Refinement of Current WisDOT HMA Mixture Application Guidelines Related to NMAS and Aggregate Characteristics

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16. Abstract

Current Wisconsin Department of Transportation (WisDOT) Specifications limit nominal maximum aggregate size (NMAS) of hot-mix asphalt (HMA) to 12.5 mm in the surface layer and 19.0 mm in lower layers. This potentially places unnecessary limits on both construction practice and pavement design. The purpose of this research project was to evaluate the feasibility of expanding WisDOT specifications to include a wider range of aggregate NMAS and lift thicknesses in HMA pavements. In addition to a literature review, a variety of models and laboratory tests were used to determine the likely effects of varying NMAS and lift thickness on various aspects of pavement performance. Based upon this research, it is recommended that WisDOT allow use of 9.5 and 12.5 mm NMAS in HMA surface layers, and 12.5 mm and 19 mm NMAS in intermediate and base layers. Additional recommendations, including restrictions on lift thickness, are also given in the report.

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EXECUTIVE SUMMARY

This report documents the results of WHRP Project 0092-12-01: Refinement of Current WisDOT HMA Mixture Application Guidelines Related to NMAS and Aggregate Characteristics. The background and problem statement for the proposed research is concisely stated in the RFP:

> *…Current specifications restrict the NMAS of the WisDOT standard pavement design structure to 12.5mm in the upper layer over a 19.0mm lower layer, a practice that limits both the products available to designers/contractors and the flexibility to specify NMAS base on intended application…The research will revise and update recommendations made at a national level based on results obtained using Wisconsin materials and conditions.*

Based upon the state of the practice, it appears there is strong evidence to allow the use of 9.5-mm mixes in surface courses in Wisconsin. These mixes should exhibit performance at least as good as that of 12.5-mm mixes, with the possible exception of rut resistance. Allowable NMAS for base course mixes should be extended to 9.5-mm and 12.5-mm mixes, since these mixes should exhibit low permeability and excellent fatigue resistance, both qualities that are essential in base course mixtures. Current requirements for lift thickness to NMAS ratio are generally consistent with recommendations from recent research projects and practices by other states, and probably do not need to be changed, although some consideration should be given to a minimum lift thickness to prevent mats from cooling to quickly and potentially contributing to poor compaction.

Various predictive models were used to analyze the relationship between NMAS and various performance-related properties, including modulus, permeability, layer coefficient and predicted thermal cracking temperature. Predictive models showed little relationship between NMAS and modulus or layer coefficient, suggesting there should be no significant structural issues in using either 9.5-mm or 25-mm mixes in Wisconsin. The predicted permeability of 9.5-mm mixes should in general be somewhat lower than the mixes currently used in Wisconsin. However, the estimated permeability of 25-mm mixes could in some cases be higher than mixes currently in use, and could exceed the maximum value recommended by the National Center for Asphalt Technology. Based upon theoretical stress analyses, resistance to thermal cracking should be similar for 9.5-mm mixes compared to existing 12.5-mm and 19-mm mixes. However, 25-mm mixes could exhibit somewhat higher critical low temperatures.

The laboratory experiment was designed to isolate the effects of NMAS on dynamic modulus by holding other mixture volumetric properties (% AC, VMA) constant, and because of this some of the mixes did not conform to current WisDOT volumetric specifications for HMA. Results indicate that when these volumetric composition is controlled NMAS influences dynamic modulus significantly, particularly for softer binder grades. However, the sensitivity of dynamic modulus to NMAS decreased significantly when the asphalt binder grade was increased to PG 70-28, particularly at lower frequencies. Because the low frequency response is indicative of the aggregate structure, results suggest that this structure is better maintained with an increased binder grade. Furthermore, for softer binder grades selection of larger NMAS does not always result in improved aggregate structure as the dynamic modulus of the 9.5 mm mix was substantially higher than that of the 12.5 mm NMAS. Although some sensitivity of modulus to NMAS was observed in this study, these differences were not large and the number of mixes tested was quite limited. Given that the review of current practice and the results of modeling suggest that there is little or no effect of NMAS on modulus, it is concluded that WisDOT specifications concerning allowable NMAS in HMA applications can be broadened.

The AASHTO MEPDG was also used to analyze the effect of changing NMAS on HMA performance. This analysis largely supported the findings of the review of the state of practice, performance prediction using empirical and semi-empirical models, and modulus testing (in fact, the results of the modulus testing described above were used as input in the MEPDG analysis). In general, little difference was found in the performance of HMA pavements constructed using different NMAS. Some increase in rutting was predicted for 9.5-mm HMA relative to that with larger NMAS, however, this is no doubt directly related to the differences in modulus discussed above, and so the same limitations apply.

Based upon the results of this research, it is recommended that WisDOT specifications allow both 9.5-mm and 12.5-mm NMAS HMA in surface courses, 12.5-mm and 19-mm NMAS HMA in intermediate courses and base courses, and 9.5-mm NMAS HMA in leveling courses. Recommended minimum and maximum lift thicknesses are in general based upon a ration of lift thickness/NMAS of 3 to 5 for fine mixtures and 4 to 5 for coarse mixtures, and a minimum lift thickness of 1.5 inches to prevent unacceptably fast cooling of the mat during compaction. The final recommendations of the report vary somewhat from these guidelines because of practical considerations.

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INTRODUCTION

This report documents the results of WHRP Project 0092-12-01: Refinement of Current WisDOT HMA Mixture Application Guidelines Related to NMAS and Aggregate Characteristics. The background and problem statement for the proposed research is concisely stated in the RFP:

> *Recent research in the area of Super Pave mix design has resulted in expansion of the nominal maximum aggregate size (NMAS) mix designs available and established the relationship between NMAS and pavement performance. The NMAS mixes evaluated in this work ranged from 4.75 to 37.5 mm. This range in sizes coupled with known performance characteristics allows state agencies to design and specify different NMAS mixes based on the intended application of the mix.*
>
> *Current specifications restrict the NMAS of the WisDOT standard pavement design structure to 12.5mm in the upper layer over a 19.0mm lower layer, a practice that limits both the products available to designers/contractors and the flexibility to specify NMAS base on intended application. The aforementioned recent advances in mix design at a national level present a valuable opportunity for WisDOT to promote the use of a wider range of mix types through development of application guidelines for mixes of different NMAS. The research will revise and update recommendations made at a national level based on results obtained using Wisconsin materials and conditions.*

As stated above, many state agencies currently allow Superpave mixes with a wide range of NMAS to be placed in a variety of lift thicknesses. Wisconsin currently severely restricts NMAS and lift thickness in its HMA specification and guidance. This lack of flexibility potentially negatively impacts both the cost and performance of HMA pavements in Wisconsin. By a thoughtful revision of current standards to allow a wider range of NMAS and lift thicknesses, the cost effectiveness of HMA pavement construction in Wisconsin can probably be significantly improved.

However, any revisions in current standards must be done carefully, considering a wide range of mechanisms through which both NMAS and lift thickness could impact pavement performance. The research presented in this report examined five areas of importance in reviewing and revising WisDOT's current standards on NMAS and lift thickness:

1. Air voids, permeability, compactibility and rate of mat cooling
2. Effect of NMAS on modulus and various other performance related properties

3. Pavement performance as indicated by various empirical models and pavement design and analysis programs
3. Effect of NMAS on resistance to segregation
4. Contractor experience with NMAS and lift thickness
5. Experience in neighboring states

This report is divided into six sections:

- Synthesis of Current Practice
- Approach to Laboratory Test Program and Simulations
- Results
- Analysis and Discussion
- ` Conclusions and Recommendations
- References

The Synthesis of Current Practice, which immediately follows the Introduction, is largely based on the document submitted earlier in this project. The other sections are new and summarize experimental work and analysis performed after completion of the Synthesis of Current Practice.

SYNTHESIS OF CURRENT PRACTICE

Current Wisconsin Standards

Current specifications for NMAS and lift thickness for HMA pavements in Wisconsin are contained in two standards, both sections of the document *Roadway Standards*: "Section 460: Hot Mix Asphalt Pavement;" and "Section 14-10-5.6: HMA Mixture Layers." Section 14-10-5.6 contains probably what is the best summary of current standards in Wisconsin governing NMAS and lift thickness for HMA pavements:

> *By the Standard Specifications (standard spec 460.2.2.3), the normal nominal size mixture required for se is as follows (Lower Layer and Upper Layer are defined in standard spec 450.2.1):*
>
> *- Lower pavement layer - 19.0 mm*
>
> *- Upper pavement layer - 12.5 mm*
>
> *- SMA pavement layer - 12.5 mm*
>
> *Based on these nominal sizes and the minimum layer thickness from standard spec 460.3.2 the standard structure results in a 4.0-inch pavement. It is allowed to place a 12.5 mm nominal size mixture for thelower layer, which results in a 3.5-inch pavement. However, a change such as this requires a special provision stating that the lower layer would be a 12.5 mm nominal size mixture.*

The use of a 9.5 mm nominal size mixture is also being allowed for the following applications:

- *In a wedging or leveling layer,*
- *For bridge deck overlays,*
- *In urban situations to aid in matching the curb line and to provide a finer appearing mixture often desired in local paving.*

The 9.5 mm nominal size mixture allows a minimum layer thickness of 1.5 inches to be used. In these applications, the 9.5 mm nominal size mixture may be used for both upper and lower layers, resulting in a 3.0 inch pavement.

Maximum layer thicknesses are also specified in 460.3.2, with values that depend on whether the mix is placed in an upper layer or a lower layer. Maximum thickness for lower layers are 4, 3 and 3 inches for 19-, 12.5- and 9.5-mm mixes, respectively. Maximum thickness for upper layers are 3, 2.5 and 2 for 19-, 12.5- and 9.5-mm mixes, respectively.

Thus, the vast majority of dense-graded HMA pavements in Wisconsin consist of 19-mm base layers and 12.5-mm surface course layers. The "standard" structure using minimum layer thicknesses results in a 4-inch thick pavement; this can be increased to 6 inches if the maximum layer thicknesses are used. As discussed in the introduction, the primary objective of this research is to evaluate whether or not Wisconsin should freely allow the use of 25- and 9.5-mm mixes, and if the specifications on layer thicknesses should be loosened or otherwise modified. If appropriate, such changes in the specification could result in increased efficiency in the design and construction of HMA pavements in Wisconsin, and could have additional benefits in life cycle costs if the changes result in improved performance.

Other Agency Standards

Compilation of agency standards guidelines concerning Nominal Maximum Aggregate Size (NMAS) and Nominal Aggregate Size in states adjacent to Wisconsin and states with large population was completed. The Hot Mix Asphalt (HMA) specifications for nine states have been reviewed. Table 1 and Figure 1 show the maximum and minimum aggregate size specified by different transportation agencies.

Out of the nine states reviewed, four allow the use of asphalt mixtures with maximum aggregate size of 37.5 mm (i.e., Iowa, Michigan, Texas, and New York). Also, Michigan allows the use of mixes with NMAS of 4.75 mm. The majority of the agencies specify a minimum maximum aggregate size of 9.5 mm and a maximum limit in the range of 25.0-37.5 mm. Note that Wisconsin limits/guidelines for aggregate size are between the ranges specified by other major transportation agencies.

Table 1. Aggregate size specification for different states.

State	Max (mm)	Min (mm)
Wisconsin (WI)	19	9.5
Minnesota (MN)*	25	9.5
Iowa (IA)*	37.5	9.5
Illinois (IL)*	25	12.5
Michigan (MI)*	37.5	4.75
Indiana (IN)*	25	9.5
New York (NY)**	37.5	9.5
Texas (TX)**	37.5	9.5
Florida (FL)**	12.5	6.3

*Neighboring **Large population

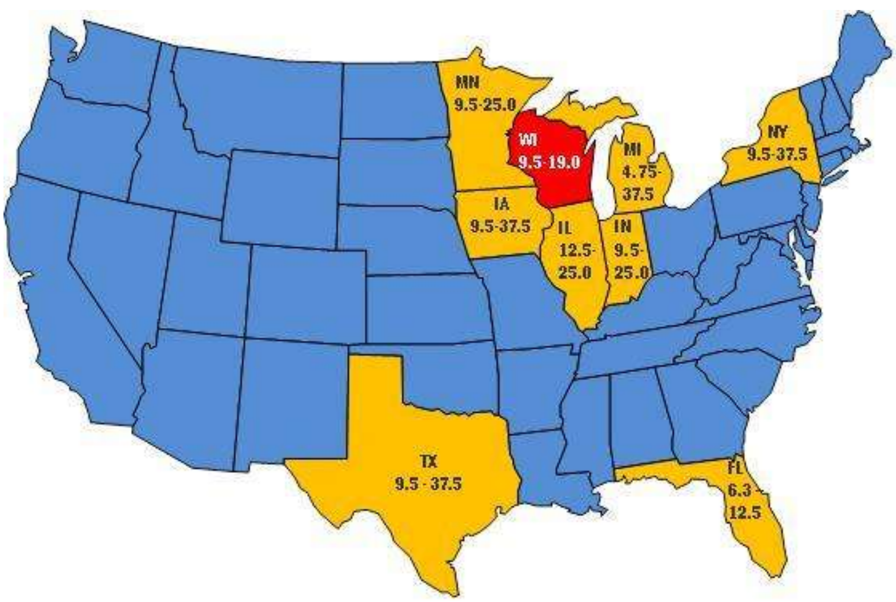


Figure 1. Nominal Aggregate Size Specification [min-max] for Different States.

Bahia and Paye (2001) reported a survey conducted in eight states on the use of different lift thickness/NMAS ratios. As indicated in Table 2, the lift thickness/NMAS ratio varied widely between 2 to 5. The only states clearly specifying lift thickness based on NMAS are Wisconsin (WisDOT specification section 460.3.2), New York (Table 3), and Texas (Table 4).

Table 2. Summary of Survey Results of Lift Thickness /NMAS ratio (Bahia and Paye, 2001).

State	Range of Lift Thickness/NMAS ratio
Indiana	2 to 4
Iowa	3
Kentucky	3
Minnesota	4
Missouri	3 to 4
Nebraska	3 to 4
South Dakota	4 to 5
Wisconsin	3 to 5

Table 3. Limits on Permissible Lift Thickness for New York DOT.

NMAS (mm)	Minimum Lift Thickness (mm)	Maximum Lift Thickness (mm)
37.5	75	125
25.0	65	100
19.0	50	75
12.5	40	50
9.5	40	50

Table 4. Limits on Permissible Lift Thickness for Texas DOT.

Mixture Type	NMAS (in)	Minimum Lift Thickness (in)	Maximum Lift Thickness (in)	Typical Location of Layer
Type A Mix	1 ½	3.0	6.0	Base
Type B Mix	1	2.5	5.0	Base/Intermediate
Type C Mix	¾	2.0	4.0	Intermediate/Surface
Type D Mix	½	1.5	3.0	Surface
Type F Mix	3/8	1.25	2.5	Surface
PFC (PG 76)	½	0.75	1.5	Surface
PFC (AR)	½	0.75	1.5	Surface
SP A	1	3.0	5.0	Base
SP B	¾	2.25	4.0	Base/Intermediate
SP C	½	1.5	3.0	Intermediate/Surface
SP D	3/8	1.25	2.0	Surface
CMHB-C	¾	2.0	4.0	Intermediate/Surface
CMHB-F	3/8	1.5	3.0	Surface
SMA-C	¾	2.25	4.0	Intermediate/Surface
SMA-D	½	1.5	3.0	Intermediate/Surface
SMA-F	3/8	1.25	2.5	Surface
SMAR-C	¾	2.0	4.0	Intermediate/Surface
SMAR-F	3/8	1.5	3.0	Surface

Summary of Research Concerning Effects of NMAS and Lift Thickness on Pavement Performance and Performance-Related Properties

A large number of research papers and technical reports concerning the relationships among NMAS. lift thickness and various aspects of pavement performance have been reviewed by the research team. The majority of these documents have been published since 2000, and so emphasize relatively recent research efforts. In the review presented below, the summaries have been broken down according to the performance or performance-related property addressed by the research. This is followed by a summary.

Effect of NMAS on Air Voids, Permeability, Compactibility and Rate of Mat Cooling

Bahia and Paye (2001) investigated the effect of lift thickness/NMAS ratio on the density of HMA. The research project was divided in two phases: (a) laboratory experiment and (b) field study. The laboratory results showed a clear relation between lift thickness and density. However, no significant trend between lift thickness and density was observed in the field study.

In the laboratory study, Bahia and Paye (2001) used the Superpave Gyratory Compactor (SGC) to indicate that sample thickness (i.e., size) has a very important effect on obtaining the target density. The researchers found that in the laboratory, a ratio of thickness/NMAS between 4-6 is required to guarantee that sample size does not affect target density. In the field study no significant trend between lift thickness and density was observed. The authors concluded that there is no strong evidence which indicates that projects with lift thickness/NMAS ratios below 3 are significantly more difficult to compact (i.e., more roller passes). In this research project, Bahia and Paye (2001) also conducted a survey of contractors and Midwestern transportation agencies about requirements for compaction of Superpave mixes. A total of 12 DOTs and different Wisconsin contractors were interviewed. The response from the contractors and DOTs with regard to lift thickness/NMAS ratio ranged from 1.75 to 4 and from 3 to 4, respectively.

Cooley *et al.* (2002) showed a good relationship between permeability and pavement density measured in the field and laboratory. The authors indicated that NMAS and lift thickness placed in the field affect the observed relationship between permeability and density. It is suggested that higher densities are required for mixes with large NMAS to ensure an impermeable pavement. In this study it is observed that for a given mixture at a specific density level, as lift thickness increases permeability decreases. It was noted that coarse-graded mixes with NMAS of 9.5 and 12.5 mm have similar permeability. Generally, mixes with larger NMAS have higher permeability than mixes with smaller NMAS.

Mallick *et al.* (2003) found that NMAS has a strong effect on permeability. For example, at 6 % air voids, the average permeability for 9.5, 12.5, 19, and 25 mm mixes is 6, 40, 140, and 1200 x 10^{-5} cm/s, respectively. To obtain reasonably low permeability in HMA, NMAS needs to be relatively small.

Brown *et al.* in NCHRP Report 531 (2004) indicated that the thickness/NMAS ratio should be at least three for fine graded mixes and at least four for course grade and SMA mixes. These researchers also suggested a maximum thickness/NMAS ratio of 5. Using lower t/NMAS ratios results in pavement layers that require more effort to compact to an adequate density due to the rapid cooling of thinner lifts (e.g., 1" lift cools to 80°C in about 13 minutes while a 2" lift does not reached this temperature after 40 minutes). The authors reported that a 25 mm lift typically cools twice as fast as a 37.5 mm lift. This research indicates the importance of maintaining a certain minimum lift thickness in order to allow adequate time for proper compaction in the field. Therefore, a revised specification should potentially look at not only t/NMAS but minimum lift thickness as well.

Walubita *et al.* (2009) showed that coarser graded mixes are more susceptible to large variation in air void distribution along the depth of the pavement layer. This can result in structures with localized areas of large voids and non-uniform density. The authors also suggested that coarser mixes are prone to segregation.

In research performed under the Asphalt Research Consortium (ARC), Christensen has compiled permeability data from a number of independent research projects and developed an empirical relationship for predicting permeability based upon air voids, NMAS and D_{50} (the mean particle size of the aggregate gradation). Data from five studies was used in the analysis:

1. NCHRP Project 9-27, as reported by Brown et al. (2004)
2. Research on the permeability of Superpave mixes in Florida, as reported by Choubane et al. (1998)
3. A study on factors affecting the permeability of Superpave mixes in Kansas (Gogula et al., 2004)
4. Research by NCAT on factors affecting the permeability of Superpave mixes (Mallick et al., 2003)
5. NCHRP Projects 9-25 and 9-31, as reported by Christensen and Bonaquist (2006)

Also included as a predictor was whether or not the specimen was from a pavement or was laboratory compacted. The results of the model are shown graphically in Figure 2. The statistical model suggests that permeability decreases with decreasing air voids, increasing aggregate fineness (as indicated by D_{50}), and increasing NMAS. This last finding is somewhat counter-intuitive and not in agreement with findings by Mallick *et al.* and Cooley *et al*. as cited above. One possible explanation is that for HMA mixes, as NMAS increases, the average air void size increases and the air void system exhibits a significantly lower degree of interconnectedness, leading to lower permeability. The reason that Mallick, Cooley and others found that permeability increased with increasing NMAS is perhaps that they did not consider mean particle size as a separate factor in predicting permeability. In other words, overall gradation fineness, as reflected by mean particle size, is more important in determining permeability than NMAS. Fine-graded mixes with relatively large NMAS values could still exhibit adequately low permeability if properly compacted. It should be emphasized that the data set analyzed by Christensen included 75 points with an NMAS of 9.5 mm, 190 with an NMAS of 12.5 mm, 76 points with an NMAS of 19 mm, and only five points with an NMAS of 25 mm. No 37.5 mm mixes were included in the data. Therefore, the findings cannot be extended to mixes having NMAS values above 19 mm. Furthermore, the magnitude of the effect of NMAS on permeability is not extremely large; compared to a 12.5 mm mix, a 19 mm mix would typical have a permeability about 39 % lower, while the permeability of a 9.5 mm mix, all else being equal, would be about 38 % higher. These differences may seem large, but not compared to the overall variability seen in permeability for HMA mixes, which can range over three orders of magnitude. Another important consideration is the effect of NMAS on workability, compactibility and resistance to segregation. Even though larger NMAS mixes—as long as their overall gradation is on the fine side—may exhibit low permeability when properly placed and compacted, if such mixes tend to segregate and/or are more difficult to compact properly, their in-place permeability may still be unacceptable.

