USE OF RECLAIMED ASPHALT PAVEMENTS (RAP) IN AIRFIELDS HMA PAVEMENTS

AIRFIELD ASPHALT PAVEMENT TECHNOLOGY PROGRAM
Auburn University, 277 Technology Parkway
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FINAL REPORT APPENDICES

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RENO

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for

AAPTP Project 05-06

Submitted to

Airfield Asphalt Pavement Technology Program

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APPENDIX A – EXPANDED BACKGROUNDS FOR LITERATURE REVIEW

Appendix A includes additional information on some of the reviewed literature included in the final report. It should be noted that not all studies presented in the final report are coupled with expanded information in appendix A.

EXPANDED REVIEW OF RESEARCH EFFORTS

Minnesota Department of Transportation

Recycled Asphalt Pavement has been used in Minnesota for over 25 years. The most commonly used method is to mill material from an existing pavement and incorporate it into a new asphalt mix. The Minnesota DOT Specification 2350 allows up to 30% recycled material depending on the traffic level and Specification 2360 allows 20%. In 2004, Li et al. investigated the effect of RAP type and percentage on the final asphalt mixture properties using both traditional methods as well as the dynamic modulus as proposed by the new AASHTO design guide *(1)*.

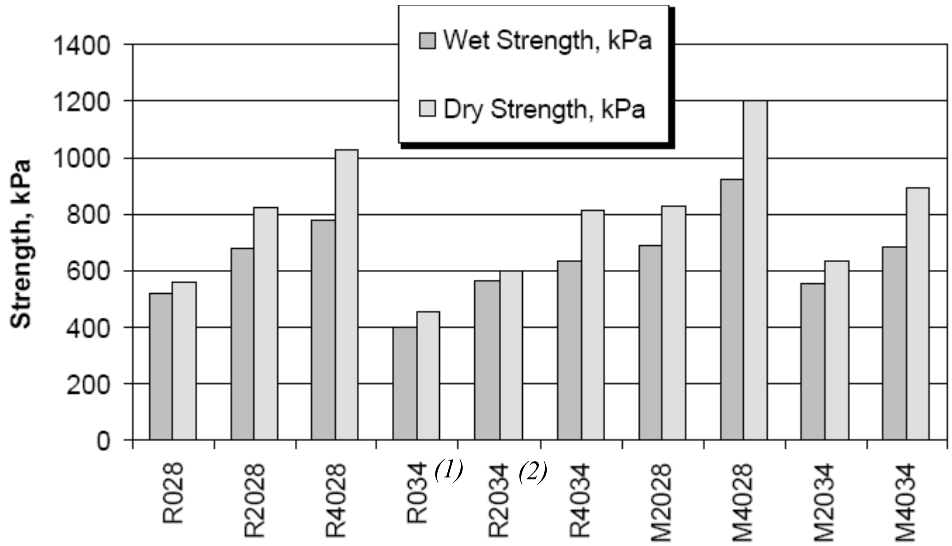
Ten mixtures consisting of three RAP percentages (0, 20% and 40%), two virgin asphalt binders (PG58-28 and PG58-34), and two RAP sources (RAP and millings), were studied. The RAP sources were provided by a local contractor and were identified as follows:

- Millings – RAP from a single source, milled up from I-494 in Maple Grove. The RAP has a binder content of 4.3% and an extracted binder grade of PG76-22.-
- RAP – RAP combined from a number of sources and crushed at the HMA plant. The RAP has a binder content of 5.4% and an extracted binder grade of PG70-22.

The RAP material was blended with virgin aggregate such that all samples tested had approximately the same gradation. The Superpave mix design process was used to determine the optimum asphalt content for the mixtures. Table A1 shows the optimum binder content of the various RAP containing mixes along with the percent of RAP binder in the total designed mix. It should be noted that the mix design was performed only for the mixes with the PG58-28 asphalt binders and the same optimum binder content was used for the mixes with the PG58-34 asphalt binders. Moisture susceptibility of the various mixtures was evaluated in accordance with AASHTO T283. Test results showed all ten mixtures pass the minimum tensile strength ratio of 75%. Additionally, the test results indicated an increase in tensile strength and a decrease in the tensile strength ratio as the percentage of RAP or millings increases. Figures A1 and A2 show the indirect tensile strength and the tensile strength ratio of the evaluated mixtures at 77°F, respectively.

Table A1 Optimum Binder Content and Percent of RAP Binder in Total Mix

Mixture Type	Mixture ID	Optimum Binder Content	RAP Binder Content in Total Mix
0% RAP + PG58-28	R028	5.85%	0.00%
20% RAP + PG58-28	R2028	5.38%	0.86%
40% RAP + PG58-28	R4028	5.29%	1.72%
20% Millings + PG58-28	M2028	5.32%	1.08%
40% Millings + PG58-28	M4028	5.05%	2.16%



(1) 0% RAP + PG58-34
(2) 20% RAP + PG58-34

Figure A1 Indirect tensile strength data of Minnesota evaluated mixtures

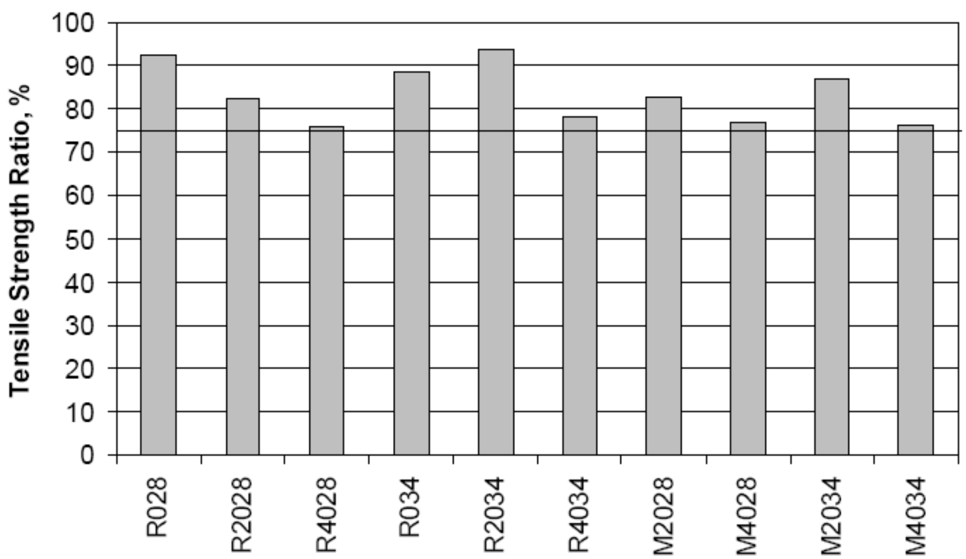


Figure A2 Tensile strength ratio data of Minnesota evaluated mixtures

All ten mixtures were subjected to dynamic modulus testing and indirect tensile (IDT) creep and strength testing. The asphalt binder was extracted from the tested dynamic modulus samples and the PG grades were determined according to current Superpave specifications (AASHTO M320).

The limited data obtained in this project showed that the addition of RAP increased the dynamic modulus and that the asphalt binder grade and RAP source had a significant effect on the mixture modulus. However, this effect was reduced at low temperatures and high frequencies. Figure A3 shows the variation of dynamic modulus with the RAP content. It was also found that the mixtures containing RAP exhibited higher variability than virgin mixtures (i.e., 0% RAP) and that the variability increased with the increase in RAP content as shown by the separation of the cores in Figures A4-A6. Dynamic modulus test results were observed to have more variability at low temperatures.

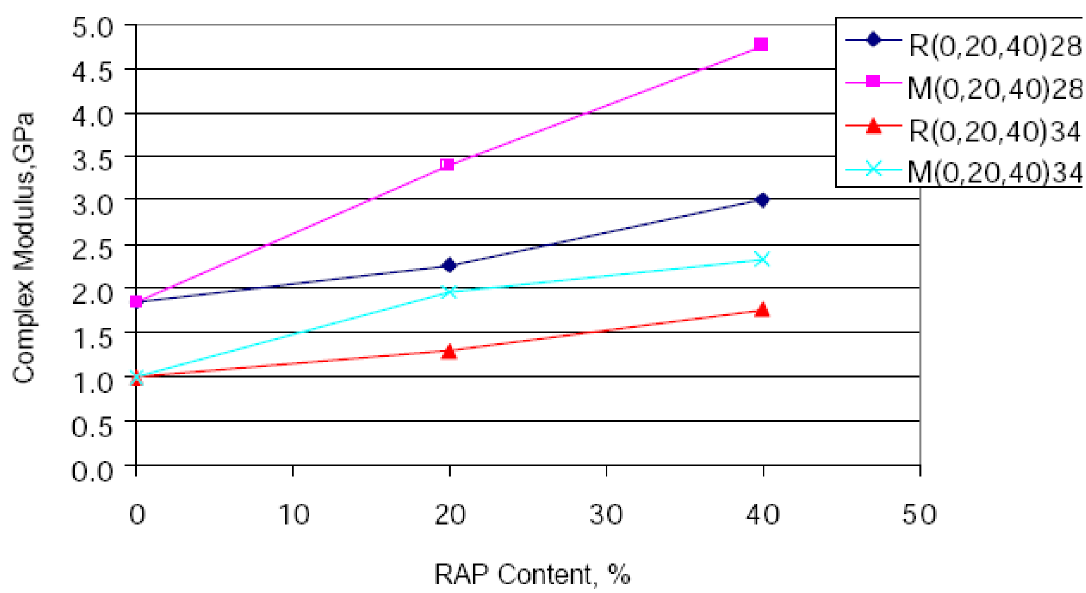


Figure A3 Effect of RAP on complex modulus at 21°C and 1.0 Hz

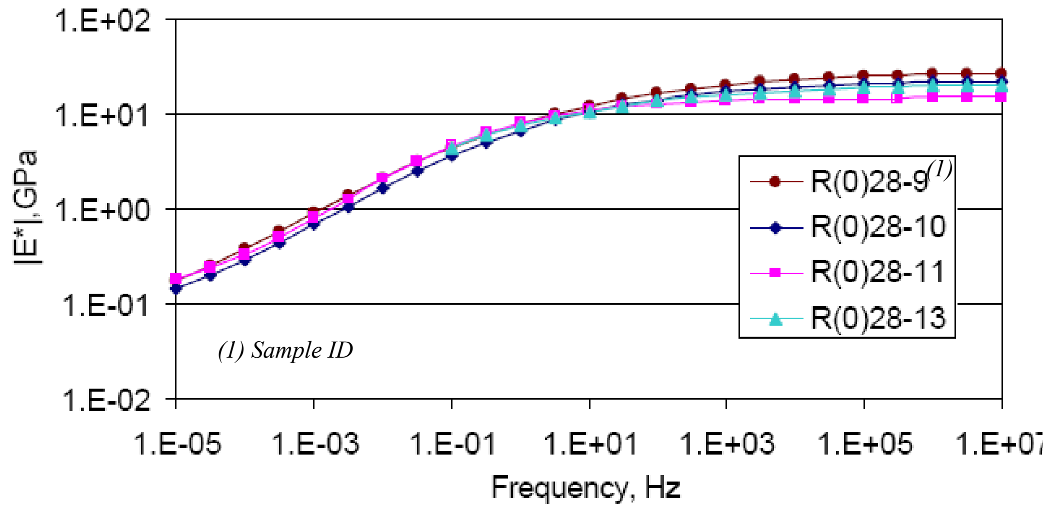


Figure A4 Dynamic modulus master curves at 4°C for 4 replicates of the R028 mix

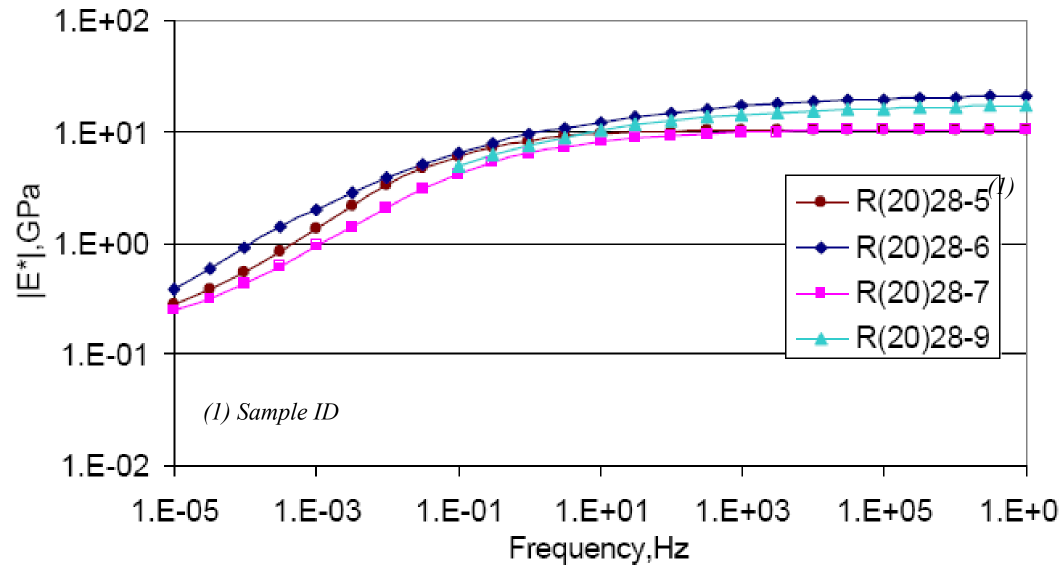


Figure A5 Dynamic modulus master curves at 4°C for 4 replicates of the R2028 mix

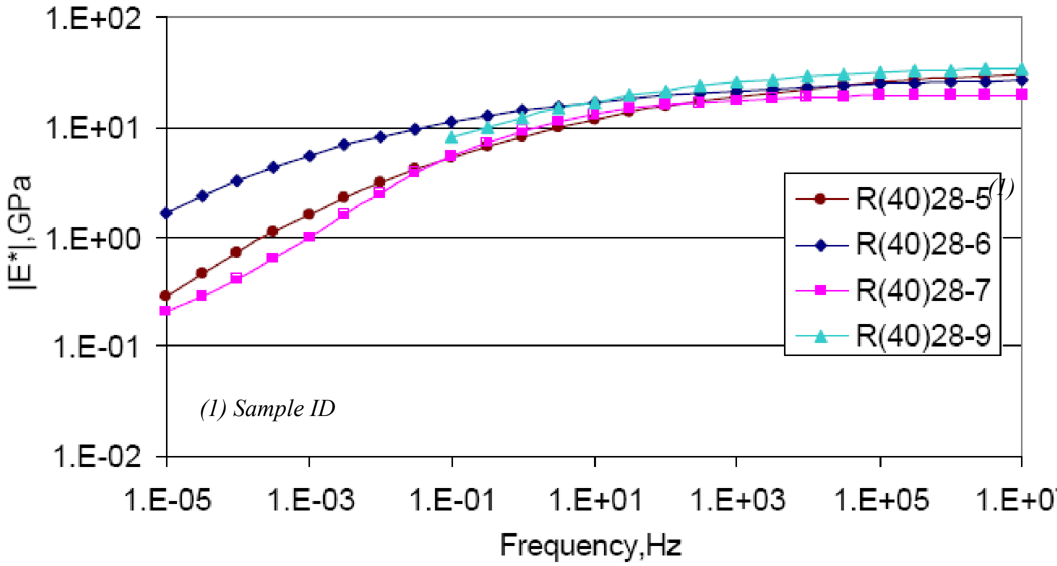


Figure A6 Dynamic modulus master curves at 4°C for 4 replicates of the R4028 mix

Indirect tensile creep and strength tests were performed on the ten mixtures at -18°C and -24°C according to AASHTO TP9 and the results are shown in Table A2. The researchers indicated that it was difficult to obtain consistent results with the creep tests because of problems with the extensometers. The creep test data showed that as the percentage of RAP or millings increases, the stiffness increases and that the mixtures with PG 58-34 binder were softer than the mixtures with PG58-28 binder at the temperature of -18°C.

Table A2 Indirect Tensile Creep Test Results

Mixture	-18 °C		-24 °C	
	Stiffness (GPa)	COV (%)	Stiffness (GPa)	COV (%)
R028	7.89	6.5	15.12	14.6
R2028	13.34	14.0	9.05	56.4
R4028	17.68	3.0	11.47	77.3
R034	6.32	16.5	4.84	14.2
R2034	9.00	12.6	4.70	29.2
R4034	14.42	15.5	17.26	-
M2028	17.93	12.9	24.44	-
M4028	19.25	37.0	-	-
M2034	7.44	38.9	16.18	47.5
M4034	11.83	36.1	11.19	2.4

The Minnesota researchers recommended that further studies need to be conducted which include testing of additional mixtures and asphalt binders to encompass a wider range of materials used in Minnesota, comparing the laboratory test data to the field, and studying the effects of the recycled materials on the performance of the mixtures at low temperature *(1)*.

In 2008, Li et al. *(2)* evaluated the resistance to low temperature cracking of the same ten mixtures using the Semi Circular Bending (SCB) fracture test. The SCB test is used to simulate the lower temperature cracking of asphalt pavements by measuring the fracture energy of the mixes. Higher fracture energy in the SCB test reflects a higher resistance to low temperature cracking. All ten mixtures were tested for SCB fracture energy at three temperatures (-18, -24, and -36°C). Table A3 shows the fracture energy for all 10 mixtures at various temperatures along with the corresponding coefficient of variations. The researchers found that the percentage of RAP in the mixtures significantly affect the fracture resistance. Fracture testing results indicated that the control mixtures have the highest fracture energy, hence the best resistance to low temperature cracking, with the 20% RAP mixtures exhibiting fracture resistance abilities similar to the control mixtures. The addition of 40% RAP significantly decreased the low temperature fracture resistance. Additionally, the experimental data indicated that the RAP source does not significantly affect the fracture resistance of the asphalt mixtures at low temperatures.

Table A3 Fracture Energy and Coefficient of Variations

Mix	Fracture Energy					
	At -12°C		At -24°C		At -36°C	
	Gf (J/m^2)	CV (%)	Gf (J/m^2)	CV (%)	Gf (J/m^2)	CV (%)
R028	742.3	13.64	275.9	16.07	225.3	5.35
R2028	530.3	0.96	266	22.59	246.5	8.89
R4028	511	14.09	217.9	14.5	166.1	18.62
M2028	577.4	9.64	281.4	10.6	215.3	2.83
M4028	446.9	6.52	250.5	4.8	165	10.55
R034	1104.3	22.85	440.3	14.2	298	12.03
R2034	1009.5	14.96	381.9	19.76	252.6	17.01
R4034	697.6	15.51	263.6	4.42	197.4	19.69
M2034	792.8	4.99	328.9	6.58	244.6	3.8
M4034	527.9	11.2	280	13.9	219.6	16.72

National Cooperative Highway Research Program

In 1997, in an effort to incorporate the usage of RAP in Superpave HMA mixtures, the National Cooperative Highway Research Program (NCHRP) funded a three years research study to evaluate the effects of RAP on Superpave mixtures *(3)*. The objective of this research was to develop guidelines for incorporating RAP in the Superpave System, and prepare a technician's manual for the various laboratory tests involved in the design process. The main research was conducted in three separate, but related, laboratory studies: black rock study, binder effects study, and mixture effects study.

The research used three sources of RAP, two virgin binders and one virgin aggregate. All mixtures were produced following the Superpave specification for the 12.5 mm nominal maximum size mix.

- RAP properties: The three RAP sources were selected based on the viscosity of the recovered RAP binders. The sources covered a range of low (Florida, PG82-22, 5.9% RAP binder), medium (Connecticut, PG82-22, 4.9% RAP binder), and high stiffness (Arizona, PG82-10, 5.3% RAP binder) RAP materials.
- Virgin binder properties: Two virgin binders; a PG52-34 representing a soft base asphalt that could be blended with RAP in cool climates and a PG64-22 representing a medium grade asphalt binder that could be blended with RAP in warm climates, were selected.
- Virgin Aggregate: Limestone and natural sand from Kentucky were chosen as the virgin aggregate source for this project. The gradations of these materials were artificially manipulated to meet the required gradation of the blends.
- Mixtures (Blend): The mix gradation was selected to meet the Superpave 12.5 mm nominal maximum size specification. In order to study the impact of RAP content on the properties of the binders and mixtures, four mixtures were generated at 0%, 10%, 20% and 40% RAP materials. The gradation of the virgin aggregate was adjusted to produce blended mixtures that are consistent with the Superpave specifications for the 12.5 mm nominal maximum size.

Black Rock Study

The objective of this effort was to assess the impact of the manufacturing method on the final properties of the blended mixtures. This effort was conducted on two RAP contents of 10% and 40%. Three methods of introducing the RAP into the mix were evaluated.

- **Black Rock:** Samples were prepared using virgin and extracted RAP aggregates mixed with a virgin binder. This method only uses the aggregates extracted from the RAP and discards the binder in the old mix.

- **Actual Practice:** Samples were prepared using virgin binder and virgin aggregates mixed with un-separated RAP materials. This method uses the RAP material as is without separating the two components prior to mixing.
- **Total Blending:** Samples were prepared using virgin and extracted RAP aggregates mixed with a blend of virgin and recovered RAP binders at a specific percentage. This method uses the properties of the RAP binder and the required final binder grade to estimate the properties and percentage of the virgin binder.

The resistance of the blended mixtures to rutting, fatigue, and thermal cracking were evaluated. The repeated shear constant height (RSCH) test was used to measure the mixtures resistance to rutting. The frequency sweep (FS) test was used to measure the mixtures resistance to fatigue cracking. The indirect tension (IDT) test was used to evaluate the mixtures resistance to thermal cracking. Full descriptions of the various tests are presented in the NCHRP 9-12 report.

The results of the performance tests showed no significant differences among the three blending methods at the low RAP content of 10% while the three blending methods behaved significantly differently at the 40% RAP content. The magnitude of the influence that the RAP has at the 40% level on the final mix properties depends on the method of blending (i.e. Black Rock, Actual Practice or Total Blending). The Actual Practice technique was recommended for the other parts of the study. The results of this part of the study supported the common belief that each RAP mix should be individually designed to fully assess the interaction between the RAP materials and the virgin materials in the blended mix.

Binder Effect Study

The objective of this effort was to evaluate the impact of the properties of the RAP binder on the properties of the virgin binder. The study evaluated the impact of RAP at 10%, 20%, and 40% on the critical temperatures of the virgin binder. The critical temperatures are the temperatures at which a binder just meets the specified Superpave criteria, for example, a G*/sinδ of 1.00 kPa for the unaged (original) binder. Tables A4-A6 show the test results for the various binders evaluated in this study.

Table A4 Measured Critical Temperatures and Performance Grades of the Florida Blended Binders

Aging	Property	PG 52-34 Blends, % RAP binder			PG 64-22 Blends, % RAP binder		
		10%	20%	40%	10%	20%	40%
Original	DSR G*/sinδ	57.9	60.3	66.7	70.1	72.4	75.2
RTFO	DSR G*/sinδ	57.5	62.5	66.7	69.5	71.4	73.6
PAV	DSR G*sinδ	13.3	14.9	18.2	19.2	20.1	24.8
	BBR S	-23.3	-22.5	-20.3	-18.9	-17.7	-14.1
	BBR m	-23.2	-21.8	-19.5	-16.1	-14.6	-12.4
PG	Actual	PG 57-33	PG 60-31	PG 66-29	PG 69-26	PG 71-24	PG 73-22
	MP1	PG 52-28	PG 58-28	PG64-28	PG 64-22	PG 70-22	PG 70-22

Table A5 Measured Critical Temperatures and Performance Grades of the Connecticut Blended Binders

Aging	Property	PG 52-34 Blends, % RAP binder			PG 64-22 Blends, % RAP binder		
		10%	20%	40%	10%	20%	40%
Original	DSR G*/sinδ	57.7	60.1	67.3	70.6	71.8	77.5
RTFO	DSR G*/sinδ	57.8	60.1	67.0	69.5	70.3	77.4
PAV	DSR G*sinδ	15.0	15.8	15.9	20.4	20.4	23.8
	BBR S	-23.0	-21.8	-20.5	-18.2	-17.6	-16.9
	BBR m	-23.2	-21.9	-19.8	-15.5	-14.8	-12.5
PG	Actual	PG 57-33	PG 60-31	PG 67-29	PG 69-25	PG 70-24	PG 77-22
	MP1	PG 52-28	PG 58-28	PG 64-28	PG 64-22	PG 70-22	PG 76-22

Table A6 Measured Critical Temperatures and Performance Grades of the Arizona Blended Binders

Aging	Property	PG 52-34 Blends, % RAP binder			PG 64-22 Blends, % RAP binder		
		10%	20%	40%	10%	20%	40%
Original	DSR G*/sinδ	57.4	63.5	71.9	69.4	74.4	79.5
RTFO	DSR G*/sinδ	57.6	63.0	70.5	68.6	72.8	78.2
PAV	DSR G*sinδ	14.5	17.4	22.5	20.5	22.7	27.0
	BBR S	-22.4	-20.9	-17.6	-18.1	-17.0	-14.3
	BBR m	-22.8	-19.9	-14.8	-14.7	-11.9	-9.5
PG	Actual	PG 57-32	PG 63-29	PG 70-24	PG 68-24	PG 72-21	PG 78-19
	MP1	PG 52-28	PG 58-28	PG 70-22	PG 64-22	PG 70-16	PG 76-16

The results of this part of the research supported the following recommendations: a) at the 10% RAP, the effects of the RAP binder are negligible, b) at the 20% RAP content, the effects of the RAP binder can be compensated for by using a virgin binder that is one grade softer on both the high and low temperature grades, and c) at the 40% RAP content, a blending chart should be used to either determine the appropriate virgin binder grade or to determine the maximum amount of RAP that can be used with a given virgin binder.

