required. Flow number testing on HMA and FASB cores from the MD 295 test site found that the permanent deformation resistance of the FASB was satisfactory as compared to HMA. The average permanent strain at 10,000 cycles (102.2°F, 10 psi confining pressure, 70 psi deviatoric stress) was 1.14% for the HMA cores and only 0.80% for the FASB cores. In addition, testing the rutting susceptibility of FASB under the 10 psi confining pressure and 70 psi deviatoric stress recommendations for HMA (Von Quintus *et al.*, 2011) likely overstates the permanent deformation potential for FASB. The FASB is usually lower in the pavement structure than HMA and thus experiences lower stresses in the field, at least for high volume pavements. Consequently, rutting is not expected to be a concern for the 60%RC+40%RAP FASB material provided that the mix design meets specifications, it does not have an excessively high foamed asphalt content, and it is allowed to cure adequately in the field.

100% RAP

Stiffness

Much less information is available for the 100%RAP FASB mixture (Mix G in Chapter 2), primarily because no suitable field test sites for this material could be found during the project. However, an estimate of the stiffness of the 100%RAP material can be obtained using the measured stiffness ranges for the 40%RAP+60%RC material and the observation that stiffness tends to vary proportionately with strength. In other words, the ratio of the dry IDT strengths of the 100%RAP relative to the 40%RAP+60%RC material can be used to scale the measured/estimated stiffness ranges for the 40%RAP+60%RC FASB to give reasonable estimates of the stiffness ranges for the 100%RAP material.

As documented in Chapter 2, the dry IDT strength for the 100%RAP material (Mix G) ranged from 63 to 68 psi while the corresponding strength for the 40%RAP+60%RC material (Mix A) ranged from 65 to 85 psi. Using the midpoint strength for each range gives a scaling factor of 0.87—in other words, the stiffness of the 100%RAP material is estimated to be only 87% of the stiffness of the 40%RAP+60%RC mixture.

The estimated range of modulus values for the 100%RAP (Mix G) material at 70^{o}F and a 0.1 second load time are summarized in Figure 25. The UMD E*, Bonaquist E*, and IDT strength entries are based on the measured values for the 40%RAP+60%RC in Figure 20 after reduction by the 87% scaling factor. One additional source of stiffness data for the 100%RAP material is the indirect tension resilient modulus testing performed by the Virginia Center for Transportation Innovation and Research (VCTIR) on cores extracted from the I-81 reconstruction project in Virginia. These results are labeled VCTIR in Figure 25.

As summarized in Figure 25, there is considerable divergence in the spread of measured and estimated stiffness values from the various approaches. Using engineering judgment with qualitative weighting for confidence level in the results, 350 to 750 ksi appears to be a reasonable range for the stiffness values for the 100%RAP FASB under field cured conditions. This corresponds to a lower bound of 350 ksi, a mean value of 550 ksi, and a lower quartile break (assuming a uniform distribution) at 450 ksi.