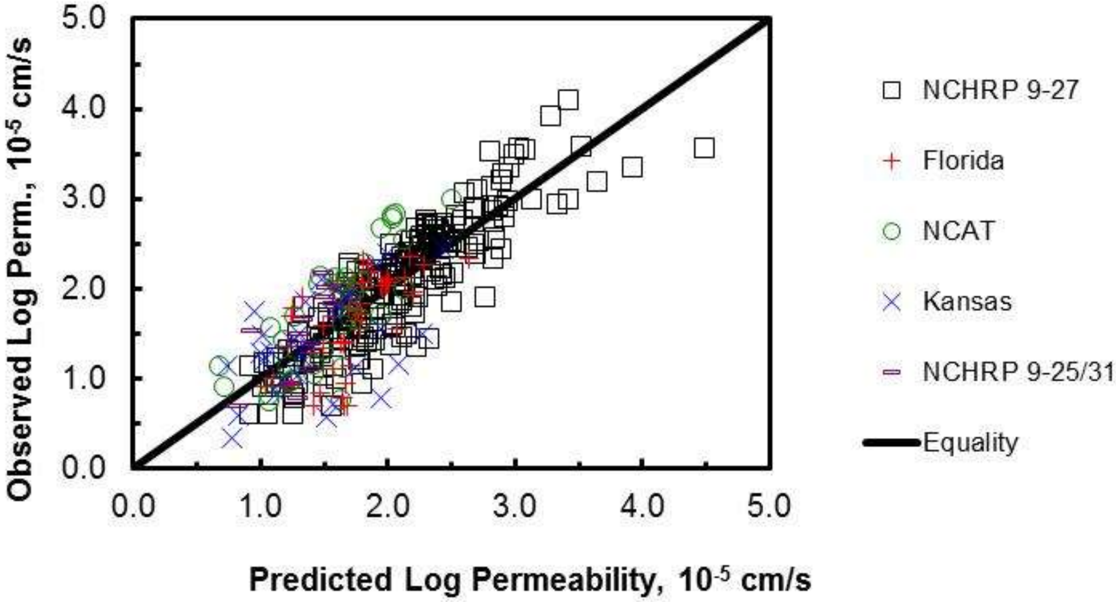


Figure 2. Plot of Observed Permeability and Permeability Values Predicted Using Christensen's Equation, for Five Independent Studies (Christensen, 2011).

Effect of NMAS on Modulus

The effect of NMAS on modulus impacts both the inherent performance of the HMA and the design of the pavement made with a given mixture. Fonsecca and Witczak (1996) developed a well-known model for estimating the dynamic modulus of HMA mixes. This model included as predictors various parameters related to aggregate gradation and hence aggregate NMAS. Christensen *et al.* (2003) indicated that NMAS has little or no influence on the modulus of hot mix asphalt (HMA), since NMAS (nor other parameters based on aggregate gradation) did not appear as a significant predictor in the final model for modulus. However, it is important to note that VMA and VFA are significant parameters in the model and therefore NMAS has probably an indirect effect on E^*.

Effect of NMAS on Resistance to Segregation

Stroup-Gardiner and Brown (2000) developed methods to quantify segregation in HMA and investigated the effects of segregation on the properties and performance of HMA. The authors identified two main factors affecting segregation: gradation and temperature. Segregation due to gradation was found to affect many other mix properties, including permeability, modulus, and strength. It also causes a significant reduction in performance. Unfortunately, for this study no attempt was made to relate segregation to aggregate properties such as gradation type or NMAS. However, an analysis of data presented by the authors show that segregation increases significantly with NMAS. The percent of cores showing none to low segregation by NMAS were 63 % for 25.0 mm; 72 % for 19 mm; and 97 % for 12.5 mm mixes. These data suggest that using an NMAS greater than 12.5 mm greatly increases the chances of segregation in HMA pavements. Another important finding of this study is that because segregation strongly affects HMA properties, and that segregation increases with NMAS, the variability in properties in an HMA pavement will increase significantly as NMAS increases.

Gudimettla *et al.* (2003) developed a prototype device for measuring the workability of HMA. The device consists of a paddle-type stirrer with provisions for measuring the torque required to stir the HMA at a constant rotational speed. It was found that mix workability decreases with increasing NMAS. This suggests that some caution is needed in designing and placing HMA with larger aggregate sizes. This is especially true when the effect of large NMAS on segregation is considered.

Effect of NMAS on Other Aspects of HMA Performance

Effect of NMAS on General HMA Performance—Cooley (2001) showed that asphalt mixes with small NMAS (i.e., 4.75-mm) have good resistance to rutting and cracking. To reduce cost and for economic considerations, Cooley suggested that lower asphalt content can be used for this fine mixes. It was noted that durability of mixes with small NMAS is not significantly affected by lowering asphalt binder content due to the lower permeability of these mixes as compared to mixes with larger NMAS.

Brown *et al*. (2002) summarized results of first phase of NCAT test track design, construction, and performance. Unfortunately, the effect of NMAS on performance was not reported. The authors indicated that a significant reduction in field permeability occurs with increased mix fineness. Similarly, Timm *et al*. (2006) did not report the effect of NMAS on performance of the Phase II NCAT sections. However, comparison between fine and coarse graded mixes was presented. The authors concluded that the fine-graded mixes were equally rut resistant, less permeable, similar in friction, and possibly easier to compact when compared to coarse graded mixes.

Bhasin *et al.* (2004) investigated the effect of NMAS on the performance HMA mixes in south Texas. The authors found that cracking resistance was not significantly affected by the change in NMAS. A reduction in NMAS can negatively affect rutting resistance unless aggregate shape characteristics are improved. Most of the aggregates studied were siliceous river gravels. The findings from this project did not support the use of fine south Texas aggregate (i.e., 9.5-mm) even when combined with crushed coarse aggregate as rutting performance of these mixtures was poor. The researchers reported optimum rutting resistance for mixtures with NMAS of 12.5-mm.

Button *et al.* (2006) summarized the findings of a research project designed to address several practical issues related to HMA design, including selection of aggregate NMAS and gradation. It should be emphasized that the aggregate NMAS issue focused on specific aggregates from South Texas (i.e., largely crushed gravels of poor quality). It was found that optimum rut resistance as measured using the SPT was obtained with 12.5 mm NMAS aggregates, although 12.5-mm fine gradations showed poor performance in the Hamburg wheel-tracking test. Mixes with 9.5-mm NMAS showed the poorest rut resistance in laboratory testing. The authors noted that the observed relationship between rut resistance and NMAS would be different for higher quality aggregates.

Christensen and Bonaquist (2006), in NCHRP Report 567, presented the results of NCHRP Project 9-25 and 9-31. The purpose of this research was to better understand the effects of HMA composition on performance and to use this information in developing improved volumetric requirements for Superpave mixes. Various relationships between the results of laboratory performance tests and mixture composition were developed. These relationships can be used to infer effects of NMAS on performance, since NMAS is directly related to VMA, one of the essential compositional factors. A 1 % increase in VMA, corresponding to a decrease in NMAS of one size, typically results in a 16 % improvement in fatigue life. For other performance related tests--including age hardening, permeability, and rut resistance, VMA appeared to have little effect. Instead, these properties related to other properties: air voids, degree of compaction, and aggregate surface area. VMA did have an effect on rut resistance, but it is in fact the relationship of aggregate surface area to VMA that is in this case critical; if VMA is increased, then aggregate surface area must also increase in order to maintain rut resistance. This would suggest that HMA mixes with small aggregate size, coarse gradations and low dust/binder ratios might be prone to rutting.

Suleiman (2009) evaluated the use of 4.75-mm HMA mixtures in North Dakota. Results indicated that higher percentage of crushed aggregate in these mixtures allow for better rutting performance. According to Suleiman (2009), 4.75-mm mixes have the potential of improving pavement life, reduce noise, create a smooth surface, and reduce permeability. However, the author indicated that some adjustments to air voids, dust content, and VMA, among other factors are required to ensure performance.

Effect of NMAS on Rut Resistance—Hand et al. (2001) evaluated the rut resistance of 21 different Superpave mixtures to ascertain the effect of aggregate size and gradation using laboratory tests such as the triaxial shear strength and wheel loading test. The authors concluded that NMAS did not have a significant effect on rut resistance as measured by these laboratory procedures.

Bhasin *et al.* (2006) evaluated the effect of changing aggregate angularity and NMAS on the rutting resistance of HMA prepared with siliceous gravels. The authors showed that rutting resistance improves with a reduction in NMAS initially. However, after an optimum NMAS, rutting resistance is reduced as NMAS decreases. Bhasin *et al.* (2006) suggested using an intermediate NMAS in projects where only river gravel is available as it can reduce the percentage of rounded aggregates and therefore improve rutting resistance. The authors concluded that a reduction in NMAS can negatively affect mix rutting resistance unless it is balanced by improving angularity and surface texture of aggregates. Other properties and factors that affect rutting resistance of mixtures are: aggregate shape, maximum aggregate size, filler type, and filler to binder ratio.

Rahman *et al.* (2011) proposed 4.75-mm mixtures to be used as a low-cost pavement preservation strategy for Kansas. The study concluded that higher PG grade does not necessarily improve the rutting resistance of 4.75 mm mixes. Rutting performance measured by the Hamburg Wheel Device and moisture damage using Tensile Strength Ratio (TSR) of these mixtures depend on aggregate type/source and not on the binder type.

Fatigue Cracking—there are a number of well-known studies that have produced empirical equations for predicting fatigue resistance (Bonnaure et al., 1980; Shook et al., 1982; Tayebali et al., 1995). Virtually all these equations suggest that increasing binder content—as indicated by either effective binder content or voids filled with asphalt (VFA)—will increase fatigue resistance. These leads to the conclusion that fatigue resistance of HMA mixtures should improve with decreasing NMAS. Shen and Carpenter (2007) performed more recent research indirectly linking NMAS and fatigue resistance for HMA mixtures. The authors developed a statistical model for predicting plateau value (strain endurance limit) as a function of various mixture compositional variables. This directly relates to traditional fatigue life as defined by cycles to 50 % damage. The authors presented a table showing effect of changes in various parameters on cycles to failure. Based upon their findings, it is estimated that each size increase in NMAS produced a decrease in fatigue life of about 15 %.

Effect of NMAS on Low Temperature Cracking—Christensen and Bonaquist (2004) reported on the results of research on various aspects of the indirect tension test for evaluating resistance to low temperature cracking. There is a section in this report on the calculation of the coefficient of thermal expansion for HMA mixes (CTEM). An improved equation for calculating CTEM is presented, which is based on mix m-value and VBE. This suggests that as NMAS increases and VBE decreases, CTEM should decrease improving the low temperature properties of the mix. However, in this same report an equation is presented which gives direct tensile strength as a function of VFA, showing a distinct increase in strength with increasing VFA, which will in general increase with decreasing NMAS. An important question is which effect is stronger, or in other words, will decreasing NMAS improve or reduce resistance to low temperature cracking. Using the equations presented in this report for estimating CTEM and IDT strength, the overall effect of NMAS on resistance to low temperature cracking was estimated as follows. The relative CTEM was estimated for NMAS values from 9.5 mm through 37.5 mm, using typical in-place volumetrics (7 % air voids). The IDT strength was similarly estimated, and since the authors presented equations for mixes containing modified and non-modified binders, both cases were considered and an average IDT strength found for each NMAS. Performance indices were calculated from these values; for CTEM, the performance index was calculated as the ratio of the

CTEM for the typical 19 mm mix divided by the CTEM for the selected NMAS. The performance index for IDT strength was calculated as the ratio of the strength for the NMAS selected to the strength of the typical 19 mm mix. In each case, as the performance index increases, the low temperature performance, in general, should increase. Furthermore, since the tensile stress generated in a pavement will be directly proportional to the CTEM, and since the ability to withstand that stress will be directly proportional to the IDT strength, the overall performance of the mix can be estimated by multiplying the CTEM and IDT strength performance indices. The results of this analysis are summarized in Table 5 below. It appears as though the typical improvement in IDT strength with decreasing NMAS overrides the typical increase in CTEM, so the overall effect of decreasing NMAS in HMA mixes appears to be to somewhat improve resistance to low-temperature cracking.

Table 5. Estimated Effect of NMAS on Resistance to Low Temperature Cracking Based on Estimated Coefficient of Thermal Expansion on Indirect Tension Strength.

NMAS *mm*	**Estimated Coefficient of Thermal Expansion (CTEM)** *mm/mm*	**Est. Average IDT Strength** *MPa*	**CTEM Performance Index**	**IDT Strength Performance Index**	**Overall Low Temperature Performance Index**
9.5	1.90E-05	2,436	0.89	1.28	1.14
12.5	1.79E-05	2,187	0.94	1.15	1.08
19	1.69E-05	1,909	1.00	1.00	1.00
25	1.58E-05	1,597	1.07	0.83	0.89
37.5	1.48E-05	1,244	1.14	0.65	0.74

Summary: Effect of NMAS on HMA Pavement Performance as Reported in the Literature

Based upon the literature review presented above, several conclusions can be made concerning the relationships among NMAS, lift thickness and performance. Findings concerning compactibility, air void content and NMAS and lift thickness are somewhat inconsistent. Research by Bahia and Pave (2001) found no difference in density between pavements constructed with lift thickness/NMAS ratios below 3 and those with higher ratios. Cooley et al. (2002), on the other hand, found a decrease in density at lower lift thickness/NMAS ratios. Because the field and laboratory portions of the study by Bahia and Pave produced somewhat contradictory results, and because of the somewhat larger scope of the work by Cooley et al., it is suggested that at this time the recommendations of the latter study be given more weight—that pavements with lower lift thickness/NMAS ratios can be more difficult to compact, leading to higher air voids and greater permeability. This in turn would suggest that mixes with larger

NMAS must be placed at higher lift thicknesses in order to obtain sufficient in place density. The later study by Brown *et al*. (2004) continued the work of Cooley *et al*. (2002) and gave specific recommendations that fine mixes should have a minimum lift thickness/NMAS ratio of three, and that coarse mixes should have a minimum lift thickness/NMAS ratio of four. Brown *et al.* also pointed out that thin lifts of HMA cool much more quickly than thicker ones, therefore consideration should be given to specifying a minimum lift thickness, regardless of NMAS. The research linking NMAS directly to permeability is contradictory. However, it appears at this time that it is not NMAS but overall mix fineness that has the stronger effect on permeability, with finer gradations exhibiting significantly lower permeability. This might suggest that mixes designed with larger NMAS—say 19 and 25 mm, should be designed with fine gradations in order to ensure adequately low in situ permeability.

There is good agreement concerning the relationship between modulus and NMAS. In general, all else being equal, mixes with larger NMAS will exhibit somewhat higher modulus values compared to mixes with lower NMAS. However, this effect is much less than the effect of binder grade and temperature on modulus, and probably will not significantly impact pavement design. However, mixes with larger NMAS will typically contain less asphalt binder and will hence be less expensive compared to those having a smaller NMAS.

There was also good agreement that resistance to segregation in HMA mixes increases with decreasing NMAS. One researcher also reported that resistance to segregation is improved in mixes with fine gradation relative to those with coarse gradations. The high degree of segregation observed in 25 mm NMAS mixes calls into question whether such mixes should be routinely used in pavement mixes. Limiting 19 mm mixes to those with a fine gradation would potentially not only help reduce segregation, but would also lower permeability.

The effect of NMAS on rut resistance is not completely clear at this time. Research on Texas aggregates suggests that there may be a slight decrease in rut resistance when NMAS is reduced from 12.5 mm to 9.5 mm, but this finding may only apply to a specific set of aggregates. There is strong evidence that mixes with smaller NMAS, because they typically have higher binder contents, will in general exhibit improved fatigue resistance compared to mixes with larger NMAS. There is also evidence that resistance to low temperature cracking should improve slightly with decreased NMAS. A final consideration in rut resistance, fatigue resistance and resistance to low temperature cracking is the effect of segregation and especially compaction on these aspects of performance. Because smaller NMAS and finer mixtures will tend to exhibit greater resistance to segregation, this should further improve the performance of these mix types in terms of fatigue and low temperature cracking. Christensen and Bonaquist (2006) demonstrated the very strong effect of proper field compaction on almost all aspects of performance; therefore, since mixes with smaller NMAS tend to be easier to compact, it should be expected that these mixes will in general tend to exhibit better field performance compared to mixes with larger NMAS.

Comparing the Resilient Modulus and Dynamic Modulus of HMA

In order to evaluate the effect of NMAS on pavement designs using the WisPave program, the resilient modulus of the mix must be known (or estimated). The Hirsch model is a reasonably accurate means of estimating the dynamic modulus of HMA mixes, but unfortunately cannot be used to directly estimate M_R (Christensen *et al*., 2003). Therefore, a method for at least approximately converting E^* values to M_R values is needed to use the Hirsch model in studying the effect of NMAS on pavement designs using WisPave. Kim and Lee (1995) proposed inter-relationships among different definitions of stiffness for HMA. Two different laboratory mixes along with field cores were tested and analyzed. However, very limited data on resilient modulus was presented by the authors which limits the development of a generalized model/relationship between E^* and resilient modulus.

Loulizi *et al.* (2006) compared the M_R and E^* values of two mixes, a 25-mm base mix and a 9.5-mm surface mix. The authors proposed that $E^* = 1.4 \times M_R$ for the 25-mm mix and that $E^* = 1.07 \times M_R$ for the 9.5 mm mix. These equations were developed based on resilient modulus measurements using a loading time of 0.03 s, and E^* measurements at 5.2 Hz (i.e., equivalent loading time using $\omega = 1/t$). It is important to note that the data provided is not large enough for generalization, especially given that the variability in E^* and M_R measurements is often very large.

One possible approach to estimating M_R values from E^* data is to use the storage modulus $E'(\omega = 1/t)$ as an estimate of $M_R(t)$. An analysis performed as part of this literature review shows that at a frequency of 5.2 Hz, and temperatures of 20 and 40°C, $E^* = 1.09 \times E'$. This suggests that E' is probably a reasonable surrogate for M_R, at least for the purposes of examining the effect of NMAS on HMA pavement designs.

Preliminary Findings and Recommendations

Based upon the review of the state of the practice, it appears there is strong evidence to allow the use of 9.5-mm mixes in surface courses in Wisconsin. These mixes should exhibit performance at least as good as that of 12.5-mm mixes, with the possible exception of rut resistance. Allowable NMAS for base course mixes should be extended to 9.5-mm and 12.5-mm mixes, since these mixes should exhibit low permeability and excellent fatigue resistance, both qualities that are essential in base course mixtures. Current requirements for lift thickness to NMAS ratio are generally consistent with recommendations from recent research projects and practices by other states, and probably do not need to be changed, although some consideration should be given to a minimum lift thickness to prevent mats from cooling to quickly and potentially contributing to poor compaction. A minimum lift thickness of 38 mm would represent a minimum lift thickness/NMAS ratio of 4 for 9.5-mm mixes, and would prevent these from being placed at a lift thickness of 28.5 mm, which potentially might cool too quick to allow proper compaction.

APPROACH TO LABORATORY TEST PROGRAM AND SIMULATIONS

The efforts in this project consisted of both laboratory testing and various simulations; the approaches used for both are discussed in the sections below. The specific tasks addressed in this part of the project were as follows:

- Analyze effect of NMAS on modulus using predictive equations
- Analyze effect of NMAS on pavement performance using WisPave and predictive equations for modulus
- Evaluate effect of NMAS on modulus as measured in the laboratory and on segregation using image analysis
- Evaluate effect of NMAS on pavement performance using MEPDG
- Construct a database of HMA properties from QC data
- Analyze effect of NMAS on modulus and permeability using QC database

This section of the report presents details on the laboratory testing and simulation studies. These sections include Materials, Methods and Experiment Design. Results of the testing and simulations are summarized in the following sections of the report.

Materials

Much of this research was done using models to estimate engineering properties from compositional properties of HMA mixes, and so did not involve materials per se. Some of this modeling was done using a database of QC properties compiled for Wisconsin HMA mixes; this allowed a more realistic estimate of properties compared to simply using typical compositions. The general properties of the database are described below, while a more detailed description is given in the Results section of this report.

Materials Included in the QC Database

Table 6 is a summary of project information for mixes included in the QC database. The database includes 9 mixes with a 19-mm NMAS, and 22 with a 12.5-mm NMAS. Mix types include E-0.3, E-1, E-3, E-10 and E-30. Binders used include PG 58-28, PG 64-22 and PG 64-28. Additional details on the database are given in the results section of this report. The information in this table was used in conjunctions with various predictive models to analyze the effect of NMAS on permeability, modulus and performance as indicated using the WisPave program. The methods and results of these various analyses are discussed in later sections of this report.

Table 6. Summary of Project Information on Mixes Included in QC Database.

Project ID	Year	NMAS	Location	Mix Type	PG Grade
1001-01-61	2011	19	Dane	E-0.3	[redacted]
5567-00-72	2011	12.5	Dane	E-10	64-28
5886-02-02	2011	19	Dane	E-1	64-22
1066-02-75	2010	12.5	Dane	E-10	64-28
1206-00-73	2010	12.5	Dane	E-3	[redacted]
3670-00-64	2010	12.5	Dane	E-1	58-28
5825-00-70	2010	12.5	Dane	E-3	58-28
5924-00-61	2010	12.5	Dane	E-1	64-28
5992-00-76	2010	12.5	Dane	E-3	[redacted]
1066-00-72	2009	19	Dane	E-30	64-22
1066-02-72	2009	12.5	Dane	E-30	64-22
5390-00-71	2009	12.5	Dane	E-10	64-22
5926-00-73	2008	12.5	Dane	E-1	58-28
1066-03-61	2007	12.5	Dane	E-30	64-22
5909-00-72	2007	12.5	Dane	E-3	58-28
5954-02-76	2007	12.5	Dane	E-3	58-28
1620-00-79	2011	19	Marathon	E-10	[redacted]
1166-01-76	2009	12.5	Marathon	E-10	64-22
6999-08-81	2007	19	Marathon	E-3	64-28
4337-09-71	2009	12.5	Manitowoc & Brown	E-3	58-28
9280-04-72	2011	12.5	Manitowoc & Brown	E-3	58-28
1150-22-71	2009	19	Manitowoc & Brown	E-10	58-28
1227-10-60	2009	12.5	Manitowoc & Brown	E-10	58-28
1211-18-71	2008	19	Manitowoc & Brown	E-30	64-22
4987-02-17	2008	19	Manitowoc & Brown	E-3	64-28
8160-14-72	2010	12.5	Douglas & Bayfield & Ashland	E-3	[redacted]
8343-05-72	2009	12.5	Douglas & Bayfield & Ashland	E-1	58-28
8727-06-60	2011	19	Douglas & Bayfield & Ashland	E-3	64-28
9926-00-70	2010	12.5	Douglas & Bayfield & Ashland	E-0.3	58-28
9932-00-70	2009	12.5	Douglas & Bayfield & Ashland	E-1	58-28
1180-40-71	2008	12.5	Douglas & Bayfield & Ashland	E-10	64-28

Materials Used in the Laboratory Evaluation of the Effects of NMAS on Dynamic Modulus

The effect of NMAS on dynamic modulus was evaluated by preparing mixes at four levels of NMAS (9.5mm, 12.5mm, 19.0mm, and 25.0mm) using a granite aggregate source from north central Wisconsin. Gradations for each NMAS were selected to meet the requirements of Table 460-1 for dense graded mixes in the WisDOT Standard Specifications, percent passing vs. Sieve size plots are presented in Figure 3. In the figure the plots were constructed using the sieve size raised to the 0.45 power, in labels for the x-axis these values were replaced with actual sieve sizes.