This experiment also evaluated the possibility of analytically evaluating the impact of the RAP binder on the critical temperatures of the blended binder (i.e. RAP binder plus the virgin binder). The Asphalt Institute (AI) equation shown below was used to analytically determine the critical temperatures of the blended binder.

$$T_c = T_{virgin} + (\%RAP)(T_{RAP} - T_{virgin}) \quad \text{(Equation A1)}$$

where, T_c = the critical high, intermediate, or low temperature of the blended binder
T_{virgin} = the critical high, intermediate, or low temperature of the virgin binder
T_{RAP} = the critical high, intermediate, or low temperature of the RAP binder

%RAP = percentage of RAP in decimal

The results of the AI equation were compared to the actual measured critical temperatures of the blended binder with and without RTFO aging of the RAP binder. The data in Table A7 shows close agreement between measured and estimated temperatures with the RTFO aging of the RAP binder. This indicates that the AI equation can be used to get reasonable estimates of the impact of the RAP binder on the critical temperatures of the blended binder. However, the estimated critical temperatures should only be used at the RAP source approval stage and actual testing of the blended binder should be conducted during the mix design process.

Table A7 Comparison of Estimated and Actual Blended Binder Grades.

RAP	**Blend**	**PG52-34 Blends**		**PG64-22 Blends**	
		Estimated	**Actual**	**Estimated**	**Actual**
Florida	0% (Virgin)		PG52-28		PG64-22
	100%		PG82-22		PG82-22
	10%	PG52-28	PG52-28	PG64-22	PG64-22
	20%	PG58-28	PG58-28	PG70-22	PG70-22
	40%	PG64-28	PG64-28	PG70-22	PG70-22
Connecticut	0% (Virgin)		PG52-28		PG64-22
	100%		PG82-16		PG82-16
	10%	PG52-28	PG52-28	PG64-22	PG64-22
	20%	PG58-28	PG58-28	PG70-22	PG70-22
	40%	PG64-28	PG64-28	PG70-22	PG76-22
Arizona	0% (Virgin)		PG52-28		PG64-22
	100%		PG88-4		PG88-4
	10%	PG52-28	PG52-28	PG64-22	PG64-22
	20%	PG58-28	PG58-28	PG70-22	PG70-16
	40%	PG64-22	PG70-22	PG76-16	PG76-16

Mixture Effect Study

The mixture effects study was performed to investigate the impacts of adding RAP on mixtures properties. This study evaluated the impact of adding 0%, 10%, 20%, and 40% RAP on the properties of the final mix. All combinations of the three RAP sources and two virgin binders were evaluated. The virgin binder grades were not changed according to blending chart calculations. The RSCH test was used to assess the mixtures resistance to rutting, the flexural beam fatigue test was used to assess the mixtures resistance to fatigue cracking, and the IDT test was used to assess the mixtures resistance to thermal cracking. Table A8 shows the percent of RAP binder in the total designed mix.

Table A8 Optimum Binder Content and Percent of RAP Binder in Total Mix

Mixture	RAP Binder Content in Total Mix
AZ RAP 10%	0.53%
AZ RAP 20%	1.06%
AZ RAP 40%	2.12%
CT RAP 10%	0.49%
CT RAP 20%	0.99%
CT RAP 40%	1.97%
FL RAP 10%	0.50%
FL RAP 20%	1.00%
FL RAP 40%	2.00%

The permanent shear strains from the RSCH test for all three RAP sources tended to decrease with increasing RAP content, but there was variability in the data. In some cases, the shear strain increased or did not change significantly. The researchers related this to possibly the variability in the data or to the balance between increasing binder stiffness and, possibly, decreasing aggregate shear resistance when higher percentages of RAP are used. Overall, as RAP content increased the permanent shear strain was found to decrease. The IDT data in Tables A9 and A10 showed that no effects on creep stiffness with RAP contents up to 10%, but over 10% the stiffness increases.

Table A9 IDT Stiffness at 60 sec using PG52-34.

% RAP	Temperature (°C)	Stiffness (MPa)	
		Arizona	Connecticut
0	0	1,142	
	-10	4,415	
	-20	10,298	
10	0	1,831	1,467
	-10	6,326	5,226
	-20	14,196	13,291
20	0	2,823	1,869
	-10	7,255	6,275
	-20	15,183	13,171
40	0	5,231	2,766
	-10	11,395	7,293
	-20	18,831	14,303

Table A10 IDT Stiffness at 60 sec using PG64-22.

% RAP	Temperature (°C)	Stiffness (MPa)	
		Arizona	Connecticut
0	0	5,076	
	-10	11,736	
	-20	19,113	
10	0	5,243	3,357
	-10	11,483	8,071
	-20	19,400	16,305
20	0	8,727	6,228
	-10	16,385	11,908
	-20	24,365	21,536
40	0	9,226	6,731
	-10	14,281	12,477
	-20	21,033	16,416

The flexural beam fatigue results (Table A11) showed that the fatigue life of the mix decreases with the addition of the RAP if the grade of the virgin binder is not adjusted to account for the inclusion of the RAP. In general, the researchers concluded that a softer binder is needed to compensate for the increased mixtures stiffness due to the inclusion of the RAP materials and to help improve the fatigue and low temperature cracking resistance of the mixtures.

Overall, this research revealed that the impact of RAP on the properties of the mix depends on the stiffness of the RAP materials. The stiffer the RAP materials, the more adversely the properties of the final mix are affected.

Table A11 Beam Fatigue Average Life Cycles for Various Mixtures.

RAP source	RAP content (%)	Average Fatigue life (cycles)			
		PG 52-34		PG 64-22	
		Low strain level (400μ)	High strain level (800μ)	Low strain level (400μ)	High strain level (800μ)
Virgin	0	500,000+	129,106	500,000+	12,589
Connecticut	10	500,000+	131,121	428,842	18,164
	20	500,000*	73,767	500,000	12,822
	40	421,909*	33,533	330,424*	19,042
Arizona	10	500,000+	150,530	352,578	13,332
	20	500,000+	41,259	233,199	6,526
	40	278,816	16,892	87,090	6,608

+ Beam did not reach half stiffness even after 500,000 cycles.
* One beam failed at or below 500,000 cycles and other beam did not fail after 500,000 cycles.

North Central Superpave Center

In 2002, the North Central Superpave Center conducted a regional pooled fund study to investigate the laboratory performance of superpave asphalt mixtures incorporating RAP *(4)*. This study was closely coordinated with the national study of NCHRP 9-12. Specifically, this regional project study looked at typical materials from the north central United States to determine if the findings of NCHRP 9-12 were valid for Midwestern materials and to expand the NCHRP findings to include higher RAP contents.

Three RAP materials from Indiana, Michigan, and Missouri were evaluated. Mixtures were designed and tested in the laboratory with each RAP, virgin binder and virgin aggregate at RAP contents up to 50%. The laboratory mixtures were compared to plant produced mixtures with the same materials at the medium RAP content of 15-25%. Binder and mixture tests were performed. Table A12 shows the properties of the virgin binders as well as the recovered binders from the plant and RAP containing mixes.

Table A12 Critical Temperatures and Binder Grades

Property	Critical Temperatures (°C)								
	Indiana			Michigan			Missouri		
	Virgin	Plant	RAP	Virgin	Plant	RAP	Virgin	Plant	RAP
RAP Content (%)	0	15	100	0	25	100	0	20	100
Optimum Binder Content (%)	5.0	5.0	4.70	5.5	5.5	3.80	4.2	4.2	4.40
RAP Binder Content in Total Mix (%)	0	0.71	4.70	0	0.95	3.80	0	0.88	4.40
Original G*/sinδ	68.9	76.4	77.5	57.8	71.1	84.7	60.8	73.8	86.6
RTFO G*/sinδ	67.6	71.7	73.6	60.1	63.4	79.6	60.1	64.6	80.4
PAV G*sinδ	15.9	20.8	24.9	11.5	15	27.6	17.3	17.5	21.8
Stiffness	-21.1	-16.4	NA	-21.5	-29	NA	-17.6	-17.2	-19.9
m- value	-26.2	-25.3	-24.2	-24.7	-21	NA	-21.9	-17.4	-20.2
Temp. Range	67-31	71-26	73-??	57-31	63-31	79-??	60-27	64-27	80-29
PG grade (MP1)	64-28	70-22	70-??	52-28	58-28	76-??	58-22	64-22	76-28

Briefly, the results showed that mixtures with up to 50% RAP could be designed under Superpave, provided the RAP gradation and aggregate quality were sufficient. In some cases, the RAP aggregates limited the amount of RAP that could be included in a mix design to meet the superpave volumetric and compaction requirements. Linear binder blending charts were found to be appropriate in most cases.

Table A13 summarizes the frequency sweep tests results. The frequency sweep test results showed that the mixture stiffness increased with the increase in RAP content. The plant mixtures from for Missouri and Michigan performed similar to the laboratory produced mixtures with the same RAP content, whereas the plant mixtures from Indiana showed significantly higher stiffness than the corresponding laboratory produced mixtures.

Table A13 Summary of Shear Moduli from Frequency Sweep Tests at 10 Hz.

State	**Mix**	**Shear modulus (psi)**	
		@ 20°C	**@ 40°C**
Indiana	Plant mix 15% RAP	563,312	155,803
	Lab 0% RAP	289,966	66,166
	Lab 15% RAP	304,183	37,367
	Lab 50% RAP	355,048	87,419
Michigan	Plant mix 25% RAP	377,154	31,197
	Lab 0% RAP	281,139	28,832
	Lab 25% RAP	406,986	36,002
	Lab 40% RAP	297,109	48,971
Missouri	Plant mix 20% RAP	458,283	119,852
	Lab 0% RAP	390,118	75,295
	Lab 20% RAP	536,511	107,307
	Lab 50% RAP	613,493	168,104

The simple shear tests produced highly variable results; therefore it was difficult to interpret the data. In general, the simple shear tests showed the same trend as the frequency sweep tests, an increase in stiffness with the increase in RAP content.

The repeated shear constant height tests (Figure 7 through 9) showed that all the evaluated mixtures had a good rutting resistance by having a permanent shear strain less than 2.0% after 5,000 cycles except for the laboratory prepared mix with 40% RAP from Michigan which exhibited a permanent shear strain of 2.5% after 5,000 cycles. This test is more sensitive to aggregate structure, therefore in some cases increase in RAP content only lead to a slight decrease in performance. This supports the reason for considering the RAP aggregate shape, gradation and quality.

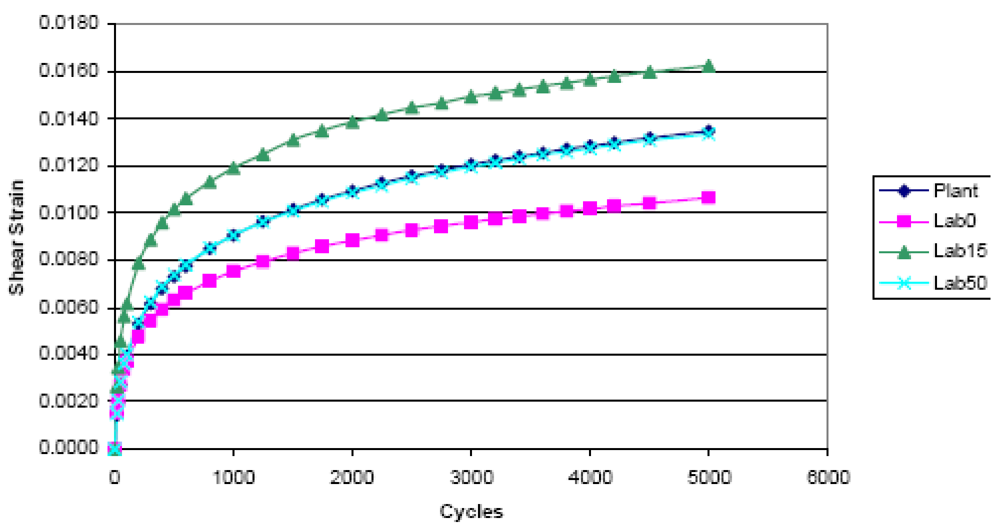


Figure A7 Average Shear Strain from RSCH Test, Indiana Mixes @ 58°C

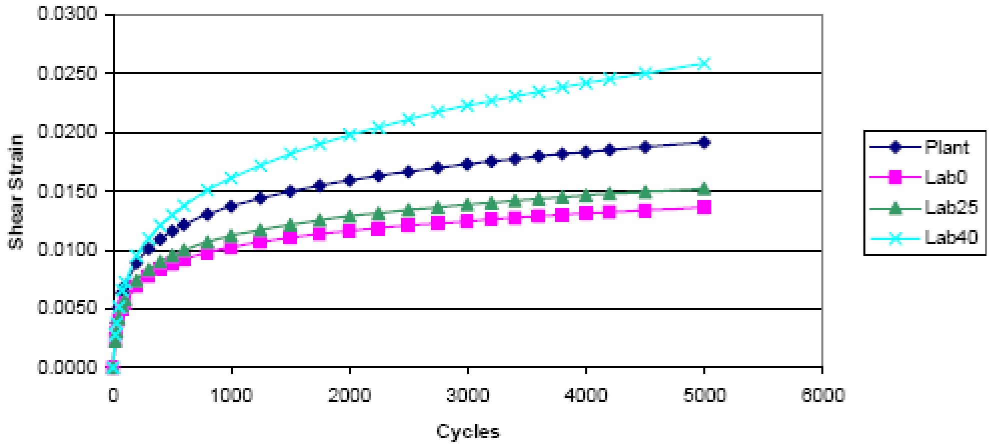


Figure A8 Average Shear Strain from RSCH Test, Michigan Mixes @ 58°C

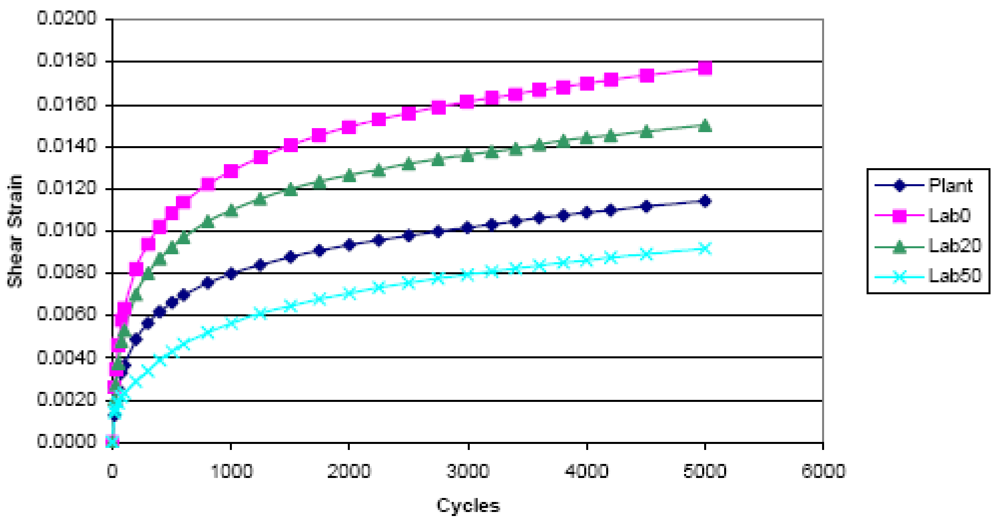


Figure A9 Average Shear Strain from RSCH Test, Missouri Mixes @ 58°C

In general, increasing the RAP content of a mixture increased its stiffness and decreased its permanent shear strain, indicating increased resistance to rutting if the virgin binder grade was unchanged. It was recommended that RAP aggregate gradation and quality should be considered in the mix design, since a poor aggregate structure could reduce mixture stiffness and ultimately performance. Provided the properties of the RAP materials are properly accounted for in the material selection and mix design process, superpave mixtures with RAP can perform very well *(4)*.

In 2006, McDaniel et al. evaluated the influence of RAP content on the mixture and recovered binder properties of plant-produced hot mix asphalt by studying the dynamic moduli of RAP mixtures and binders properties *(5)*. RAP was added at 15%, 25% and 40% levels to HMA with PG64-22 and at 25% and 40% levels to HMA with PG58-28 binder. In addition, control mixture samples with PG64-22 and no RAP were also collected and tested for comparison. Compacted specimens were tested to determine the complex dynamic moduli (|E*|) at three temperatures. The shear moduli (|G*|) of the binders recovered from the HMA samples, RAP samples and original binders were also determined at the same test temperatures. Low temperature creep compliance and indirect tensile strength of the mixtures were also determined, and these results were used to estimate the critical cracking temperature of the pavement. Figures A10 and A11 show the test results for the dynamic modulus and the indirect tensile strength, respectively. Statistical analyses indicated that there were no differences in mean strength and |E*| of the control mixture and mixtures with 15% and 25% RAP. Some differences between the control mixture and the 40% RAP mixtures were found only at the higher test temperatures *(5)*.

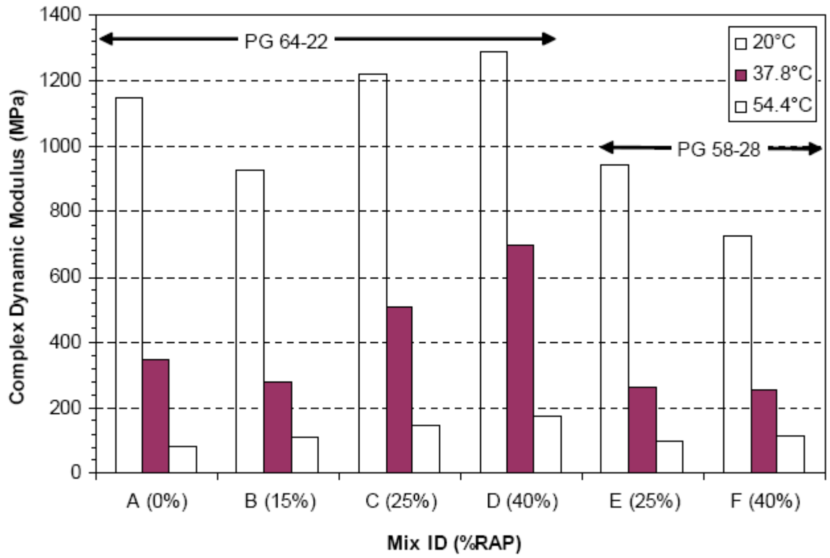


Figure A10 Average dynamic modulus test data

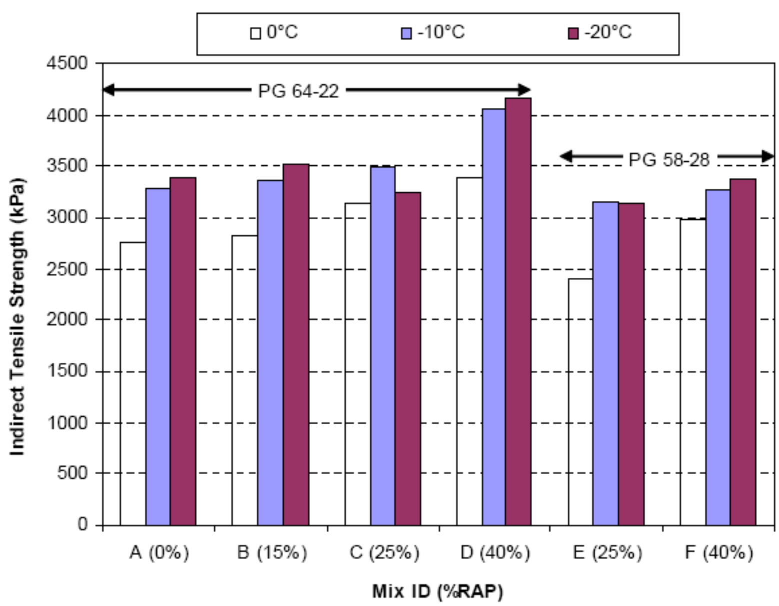


Figure A11 Indirect tensile strength test data

In summary, this study showed that adding small amounts of RAP may not change the mix properties greatly. As the percentage increases, some effect on the mixture properties is noted, but not in proportion to the amount of RAP being added. When the percentage is high enough, the RAP binder would create a significant change in the mixtures properties. In a similar manner, the influence of RAP on the final HMA property also varies with the RAP amount. It was also recommended that the percent of binder in the RAP material should be considered in addition to the percent of RAP being used *(5)*.

Saskatchewan Highways and Transportation, Canada

In 1996, Puttaguanta et al. *(6)* compared the predicted fatigue performance and moisture damage of virgin HMA mixtures and RAP containing HMA mixtures. Core samples for the HMA mix with 50% RAP obtained one month after construction from a pavement section of Highway 11 between Regina and Saskatoon, Canada, were tested for density, air voids, aggregate gradation, asphalt content, and penetration of the recovered binder. The RAP and virgin aggregates used in the recycling project were also obtained from the site and used to prepare mixtures with RAP contents of 0, 25, and 50%. The RAP containing mixtures were prepared and compacted to have the same physical properties as the core samples. The original intention was to use three RAP contents of 25%, 50%, and 75%. However, the last content could not be achieved because of the lack of a suitable virgin binder grade to reduce the penetration of the blended binder to the field target value. A rejuvenator could not be used because it would change the chemical composition of the asphalt binder in the mix. For the purpose of comparisons a 100% virgin mix (i.e., 0% RAP) was also prepared. The predicted numbers of load repetitions to fatigue failure of the various mixtures at three temperatures are shown in

Table A14. The data show that the virgin mix can sustain a higher number of load repetitions than the RAP containing mixes at 5°C, whereas at higher temperatures all the mixes had an equal number of load repetitions to failure. Additionally, there is negligible difference between the 25% and 50% recycled mixes. Table A15 shows the test results for the moisture damage evaluation. The data show that the virgin mix had tensile strength and resilient modulus ratios less than 80% while the recycled mixes had ratios greater than 80.

Table A14 Resilient Modulus and Allowable Number to Fatigue Failure.

Property	Temperature								
	5°C			22°C			40°C		
	Virgin mix	RAP mix		Virgin mix	RAP mix		Virgin mix	RAP mix	
		25%	50%		25%	50%		25%	50%
Resilient modulus, E_R (Mpa)	6,928	5,261	5,380	2,020	1,594	1,621	1,259	1,021	1,004
Allowable number of loads*	8.3×10^{8}	4.8×10^{7}	7.6×10^{7}	7.6×10^{6}	2.4×10^{6}	2.6×10^{6}	2.0×10^{5}	1.4×10^{5}	1.5×10^{5}

* $N_f = k_1(1/\varepsilon)^{k2}$
Where: $k_1 = 7.87\times10^{-7}\,(E_R/3447.5)^{-4}$ and $k_2 = 1.75 - 0.252\log k_1$

Table A15 Results of Moisture Damage Test

Property	Virgin mix	RAP mix		Allowable limit
		25%	50%	
Tensile strength ratio, %	59	81	91	> 80
Resilient modulus ratio, %	68	85	90	> 80

Western Regional Superpave Center (WRSC)

In 2007, Hajj et al. *(7)* conducted a research study for the Regional Transportation Commission (RTC) of northern Nevada on the use of RAP in HMA mixtures. The laboratory performance of the control and RAP mixtures were evaluated in terms of their resistance to:

- Moisture damage: AASHTO T283
- Rutting: asphalt pavement analyzer (APA)
- Fatigue: flexural beam fatigue test
- Thermal cracking: thermal stress restrained specimen test (TSRST)

This study covered three sources of RAP at the contents of 0, 15 and 30%, one source of virgin aggregates, and one source of virgin asphalt binders to design HMA mixtures with two target asphalt binder grades. The testing matrix consisted of six RAP mixtures and one control mix (0% RAP) for each of the target binders of PG64-22 and PG64-28.

Currently, RTC uses the Marshall mix design method to design HMA mixtures and specifies two binder grades for all HMA mixtures: PG64-22 and PG64-28. The PG64-22 is a neat asphalt binder mostly used in the bottom and middle lifts of the HMA layer. The PG64-28 is a polymer-modified binder mostly used in the top lift of the HMA layer. The three RAPs used in this study were selected from three different local sources.

- Source I: plant waste from Granite Construction Company's Lockwood quarry, Reno, Nevada (4.6% binder content by weight of RAP mix).
- Source II: reclaimed asphalt from a 15-year old HMA pavement in Reno, Nevada (5.4% binder content by weight of RAP mix).

- Source III: reclaimed asphalt from a 20-year old HMA pavement in Reno, Nevada (5.8% binder content by weight of RAP mix).

Based on the data generated from this experiment, the following conclusions were made. While reviewing the findings and conclusions, it should be well recognized that in most cases the addition of RAP materials necessitated a change in the virgin binder grade from the target binder grade as shown in Table A16. This change in the virgin binder grade had impact on the measured performance properties of the final mix. Table A17 shows the optimum binder content of the various mixes evaluated in this study along with the percent of RAP binder in the RAP containing mix.

Table A16 Required Virgin Binders Grades for the Various RAP Sources and Contents.

RAP	**Recovered RAP Binder Grade**	**Required Virgin Binder Grade (Based on Blending Chart)**			
		Target Binder: PG64-22		**Target Binder: PG64-28NV**	
		15% RAP	**30% RAP**	**15% RAP**	**30% RAP**
Source I	PG82-16	PG64-22	PG58-28	PG64-34	PG58-34
Source II	PG82-16	PG64-28	PG58-28	PG64-34	PG58-34
Source III	PG82-16	PG64-28	PG58-28	PG64-34	PG58-34

Table A17 Optimum and RAP binder contents of various Mixtures

Target Binder	Mix Proportions	Optimum Binder Content (% by total weight of mix)	% RAP Binder in the Total Mix
PG 64-22	0% RAP	4.5	0
	15% RAP (Source I)	4.4	0.66
	30% RAP (Source I)	4.5	1.34
	15% RAP (Source II)	4.4	0.78
	30% RAP (Source II)	4.5	1.57
	15% RAP (Source III)	4.2	0.84
	30% RAP (Source III)	4.4	1.69
PG 64-28 NV	0% RAP	4.7	0
	15% RAP (Source I)	4.3	0.66
	30% RAP (Source I)	4.5	1.34
	15% RAP (Source II)	4.2	0.78
	30% RAP (Source II)	4.3	1.57
	15% RAP (Source III)	4.2	0.84
	30% RAP (Source III)	4.4	1.69

- The Marshall Mix Design method as outlined in the Asphalt Institute's Mix Design Methods Manual MS-2 and implemented by the Washoe RTC can be used to design HMA mixtures with 15 and 30% RAP.