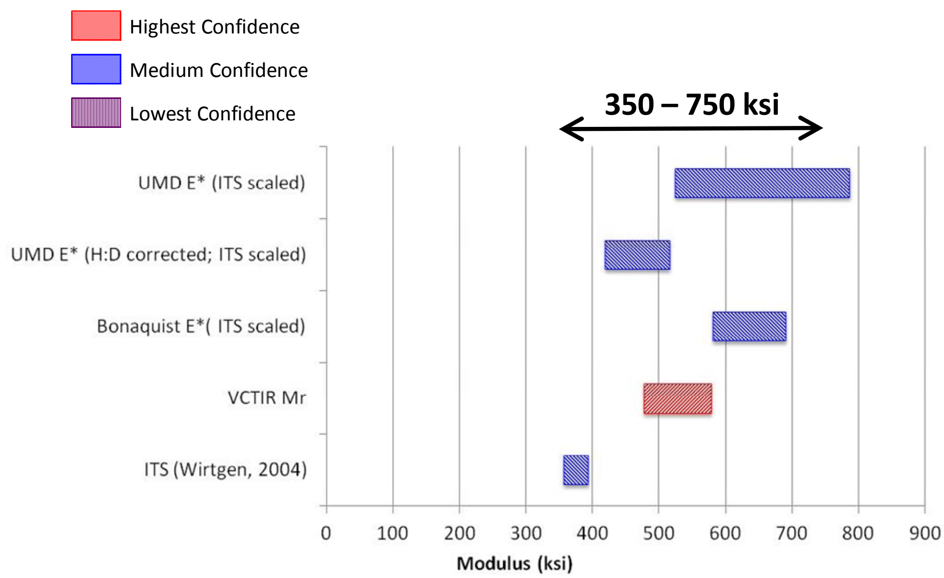


Figure 25. Estimated modulus ranges for 100%RAP FASB (Mix G). Dynamic modulus E* values are at 70°F and 0.1 Hz, approximately equivalent to resilient modulus M_R conditions.

Structural Layer Coefficient

Similar to the procedure followed for the 40%RAP+60%RC material, the FASB stiffness range of 350 to 750 ksi for the 100%RAP FASB from Figure 25 used to estimate the corresponding ranges for the structural layer coefficient using the same approaches as before. These results are summarized in Figure 26. As before, an attempt has been made to characterize qualitatively the various estimates as either high, medium, or low confidence.

There is reasonable agreement in Figure 26 for the spread of measured and estimated structural layer coefficient values from the various approaches. Using engineering judgment with qualitative weighting for confidence level, a reasonable range for the structural layer coefficient values for the 100%RAP FASB under field cured conditions is from 0.30 to 0.48. This corresponds to a lower bound of 0.30, a mean value of 0.39, and a lower quartile break (assuming a uniform distribution) at 0.345.

Permanent Deformations

The arguments in the preceding section regarding rutting susceptibility for the 60%RC+40%RAP material apply equally well to the 100%RAP material. The 100%RAP FASB can be expected to demonstrate satisfactory rutting resistance given a satisfactory mix design, moderate foamed asphalt content, and adequate field curing.

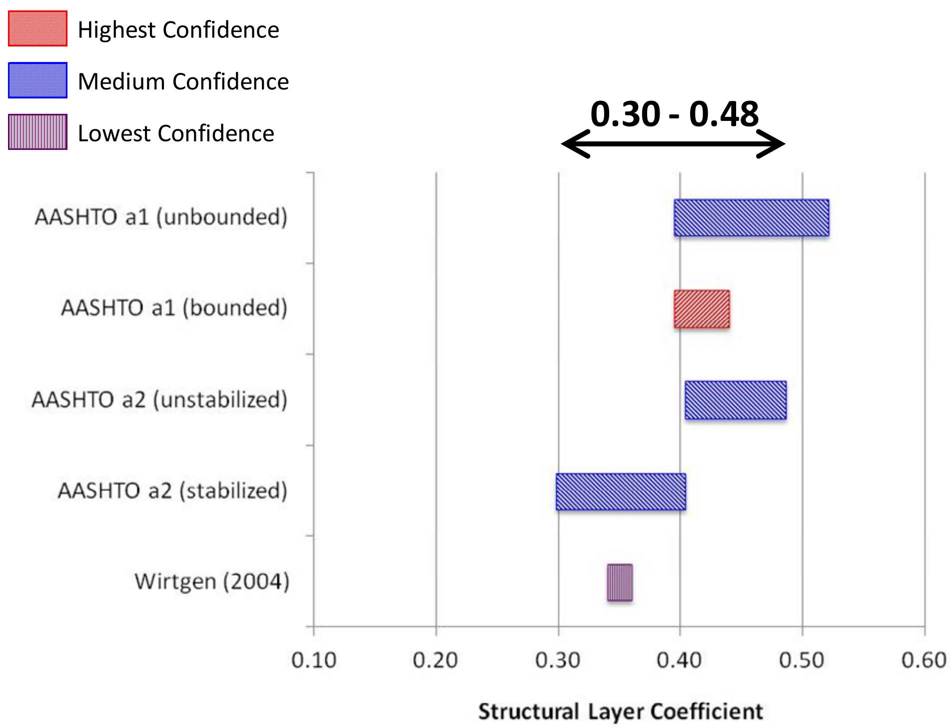


Figure 26. Estimated layer coefficient ranges for 100%RAP FASB (Mix G).

