Five-Year Evaluation of HMA Properties at the AAMAS Test Projects

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To evaluate the process of densification in hot mix asphalt (HMA) pavements and its effect on laboratory properties, the National Center for Asphalt Technology at Auburn University initiated a performance study. It was to be a follow-up study to the Asphalt-Aggregate Mixture Analysis System (AAMAS), developed in the late 1980s. The first phase involved collection and analysis of data from 5-year-old pavements on AAMAS projects. Changes in HMA properties with time were evaluated. Pavement densification was found to continue after 2 years of traffic. Sections with higher initial voids showed greater changes in properties with time. For the first 2 years after construction, properties of HMA mixes were found to be affected significantly by changes in air voids; values of tensile strength and resilient modulus increased with a decrease in voids. Tensile-strain-at-failure values decreased continuously with time. In a majority of cases, 5-year in-place voids were found to be less than mix design voids. Densification generally increased with traffic during the 5-year investigation.

Several studies (*1–3*) indicate that air-void content substantially affects other properties of hot mix asphalt (HMA). To design HMA properly and to avoid distresses in pavements, it is necessary to understand and evaluate changes in air voids under actual field conditions. Changes in other engineering properties of mixes should be evaluated also, as these can be affected by air voids.

This study is a follow up of a larger, overall study that was developed by Von Quintas et al. (4) in the late 1980s the Asphalt-Aggregate Mixture Analysis System (AAMAS). The authors present the changes of properties of pavement mixes as noted from five different projects over a period of 5 years.

BACKGROUND

Part of the broad-based AAMAS study was to establish field-paving projects in different parts of the country and observe changes in the properties of HMA mixes for a period of 2 years. Properties of initial in-place mixes were compared with laboratory-compacted mixes to evaluate laboratory compaction devices. Laboratory specimens were prepared by compactive efforts to match the air-void levels measured in the field immediately after construction. By excluding the effect of difference in air voids between the laboratory specimens and the field cores, the difference in engineering properties between the specimens and the cores was related to the type of compaction device used. By statistical analysis of results from tests on cores and specimens, the earlier study concluded that the Texas gyratory compactor specimens best matched the field cores (*4*). However, one question remained: How do a mixture's properties change as the mix approaches refusal air-void content or density? The AAMAS study evaluated (*4*) changes in mix properties for 2 years. A longer time frame may be required to evaluate changes of mix properties that occur with time.

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OBJECTIVE AND SCOPE

The objective of this paper is to evaluate the changes in HMA properties with time. To achieve that purpose, we obtained field data from 5 AAMAS projects 5 years after their construction.

Field cores were obtained after 5 years of traffic from test sections in the five projects that comprised the AAMAS study. Air voids, indirect tensile strength, tensile strain at failure, and resilient modulus properties of the cores were measured. Data from the 5-year-old cores were compared with the 0 and 2-year data related to cores taken from the same test sections by AAMAS (*4*).

TEST SECTIONS

Table 1 provides a summary of each of the AAMAS test projects including traffic data. Type, depth, and thickness of each of the pavement sections are shown. Detailed discussions are found elsewhere (*4*). Twenty-one cores were taken at random from each of the sections, which were about 300 ft long. Cores were extracted in the same manner they were for the original AAMAS study. The same number of cores was taken from inside, outside, and between the wheel paths. Two sections in each project site were compacted by different rollers at construction. Air-void data from both of the test sections are presented in Table 2. Designated as the standard test sections by AAMAS, other properties were evaluated for those sections only. The breakdown roller used on the standard section was the same as that used over the entire project.

TEST PLAN

Field cores were obtained from the five projects originally used for the AAMAS study (*4*). Cores of 4 and 6 in. were obtained using diamond core barrels from two test sections at each site. Cores were obtained randomly from three lines parallel to the centerline from three locations: inside, outside, and between the wheel paths. The difference in air-void contents of cores from different wheel paths was not found significant, and hence the cores were combined into a single lot for tests. Twenty-one cores were taken from each section: fifteen 4-in. and 6-in. cores were taken. Only the 4-in. cores were used for this part of the study, however.

Figure 1 shows the laboratory test plan. In the laboratory, the cores were used for various tests according to applicable standard ASTM procedures. The Virginia cores were sawed to 50.8 mm (2 in.) thicknesses before testing. Air voids were determined from tests for bulk gravity (ASTM D726) and theoretical maximum density

