RUBBER MODIFIED AND PERFORMANCE BASED ASPHALT BINDER PAVEMENTS

I- 5
Nisqually River to Gravelly Lake
Contract 4250

Post Construction Report
Federal Aid No. IM-005-3(802)

by

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Pavement Design Engineer

Prepared for
Washington State Department of Transportation
and in cooperation with
US Department of Transportation
Federal Highway Administration
January 1996

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO. WA-RD 407.1	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
4. TITLE AND SUBTITLE Rubber Modified and Performance Based Asphalt Binder Pavements, I-5 Nisqually River to Gravelly Lake		5. REPORT DATE January 1996 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Linda M. Pierce		8. PERFORMING ORGANIZATION REPORT NO. WA93-01A
9. PERFORMING ORGANIZATION NAME AND ADDRESS Washington State Department of Transportation Materials Laboratory PO Box 47365 Tumwater, Washington 98504-7365		10. WORK UNIT NO. 11. CONTRACT OR GRANT NO.
12. SPONSORING AGENCY NAME AND ADDRESS Washington State Department of Transportation Transportation Building Olympia, Washington 98504-7370		13. TYPE OF REPORT AND PERIOD COVERED Post Construction 14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

This report describes the construction of asphalt pavements made with three types of asphalt binders. The three types of binders were PBA-6, PBA-6GR (ground rubber), and AR4000W. The two modified binders, PBA-6 and PBA-6GR, are being evaluated to determine their resistance to rutting as compared to the conventional binder, AR4000W.

Construction was not without problems. Cyclic segregation, defined as repetitive areas of asphalt mix segregation that occur at approximately every truck load, was a major problem on the northbound lanes. A material transfer device was used on the southbound lanes which minimized the cyclic segregation effect.

At the time of this report (approximately two years and 1.0 to 2.0 million ESALs), there is no data that indicate which of the three asphalt binder types will have better performance. The project will be monitored over the next five to ten years to identify if any added performance is gained in any one of the mixes.

17. KEY WORDS rubber asphalt pavement, performance based asphalt pavement, cyclic segregation	18. DISTRIBUTION STATEMENT No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None	43	

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Washington State Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

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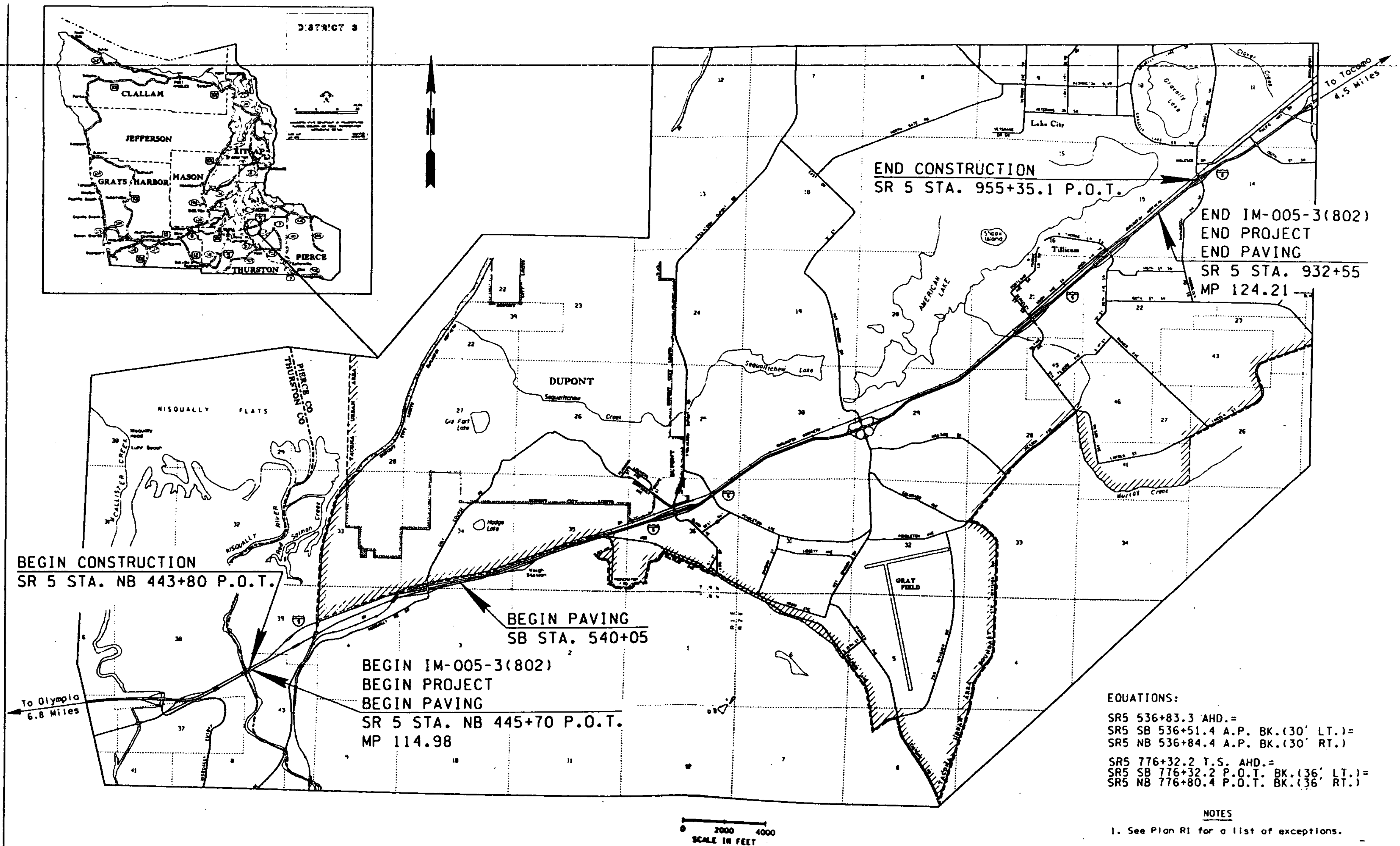