RECOMMENDED STRUCTURAL PROPERTIES FOR DESIGN

Although it is theoretically possible to measure mix-specific dynamic moduli and estimate mix-specific layer coefficient values, in practice this will rarely if ever happen. First, it is very unlikely that laboratory dynamic modulus tests of FASB will be conducted on an individual project basis. Second, even if such tests were to be conducted, the results would not be representative of field conditions since it is difficult if not impossible to replicate field compaction and curing in the laboratory. Consequently, conservative but reasonable default material properties are required for pavement structural design.

The data collected in this project provide a starting point for establishing default structural properties for FASB materials in Maryland. However, the estimated modulus ranges in Figure 20 (40%RAP+60%RC) and Figure 25 (100% RAP) and the corresponding estimates for structural layer coefficient ranges in Figure 22 and Figure 26 are based on mixtures having strength properties similar to those measured for Mixes A and G* in Chapter 2. These mix designs exhibit IDT strength values that are substantially above the lower bound values required by the Maryland SHA provisional specification. Other FASB mix designs may not satisfy the provisional specifications by as wide a margin as the materials evaluated here. Consequently, it is necessary to determine more conservative estimates of moduli and structural layer coefficients that are compatible with the minimum IDT strength thresholds in the specification. These more conservative values can then be used as default design inputs for pavement structural design.

The default structural design properties compatible with minimum IDT strength thresholds can be estimated using the same strength scaling approach employed previously. Table 16 summarizes the dry IDT strength values from the mix designs in Chapter 2 and the estimated structural design properties from Figure 20 and Figure 25 for the 40%RAP+60%RC mixture and from Figure 22 and Figure 26 for the 100%RAP mixture.

For design purposes, the values in columns 2 and 3 in Table 16 must be scaled by the ratio of the minimum dry IDT strength permitted by the specification to the actual IDT strength in column 1 as measured during mix design. The current Maryland SHA provisional specification requires a minimum 50 psi soaked IDT strength and a minimum 70% TSR. The minimum acceptable dry IDT strength can then be determined from this minimum soaked IDT strength at the maximum TSR of 100%. This gives a minimum acceptable dry IDT strength of 50 psi, identical to the soaked value. As will be described subsequently in Chapter 6, there is an argument for lowering the Maryland specification threshold for soaked IDT strength to as low as 40 psi, which by the same argument as above corresponds to a minimum dry IDT strength of 40 psi. Table 16 includes the scaled values of dynamic modulus and structural layer coefficient for a 50 psi IDT strength specification (columns 4 and 5) and the lower 40 psi IDT strength threshold (columns 6 and 7).

Table 16. Scaled moduli and layer coefficient values for pavement structural design (ITS=Indirect Tensile Strength).

	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Material	Mix Design	Measured/Estimated		Spec ITS = 50 psi		Spec ITS = 40 psi	
	ITS – psi[1]	M_R - ksi[2]	a_2[3]	M_R - ksi	a_2	M_R - ksi	a_2
40%RAP+60%RC	65-85 (75)	250-800 (525)	0.25-0.50 (0.38)	168-536 (352)	0.25-0.45 (0.35)	133-427 (280)	0.18-0.42 (0.30)
100%RAP	63-68 (65.5)	350-750 (550)	0.30-0.48 (0.39)	267-573 (420)	0.26-0.45 (0.36)	214-458 (335)	0.22-0.43 (0.32)

[1]From Chapter 2.
[2]From Figure 20 (40%RAP+60%RC) and Figure 25 (100%RAP).
[3]From Figure 22 (40%RAP+60%RC) and Figure 26 (100%RAP).

Note that the scaled moduli and layer coefficient values in Table 16 do not vary greatly between the 40%RAP+60%RC and 100%RAP mixtures. Although the scaled moduli and layer coefficient values in Table 16 are given as ranges, it is arguably overconservative to take the lower bound for each range since the table is already based on the assumption that the FASB strength will be at the specification limit. The mean values with each range are more appropriate default values for pavement structural design. Combining the values for the two mixtures produces in the following recommendations for default structural properties:

- 50 psi IDT strength specification limit: E* = 400 ksi, a = 0.35
- 40 psi IDT strength specification limit: E* = 300 ksi, a = 0.30

Design values for an intermediate 45 psi IDT strength specification limit can be reasonably interpolated from the above.