TABLE 1 Description of AAMAS Test Sections

State Project	Colorado CO-009	Michigan MI-0021	Texas TX-0021	Virginia VA-0621	Wyoming WY-0080
Type of Section	Lower Surface Course	Surface course	Base course	Base Course	Lower Surface Course
Average Thickness of Section (mm)	34.2	45.7	71.6	96.5	55.3
Depth from Surface (mm)	57.1	0.00	76.2	>100.0	50.8
Roller Compaction					
Section VB/SS	Vibratory roller breakdown, static steel wheel roller finish	Vibratory roller breakdown, static steel wheel roller finish	Vibratory roller breakdown, static steel wheel roller finish	Vibratory roller breakdown, static steel wheel roller finish	Vibratory roller breakdown, static steel wheel roller finish
Section PB/SS	Pneumatic rubber-tired roller breakdown, static steel wheel roller finish	Pneumatic rubber-tired roller breakdown, static steel wheel roller finish	------	------	Pneumatic rubber-tired roller breakdown, static steel wheel roller finish
Section SB/PS	------	------	Static steel wheel roller breakdown, rubber-tired roller and static steel wheel roller finish	-------	------
Section SB/SS	------	------	------	Static steel wheel roller breakdown and finish	------
Standard Section	VB/SS	PB/SS	SB/PS	VB/SS	VB/SS
Average Annual Daily Traffic (AADT) (1991 data)	1700	11,100	11,410	680	9630
Percent Commercial Vehicle	3.5	6	10	10	44
Estimated Total ESAL (in millions) 2 year	0.01	0.16	0.26	0.01	0.96
5 year	0.03	0.42	0.69	0.04	2.57

------ Section not used in project
ESAL - 18-kip single axle load
1 inch = 25.4 mm

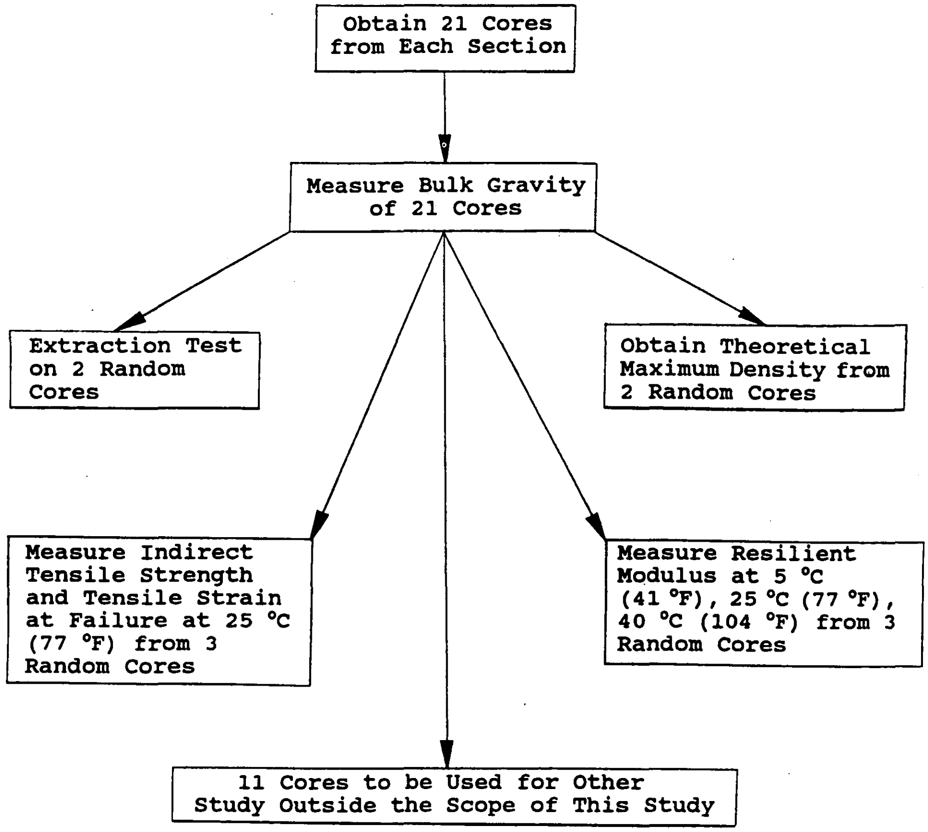

FIGURE 1 Laboratory test plan.

(ASTM D2041). Three random samples were then used to determine indirect tensile strength and tensile strain at failure (ASTM D4123) at 25°C (77°F). Another set of three samples was used to determine resilient modulus (ASTM D4123) at 5°C (41°F), 25°C (77°F), and 40°C (104°F). Other tests included gradation and asphalt content determination by extraction. Because of a shortage of materials, resilient modulus testing was not done on the Michigan cores. Test results were compared with those obtained at construction from in-place samples and after 2 years of traffic (*4*).

RESULTS

A summary of the results from tests on pavement cores at different times after construction is provided in Tables 2–6. Results of air voids, indirect tensile, and resilient modulus properties are compiled in Tables 2–4. Table 6 shows the asphalt content and aggregate gradations measured from the cores recovered at different times after construction. The results on 0- and 2-year cores were obtained from the AAMAS study.

ANALYSIS AND DISCUSSION

Change of HMA Properties with Time

Mix properties include air voids, indirect tensile strength at 25°C (77°F), tensile strain at failure at 25°C (77°F), and resilient modulus at 5°C (41°F), 25°C (77°F), and 40°C (104°F). Because the cores were tested at a different laboratory, interlaboratory variation and the use of different apparatus account for at least some of the difference in properties. Discussed are changes in such mix properties with time.