EQUATIONS:

SR5 536+83.3 AHD.=
SR5 SB 536+51.4 A.P. BK.(30' LT.)=
SR5 NB 536+84.4 A.P. BK.(30' RT.)

SR5 776+32.2 T.S. AHD.=
SR5 SB 776+32.2 P.O.T. BK.(36' LT.)=
SR5 NB 776+80.4 P.O.T. BK.(36' RT.)

NOTES

1. See Plan R1 for a list of exceptions.

DESIGNED BY	L. SCHMITT/G. LYDON	12-21-92			
ENTERED BY	A. LEINGANG				
CHECKED BY	H. DIEP/J. VESPER				
PROJ. ENGR.	D. HILDERBRANT				
DIST. ADM.	G. DEMICH				
		DATE	DATE	REVISION	BY

REGION NO.	STATE	FED. AID PROJ. NO.
10	WASH	IM-005-3(802)
JOB NUMBER 93W022		
CONTRACT NO.		

PROGRAM DEVELOPMENT DIVISION

Washington State Department of Transportation

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SR 5
NISQUALLY RIVER TO GRAVELLY LAKE I/C

VICINITY MAP

NISQUALLY RIVER BR. TO GRAVELLY LAKE I/C
PIERCE CO.
SHEET OF SHEETS

OBJECTIVE

The primary objective of this experimental feature is to evaluate the performance of two performance based asphalt (PBA) binders against the performance of the Washington State Department of Transportation (WSDOT) conventional AR4000W asphalt binder. For this particular project, the main concern of the pavement Engineers was to develop an asphalt mix which would be resistant to rutting. Since this section of I-5 is structurally sufficient, fatigue cracking is not an issue.

INTRODUCTION

The performance based asphalt binders where developed by the West Coast User/Producer Group which is composed of State Highway Agencies, the Federal Highway Administration, and asphalt producers and suppliers. The PBA grading system is performance driven and utilizes both conventional and modified binders. This grading system is based on selecting an asphalt binder based on climatic and environmental considerations, not on asphalt properties.

The three types of asphalt binder to be evaluated are PBA-6, PBA-6GR (ground rubber) and AR4000W. The PBA-6 was developed for hot/very cold climates (above 90°F and below -20°F), which this portion of the state generally does not experience, but was felt by the pavement engineers to be an ideal opportunity to evaluate this products ability to resist rutting. In addition, since the film thickness of the PBA-6 binder is greater than that of the conventional AR4000W binder, it is generally understood that a thicker film would help to minimize the effects of aging.

There are essentially two different process by which ground rubber can be added to the asphalt concrete mix: the wet process and the dry process. The wet process combines very fine (80 to 200 mesh) ground rubber with the liquid asphalt cement at the suppliers plant. The product is delivered to the contractor by tanker and put through the asphalt plant using the same procedures as a conventional asphalt cement. The dry process involves adding the ground rubber as either a mineral filler or as an aggregate substitute in the hot mix.

For this experimental project, the wet process was selected for incorporating the ground rubber into the asphalt mix. The PBA-6GR product was blended to meet the PBA-6 specifications, excluding the requirements for ductility. In addition, the PBA-6GR was developed in anticipation of the federal mandate requiring the use of recycled tires in asphalt concrete pavements (see Appendix A for detailed information on all three mix properties).

STUDY SITE

The section of roadway to be overlaid is located approximately ten miles north of Olympia on Interstate 5. The overlay paving limits for the northbound direction are from MP 114.98 to MP 124.21 and for the southbound direction are from MP 116.77 to MP 124.21. This portion of Interstate 5 is located in Pierce County and is classified as a rural interstate for the first four miles (south end) and an urban interstate for the last six miles (north end) of the project. Average daily traffic ranges from approximately 42,000 on the south end of the project to approximately 60,000 on the north end of the project. As of 1994, the truck percentages ranged from 9 to 13 percent, with an associated 18,000 lb equivalent single axle load (ESAL) of 39,000,000 to 43,000,000 over the next 15 years.

This equates to approximately 1,800,000 to 2,000,000 ESAL's per year.

The roadway consists of three twelve foot wide lanes with a four foot left shoulder and a ten foot right shoulder, in both directions. The existing roadway consist of 0.50 ft to 1.00 ft ACP over 0.50 ft CTB, or 0.25 ft to 0.50 ft CSTC, or 0.80 ft ATB. The majority of the existing roadway was last rehabilitated in 1980 with 0.06 ft ACP Class D.