It is clear from these evaluations that, even with a lowered IDT strength specification limit, FASB has structural properties approaching those of an HMA base mixture. Higher values for dynamic modulus and structural layer coefficient may be warranted if a supplier can demonstrate consistent production of material having IDT strength values well above the specification limit.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

INTRODUCTION

A wealth of information has been compiled in this study with respect to mix design, field evaluation, laboratory testing, and structural material property evaluation. The following sections summarize the principal findings from the study. Some of these findings tie directly back to material in the individual chapters. Others are more integrative findings based on observations made during several phases of the work.

MIX DESIGN

FASB mix designs were developed for eight different mixtures of RAP/RC/GAB of interest in Maryland. An SHA provision specification (Appendix A) for FASB materials was developed by the investigators early in this study to provide a starting point for the mix design study and to guide construction of the field evaluation sections. Five different aggregates, three PG 64-22 binders, and Portland cement additives were investigated. The moist aggregates were mixed with varying percentages of foamed asphalt in the laboratory, compacted into 4 inch Marshall specimens, oven cured, and then tested for indirect tensile (IDT) strength in both dry and soaked conditions. Several ancillary studies of details of the mix design procedure were also performed. Principal findings from the mix design study are as follows:

- The Maryland SHA provisional specification for FASB mix designs (Appendix A) generally agrees with most of the requirements from other agencies.
- The gradation requirements should be revised to require 100% passing the 1.5 inch sieve. This reduction of the coarsest aggregates will have no impact on the mix designs from the current Maryland producers.
- Adding Portland cement adds cementitious bonds and increases unsoaked and soaked IDT strength values. This increase was greater for the 100% RAP (Mix G) than for the 40%RAP+60%RC (Mix A) material.
- For many of the mixes, the IDT strength vs. asphalt content curves did not exhibit the expected concave downward shape. This makes it difficult to determine an optimum binder content. In these cases, it is suggested that the lowest foamed asphalt content that passes all specification requirements be used, but in no case should the foamed asphalt content be less than 2%. An upper bound of 3.5% for foamed asphalt content is also suggested to eliminate any potential for mix instability and rutting potential.
- The soaked IDT strength requirement of 50 psi in the Maryland provisional specification is at the upper limit of other states' practices and is substantially higher than the new FHWA-approved specification in Virginia (minimum dry strength of 45 psi and minimum TSR of 70% corresponding to a minimum soaked strength of 31.5 psi). It is recommended that the Maryland soaked IDT strength requirement be reduced slightly to 45 or 40 psi and that the minimum TSR be kept at 70%. This will still conform to practice in most other states and be achievable by Maryland producers. It will also still produce FASB mixtures with satisfactory structural properties.

Other findings from the mix design studies include:

- 24 hour soaking was more effective than low vacuum saturation but slightly less effective than 72 hour soaking. 24 hour soaking also has practical advantages over both vacuum saturation and 72 hour soaking. For all of these reasons, 24 hour soaking is recommended for mix design testing.
- The mixing moisture content was not found to have a significant impact on asphalt dispersion in the low fines mixtures typical for Maryland producers. The tensile strength is mainly affected by the conventional moisture-density behavior of the granular material. Consequently it is recommended that the mixing moisture content be near the optimum moisture content for compaction.
- A limited laboratory evaluation suggests that stockpiling FASB for even just a few days after production may cause a reduction of soaked IDT strength by up to 30%. More work should be done to confirm this preliminary finding. In the interim, stockpiling should therefore either be strictly curtailed in the specification or, alternatively, mixtures that will be stockpiled should be required to satisfy a higher soaked IDT strength requirement during mix design.