Air Voids

A summary of air voids data obtained 0, 2, and 5 years after construction is presented in Table 2. The AAMAS study (*4*) reported significant differences between the voids of 0- and 2-year sections. In the present study, *t* tests were performed (with an alpha value of 0.05) with the 2- and 5-year core data to observe any significant difference in voids. Table 7 shows a summary of the *t* test results. In 20 out of 30 cases analyzed, the air voids of the 5-year sections were found to differ significantly from those of the 2-year sections. As expected, in 16 out of the 20 cases, the voids after 5 years were found to be lower than those after 2 years.

Figure 2 shows the change in air voids of different sections with time. The Texas SB/PS and Virginia VB/SS sections showed the highest void changes with time. These two sections were also located in warmer parts of the country where climate might facilitate quicker compaction. The Colorado and Wyoming sections show an increase in voids. The observed difference in voids may be related to a difference in theoretical maximum density values used for calculation of voids at 2 and 5 years. Absorption of asphalt, as suggested by a decrease in asphalt content from extraction results, and

TABLE 2 Summary of Air Voids Data

Type of Specimens	AIR VOIDS (%)				
	PROJECT				
	COLORADO	MICHIGAN	TEXAS	VIRGINIA	WYOMING
MIX DESIGN	4.20	2.80	6.00	5.00	5.00
Initial In-Place	(2.4759)	(2.4748)	(2.4393)	(2.7361)	(2.4516)
VB/SS	8.19	3.74	10.17	5.85	5.77
PB/SS	8.98	4.21	—	—	8.37
SB/PS	—	—	8.75	—	—
SB/SS	—	—	—	7.44	—
Two Year In-Place	(2.4759)	(2.4748)	(2.4343)	(2.7361)	(2.4516)
VB/SS	6.18	2.33	9.38	5.25	4.66
PB/SS	6.26	2.88	—	—	6.50
SB/PS	—	—	7.07	—	—
SB/SS	—	—	—	5.91	—
Five Year In-Place	(2.487)	(2.487)	(2.399)	(2.694)	(2.447)
VB/SS	7.48	2.55	5.55	3.64	4.71
PB/SS	7.47	2.76	—	—	6.51
SB/PS	—	—	3.84	—	—
SB/SS	—	—	—	3.07	—

— Sections not used in project

*NOTE: Numbers in parentheses indicate respective theoretical maximum specific gravities.

TABLE 3 Summary of Indirect Tensile Strength (kPa) Data (25°C)

TYPE OF SPECIMENS	INDIRECT TENSILE STRENGTH (kPa)				
	PROJECT				
	COLORADO	MICHIGAN	TEXAS	VIRGINIA	WYOMING
Mix Design	----	----	----	----	----
Initial In-Place					
VB/SS	621	---	---	1545	986
PB/SS	---	621	---	---	---
SB/PS	---	---	821	---	---
SB/SS	---	---	---	---	---
Two Year In-Place					
VB/SS	483	---	---	1262	1269
PB/SS	---	600	---	---	---
SB/PS	---	---	1718	---	---
SB/SS	---	---	---	---	---
Five Year In-Place					
VB/SS	462	---	---	600	1159
PB/SS	---	600	---	---	---
SB/PS	---	---	1531	---	---
SB/SS	---	---	---	---	---

--- Section not used in project or not a standard section

1 ksi = 6.89 KPa

TABLE 4 Summary of Tensile Strain at Failure Data (25°C)

TYPE OF SPECIMENS	TENSILE STRAIN AT FAILURE (MILS/MM)				
	PROJECT				
	COLORADO	MICHIGAN	TEXAS	VIRGINIA	WYOMING
Mix Design	---	---	---	---	---
Initial In-Place					
VB/SS	15.4	---	---	6.9	6.4
PB/SS	---	14.5	---	---	---
SB/PS	---	---	9.0	---	---
SB/SS	---	---	---	---	---
Two Year In-Place					
VB/SS	1.2	---	---	1.5	1.4
PB/SS	---	5.9	---	---	---
SB/PS	---	---	2.2	---	---
SB/SS	---	---	---	---	---
Five Year In-Place					
VB/SS	1.7	---	---	1.8	2.4
PB/SS	---	3.5	---	---	---
SB/PS	---	---	2.5	---	---
SB/SS	---	---	---	---	---