A pavement survey of the existing roadway indicated scattered areas of longitudinal and transverse cracking, patching, low severity alligator cracking and wheel path wear of 6 to 10 mm in depth. The main distress present on this section of roadway was wheel path wear due to studded tires.

CONSTRUCTION SUMMARY

Woodworth and Company of Tacoma, Washington was awarded the contract on June 21, 1993, with work commencing on August 2, 1993 with the installation of construction signing. The PBA-6 mix designs were developed using two different brands of liquid asphalt. The PBA-6 binder was supplied by Chevron, while the PBA-6GR was supplied by US Oil.

This project involved overlaying the existing roadway with 0.15 ft of Class A asphalt concrete pavement. The northbound lanes were paved in 1993 and the southbound lanes were paved in 1994. The three asphalt binders (AR4000W, PBA-6 and PBA-6GR) were utilized in the asphalt overlay in both directions. Refer to Figure 1 for detailed listing of binder locations and Table 1 for placement data.

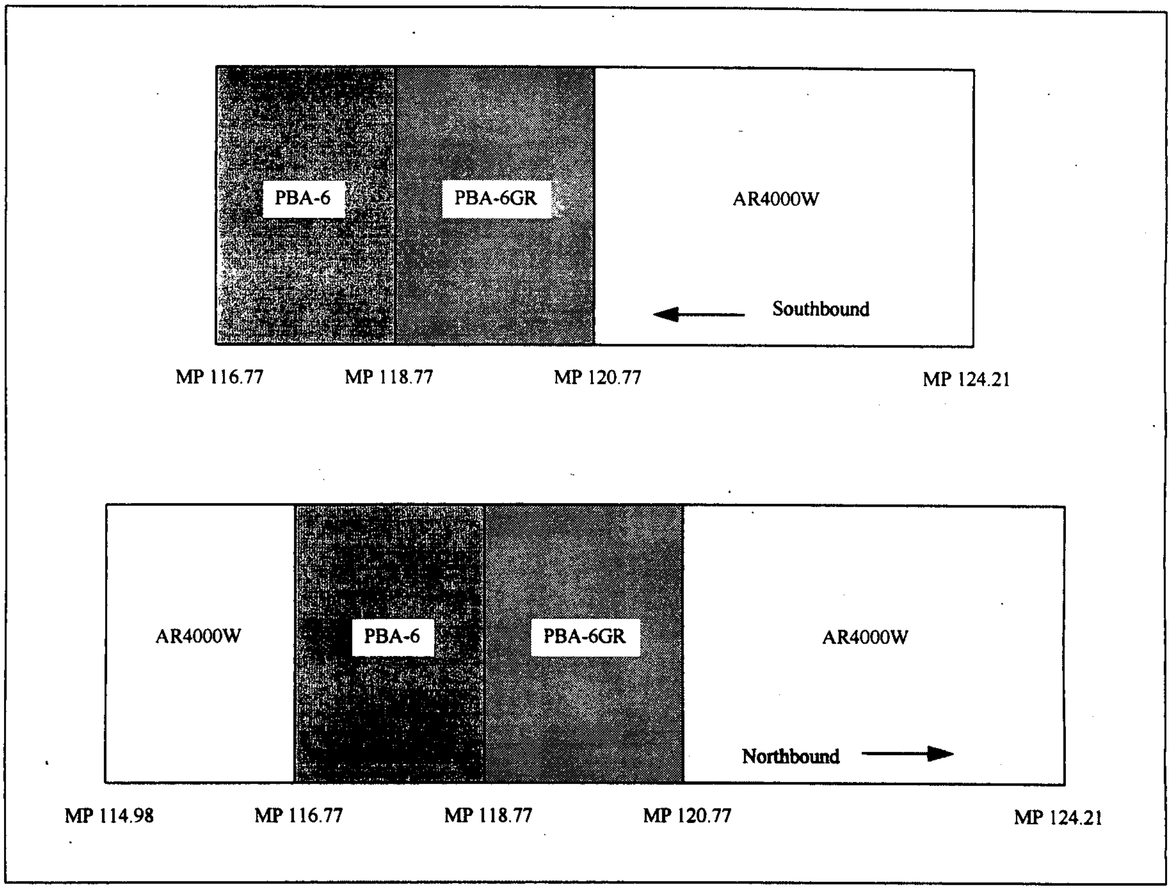


Figure 1. AR4000W, PBA-6, and PBA-6GR Binder Locations.

Table 1. Data on Mix Placement

	AR4000W	**PBA-6**	**PBA-6GR**
ACP Tons Placed	38,460	9,070	9,100
Asphalt Content (%)	5.0	4.9	5.4
Mixing Temperature (°F)	*	330	340
Lay down Temperature (°F)	*	300	300
Ambient Air Temperature (°F)+			
1993 Paving	*	40-56	42-52
1994 Paving	*	58-82	53-71

* Indicates that data is not available
+ Standard Specifications require surface temperature of 45°F.

Due to traffic considerations, paving operations, for all three mixes, occurred at night.

Therefore, ambient air temperatures became a factor in the operation. The ambient air restriction was left to the discretion of the Engineer.

Compaction was accomplished by using a 10 ton vibratory roller.

The unit bid price for each of the three mixes is listed in Table 2.