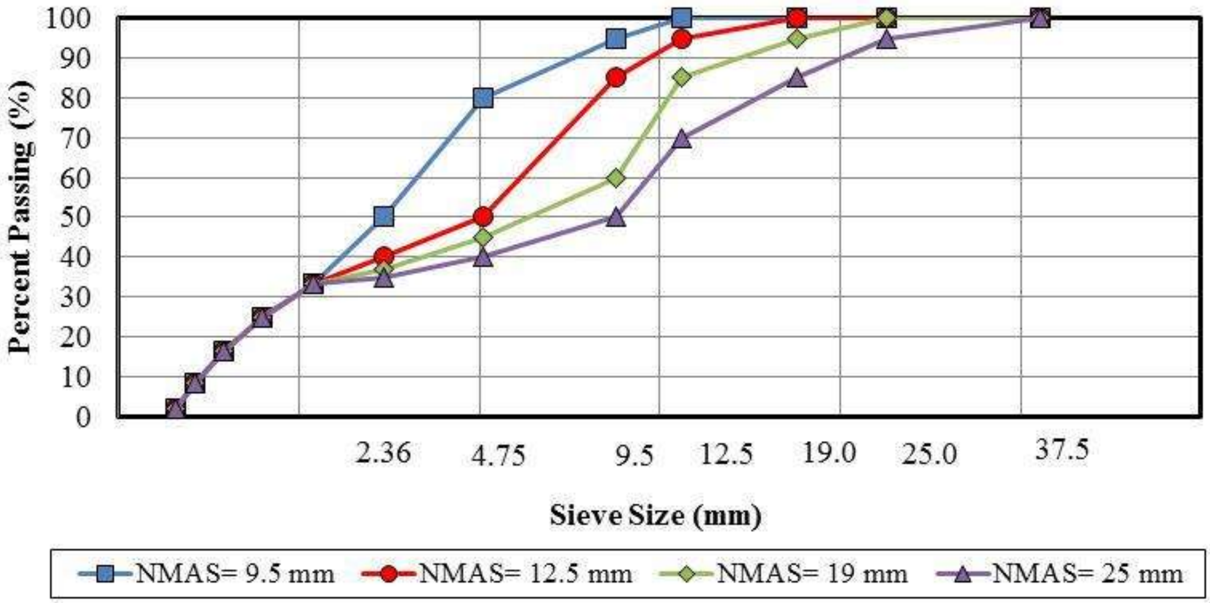


Figure 3. Summary of Gradations Used for Evaluation of the Effects of NMAS on Dynamic Modulus.

To isolate the effect of changes in NMAS on E* the gradation of the fine aggregate (P#16) and the asphalt binder content (5.1%) was held constant rather than varied to meet SuperPave volumetric criteria for all mixes. Mixes were prepared with two asphalt binder grades, PG 58-34 and PG 70-28, selected to represent the range in binder grades currently used in Wisconsin. Both asphalt binders were from the same supplier, the PG 70-28 is modified, details of the modification were not requested.

All mixes were prepared at the selected binder content of 5.1% and subjected to a compactive effort of 100 gyrations at 600 kPa using the SuperPave Gyratory Compactor (SGC). Mixing and compaction temperatures as determined by AASHTO T316 were 148°C/138°C (MT/CT) and 164°C/154°C for the PG 58-34 and PG 70-28 binders respectively. The %Air Voids after 100 gyrations in the SGC for the eight mixes used is provided in Figure 4, the VMA for all mixes is provided in Figure 5. Results indicate holding binder content constant did not produce mixes that met SuperPave volumetric criteria for both PG 58-34 and PG 70-28 binders. In regards to VMA, all mixes met minimum values specified in Table 460.1 of the WisDOT Standard specifications.

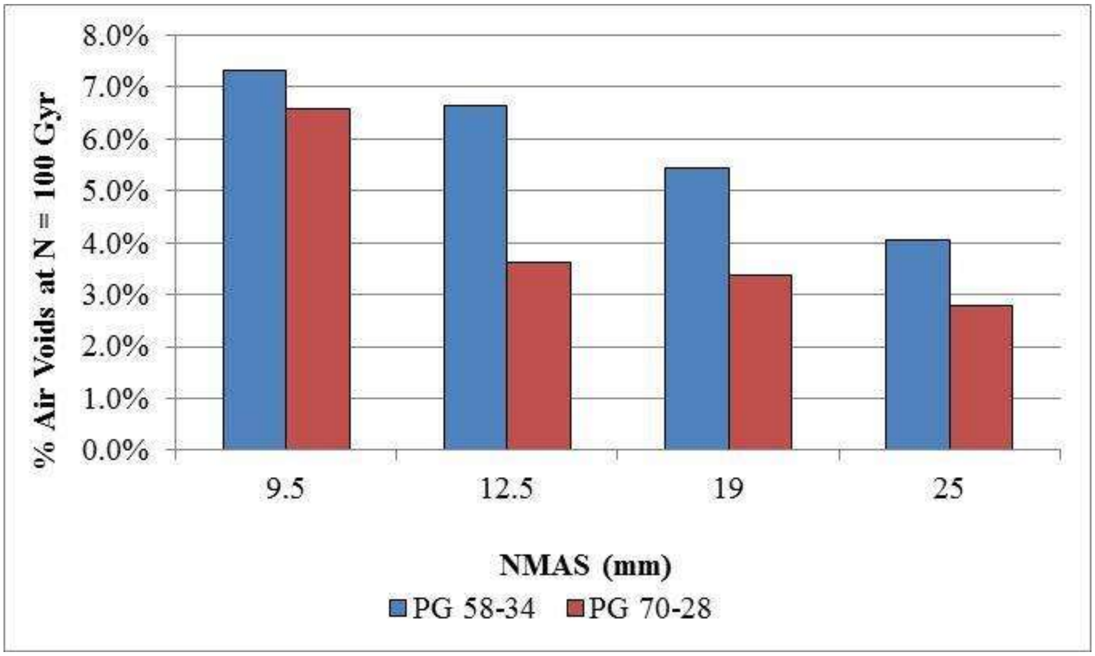


Figure 4. Summary of Mixture % Air Voids after 100 SGC Gyrations – PG 58-34 and PG 70-28 Binders.

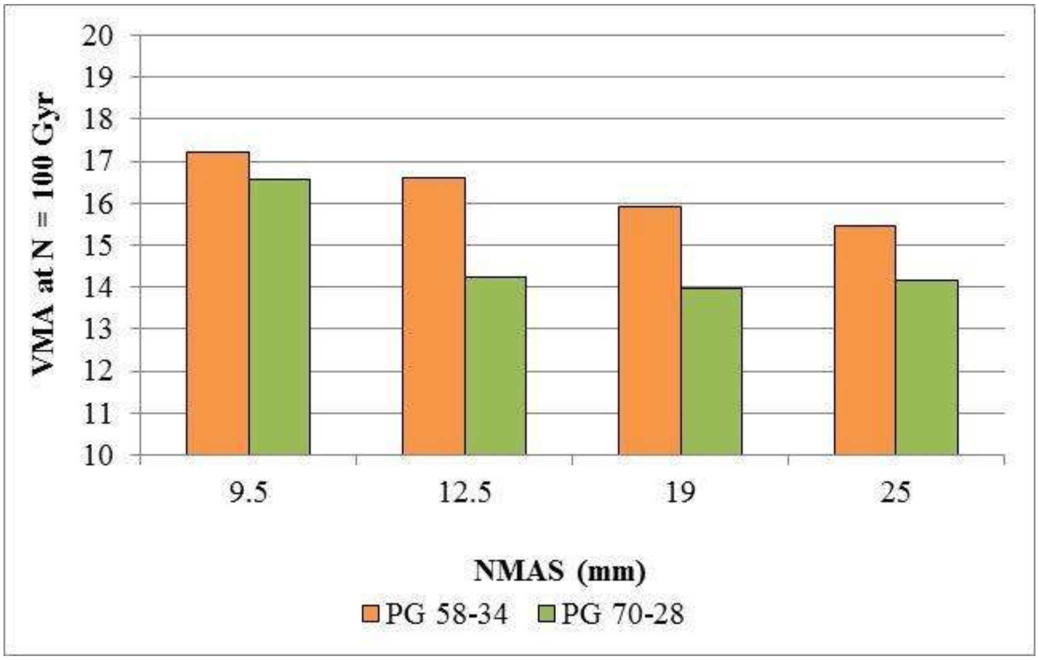


Figure 5. Summary of Mixture VMA after 100 SGC Gyrations – PG 58-34 and PG 70-28 Binders.

Asphalt binder content was intentionally held constant to isolate the effects of NMAS on dynamic modulus by removing the confounding factors of variation in VMA and VFA due to mix design changes. This approach will be further discussed in the experimental design section of the report.

Field Cores Sampled for Segregation Analysis

Based on recommendations of the TOC after presentation of the interim results in the summer of 2012, the evaluation of the effects of NMAS on segregation was limited to field compacted mixes. Field samples were cored and collected from the USH 41 reconstruction near Green Bay, WI; the samples are summarized in Table 7. The gradation of the 12.5 and 19.0 NMAS mixes is provided in Figure 6. Gradation and volumetric data for the 25.0mm mix was not available in the information provided by the contractor. Three cores were taken of each mix.

Table 7. Summary of Field Cores Collected for Segregation Analysis – USH 41

NMAS	Traffic Level	Mix Design ID	Design Binder Content	Volumetrics @Ndes	
				VMA	VFA
12.5 mm	E-3 (N_{des} = 75)	250-0086-2011	5.1%	14.3	72%
19.0 mm	E-10 (N_{des} = 100)	250-0049-2010	4.4%	13.5	70.4%
25.0 mm	E-1 (N_{des} = 60)	250-0017-2012	4.0%	N/A	N/A

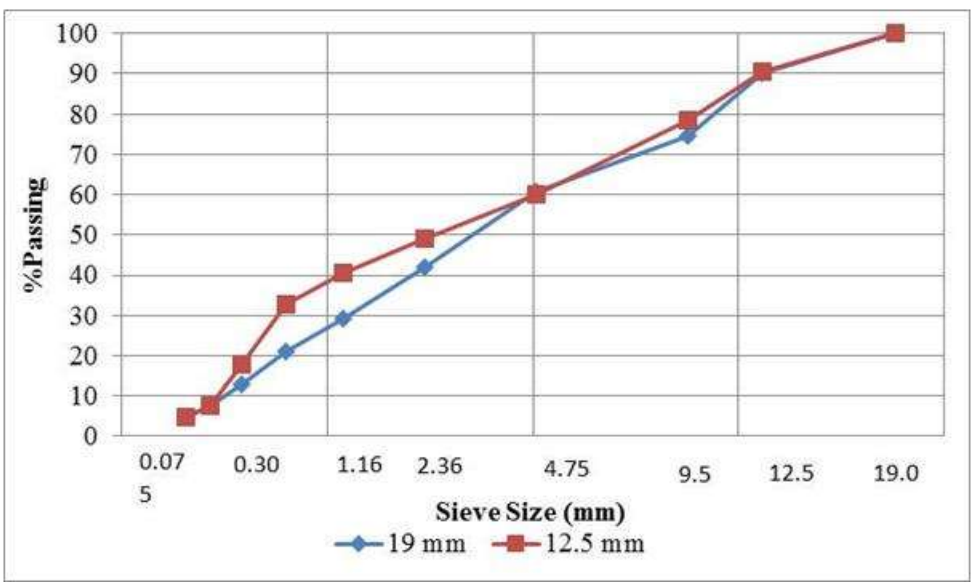


Figure 6. Gradation of Field Mixes Sampled from USH 41 – NMAS 12.5 and 19.0.

Methods

Evaluating the Effect of NMAS on Modulus Using Predictive Equations

In order to estimate the effect of NMAS on HMA modulus, both the Witczak equation (Fonsecca and Witczak, 1996) and Hirsch equation (Christensen et al., 2003) were applied in analyzing typical HMA mixtures with different NMAS values, at temperatures of 5, 20 and 40°C, loading frequencies of 0.1 and 10 Hz, and using two different binders. The binders used were from the Strategic Highway Research Program, or SHPR, and are known as AAC-1 and AAK-1. AAC-1 is a relatively soft AC-8 asphalt, while AAK-1 is a relatively hard AC-30 asphalt. These binders were selected because both DSR data (as used in the Hirsch model) and a full array of traditional rheological data (as used in the Witczak equation), were available. The aggregate gradation used were set at the midpoint for Wisconsin specifications. Air voids were fixed at 3 %, and VMA was set at 1 % above the minimum requirement.

Extending the QC Database Using Simulated 9.5- and 25-mm Mixes

One of the primary objectives of this project was to evaluate the impact of using 9.5-mm and 25-mm NMAS mixes in Wisconsin pavements. However, only data for 19 and 12.5 mm projects were available in the database. In order to evaluate the modulus and permeability of realistic 9.5-mm NMAS mixes, the following procedure was used to simulate 9.5-mm mix data in the QC data base, using data for the 12.5-mm mixes:

1. The percent passing for the top three sieve sizes in the simulated 9.5-mm gradation (100 % passing, 90 to 100 % passing and 90 maximum percent passing) are based on top three sieves for the corresponding 12.5-mm gradation.
2. The percent passing for the 2.36 mm sieve is put in same position relative to the gradation limits for the simulated 9.5-mm gradation as the corresponding 12.5-mm gradation.
3. The percent passing for the simulated 9.5-mm gradation for sieve sizes between 0.075 mm and 2.36 mm are placed in same position relative to the minimum percent passing for the 0.075-mm sieve and maximum percent passing for the 2.36-mm sieve for the corresponding 12.5-mm gradation.
4. The percent passing for 0.075-mm sieve is the same for the simulated 9.5-mm gradation and the corresponding 12.5-mm gradation.
5. VMA is adjusted 1.0 % upward for the simulated 9.5-mm mix, compared to the corresponding 12.5-mm mix.
6. Design air voids are kept the same for the simulated 9.5-mm mix and the corresponding 12.5-mm mix. Field air voids are decreased by 0.41 % for the simulated 9.5-mm mix compared to the corresponding 9.5-mm mix. This is because analysis of the QC database shows a slight increase in field air voids with increasing NMAS.

The composition of simulated 25-mm mixes was determined in a similar manner, but based on the 19-mm mixes and with the following differences:

1. The percent passing for the top four sieve sizes in the simulated 25-mm gradation (100 % passing, 90 to 100 % passing and 90 maximum percent passing) are based on top three sieves for the corresponding 19-mm gradation.
2. The percent passing for the 2.36 mm sieve is put in same position relative to the gradation limits for the simulated 25-mm gradation as the corresponding 19-mm gradation.
3. The percent passing for the simulated 25-mm gradation for sieve sizes between 0.075 mm and 2.36 mm are placed in same position relative to the minimum percent passing for the 0.075-mm sieve and maximum percent passing for the 2.36-mm sieve for the corresponding 19-mm gradation.
4. The percent passing for 0.075-mm sieve is 1.0 % less for the simulated 25-mm gradation and the corresponding 19-mm gradation.
5. VMA is adjusted 1.0 % downward for the simulated 25-mm mix, compared to the corresponding 19-mm mix.
6. Design air voids are kept the same for the simulated 9.5-mm mix and the corresponding 12.5-mm mix. Field air voids are increased by 0.82 % for the simulated 9.5-mm mix compared to the corresponding 9.5-mm mix. This is because analysis of the QC database shows a slight increase in field air voids with increasing NMAS.

Because there were nine 19-mm mixes in the QC database and 22 12.5-mm mixes, the simulated mixes added nine 25-mm mixes to the database and 22, 9.5-mm mixes. Although not actual HMA mixes placed in Wisconsin, this procedure should provide a realistic set of properties for 25-mm and 9.5-mm NMAS mixes, so that a realistic evaluation of the effect of NMAS on modulus and permeability can be performed.

Effect of NMAS on Modulus and Permeability Using Predictive Equations and QC Database

The effect of NMAS on modulus was analyzed earlier by estimating the modulus of a range of typical HMA mixtures using the Witczak and Hirsch equations. To provide a more realistic analysis, this same approach was applied to the mixes included in the QC database. In this case, the actual composition of the mixes was used in making the predictions. The results of this analysis are presented in the following section of this report.

Permeability was estimated using an equation developed by Christensen as part of the Asphalt Research Consortium. The general approach and the data used in developing this equation was presented earlier in the Synthesis of Current Practice. The actual equation is as follows:

$$\text{LOGPERM} = 0.471 + 0.222\ \text{VTM} - 0.0495\ \text{LV} + 1.64 \log(D_{50}) - 1.22 \log(\text{NMAS}) \qquad (1)$$

Where

LOGPERM	=	log permeability (10^{-5} cm/s)
VTM	=	air void content
LV	=	term for interaction between indicator variable for lab specimens and air void content
D50	=	particle size, in mm, at 50 % passing
NMAS	=	nominal maximum aggregate size, in mm

Table 8 is a summary of the statistics for this model. The r-squared value of 70 % (adjusted for degrees of freedom) is quite good considering the high amount of variability inherent in permeability values for HMA specimens. The variable LV is an interaction term between an indicator variable for lab specimens (1 if the specimen tested was compacted in the lab, 0 if it is a field core) and air void content. This is needed because laboratory compacted HMA specimens show a significantly lower permeability at a given air void content compared to field cores. The results of applying Equation X to the mixes in the QC database are presented below.

Table 8. Summary of Statistics for Regression Model for Permeability Model (Christensen, 2013).

Predictor	Coef.	St. Dev.	T	P
Constant	0.4711	0.2639	1.78	0.075
VTM	0.2218	0.0097	22.93	0.000
LV	-0.04950	0.00834	-5.94	0.000
Log (D_{50})	1.639	0.162	10.14	0.000
Log (NMAS)	-1.222	0.224	-5.45	0.000

S = 0.3770 R-Sq = 70.2% R-Sq(adj) = 69.8%

To evaluate the accuracy of Equation 1 when applied to HMA mixes in Wisconsin, data from a WHRP project on compaction and permeability were compiled (Schmitt et al., 2007). Equation 1 was then used to predict permeability values, which can be compared with permeability values measured during the project. This comparison is shown in Figure 7. The general level of permeability predicted for the mixes is accurate, but the accuracy of predictions for individual mixes is not good. For the original data set (see Figure 2) the model predicted the permeability of the mixes to within about a factor of 6. In this case, the predictions appear to be accurate only to within a factor of about 18. However, the permeability measurements for the Wisconsin mixes were made using a field permeameter, whereas those for which the model was based on were made using a laboratory permeameter. The relationship between these two types of measurements has not been well documented. It is possible that factors may be affecting the field permeability measurements that are not involved in laboratory measurements, such as surface texture and/or lift thickness. This might be especially relevant for low permeability values—such as those considered here. Unfortunately, this analysis cannot confirm the accuracy of Equation 1, but cannot be used to refute its accuracy either. Equation 1 will be assumed to be reasonably accurate in the remaining analysis of mix permeability.

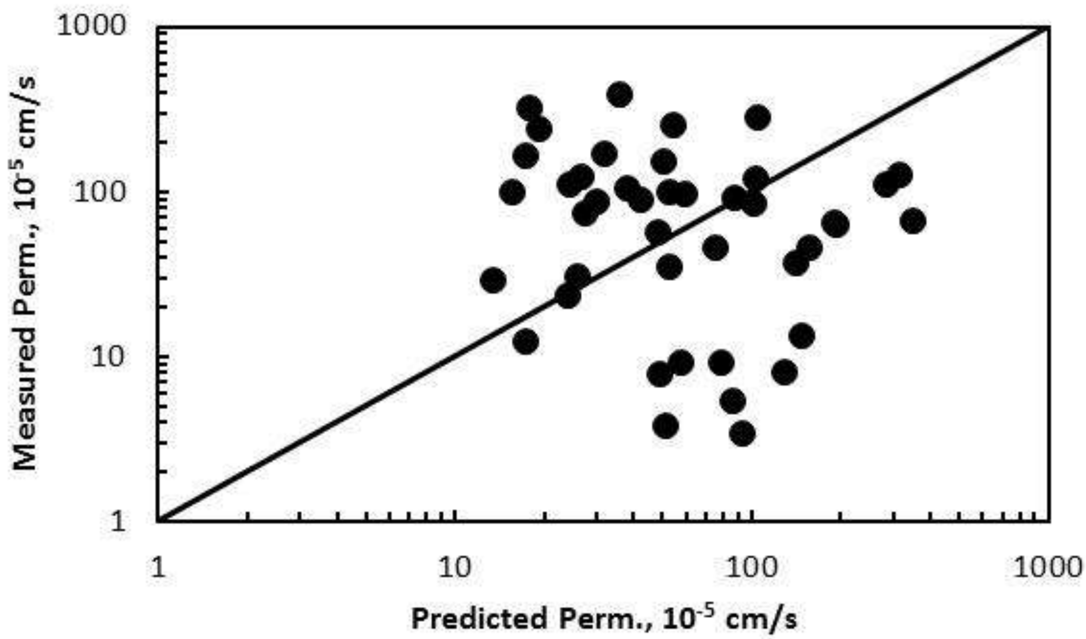


Figure 7. Comparison of Measured Permeability Values with Those Predicted Using Equation 1, for HMA Mixes Produced in Wisconsin (Schmitt et al., 2007).