--- Section not used in project <u>or</u> not a standard section

1 MILS/INCH = 1 MILS/MM

TABLE 5 Summary of Resilient Modulus (10^3MPa) Data

TYPE OF SPECIMENS	RESILIENT MODULUS (10^3 MPa)														
	PROJECT														
	COLORADO			MICHIGAN			TEXAS			VIRGINIA			WYOMING		
TEMPERATURE	5°C	25°C	40°C	5°C	25°C	40°C	5°C	25°C	40°C	5°C	25°C	40°C	5°C	25°C	40°C
Mix Design	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
Initial In-Place															
VB/SS	11.0	4.0	2.3	---	---	---	---	---	---	23.0	6.4	1.7	14.0	4.9	1.4
PB/SS	---	---	---	16.0	3.1	1.1	---	---	---	---	---	---	---	---	---
SB/PS	---	---	---	---	---	---	30.0	8.9	1.6	---	---	---	---	---	---
SB/SS	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
Two Year In-Place															
VB/SS	25.0	3.9	1.9	---	---	---	---	---	---	14.0	8.6	2.5	22.0	7.5	2.1
PB/SS	---	---	---	12.0	3.0	1.0	---	---	---	---	---	---	---	---	---
SB/PS	---	---	---	---	---	---	28.0	10.3	3.0	---	---	---	---	---	---
SB/SS	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
Five Year In-Place															
VB/SS	4.0	1.9	0.3	---	---	---	---	---	---	16.0	2.6	0.5	12.0	3.5	0.8
PB/SS	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
SB/PS	---	---	---	---	---	---	37.0	9.9	1.2	---	---	---	---	---	---
SB/SS	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---

--- Section not used in project <u>or</u> not a standard section

1 ksi = 6.89 MPa

TABLE 6 Summary of Aggregate Gradation and Asphalt Content Data

Aggregate property	Colorado			Michigan			Texas			Virginia			Wyoming		
	JMF	EX1	EX2	JMF	EX1	EX2	JMF	EX1	EX2	JMF	EX1	EX2	JMF	EX1	EX2
Sieve Size															
2	---	---	---	---	---	---	---	---	---	---	---	---	---#	---	---
1 1/2	---	---	---	---	---	---	---	---	---	100	---	---	---	---	---
1	---	---	---	---	---	100	100	100	---	---	98.8	100	---	100	100
3/4	100	100	100	100	100	92.4	94.1	96.5	100	85.0	85.6	93.1	---	95.6	91.2
1/2	93.4	90.0	93.9	92.2	88.8	81.2	77.1	81.6	89.2	---	68.3	74.4	---	72.0	76.7
3/8	68.	75.0	83.8	79.9	75.8	72.3	67.6	70.0	77.4	---	60.2	69.2	---	63.3	66.3
No. 4	45.5	53.0	61.2	60.9	53.2	52.6	51.9	54.2	63.0	46.0	46.5	50.4	---	40.0	46.1
No. 8	32.2	38.0	44.0	47.4	48.6	37.6	33.7	40.0	51.9	34.0	35.6	38.8	---	29.0	32.6
No. 16	23.8	---	32.8	---	37.0	30.4	---	34.3	42.1	---	---	29.8	---	22.0	25.0
No. 30	17.5	---	24.7	24.9	26.0	25.5	23.0	30.8	33.0	---	19.4	21.6	---	17.6	20.2
No. 50	11.2	14.0	17.5	---	14.0	18.6	19.2	27.8	23.1	---	12.1	13.3	---	13.2	15.8
No. 100	7.7	---	10.9	---	7.0	9.4	---	15.3	16.3	---	---	8.3	---	9.0	11.3
No. 200	4.9	6.0	6.7	5.3	4.0	3.7	2.7	6.8	13.6	5.5	5.6	5.7	---	5.5	7.3
Asphalt Content	5.5	5.0	4.6	4.9	5.3	5.1	5.5	5.5	5.0	4.5	4.5	4.9	2.7*	4.7*	3.6*
Placement Temperature, °C		137			137			154			137			135	

JMF Job Mix Formula from State Highway Agency

EX1 Extraction done by AAMAS (4) from bulk mixture samples at the plant during production of mix

EX2 Extraction done in the present study from five year old pavement cores

Data not available

* JMF indicates new asphalt added to recycled mix, EX1 and EX2 show total asphalt content

1 °C = 0.55 (°F-32)

stripping may also be responsible for this increase in voids. A substantial amount of stripping was observed in the cores from Colorado and Wyoming. The average density of the 2-year Colorado cores was found to be slightly higher than that of the 5-year cores. This difference may be the result of differences in sampling patterns or reflect actual loss in density between 2 and 5 years. As expected, the rate of change in voids is seen to be higher in the first 2 years (Figure 2).

Indirect Tensile Strength at 25°C (77°F)

A summary of indirect tensile strength at 25°C (77°F) from cores obtained at different times after construction is presented in Table 3; Figure 3 shows the changes with time.

In the case of Texas and Wyoming, the values increased for the first 2 years and then decreased slightly for the next 3 years. The increase may be related to the significant decrease in voids with time. However, the voids decreased rapidly with time in Michigan, and the indirect tensile strength of those cores did not change significantly with time. This indicates that mixes with higher initial voids had greater tendency for mixture properties to change with time, considering that the initial voids in the Texas and Wyoming cores were considerably higher than those from Michigan. In the cases of the Colorado and Virginia cores, indirect tensile strength decreased with time. Moisture damage may be the reason for this decrease. The AAMAS study observed that aggregates used in the Colorado and Virginia projects were susceptible to moisture damage, and that, during construction of the Colorado project, the mixture became wet when rain fell before and during the compaction process (*4*). A plot of average tensile strength with time is presented in Figure 3. After 2 years, a slight decrease occurred. Because densification and oxidation slow down after 2 years, the change in indirect tensile strength is expected to be less.