Table 2. Contract Bid Item Price

Item	Cost (per ton)
AR4000W	$27.00
PBA-6	$30.00
PBA-6GR	$36.00

ASPHALT PLACEMENT

A few problems arose during the paving operation. One problem was cyclic segregation of the asphalt concrete pavement. Cyclic segregation is defined as repetitive areas of asphalt mix segregation (or at least the appearance of segregation) that occur at approximately every truck load. Cyclic segregation was found in all mixes through most of the northbound lanes. Several theories have been examined to explain this phenomenon (i.e., low oil content, night paving which results in cooling of the asphalt, non continuous paving operation (paver stopping and starting), etc.). The actual cause of the segregation is unknown at this time, but this project will be investigated as part of a separate WSDOT research project concerning cyclic segregation. A material transfer device was used on the southbound lanes which minimized the cyclic segregation affect. The transfer device essentially stores and mixes up to 30 tons of mix and continuously feeds directly into the paving machine hopper. One of the benefits of the transfer device is minimizing the stopping and starting of the paving machine.

A second problem that occurred was the PBA-6 mix sticking to the rollers. To correct this problem water was applied to the rollers.

EVALUATION OF THE RUTTING POTENTIAL

A study was initiated to determine if one of the three mixes was more resist to rutting. Pavement cores were taken from each of the three binder sections, tested and evaluated to predict the rutting potential. Pavement cores were taken in 1993 and in 1994. The 1993 cores were sent to Applied Paving Technology, Inc. and the 1994 cores were sent to the Asphalt Institute. In each case the analysis for rutting potential was determined using the Superpave Shear Tester (SST). The SST can be used to evaluate and predict the amount of deformation (rutting) and fatigue cracking of the asphalt concrete mix. See the following tables for laboratory results.

Applied Paving Technology evaluated pavement cores taken from the 1993 paving of the northbound lanes. These cores were taken approximately 6 months after placement. As noted in the test report (see Appendix C) the above analysis does not consider the effect that as the mixes densifies, shear resistance increases with time and the binder ages with time, which also would contribute to an increase in rut resistance. Therefore, the time to a 0.4 inch rut shown in Table 3 is a worst case scenario which does not include mix densification or aging.

In 1994, after completing the southbound paving, pavement cores were sent to the Asphalt Institute for similar evaluation of the rutting potential. Results of this analysis are shown in Table 4. As stated in the Asphalt Institutes Report (see Appendix C) the

Table 3. Applied Paving Technology Evaluation for Rutting.

Grade	In Place Air Voids	ESALs to Achieve a 0.4" Rut Depth	# Years to Achieve 0.4" Rut Depth
AR4000W	7.1	12,300,000	8.5
PBA-6	5.5	23,400,000	16.1
PBA-6	6.5	13,400,000	9.2
PBA-6GR	8.9	326,000	0.2
PBA-6GR	8.8	792,000	0.5

most likely cause of the unrealistic rutting performance prediction is the use of a model (equation) which is not specific to the materials used in Washington state and that the testing temperature was probably 2° to 3° higher than what would normally be used to analysis the material.

Table 4. Asphalt Institute Evaluation for Rutting.

Grade	In Place Air Voids	ESALs to Achieve a 0.5" Rut Depth	# Years to Achieve 0.5" Rut Depth
AR4000W	6.6	661,347	0.8
AR4000W	6.6	601,419	0.7
AR4000W	6.8	540,020	0.6
AR4000W	6.8	719,997	0.9
PBA-6	4.7	784,638	0.9
PBA-6	7.0	861,954	1.0
PBA-6	7.0	335,190	0.4
PBA-6GR	6.8	646,492	0.8
PBA-6GR	7.1	646,492	0.8

Unfortunately, the results of these studies appear to be ambiguous and are not relevant to the observed conditions. The Applied Paving Technologies analysis indicated less than a years performance for the PBA-6GR mix to reach a rutting depth of 3/4 inch. The

Asphalt Institutes analysis indicated less than a year performance (rutting depth of 1/2 inch) for all three mixes. In 1995, using the South Dakota Profilometer, no significant amount of rutting (less than 1/16 inch) is occurring in any of the mixes. There appears to be a need for further research in correlating rut predication tests with actual field measurements.

CONCLUSION

At the time of this report (approximately 2 years and 1.0 to 2.0 million ESALs) there is no information that would indicate which of these materials will have better performance since all three mixes are performing well and performing equally. Based on a study conducted by Caltrans[1], which concluded that the rubber modified mix is much less resistant to rutting than the dense graded mix. In addition, it was concluded from this study that the rubber modified mix has much better fatigue performance than the dense graded mix. Even though the PBA-6GR material is not gap-graded, it may be more susceptible to rutting than the AR4000W or PBA-6 mixes. The PBA-6GR may also perform better under fatigue, but since this roadway section is of adequate thickness, improved performance in fatigue may not be validated.

This project will be evaluated over the next 5 to 10 years to identify if any added performance is gained in any one of the mixes. Continued use of the Superpave Shear Tester and correlation to field testing will be further evaluated and modified as the state continues to evaluate the SHRP testing procedures for both asphalt binders and asphalt

[1] Harvey, J. and C. L. Monismith, "Accelerated Pavement Testing Phase I - Laboratory Evaluation of Dense-Graded Asphalt Concrete (DGAC) and Asphalt-Rubber Hot Mix Gap-Graded (ARHM-GG)", January 31, 1994.

mixes.