Effect of NMAS on Pavement Performance Using WisPave

WisPave is a computer program used by the Wisconsin Department of Transportation to design HMA pavements. It is based on the AASHTO equation for flexible pavement design, in which the allowable traffic is a function of the terminal serviceability index and the structural number for the pavement. The structural number for the pavement depends upon the layer coefficient and thickness for each layer of the structure. Therefore, the performance of a given HMA layer in the WisPave program becomes only a function of its thickness and layer coefficient. In Wisconsin, a layer coefficient of 0.44 is assumed for all new HMA layers. Therefore, if the current Wisconsin design procedure using WisPave is strictly applied, no differences would be observed in the performance of HMA of any NMAS. However, the real question is if the layer coefficients were to be determined experimentally, would there be any significant difference among mixes of differing NMAS?

To answer this question, it is necessary to estimate the layer coefficient for typical HMA mixes having different NMAS values. Van Til et al. (1972) related layer coefficient to resilient modulus for HMA mixtures. Unfortunately, the Hirsch model (and the Witczak model) cannot be used to directly estimate resilient modulus. However, the Hirsch model can be used to estimate the storage modulus E'(ω), which should relate well to resilient modulus, since resilient modulus is based upon recoverable strain, while storage modulus is based upon the in-phase component of the complex modulus which is closely related to the degree of elasticity in the behavior of a material. More precisely, the conversion can be expressed mathematically:

$$M_R \cong E'(\omega)\Big|_{\omega \to 3/\pi t} \tag{2}$$

The equivalence between loading time and frequency is based on the analysis of Christensen (1983) for the relationship between relaxation modulus and storage modulus, but with a slight adjustment to account for the gradual pulse loading used in the resilient modulus test as opposed to the instantaneous loading inherent in the definition of relaxation modulus. This results in a frequency of 9.5 rad/s (1.5 Hz) for the calculation of resilient modulus from storage modulus, given the standard pulse duration of 0.1 seconds.

The layer coefficient is calculated from the resilient modulus using the relationship proposed by Van Til et al. (1972), which can be expressed mathematically using the following equation:

$$a = 0.1723 \ln\left(M_R\right) - 1.8032 \tag{3}$$

Where a is the layer coefficient for a dense graded HMA with resilient modulus M_R (lb/in^2). In the work plan, the WisPave anlaysis was described as consisting of an analysis of a set number of NMAS values (9.5, 12.5 and 19 mm) and various other factors, such as pavement structure, traffic level, mix design type (E-3 and E-10), climate and asphalt binder. However, as discussed above, the effect of NMAS on WisPave occurs only through the layer coefficient, which in turn is directly related to mix modulus, so the clearest and most direct way of evaluating the effect of NMAS on WisPave (that is, if layer coefficient were determined for each mix), is to evaluate the effect of NMAS on layer coefficient. Furthermore, to make this analysis as realistic as possible, the QC database was used in the analysis. As described below, this includes mix design and QC information on 31 mixes, 22 of which are 12.5-mm and 9 of which are 19-mm. This data base was then expanded to include 31 additional mixes, both 9.5-mm and 19-mm. The procedure used to develop these simulated mixes is described later in this section.

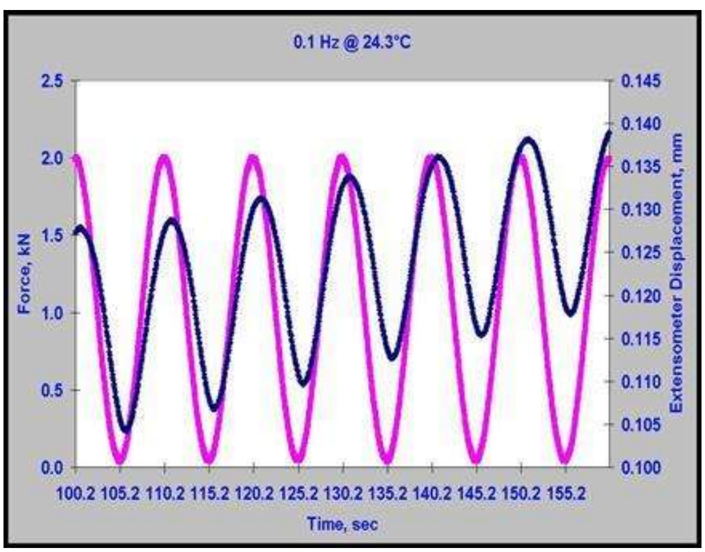
Dynamic Modulus Testing

Dynamic modulus testing was conducted according to AASHTO TP 62-06 protocols. Mixes were compacted in the laboratory using the Superpave gyratory compactor, cut and cored to obtain the cylindrical samples required for the test. The air void content of the cored samples was 7%. During testing samples are subjected to sinusoidal loading at a user selected range of test temperatures and frequencies. The temperatures and frequencies used in this study are summarized in Table 9.

Table 9. Summary of Temperatures and Frequencies Used for Dynamic Modulus Testing.

Test Temperature (°C)	Frequency (Hz)
4	0.1, 0.5, 1, 5, 10
20	
40	

During the test, the applied stress and resulting strain are recorded with time, and the dynamic modulus is defined as the ratio of peak stress (σ_o) to peak strain (ε_o). The phase angle, defined as the time lag (t_i) between the applied stress and material response multiplied by the loading frequency (ω) is also an output of the test. A schematic of the data output during the test and the equations used to calculate mixture dynamic modulus and phase angle are shown in Figure 8.

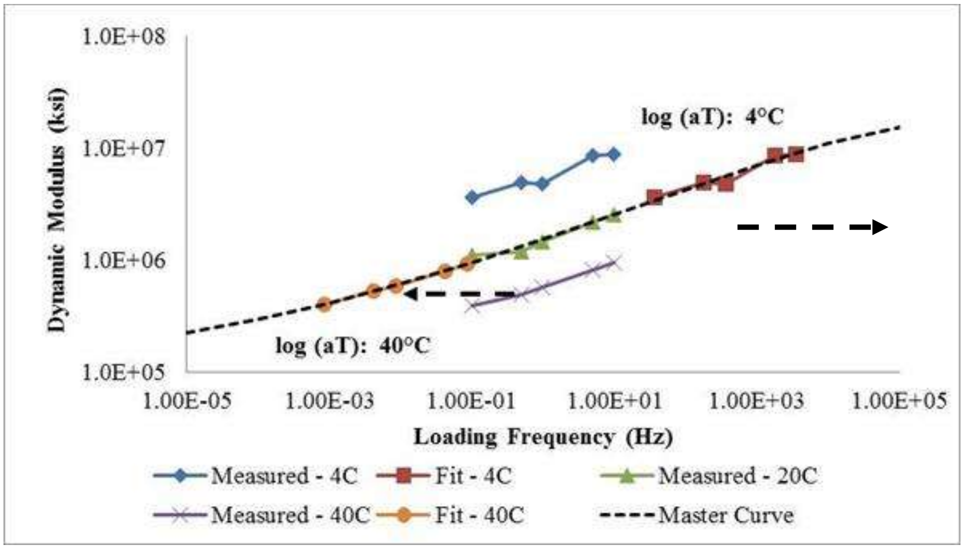


$$|E^*| = \frac{\sigma_0}{\varepsilon_0}$$

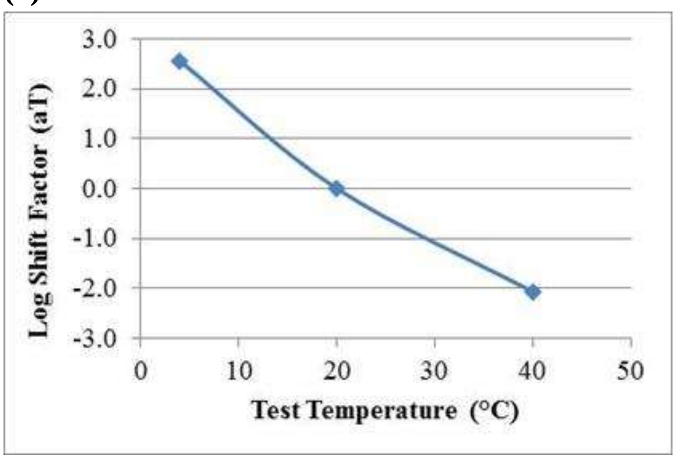
$$\phi = \omega t_i$$

Figure 8. Schematic of Dynamic Modulus Test and Equations Used to Test Outputs.

The dynamic modulus represents a single point measurement of mixture stiffness at a given temperature and frequency. Testing at multiple of frequencies and temperatures allows for definition of the mixture stiffness over a range of service conditions (loading rate/temperature) through application of the time-temperature super-position principal to determine the dynamic modulus master-curve. Use of this analysis allows for estimates of mixture stiffness at conditions that cannot be measured directly in mechanical testing. The dynamic modulus master curve is used as a design input in the Darwin-ME pavement design guide to represent the variation in modulus due to differing traffic and climatic conditions. A schematic of constructing the master curve based on the test conditions provided in Table 9 is provided in Figure 9.



(a)



(b)

Figure 9. Example of Master Curve Construction (a) and Plot of Shift Factors vs. Test Temperature (b).

In this study the dynamic modulus master curve was constructed based on the model defined in NCHRP Report 459 and temperature shift factors were determined using the Williams-Landel-Ferry (WLF) formulation, these models are provided in Equations 4 and 5 respectively. (Bahia, 2001). Model parameters were fit through minimization of squared errors using the Solver function in Microsoft Excel.

$$E^* = E^*_E + (E^*_G - E^*_E)\left[1 + \left(\frac{f_c}{a_T f}\right)^k\right]^{\frac{-me}{k}} \tag{4}$$

Where:

E^*	=	Predicted Dynamic Modulus (ksi)
E^*_E	=	Equilibrium Complex Modulus (ksi) – Modulus as frequency approaches 0.
E^*_G	=	Glassy Complex Modulus (ksi) Modulus as frequency approaches ∞
f_c	=	Cross-over Frequency (Hz)
a_T	=	WLF Shift Factor
f	=	Frequency used in test (Hz)
k, me	=	Shape parameters (dimensionless)

$$log\frac{a_T(T)}{a_T(T_r)} = \frac{c_1(T-T_r)}{c_2+(T-T_r)} \tag{5}$$

Where:

a_T	=	Temperature Shift Factor
T_r	=	Reference Temperature (°C)
c_1	=	Constant
c_2	=	Temperature Constant

Segregation Analysis through Use of Planar Imaging

Segregation analysis was conducted through use the IPas[2] (Image Processing and Analysis Software) on the field cores summarized in Table 7. To conduct the analysis the field cores were cut in the vertical direction into four slices, images from each slice were obtained using an office grade desktop scanner. The vertical direction was selected to correspond to the direction of mixture compaction and load bearing in the field, thus segregation due to differing NMAS at this orientation relates to potential performance differences between mixes. A schematic of how the field cores were sliced is provided in Figure 10.

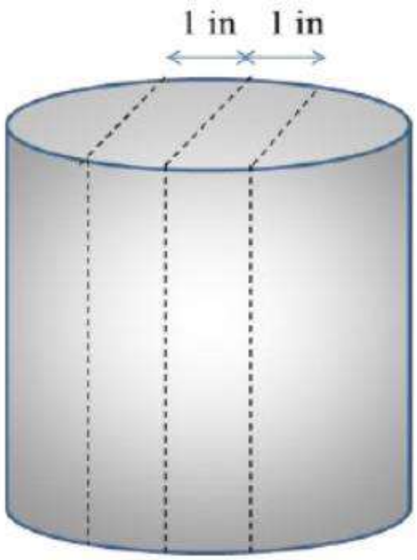


Figure 10. Schematic of Slicing Field Cores for Image Processing.

Use of four slices results in a total of six images, all of which were processed individually. The image processing and analysis consists of two steps 1) Image processing to ensure that the image captured is representative of the actual field mix, 2) Analysis of the processed image to evaluate vertical segregation. Individual results were averaged to obtain a result representative of the entire mix.

Image processing consists of application of well established filtering techniques to differentiate between air voids, aggregate, and asphalt mastic (binder + fine particles) in the mix. A schematic of the filters applied is provided in Figure 11.

To differentiate between mixture components images are first processed to remove noise within the image (Median Filter) and to remove variations in pixel intensity (color) within an aggregate particle (Hmax Filter). Particle boundaries are then defined using the water shed transformation. The final step is conversion of the gray-scale image to a black and white image based on a user defined pixel intensity threshold. An example of application of the thresholding process to differentiate between aggregates and air void/mastic is provided in Figure 12. The black/white image conversion defines all air voids and mastic as black and all aggregates as white.

An iterative process is used for quality control to ensure that the black and white processed image is representative of the mix by comparison of the mixture gradation and area fraction of the coarse aggregates. To conduct the calibration the binder content, volume of binder, aggregate specific gravity, and gradation are entered from the mix design into the software. With these parameters the size distribution of the mix is obtained and the volume of the coarse aggregate is calculated. Depending on the resolution of the scanner only aggregates retained on the #4 or #8 sieve are visible in the image. An example of comparison of the gradation and coarse volume fraction between the mix and image is provided in Figure 13, using a screen shot from the IPas^2 software.

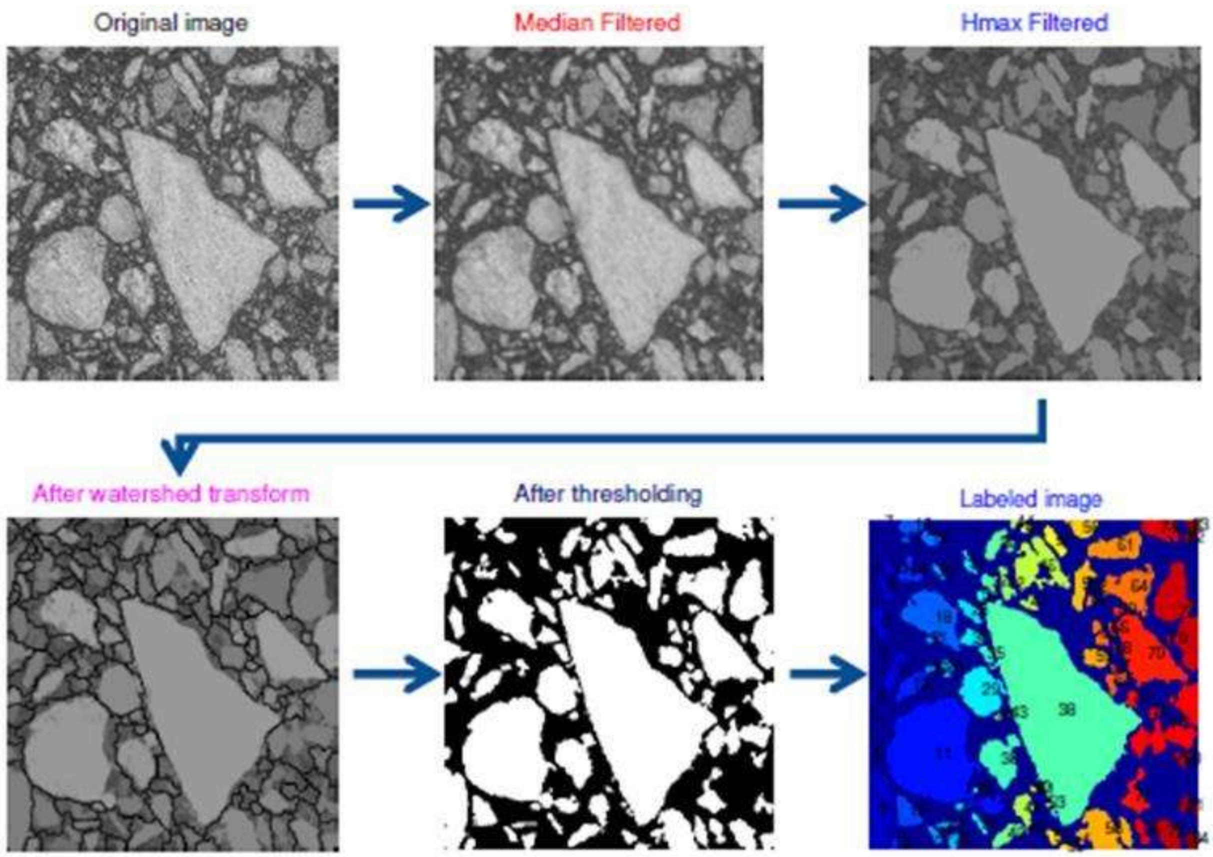


Figure 11. Example of Filters Applied During Image Processing.

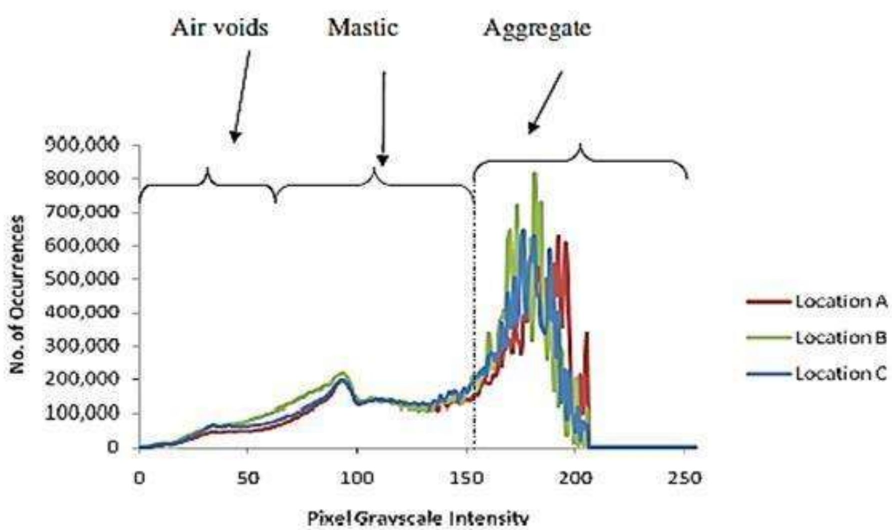


Figure 12. Example of Outcomes of Thresholding Process.

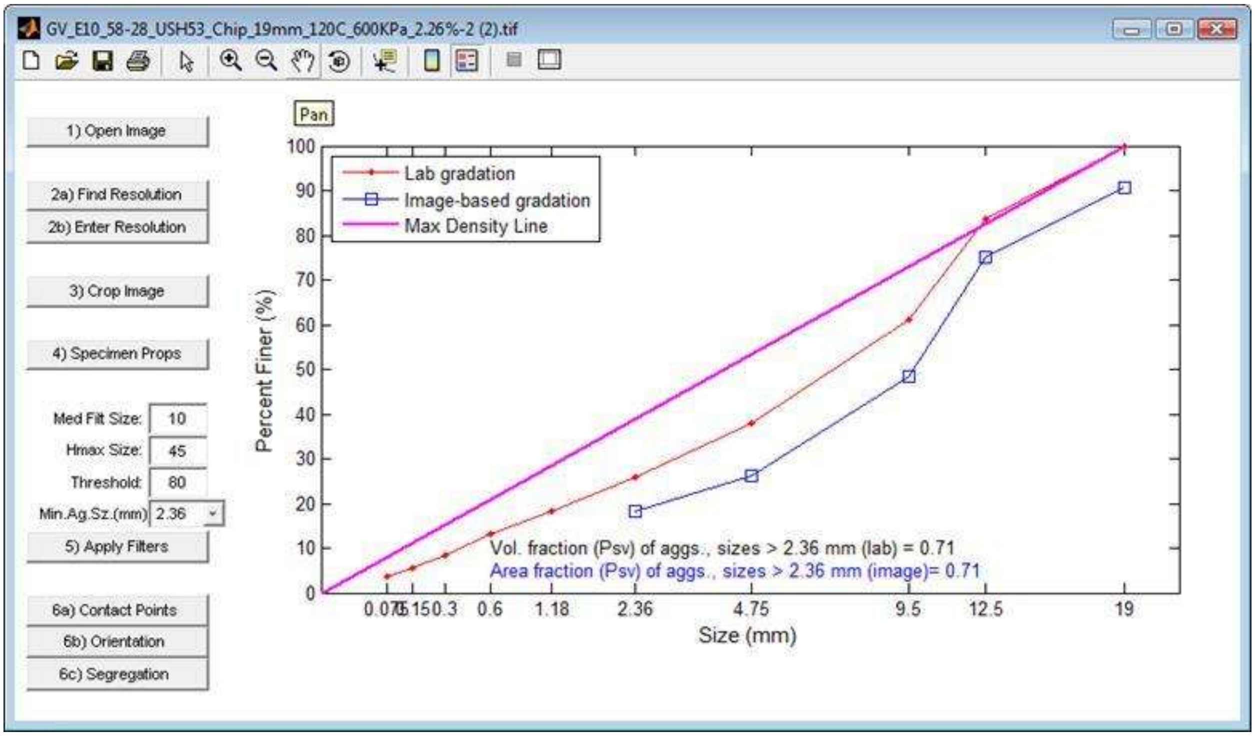


Figure 13. Example of Calibration of Image Thresholding Based on Actual Mixture Properties.

An acceptable image must meet the following criteria:

- Coarse Aggregate Volume Fraction of image within ±5% of actual volume fraction.
- Gradation curve of image parallel to actual mix gradation. The gradation curve of the image will always be below that of the actual mix design because the fine aggregates are not included in the image due to the resolution of the scanner.

If the above criteria are not met, a new threshold value is selected and the calculations are re-run.

Once the image is processed and meets the aforementioned quality control requirements analysis of vertical segregation is conducted by portioning the sample into three segregation groups, with each group representing one-third of the total depth of the sample. An example is provided in Figure 14.