Tensile Strain at Failure at 25°C (77°F)

A summary of tensile strain at failure at 25°C (77°F) from cores obtained at different times after construction is presented in Table 4.

A significant decrease in values was observed for the first 2 years. The Colorado and the Texas sections, which had the highest initial voids, showed the greatest change with time. This was expected because a greater number of voids result in greater hardening of asphalt. Despite low initial voids, the Michigan section showed a considerable decrease in tensile strain values also. The combined effect of voids and aging binder may be responsible for the decrease in tensile-strain-at-failure values with time. As may be noted from Figure 4, the values remained almost constant between the second and fifth years after construction.

Resilient Modulus at 5°C (41°F)

A summary of resilient modulus at 5°C (41°F) from cores obtained at different times after construction is presented in Table 5. Five-year resilient modulus data are not presented for Michigan cores; testing was not done because of a shortage of materials.

TABLE 7 Summary of T Tests on Air Voids from 2- and 5-Year Cores

Project	Section	Wheelpath	Results of T Tests Between Air Voids of 2 and 5 Year Old Cores		
			DF	P	Result
Colorado	VB/SS	RWP	14.0	0.2596	NOT DIFFER
		LWP	12.0	0.0000	DIFFER
		BWP	8.0	0.4584	NOT DIFFER
	PB/SS	RWP	7.5	0.0106	DIFFER
		LWP	12.0	0.0001	DIFFER
		BWP	10.0	0.0119	DIFFER
Michigan	VB/SS	RWP	12.0	0.1519	NOT DIFFER
		LWP	12.0	0.8664	NOT DIFFER
		BWP	7.4	0.9392	NOT DIFFER
	PB/SS	RWP	12.0	0.1749	NOT DIFFER
		LWP	12.0	0.0284	DIFFER
		BWP	12.0	0.0305	DIFFER
Texas	VB/SS	RWP	6.4	0.0012	DIFFER
		LWP	13.0	0.0024	DIFFER
		BWP	13.0	0.0001	DIFFER
	SB/PS	RWP	15.0	0.0000	DIFFER
		LWP	14.0	0.0000	DIFFER
		BWP	15.0	0.0001	DIFFER
Virginia	VB/SS	RWP	11.0	0.0240	DIFFER
		LWP	10.0	0.0000	DIFFER
		BWP	9.0	0.0011	DIFFER
	SB/SS	RWP	9.0	0.0001	DIFFER
		LWP	10.0	0.0000	DIFFER
		BWP	7.0	0.0141	DIFFER
Wyoming	VB/SS	RWP	12.0	0.3525	NOT DIFFER
		LWP	12.0	0.1285	NOT DIFFER
		BWP	12.0	0.2264	NOT DIFFER
	PB/SS	RWP	12.0	0.0243	DIFFER
		LWP	12.0	0.0301	DIFFER
		BWP	12.0	0.4243	NOT DIFFER

RWP right wheel path
LWP left wheel path
BWP between wheel path
DF Degree of freedom
P Probability
Alpha = 0.05

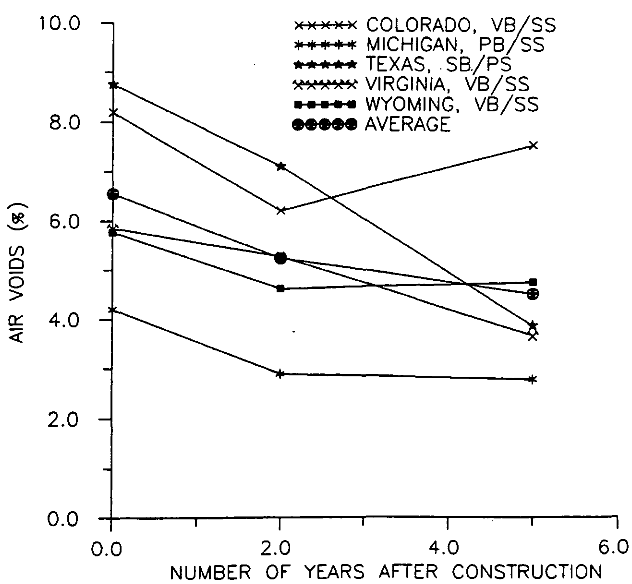

FIGURE 2 Change in percentage air voids with time.

In Colorado and Wyoming, resilient modulus values increased for the first 2 years and then decreased considerably. The pattern in Colorado may be related to change in air voids, which decreased for the first 2 years and then increased. In Texas, the values remained constant for the first 2 years despite a decrease in voids during that period. In the next 3 years, the modulus values increased considerably with a corresponding decrease in voids. In the case of Virginia, the modulus values decreased for the first 2 years, possibly because of damage to the asphalt mixture caused by moisture-susceptible aggregates. Greater changes are observed in the first 2 years than in the later 3 years.