A side benefit of the future evaluation is the comparison of how the different asphalt binders will react to the affects of the cyclic segregation. These areas have higher voids, which previous experience and studies suggest will result in early deterioration. How well each asphalt binder performs will be compared to the sections placed with the transfer device as a "control".

Appendix A
Material Specifications

Table A-1. Asphalt Concrete Pavement Class A Specifications (WSDOT 1994 Standard Specifications for Road, Bridge, and Municipal Construction, Section 9-02.1(4))

Characteristics	WSDOT Test Method	AR-4000W
Tests on Residue from RTFC[2]	*208*	
Absolute Viscosity at 140°F, poise	203	2500 - 5000
Kinematic Viscosity at 275°F cSt, min.	202	275
Penetration at 77°F 100g/5 sec, min	201	40
Percent of original penetration at 77°F min	[3]	45
Ductility at 45°F (1 cm/min)	213	10
Test on Original Asphalt		
Flashpoint (Cleveland Open Cup) °F min	206	440
Solubility in Trichloroethylene, % min	214	99.0

[2] TFO may be used but RTFC shall be the referee method.
[3] Original penetration as well as penetration after RTFC loss will be determined by WSDOT Test Method 201.

Table A-2. PBA-6 Binder Specification

Characteristic	AASHTO Test Method	Specification Requirements
Penetration (4°C 200 g, 60 sec), dmm RTFO Aged Residue (Note 1)	T-49	30+
Penetration (25°C, 100 g, 5 sec), dmm RTFO Aged Residue	T-49	...
Ductility (7.2°C, 1 cm/min), cm RTFO Aged Residue (Note 2)	T-51	...
Absolute Viscosity (60°C, P) (Note 3)		
Original Binder	T-202	2000 +
RTFO Aged Residue	T-202	5000 +
Kinematic Viscosity (135°C), cSt		
Original Binder	T-201	2000 -
RTFO Aged Residue	T-201	275 +
Absolute Viscosity Ratio (60°C) RTFO Aged Residue		4.0-
Penetration (25°C, 100 g, 5 sec) dmm Tilt Oven Residue (Note 4)	T-49	Report
Ductility (25°C, 5 cm/min) cm Tilt Oven Residue	T-51	Report
Absolute Viscosity (60°C), Poise Tilt Oven Residue	T-202	Report
Flash Point, Cleveland Open Cup, °C Original Binder	T-48	232 +
Mass Loss after RTFO Test, %	T-240	Report
Solubility in Trichloroethylene, % Original Binder	T-44	Report
Ductility (25°C, 5 cm/min), cm RTFO Aged Residue	T-51	60 +

Ground rubber shall not be used in the formulation of PBA-6.

Note 1 "RTFO Aged Residue" means the asphalt residue obtained using the Rolling Thin-Film Oven Test (RTFO Test), AASHTO T-240 or ASTM D-2872.

Note 2 Use AASHTO T-51 as modified by Washington DOT (using a special mold release agent and a special method of applying the release agent).

Note 3 The absolute viscosity (60°C) of PBA-3, PBA-5, PBA-6 and PBA-7 will be determined at 1 sec-1 using ASTM P-159 (Volume 4.03, 1985) with the Asphalt Institute Vacuum Capillary Viscometers.

Note 4 "Tilt Oven Residue" means the asphalt residue obtained using California Test 374, "Method for Determining Asphalt Durability Using the California Tilt-Oven Durability Test."

Table A-3. PBA-6GR Binder Specification

Characteristic	AASHTO Test Method	Specification Requirements
Penetration (4°C 200 g, 60 sec), dmm	T-49	30+
RTFO Aged Residue (Note 1)		
Absolute Viscosity (60°C, P) (Note 2)		
Original Binder	T-202	2000 +
RTFO Aged Residue (Note 1)	T-202	5000 +
RTFO Viscosity/Original Viscosity		4.0-
Kinematic Viscosity (135°C), cSt		
Original Binder	T-201	2000 -
RTFO Aged Residue	T-201	275 +
Flash Point, Cleveland Open Cup, °C	T-48	450 +
Original Binder		
Ductility (25°C, 5 cm/min), cm	T-51	Report
RTFO Aged Residue		

Powdered Rubber Paving Asphalt shall contain not less than 8% by weight of total material of powdered rubber meeting the requirements described below.

Note 1 "RTFO Aged Residue" means the asphalt residue obtained using the Rolling Thin-Film Oven Test (RTFO Test), AASHTO T-240 or ASTM D-2872.

Note 2 The absolute viscosity (60°C) will be determined at 1 sec-1 using ASTM P-159 (Volume 4.03, 1985) with the Asphalt Institute Vacuum Capillary Viscometers.