- Blending chart process was used to identify the required grades of the virgin binders to achieve the target binder grades. The blending chart method was found to be conservative and not highly reliable in identifying the appropriate grade of the virgin binder for the various RAP sources and RAP contents.

- Impact of RAP on moisture damage resistance (Table A18):

  PG64-22 mixtures:
    - The addition of 15 or 30% RAP to a mixture resulted in an acceptable resistance to moisture damage regardless of the source of the RAP.
    - The addition of 15 or 30% RAP to a mixture resulted in a reduction in the unconditioned and conditioned tensile strengths.
    - The 15% RAP mixtures had higher resistance to moisture damage than the 30% RAP mixtures.

  PG64-28 mixtures (polymer modified asphalt binder):
    - The addition of 15 or 30% RAP to a mixture resulted in an acceptable resistance to moisture damage regardless of the source of the RAP.
    - The addition of 15 and 30% RAP to a mixture resulted in a reduction in the unconditioned and conditioned tensile strengths.

- The 15% RAP mixtures had lower resistance to moisture damage than the 30% RAP mixtures.

Table A18 Moisture Sensitivity Properties of the Various Mixtures (AASHTO T283).

Target Binder	Mix Proportions	Tensile Strength, TS @ 77°F, psi				Tensile Strength Ratio (%)
		Un conditioned		Conditioned		
		Average	CV (%)	Average	CV (%)	
PG 64-22	0% RAP	194	7	168	5	86
	15% RAP (Source I)	224	8	174	8	78
	30% RAP (Source I)	179	9	135	7	76
	15% RAP (Source II)	157	9	139	3	89
	30% RAP (Source II)	107	5	84	10	78
	15% RAP (Source III)	180	5	160	10	89
	30% RAP (Source III)	184	5	129	4	70
PG 64-28 NV	0% RAP	167	8	137	9	82
	15% RAP (Source I)	75	5	50	8	66
	30% RAP (Source I)	91	7	69	3	76
	15% RAP (Source II)	79	8	63	9	80
	30% RAP (Source II)	180	9	146	10	81
	15% RAP (Source III)	86	10	71	6	83
	30% RAP (Source III)	131	8	94	8	72

- Impact of RAP on rutting resistance (Table A19):
  PG64-22 mixtures:
  - The addition of 15% RAP to a HMA mixture resulted in better resistance to rutting than the virgin mix when RAP from a 15 to 20-year old HMA pavement is used.
  - The addition of 30% RAP to a mixture resulted in a better resistance to rutting than the virgin mix only in the case of RAP from a 20-year old HMA pavement (source III).
  - The addition of 15 or 30% RAP from the plant waste to a mixture resulted in a lower resistance to rutting than the virgin mix.

  PG64-28 mixtures (polymer modified asphalt binder):
  - The addition of 15% and 30% RAP to a HMA mixture resulted in rutting resistance equivalent to the virgin mix and significantly lower than the APA failure criteria regardless of the source of the RAP.

- Impact of RAP on fatigue resistance (Table A20):
  PG64-22 mixtures:

- The addition of 15% RAP to a mixture resulted in either a better or equivalent resistance to fatigue than the virgin mix regardless of the source of the RAP.
- The addition of 30% RAP to a mixture resulted in a better resistance to fatigue than the virgin mix only in the case of RAP from a 20-year old HMA pavement (source III).

PG64-28 mixtures (polymer modified asphalt binder):

- The addition of 15 or 30% RAP to a mixture resulted in a significant reduction in fatigue resistance regardless of the source of the RAP.

• Impact of RAP on thermal cracking resistance (Table A20):

PG64-22 mixtures:

- The addition of 15 or 30% RAP to a mixture resulted in either a better or equivalent resistance to thermal cracking regardless of the source of the RAP.

PG64-28 mixtures (polymer modified asphalt binder):

- The addition of 15 or 30% RAP to a mixture resulted in a significantly better resistance to thermal cracking regardless of the source of the RAP.

Table A19 Rutting Resistance of the Various Mixtures.

Target Binder	Mix Proportions	APA Rut depth after 8000 cycles	
		mm	Inch
PG 64-22	0% RAP	4.6	0.18
	15% RAP (Source I)	5.9	0.23
	30% RAP (Source I)	6	0.24
	15% RAP (Source II)	2.2	0.09
	30% RAP (Source II)	7.3	0.29
	15% RAP (Source III)	1.4	0.06
	30% RAP (Source III)	2.1	0.08
PG 64-28 NV	0% RAP	2.1	0.08
	15% RAP (Source I)	2.1	0.08
	30% RAP (Source I)	3.1	0.12
	15% RAP (Source II)	2.1	0.08
	30% RAP (Source II)	2.4	0.09
	15% RAP (Source III)	2.1	0.08
	30% RAP (Source III)	2.1	0.08

Table A20 Laboratory Fatigue Life and Fracture Temperature of Various Mixtures.

Target Binder	Mix Proportions	No of cycles to failure at 72°F			TSRST Fracture temp (°C)
		Strain level			
		300 microns	500 microns	700 microns	
PG 64-22	0% RAP	191,520	20,700	4,780	-18
	15% RAP (Source I)	510,980	12,550	1,090	-17
	30% RAP (Source I)	311,000	43,330	11,830	-26
	15% RAP (Source II)	530,000	59,690	14,160	-29
	30% RAP (Source II)	126,770	23,520	7,760	-22
	15% RAP (Source III)	546,670	49,520	10,180	-16
	30% RAP (Source III)	2,635,400	39,200	2,450	-23
PG 64-28 NV	0% RAP	Will not fail	1,683,400	119,850	-24
	15% RAP (Source I)	3,827,100	925,600	363,400	-39
	30% RAP (Source I)	9,545,000	669,000	116,180	-35
	15% RAP (Source II)	Will not fail	3,715,000	918,750	-40
	30% RAP (Source II)	13,194,000	419,850	43,340	-40
	15% RAP (Source III)	3,621,100	773,400	279,800	-39
	30% RAP (Source III)	1,611,200	165,925	37,120	-28

Table A21 compares the properties of the RAP containing mixtures to the properties of the control (i.e., 0% RAP) mix.

Table A21 Overall Summary of the Laboratory Evaluation of RAP Containing Mixtures.

Target Binder Grade	RAP Source#	RAP %	Impact of RAP on Resistance to+							
			Moisture		Rutting		Fatigue		Thermal Cracking	
PG64-22	I	15	Pass	--	--	Worse	Better	--	Same	--
		30	Pass	--	--	Worse	--	Worse	Better	--
	II	15	Pass	--	Better	--	Same	--	Better	--
		30	Pass	--	--	NA	--	Worse	Better	--
	III	15	Pass	--	Better	--	Better	--	Same	--
		30	Pass	--	Better	--	Better	--	Better	--
PG64-28*	I	15	--	Fail	Same	--	--	Worse	Better	--
		30	Pass	--	Same	--	--	Worse	Better	--
	II	15	Pass	--	Same	--	--	Worse	Better	--
		30	Pass	--	Same	--	--	Worse	Better	--
	III	15	Pass	--	Same	--	--	Worse	Better	--
		30	Pass	--	Same	--	--	Worse	Better	--

** Polymer modified asphalt binder.*
+ Statistically compared to control mixture (0% RAP).

In addition to the above laboratory evaluation, two field HMA mixtures containing 15% RAP from a pavement in Sparks, Nevada were sampled during construction and evaluated in the laboratory in terms of their resistance to moisture damage, rutting, fatigue, and thermal cracking *(7)*. The constructed HMA layer consisted of 3 lifts of 2.5 inch each. The bottom lift consisted of a dense graded HMA with 15% RAP material manufactured with a PG64-22 neat asphalt binder (F-22-15). The middle and the top lifts consisted of a dense graded HMA with 15% RAP material manufactured with a PG64-28 polymer modified asphalt binder (F-28-15). Table A22 shows the moisture sensitivity test results (AASHTO T283) along with the APA rut depth at 140°F. Figure A12 shows the fatigue relationships of both mixtures at 72°F. Based on the data generated from this experiment, the following conclusions were made:

- The F-22-15 mix failed to meet the minimum tensile strength ratio (TSR) of 70% required by RTC indicating a poor resistance to moisture damage. The F-28-15 mix barely passed the minimum required TSR indicating a marginal resistance to moisture damage.

- In the case of resistance to rutting, both field mixtures met the Nevada DOT APA criterion of 8 mm under 8,000 cycles at 140°F. The use of polymer modified binder reduced the APA rut depth by about 42% compared to the neat asphalt binder.

- The use of RAP in a polymer modified mixture (F-28-15) increased the mixture’s laboratory resistance to fatigue cracking when compared to the mix with neat asphalt binder (i.e., F-22-15).

- In the case of resistance to thermal cracking, the field mixtures exhibited a fracture temperature within 1°C of the low performance temperature of the corresponding target binder grades (i.e. -22°C and -28°C).

- In a summary, the evaluated pavement section is expected to have acceptable performance in rutting, fatigue, and thermal cracking, but might show signs of failure due to moisture sensitivity problems. Moisture sensitivity problems lead into a reduction in the HMA strength and hence a reduction in the overall pavement performance.

It should be noted that all mixtures were treated with 1.5% hydrated lime by dry weight of aggregate without any marination and fulfilled the RTC requirement for TSR at the mix design stage. Therefore, attention should be given to the durability property of the filed produced mixtures. Previous studies on field mixtures sampled from behind the paver showed higher percentage of TSR failures for mixes treated with lime without marination when compared to mixes treated with lime followed by 48 hours marination.

Table A22 Moisture Sensitivity Properties and APA Rut Depths of field Mixtures.

Mix	**Tensile strength at 77°F (psi)**				**Tensile strength ratio (%)**	**APA Rut Depth at 140°F**	
	Unconditioned		**Conditioned**				
	Average	**CV (%)**	**Average**	**CV (%)**		**mm**	**inch**
F-22-15	159	7	82	7	52	5.2	0.2
F-28-15	146	6	103	4	71	3	0.12

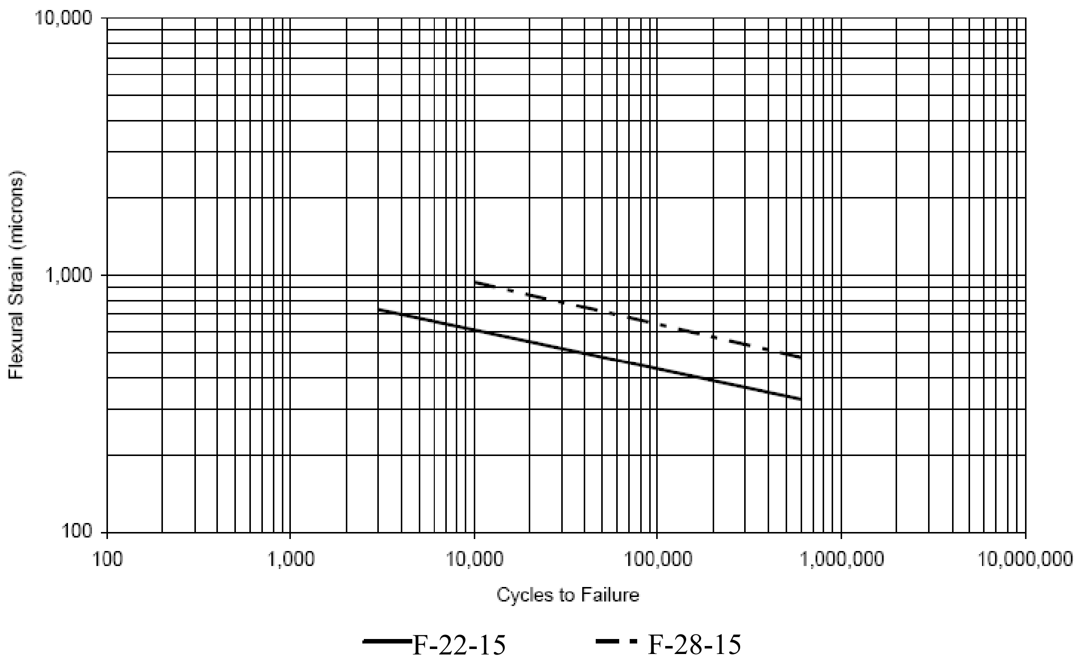


Figure A12 Fatigue Relationships of the Stanford Way Mixes at 72°F

Other Research Studies

In 2005, Daniel and Lachance evaluated the effect of RAP and its content on the volumetric and mechanistic properties of HMA mixes *(8)*. The 0% RAP mix was used as the control for evaluating the properties of the mixes containing 15%, 25%, and 40% RAP. Two types of RAP were evaluated: a processed RAP and an unprocessed RAP (grindings). The processed RAP had a 3.6% of PG94-14 asphalt binder and consisted of a mix of recycled asphalt pavement, Portland cement concrete and sometime slight amount of organic material. The unprocessed RAP (grindings) consisted of recycled asphalt pavement that was milled from a pavement surface and had a 4.9% of PG82-22 asphalt binder. Both of these RAP sources were used to study the volumetric effects on

HMA mixes and only the processed RAP was used to study the mechanistic effects on the HMA mixes.

A 19 mm Superpave mix design with an unmodified PG58-28 asphalt binder was conducted for the 15% RAP content according to the New Hampshire Department of Transportation (NHDOT) specifications. For the remaining mixtures (control, 25% RAP, 40% RAP), the stockpile percentages were adjusted to achieve an overall mixture gradation similar to the 15% RAP gradation. The relative proportions of blast rock and sand stockpiles were held constant for the different mixtures to maintain the same relative structure (particle angularity, type of material) for the virgin material in the mixture. For the HMA mixtures containing processed RAP, the increasing percentages of RAP caused the gradations to become finer at the smaller sieve sizes, with the 40% RAP mixture going into the restricted zone. All of the other gradations were on the coarse side of the restricted zone. The three mixtures containing the grindings produced slightly finer gradations at the #30 sieve size, but were essentially the same as the control mix. The asphalt contents and volumetric properties of the mixtures are shown in Table A23.

Table A23 Mix Design Parameters.

Property	**Control Mix**	**Processed RAP mixes**			**Unprocessed RAP mixes**		
		15% RAP	**25% RAP**	**40% RAP**	**15% RAP**	**25% RAP**	**40% RAP**
% Optimum binder	4.8	5.1	5.4	4.9	4.9	5.2	5.2
% RAP binder	0.00	0.54	0.90	1.44	0.74	1.22	1.96
G_{mm}	2.451	2.483	2.445	2.466	2.452	2.460	2.475
VMA	13.1	13.3	16.3	15.2	13.8	14.3	14.7
VFA	69.4	69.9	75.4	73.6	71.8	71.0	73.0
DP	1.14	1.10	0.88	1.02	0.91	0.75	0.75

The data in Table A23 shows that the VMA and VFA of RAP mixtures are higher than that of control mixture and the VMA of unprocessed RAP increases with the increase in RAP content. As part of this study the heating time effect on the volumetrics of the 40% processed RAP mixture was evaluated by heating the RAP before mixing for 2 hours, 3.5 hours, and 8 hours. The two hour time is the standard procedure used by the New Hampshire DOT. The 3.5 hour time is the time required for the RAP to reach mixing temperature, and 8 hours is equivalent to the time the aggregate is heated (usually overnight) in the oven. The same compaction effort was used in fabricating all of the specimens and the result of this testing are shown in Table A24.

The data in Table A24 show a decrease in the VMA by 0.5% when the heating time increases from 2 to 3.5 hours, and then an increase by almost 3% with the longer heating time. The researchers claimed that: a) at the shorter heating time, the RAP is not heated enough to allow the RAP particles to break up into smaller pieces and blend with the virgin materials, and b) at the longer heating time, the RAP was likely aged further and the RAP particles have hardened and even fewer of them were able to break down and blend with the virgin material. This indicated that there is an optimum heating time for the RAP material to allow for the greatest extent of blending between the virgin and RAP materials.

Additionally, Table A24 shows the VMA values for the same design air void content of 4%. The data shows that longer heating times decrease the VMA values, and may affect the mixture design and design asphalt content. Therefore, a RAP mixture may not meet the Superpave VMA requirements when the RAP is heated for a particular amount of time, but may meet the requirements if the RAP is heated for a different amount of time. Therefore, it is very important that the laboratory procedures for producing RAP mixture simulate the plant operations as close as possible *(8)*.

Table A24 Effect of RAP Pre-heating Time on 40% Processed RAP Mixture Volumetrics.

Compaction method	**Property**	**Preheating duration**		
		2 Hrs	**3. 5 hrs**	**8 hrs**
	G_{mm}	2.484	2.480	2.479
Same compaction effort	Air void (%)	4.0	4.4	7.6
	VMA (%)	15.1	14.6	17.5
	VFA (%)	73.6	70.1	56.3
Same air void content	Air void (%)	4.0	4.0	4.0
	VMA (%)	15.1	14.2	14.4
	VFA (%)	73.6	71.2	72.2

The samples produced according to above mix designs were tested for dynamic modulus by means of both tension and compression and for creep compliance by means of compression.

When samples were tested for dynamic modulus under compression the variability of the results increased with increasing RAP content, but when the samples were tested in tension the variability of all RAP mixtures were lower than that of the control mixture. Table A25 shows the mean square error of dynamic modulus master curve regressions for various mixtures in compression and tension. The dynamic modulus master curves in compression and in tension are shown in Figures A13 and A14 respectively. The data show that the 15% RAP mix has a higher stiffness than control mixture in both tension and compression. The 25% and 40% RAP mixtures showed similar stiffness as the control mixture in both tension and compression, though these were expected to have higher stiffness than the 15% RAP mixture. The stiffness reduction of the 25% and 40% RAP mixtures was attributed to the finer gradations and higher VMA and VFA values.

Table A25 Mean Square Errors from Dynamic Modulus Master Curve Regressions.

Test method	Mixture	Mean Square Error (kPa)
Compression	Control	12,690
	15% Processed RAP	13,817
	25% Processed RAP	24,876
	40% Processed RAP	28,935
Tension	Control	21,513
	15% Processed RAP	16,144
	25% Processed RAP	14,470
	40% Processed RAP	14,683

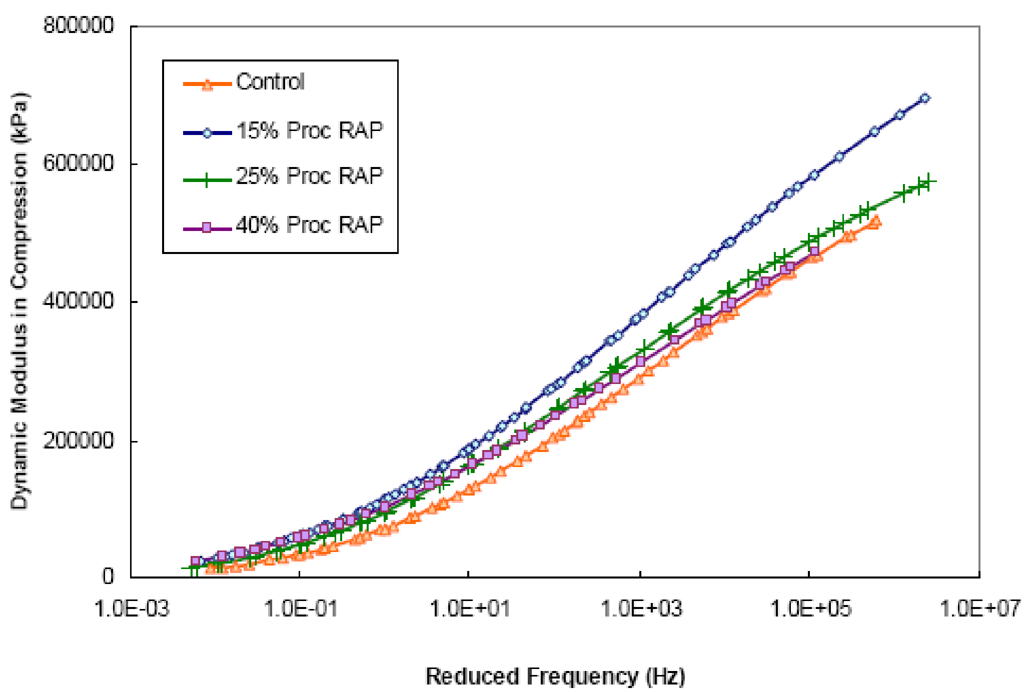


Figure A13 Dynamic Modulus Master Curves in Compression for All Mixtures

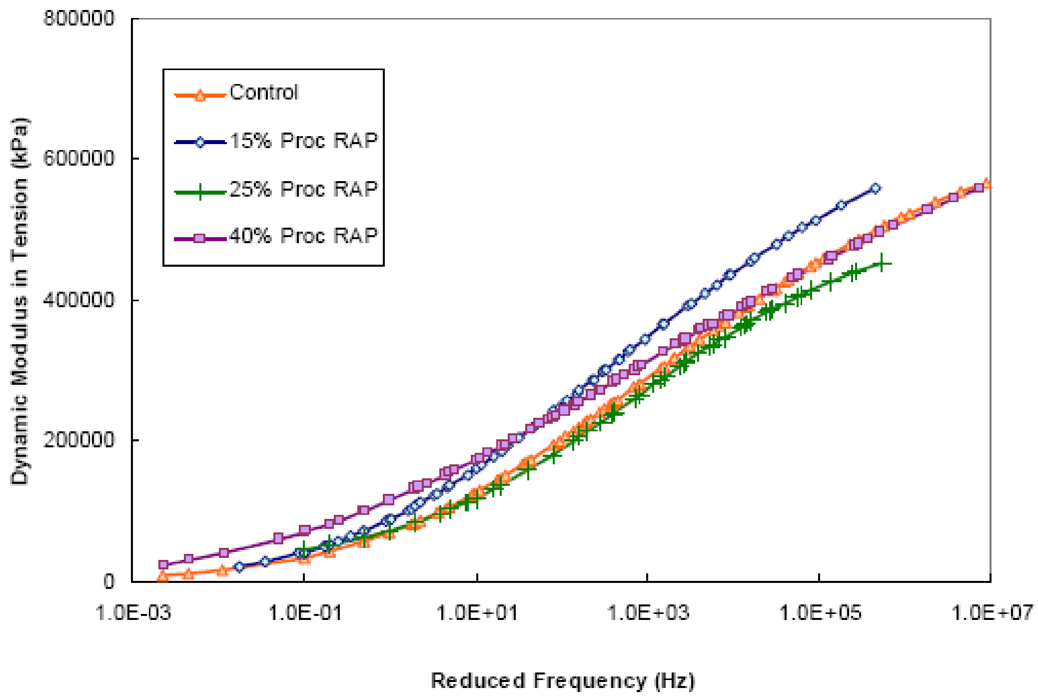


Figure A14 Dynamic Modulus Master Curves in Tension for All Mixtures.

The creep compliance test results in Figure A15 shows that the 15% RAP mixture has higher stiffness and lower compliance when compared to the control mixture. But the 25% and 40% RAP mixtures did not follow the same trend set by the 15% RAP mix. This behavior was again attributed by the researchers to the finer gradations and higher VMA and VFA values. Table A26 shows the creep flow time test results at 87 psi stress and 45°C temperature. The creep flow time increased with increasing RAP content with the exception of 25% RAP mixture. The reduction in creep flow time was attributed to the higher binder content of the particular mix.

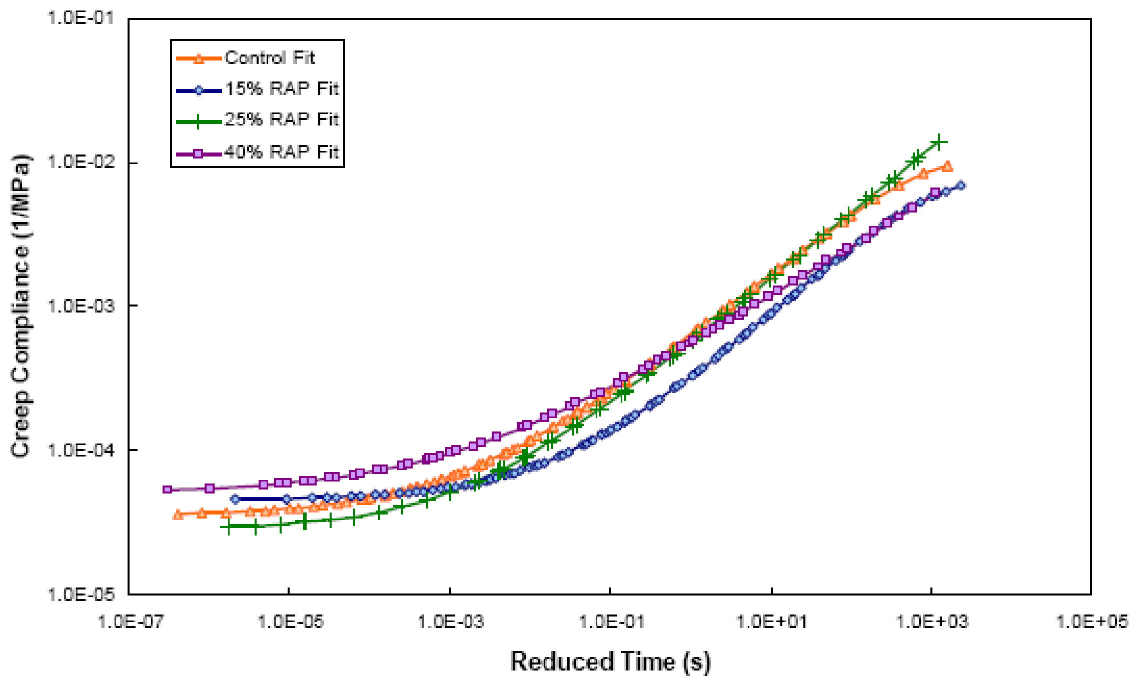


Figure A15 Compression Creep Compliance Master Curves for all Processed RAP Mixtures

Table A26 Creep Flow Times for Processed RAP Mixtures.