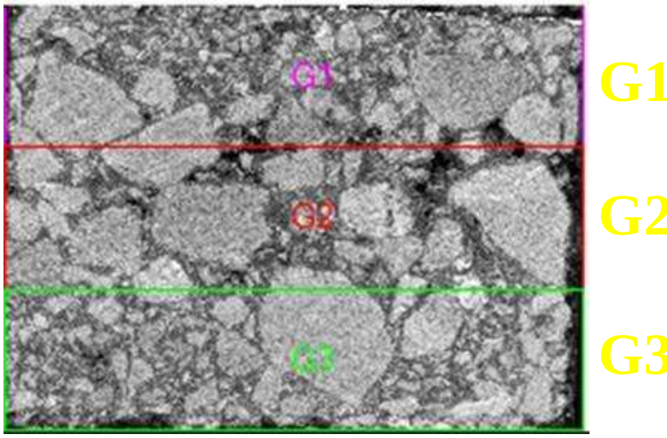


Figure 14. Illustration of Vertical Segregation Groups.

Segregation analysis is based on the location of the centroid of each aggregate in Groups 1, 2, and 3. The centroid is determined based on a Cartesian coordinate system using Equation 6:

$$\textbf{Centroid:}\ x_j^c = \frac{1}{N_j}\sum_{k=1}^{N_j} x_k \ ,\ \ y_j^c = \frac{1}{N_j}\sum_{k=1}^{N_j} y_k \tag{6}$$

Where, x_j^c and y_j^c are the x- and y- coordinate of the centroid of the labeled region j, respectively; and x_k and y_k are the individual coordinates of each pixel within labeled region (aggregate) j.

All aggregates in the mix are identified based on the labeling system shown in Figure 10. Segregation is evaluated by constructing a histogram of the percent aggregates of a given size located within each of the previously defined vertical segregation groups and comparing the distribution of aggregates throughout the depth of the image. The mix is defined as segregated if significant variation in the distribution of each aggregate size observed between groups. An example of the histogram used to assess segregation is provided in Figure 15. In this example, the mix is segregated as the Group 1 (top of mix) is composed of considerably more fine aggregates than other analysis groups.

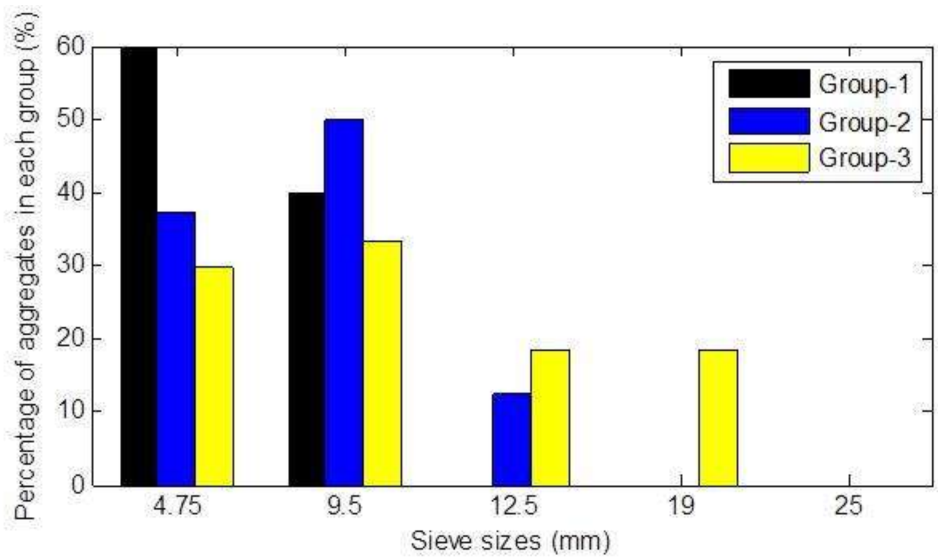


Figure 15. Example of Histogram Identifying the Distribution of Each Aggregate Size within the Vertical Segregation Group.

The IPas^2 software is a product of the Asphalt Research Consortium (ARC) developed by UW-MARC in collaboration with Michigan State University. It was applied to this study as an automated method to process images and conduct segregation analysis. The software was developed based on well established image processing techniques, and was used as a tool to facilitate analysis. It is not necessary to have the software to obtain similar outputs. The analysis methods and image processing techniques applied by IPas^2 were recently summarized and submitted to the FHWA Mixtures Expert Task Group as a draft AASHTO procedure. More information regarding the development and application of IPas^2 is available in (Coenen, 2011) and (Roohi, 2012).

Experiment Designs for Laboratory Tests

Laboratory Evaluation of the Effects of NMAS on Dynamic Modulus

A two factor experiment designed to assess the effects of NMAS and asphalt binder grade is summarized in Table 10. Levels of binder PG and NMAS were selected based on the range of materials currently specified by WisDOT for binder and surface courses on projects constructed in Wisconsin.

Table 10. Experimental Design for Evaluation of Effects of NMAS on Laboratory Measured Dynamic Modulus.

Factor	Levels
Binder PG (2)	PG 58-34 PG 70-28
NMAS (4)	9.5 mm 12.5 mm 19.0 mm 25.0 mm

Prior to testing, all samples were compacted to a nominal target core air void content of 7%. As previously stated, during sample preparation asphalt binder content was held constant at 5.1% to limit variations in VMA between mixes and isolate the effects of NMAS and binder grade. Results presented in Figure16 indicate that this approach was successful as values of VMA average approximately 17% and are all within by 1%. Given the relationship between VMA and VFA, the VFA for all values was consistent as well ranging from 59% - 61%. As previously stated, holding the binder content constant resulted in mixtures that did not meet the air voids at N_{des} limit of 4%, therefore while this approach isolates the selected experimental factors, the mixtures tested may not be representative of what would be accepted in the field.

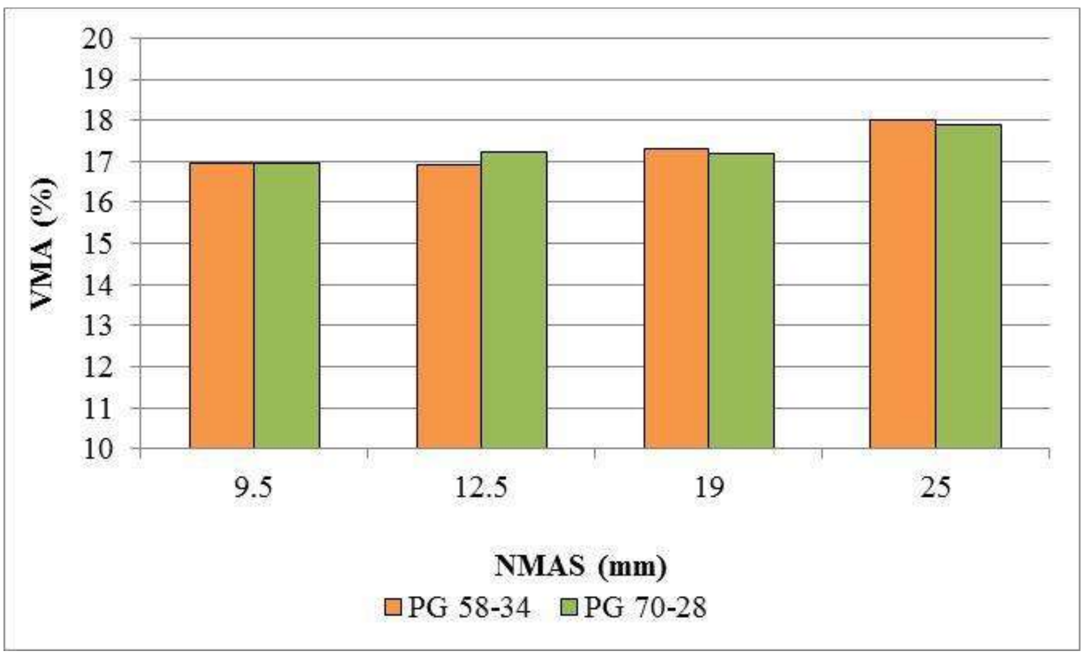


Figure 16. Variation in VMA for Dynamic Modulus Samples.

The effect of NMAS on dynamic modulus was evaluated using individual frequency sweep data and master curves. Statistical analysis was also used to verify observations. Specifically, two methods were used, for a given binder PG grade one-way analysis of variance was applied to compare the effects of NMAS at three levels of reduced frequency selected to represent mixture stiffness at a range of service conditions. The reduced frequencies selected were 0.001, 1, and 100 Hz. A second statistical analysis was conducted on the entire data set at the same frequencies to assess the impacts of changing NMAS relative to the effects of changing binder PG grade.

Segregation Analysis

The IPas^2 software was used to conduct vertical segregation analysis on the field cores sampled from USH 41 detailed in Table 7Table , which include NMAS sizes of 25 mm, 19.0 mm, and 12.5 mm. As detailed previously, the output of the analysis is the histogram of aggregate particles in each vertical analysis group for aggregate sizes ranging from NMAS to the #8 sieve (1.16mm). Segregation is defined as deviations in the number of aggregates between groups (increasing depth within the core sample).

Effect of NMAS and Lift Thickness on Pavement Performance Using the AASHTO MEPDG

Simulations using AASHTO MEPDG were conducted to evaluate the effects of pavement structures utilizing different combinations of NMAS on pavement performance. The free version of MEPDG available prior to the release of Darwin-ME was used because purchase of the Darwin ME was cost prohibitive to the project. This decision was made based on the conclusion that while differences may exist in the magnitude of performance predictions from the two versions of the software the effects of NMAS and other factors will remain unchanged. The factors varied in MEDPG simulations are provided in Table 11, the trial pavement sections are provided in Table 12. For all sections the HMA thickness used was selected based on the lift thickness requirements for NMAS presented in Section 460.3.2 of the WisDOT Standard Specifications.

Table 11. Factors Varied for MEPDG Simulations.

Factor	Levels
Average Annual Daily Traffic (AADT) – 2	2000 4500
Wisconsin Climate – 2	North (Rhinelander, WI) South (Madison, WI)
Asphalt Binder PG Grade – 2	PG 58-34 PG 70-28

Table 12. Summary of Pavement Sections Used in MEPDG Simulations.

Placed on	Lower Layer NMAS (mm)	Upper Layer NMAS (mm)
PCC	N/A	9.5
		12.5
Base Course	9.5	9.5
	12.5	9.5
	19	9.5
	9.5	12.5
	12.5	12.5
	19.0	12.5

All simulations were conducted using Level 3 analysis and an analysis period of 10 years. The distresses selected for evaluation include:

- International Roughness Index (IRI)
- Alligator Cracking
- AC Rutting
- Total Rutting

Longitudinal and thermal cracking distresses were omitted from the analysis. Longitudinal cracking was not included because at the time of simulation the transfer functions used to relate material properties to distress were still under development as preliminary results showed that they were inaccurate and highly variable. For Level 3 analysis thermal cracking is only a function of low temperature asphalt binder PG grade and thus not relevant to this study.

RESULTS

Evaluating the Effect of NMAS on Modulus Using Predictive Equations

The results of the analysis of NMAS on modulus using predictive equations are shown graphically in Figure 17 (Witczak Equation) and Figure 18 (Hirsch model). The results are similar in that they show that in general, HMA modulus will increase with increasing NMAS. The modulus increase seen for the 25 mm mix is predicted to be somewhat greater using the Witczak equation (10 to 20 %) compared to the increase predicted using the Hirsch model (5 to 8 percent).

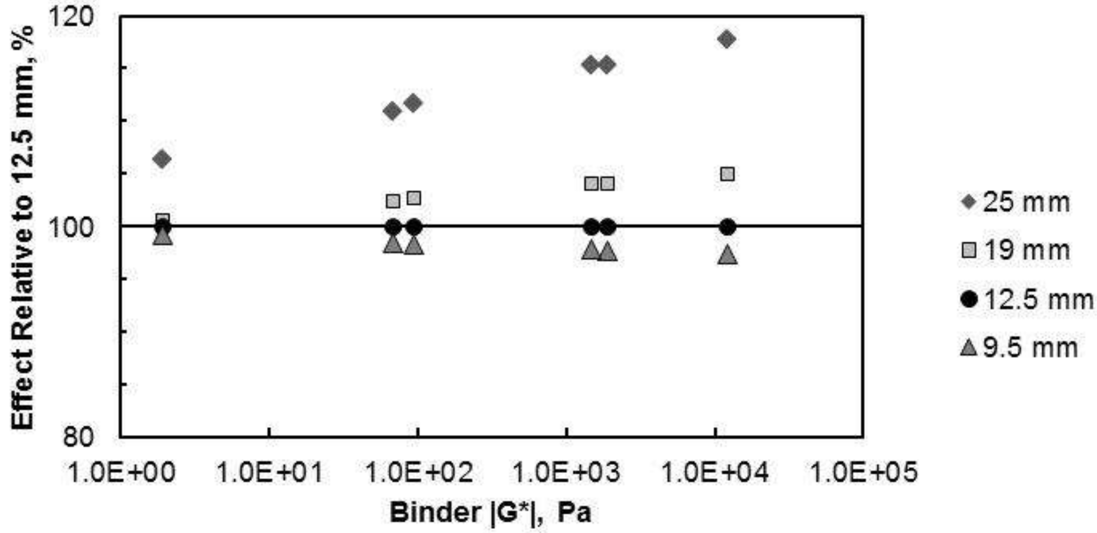


Figure 17. Effect of Binder Modulus and Aggregate NMAS as Estimated using the Mirza-Witczak Equation.

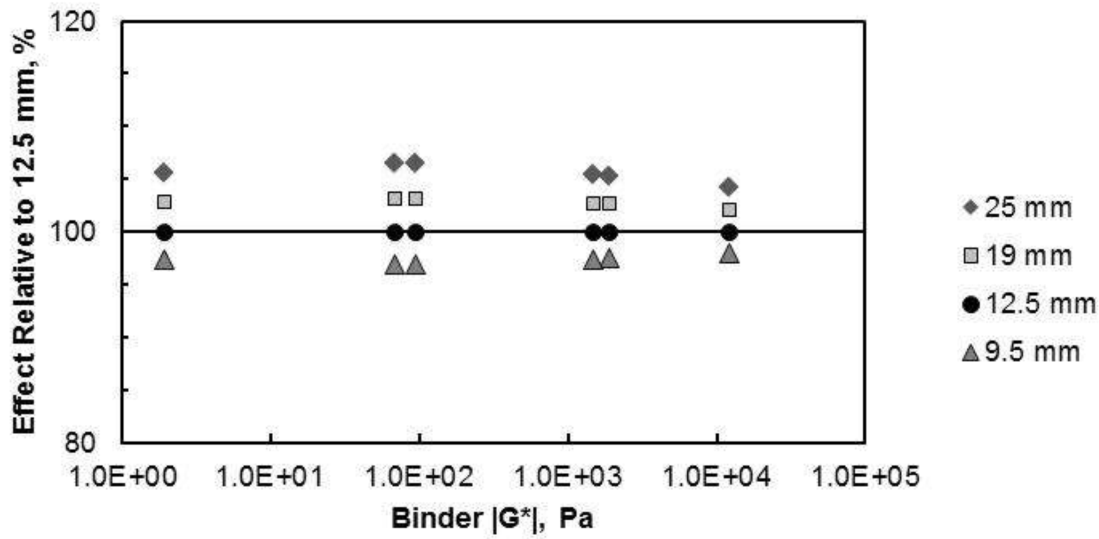


Figure 18. Effect of Binder Modulus and Aggregate NMAS as Estimated using the Hirsch Model.

QC Database: Effect of NMAS on Air Voids

The results of the analysis of the in place air voids as compiled in the QC database are summarized in Table 13. Air voids were slightly higher for the 19-mm mixes compared to the 12.5-mm mixes (7.68 vs. 6.80 %). The pooled standard deviation—representing the average within-project variability—was nearly identical for the two cases, 1.36 % for the 19-mm mixes and 1.30 % for the 12.5-mm mixes.

Table 13. Summary of In Place Air Voids from QC Database.

Property	NMAS	
	12.5 mm	**19 mm**
Average	6.80	7.68
Minimum	3.58	5.54
Maximum	8.96	9.24
Sample Size	22	9
Pooled Std. Dev.	1.30	1.36

QC Database: Effect of NMAS on Modulus

Figure 19 shows the results of the analysis of the relationship between NMAS and estimated dynamic modulus for the QC Database mixes. The relationship is very weak, if there is a relationship at all. Examining the various mix properties, it was found that in-place air voids correlated very well with estimated dynamic modulus. Figure 20 is a plot of estimated dynamic modulus as a function of air voids.

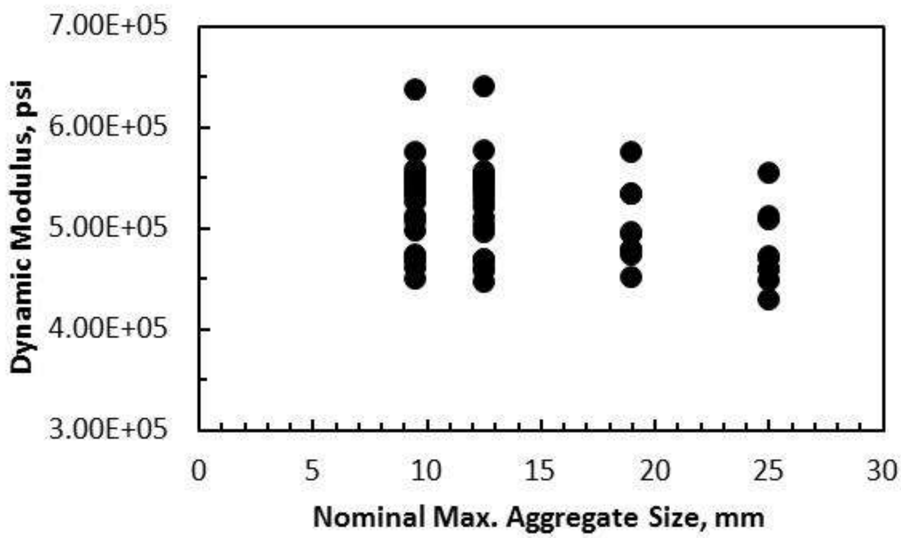


Figure 19. Plot of Estimated Dynamic Modulus at 1.6 Hz and 21°C as a Function of NMAS for QC Database Mixes.

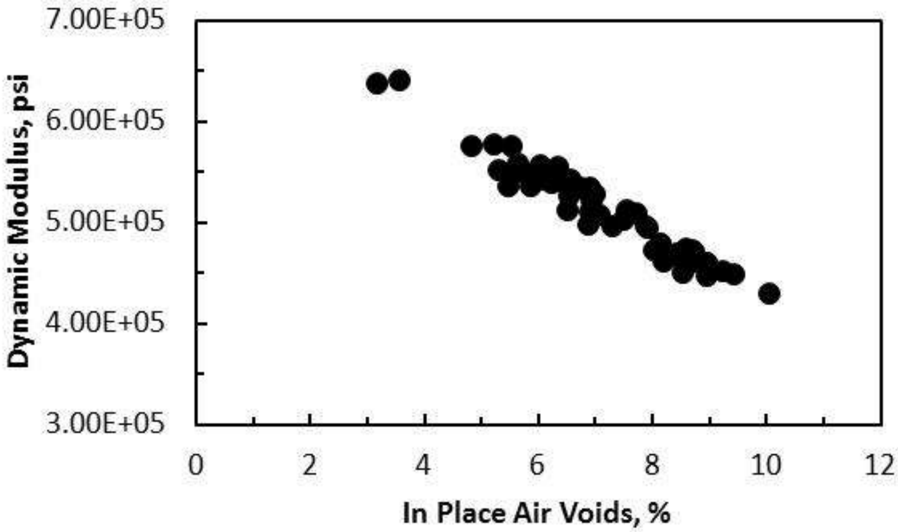


Figure 20. Plot of Estimated Dynamic Modulus at 1.6 Hz and 21°C as a Function of In-Place Air Voids for QC Database Mixes.

QC Database: Effect of NMAS on Permeability

The results of the analysis of the effect of NMAS on permeability for the QC Database mixes is shown graphically in Figure 21, which is a plot of estimated permeability as a function of NMAS. The plot includes a line representing 125×10^{-5} cm/s, which is the upper limit for in-place permeability suggested by the National Center for Asphalt Technology (Brown et al., 2004). There is in this case a moderately strong relationship between NMAS and estimated permeability. All mixes below 25-mm NMAS are below the suggested maximum permeability value; several of the 25-mm exceed this limit. As with dynamic modulus, the in-place air voids exert a stronger influence on permeability than NMAS, as shown in Figure 22.

There is reasonable agreement between these predictions and permeability values reported as part of NCHRP Project 9-27 (Brown et al., 2004). In NCHRP Report 531, fine graded 9.5 mm mixes were reported to have permeability values ranging from 1 to 37×10^{-5} cm/s, with the majority of values below 10×10^{-5} cm/s. Although somewhat higher than the values predicted here, it is clear from data reported in NCHRP Report 531 that permeability is very strongly related to aggregate gradation, so the observed difference here is easily attributable to differences in gradation. Permeability values reported by Brown and his associates for fine graded 19-mm mixes ranged from 1 to 77 × 10-5 cm/s, with values typically ranging from about 3 to 30 × 10-5 cm/s. Again, although slightly lower than values reported here they are in general agreement and the differences are possibly due to slight differences in aggregate gradation.

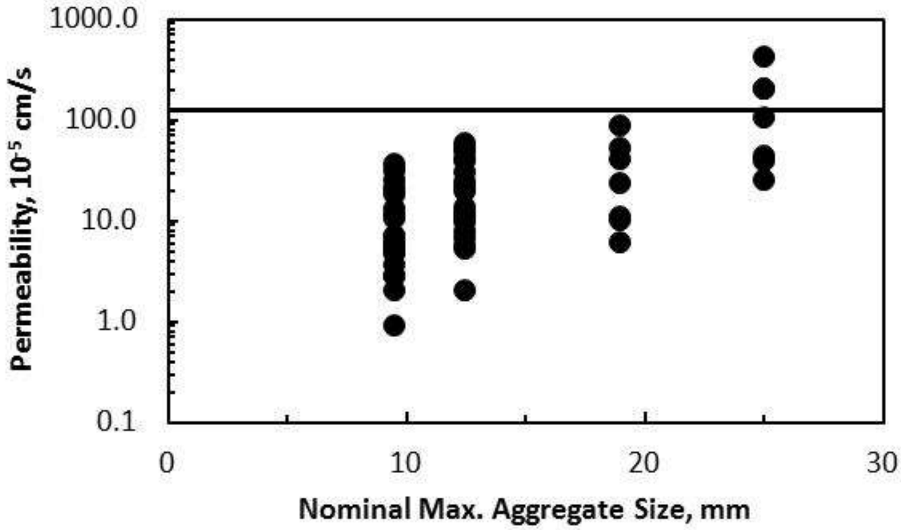


Figure 21. Plot of Estimated Permeability as a Function of NMAS for QC Database Mixes.