The powdered rubber used in the PBA-6GR shall conform to the following specification requirements:

Chemical Properties:

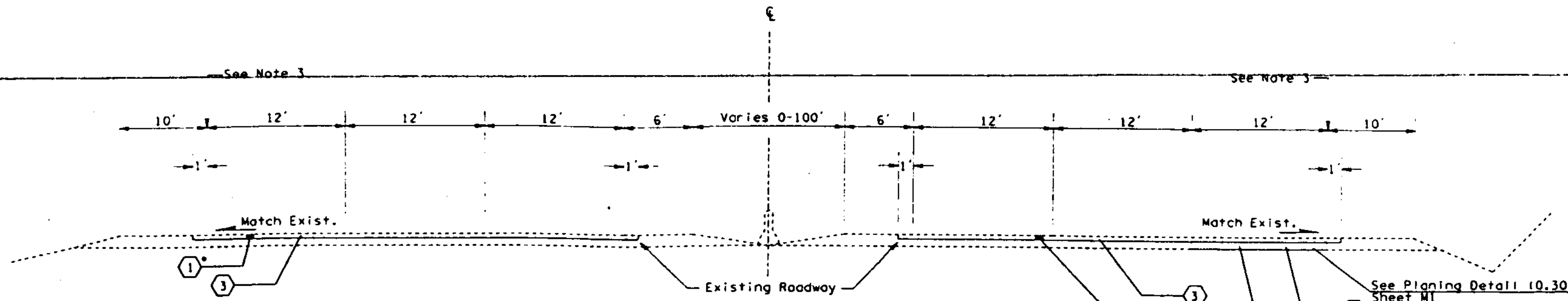
Acetone Extract (ASTM D-297) % max.	23
Ash (ASTM D-297B) % max.	7
Carbon Black (ASTM D-297B) % max.	34
Rubber Hydro Carbon (by difference) % max.	42
Specific Gravity (ASTM D-297)	1.15 ± 0.02
Moisture Content % max.	1.0

Sieve Analysis:

Sieve Size	% Passing
#60	99 - 100
#80	89 - 100
#100	74 - 90
#200	24-90

A job mix formula and mix design will be required for each asphalt cement.

Appendix B
Roadway Sections



LEGEND

1. ASPHALT CONC. PAVEMENT CL. A (AR 4000W) - 0.15' COMP. DEPTH
   - *ASPHALT CONC. PAVEMENT CL. A (PBA-6) - 0.15' COMP. DEPTH FROM STA. NB & SB 540+05 TO STA. NB & SB 645+65
   - *ASPHALT CONC. PAVEMENT CL. A (PBA-6 GR) - 0.15' COMP. DEPTH FROM STA. NB & SB 645+65 TO STA. NB & SB 751+25
2. ASPHALT CONC. FOR PRELEVELING CL. G - VARIABLE COMP. DEPTH
3. PLANING ASPHALT CONC. PAVEMENT - 0.15' DEPTH
4. PLANING ASPHALT CONC. PAVEMENT - 0.30' COMP. DEPTH STA. NB 527+00 TO STA. NB 560+00
5. ASPHALT CONC. PAVEMENT CL. G - 0.08' COMP. DEPTH
6. ASPHALT CONC. PAVEMENT CL. A FOR PAVEMENT REPAIR - VARIABLE COMP. DEPTH (SEE NOTE 1)
7. CRUSHED SURFACING BASE COURSE - VARIABLE COMP. DEPTH (SEE NOTE 2)
8. ASPHALT CONC. PAVEMENT CL. A - 0.60' COMP. DEPTH (SEE NOTE 1)
9. CRUSHED SURFACING BASE COURSE - 0.40' COMP. DEPTH (SEE NOTE 2)
10. CRUSHED SURFACING BASE COURSE - 0.25' COMP. DEPTH (SEE NOTE 2)
11. ASPHALT CONC. PAVEMENT CL. G - 0.15' COMP. DEPTH

ROADWAY SECTION A

(SEE NOTE 4)

STA. NB 445+70 TO STA. NB 469+10
STA. NB 473+10 TO STA. NB 622+50
STA. NB 624+40 TO STA. NB 677+60
STA. NB 678+38 TO STA. NB 757+13
STA. NB 758+73 TO STA. NB 784+38

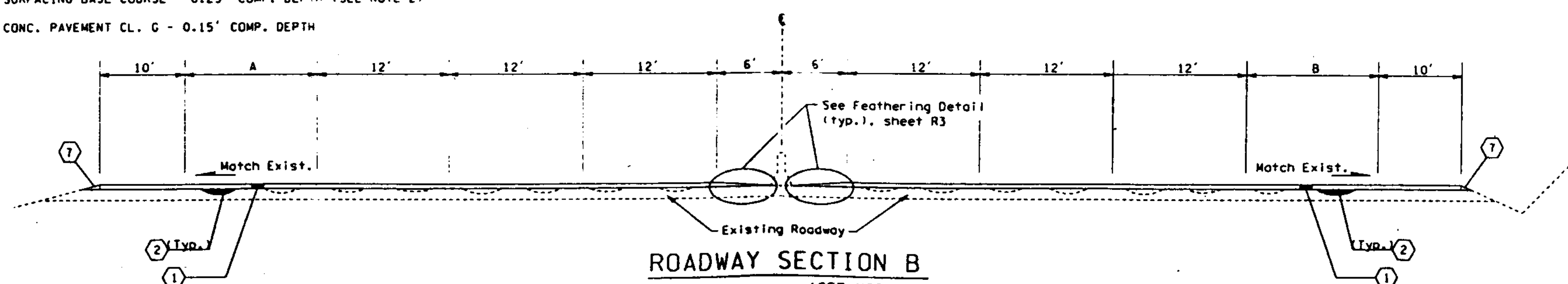
STA. SB 540+05 TO STA. SB 622+50
STA. SB 624+40 TO STA. SB 677+60
STA. SB 678+38 TO STA. SB 756+69
STA. SB 758+48 TO STA. SB 781+40