Resilient Modulus at 25°C (77°F)

A summary of resilient modulus at 25°C (77°F) from cores obtained at different times after construction is presented in Table 5; Figure 5 shows the changes of values with time.

In all cases, the values either increased or remained constant during the first 2 years. In the case of Colorado, the values remained constant for the first 2 years despite a decrease in voids. In the next

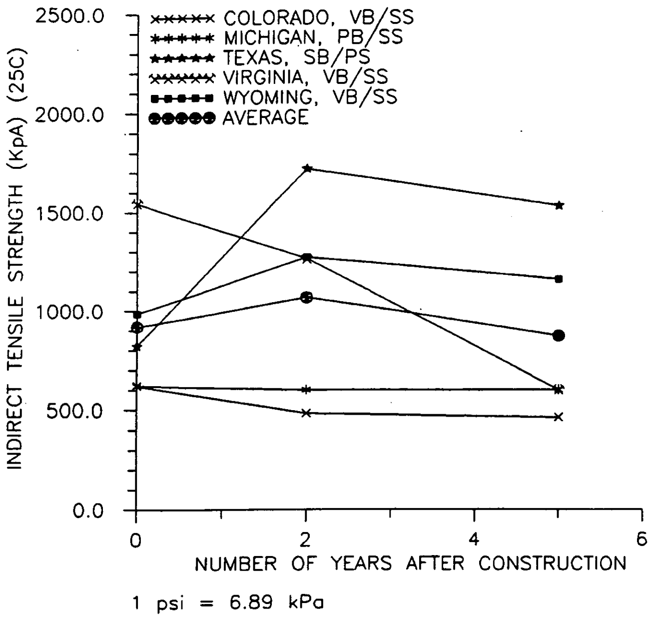

FIGURE 3 Change in indirect tensile strength with time.

3 years, the values dropped considerably with a simultaneous increase in voids. The reason for the decrease may be the use of moisture-susceptible aggregates in the mix. In Texas, the values increased considerably, with an accompanying decrease in voids, and then decreased slightly for the next 3 years, even though the decrease in voids continued. The decrease may be related to a decrease of air voids below optimum and a corresponding decrease in indirect tensile strength during that period. In Virginia, the values increased considerably in the first 2 years and then dropped below the original values in the next 3 years. Use of moisture-susceptible aggregates may have some influence on the decrease of modulus values. In Wyoming, the values increased considerably in the first 2 years with a corresponding decrease in voids. However, for the next 3 years the values dropped considerably, with a simultaneous drop in tensile strength values. A lowering of asphalt content from absorption or stripping may be the reason for this decrease. A plot of average values with time is presented in Figure 5. The pattern of change in resilient modulus is similar to that of indirect tensile strength.

Resilient Modulus at 40°C (104°F)

A summary of resilient modulus at 40°C (104°F) data obtained from cores recovered at different times after construction is provided in Table 5; Figure 5 shows the changes in values with time.

All the sections except Colorado showed an increase in values for the first 2 years, with a corresponding decrease in voids. The mixes having higher initial voids had higher changes with time. In the next 3 years, the values dropped in all the cases. This decrease may be the result of mixture damage caused by a combined action of moisture and aging.

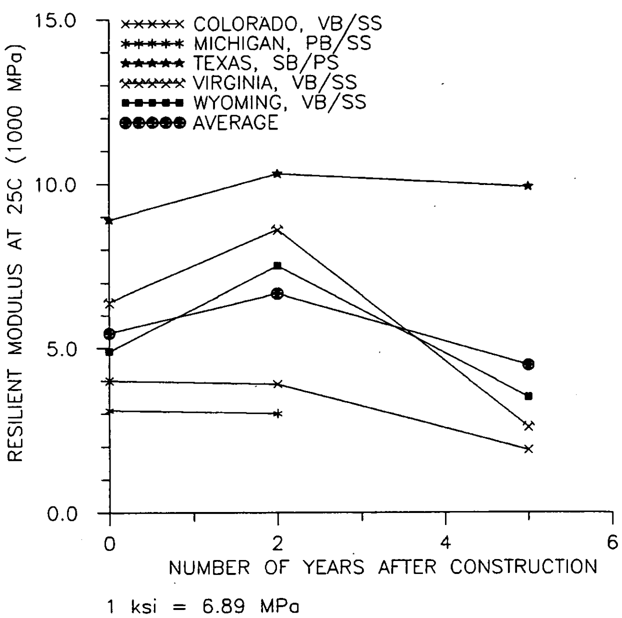

FIGURE 4 Change in resilient modulus at 25°C with time.