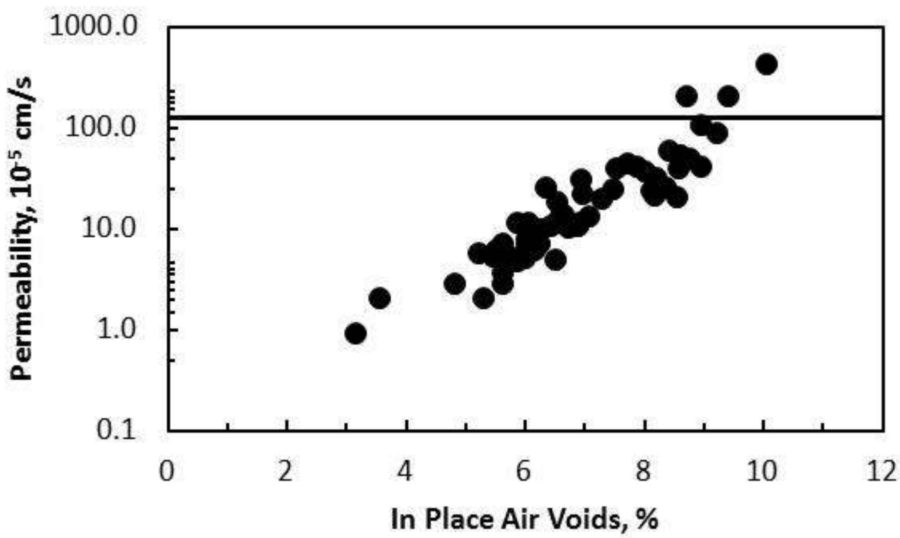


Figure 22. Plot of Estimated Permeability as a Function of In-Place Air Voids for QC Database Mixes.

QC Database: Effect of NMAS on Performance Using WisPave

As discussed in the methods section, the potential performance of an HMA mix as indicated by the WisPave program is solely a function of layer coefficient. For the QC Database mixtures, the value for layer coefficient was estimated from the storage modulus, which in turn was estimated from mixture composition using the Hirsch model. The details of this procedure are described above in the Methods section of this report. The results of this analysis are shown in Figures 23 and 24. Figure 23 is a plot of estimated layer coefficient as a function of NMAS, while Figure 24 is a plot of layer coefficient as a function of air voids. The good relationship between air voids and layer coefficient is not surprising, since there is a very strong relationship between modulus and air voids. The accuracy of the estimated layer coefficients is to some extent validated by the values, which range from 0.40 to 0.48 with an average of 0.44. The assumed value for layer coefficient for new HMA in Wisconsin is 0.44, which agrees exactly with the estimated average for the QC Database mixes.

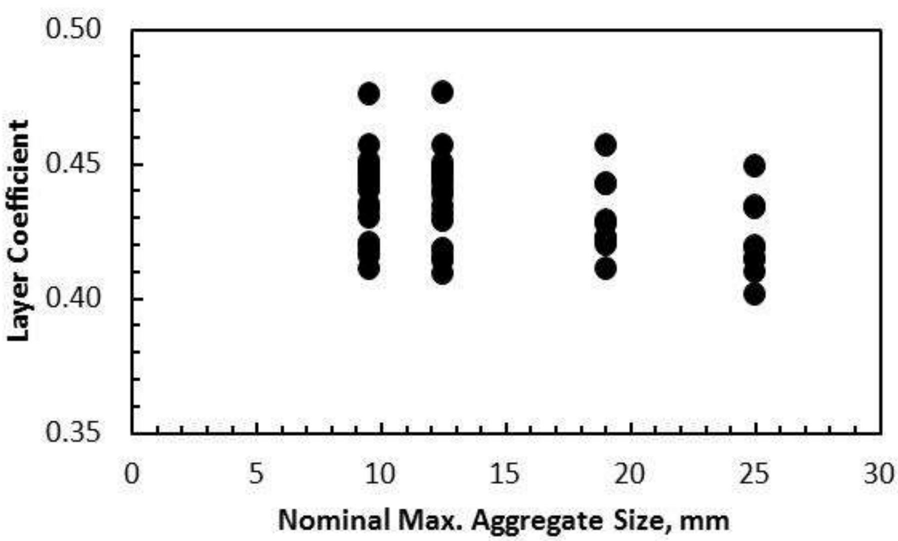


Figure 23. Plot of Estimated Layer Coefficient as a Function of NMAS for QC Database Mixes.

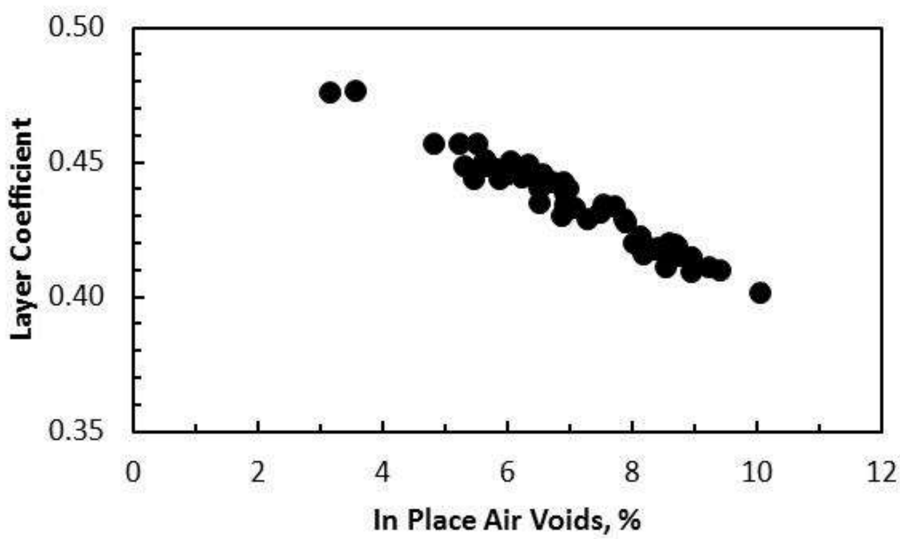


Figure 24. Plot of Estimated Layer Coefficient as a Function of In-Place Air Voids for QC Database Mixes.

Modulus Tests

Laboratory collected data is presented at three selected frequencies (0.1, 1, and 10 Hz) and three test temperatures to provide an initial assessment of the effects of NMAS on dynamic modulus and the variability within the test data. Two replicates were tested for each combination of NMAS and asphalt binder; the results are summarized in Table 14. Results for the PG 58-34 binder grade are presented in Figure 25. In the figure, the error bars represent the range between the two test replicates.

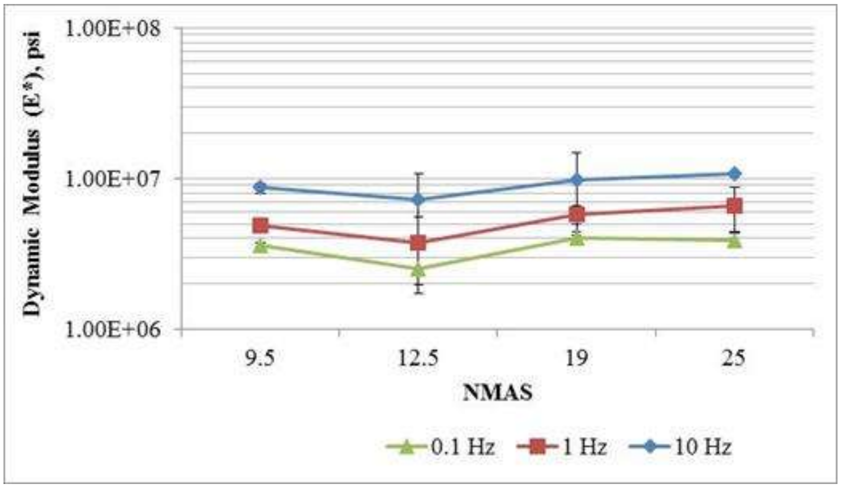
Results presented in Figure 25 indicate that NMAS values from 9.5 mm to 19.0 mm have a negligible effect on dynamic modulus across all test temperatures and frequencies. However, a significant increase in dynamic modulus for the 25 mm NMAS mix is observed at test temperatures of 20°C and 40°C for all frequencies. Variability between replicates is shown at high test temperature/low frequency and low test temperature/high frequency conditions particularly for mixes with NMAS of 12.5 mm and 19.0 mm. The data is presented in a similar fashion for the PG 70-28 binder in Figure 26.

Differing trends are observed for the PG 70-28 binder relative to those discussed previously for the PG 58-34 mixes. Specifically, there is a negligible effect of NMAS on the dynamic modulus and overall the test results are less variable. The only instance in which dynamic modulus is significantly higher within mixes is the 9.5 mm mix at 40°C and 10 Hz, however significant variation between replicates was observed for this combination of temperature and frequency, thus these differences are attributed to experimental error.

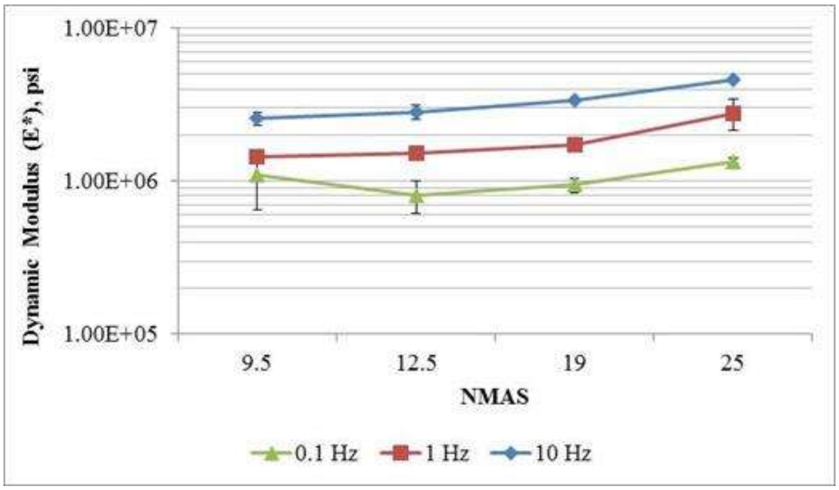
The repeatability of the dynamic modulus was further assessed using precision statement guidance provided in NCHRP 9-29 (Bonaquist, 2011). This report recommends a precision statement for the dynamic modulus based on number of replicates, average value of E* between replicates, and the NMAS of the mix. Precision is evaluated based on the range in test observations relative to the average measured value of E*, and is reported as % acceptable range for specimens. A summary repeatability of the test data collected in this study compared to the % acceptable range for 2 replicates is provided in Table 14.

Results indicate that a majority of the data points collected were below or near (within 10%) of the limits on precision presented in NCHRP 9-29. The average range between replicates across all frequencies was 17%, 17%, and 28% for test temperatures of 4°C, 20°C, and 40°C respectively.

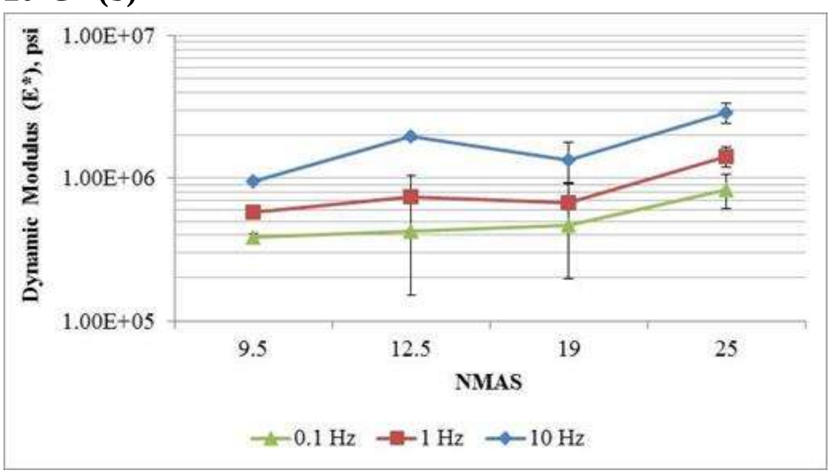
The dynamic modulus master curves for the PG 58-34 and PG 70-28 mixes are presented in Figure 27 and Figure 28 respectively. In general, results indicate that the PG 58-34 binder is more sensitive to changes in NMAS, relative to the PG 70-28 grade, particularly at lower frequencies. For the PG 58-34 mixes the 25 mm mix exhibits the highest values of stiffness followed by the 19.0 mm mix. The ranking between the smaller NMAS mixes varies with frequency, at high frequencies the 9.5 mm mix has the lowest value of stiffness, as frequency decreases from the 9.5 mm mix performs similar to the 19 mm mix, and the 12.5 mm mix stiffness is substantially lower. In contrast, rankings remain consistent across all frequencies for the PG 70-28 mixes and the 9.5 mm mix performs similarly to the 12.5 mm mix.



4°C – (a)

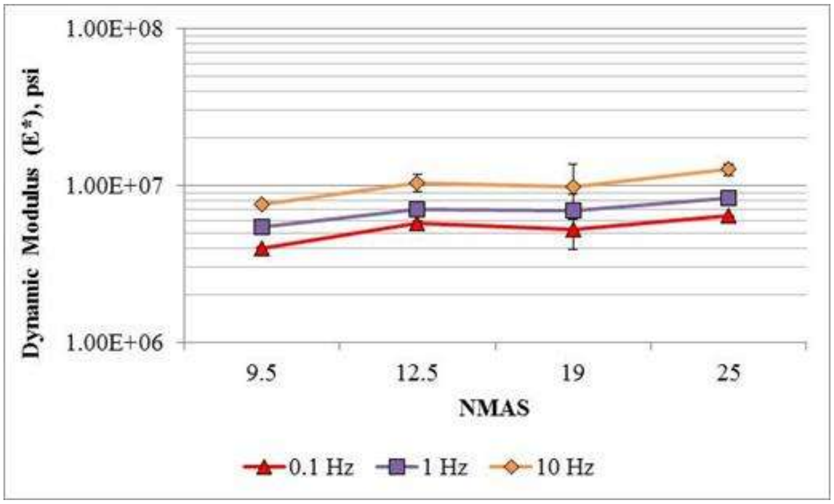


20°C - (b)

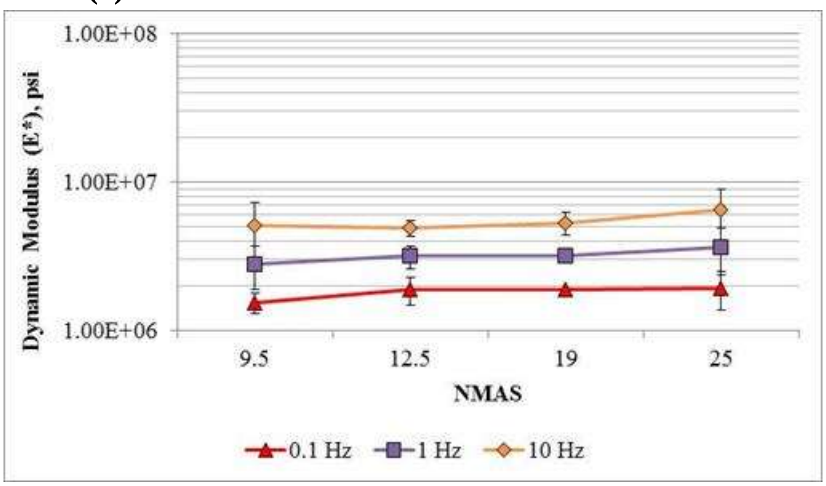


40°C – (c)

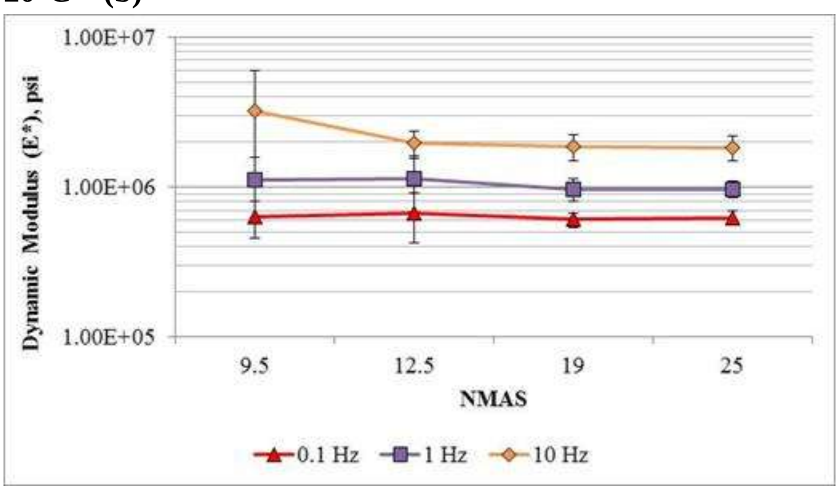
Figure 25. Summary of Laboratory Measured Dynamic Modulus Data at Selected Frequencies and Test Temperatures of 4°C (a), 20°C (b), and 40°C (c).



4°C – (a)



20°C – (b)



40°C – (c)

Figure 26. Summary of Laboratory Measured Dynamic Modulus Data at Selected Frequencies and Test Temperatures of 4°C (a), 20°C (b), and 40°C (c).

Table 14. Comparison of Laboratory Measured Data to NCHRP 9-29 Recommended E* Precision Statement - %Acceptable Range.

NMAS (mm)	Test Temp. (°C)	0.1 Hz		1 Hz		10 Hz		Limit Range N=2	% Greater
		58-34	70-28	58-34	70-28	58-34	70-28		
9.5	4	3.3%	1.4%	10.4%	10.0%	7.6%	1.3%	16%-24%	33%
12.5		31.6%	2.4%	47.8%	10.9%	49.9%	12.8%		
19		4.9%	24.8%	14.4%	28.5%	54.3%	38.7%		
25		11.5%	0.5%	32.0%	4.8%	3.2%	7.8%		
9.5	20	40.9%	15.5%	10.5%	32.8%	10.1%	43.1%	19%-24%	29%
12.5		24.2%	21.6%	1.0%	17.3%	10.7%	12.0%		
19		10.6%	0.2%	2.5%	5.4%	3.3%	17.8%		
25		5.9%	29.6%	22.5%	34.7%	0.4%	37.1%		
9.5	40	6.2%	27.6%	2.9%	43.4%	0.9%	85.8%	26%-37%	29%
12.5		64.1%	36.5%	41.3%	40.8%	5.2%	21.0%		
19		57.7%	10.7%	37.3%	17.2%	32.6%	20.0%		
25		26.8%	10.8%	16.1%	12.7%	16.2%	18.2%		

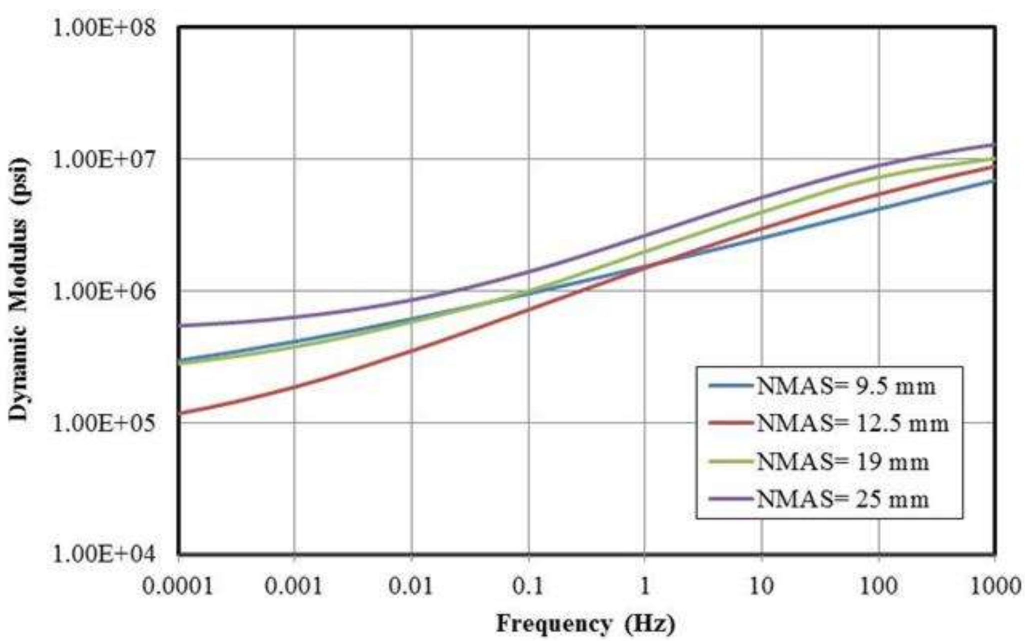


Figure 27. Effect of NMAS on Dynamic Modulus Master Curve – PG 58-34.