PAVING EXCEPTIONS-SECT. A

STATIONS		BRIDGE NO. (INCL. APPROACH SLABS)
NB 469+10 TO	NB 473+10	5/402W
NB & SB 622+50 TO	NB & SB 624+40	5/407
NB & SB 677+60 TO	NB & SB 678+38	5/409
NB 756+69 TO	NB 758+48	5/411E
NB 757+13 TO	NB 758+73	5/411W

NOTES:

1. ASPHALT CONC. PAVEMENT CL. A COMP. DEPTH SHALL NOT EXCEED 0.35' PER LIFT.
2. CRUSHED SURFACING BASE COURSE SHALL NOT EXCEED 0.35' PER LIFT.
3. FOR RAMP TAPERS, SEE RAMP TAPER DETAILS, SHEET M5.
4. INSTALL LONGITUDINAL JOINT SEAL IN ACCORDANCE WITH STANDARD PLAN A-1 (TYPICAL ACP/CCP JOINT) AT BRIDGE APPROACH SLABS AND CEMENT CONCRETE PAVEMENT SEATS.



ROADWAY SECTION B

(SEE NOTE 4)

	A	B
STA. NB 784+38 TO STA. NB 838+10	0'	0'
STA. NB 838+10 TO STA. NB 842+10	0'	0'-48'
STA. NB 842+10 TO STA. NB 850+45	0'	0'
STA. NB 853+45 TO STA. NB 862+90	0'	0'
STA. NB 862+90 TO STA. NB 871+00	0'	48'-0'
STA. NB 871+00 TO STA. NB 884+50	0'	0'
STA. NB 884+50 TO STA. NB 890+00	0'	0'-48'
STA. NB 890+00 TO STA. NB 898+33	0'	0'
STA. NB 901+33 TO STA. NB 910+30	0'	0'
STA. NB 910+30 TO STA. NB 912+20	0'	48'-12'
STA. NB 912+20 TO STA. NB 932+55***	0'	12'

	A	B
STA. SB 781+40 TO STA. SB 833+40	0'	0'
STA. SB 833+40 TO STA. SB 844+60	0'-48'	0'
STA. SB 844+60 TO STA. SB 850+45	0'	0'
STA. SB 853+45 TO STA. SB 863+30	0'	0'
STA. SB 863+30 TO STA. SB 868+20	48'-0'	0'
STA. SB 868+20 TO STA. SB 886+00	0'	0'
STA. SB 886+00 TO STA. SB 893+30	0'-48'	0'
STA. SB 893+30 TO STA. SB 898+33	0'	0'
STA. SB 901+33 TO STA. SB 910+20	0'	0'
STA. SB 910+20 TO STA. SB 919+00	48'-12'	0'
STA. SB 919+00 TO STA. SB 932+55***	12'	0'

***SEE PAVEMENT REMOVAL DETAIL, SHEET M2 FOR DETAIL

DESIGNED BY	L. SCHMITT/G. LYDON	12-21-92		
ENTERED BY	A. LEINGANG			
CHECKED BY	H. DIEP/J. VESPER			
PROJ. ENGR.	D. HILDERBRANT			
DIST. ADM.	G. DENICH			
		DATE	DATE	REVISION

REGION NO.	STATE	FED. AID PROJ. NO.
10	WASH	IM-005-3(802)
JOB NUMBER 93W022		

PROGRAM DEVELOPMENT DIVISION

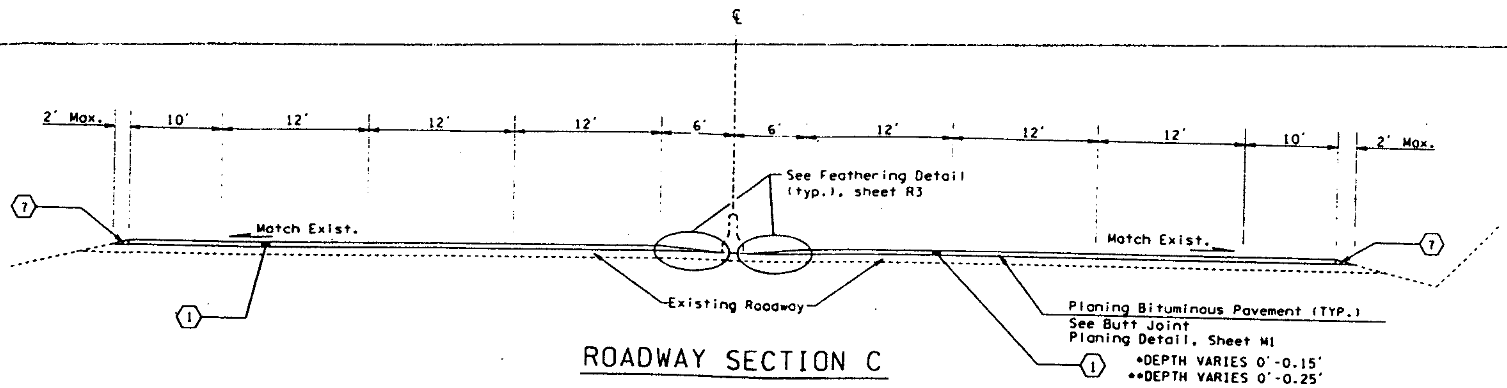
Washington State Department of Transportation