FIELD EVALUATION

A major objective of this study was to evaluate the in-place properties of FASB materials, and in particular the increase in in-place stiffness gain during curing. Several attempts to do this during the early stages of the project failed for a variety of reasons. However, a successful field evaluation at the MD 295 lane addition project near BWI airport provided a wealth of high quality data. Field tests included GeoGauge, lightweight deflectometer (LWD), falling weight deflectometer (FWD), nuclear density gauge, and field moisture content testing. Principal findings regarding the stiffness gains of FASB during curing are as follows:

- The Zorn LWD and GeoGauge devices gave significantly different values for the *in-situ* stiffness, with the Zorn LWD systematically reporting values approximately 0.5 times those from the GeoGauge at the same locations. The reasons for these differences include different load levels, loading rates, depth of zones of influence, analysis assumptions, and other factors. Given these issues, neither of the devices can be considered to give the "true" in place stiffness. The more useful measures are the percentage increase in stiffness with time and the relative stiffness of the FASB versus conventional GAB.
- Curing of the FASB in the Control Strip and mainline Segment B placement produced stiffness increases of 188 to 234% within one week after placement as measured by the GeoGauge and Zorn LWD respectively. The stiffness increases measured using the Zorn LWD tended to be slightly higher than those measured using the GeoGauge. The initial stiffness of the FASB sections (excluding Segment A) was on average 1.4 (GeoGauge) to 2 (Zorn LWD) times the equivalent GAB sections. The gain in stiffness after one week of drying and curing of the FASB sections was also greater than the corresponding stiffness gains for the GAB; the FASB sections (excluding Segment A) increased in stiffness by a factor of 2.9 (GeoGauge) to 3.3

(Zorn LWD) while the GAB increased by a factor of 2.1 (GeoGauge) to 2.9 (Zorn LWD).

- FWD testing conducted 4 to 6 months after paving and just prior to opening to traffic found that the FASB became significantly stiffer than the final GeoGauge and LWD measurements seven days after placement. The long-term stiffness of the field-cured FASB as measured using the FWD was about 295 ksi as compared to 35 to 70 ksi as measured using the LWD and GeoGauge, respectively. The long-term stiffness of the GAB backcalculated from the FWD results was about 24 ksi; the corresponding stiffness of the FASB was 12.3 times that of the GAB. Placement of the HMA layer may have improved the curing of the underlying FASB by applying additional heat and enhancing moisture evaporation.

Other findings during the field study, especially with regard to appropriate construction and QA practices include:

- In situ stiffness test devices are not yet sufficiently mature for use in construction QA of FASB (or GAB) layers. The effects of load level, loading rate, depth of zone of influence, analysis assumptions, and other factors must be better understood. Until then, nuclear density gauge testing remains the best practical approach toward construction QA.
- FASB materials are inherently variable. Consequently, nuclear density gauge results are also variable. Sufficient Proctor compaction testing should be performed in the laboratory prior to construction to enable one-point compaction testing in the field at the beginning of each day of construction as a check of nuclear density gauge readings.
- The bitumen in the RAP and foamed asphalt causes bias in the moisture content reported by nuclear density gauges. Field moisture content should be evaluated independently in order to determine the appropriate moisture offset for input to the nuclear gauge. Sending specimens to the laboratory for moisture content measurement is usually not practical during construction. Moisture content can be determined in the field using microwave drying, but care must be taken to avoid high temperatures in the mixture that may burn off some of the bitumen.
- Breaking the installation of FASB layers into two separate days affected the final stiffness of the material even with rewetting of the surface before placement of the second lift. All lifts should be placed in a single day.
- FWD testing on subgrades and granular base layers is pointless unless the standard 12 inch diameter load plate is replaced with a larger plate. The high stresses under the 12 inch diameter plate cause excessive plastic deformations in the unbound materials.
- Laboratory and field permeability tests found that the permeability of FASB is comparable to and in some cases slightly higher than that of GAB. This suggests that the drainage function of FASB should be comparable to that of GAB.

STRUCTURAL PROPERTY TESTING

Post-construction laboratory testing was conducted on field cores of FASB material from two sites. The emphasis of this testing was on the material properties relevant to pavement structural design, specifically the stiffness and permanent deformation characteristics of the material. Dynamic modulus tests (AASHTO TP 62-07) and repeated load permanent deformation tests (NCHRP 9-30A protocols) were conducted on 7 field cores of FASB taken from the MD 295 test site (M cores), 6 field cores taken from the P. Flanigan and Sons demonstration site (F cores), and 3 laboratory compacted and cured specimens of the 100%RAP FASB used in the I-81 reconstruction (I cores). In addition, 4 sets of tests were conducted on HMA cores taken from MD 295.