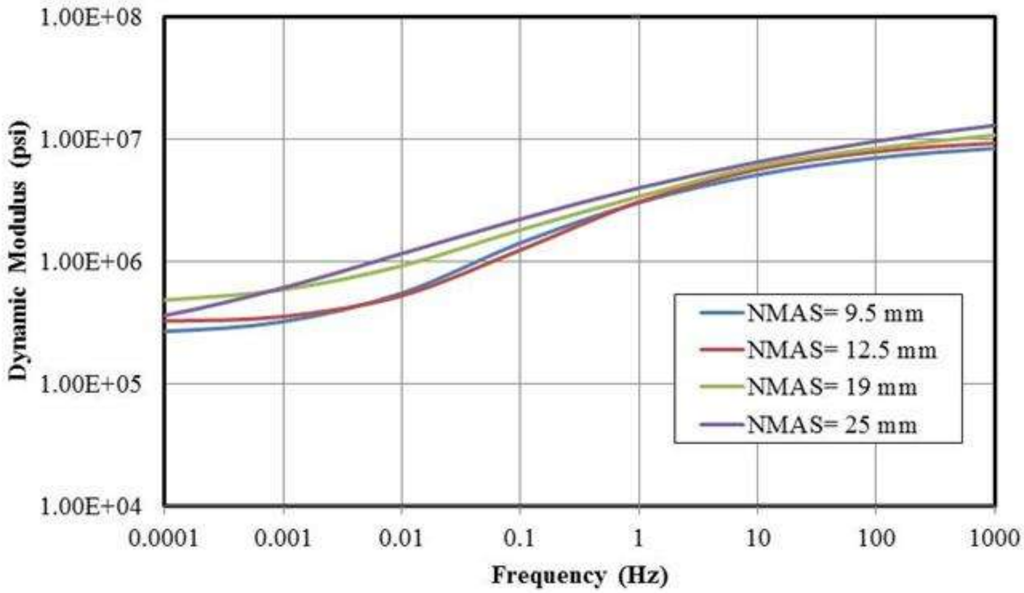


Figure 28. Effect of NMAS on Dynamic Modulus Master Curve – PG 70-28.

The observations previously stated were quantified using statistical analysis to assess the significance of differences observed in NMAS relative to the error inherent to the testing procedure. To conduct the analysis values of E* from the predicted master curve were used at three selected frequencies, 0.01 Hz, 1 Hz, 100 Hz. To represent the variability of the test procedure, predicted values from the master curve were then varied based on the acceptable range provided in the NCHRP 9-29 precision and bias statement. As previously discussed, guidance suggests different limits based on NMAS and average value of E*. For this portion of the study limits ranged from 16%-24%. As a result the analysis included three values for each combination of NMAS and frequency, the average value predicted by master curve and the upper and lower limits calculated based on the precision statement in NCHRP 9-29. Analysis of Variance at a confidence level of 95% was used first to evaluate the effect of NMAS by separating the mixes by binder grade, then to compare the relative effects of NMAS and binder grade on dynamic modulus through use of the entire data set. Results of ANOVA for the effect of NMAS within a given binder grade are presented in Table 15.

Table 15. Effect of NMAS on Dynamic Modulus for PG 58-34 and PG 70-28 Binders.

Binder	Frequency (Hz)					
	0.01		1		100	
	F	p-value	F	p-value	F	p-value
PG 58-34	26.8	<0.0001	20.7	<0.0001	13.3	0.002
R^2adj – PG 58-34	**87.6**		**84.3**		**77.1**	
PG 70-28	30.06	<0.0001	5.72	0.022	6.03	0.019
R^2adj – PG 70-28	**88.0**		**56.3**		**57.8**	

As shown in Table 15 the analysis found that the effect of NMAS was statistically significant for both binders and all frequencies. In general, results are consistent with the observations previously discussed, as the significance of the effect of NMAS is consistent across all frequencies for the PG 58-34 mixes, whereas the effect diminishes at higher frequencies for the PG 70-28 mixes. Tukey pair-wise comparison was used to identify specific combinations of frequency and NMAS that are statistically different at a 95% confidence level. Results are presented in Table 16, the p-value is reported for each comparison, significant differences are highlighted in bold and shaded.

Table 16. Significant Effects of NMAS by Binder PG Grade and Frequency.

Difference in NMAS	PG 58-34			PG 70-28		
	0.01 Hz	1.0 Hz	10 Hz	0.01 Hz	1.0 Hz	10 Hz
12.5 – 9.5	**0.007**	0.996	0.108	0.988	0.999	0.522
19.0 – 9.5	0.945	**0.093**	0.909	**0.007**	0.552	0.186
25.0 – 9.5	**0.011**	**0.001**	**0.023**	**0.0003**	**0.027**	**0.014**
19.0 – 12.5	**0.014**	**0.068**	**0.043**	**0.004**	0.613	0.832
25.0 – 12.5	**0.0001**	**0.006**	**0.001**	**0.0002**	**0.032**	**0.097**
25.0 – 19.0	**0.006**	**0.019**	0.069	0.068	0.184	0.304

Significant differences were observed in the dynamic modulus for the 25.0 mm mix compared to both the 9.5 mm and 12.5 mm mixes for both binder grades. Although this result is inconsequential as in most cases these mixes will be used in different layers of the pavement structure. In regards to applications of mixes as binder courses, the performance of 19.0 mm and

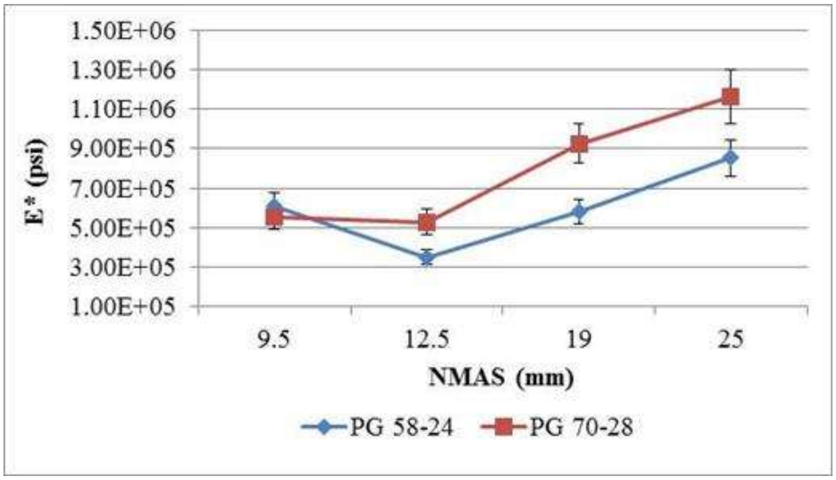
25.0 mm mixes is similar for the PG 70-28 binder and at high frequencies for the PG 58-34. Based on review of the master curves and results in Table 16 the dynamic modulus of the 25.0 mm and 19.0 mm mixes is significantly higher than that of the 12.5 mm mixes for most conditions, a factor that should be considered if a new pavement structure using a 12.5 mm binder course is considered. Assuming that surface courses are limited to NMAS sizes of 12.5 mm and 9.5 mm, results indicate that the performance of these mixes is similar across most conditions within each binder grade.

The effect on dynamic modulus of changing either NMAS or binder grade was assessed through conducting analysis of variance on the complete data set for the three frequencies selected previously. Results are presented in Table 17.

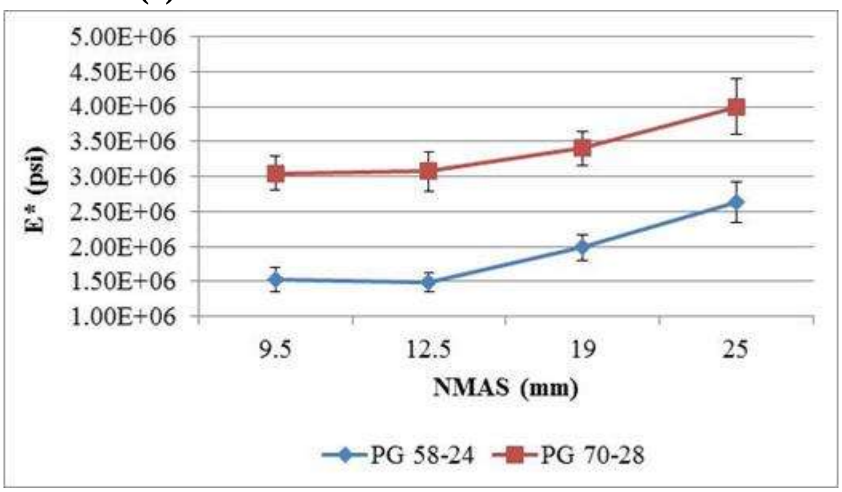
Table 17. Comparison of Effects of Changing NMAS and Binder Grade on Dynamic Modulus.

Factor	Frequency (Hz)					
	0.01		1		100	
	F	p-value	F	p-value	F	p-value
NMAS	25.83	<0.0001	22.61	<0.0001	12.00	<0.0001
Binder PG	16.32	0.001	207.57	<0.0001	11.00	0.004
R^2 adj.	79.6%		92.2%		65.2%	

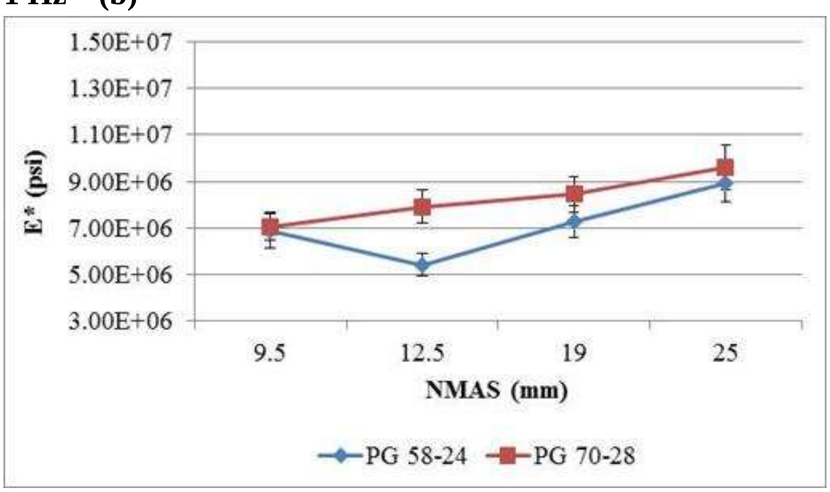
Results indicate that both factors significantly influence dynamic modulus across all selected frequencies. Comparison of the F statistics demonstrates that performance impacts of changing NMAS and binder grade are similar at high and low testing frequencies. However, at the intermediate frequency (1 Hz) changing binder grade from a PG 58-34 to a PG 70-28 has a higher impact on dynamic modulus as the F statistics differ by an order of magnitude. The relative impacts of changing binder grade and NMAS at different testing frequencies is further assessed in Figure 29. Similar to data presented previously the figures depict the dynamic modulus predicted by the master curves at the selected frequencies, the error bars represent the acceptable range in values based on the precision statement provided in NCHRP 9-29 (Bonaquist, 2011).



0.01 Hz – (a)



1 Hz – (b)



100 Hz – (c)

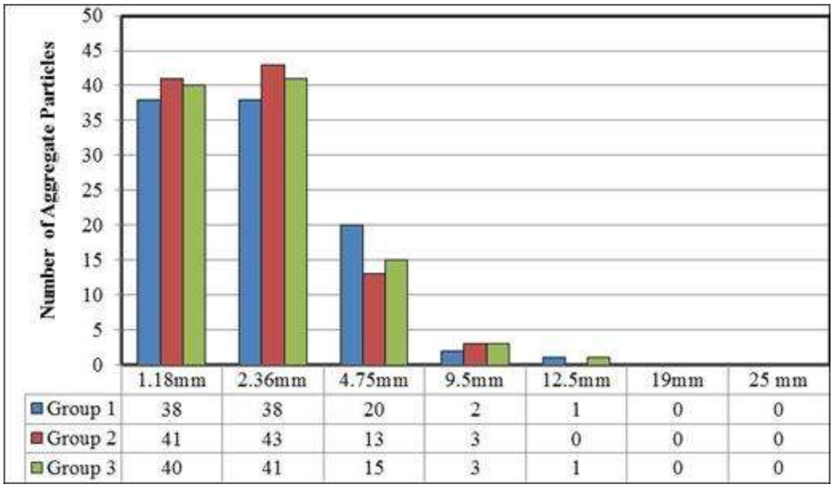
Figure 29. Effects of Binder Grade and NMAS on Dynamic Modulus at Reduced Frequencies of 0.01 Hz (a), 1 Hz (b), and 100 Hz (c).

Data presented in Figure 29 supports results of the statistical analysis as for the testing frequency of 1 Hz, dynamic modulus increases by a factor of two for an increase in binder grade from PG 58-34 to PG 70-28. The effects of binder grade at other testing frequencies are dependent on NMAS, as there is virtually no improvement in E* caused by increasing binder grade at the 100 Hz testing frequency. At 0.01 Hz, significant effects of binder grade are only observed at higher values of NMAS (19.0 and 25.0 mm).

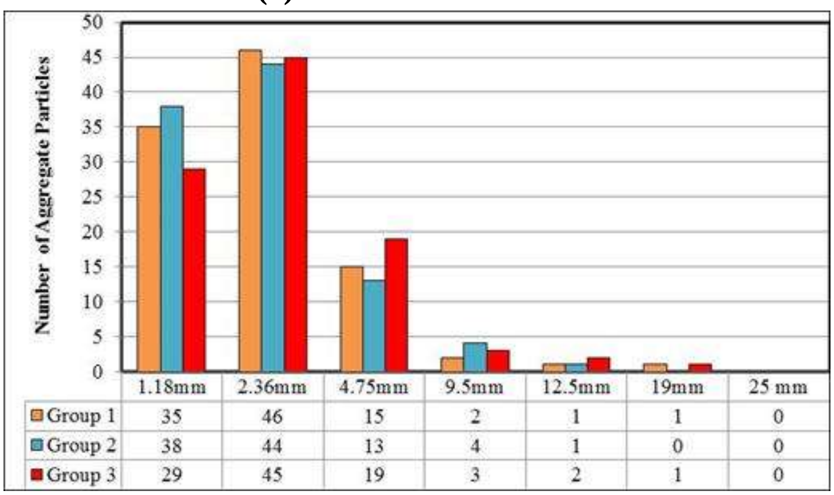
Image Analysis

The effect of NMAS on vertical segregation was evaluated on field cores through use of the IPas[2] image processing software. Sample preparation involves cutting the sample in the vertical direction and capturing the image of the cross-section through use of an office grade desktop scanner. For the analysis the cross-section divided into three sections as shown in Figure 10 and the location of aggregate particles of a given size are located and assigned to one of the three analysis groups. The output of the analysis is a histogram of the number of aggregate particles per analysis group for sizes ranging from NMAS to R8 (1.16 mm). The minimum aggregate size is controlled by the resolution of the scanner. This study was limited to field cores as instructed by the TOC due to the potential differing effects of laboratory and field compaction. Results are presented in Figure 30.

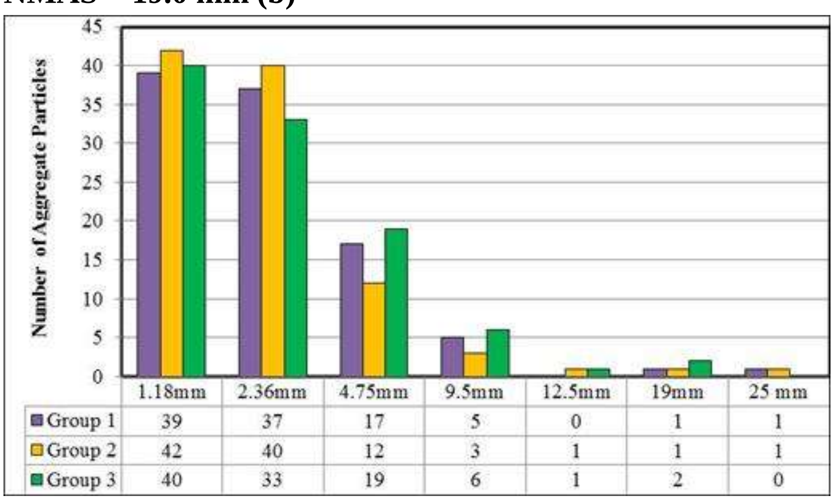
Evaluation of aggregate segregation using image analysis is based on the concept that if significant segregation is present in the mix the distribution of aggregate particles will be significantly different in the analysis sections selected as 1/3 the depth of the core sample. Furthermore, the trend will be consistent, showing an obvious trend with depth in the pavement if segregation is present. Given that this is a relatively new approach, limits to differentiate between mixes that are segregated and those that are not currently do not exist. However, based on analysis of the field cores in this study it appears that there is no segregation in mixes of all NMAS sizes as the number of aggregates for a given size in the analysis groups are similar. The largest variation occurs for the 4.75 mm particle size in which the largest difference between analysis groups is approximately 15% with the lowest number of aggregate particles observed in analysis Group 2 (middle of the sample).



NMAS = 12.5 mm (a)



NMAS = 19.0 mm (b)



NMAS = 25.0 mm (c)

Figure 30. Segregation Analysis – Histogram of Particle Size by Analysis Group for NMAS = 12.5 mm (a), 19.0 mm (b), and 25.0 mm (c).

ANALYSIS AND DISCUSSION

Effect of NMAS on Potential Performance as Indicated by Predictive Models

Various predictive models were used to analyze the relationship between NMAS and various performance-related properties, including modulus, permeability, layer coefficient and predicted thermal cracking temperature. There was little relationship between NMAS and modulus or layer coefficient, suggesting there should be no significant structural issues in using either 9.5-mm or 25-mm mixes in Wisconsin. The permeability of 9.5-mm mixes should in general be somewhat lower than the mixes currently used in Wisconsin. However, the permeability of 25-mm mixes could in some cases be higher than mixes currently in use, and could exceed the maximum value recommended by NCAT. Resistance to thermal cracking should be similar for 9.5-mm mixes compared to existing 12.5-mm and 19-mm mixes. However, 25-mm mixes could exhibit somewhat higher critical low temperatures.

Overall, the analysis using the predictive equations suggests that there should be no performance problems with 9.5-mm HMA mixes in Wisconsin. However, 25-mm HMA mixes could in some cases exhibit unacceptably high permeability values and could also be somewhat more prone to thermal cracking. For these reasons, use of 25-mm mixes in Wisconsin cannot be recommended at this time. Further research on the permeability and low temperature cracking properties of 25-mm mixes should be performed, and there use adopted only if such research indicates satisfactory performance.

Laboratory Evaluation of the Effects of NMAS on Dynamic Modulus

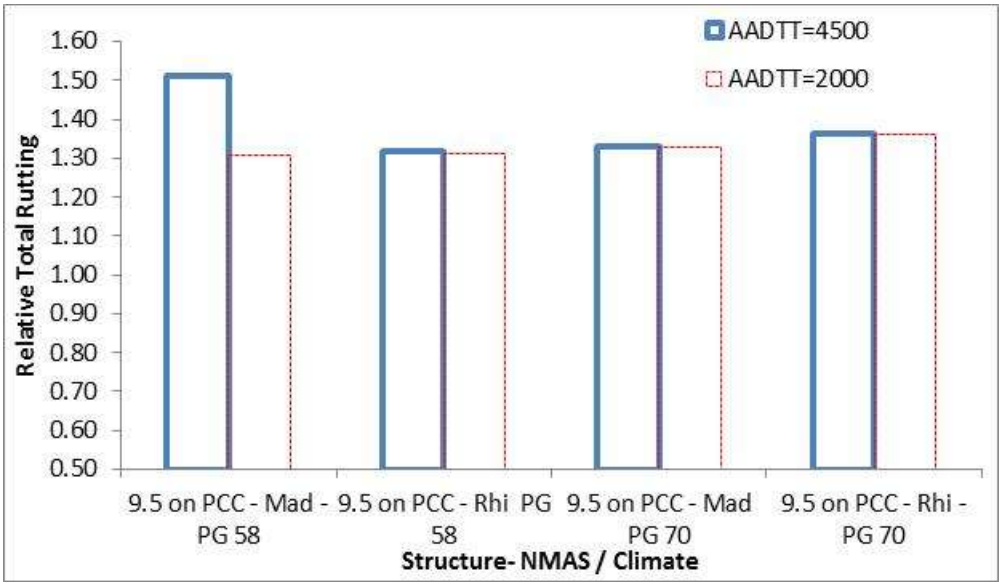
The laboratory experiment was designed to isolate the effects of NMAS on dynamic modulus by holding other mixture volumetric properties (% AC, VMA) constant. For this reason some of the mixes did not meet WisDOT volumetric specifications and the data must be interpreted with caution. Results indicate that when most volumetric properties are held constant NMAS influences dynamic modulus significantly, particularly for softer binder grades. The current mixture design and materials selection process aims to produce a mix that is capable of performing under the service conditions considered in design. Conceptually, the goal of this process is to select a gradation that provides a quality aggregate structure and an asphalt binder capable of maintaining that structure while in-service. The contributions of these two components to overall mixture performance was demonstrated in this study as sensitivity of dynamic modulus to NMAS decreased significantly when the asphalt binder grade was increased to PG 70-28, particularly at lower frequencies. The low frequency condition is representative of the aggregate structure within the mix, results indicate that this structure is better maintained with an increased binder grade. Furthermore, for softer binder grades selection of NMAS higher NMAS does not necessarily result in improved aggregate structure as the dynamic modulus of the 9.5 mm mix was substantially higher than that of the 12.5 mm NMAS.

Given that the mixes used for the laboratory portion of the study were designed to isolate the effects of NMAS and binder grade rather than to evaluate volumetric criteria (which are in practice strongly related to NMAS) the results of the modulus testing cannot be used in isolation to develop guidance concerning the effect of NMAS on HMA performance. However, test results indicate that for mixes with similar NMAS (i.e. 9.5 mm vs. 12.5 mm and 19.0 mm vs. 25.0 mm) similar performance was observed indicating that there is potential to expand the NMAS sizes of products used in binder and surface courses. Furthermore, the analysis using the models for predicting HMA modulus indicate that there is a marginal effect of NMAS on modulus when HMA mixes are designed to current volumetric standards. Therefore it is concluded that modifying the current WisDOT specification to include a wider range of NMAS values should not significantly change typical modulus values for the HMA pavement, or those performance properties related to modulus..