Mixture	Creep Flow Time (seconds)
Control	553
15% RAP	1,445
25% RAP	350
40% RAP	3,050

In 2007, Xiao et al. investigated the use of both RAP and crumb rubber in HMA mixes *(9)*. The goal of this study was to gain an improved understanding of the rutting resistance characteristics of the rubberized asphalt mixtures containing RAP. The experimental design was divided into two parts. For the first phase of the research work, two rubber types (Ambient and Cryogenic), four rubber contents (0, 5, 10, and 15% by weight of virgin binder), and three crumb rubber sizes (−14 mesh, −30 mesh, and −40 mesh) were used to make various mixtures. To avoid the influence of blending, one aggregate source (designated as L) and one binder source and grade (PG64-22) were used for preparing the samples. A total of 13 mix designs were conducted in this phase. The second part of the work included the validation of the findings from the first phase by using another aggregate source (designated as C) and another binder grade (PG52-28). A total of three mix designs were conducted for the second phase of the research. The RAPs were taken from the same geographical area as the new aggregates to ensure that the aggregate in the RAP had similar properties as the virgin aggregates. Both RAP sources (L and C) were approved by the South Carolina DOT and mixed with an original binder equivalent to a PG64-22. Four RAP percentages (0, 15, 25, and 30%) were used in the mixtures made with aggregate L and three RAP percentages (0, 15 and 38%) with aggregate C.

Experiments were carried out to evaluate the indirect tensile strength (ITS) and rutting susceptibility of the various mixtures using the asphalt pavement analyzer (APA). Figures A16-A18 show the test results. Tests were also performed to determine the rutting properties of various mixtures with respect to rubber production type, content, and size in the mixture. Based on the test results the following conclusions were made *(9)*.

- Increasing the percentages of RAP in the mixtures containing crumb rubber resulted in higher stiffness and ITS values, indicating higher stability. This increase was also very effective in improving rutting resistance over the conventional mixtures.
- Increasing the rubber content resulted in a decrease in the ITS value and creep stiffness. However, adding crumb rubber into the HMA effectively increased the rutting resistance. Increasing the percentage of rubber considerably improved the

ability of the mixtures to resist deformation as measured by the APA test. In general, the mixtures containing rubberized binder produced samples that exhibited lower rut depths than the mixes using the virgin binder.

- The results of the ITS tests suggested that the ambient rubber has produced results similar to those of the cryogenic rubber when the same rubber content is used. However, the rut depth of the two types of rubber mixtures suggested that the ambient rubber has higher rutting resistance when mixed with 25% RAP.
- The results of the ITS and rutting tests of mixtures made with 10% ambient rubber and 25% RAP showed that the effect of rubber size is rather small; the ITS values and the rut depths of these mixtures using various rubber sizes were similar.
- The results of the study show that as air voids in the modified mixtures decrease, the rut depth from the APA test decreases, exhibiting a similar trend as in the conventional asphalt mixtures.

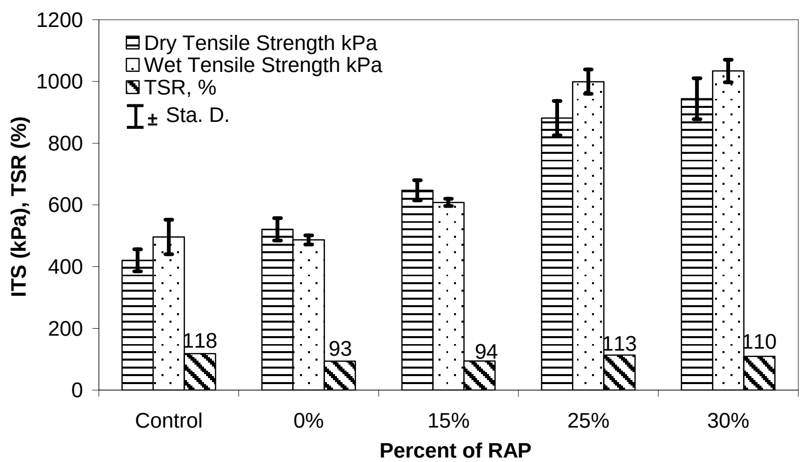


Figure A16 ITS and TSR values for specimens made with 10% -40 mesh cryogenic rubber using aggregate source L

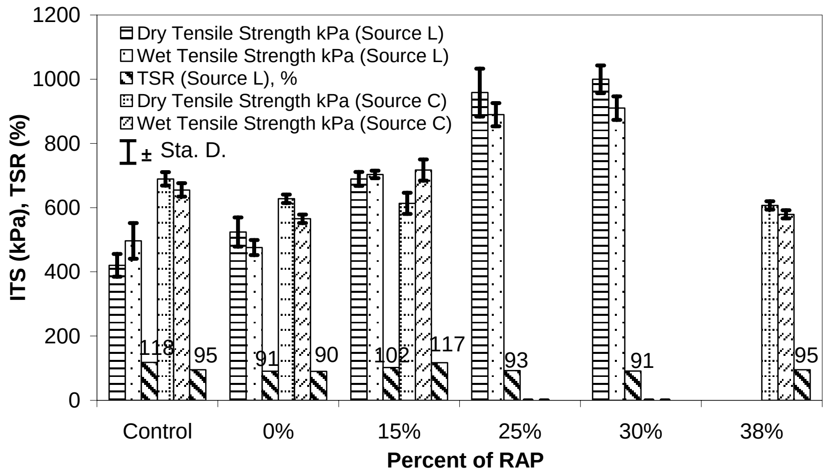


Figure A17 ITS and TSR values for specimens made with 10% -40 mesh ambient rubber

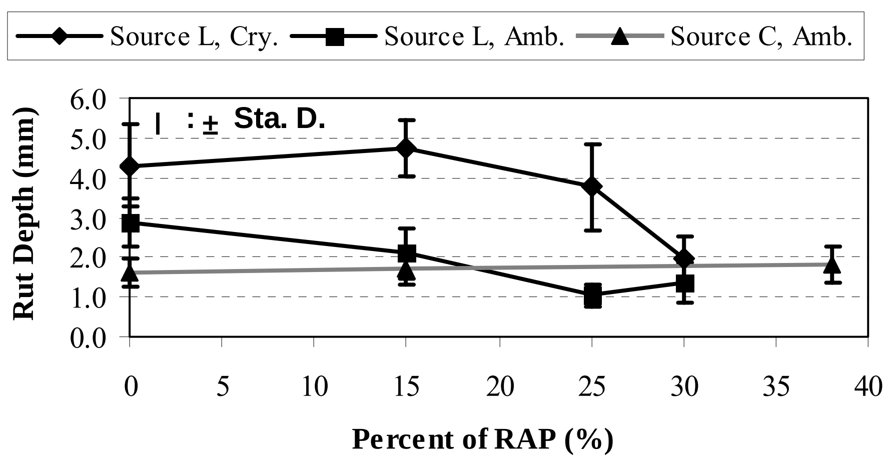


Figure A18 Rut depth of mixtures made with 10% -40 mesh rubber

PERFORMANCE OF RAP CONTAINING MIXES IN HIGHWAY PAVEMENTS

Performance of Pavements Containing RAP in California

The California Department of Transportation (Caltrans) initiated a study to evaluate the performance of in-service recycled asphalt pavements in California. As part of this study, sixty 15% RAP test sections located in three of California's environmental zones – Desert (DS), Mountain (MT) and North Coast (NC) – along four routes (one in each of Caltrans' Districts 1, 7, 9 and 11) were considered. Five of these sections have a Cement Treated Base (CTB), while the rest of the sections have an aggregate base course. Deflection, roughness, distress and cores/bores were among the data attributes collected from the test sections. Laboratory tests were performed on the cores recovered from the field. Also, analysis was performed on the data collected from these sections to evaluate the actual field performance of RAP in different environmental zones.

In 2007, Zaghloul et al. *(10)* reported the observed field performance of the RAP containing sections, as well as the results of the evaluation analysis, and compared the performance of the RAP containing sections by environmental zone.

The sections covered a wide range of layer thicknesses. The total Asphalt Concrete (AC) thickness ranged from 5.76" to 10.8" while the total aggregate base thickness ranged from 4" to 15.6". The total pavement thickness above the subgrade ranged from 13" to 24". All sixty sections had been in service for 5 to 9 years.

The evaluation of the field pavement performance included both structural and functional performance of the sections, as well as the construction consistency using the following four performance indices.

- Structural adequacy index (SAI).
- Distress index (DI).
- Roughness index (RI).
- Construction consistency index (CCI).

The in-situ structural capacity and pavement properties of the test sections were evaluated through the Backcalculation analysis of the FWD measured deflections. A structural adequacy index (SAI) was developed to account for the difference in the pavement structures that resulted from the differences in expected traffic. The SAI was developed by normalizing the effective Gravel Equivalent (GE_{eff}), which evaluates the in-situ structural capacity of the pavement section in its current condition, with respect to the gravel equivalent calculated based on the as-built pavement structure ($GE_{as\text{-}built}$). The GE_{eff} is backcalculated from FWD testing. The $GE_{as\text{-}built}$ is calculated from core/bore results by summing the product of each layer thickness and its corresponding equivalent gravel factors (G_f). SAI allows pavement sections subject to different traffic loads to be compared, as each section is evaluated relative to its design conditions.

Therefore, the SAI was used to evaluate the deterioration of the layers' material properties by normalizing them with respect to the design conditions in terms of the design gravel equivalency factors. The SAI uses a scale of 0.0 to 1.0, with a value of 1.0 representing the expected SAI value for a new pavement section and 0.5 being the value used as a trigger for rehabilitation.

The overall distress condition of the pavement test sections were compared using the distress index (DI) which is a re-scaled version of the PCI used in the Micro-Paver. The DI uses a 0.0 to 1.0 scale, with 1.0 being a perfect section and 0.5 being the value used as a trigger for rehabilitation.

The IRI data collected from all sections considered in this study were re-scaled using an exponential formula to fit a 0.0 to 1.0 scale, with 1.0 being a perfect section and 0.5 being the value used as a trigger for rehabilitation.

A Construction Consistency Index (CCI) that evaluates the variability in the in-situ structural capacity was developed and used in this study. The CCI is a function of the FWD deflections at sensors 1 and 7, the pavement remaining structural life as determined from the FWD results, and the backcalculated subgrade resilient modulus. The CCI is not impacted by the absolute values of these parameters, but rather by the consistency of the values along a section. For example, a section with relatively high deflection that is consistent along its length will have better CCI than a section with relatively low deflection that shows large variability along its length. CCI uses a scale of 0.0 to 1.0, where 1.0 indicates the highest possible construction quality.

Based on the analysis of the performance data from all sixty sections, the following conclusions were made by the researchers *(10)*.

- The expected service lives for the RAP containing sections in the north coast environmental zone based on structural, distress and roughness performances are 18, 21 and 17 years, respectively. Therefore, the RAP containing sections will be triggered for ride quality first, after 17 years.
- The expected service lives for the RAP containing sections in the desert environmental zone based on structural, distress and roughness performances are 15, 9 and 15 years, respectively. Therefore, the RAP containing sections will be triggered for distress first, after 9 years. However, the distress service life can be significantly increased if the appropriate maintenance activities, such as crack sealing, are applied in a timely fashion. In such a case, it may be expected that the RAP containing sections in the desert environmental zone will last for 15 years, i.e. the lowest of the other two service life values.
- The expected service lives for the RAP containing sections in the mountain environmental zone based on structural, distress and roughness performances are

11, 13 and 15 years, respectively. Therefore, the RAP containing sections will be triggered for structural adequacy first, after 11 years.

- The long-term performance of the RAP containing sections in the north coast is expected to be better than that of the RAP containing sections in the desert and mountain in terms of SAI, DI, and RI. This may be attributed to the use of CTB, since CTB typically has a higher modulus than an aggregate base course. However, it should be noted that only 5 RAP containing sections were considered from the north coast, compared with 20 and 35 sections from the desert and mountain, respectively.
- The construction consistency in the north coast is significantly lower than in DS and MT.

In 2008, Zaghloul and Holland *(11)* compared the performance of 47 RAP sections located in the three California environmental zones (i.e., Desert (DS), Mountain (MT) and North Coast (NC)) to the performance of other treatments, located within a reasonable distance on the same route, such as AC overlay, Mill & AC overlay and Rubber Asphalt Concrete overlay (RAC). In total, 131 sections covering 7 different treatments were considered in the analysis. The performance comparisons were made with respect to in-situ structural capacity, distress condition, roughness condition, and consistency of construction. Deterioration models were developed and used to estimate the in-situ structural capacity, distress condition, and roughness condition for all sections at the same age (5 years) to allow fair comparisons. Also, the expected structural, distress, and roughness service lives were estimated for all treatments based on the field-observed conditions. The results of the analyses suggested that in all three environmental zones, long-term performance of RAP containing mixtures is likely to be comparable to other treatments located within a reasonable distance on the same route.

Performance of Pavements Containing RAP in Louisiana

The main objective of this study was to compare the relative performance of mixes containing RAP with conventional mixes on different projects in Louisiana *(12)*. Five projects that used between 20% and 50% RAP and four conventional HMA mixtures that were constructed during the same time frame were chosen for the study. The conventional and RAP containing projects were selected using the following criteria: same contractor, similar mix designs, similar design traffic, and same geological region.

The functional and structural performance of the various projects were measured and compared. The functional performance included; roughness, surface conditions, and rutting. The structural performance was evaluated in terms of the pavement's load-deflection response using the Dynaflect device. The Dynalflect data were used to estimate the structural number (SN). Ten evaluation locations within each project were monitored annually for five years. The major forms of distresses recorded were longitudinal and transverse cracking and rutting. Recycled pavements showed moderate transverse cracking whereas control sections showed slight transverse cracking. Rutting was less than 0.25 inch on all projects. Table A27 summarizes the performances of the sections.

Table A27 Performance Data of the Louisiana Projects.

Parameter	Mean of RAP Sections	Mean of Conventional Sections	Significant Difference
Mays Ridemeter, PSI	3.9	4	No
Pavement Condition Rating (PCR)	41	42	No
Pavement Distress Rating (PDR)	21.5	21.9	No
Longitudinal Cracking, ft	3.1	2.1	Yes
Transverse Cracking, ft	3.2	2.7	No
Rutting, in	0.152	0.184	No
Structural Number (SN)	4.6	4.7	No

Field samples were collected to examine the physical properties of the extracted binders and aggregates. Samples were tested for specific gravity, asphalt content, gradation, viscosity, penetration, and ductility. All pavements showed increased densification from traffic beyond the initial construction compaction. The aggregate gradation results showed no significant changes after 5 years. The two plant produced mixtures with a recovered binder viscosity of 18,096 and 13,684 poises that did not satisfy the Louisiana requirement of a viscosity less than 12,000 poises after plant production for recycled mixes showed higher cracking than their control pairs. The physical properties of the recovered asphalt binders summarized in Table A28 revealed no significant differences between the conventional and RAP mixes.

Table A28 Performance Data of the Louisiana Projects.

Parameter	Mean of RAP Sections	Mean of Control Sections	Significant Difference
Viscosity, Poises	60,618	75,467	No
Penetration, 0.1 mm	20.5	25.5	No
Ductility, cm	24.8	41	No

Based on the data generated in this study, the researchers made the following conclusions:

- Overall, pavements containing 20-50% RAP performed similarly to the conventional pavements for a period of six to nine years after construction. (Note: the research collected the data between years 5 and 9 while the data for years 1-5 were obtained from the pavement management database).
- No significant differences existed in the recovered asphalt binder properties from pavements containing RAP and pavements without RAP.

Performance of Pavements Containing RAP in Georgia

The Georgia Department of Transportation (GDOT) has been constructing asphalt pavements containing RAP routinely since 1991. In 1995 a research project was undertaken to evaluate the performance of pavements containing RAP compared to virgin (control) asphalt pavements *(13)*. Five projects, each consisting of a RAP containing section and a control section, were subjected to detailed evaluation. In situ mixture properties (such as air voids, resilient modulus, and indirect tensile strength), recovered asphalt binder properties (such as penetration, viscosity, $G^*/\sin\delta$, and $G^*\sin\delta$), and laboratory re-compacted mix properties (such as gyratory stability index and confined, dynamic creep modulus) were measured.

The GDOT limits the amount of RAP to 40 percent of the total HMA mixture for continuous type plants and to 25 percent for batch type plants. According to the GDOT specification, when the virgin asphalt is blended with asphalt recovered from the RAP material, its absolute viscosity at 140°F after aging in the thin film oven should be between 6,000 to 16,000 poises. Consequently, the RAP material proportion in the mixtures from all five projects varied between 10 and 25%.

Table A29 Pavement Surface Evaluation Results

County	Mix	Ave. Rut depth (inch)	Raveling & Weathering	Alligator cracking	Transverse cracking	Longitudinal cracking	Other surface distress
Coffee	Virgin	0.069	None	None	Low	None	WB lane has more transverse cracks
	RAP	0.069	None	None	None	None	None
Ware	Virgin	0.069	None	None	None	None	None
	RAP	0.063	None	None	None	None	None
Chatham	Virgin	0.044	None	None	None	None	None
	RAP	0.066	None	None	None	None	None
Emmanuel	Virgin	0.009	None	None	None	Low (Continuous)	Map cracking
	RAP	0.063	None	None	None	Low (Continuous)	Long. Refl. Crack
Columbia/ Richmond	Virgin	0.013	None	None	None	None	None
	RAP	0.078	None	None	None	None	Open surface texture

Statistical analysis indicated no significant differences between the measured properties of 100% virgin and RAP containing pavements after 1.5 to 2.25 years in-service. Both virgin and RAP containing sections of the five projects are performing satisfactory with no significant rutting, raveling and weathering, and fatigue cracking (Table A29). Even though the virgin sections showed a slightly higher indirect tensile strength, no visual distress was found in RAP containing sections as a result of this difference.

It should be noted that the recovered binders exhibited a G*/sinδ value well above the 1 kPa criterion for original binders and a G*sinδ value well below 5,000 kPa at the PAV aged condition; hence, indicating higher resistance to rutting and fatigue, respectively (Table A30).

Table A30 Recovered binder test results

Recovered binder Property	**Average of 5 Projects**		**Are differences Significant at 5% Level**
	Control	**RAP**	
Penetration @ 25°C (0.1 mm)	20	20	No
Viscosity @ 60°C (Pas)	5,466	4,688	No
G*/sinδ kPa @ 64°C	17.9	15.4	No
G*sinδ kPa @ 22°C	1,356	1,288	No

Accordingly, ten additional virgin mix wearing courses projects and thirteen additional RAP containing wearing courses projects constructed during the same period throughout the state of Georgia were also evaluated. No statistically significant differences were found between the recovered asphalt properties (penetration and viscosity) from the virgin and RAP containing pavements. Additionally, based on visual inspection there was no significant overall difference in the performance of virgin and RAP containing pavements.

Based on the findings of this study, it was concluded that the RAP containing pavements are generally performing as well as the virgin pavements. Therefore, it was implied that the GDOT recycling specifications, recycled mix design procedures, and quality control are satisfactory. Additionally, the evaluation showed that the specification to achieve a viscosity between 6,000 and 16,000 poises for the blended binder (RAP binder + virgin binder) is reasonable.

PERFORMANCE OF RAP CONTAINING MIXES IN AIRFIELD PAVEMENTS

US Army Research and Development Center

In 2005, Shoenberger and Demoss *(14)* reported on the performance of HMA airfield pavements containing RAP at four air force bases (AFB): Columbus AFB, Lajes Field, MacDill AFB, and McGuire AFB. During construction, the U.S. Army Engineer Research and Development Center, Waterways experiment station (WES) was actively involved in the design or verification of the job mix formula (JMF) and also provided on-site assistance to the Air Force bases at the start of paving. The evaluated pavements were between 8 and 12 years old.

Three out of the four bases were located in different areas of the US, with the Lajes Field is located on Terceira Island in the Azores, Portugal, which has mild climate with warm summers and no freezing winter temperatures. Columbus AFB is located in Columbus, Mississippi, which has hot and humid summers and mild and wet winters. MacDill AFB is located in Tampa, FL, which has hot and humid summers and mild winters. McGuire AFB is located near Wrightstown in central New Jersey and has warm summers and moderate winter temperatures.

The existing pavement at all four bases was cold milled to a depth of at least 3.0 inches, leaving a minimal depth of HMA on top of the base course. The resulting cold milled material (RAP) combined with new aggregate, neat asphalt binder, and possibly either rejuvenators or hot mix recycling agent produced the RAP containing mix at each location. The percent of RAP used in the mixes ranged from 35 to 60%. The underlying pavement structures varied at each location and none of the pavements required repairs for structural deficiencies during construction.

The RAP containing mixtures were all placed with paving machines using the widest placement widths possible to reduce the number of construction cold joints. Steel wheel and vibratory rollers, in the static mode, were used for breakdown and finish rolling. Vibratory and pneumatic rollers, weighing a minimum of 20 tons (18 kg), were used for compaction of the pavements. The cold construction joints at all locations, except for Columbus AFB, were cut back from 2 to 6 inch (5 to 15 cm) to assist in achieving the required joint density.

A pavement condition survey was performed at each location to determine its condition and amount and types of distress experienced by the pavement. The performance of the pavement sections was assessed by performing the pavement condition index (PCI) survey over time and compared to virgin HMA in place for the same period of time. Additionally, pavement samples from all locations were tested to determine the material properties of the binder, aggregate, and mixture. The results of these tests were then compared with test data obtained during construction to verify that the field mixture met specification requirements.

A combination of several factors limited the effectiveness of this performance evaluation, including: the limited number of RAP containing pavements evaluated, differing times of service, differing climatic conditions, variations in the amounts of RAP material used, and variations in virgin aggregates, asphalts, and recycling agents used in the blended mixtures. Therefore, only general trends were feasible due to the multiple variations of the various properties of the pavements investigated in this study.

The researchers reported that the PCI values varied from 37 (poor) to 80 (very good) with block cracking at low severity levels being the major distress noted on all pavements and at all locations except for Lajes Field where the block cracking was at high severity level. The Lajes Field had the only RAP containing mixes that contained a recycling agent. A laboratory study conducted by Brown *(15)* with this type of recycling agent and others showed a decrease in durability in the RAP containing mixtures versus only using asphalt cement without a recycling agent. The pavement at the MacDill AFB had low to medium severity patching and raveling distresses in addition to low severity block cracking.

Pavement samples were obtained from each section. Properties such as field density, maximum theoretical density, asphalt content, and aggregate gradation were obtained. The recovered asphalt cements were evaluated for penetration, viscosity, and specific gravity (Table A31). Aggregate properties in terms of specific gravity, absorption, percentage of crushed and flat and elongated particles were also determined. The mixtures were re-compacted to determine a laboratory density and void properties.

Based on the performance and materials evaluations of the various pavements the researchers were able to make the following conclusions *(14)*.

- Aging or hardening of the asphalt binder of the RAP containing mixtures was illustrated by decreasing penetration values and by an increased specific gravity.

- The RAP containing mixtures evaluated in this study generally contained excessive amounts of asphalt cement which should have made them susceptible to load related distresses. However, the pavements generally had only climatic or durability related distresses.
- The durability of these pavements was probably adversely affected by densities that were below the minimum specified values.
- The overall performance of these pavements suggests that, at least for the airfields evaluated, possibly the current airfield pavement mixture design should be adjusted to provide for increased pavement durability rather than load carrying capability.
- HMA pavements containing RAP have been successfully used by the Air Force. These airfield pavements have provided good performance. The pavements investigated have performed satisfactorily for 8 to 12 years (except for a high speed taxiway at McGuire AFB). The majority of distresses found in the evaluated RAP containing pavements, as with virgin mixtures, were from environmental or climatic causes with very few load related distresses even in the parking and taxiway areas.
- PCI values, obtained for the RAP containing pavements, showed that the rates of deterioration appear to be similar for all sections. Condition surveys showed that the investigated RAP containing pavements performed similar to virgin HMA mixtures under similar circumstances.
- The use of RAP in HMA pavements on airfield can be an economical solution while being beneficial to the environmentally conscious society.

Table A31 Asphalt Binder Contents and Properties.