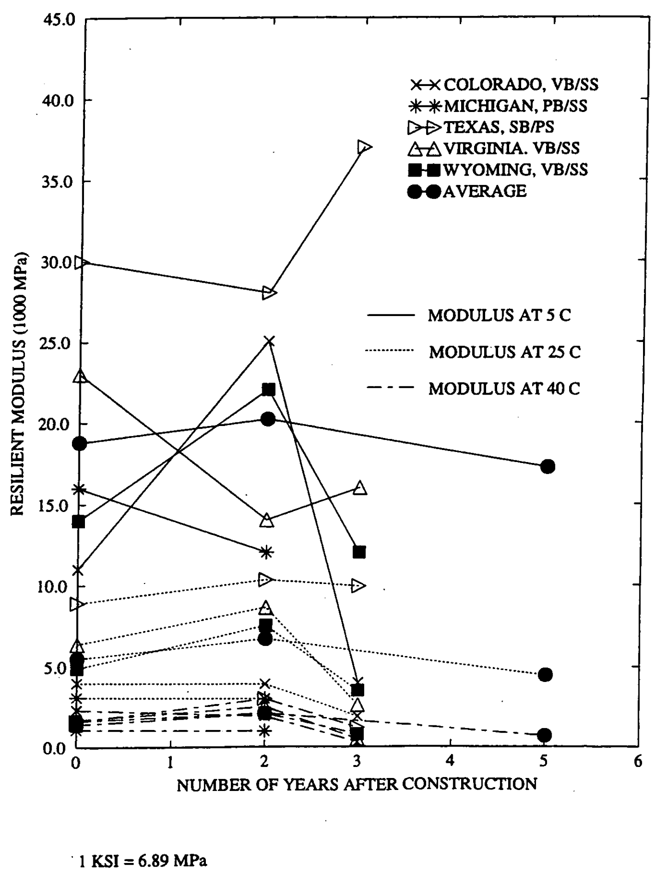

FIGURE 5 Change of resilient modulus with time.

Comparison of Five-Year In-Place Air Voids with Mix Design Voids

A summary of mix-design and 5-year in-place air voids is presented in Table 2; a graphic comparison is presented in Figure 6. Table 1 shows the average annual daily traffic on the different sections. In most cases, the in-place voids are found to be lower than the mix design voids. In Michigan, which had the lowest initial voids, the 5-year voids are found to be equal to the mix design voids. In Colorado, the 5-year voids were greater than mix-design voids, possibly because of moisture damage and related stripping of mixes. It may also be related to very low traffic. As may be observed from the figure, the Texas and Virginia sections have considerably lower air voids than the corresponding mix-design voids. Such high densification may be the result of high temperature in areas in which those sections are located. It may also be caused by changes in mixtures or improper compactive effort in mix design.

Changes of Air Voids with Traffic

Traffic compaction is an important part of the densification process in HMA pavements. Plots of changes in air voids with traffic between 0 and 2 years and between 0 and 5 years are presented in Figures 7 and 8. Traffic is expressed as total equivalent 18-kips single-axle loads in millions, and pavement densification is represented by a ratio of the difference between in-place (2 or 5 year) and original voids and the difference between original and mix design voids. The denominator is the estimated potential change in voids, and the numerator is the actual change in voids to date. A number of 100 percent would mean that the in-place voids were equal to the mix-design voids. Figure 7 shows an increase in densification with an increase in traffic. In a work by Brown and Cross (*5*) similar correlations were obtained. Figure 8 shows a wide scatter in the data, however, even though the trend for increase in densification with increase in traffic is clearly visible.

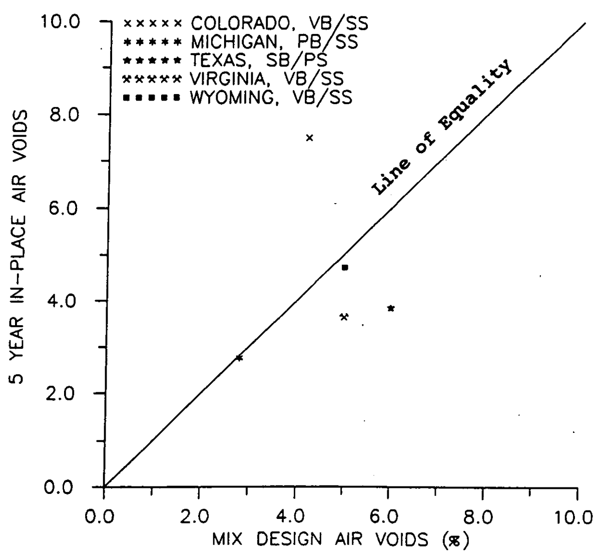

FIGURE 6 Comparison of 5-year in-place and mix design air voids.