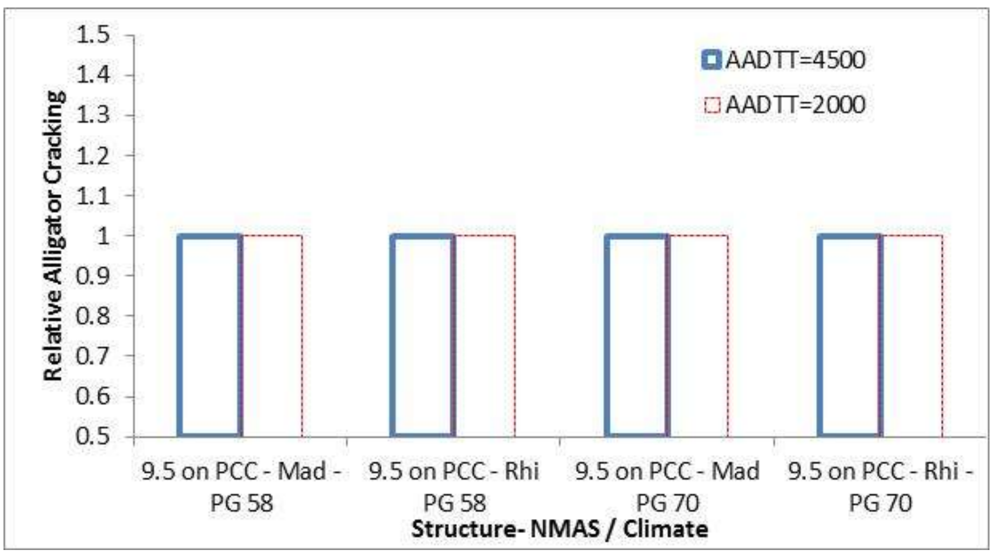
Effect of NMAS and Lift Thickness on Pavement Performance Using the AASHTO MEPDG

A total of 64 MEPDG simulations were conducted in this study, 16 of HMA overlays on PCC and the remaining 48 on newly constructed asphalt pavements. The factors varied to conduct simulations and pavement sections used in the analysis are provided in Table 11**Table** and Table 12**Table** . Results are presented first for HMA overlay on PCC, then newly constructed asphalt pavements. In presentation of the results, side by side comparison of the predicted distress using PG 58-34 and PG 70-28 is provided to allow for comparison of the effects of binder grade in addition to the selected NMAS/pavement structure on performance. Results for HMA overlays on PCC are presented in Figure 31. All figures use the ratio of distress at 9.5 mm to 12.5 mm to represent the change in distress relative to use of the conventional overlay NMAS of 12.5 mm.

Results indicate that NMAS and binder PG grade have a significant effect on predicted rutting. In regards to NMAS the use of the 9.5 mm mix causes an increase in rutting ranging from 30% to 50% relative to the conventional 12.5 mm NMAS thickness. The largest change in rutting performance is realized for the PG 58 binder in the southern WI (Madison) climate. Results also demonstrate that similar rutting performance relative to the PG 58-34 12.5 mm mix can be achieved for the 9.5 mm mix if the binder grade is improved from a PG 58-34 to a PG 70-28. None of the factors selected, including traffic level were shown to influence resistance to alligator cracking. The combined effect of these distresses on IRI is provided in Figure32.



AC – Rutting (a)



Alligator Cracking (b)

Figure 31. Effect of NMAS and Binder PG Grade on MEPDG Predicted Distress for 9.5 NMAS HMA Overlay of PCC Relative to 12.5 mm NMAS – AC Rutting (a), Alligator Cracking (b) .

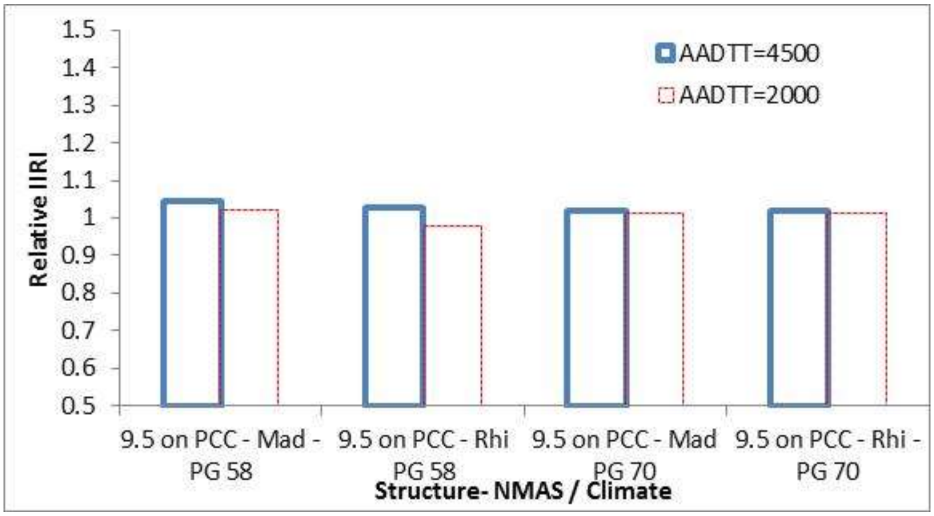
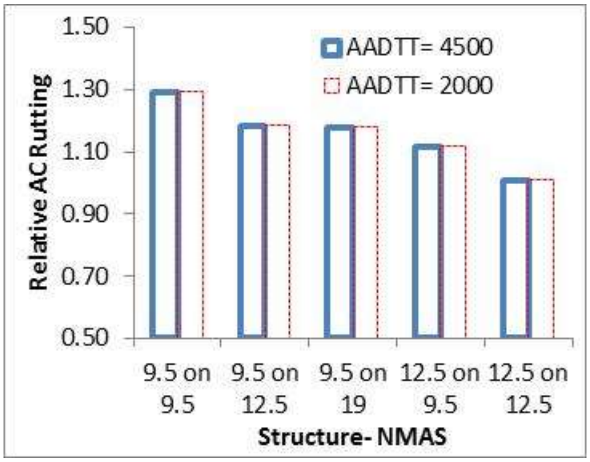


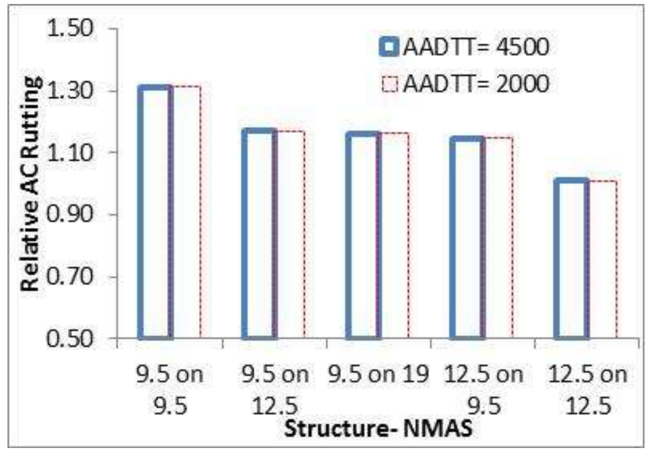
Figure 32. Effect of NMAS and Binder PG Grade on MEPDG Predicted IRI for 9.5 mm NMAS HMA Overlay of PCC Relative to 12.5-mm NMAS.

The increase in rutting observed for the 9.5 mm NMAS mixes relative to the 12.5 mm NMAS mixes did not produce a significant change in IRI as increases in IRI less than 10% were observed across all binder grades and climatic conditions..

Results of MEPDG simulation for pavement structures with various combinations of NMAS are provided below. The analysis is focused on results for the southern WI climate (Madison), similar results regarding the effect of NMAS and binder PG grade were observed for the northern WI climate (Rhinelander) used in the simulation. Results of the predicted distresses for various pavement structures are provided in Figure 33 and Figure 34 for rutting and cracking respectively. Similar to the data presented previously, all distress results are presented relative to the conventional pavement structure which was defined as a 12.5 mm surface course over a 19.0 mm binder course.

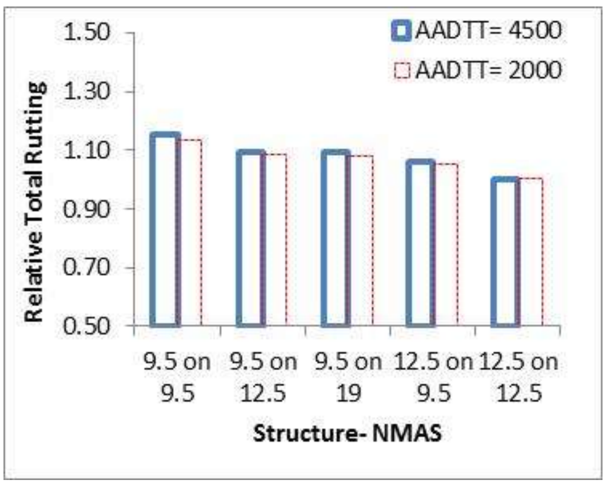
Results presented in Figure 33 indicate that for a given binder grade the alternative pavement structures selected result in an increase in rut depth ranging from 10% to 30%. For both binder grades the largest increase in rutting was observed for a pavement structure consisting of two 9.5 mm lifts and the smallest increase was observed for a pavement structure consisting of two 12.5 mm lifts. No sensitivity to traffic level was observed across both binder grades. As shown in Figure 34, the NMAS combinations for the pavement structures selected result in increases in alligator cracking ranging from less than 10% to approximately 30% relative to the conventional pavement structure of a 12.5 mm surface course placed on a 19.0 mm binder course for the combinations selected in this study. In general, the combination of two 9.5 mm lifts is the only pavement structure that results in an increase in alligator cracking that exceeds approximately 10%.

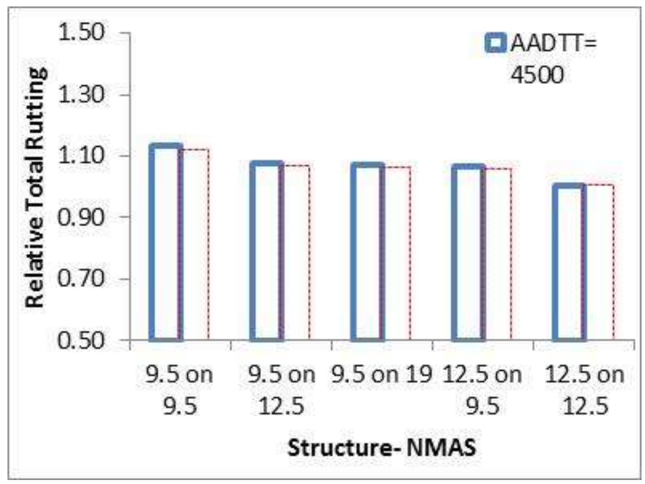




PG 58-34 **PG 70-28**

AC Rutting (a)

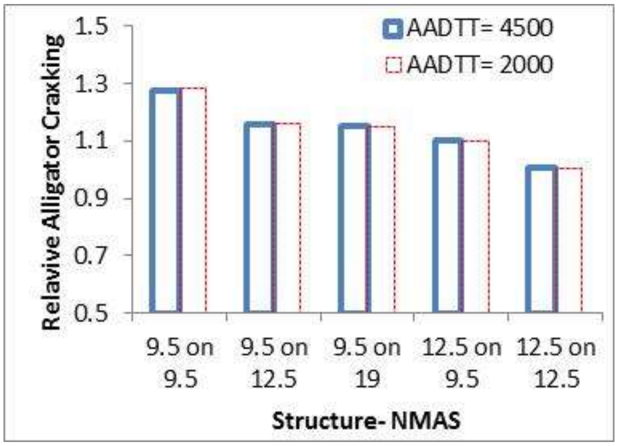




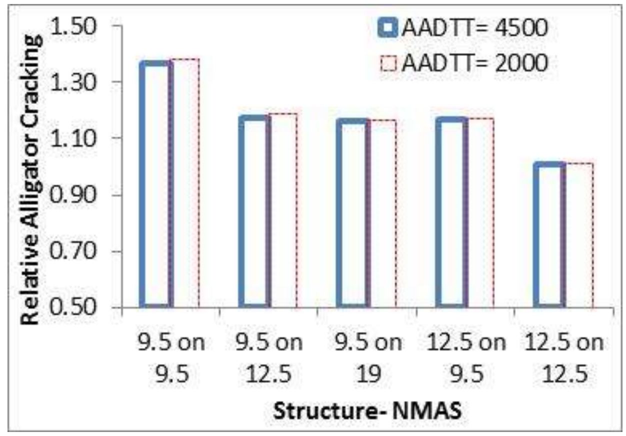
PG 58-34 **PG 70-28**

Total Rutting – (b)

Figure 33. Effect of NMAS and Pavement Structures with Different Combinations of NMAS on MEPDG Predicted Rutting Relative to a Pavement Structure of 12.5 mm on 19.0 mm – AC Rutting (a), Total Rutting (b).



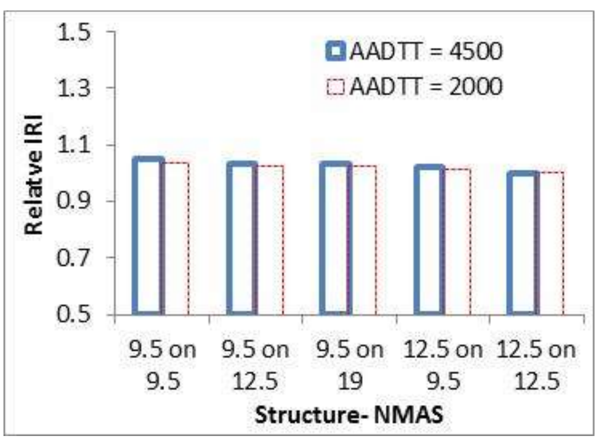
PG 58-34



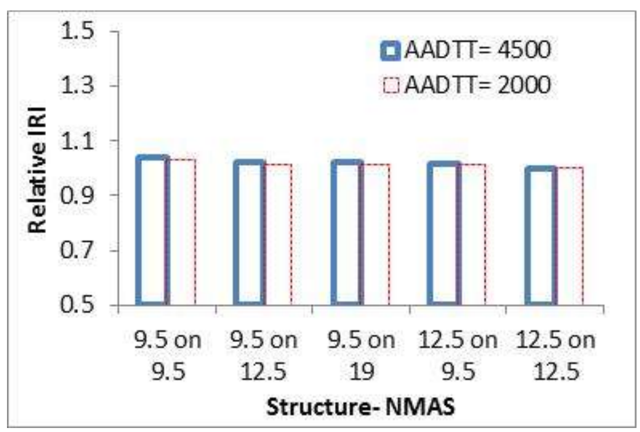
PG 70-28

Figure 34. Effect of NMAS and Pavement Structures with Different Combinations of NMAS on MEPDG Predicted Alligator Cracking Relative to a Pavement Structure of 12.5 mm on 19.0 mm.

Results presented in Figure 35 indicate that the observed performance differences due to use of alternative NMAS combinations in the pavement structure are insufficient to significantly affect IRI as increases in IRI of less than 5% relative to the conventional pavement structure (12.5 mm NMAS on 19.0 NMAS) were observed.



PG 58-34



PG 70-28

Figure 35. Effect of NMAS and Pavement Structures with Different Combinations of NMAS on MEPDG Predicted IRI Relative to a Pavement Structure of 12.5 mm on 19.0 mm.

In summary, for newly constructed pavements or asphalt overlays on existing asphalt pavements the results of the MEPDG simulation support the findings of the laboratory study—9.5-mm NMAS mixes are suitable for use in HMA surface courses. Results also suggest that pavement structures composed of small NMAS sizes (9.5 mm and 12.5 mm) will exhibit similar performance to mixes that use 19.0 NMAS in the binder course. For asphalt overlays of PCC pavements distresses were low at the end of the design period, however the 9.5 mm NMAS mixes exhibited higher rutting relative to the 12.5 mm NMAS. Sensitivity to NMAS size was mitigated by use of the PG 70-28 binder, indicating that for overlays of PCC increasing binder grade should be considered to improve the reliability of in-service performance. It should be noted that these simulations were made by using the laboratory measured HMA dynamic modulus results presented previously as a level one HMA input into the MEPDG software. Therefore, the MEPDG analyses are to a certain extent subject to the same limitations as the modulus testing—the number of mixes were limited and were designed not using typical volumetric composition but with constant VMA and asphalt content.

Effect of NMAS on Segregation as Evaluated Using Image Analysis

The image analysis tool implemented for evaluating mixture segregation proved as an effective, quantitative method for capturing the as-built aggregate structure and how aggregate particles are distributed throughout the depth of the mix. However, application of this tool was limited to one project consisting of only one mix for each NMAS type. Based on the data collected it appears that segregation is not an issue as the number of aggregate particles in each analysis group was similar for the 12.5, 19.0, and 25.0 mm NMAS mixes tested.

Constructability is a critical aspect pavement performance that is not directly addressed in the design phase. One concern with high NMAS mixes is increased potential for segregation leading to decreased mixture durability due to moisture damage and accelerated cracking. Although preliminary results indicate that there is not a substantial risk in segregation for higher NMAS mixtures based on image analysis results, no conclusions regarding the relationship between segregation and NMAS can be made. This is attributed to both the limited data set collected in this study and the absence of standardized test procedures or specifications limits to determine if the mix is segregated.

NMAS and Lift Thickness: Current Practice

The most important current specifications for HMA lift thickness in Wisconsin are given in Table 18. As discussed in the Synthesis of Current Practice presented above, recommendations in NCHRP Report 531 provide for a minimum lift thickness/NMAS ratio of 3 for fine mixtures and 4 for coarse mixture, with a maximum of 5 for both gradation types (Brown et al., 2004). Furthermore emphasis is placed on this report of how rapidly thin lifts cool, which can cause difficulty in compaction. Based upon current Wisconsin specifications, a reasonably value for minimum lift thickness would appear to be 1.5 inches. By applying recommendations of NCHRP Report 531 and this minimum lift thickness to NMAS, an alternate set of lift thickness specifications can be developed, which are shown in Table 19. Table 20 gives the primary control sieve size and percent passing used to define fine and coarse mixes. Some flexibility has been taken in constructing this table to maintain current limits where possible. For instance, applying a minimum thickness/NMAS value of 3 to a 12.5-mm mix results in a minimum lift thickness of 1.5 inches. However, the current minimum of 1.75 inches is only slightly higher than this, and maintaining this value will help prevent confusion during implementation. Adopting these limits would provide additional flexibility in HMA pavement design and would be consistent with current best practice.

In terms of restrictions on the location of different NMAS mixes within the pavement structure, NCHRP Report 673 (2011), a recently developed HMA mix design manual, recommends the use of 9.5- and 12.5-mm mixes in surface courses and 19- and 25-mm mixes in intermediate and base courses. Leveling courses should consist of 9.5-mm HMA. However, this document also discusses the typical structure of perpetual pavements, which make use of rich, small NMAS lower pavement layers to enhance fatigue resistance. Also as discussed previously in this report, it appears that the permeability of 25-mm HMA mixes might often be higher than is consistent with good performance. Based upon these considerations, an NMAS selection guide is given in Table 21.

Table 18. Current Wisconsin Specifications for HMA Layer Thickness.

NMAS, mm	Minimum Layer Thickness, in.	Maximum Lower Layer Thickness, in.	Maximum Upper Layer Thickness, in.
25	3.25	5	4
19	2.25	4	3
12.5	1.75	3	2.5
9.5	1.5	3	2

Table 19. Potential Alternative Specifications for HMA Layer Thickness.

NMAS, mm	*Minimum Layer Thickness, in.*		Maximum Layer Thickness, in.
	Fine Mixes	Coarse Mixes	
25	3	4	5
19	2.25	3	4
12.5	1.75	2	2.5
9.5	1.5	1.5	2

Table 20. Primary Control Sieves and Percent Passing Used to Define Coarse and Fine HMA Mixes.

Nominal Maximum Aggregate Size	Primary Control Sieve	*Percent Passing for*	
		Fine Mixes	**Coarse Mixes**
37.5 mm	9.5 mm	≥ 47	< 47
25.0 mm	4.75 mm	≥ 40	< 40
19.0 mm	4.75 mm	≥ 47	< 47
12.5 mm	2.36 mm	≥ 39	< 39
9.5 mm	2.36 mm	≥ 47	< 47

Table 21. Suggested HMA NMAS for Different Applications.

Design Traffic, ESALs	*Suggested HMA NMAS (mm) for*			
	Surface Courses	**Intermediate Courses**	**Base Courses**	**Leveling Courses**
All levels	9.5, 12,5	12.5, 19	12.5, 19	9.5

CONCLUSIONS AND RECOMMENDATIONS

1. Analysis of NMAS using predictive models indicates that 9.5-mm HMA mixes should have similar performance to 12.5 and 19 mm mixes. However, 25 mm mixes might exhibit unacceptably high permeability values and also could be more prone to low temperature cracking.

2. When all other volumetric properties are held constant NMAS significantly effects dynamic modulus, however the magnitude of the effect is influenced by loading frequency and asphalt binder PG grade. Results are limited to laboratory produced mixes designed specifically to isolate the effects of NMAS. It is recommended that these results are verified with field produced mixes designed to meet current volumetric requirements.

3. In combination laboratory testing and MEPDG simulation indicate that the 9.5 mm NMAS mix is a viable alternative as a surface course as similar values of dynamic modulus and MEPDG predicted distresses were observed. However, conflicting results using these two methods were obtained regarding use of lower NMAS mixes in the binder course. Laboratory testing indicates that significant decreases in dynamic modulus are realized for smaller NMAS sizes relative to 19.0 mm and 25.0 mm mixes, particularly at lower loading frequencies. Whereas, MEPDG analysis indicates a marginal increase in distress when

smaller NMAS sizes are used in the binder course, particularly when the pavement structure is composed of two 9.5 mm NMAS lifts. Further research is recommended to measure dynamic modulus and conduct MEPDG Level 1 analysis on mixes that represent the current pavement structure used and smaller NMAS mixes to resolve this discrepancy.

4. The IPas^2 image analysis technique proved as an effective method to quantify the aggregate sizes present at different depths within the mix. The mixes evaluated in this study did not indicate increased potential for segregation with NMAS size, however the study is limited to one set of cores taken from one project. If mixture segregation is a concern it is recommended that application of the IPas^2 analysis method be expanded to provide an assessment of in-project and between project segregation. This work would benefit WisDOT by identifying if segregation is occurring during field production and would provide an expanded data set to support development of vertical segregation limits using the IPas^2 software.

5. Based upon the research summarized in this report, is it recommended that the Wisconsin specifications for HMA NMAS and lift thickness be modified as described above and summarized in Tables 19, 20 and 21. Restriction of HMA pavement structure to a 12.5-mm mix in the surface coarse and a 19-mm mix in the binder and/or base coarse is not warranted by the results of this study.

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