<table>
<tr><th colspan="3">Location</th><th>Columbus AFB</th><th>Lajes Field*</th><th>Macdill AFB</th><th>McGuire AFB</th></tr>
<tr><td rowspan="2">Asphalt content</td><td colspan="2">Mean (M)</td><td>7.50</td><td>6.60</td><td>5.27</td><td>5.10</td></tr>
<tr><td colspan="2">Standard deviation (SD)</td><td>0.141</td><td>-</td><td>0.350</td><td>0.000</td></tr>
<tr><td rowspan="4">Penetration (0.1 mm)</td><td rowspan="2">25°C</td><td>M</td><td>23.6</td><td>15.0</td><td>25.0</td><td>21.5</td></tr>
<tr><td>SD</td><td>12.50</td><td>-</td><td>5.30</td><td>0.71</td></tr>
<tr><td rowspan="2">1.1°C</td><td>M</td><td>16.6</td><td>10.0</td><td>17.0</td><td>10.0</td></tr>
<tr><td>SD</td><td>11.1</td><td>-</td><td>6.1</td><td>1.4</td></tr>
<tr><td rowspan="4">Viscosity</td><td rowspan="2">60°C (Pas)</td><td>M</td><td>133,665</td><td>107,136</td><td>27,972</td><td>25,764</td></tr>
<tr><td>SD</td><td>101,880</td><td>-</td><td>22,150</td><td>303</td></tr>
<tr><td rowspan="2">135°C (cm^2/s)</td><td>M</td><td>26.66</td><td>19.84</td><td>11.68</td><td>12.66</td></tr>
<tr><td>SD</td><td>11.74</td><td>-</td><td>28.70</td><td>0.25</td></tr>
<tr><td colspan="2" rowspan="2">Specific gravity</td><td>M</td><td>1.060</td><td>1.064</td><td>1.050</td><td>1.050</td></tr>
<tr><td>SD</td><td>0.00419</td><td>-</td><td>0.00502</td><td>0.0</td></tr>
</table>

*Sample from only one location, no statistical data available

Massachusetts Port Authority

The hot weather pavement rutting and moisture induced damage at the Logan International Airport in Boston, MA, have become problematic on several taxiways and aprons that are subjected to slow moving and standing aircraft loads. In 2003, the Massachusetts Port Authority (Massport) evaluated the performance of seven mix designs that are presently in service by the testing of field cores at the Worcester Polytechnic Institute (WPI). Four mixes out of the seven were selected for more extensive evaluation in terms of rutting and moisture damage: a PG76-28 modified P-401, a reclaimed asphalt pavement (RAP)/latex modified P-401, a latex modified stone matrix asphalt (SMA), and a Rosphalt 50™ modified P-401. The RAP P-401 mixture contained 18.5% RAP material and a 4% SBR latex modified PG64-28 asphalt binder. The Rosphalt 50™ is a trademark of the Royston Laboratories Division of Chase Corporation and is described as a concentrated thermoplastic virgin polymeric material which is added to an HMA to improve its rut resistance. Dry and wet asphalt pavement analyzer (APA) rut tests were carried out at 140°F (60ºC). The tested cores had in-place air voids between 3.0 and 4.5% except for the Rosphalt 50™ modified P-401 mixture which had air voids close to 2.0% *(16)*.

Under dry condition in the APA test, the PG76-28 P-401 mix exhibited the highest rut depth (5.3 mm) whereas the RAP P-401 exhibited the lowest rut depth (1.9 mm) at 140°F (60ºC) and under 8,000 cycles. The statistical analysis of the rut depths indicated a significant difference in rutting of the various mixtures, and that the RAP P-

401 and Rosphalt 50™ P-401 mixtures ranked better than the PG76-28 P-401 and SMA mixtures.

Under wet condition in the APA test, all mixtures exhibited a wet rut depth comparable to the dry rut depth except for the RAP P-401 which indicated aggravation in rutting in the presence of moisture. However, since none of the wet rut depths are above the 4.5-5.0 mm range, it was difficult for researchers to predict whether any of these mixtures is moisture susceptible or not. Additionally, a visual observation showed no evidence of stripping in these mixtures. From a purely ranking point of view, the results indicated that the Rosphalt 50™ P-401 mix had the lowest wet rutting, and the PG76-28 P-401 had the highest wet rutting while no statistically significant differences in the wet rut depths were identified.

As of 2003, the RAP P-401 mixture has performed very well in the field through two summers and tested well in the laboratory. The PG76-28 and SMA mixtures have shown slight indications of rutting in the field. Since these three mixes were the least costly to produce, they were recommended for further evaluations. Researchers recommended that long-term field performance of these mixtures be evaluated, and that accelerated loading and testing of these mixtures be conducted *(16)*.

In the past, Massport has tried to prevent moisture damage problems by modifying the FAA P-401 specification to include a higher retained tensile strength for their HMA mixtures. In view of those unsuccessful attempts, Massport has recently started a project to conduct a study to recommend changes in their specification to produce more durable mixes. As part of this effort, Mallick et al. evaluated locally available aggregates typically used to manufacture HMA mixtures for the Logan International Airport to see if they are prone to stripping and to determine what additives would be required to improve the mixtures' ability to resist moisture and stress induced pavement damages *(17)*.

This project was broken into multiple phases, each one building on the results of the previous ones. During the different phases, a number of different types of aggregates, asphalt, accelerated loading testing and laboratory tests were used. Tests were conducted on laboratory mixed and compacted samples, plant mixed and laboratory compacted samples, as well as cores from field compacted mixes. The results were analyzed and used for drawing conclusions and making recommendations to be implemented and applied for HMA paving jobs at Logan airport only. In addition to the evaluated mixtures, the study also included several mixtures with 18% RAP and a 4% latex PG64-28 binder.

As a result of the study, it was found that the TSR test alone is not a good indicator of performance for local mixes. The effectiveness of hydrated lime in reducing moisture damage at high temperatures was demonstrated in the study. Mixes containing about 18% RAP were found to perform adequately but quality control was a concern due to varying sources of RAP. Multiple cycles of freeze thaw were found necessary to identify moisture susceptible mixes. Finally, an accelerated loading test was found to be effective in identifying good and poor performing mixes.

It has been recommended that hydrated lime be used in paving mixes, the tensile strength test be run after six cycles of freeze thaw and supplemented with an accelerated loading test to be conducted for paving mixes before accepted for placement. Accelerated loading test result limits for acceptance were established on the basis of tests conducted on good and poor performing field cores. The feasibility of producing a desirable mix, conforming to these limits, with locally available materials was confirmed.

Although the recently installed surface mixtures containing RAP at Logan are performing well in the field and the laboratory, the researchers recommended that only virgin materials be used for better quality control. Therefore, researchers did not allow the use of RAP within the proposed job mix formula unless the contractor can demonstrate that the RAP mix can meet the performance standards of a virgin material mix *(17)*.

Naval Civil Engineering Laboratory, Port Hueneme, California

In 1986, Cline and Hironaka documented the relative airfield performance of pavement containing RAP versus virgin asphalt concrete pavement surfaces *(18)*. At that time there were about 20 airports that included RAP in HMA. However, only 5 of them had RAP in the surface course out of which only two were used in this study: the Needles airport in California with 50% RAP material and the Barnes county municipal airport at Valley City in North Dakota with 70% RAP material. Both of these airports had general aviation traffic of low volume.

The selected airfield pavements were evaluated using the pavement condition index (PCI) after 5 years of reconstruction along with laboratory testing on field cores. The field cores were tested for Marshal stability and flow, resilient modulus, indirect tensile strength, Lottman water susceptibility, asphalt content, and asphalt properties. The properties of the field cores were compared to those of other RAP containing highway pavements.

The pavement at the Needles airport Runway 2-20 was reconstructed in 1981. The top 2.0 inch of the existing 5.5 inch asphalt layer was removed through cold planning and overlaid with a 2.5 inch HMA with 50% RAP material. In 1985, a fog seal was applied to the raveled surface. After 5 years of construction, the PCI of the runway averaged 85 with an overall rating classified as very good in accordance with FAA specifications. The major distresses observed at the runway were longitudinal and transverse cracking and raveling of low severity. The primary distress mechanism was climatic effects on material durability. The laboratory tests on the field cores from the Runway 2-20 (Table A32) showed higher Marshall stability and flow values than those obtained on the highway projects with RAP covered in this study. The resilient modulus values at Needles were considerably higher than those on highway projects with RAP indicating a stiffer mix. The dry tensile strength of the field cores was higher than previously built projects with RAP. The Lottman water susceptibility test showed 87% retained tensile strength ratio indicating that stripping and loss of strength in the presence of water should not be a major problem. The extracted/recovered asphalt binder from cores exhibited high viscosity and low penetration values indicating an aged binder *(18)*.

Table A32 Summary of results from tests on core samples

Property		Needles airport	Valley city airport
Resilient modulus at 77°F 106 (psi)	Before Lottman	1.734	0.404
	After Lottman	1.509	0.103
	% Retained	87.0	25.5
Splitting tensile strength at 77°F (psi)	Before Lottman	354	101
	After Lottman	307	35
	% Retained	86.7	34.6
Marshall stability (lbs)		3160	773
Marshall flow (0.01 inch)		27	21
Asphalt content	Base AC	6.9	6.4
	Surface AC		6.2
Penetration at 77°F (0.1mm)	Base AC	16	25
	Surface AC		58
Viscosity at 77°F (poises)	Base AC	12800	17000
	Surface AC		2009

The test pavements at the taxiway, runway, and parking apron of the Valley City, North Dakota airport were reconstructed in 1980. The existing pavement was removed with a milling machine down to the base course and overlaid with a full depth HMA with 70% RAP material and a virgin binder of 200-300 penetration grade. The RAP HMA mix was placed with a conventional paving machine. The contractor had some problems with the plant due to the drum clogging and chutes plugging with the RAP material. Immediately after construction a 3/8-inch chip seal was placed on top of the RAP HMA surface. Following construction, the pavement was subjected to crack filling and routing every year as required. In 1985, when the PCI survey was performed the taxiway averaged a 75 rating as very good in accordance with the FAA specifications. The primary pavement distresses at the Valley City airport were longitudinal and transverse cracking and raveling of the chip seal at low severity. The primary distress mechanism was climate and material durability. The laboratory tests on the field cores from the Valley City airport (Table A32) showed lower Marshall stability value than the other RAP containing highway projects covered in this study. The resilient modulus values of the Valley City airport mixtures were generally high. The dry tensile strength of the field cores was similar to previously constructed RAP mixtures. The Lottman water susceptibility test showed 25 to 35% retained tensile strength ratio indicating potential stripping and loss of strength in the presence of water. The extracted/recovered asphalt binder from cores exhibited typical values for the viscosity and penetration for asphalt surfaces *(18)*.

HIGHWAY AGENCIES SPECIFICATIONS

A large number of highway agencies allow RAP in HMA pavements. Several agencies have their own specifications on RAP usage in HMA mixtures. Table A33 summarizes the various highway agencies specifications for the use of RAP materials in HMA mixtures along with the mix design method used *(19, 20)*. Additionally, Figures A19 through A24 show the responses from a survey conducted by the North Carolina Department of Transportation for the specified and average use of RAP in 38 different U.S. states.

Table A33 Summary of State Specifications on RAP

State	Maximum Amount of RAP	General Information	Mix Design Method
Alabama	Type Mix / % RAP / Remarks PATB / 0 / Permeable Asphalt Treated Base OGFC / 0 SMA / 15 / All layers-No RAP with Chert Gravel SP&IMP / 0 / Wearing layers- if RAP with Chert Gravel SP&IMP / 15 / Other layers- if RAP with Chert Gravel SP&IMP / 20 / All layers- if RAP without Chert Gravel * SP refers to “Superpave” * IMP refers to “Improved bituminous” No recycled asphalt pavement shall be allowed in Polymer Modified Open Graded Friction Course.	The total amount of RAP allowed in SMA is limited to 15% by weight {mass} of aggregate. RAP containing gravel or fine aggregate manufactured from gravel with a bulk specific gravity less than 2.550 is not allowed in SMA. The Contractor may use reclaimed material that was previously milled from another Alabama Department of Transportation project provided the source can be identified in regard to route, mile {kilometer} post, and layers (based on thickness) milled. The reclaimed material must be traceable back to the original job-mix formula.	Superpave
Alaska	-	-	-
Arizona	-	-	-
Arkansas	30% RAP Max	This item shall consist of an asphalt concrete base, binder, or surface course mixed at a central plant and constructed on the completed and approved sub-grade, base, or surface course in conformance with the lines, grades, and dimensions shown on the plans, and according to the provisions of these specifications. The mixture shall contain a minimum of 70% virgin material. For RAP < 15% and a PG64-22 grade, temperature viscosity curved not required	Superpave
California	Cal-Trans allows 15% RAP in new HMA	Reportedly not commonly used (Rita Leahy)	Hveem
Colorado	The hot bituminous pavement shall not contain more than 15% RAP.	-	Superpave
Connecticut	The blend percentage of RAP shall be a maximum of 40% reclaimed material; a maximum of 15% RAP may be routinely used after prior notification and approval by the Director of Research and Materials.	Final Binder grade must meet the Superpave requirement.	Marshall

State	Maximum Amount of RAP	General Information	Mix Design Method
	RAP may be used in class 1, 2, 3, or 4.		
Delaware	Mixtures Deep lift Type B Type C Drum 20 10 10 Batch 20 10 10 Deep lift refers to "Base Course" Type B refers to "Dense graded base and binder course" Type C refers to "Dense graded surface course"	In all mixture types, the contribution of the RAP asphalt cement shall not exceed 50% of the design asphalt content for the recycled mix. The testing of the physical properties will govern the percentage of RAP permitted in the recycled mix.	Marshall
Florida	The use of RAP material will not be permitted for asphalt concrete friction course pavement ESAL<10,000,000, limit the amount of RAP material used in the mix to a maximum of 50% by weight of total aggregate. ESAL>10,000,000, limit the amount of RAP material used in the mix to a maximum of 30% by weight of total aggregate.	When using a PG 76-22 Asphalt Binder, limit the amount of RAP material used in the mix to a maximum of 15% by weight of total aggregate; when using PG 64-22, limit the amount of RAP material used in the mix to a maximum of 20-29% by weight of total aggregate; use Recycling agent, if RAP ≥ 30%. Use RAP from an FDOT approved stockpile or RAP that has an FDOT furnished Pavement Composition Data Sheet. Provide RAP material having a minimum average asphalt content of 4.0% by weight of total mix. The Engineer may sample the stockpile to verify that this requirement is met. Minimum dry tensile strength of 100 psi.	Superpave
Georgia	The maximum ratio of RAP material to the recycled mixture is 40% for continuous mix type plants and 25% for batch type plants.	Add lime to recycled mixtures at a min rate of 1% of virgin aggregate plus a min of 0.5% of aggregate in RAP portion. Use Neat asphalt binder. Do not use RAP materials that contain alluvial gravel or local sand in any mixture placed on interstate projects except for mixtures used in shoulder construction. When used in shoulder construction, limit RAP containing local sand or alluvial gravel so that the sand or gravel contributes no more than 20% of the total aggregate	Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
		portion of the mix. Max RAP size 2”. Asphalt binder with recovered RAP binder shall have a viscosity of 6,000 to 16,000 poises (600 to 1600 Pa) after TFOT.	
Hawaii	-	-	-
Idaho	RAP shall not be allowed as part of the Job Mix formula unless specified in the contract.	Stockpiles of RAP shall be limited to 10 ft (3 m) in height and no equipment will be allowed on top of the stockpile.	Hveem
Illinois	ESALs (millions) / Ndes / %RAP Binder/Leveling / Surface <0.3 / 30 / 30 / 30 0.3-3 / 50 / 25 / 15 3-10 / 70 / 15 / 10 10-30 / 90 / 10 / 10 >30 / 105 / 0 / 0 RAP containing steel slag will be permitted for use in top-lift surface mixtures only.	If the Contractor is allowed to use more than 15 percent RAP, as specified on the plans, a softer performance graded binder may be required as determined by the Engineer. RAP will not be permitted in mixtures containing polymer modifiers.	Marshall
Indiana	Recycled materials shall not be used in ESAL (> 3,000,000) surface mixtures and open graded mixtures.	Mixtures containing 15.0% or less RAP shall use the same grade of binder as specified. The binder for mixtures containing greater than 15.0% and up to 25.0% RAP shall be reduced by one temperature classification, 6°C, for both the upper and lower temperature classifications. When only RAP is used in the mixture (i.e., no shingles), the RAP shall not exceed 25.0% by weight (mass) of the total mixture.	Superpave
Iowa	Not more than 30% of the asphalt binder in a final surface course mixture shall come from the RAP.	Designated RAP : When RAP is taken from a project, or is furnished by the Contracting Authority, the contract	Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
	Up to 10% of unclassified RAP may be incorporated into intermediate mixes for under 3,000,000 ESALs and all base mixes	documents will indicate quantity of RAP expected to be available. Certified RAP : The RAP shall be from a known source and of the proper quality for the intended use, with no material added from other sources during the time in stockpile. The Contractor shall certify to this before use. RAP from not more than two known sources at a time will be allowed. Certified RAP may be used in the base and intermediate course of mixes for which the RAP aggregate qualifies. RAP may also be used in surface courses when authorized by the Engineer. A certified RAP stockpile shall be sealed or protected. Unclassified RAP shall not be used in surface courses. Unclassified RAP shall not be used in intermediate or base mixtures containing designated or certified RAP. The Engineer will inspect the unclassified RAP stockpile visually for uniformity. Unclassified RAP stockpiles containing concrete chunks, grass, dirt, wood, metal, coal tar, or other foreign or environmentally restricted materials shall not be used, unless approved by the Engineer.	
Kansas	-	-	-
Kentucky	Do not use reclaimed materials in open-graded friction courses. Use reclaimed asphalt pavement (RAP) from Department projects or other approved source in hot asphalt mixtures, provided mixture requirements are satisfied. Kentucky Method 64-427 implies over 30% RAP can be used. No	When RAP in asphalt mixtures requiring polish-resistant aggregate, provide documentation to the Engineer's satisfaction that the reclaimed material consists of a given portion of polish-resistant aggregate.	Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
	maximum value identified.		
Louisiana	Reclaimed asphaltic pavement (RAP) will not be allowed in final wearing courses on roadway travel lanes and airports. Base courses containing 20-30 percent RAP shall use PG 58-28 asphalt cement and shall meet the viscosity limits. Up to 20% RAP could be used for Type 3 mixture (1” Nominal, Binder Course). Up to 20% RAP could be used for Type 8 mixture (1” Nominal, Binder Course). Up to 30% RAP could be used for Type 5 mixture (1” Nominal, Base Course). All shoulder wearing courses may include up to 20% Reclaimed Asphaltic Pavement (RAP).	Reclaimed asphaltic pavement shall be stockpiled separate from other materials at the plant and will be subject to approval prior to use. Mixtures containing a maximum of 20 percent RAP will have no viscosity limits.	Marshall
Maine	The Contractor may use a maximum of 15% RAP in any base, binder, surface, or shim course. The Contractor may be allowed to use more than 15% RAP, up to a maximum of 25% RAP, in a base, binder, or shim course provided that PG 58-34 asphalt binder is used in the mixture.	-	Superpave
Maryland	When using 15 percent or less of RAP, binder viscosity adjustments are not required.	The Contractor may elect to use crushed, recycled asphalt pavement (RAP) material or a maximum of 5% roofing shingles from manufacturing waste. The use of RAP may be considered for applications where higher polish value aggregates are required. Approval for use will be on an individual project basis. Crushed glass shall not be used in surface mixes. RAP and roofing shingles from manufacturing waste shall not be used in gap-graded mixes, surface mixes requiring	Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
		high polish aggregate, or mixes requiring elastomer type polymer binder.	
Massachusetts	The maximum amount of RAP for surface courses shall be 10% except no RAP will be allowed in the open graded friction course (OGFC).	The proportion of RAP to virgin aggregate shall be limited to a maximum of 40% for drum mix plants and 20% for modified batch plants.	Marshall, Superpave
Michigan	0-17 % RAP binder by total binder weight, no adjustment 18-27 % RAP binder by total binder weight, one grade lower for high temperature or using blending chart for high and low temperatures. >28 % RAP binder by total binder weight, the binder grade is selected by using blend chart for high and low temperatures.	In mixtures containing RAP, the required minimum fine aggregate angularity must be met by the virgin material. An aggregate wear index (AWI) of 240 will be assigned to all RAPs unless evidence is presented to support a different value. Complete mix design analysis including gradation of aggregate, asphalt content, and theoretical max specific gravity for every 1000 tons of processed RAP. For stockpiles and projects less than 3000 tons, a min of 3 mix analyses is required.	Marshall, Superpave
Minnesota	The Contractor may use up to 30 percent recycled asphaltic pavement (RAP) in all wearing layers and High Volume (HV) non wear layers. (HV, >3 millions ESAL). Medium Volume (MV) mixtures may contain a maximum of 30% RAP in the non wear layers above 90 mm (3.5 inch) and a maximum of 40% RAP in layers 90 mm (3.5 inch) and greater in depth from the surface. (MV, 1-3 millions ESAL). Low Volume (LV) mixtures may contain a maximum of 40% RAP in the non wear layers. (<1 millions ESAL)	The combined RAP and virgin aggregate shall meet the composite fine aggregate angularity or calculated crushed requirements.	Superpave, Marshall
Mississippi	HMA mixture Max. RAP % 4.75mm 0 9.5 mm 15 Top 12.5mm 15 Underlying	-	Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
	12.5mm 30 19.0mm 30 25.0mm 30		
Missouri	Up to 15% of RAP may be substituted in lieu of mineral aggregate	All Reclaimed Asphalt Pavement (RAP) material not having a direct verifiable tie to the MoDOT system shall be tested in accordance with AASHTO TP 58, Method for Resistance of Coarse Aggregate Degradation by Abrasion in the Micro-Deval Apparatus. Material with a Micro-Deval percent loss of more than 20 shall not be used.	Marshall
Montana	-	-	-
Nebraska	Asphalt concrete % RAP SPS 50 SP0 45 SP1 35 SP2 25 SP3 25 SP4 15 SP5 15 * These mix designations essentially confirm to AASHTO traffic levels. SPS and SP0 are low traffic volume; SP1 is equal to 0.3 million ESALs; SP5 is equal to 30 million ESALs.	In recycled asphaltic concrete mixtures with a percent of Reclaimed Asphalt Pavement (RAP) of 25 or less, the asphalt cement shall be AC-10. In recycled asphaltic concrete mixtures with a percent of Reclaimed Asphalt Pavement (RAP) of greater than 25 percent, the asphalt cement shall be AC-5.	Marshall
Nevada	-	-	-
New Hampshire	RAP incorporated in wearing courses for both a drum mixer and a batch plant shall not exceed 15%. The blend percentage or RAP for a drum mixer shall not exceed 50% and for a batch plant shall not exceed 35%.	If the source of RAP is unknown, but is of acceptable quality as described above, it will be allowed to a maximum of 15% of the total batch weight.	Marshall, Superpave
New Jersey	Up to 15% RAP for Superpave HMA surface course. Up to 25% of RAP for base or intermediate course	-	Marshall
New Mexico	Reclaimed Asphalt Pavement (RAP) shall not be used in the production of Superpave asphalt mixtures. Reclaimed asphalt pavement (RAP) shall not be used in the	RAP maximum size of 1.5”.	Marshall

State	Maximum Amount of RAP	General Information	Mix Design Method
	production of SMA. Reclaimed Asphalt Pavement (RAP) will be permitted in all plant-mix bituminous pavement (PMBP) for Dense-Grade A, B, C, D.		
New York	(Table 703-09A) Plant type % Reclaimed material Max Drum 70 Batch 50	RAP from each pavement source shall be stockpiled on a free draining base separately from other aggregate or RAP sources.	Superpave
North Carolina	For Type S 12.5D mixes, the maximum percentage of reclaimed asphalt material is limited to 15% and must be produced using virgin asphalt binder grade PG 76-22. Reclaimed asphalt shingle (RAS) material may constitute up to six (6) percent by weight of total mixture for any mix.	Use separate stockpiles for RAP with differences in properties. RAP may constitute up to 50% of the total material used in recycled mixtures, except for mix Type S 12.5D and mixtures containing reclaimed asphalt shingle material (RAS). When both RAP and RAS are used, do not use a combined percentage of RAS and RAP greater than 15% by weight of total mixture, unless otherwise approved. 15-25%, the virgin binder PG grade must be one grade below the specified grade (both high and low temperature grade). RAP maximum size of 2.0".	Superpave
North Dakota	Section 407.04 F implies that more than 50% RAP can be used. Section 407 has separated specification and pay item for Hot Recycled Bituminous Pavement.	Hot recycled bituminous mix shall not be placed on a damp surface, on a frozen roadbed, or when weather conditions prevent the proper handling or finishing of the bituminous mixtures. Presence of frost particles in the roadbed is sufficient evidence of being frozen. Hot recycled bituminous mix shall not be placed when the air or mat surface temperatures are below the following minimums: Surface course subsurface Existing Mat	Marshall, Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
		Thickness Temp. Temp. Temp. 1-1/2" or less 45F 40F 40F More than 1-1/2" 40F 35F 40F	
Ohio	For surface courses with polymer modified asphalt binder, the Contractor may use a maximum of 10% reclaimed asphalt concrete pavement. For surface courses, the Contractor may use up to a maximum of 20% of reclaimed asphalt concrete pavement. For intermediate courses, the Contractor may use up to a maximum of 35% of reclaimed asphalt concrete pavement. For Asphalt Concrete Base, the Contractor may use 10 to 50% of reclaimed asphalt concrete pavement	No change in the job mix formula is necessary.	
Oklahoma	Bituminous mixtures containing up to 25 percent reclaimed asphalt concrete pavement will be accepted for bases and surfaces. For roadways with 0.3 million ESALS or more, bituminous mixtures containing reclaimed asphalt concrete pavement will not be accepted in the wearing course.	-	Superpave
Oregon	No more than 30% RAP material will be allowed in the new HMAC pavement. RAP material will not be permitted in open graded HMAC or Level 4 (very heavy traffic volume and truck traffic) dense graded HMAC wearing courses.	When RAP material is used at a rate of less than 15%, no adjustment to the new asphalt will be required. When utilizing RAP at a rate at or above 15%, the combined RAP and new asphalt shall provide blended properties equivalent to the specified grade. Determine the blended properties according to ASTM D 4887. Determine asphalt cement properties for the RAP material from asphalt cement recovered from the RAP according to AASHTO T 170.	Marshall, Superpave
Pennsylvania	Unless the source(s) of RAP and/or RAM (Recycled Aggregate Material) is documented, such that the original SRL (Skid Resistance Level) designation of the coarse aggregate in the reclaimed material is known, limit RAP and RAM to a maximum of 15% of the mix composition.	For bituminous ID courses constructed under RPS (Restricted Performance Specifications) specifications, printed ticket acceptance may be elected if RAP in the reviewed job-mix formula does not exceed 15%. Obtain at least 10 samples from the stockpile at different	Marshall