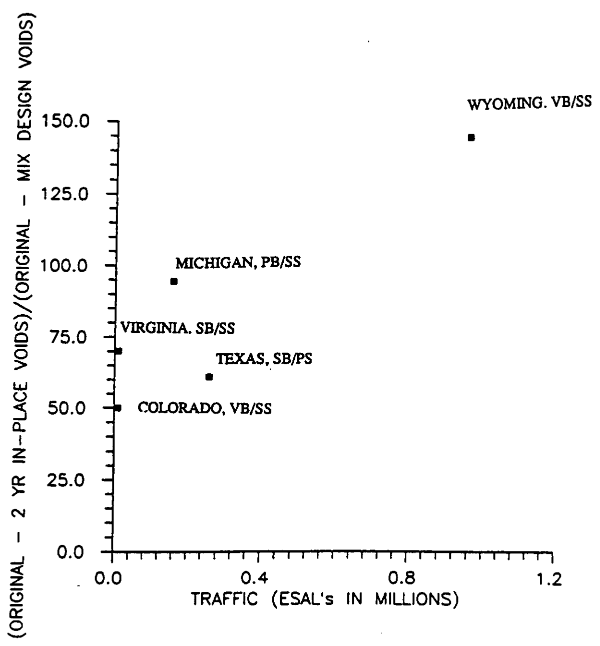

FIGURE 7 Plot of densification versus traffic 2 years after construction.

CONCLUSIONS

On the basis of this study the following conclusions can be made:

- Densification of pavements continues beyond 2 years after construction. In this study, substantial differences in air voids were found between the 2- and 5-year sections.

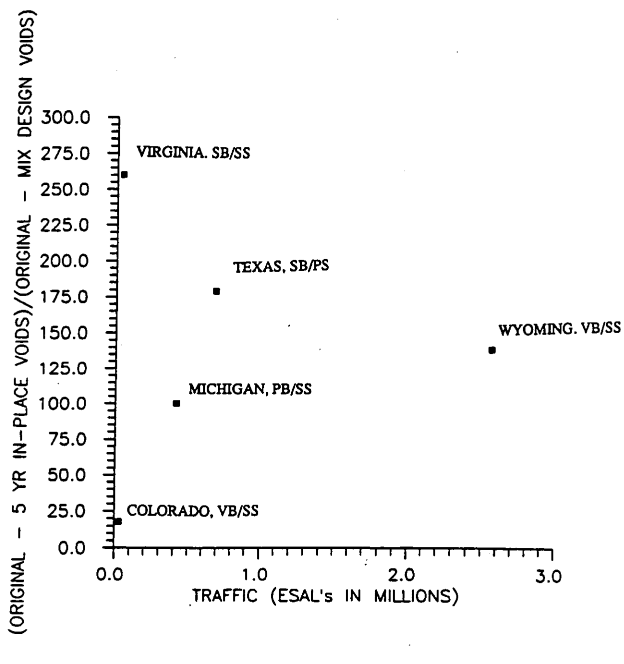

FIGURE 8 Plot of densification versus traffic 5 years after construction.

• Sections with higher initial (at construction) voids have higher rates of void change.

• Tensile strength values increase as air voids decrease; however, the increment may be dependent on initial air voids. Mixes with moisture-susceptible aggregates may suffer a decrease in strength with time.

• Tensile strain at failure values (77°F) for HMA mixes decreases with time, with a sharp decrease within 2 years of construction, and little change between 2 and 5 years.

• There is an increase in resilient modulus at 41F°, 77F°, and 104°F values, with a corresponding decrease in air voids within the first 2 years of construction. After 2 years, the pattern of changes of modulus values is not clear, and it is possibly influenced by a combination of factors like change in voids, moisture damage, and aging.

• In most cases, after 5 years, in-place voids were less than the mix design voids.

• Densification of HMA pavements is substantially influenced by traffic. As traffic increases, the HMA continues to densify—but at a slower rate with time.

RECOMMENDATIONS

On the basis of this study the following recommendations are made regarding further research:

• Densification studies should be carried out with surface courses and heavy traffic-volume pavements for a period of 3 to 4 years to determine changes in various properties of HMA with time and to evaluate the effects of various factors on such changes.

• Available SHRP binder equipment should be used to observe changes in binder properties with time, and to correlate changes in binder properties with changes in HMA properties.

REFERENCES

1. Tunnicliff, D. G., and R. E. Root. *NCHRP Report 274: Use of Antistripping Additives in Asphaltic Concrete Mixtures—Laboratory Phase.* TRB, National Research Council, Washington, D.C., 1984.
2. Powell, W. D., N. W. Lister, and D. Leech. Improved Compaction of Dense Graded Bituminous Macadams. *Proc. Association of Asphalt Paving Technologists,* Vol. 50, St. Paul, Minn., 1981.
3. Huber, G. A., and G. H. Heiman. Effects of Asphalt Concrete Parameters on Rutting Performance: A Field Investigation. *Proc., Association of Asphalt Paving Technologists,* Vol. 57, St. Paul, Minn., 1987.
4. Von Quintas, H. L., J. A. Scherocman, C. S. Hughes, and T. W. Kennedy. *NCHRP Report 338, Asphalt-Aggregate Mixture Analysis System (AAMAS).* TRB, National Research Council, Washington, D.C., 1991.
5. Brown, E. R., and S. A. Cross. Comparison of Laboratory and Field Density of Asphalt Mixtures. In *Transportation Research Record 1300,* TRB, National Research Council, Washington, D.C., 1991.

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