At a 77 °F temperature and a 10 Hz loading rate typical for base layer conditions in high volume highway pavements, the mean value of dynamic modulus ± one standard deviation for the M and F cores was 629 ±134 and 462 ±145, respectively. The influence of confining stress on dynamic modulus was slight compared to the influence of loading rate and temperature. The lower dynamic modulus limit measured for the FASB is substantially greater than the typical 25 ksi design modulus for GAB material and the upper limit of the FASB dynamic modulus is close to the lower bound of HMA at this temperature and loading rate (e.g., a mean modulus of 851 ksi for the HMA cores from MD 295). The effects of improper construction were observed in the measured dynamic modulus from field cores from the MD 295 site. Field cores from Segment A where the material was placed in two lifts at different times (4 days apart) exhibited lower dynamic moduli and higher permanent deformation as compared to Segment B and the Control Strip where the FASB material was placed on the same day.

None of the M or F cores entered the tertiary stage of permanent deformation. The laboratory prepared I specimens gave significantly higher permanent deformations, but it is believed that tests on laboratory prepared specimens significantly underestimate the rutting resistance of FASB. The permanent deformation resistance of FASB cores from both mixtures was found satisfactory as compared to HMA. This is especially true given that FASB in thick pavement sections will experience lower stress levels in the field as it is placed deeper in the pavement structure than the HMA layer. However, it is important that the mix designs and field placement conform to specifications and that the foamed asphalt content be limited to avoid mix instability.

PAVEMENT STRUCTURAL DESIGN PROPERTIES

Appropriate structural property values for FASB must be defined if these materials are to be used rationally in pavement design. The required structural properties vary by pavement design methodology. For the MEPDG, the relevant property for asphaltic materials like FASB is the dynamic modulus E* as a function of temperature and loading rate. For the empirical 1993 AASHTO design procedure, the relevant property is the structural layer coefficient, which in turn is a function of material stiffness. Although not explicitly required by either design procedure, the permanent deformation (rutting) resistance is also important.

The present study has compiled a wealth of information that is directly relevant to the estimation of structural material properties for FASB materials relevant to Maryland. Sources of information on the stiffness characteristics of FASB include the following:

- Laboratory E* testing performed by the University of Maryland as part of this study

- Laboratory E* testing performed by others on the same materials considered in the present study
- Laboratory resilient modulus M_R testing performed by others on materials similar to those in the present study
- Field modulus estimates backcalculated from FWD data
- Modulus estimates from standard empirical correlations with mix design test values (e.g., IDT strength)

The empirical 1993 AASHTO design procedure requires that stiffness be converted to an equivalent structural layer coefficient value. Because FASB exhibits behavior sharing characteristics of both HMA and GAB, there are consequently several approaches for estimating the structural layer coefficient values:

- The 1993 AASHTO relationship for a_1 vs. M_R for HMA
- The 1993 AASHTO relationship for a_2 vs. M_R for non-stabilized base materials
- The 1993 AASHTO relationship for a_2 vs. M_R for bituminous stabilized base materials
- The Wirtgen (2012) empirical relationships for structural layer coefficient

Some of these approaches will provide higher quality estimates than others. In all cases, though, there will be a range of modulus and layer coefficient values and thus some judgment will be required to determine appropriate values for design. Given these caveats, the recommended default structural properties are as follows:

- 50 psi soaked IDT strength specification limit: E* = 400 ksi, $a_2 = 0.35$
- 40 psi soaked IDT strength specification limit: E* = 300 ksi, $a_2 = 0.30$

Design values for an intermediate 45 psi IDT strength specification limit can be reasonably interpolated from the above. It is clear from these evaluations that, even with a lowered IDT strength specification limit, FASB has structural properties approaching those of an HMA base mixture.

OVERALL CONCLUSION

The findings from this study clearly confirm the suitability of FASB material for high volume pavement applications if designed and installed properly and cured under favorable climatic condition. The final in place structural capacity of this flexible, partially bound material is substantially higher than unbound GAB and approaches that of base HMA mixtures. Proper use of FASB can reduce pavement cost and help Maryland SHA meet its recycled materials goals.

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