State	Maximum Amount of RAP	General Information	Mix Design Method
		locations and extract them to determine the average RAP mix composition. At least 95% of RAP shall pass the 2.0" sieve.	
Puerto Rico	The maximum percentage of RAP allowed to be incorporated in surface courses (S-1 and S-2) shall be 5% by weight of total mix. The maximum percentage of RAP allowed to be incorporated in base and leveling courses (B-1, B-2, L-1 and L-2) shall be 10% by weight of total mix.	-	
Rhode Island	Allow RAP between 10 and 30%.	Virgin binder should be an AC20. Gradation and binder content of RAP are required.	Marshall
South Carolina	Type Mix % RAP Base types 1&2 10-30 Binder types 1&2 and Surface type 3&4 10-25 Surface type 1 10-20 Reclaimed asphalt material will not be allowed in Superpave mixes or Surface Types 1B and 1C. RAP is limited to 15% maximum when introduced in the hot elevator.	The RAP will be milled material from Department projects. Stockpiles of RAP material shall be separated by project and a sign satisfactory to the Engineer shall be erected and maintained by the Contractor on each stockpile to identify the source(s). Extraction tests shall be performed at a rate of one per 1000 tons of RAP, with a minimum of 3 per stockpile. RAP maximum size of 2".	Marshall
South Dakota	No reclaimed asphalt pavements (RAP) are allowed in the asphalt concrete unless specified in the plans.	Materials and additives shall be fed simultaneously into the dryer. Recycled asphalt (RAP) when specified to be used shall be fed at the midpoint of the drum unless the drum mix plant has a manufacturer's design technology made specifically for RAP entry at a different location.	Marshall
Tennessee	A maximum of 30% of the recycled material may be incorporated into the final mixture of Grading "E" mix on shoulders. (Base) A maximum of 15% of the recycled material may be incorporated into 307 "A" mix. A maximum of 20% of the recycled material may be incorporated into the final mix for 307 "B", "B-M", "B-M2", "C-W" and "C";	-	Marshall, Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
	except that if the material is of uniform quality as documented by asphalt content and gradation test, the 20% maximum will not apply. In any event, at least 65% of the asphalt cement in the final mix shall be new material.		
Texas	Only RAP from designated sources may be used in surface courses. Only RAP from state-owned sources will be allowed in mixes using more than 20% RAP	The RAP to be used in the mix shall be crushed or broken to the extent that 100% will pass the 2" sieve. Unless otherwise shown on the plans, the Engineer will furnish the mix design for mixtures when using 20% or less RAP; the Contractor shall furnish the mixture design for all mixtures containing more than 20% RAP. RAP is rejected if the decantation value exceeds 5% and the PI greater than 8. The decantation and PI requirement do not apply to RAP samples with asphalt removed by extraction.	Texas Gyratory design with Hamburg wheel test
Utah	May use up to 25% RAP by total weight in the HMA. RAP may only be used if approved by Engineer.	Only material rotomilled from 2" below the existing surface will be allowed to be recycled into the dense-graded asphalt concrete material. Stockpile will be separated into two piles: (1) passing the 1-1/2" sieve and retained on the No. 4 sieve. (2) Material passing the No. 4 sieve. (3) Use separate cold feed bins for each stockpile. high temperature PG grade may be 1 grade > grade specified. >= 5% change in either RAP or virgin materials requires a new mix design.	Superpave
Vermont	The RAP from different projects shall be separated in individual stockpiles according to specific pavement source and type of mix by the Contractor, unless otherwise directed by the Engineer.	-	Marshall

State	Maximum Amount of RAP	General Information	Mix Design Method
Virginia	% RAP Mix ESAL (millions) PG grade 0-20 >20.1 SM 9.0A 0-3 64-22 64-22 58-28 SM-9.0D 3-10 70-22 70-22 64-28 SM-9.0E >10 76-22 76-22 70-28 SM-9.5A 0-3 64-22 64-22 58-28 SM-9.5D 3-10 70-22 70-22 64-28 SM-9.5E >10 76-22 76-22 70-28 SM12.5A 0-3 64-22 64-22 58-28 SM-12.5D 3-10 70-22 70-22 64-28 SM-12.5E >10 76-22 76-22 70-28 IM-19.0A <10 64-22 64-22 58-28 IM-19.0D >=10 70-22 70-22 64-28 BM-25.0 all ranges 64-22 64-22 64-22 BM-37.5 >=10 64-22 64-22 64-22 BM-25.0 and BM-37.5 mixes using more than 25 percent RAP shall use a PG 58-22. * SM=Surface Mix, IM=Intermediate Mix, and BM= Base Mix	RAP shall be processed in such a manner as to ensure that the maximum top size introduced into the mix shall be 2”. Reclaimed asphalt pavement may not be used as component material unless approved by the Engineer for stabilized open-graded material on a prepared subbase or subgrade. The aggregate specific gravity of RAP shall be the effective aggregate specific gravity calculated from the result of AASHTO T 209 and VTM 102	Superpave
Washington	The RAP from different projects shall be separated in individual stockpiles according to specific pavement source and type of mix by the Contractor, unless otherwise directed by the Engineer. Recycled materials shall not be used in asphalt concrete Class D. (Open graded mixes)	-	Superpave
West Virginia	-	-	Marshall, Superpave

State	Maximum Amount of RAP	General Information	Mix Design Method
Wisconsin	The contractor may use up to 35 percent RAP material in lower layer and base mixtures and up to 20 percent in upper layer mixtures. The contractor may use up to 25% RAP for lower layers and up to 20% RAP for upper layers without changing the asphaltic binder grade. If using more than that amount of RAP, furnish binder with a low temperature rating one grade lower than the contract designates, unless testing indicates the resultant binder meets the grade the contract originally specified.	-	Marshall
Wyoming	Only RAP from state-owned sources will be allowed in mixes using more than 35% RAP	RAP may be used for all courses.	Marshall, Superpave

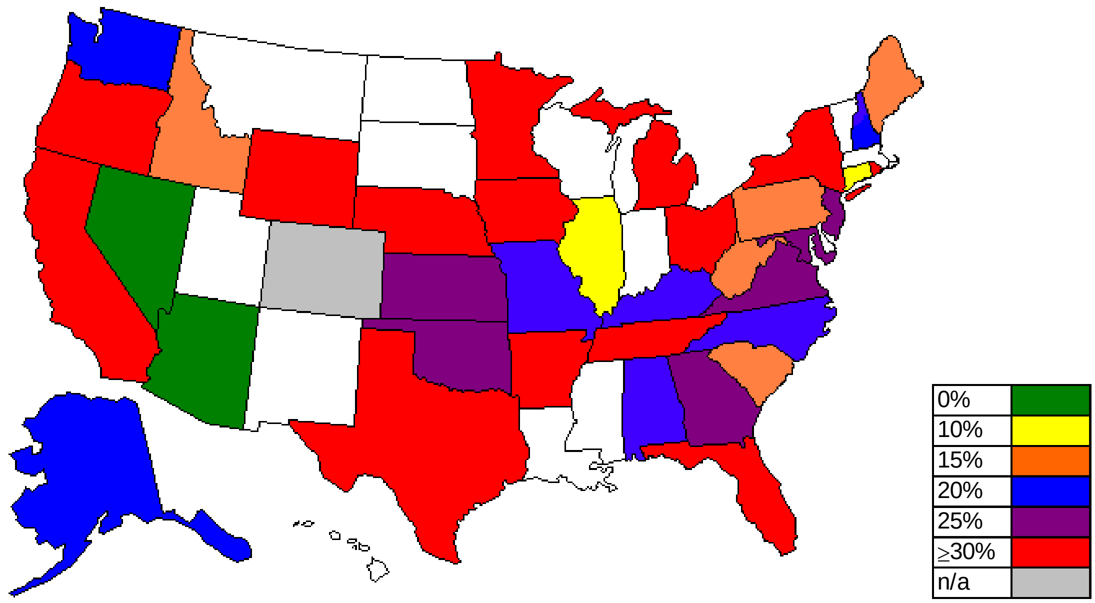


Figure A19 Specified use of RAP in HMA base mixes (After North Carolina DOT)

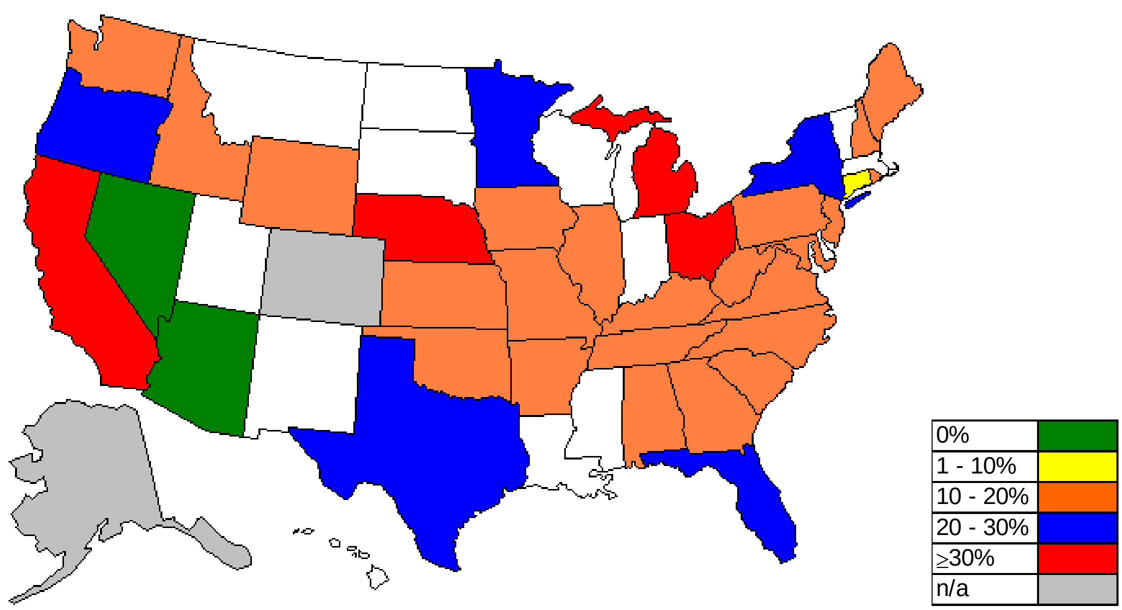


Figure A20 Average use of RAP in HMA base mixes (After North Carolina DOT)

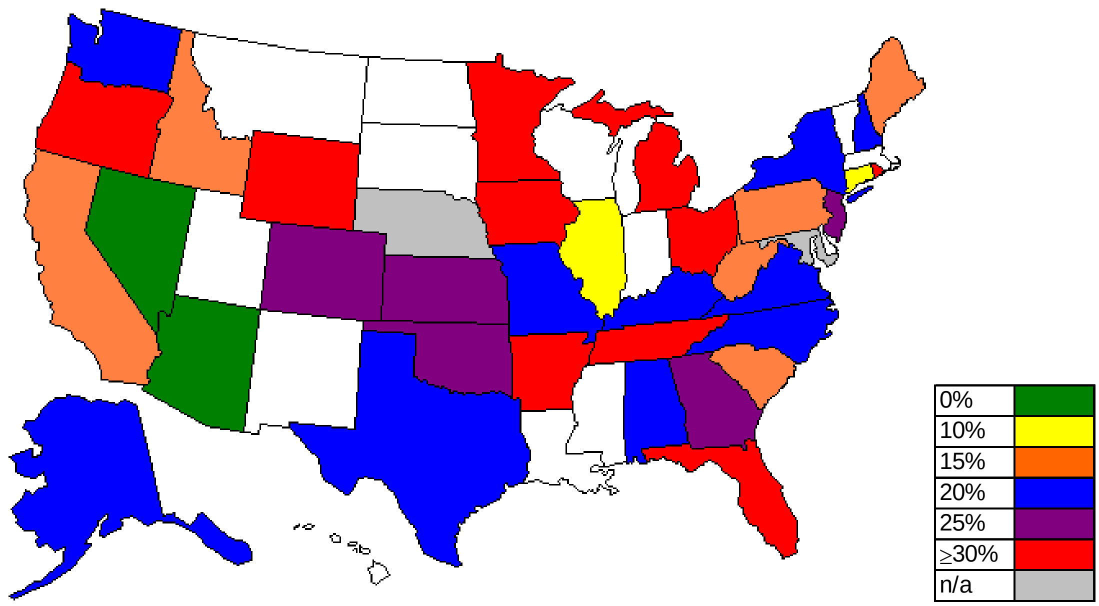


Figure A21 Specified use of RAP in HMA intermediate mixes
(After North Carolina DOT)

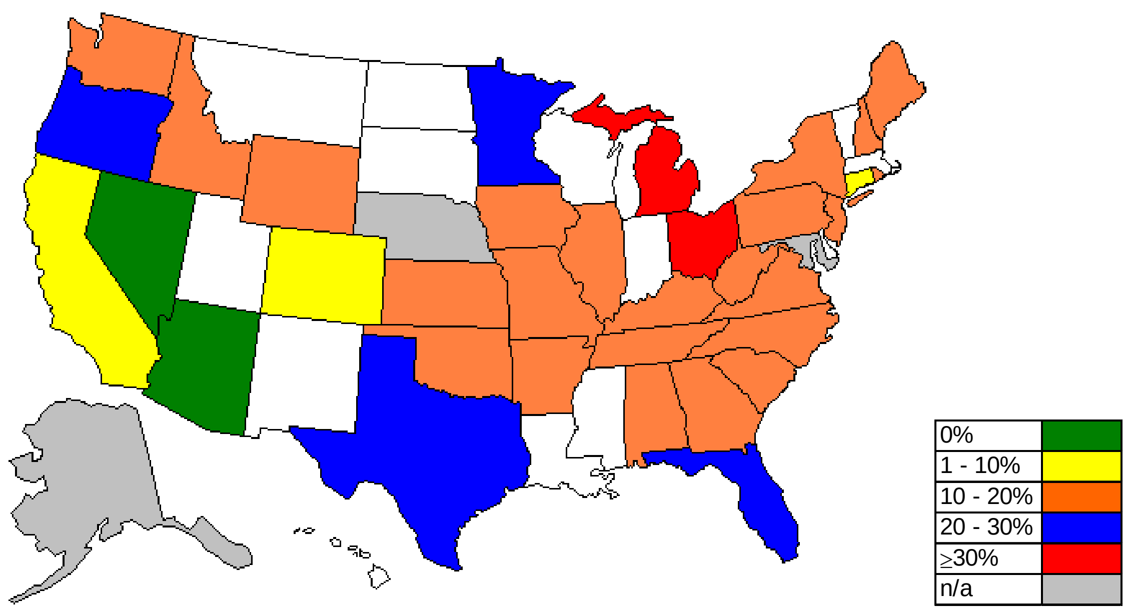


Figure A22 Average use of RAP in HMA intermediate mixes
(After North Carolina DOT)

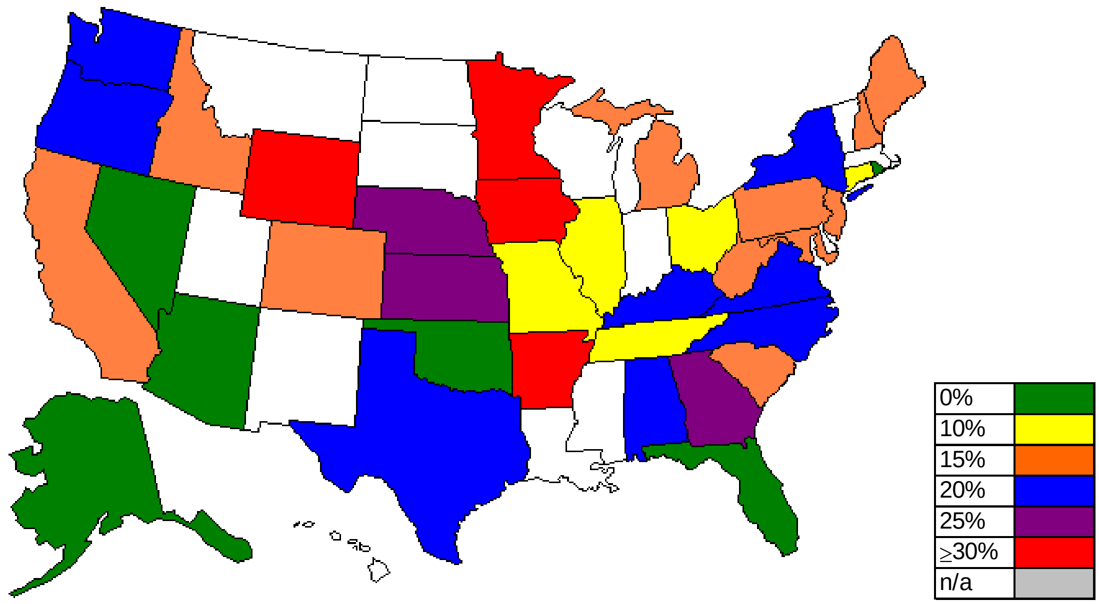


Figure A23 Specified use of RAP in HMA surface mixes (After North Carolina DOT)

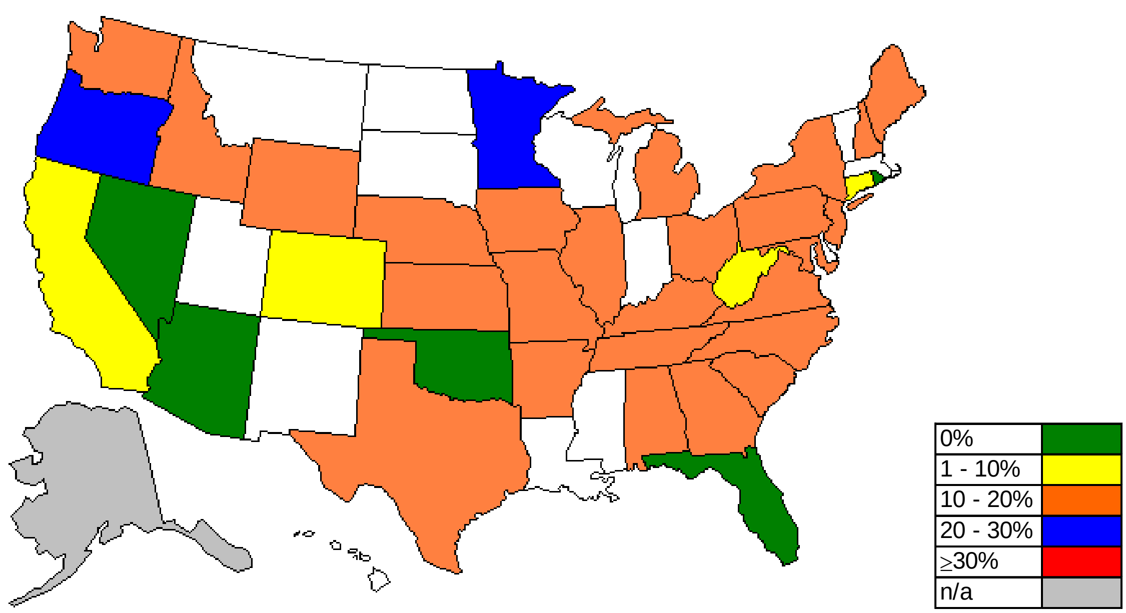


Figure A24 Average use of RAP in HMA surface mixes (After North Carolina DOT)

Based on the review of the data presented in Table A33 and Figures A19 through A24, the following observations can be made.

- Most highway agencies allowed the use of RAP in HMA Mixes.
- Most specifications limit practical use of higher percentages of RAP in HMA mixes.
- Most highway agencies specifications change with the mix type (i.e., dense graded mix, SMA, open graded mix…) and production method (batch plant versus drum mix plant).
- Most highway agencies allow maximum 10-25% of RAP in surface mixes and a higher percentage of RAP in base mixes. However, some agencies restrict the use of RAP in the surface course for pavements with high applied ESAL.
- Some highway agencies require the sources of the RAP materials to be approved prior to their usage in the HMA mix.
- The majority of highway agencies specify maximum size for the RAP material that is greater than the maximum size of the regular HMA mix.
- Some highway agencies restrict or limit the use of RAP to 10% with polymer modified HMA mixtures.
- Most highway agencies require an adjustment to the binder grade when more than 15-20% RAP is used.
- RAP is used with Marshall, Hveem, and Superpave mix design methods.

Figures A25 through A30 show the results of a survey of HMA contractors conducted by the National Asphalt Pavement Association (NAPA) on the extent of RAP usage in the U.S. The results were based on responses from 131 contractors from all over the U.S.

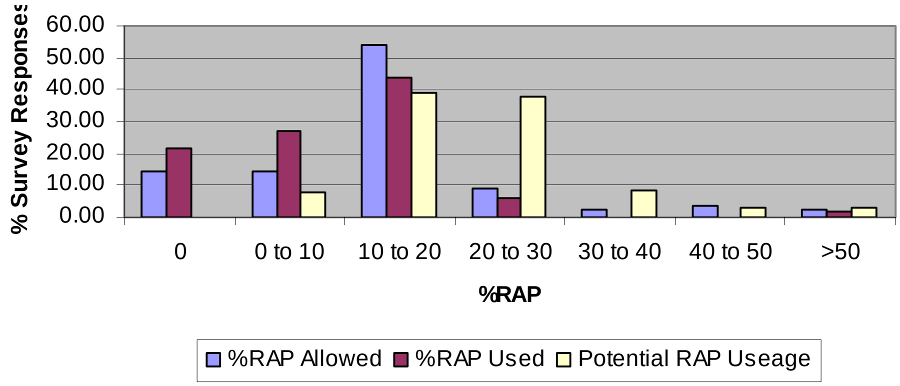


Figure A25 Allowed and usage of RAP in HMA surface mixes
(average allowed = 18, average used = 12, average Potential = 26)

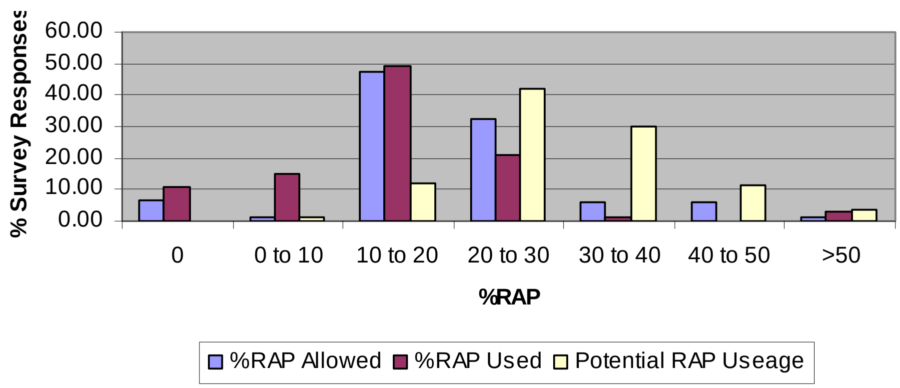


Figure A26 Allowed and usage of RAP in HMA binder mixes
(average allowed = 23, average used = 18, average Potential = 33)

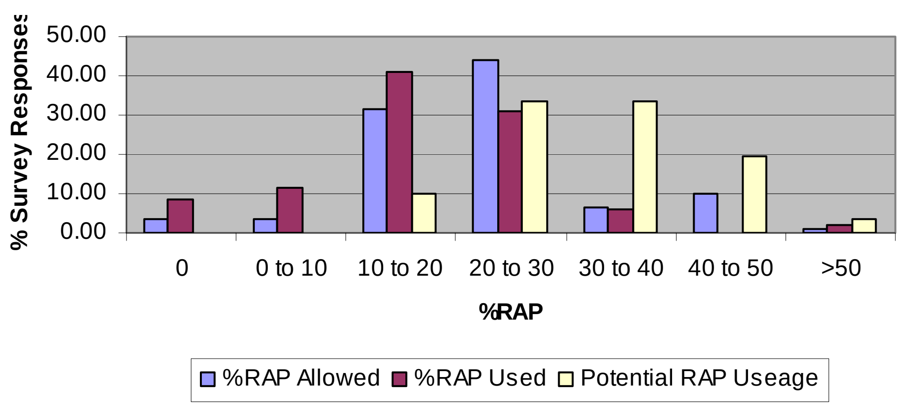


Figure A27 Allowed and usage of RAP in HMA base mixes
(average allowed = 27, average used = 21, average Potential = 36)

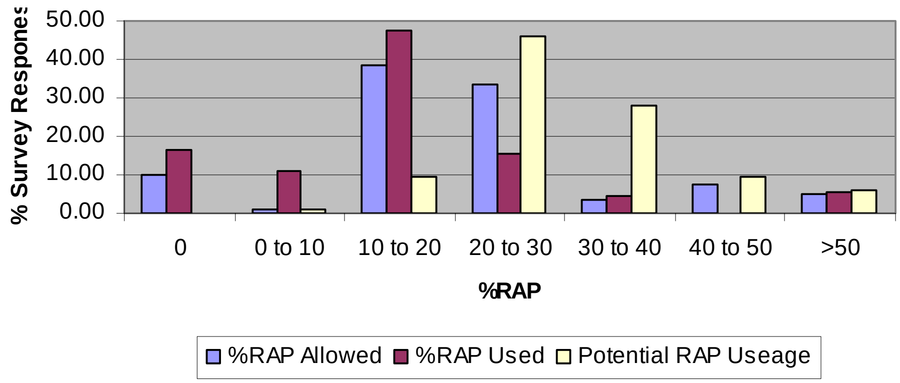


Figure A28 Allowed and usage of RAP in HMA shoulder mixes
(average allowed = 26, average used = 20, average Potential = 36)

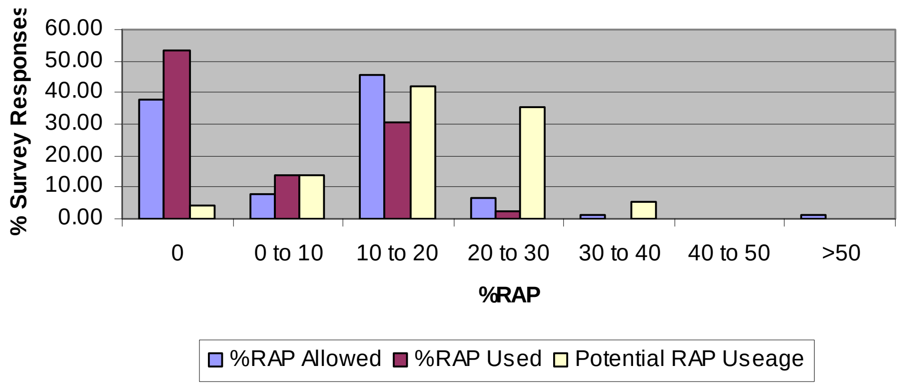


Figure A29 Allowed and usage of RAP in HMA polymer modified mixes
(average allowed = 12, average used = 6, average Potential = 20)

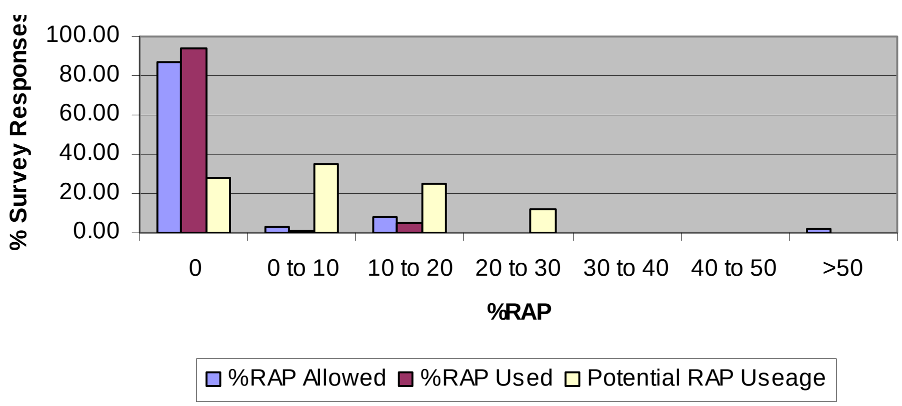


Figure A30 Allowed and usage of RAP in SMA mixes
(average allowed = 3, average used = 1, average Potential = 11)

Since moisture damage and durability of RAP mixtures has been a concern for highway agencies, a review of the State DOTs specifications for moisture sensitivity and durability tests was conducted. It was found that no particular test or specification for moisture resistance other than what is specified for regular HMA mixtures is implemented for mixtures with RAP. The state DOTs specifications for moisture resistance and other laboratory performances tests for HMA mixtures were reviewed and are summarized in Table A34.

Table A34 shows that the AASHTO T283 test is the most widely adopted test for evaluating HMA mixtures resistance to moisture damage. The minimum required tensile strength ratio (TSR) varied among the state agencies and ranged from 70 to 85%. Some highway agencies required in addition to the minimum TSR, a minimum value for the unconditioned tensile strength (TS).

Additionally, Table A34 shows the asphalt pavement analyzer and the Hamburg wheel tests are the most commonly used tests by some state highway agencies for determining rutting resistance of HMA mixtures. The criteria for both tests varied with the traffic level.

Table A34 Summary of States Specifications for Moisture Sensitivity.

State	Moisture Sensitivity Requirement	Other Required Mixture Test
Alabama	80% TSR (AASHTO T283)	NA
Alaska	NA	NA
Arizona	--	--
Arkansas	80% TSR (AASHTO T283)	NA
California	Min Hveem stability of 30 for mix A and 25 for Mix B after moisture vapor susceptibility (Cal Test 307)	
Colorado	70% TSR, CP L-5109 Method B	NA
Connecticut	NA	NA
Delaware	80% TSR (AASHTO T283)	NA
Florida	80% TSR (AASHTO T283) Minimum unconditioned tensile strength of 100psi	
Georgia	80% TSR Min uncond. strength of 60 psi at 55°F (GCT 66) (A tensile splitting ratio >70% may be acceptable so long as all individual test values >100 psi (690 kPa).	rutting on APA after 8000 cycles (49°C) of max 7, 6 & 5mm for level A, level B, level C&D mix designs respectively (GD115)
Hawaii	NA	NA
Idaho	85% Immersion - Compression (AASHTO T 165)	NA
Illinois	75% TSR for 4 inch and 85% TSR for 6 inch (AASHTO T283)	NA
Indiana	80% TSR (AASHTO T283)	NA
Iowa	80% TSR (AASHTO T283)	NA
Kansas	80% TSR (AASHTO T283)	NA
Kentucky	80% TSR according to ASTM D4867 using 150mm samples with 65±5 % saturation	NA
Louisiana	80% TSR for modified asphalt and 75% for unmodified asphalt (AASHTO T283)	NA
Maine	NA	NA
Maryland	85% TSR according to ASTM D4867	NA
Massachusetts	--	--
Michigan	80% TSR (AASHTO T283)	NA
Minnesota	75% TSR (AASHTO T283)	NA
Mississippi	85% TSR - MT-63	NA
Missouri	80% TSR (AASHTO T283)	NA
Montana	--	--
Nebraska	80% TSR (AASHTO T283)	NA
Nevada	70% TSR at 77°F Unconditioned tensile strength min 65psi for PG64-28 NV binder and 100 psi for PG76-22NV binder	Max of 8 mm rut depth after 8,000 cycles at 60°C in APA
New Hampshire	80% TSR (AASHTO T283)	NA
New Jersey	NA	NA
New Mexico	85% Minimum retained strength according to AASHTO 165	NA
New York	80% TSR (AASHTO T283)	NA
North Carolina	85% TSR (AASHTO T283)	Max. rut depth of 11.5 to 4.5 mm respectively for 0.3 to 30 million ESALs at 60°C after 8,000 cycles in APA
North Dakota	NA	NA
Ohio	80% TSR (AASHTO T283)	NA

Table A34 Summary of States Specifications for Moisture Sensitivity (cont'd).

State	Moisture Sensitivity Requirement	Other Required Mixture Test
Oklahoma	Permeability should be less than 12.5×10-5 cm/s	Max of 3-8 mm rut depth depending on traffic level at 64°C after 8,000 cycles in APA
Oregon	80% TSR (AASHTO T283)	Max of 4-6 mm rut depth depending on traffic level at 64°C after 8,000 cycles in APA
Pennsylvania	80% TSR (AASHTO T283)	NA
Rhode Island	NA	NA
South Carolina	80% TSR (SC-T-70) with 60 psi minimum wet strength	Max rut depth of 3mm for Type A and 5 mm for Type B and Type CM at 64°C after 8000 cycles in APA
South Dakota	NA	NA
Tennessee	TSR of 80% and a minimum tensile strength of 100 psi for polymer modified binder and 80psi for non polymer binder	Max. of 0.35 and 0.40 inch rut depths for 10,000 & 5,000 ADT respectively at 147°F after 8,000 cycles in APA
Texas	NA	Max. rut depth of 12.5 mm at 50°C after 20,000, 15,000, and 10,000 cycles respectively for PG76-xx, PG70-xx, PG64-xx under Hamburg wheel test
Utah	--	Max. rut depth of 10 mm after 20,000 cycles in Hamburg wheel test
Vermont	80% TSR (AASHTO T283)	NA
Virginia	80% TSR (AASHTO T283)	Max. rut depth of 3.5, 5.0, and 7.0 mm respectively for more than 10, 3-10, 0-3 million ESALs at 49°C after 8,000 cycles in APA
Washington	NA	NA
West Virginia	NA	NA
Wisconsin	75% TSR (ASTM D4867)	NA
Wyoming	75% TSR (ASTM D4867)	NA

APPENDIX B – DETAILED INFORMATION OF AIRFIELD PROJECTS

Three civilian airports and one military airport were identified as using HMA pavements with RAP. The four airports are Logan International Airport (BOS), Griffin-Spalding County Airport (6A2), Pekin Municipal Airport (C15), and Oceana Naval Air Station (NTU). Three of those airports (BOS, 6A2, NTU) had RAP in the HMA surface course. The C15 airport had a base course with a 100% RAP material. This appendix presents detailed information on the performance of pavements containing RAP at the four airports.

BOSTON-LOGAN INTERNATIONAL AIRPORT, BOSTON, MASSACHUSETTS

Boston-Logan International Airport (BOS) is located 3 miles east of Boston, Massachusetts, and is publicly owned by the Massachusetts Port Authority (Massport). As of October of 2006, Logan Airport has an average of 1120 flights per day, or about 409,000 flights per year, among which 60% are commercial aviation, 32% are air taxi aviation, and 8% are transient general aviation.

The airport is located in the FAA New England Region. The airport has an elevation of 20 feet above sea level. According to the LTPPBind Software the average yearly highest and lowest air temperatures for the airport are 90 and 1°F, respectively. For the airport location, the LTPPBind Software calls for a PG64-28 asphalt binder grade for less than 10 million ESALs application and 98% reliability.

At Logan Airport, the runways, taxiways, and terminal area taxilanes (referred to as alleyways) are constructed of HMA pavements supplemented with Portland cement concrete aprons for aircraft parking at the terminal (Figure B1). The pavements at Logan must support loads up to 873,000 lb for Boeing 747 at the maximum takeoff weight. Tire pressures can be in excess of 200 psi and the traffic is highly channelized.

Figure B1 Logan International Airport, Boston, MA

The pavements at Logan are designed according to the Federal Aviation Administration standards for a 20-year service life, but in reality the HMA pavements that are the most heavily used will only last for 10 to 12 years *(21)*. Prior to the proliferation of wide body aircraft, pavements were rehabilitated primarily for cracking and oxidation due to age. By the early 1990s, with increasing numbers of heavier aircraft, the pavements at Logan constructed with AC-20 binder, the same binder used for highway work, were experiencing rutting and shoving, particularly in the summer months *(21)*.

Since 1995, permanent deformation was mainly resisted by adding modifiers such as Trinidad Lake Asphalt (TLA) and polymers (e.g., SBS) to the asphalt binder. However, with the introduction of modifiers, moisture-induced damage in the HMA pavements started to show up in the form of stripping.

In 2001, Aggregate Industries of Saugus, MA, developed an HMA mix containing RAP for repairs on Taxiway November to combat rutting and stripping (Figure B2). A 1,000 feet section was inlaid with 4” of the RAP containing mix. This portion of Taxiway November handles l00,000 operations annually (14,000 equivalent A330 operations), particularly for hot weather departures from Runways 22R and 22L *(21)*. The original RAP mix used 1” maximum aggregate size gradation, PG64-28 binder, 18% RAP, 4% latex, and 0.5% liquid antistrip and is still performing well today.

This RAP containing mix was used for repairs only from 2001 to 2003. Based on its success for local repairs, the same 15-20% RAP containing mix became Logan’s “everyday” mix in 2004 to the present. The current RAP containing mix uses a 0.75” maximum aggregate size gradation, PG64-28 binder, 4% latex, and 1% lime for antistripping and is performing well. Accelerated loading tests in the laboratory for rutting and moisture induced damage have confirmed that this mix is equivalent to a virgin HMA mix with a PG76-28 binder.

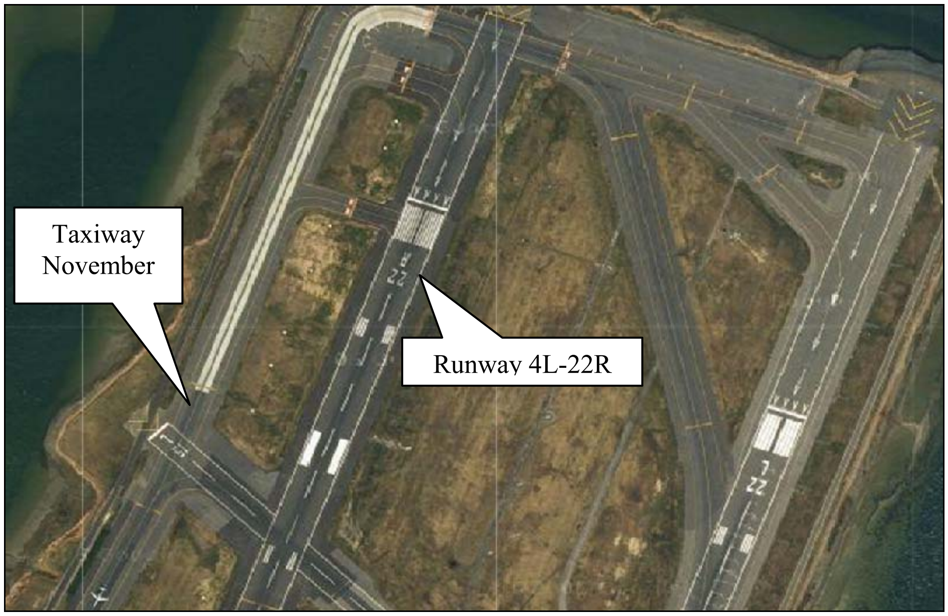


Figure B2 Taxiway November at Logan International Airport, Boston, MA

The Massport P-401 specification requires the RAP to be of a consistent gradation and do not allow the use of RAP obtained from the project site. Table B1 summarizes the mix design requirements for the base and surface course of a P-401 PG64-28 latex modified HMA mix containing RAP.

The following data is from a project in 2006 which reconstructed the 22L end of the Runway 4R-22L (Figure B3) using a RAP containing mix that was designed according to the specifications shown in Table B1.

Table B1 Marshall Mix Design Requirements, Logan Airport.

Property	**Requirement**
RAP content (%)	15-20
Number of blows	75
Hydrated lime (% by total weight of aggregate)	Min 1.0
Air Voids (%)	3.0-5.0
VMA (%)	Min 15
VFA (%)	NA
Marshall stability (lb)	Min 2150
Flow (0.01 inch)	10-14
Optimum Binder content (%)	5.0-7.5
Tensile strength ratio after 5 cycles at 77°F, TSR (%)	92
Sieve Size	**P-401 Spec (3/4” max)**
2 inch	100
1 inch	100
3/4 inch	100
1/2 inch	79-99
3/8 inch	68-88
# 4	46-68
# 8	33-53

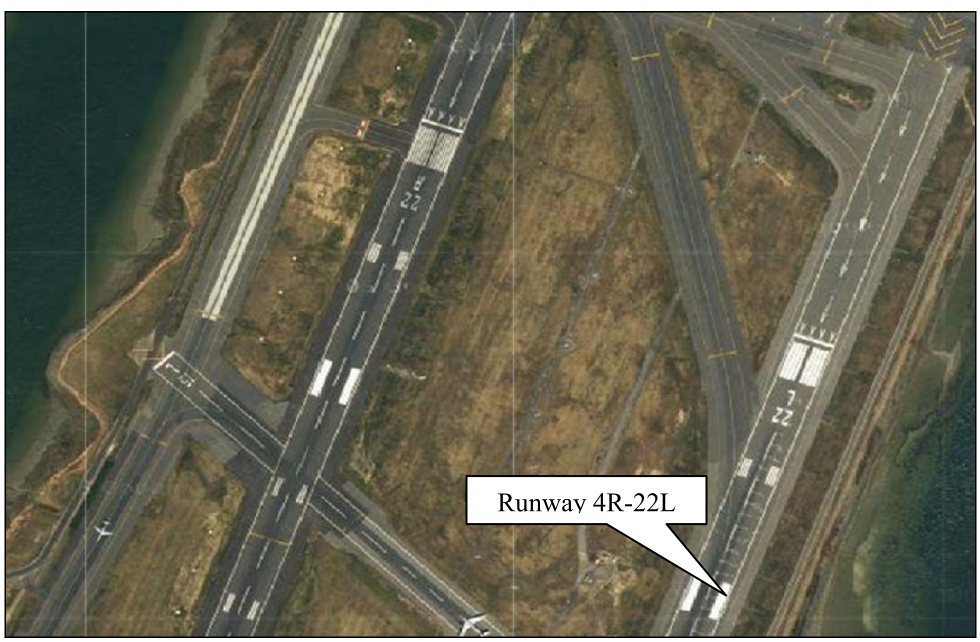


Figure B3 Runway 4R-22L at Logan International Airport, Boston, MA

In 2000, the overall condition of the northern portion of the Runway 4R-22L at Logan Airport was good to very good with PCI values between 64 and 81. The runway had mostly longitudinal, transverse cracking, raveling and weathering distresses. On average, the runway had around 60% of materials related distresses and 25% of load related distresses. The PCI values for 2006 were estimated between 47 and 72. Following this evaluation in 2006, the center 75 feet of the northern portion of the Runway 4R-22L was reconstructed. The last time any major pavement rehabilitation work was performed on the runway was in 1990, when a 5 inch AC20 P-401 overlay with a stress membrane was constructed. Routine crack sealing has been performed on the pavement.

The reconstruction consisted of milling down 12 inches from the center 75 feet of the runway and then placing 12 inches of new pavement. The HMA mix consisted of a 0.75 inch maximum size, 18.5% RAP with a PG64-28 binder modified with 4% latex and 1% lime (Table B2).

The pavement was reconstructed in from August through October of 2006 during six weekends when the runway was closed for 52 hours from Friday night until Sunday afternoon. About 4,000 tons of mix was placed each weekend for a total of around 26,000 tons. No specific problems related to the use of RAP were encountered during construction. The majority of the pavement sections met the in-place density specification.

Table B2 Marshall Mix Design Summary, Logan Airport.

Property	**Value**	**Requirement**
Optimum Binder content (%)	5.2	5.0-7.5
Number of blows	75	75
Hydrated lime (% by total weight of aggr.)	1.0	Min 1.0
Max theoretical specific gravity, Gmm	2.641	--
Bulk specific gravity, Gmb	2.535	--
Air Voids (%)	4.01	3.0-5.0
VMA (%)	17.1	Min 15
VFA (%)	76.5	--
Unit weight (lbs/ft^3)	158	--
Marshall stability (lb)	2340	Min 2150
Flow (0.01 inch)	11	10-14
Tensile strength ratio after 5 cycles at 77°F, TSR (%)	92	92
Aggregate properties		
Aggregate effective specific gravity, Gse	--	--
Sieve Size	**%Passing**	**P-401 Spec (3/4" max)**
2 inch	100	100
1 inch	100	100
3/4 inch	100	100
1/2 inch	94	79-99
3/8 inch	77	68-88
# 4	55	46-68
# 8	38	33-53
# 16	25	20-40
# 30	18	14-30
# 50	13	9-21
# 100	9	6-16
# 200	5	3-6

On September 18, 2007 Dr. Hajj, a member of the UNR research team visited the Logan Airport and conducted a windshield visual inspection of the Runway 4R-22L. It was determined that the runway is in excellent condition with no visible rutting observed. Figure B4 shows the center 75 feet of the runway along with the existing crack-sealed HMA old pavement on both side of the center part. No signs or potential of foreign object damage (FOD) was observed because of the use of RAP in the mix.

Figure B4 Runway 4R-22L at Logan International Airport

GRIFFIN-SPALDING COUNTY AIRPORT, GRIFFIN, GEORGIA

The griffin-Spalding County Airport (6A2) is located in Griffin approximately 40 miles southwest of Atlanta, Georgia. The Griffin-Spalding Airport is open to public use and is jointly owned by the City of Griffin and Spalding County, GA. As of March 2006, the Griffin-Spalding Airport has an average of 55 flights per day, or about 20,000 flights per year, among which half of them are transient general aviation and the rest are local general aviation. A total of 101 aircrafts are based at the airport: 73 single engine airplanes, 17 multi engine airplanes, 4 jet airplanes, 4 helicopters, and 3 ultralights aircrafts.

The airport is located in the FAA Southern Region. The airport has an elevation of 958 feet above sea level. According to the LTPPBind Software the average yearly highest and lowest air temperatures for the airport are 94 and 9°F, respectively.

Figure B5 shows the Runway 14-32 and the Taxiway A at the Griffin-Spalding Airport that were rehabilitated in 1999 and 2000, respectively. The Runway 14-32 is 75 feet wide and 3701 feet long. The runway is the only runway for the airport and has a weight bearing capacity of 26,000 lbs for single wheel aircraft and 30,000 lbs for double wheel aircrafts.

Figure B5 Griffin-Spalding Airport

In 1999, the Superpave designed mix (Table B3) used to rehabilitate the Runway 14-32 and Taxiway A was conducted by Couch Construction using 17% of recycled material from highway pavements in Georgia. The RAP material was tested for gradation and binder content (4.5% by total weight of mix). A typical PG67-22 asphalt binder for the Griffin area was used. The Georgia department of transportation (GDOT) requires that the blend of the virgin asphalt binder with the recovered RAP binder after aging in the thin film oven test meets a viscosity of 6,000 to 16,000 poises (600 to 1600

Pa). The mix was treated with 0.9% hydrated lime by total weight of aggregates (virgin + RAP aggregates). The mix was designed for an initial and design number of gyrations (i.e., N_{ini} and N_{des}) of 7 and 75, respectively. The optimum binder content was 4.8% by total weight of mix for 4% air voids at N_{des}.

Table B3 Superpave Mix Design Summary, Spalding-Griffin Airport.

Property	**Value**	**Requirement**
RAP Content (%)	17.0	--
Optimum Binder content (%)	4.8	--
Hydrated lime (%)	0.9	*
Max theoretical specific gravity, Gmm	2.474	--
Bulk specific gravity, Gmb	2.374	--
%Gmm at Ndes = 75	96	96
VMA (%)	15.1	Min 14
VFA (%)	73.2	65-78
Unconditioned tensile strength (psi) at 55°F	123	60
Conditioned tensile strength (psi) at 55°F	102	--
Tensile strength ratio, TSR (%) at 55°F	82.6	80.0
Aggregate properties		
Aggregate effective specific gravity, Gse	2.662	--
Sieve Size	**%Passing**	**P-401 Spec (3/4" max)**
2 inch	100	100
1 inch	100	100
3/4 inch	100	100
1/2 inch	99	79-99
3/8 inch	85	68-88
# 4	50	46-68
# 8	32	33-53
# 16	24	20-40
# 30	18	14-30
# 50	13	9-21
# 100	8	6-16
# 200	4.7	3-6

* Hydrated lime shall be added at the rate of 1.0% by dry weight of virgin aggregates + 0.5% by dry weight of RAP aggregates.

As part of the GDOT mix design, the diametral tensile strength test of the mix on dry and wet specimens at optimum binder content are conducted. The mix exhibited a dry tensile strength (TS) of 123 psi at 55°F and a tensile strength ratio (TSR) of 82.6%. The mix met the minimum TS of 60 psi and TSR of 80% required by GDOT.

During construction the typical GDOT requirements for regular HMA mixtures were followed and no specific problems due to the use of RAP were reported. Some pavement sections failed to meet the in-place density specifications imposed by FAA.

In 2001, the pavement condition at the Griffin-Spalding County Airport was evaluated by Wilbur Smith Associates in association with Applied Pavement Technology, Inc. (APTech). Pavement conditions were assessed using the Pavement Condition Index (PCI) procedure – the industry standard in aviation for visually assessing the condition of pavements. The types, severities, and amounts of distress present in a pavement are quantified during the pavement survey. This information is then used to develop a composite index (PCI number) that represents the overall condition of the pavement in numerical terms, ranging from 0 (failed) to 100 (excellent) *(22)*.

One year after rehabilitation (2000), the airport Taxiway A was in very good condition with isolated distresses and a calculated PCI value of 97. The distresses include unsealed, low-severity longitudinal and transverse (L&T) cracks, oil spillage, and a low-severity patch.

Two years after rehabilitation (2001), the Runway 14-32 was in very good condition with very little distress and a calculated PCI value of 98. The only distress type noted on the runway was low-severity patching in several areas along the length. The patched areas were relatively large and scattered along the length of the runway. It should be noted that in 1998 (i.e., one year before rehabilitation) the runway had a PCI value of about 70.

Both, the runway and the taxiway did not show any type of load related distresses that are attributed to a structural deficiency in the pavement such as fatigue cracking and rutting. However the distresses that were observed were more of climate or durability type of distresses.

On September 17, 2007 the Griffin-Spalding County airport was visited by Dr. Hajj and visual inspections of the Runway 14-32 and the taxiway A were performed. Overall, it was determined that the runway exhibits moderate cracking at the longitudinal construction joints and moderate transverse cracking over the entire runway (Figure B6). The transverse cracks were approximately 20 to 30 feet apart and did not extend across the entire runway width. Additionally, moderate raveling was observed especially along the longitudinal joints. However, no visible rutting was observed.

On the other hand, Taxiway A overall is exhibiting low severity transverse cracking that are 20 to 30 feet apart (Figure B7). The transverse cracks did not extend across the entire taxiway width. Additionally, low raveling was observed especially along the longitudinal joints. However, no visible rutting was observed.

No signs or potential of foreign object damage (FOD) was observed at both the Runaway 14-32 and Taxiway A because of the use of RAP in the mix.

Figure B6 Transverse cracking along Runway 14-32 at Griffin-Spalding Airport

Figure B7 Transverse cracking along Taxiway A at Griffin-Spalding Airport

PEKIN MUNICIPAL AIRPORT, PEKIN, ILLINOIS

The Pekin Municipal Airport (C15) is located approximately 15 miles south of Peoria, Illinois. The Pekin Municipal Airport is open to public use and owned by the City of Pekin, IL. As of December 2006, the Pekin Municipal Airport has an average of

25 flights per day, or about 10,000 flights per year, among which 44% are transient general aviation, 33% local general aviation, and 22% are air taxi aviation. A total of 36 aircrafts are based at the airport: 33 single engine airplanes and 3 multi engine airplanes.

The airport is located in the FAA Great Lakes Region. The airport has an elevation of 530 feet above sea level. According to the LTPPBind Software the average yearly highest and lowest air temperatures for the airport are 92 and -13°F, respectively.

Figure B8 shows the Runway 9-27 at the Pekin Municipal Airport which is the only runway for the airport and has a weight bearing capacity of 15,000 lbs for single wheel aircraft. The runway is 75 feet wide and 5000 feet long.

Figure B8 Pekin Municipal Airport

The Illinois Division of Aeronautics conducts pavement condition surveys at airports throughout Illinois on a two to three year cycle. The results of the surveys are expressed in terms of the Pavement Condition Index (PCI) value. An overall summary of the Pekin Municipal Airport runway deterioration is shown in Table B4.

Table B4 Runway 9-27 condition Distribution

Survey Date	**PCI Range/Condition**			
	100–86 / Excellent	**85–71 / Very Good**	**70–56 / Good**	**55–41 / Fair**
8/91	100%	0%	0%	0%
6/93	76%	24%	0%	0%
6/95	64%	36%	0%	0%
5/95	0%	0%	76%	24%
8/99	0%	0%	76%	24%

The Runway 9-27 consists of three sections. The first pavement section was constructed in 1963 and had a PCI of 59 in August 1999. The second pavement section was constructed in 1967 and had a PCI of 58 in August 1999. The third pavement section was constructed in 1988 as part of the runway extension and had a PCI of 53 in August 1999.

In 1999, the pavement condition survey reports stated the following: "*Runway 9/27 is in fair condition, with extensive cracking, mostly medium severity. The paving lane joints are cracked, and there are several additional random cracks. Most of the cracks are unsealed, but some have failed sealant in them. A pattern of block cracking is developing in section 3, which is located on the 9 approach. Light vegetation is present throughout, especially in the outer areas*."

In September of 2000, a crack survey was performed to determine the extent and severity of the cracking. The severity, length and approximate location of each crack were noted. The PCI severity ratings for transverse and longitudinal cracks were used to define the crack severity and were reported for the east portion of the runway as follows:

- Number of high severity full width cracks = 31.
- Number of high severity partial width cracks = 16.

- Number of medium severity full width cracks = 18.
- Number of medium severity partial width cracks = 42.
- Total number of cracks = 107.

The total length of the east portion of the runway is 3,775 feet with a total of 107 cracks with about one crack every 36 feet. There was a total of 49 high and medium severity full width cracks or one full width crack every 77 feet. Additionally, there was a total of 31 high severity full width cracks or one high severity crack every 122 feet. It was also noted that the severity of the crack was worse in the outside edges of the runway, where moisture accumulates. The excessive moisture also helped in leaching of the cement from the soil cement base course.

In 2002, the HMA pavement at the runway was reconstructed by milling off the entire existing HMA surface, pulverizing and re-compacting the existing cement treated base (CTB) course, placing and compacting 4-inch of the RAP millings, and then overlaying with a 6 inch of new HMA. The 100% RAP layer was used as an interlayer and a base course between the pulverized CTB course and the new HMA overlay. The RAP millings were crushed and sieved during the design process.

The goal of the reconstruction project of the runway in 2002 was to break down the cement treated base course which became very stiff and was performing like a PCC pavement. The CTB was pulverized and re-compacted in-place to reduce or eliminate the potential reflection of the existing cracks in the CTB layer through the new HMA overlay.

During construction, the contractor was concerned that his current equipment would not be supported by the pulverized cement treated base course. After pulverization this was not an issue. At one location (about 100 feet long) the pulverized soil cement was very unstable and was replaced with the RAP millings.

The RAP millings fulfilled the Pekin Municipal Airport special provision Item AR800237 for bituminous milling base course. The provision calls for a maximum particle size of 1 inch for the RAP millings. Additionally, the amount of the fraction of material passing the No. 200 sieve was not to exceed one half the fraction passing the No. 40 sieve. After spreading, the RAP millings were thoroughly compacted by rolling to not less than 98% maximum density.

On August 28, 2007 Dr, Hajj visited the Pekin Municipal Airport and conducted a visual inspection of the Runway 9-27. Overall, it was determined that the runway is in good condition with low severity transverse cracking over the entire runway (Figure B9a). Low to moderate severity cracks on the longitudinal construction joints were observed (Figure B9b). No visible rutting was observed.

(a) (b)

Figure B9 a) Transverse cracking along runway 9-27 at Pekin Municipal Airport
b) Longitudinal construction joint along runway 9-27 at Pekin Municipal Airport

There have been no maintenance activities since construction (i.e., 5 years ago). In summer of 2003 the runway had a PCI value of 100 with no distresses observed on the pavement surface. In summer of 2007, the runway had a PCI value of 89 with longitudinal and transverse cracking observed. The present condition of the eastern 3,800 feet of Runway 9-27 is mainly due to the cracks at the longitudinal construction joints whereas the cracking in the old pavement was mainly related to cracking in the cement treated base (prior reconstruction, in 1988).

Figure B10 shows the excess RAP millings at the job site. The City of Pekin used some of the excess RAP millings to pave alleyways in the city and the rest was sold out to the local Township Highway Department. The Township Highway Department used the RAP millings on low volume roads, such as alleyways, as a low cost surfacing.

Figure B10 Excess RAP millings at Pekin Municipal Airport

OCEANA NAVAL AIR STATION, VIRGINIA BEACH, VIRGINIA

The Oceana Naval Air Station (NTU), owned and managed by the U.S. Navy, is located in Virginia Beach, Virginia. The airport is located in the FAA Eastern Region. The airport has an elevation of 23 feet above sea level with respectively average yearly highest and lowest air temperatures of 92 and 6°F according to the LTPPBind Software.

The HMA pavement at the Taxiway Alpha has been resurfaced approximately every 8 to 10 years (Figure B11). The last resurfacing job was in 2000 where the middle 32 feet of the taxiway were milled and replaced with a 2.5-inch Navy airfield mix (almost identical to a P-401 HMA surface course) containing 20% RAP (Figure B12). The RAP containing mix consisted of a 1.0 inch maximum aggregate size with a PG70-22 asphalt binder (Table B5).

Figure B11 Taxiway Alpha at the Oceana Naval Air Station

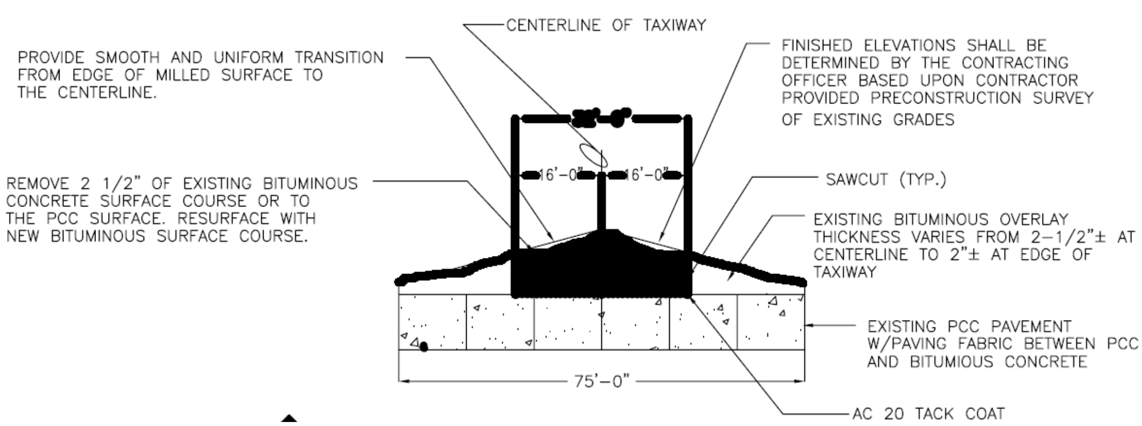


Figure B12 Typical Taxiway Alpha cross-section

Table B5 Marshall Mix Design Summary, Oceana Naval Air Station.

Property	Value	Requirement*
Optimum Binder content (%)	6.6	4-7
Number of blows	75	75
Additive (Adhere HP Plus)	5.0%	--
Air Voids (%)	3.8	3.0-5.0
VMA (%)	17.4	Min 15
VFA (%)	78.2	--
Unit weight (lbs/ft^3)	146.2	--
Marshall stability (lb)	4432	Min 2150
Flow (0.01 inch)	11.2	8-16
Tensile Stregth Ratio, TSR (%)	NA	Min 75
Aggregate properties		
Sieve Size	**%Passing**	**P-401 Spec (3/4" max)**
1 inch	100	100
3/4 inch	100	100
1/2 inch	87	82-96
3/8 inch	83	75-89
# 4	74	59-73
# 8	59	46-60
# 16	45	34-48
# 30	31	24-38
# 50	19	15-27
# 100	9	8-18
# 200	5	3-6

* Navy specifications for the P-401 surface course at the time of construction.

The pavement's daily traffic is equivalent of approximately 200 repetitions of tactical aircraft (F-14 and F-18) and 1 repetition of cargo (C-141 or C-17) aircraft. The tactical aircraft have single tricycle gear geometry with a tire pressure of 240 psi. The C-141 has a dual tandem tricycle gear with a tire pressure of 120 psi.

Before reconstruction in 2000, the pavement consisted of an HMA overlay on top of a PCC pavement with fabric between the PCC and the HMA layer. The pavement exhibited rutting in the wheelpaths at approximate distances of 8 to 14 feet left and right of the centerline. The rutting was generally described as being up to 1.0” over the 6 feet wide travel path of the wheel gear. Other major distresses in the pavement were reflective cracking from the underlying Portland cement concrete pavement. The majority of the cracks exceeding 1/4” width had been sealed as a part of routine maintenance.

In September 2007, Darrell G. Bryan of the Naval Facilities Engineering Command, Atlantic was contacted for the current condition of the HMA pavement with 20% RAP. According to Darrell, after 7 years in-service, the mix at Taxiway Alpha is again exhibiting rutting in the wheel paths (from 0.25 to 0.75 inch depth) and minor reflective cracking from the underlying PCC pavement. He associated the rutting to the constant aircraft traffic with high tire pressures and the asphalt binder grade used, and not specifically to the use of RAP in the mix. No difficulties or issues were encountered during design or construction because of the use of RAP in the HMA mix. Additionally, no signs or potential of foreign object damage (FOD) was observed at the locations where HMA mixtures with RAP were used. During construction, the RAP materials were sampled every 500 tons and tested for aggregate gradation and asphalt binder content.

In general, over the last 10 years, HMA mixtures containing 20-25% RAP have performed well for the Navy in the Mid-Atlantic and Northeast United States, except possibly for taxiway and runway pavements subjected to constant traffic having relatively high tire pressures (according to D. G. Bryan). However, the current Navy policy is to not allow the use of RAP in surface mixes of pavement trafficked by aircraft, as in recent years the consistency of the RAP material has raised some concerns. Consistency concerns include possible contamination with paving fabrics, relatively poor aggregates, and gradation control.

APPENDIX C – MECHANISTIC ANALYSIS

The review of the state highway agencies specifications showed, whenever available, that the same laboratory performance tests and criteria are used for both regular and RAP mixtures. Some agencies implemented either the asphalt pavement analyzer test (APA) or the Hamburg wheel test as part of their mix design method with different performance criteria for different traffic levels.

The performance criteria and the pass/fail values are used to distinguish between a good and a poor HMA mix. Since airport pavements are typically subjected to more severe loading conditions than highway pavements, then the transfer of the performance criteria from highway pavements into airport pavements is a delicate step requiring special analyses.

The significantly higher aircraft tire loads and tire inflation pressures along with different tire configurations impose more complex stress conditions within the structure of airport pavements that are significantly different than those encountered on highway pavements. Hence, pavements response differently to aircraft loading than to highway traffic loading.

In an attempt to study the impact of aircraft loading on the response of HMA pavements a mechanistic analysis was conducted. Several factors affect the prediction of the pavement responses to traffic loading and its long-term performance. The pavement is a layered system and the HMA surface layer exhibits viscoelastic behavior. The loading time and temperature are some of the most important factors that affect the stiffness of the HMA layer. During the past several years, the University of Nevada has developed an advanced pavement response model (3D-Move) which incorporates the effects of viscoelastic properties of asphalt layers and the speed of the moving loads in evaluating pavement responses to traffic loads *(23)*. The model can handle complex surface loadings such as multiple loads and non-uniform and non-circular stress distributions (normal and shear) at the tire-pavement interface. The 3D-Move has undergone field verifications in which responses of two full-scale road tests were used to validate the application of the model *(24)*.

In an effort to establish an equivalency between highway and airport pavements, first the 3D-Move model is used to estimate the responses of typical airport pavements under aircraft loadings and typical highway pavements under truck loading. Then, the estimated responses of the two pavements are used to transfer the technology from highway pavements to airport pavements through adjustments of the applicable specifications. For example, if a RAP specification for highway pavements includes a criterion on the maximum rut depth under the asphalt pavement analyzer (APA), such a specification will be modified to account for the stress conditions encountered in airport pavements relative to those encountered in highway pavements.

Mechanistic Analysis using 3D-Move

The 3D-Move analysis was conducted for a typical airport pavement structure and a typical highway pavement structure. The airport pavement structure was selected from the review of the recent report study titled Comparative Design Study for Airport Pavements *(25)*. The report covered multiple airport pavement structures for different traffic mix and levels and pavement materials properties. For the purpose of this study a medium strength subgrade (CBR of 8), a P-209 subbase (17 inch), a medium thick P-401 stabilized base (8 inch), and a typical 5-inch thick P-401 surface course for a total pavement thickness of 30 inch was considered for analysis.

In the case of the highway, a pavement with a 4-inch HMA layer on top of an 8 inch crushed aggregate base on top of the subgrade (CBR of 8) was selected.

Table C1 summarizes the properties of the pavement structures used in the 3D-MOVE analyses for the selected airport and highway pavements. The frequency-dependent viscoelastic properties of the HMA layer were used to characterize the rate dependent behavior of the HMA mixture under moving loads.

Table C1 Pavement Structures and Material Properties.

Layer	**Thickness (inch)**	**Modulus (psi)**	**Poisson's Ratio**	**Damping Ratio (%)**	**Unit Weight (kN/m^3)**
Airport Pavement					
P401 HMA surface	5	Viscoelastic	0.40	Viscoelastic	23.8
P401Stabilized base	8	400,000	0.40	5	23.8
P209 Crushed aggregate subbase	17	50,000	0.40	5	18.0
Subgrade	240	12,000	0.45	5	17.0
Highway Pavement					
HMA surface	4	Viscoelastic	0.40	Viscoelastic	23.8
Crushed aggregate base	8	30,000	0.40	5	18.0
Subgrade	240	12,000	0.45	5	17.0

Figure C1 shows the dynamic modulus master curve (|E*|) for the HMA mix. The dynamic modulus property is plotted as a function of loading frequency at 64°C. The loading frequency simulates the speed of traffic loads on the pavement. A higher loading frequency represents a fast moving load and a lower loading frequency represents a slow moving load. The 64°C temperature was selected to simulate the environmental condition that is critical to rutting.

In general rutting of HMA pavements can be generated from two sources: (a) the HMA layer and (b) the aggregate base, subbase, and subgrade layers. Since the objective of this investigation is to evaluate the relative performance of HMA mixes, only the rutting in the HMA layer was evaluated.

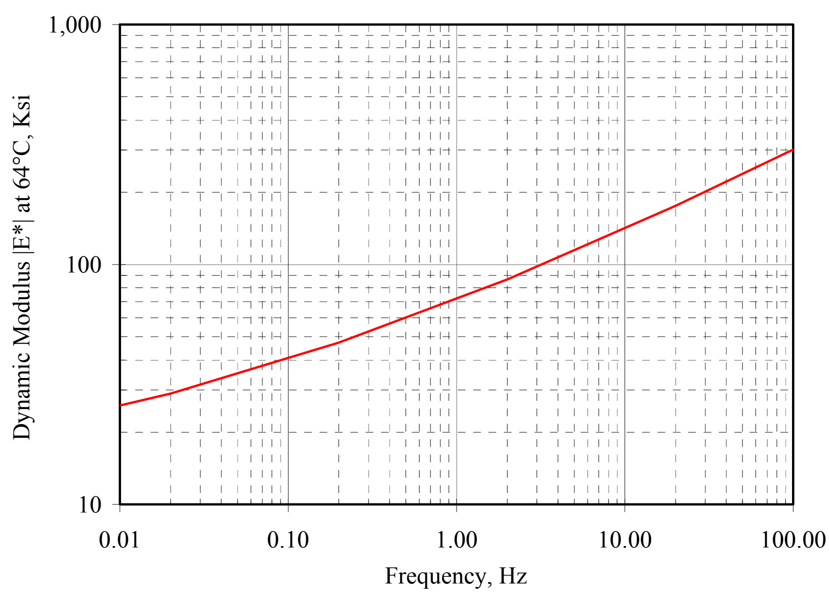


Figure C1 HMA Dynamic Modulus Curve at 64°C.

The rutting model for the same HMA mix was developed in the laboratory using the repeated load triaxial test and is shown in Equation C1.

$$\frac{\varepsilon_p}{\varepsilon_r} = k_1 \times 1.4538 \times 10^{-6} \times N^{0.354428} T^{2.663195} \qquad \text{(Equation C1)}$$

where, ε_p = Axial permanent strain (in/in).
ε_r = Axial resilient strain (in/in).
N = Number of load repetitions.
T = Temperature of the HMA layer (°F).
k_1 = Depth correction function.

The depth correction function k_1 was introduced by the MEPDG and is an empirical attempt based on engineering judgment and very limited field data to adjust the computed plastic strains for the influence of lateral confining pressure at different depths. Equation C2 shows k_1 as a function of the total asphalt layer thickness (h_{AC}, inch) and the depth (depth, inch) to the point of interest within the HMA layer.

$$\mathrm{k}_1 = (C_1 + C_2 \times depth) \times 0.328196^{depth} \qquad \text{(Equation C2a)}$$
$$C_1 = -0.1039 \times h_{AC}^2 + 2.4868 \times h_{AC} - 17.342 \qquad \text{(Equation C2b)}$$
$$C_2 = 0.0172 \times h_{AC}^2 - 1.7331 \times h_{AC} + 27.428 \qquad \text{(Equation C2c)}$$

It should be mentioned that the developed rutting model is a statistical relationship based on the laboratory analysis of the asphalt mixture and therefore shift/adjustment factors are required to provide reasonable estimates of the permanent deformation in the field. Since the shift factors are outside the scope of this research, the national field calibration factors determined in the MEPDG were used to adjust the developed laboratory model for field predictions (Equation C3).

$$\frac{\varepsilon_p}{\varepsilon_r} = k_1 \times 0.74 \times 10^{-6} \times N^{0.42531} T^{2.39688} \qquad \text{(Equation C3)}$$

The rut depth in the HMA layer can be calculated by multiplying the permanent strain (ε_p) by the corresponding thickness of the HMA layer.

In the case of airport, the mechanistic analysis was conducted for a fully loaded Boeing 727 airplane during normal taxiing at an average speed of 17 mph (i.e., 15 knots) and during braking at a speed of 2 mph (i.e., 1.8 knots). Braking causes the aircraft to decelerate and friction forces are generated between the tires and the surface of the pavement. A maximum design taxi weight of 210,000 lb for the Boeing 727 was considered. Around 93% (195,200 lb) of the total load is carried by the main gear dual wheel system. A tire pressure of 173 psi with an elliptical tire print of 24 inch length by 15 inch width was used in the analysis with a braking friction coefficient of 0.8 at the tire-pavement interface.

In the case of highway pavement, the mechanistic analysis was conducted for a fully loaded 18-wheel tractor-semitrailer during normal highway traffic and at intersections (i.e., braking). The 18-wheel truck is the most commonly used commercial truck on highway pavements. Braking at intersections causes the vehicle to decelerate and the loads to transfer to the front of the vehicle. The resulting axle load can be higher or lower than the initial static load, depending on the axle location. Two vehicle speeds were considered: 40 mph (away from intersection) and 2 mph (at intersection). The braking forces at intersection were incorporated in the analysis as interface shear stresses.

The pavement properties along with the characteristics of the Boeing 727 and the 18-wheel truck were used in the 3D-MOVE model to predict the dynamic resilient responses of the various pavements. Table C2 summarizes the maximum resilient vertical strains developed at the middle of the various HMA layers. The dynamic resilient responses determined from the 3D-MOVE analysis were used to estimate the number of repetitions for 0.5 inch rut depth as well as the rut depth in the HMA layer for 50,000 load repetitions (Table C2).

The data in Table C2 show that the maximum rut depth in the HMA layer at an airfield pavement during braking is as much as 1.6 times the response of the same HMA under normal taxiing traffic. In other words, the braking of the Boeing 727 combination at slow speed is expected to result in permanent deformation in the HMA layer that is 1.6 times higher than under the aircraft moving at 17 mph and without braking.

Table C2 3D-Move Pavement Analysis Results.

Pavement	HMA Thickness (inch)	Max Resilient Vertical Strain, ε_r (microns)	Number of Repetitions for 0.5 inch Rut Depth	Axial Permanent strain for a 50,000 load repetitions (%)	Rut Depth for a 50,000 load repetitions (inch)
Airfield at taxiway (17 mph)	5	3,209	60,000	9.3	0.46
Airfield during braking (2 mph)	5	5,237	19,000	15.2	0.76
Highway at normal speed (40 mph)	4	1,104	560,000	4.5	0.18
Highway at intersection (2 mph)	4	2,694	69,000	10.9	0.44

The comparison of the responses in the airfield and highway pavements reveals for a 50,000 load repetition that the rut depth at the taxiway is about 2.5 times higher than the rut depth at normal highway speed. On the other hand, the braking at the airfield pavement resulted in a HMA rut depth 1.7 times higher than that at intersection under the braking of 18-wheel truck.

The data in Table C2 can be used to identify the performance criteria for HMA mixtures on airfield pavements under a specific laboratory performance test. The proposed concept is explained in the following two examples for the third scale Model Mobile Load Simulator (MMLS3) test and the asphalt pavement analyzer (APA) test.

Third scale Model Mobile Load Simulator (MMLS3)

The MMLS3 test is typically performed on pavements in the field or test slabs prepared in the laboratory. A new MMLS3 procedure was developed to allow testing of cylindrical laboratory specimens compacted to specific densities or asphalt cores retrieved from the field. This allows a rapid assessment of the permanent deformation and moisture sensitivity of these specimens. Testing may be done at elevated temperatures both wet and dry. The specimens in the MMLS3 wheel trafficking tests are tested with a pneumatic tire inflated to 690 kPa (100 psi) running in one direction over the specimens at significantly high loading rates and applied forces.

Studies are underway to formalize criteria to assess the rutting and moisture susceptibility of mixes tested using the MMLS3 in the laboratory. Interim criteria were established by South African researchers for acceptable rutting performance at critical temperature (>50°C) and after 7200 load applications per hour.

- For roads and highways: maximum 3.0 mm after MMLS3 load applications
- For airfields: maximum 1.8 mm after MMLS3 load applications.

The following calculations show how the South Africa Criteria for MMLS3 compare to the criteria developed by the approach proposed in this research. Since the 3.0 mm criterion is developed for normal highway traffic then the estimated rut depths under normal highway and taxiing loads will be used instead of the rut depths during braking. This research proposes the use of the following relationship to convert highway criterion to airfield criterion.

$$\text{Airfield criterion} = \text{Highway criterion} \times \left(1 - \frac{\text{Highway Rut Depth}}{\text{Airfield Rut Depth}}\right)$$

$$= 3.0 \times \left(1 - \frac{0.18}{0.46}\right) = 1.83 \text{ mm.}$$

The 1.83 mm criterion developed in this research is very close to the 1.8 mm criterion recommended by the South Africa research, thus proving the validity of the proposed conversion technique.

If the estimated rut depths during braking were to be used then the criterion for the MMLS3 at the slow moving braking areas would be:

$$\text{Airfield criterion} = \text{Highway criterion} \times \left(1 - \frac{\text{Highway Rut Depth}}{\text{Airfield Rut Depth}}\right)$$

$$= 3.0 \times \left(1 - \frac{0.44}{0.76}\right) = 1.26 \text{ mm.}$$

Asphalt Pavement Analyzer (APA)

Depending on traffic level, the Oregon DOT specifies a maximum APA rut depth at 64°C of 4 to 6 mm after 8,000. Applying the relationship proposed in this research, the criterion at the airfield will have to be dropped from 4.0 mm to 2.5 mm at 64°C:

$$\text{Airfield criterion} = \text{Highway criterion} \times \left(1 - \frac{\text{Highway Rut Depth}}{\text{Airfield Rut Depth}}\right)$$

$$= 4.0 \times \left(1 - \frac{0.18}{0.46}\right) = 2.5 \text{ mm}$$

Summary

The mechanistic analysis for the airport and highway pavements showed promising results in converting highway performance criteria to airport applications. It should be noted that the mechanistic analysis results are highly dependent on the pavement structure, material properties, temperature of analysis, and the applied loads. For example Georgia DOT has different APA criteria at 49°C for their different standard mix levels. Therefore, adjusting the GDOT criteria to airfield pavements requires the mechanistic analysis to be run at 49°C for the GDOT different materials characteristics.

Additionally the recommended performance criteria from the mechanistic analysis need to be validated in the field before full implementation.

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