SR 5
NISQUALLY RIVER TO GRAVELLY LAKE I/C

ROADWAY SECTIONS

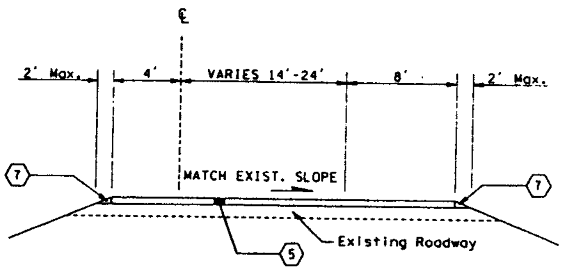
R1

SHEET 5 OF 84 SHEETS



ROADWAY SECTION C

STA. NB & SB 850+45 TO STA. NB & SB 853+45 *
STA. NB & SB 898+33 TO STA. NB & SB 901+33 **



ROADWAY SECTION D

MOUNTS RD. I/C

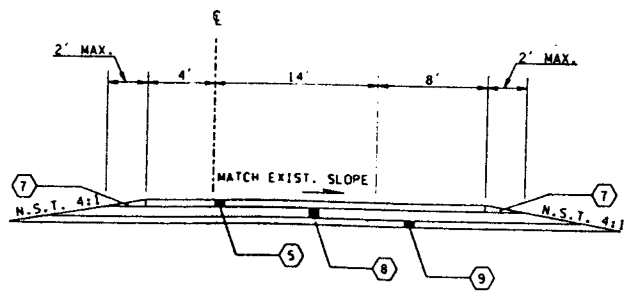
NB:	STA	A 1+66	TO	STA	A	11+90
	STA	B 0+50	TO	STA	B	7+40
SB:	STA	WS 1+60	TO	STA	WS	6+50
	STA	WS 8+00	TO	STA	WS	24+94.3
	STA	D 0+00	TO	STA	SB	528+30

WEST TILLICUM I/C

NB:	STA	BR 0+90	TO	STA	BR	10+02
	STA	CR 0+70	TO	STA	CR	10+50
SB:	STA	CL 0+90	TO	STA	CL	8+98
	STA	BL 0+75	TO	STA	BL	8+95

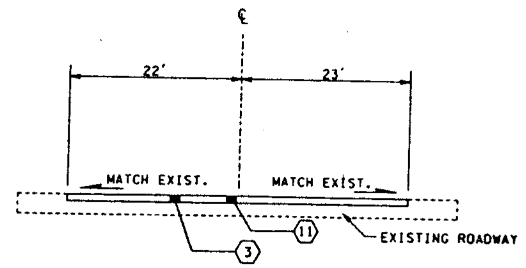
EAST TILLICUM I/C

NB:	STA	BR 0+50	TO	STA	BR	10+19
	STA	CR 0+50	TO	STA	CR	10+50
SB:	STA	CL 0+50	TO	STA	CL	9+29
	STA	BL 0+60	TO	STA	BL	8+58



ROADWAY SECTION E

STA. WS 6+50 TO STA. WS 8+00



ROADWAY SECTION F

STA. AL4 0+96 TO STA. AL4 1+36 (Steilacoom-Dupont Rd.)

NOTE:
N.S.T.= NOT STEEPER THAN

SEE LEGEND SHEET R1

DESIGNED BY L. SCHMITT/G. LYDON 12-21-92
ENTERED BY A. LEINGANG
CHECKED BY H. DIEP/J. VESPER
PROJ. ENGR. D. HILDERBRANT
ASST. ADM. G. DEMICH

DATE	DATE	REVISION	BY

REGION NO.	STATE	FED. AID PROJ. NO.
10	WASH	IM-005-3(802)
JOB NUMBER 93W022		
CONTRACT NO.		

PROGRAM DEVELOPMENT DIVISION

Washington State Department of Transportation

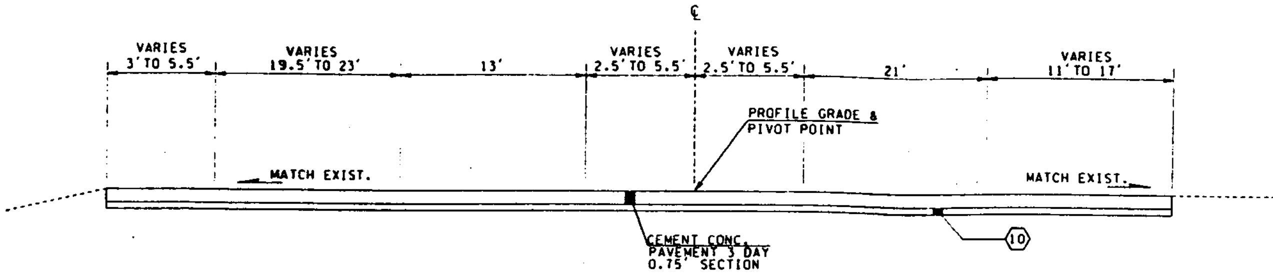
/plans2/hak/w93022e.dgn

SR 5
NISQUALLY RIVER TO
GRAVELLY LAKE I/C

ROADWAY SECTIONS

R2

SHEET 6 OF 84 SHEETS

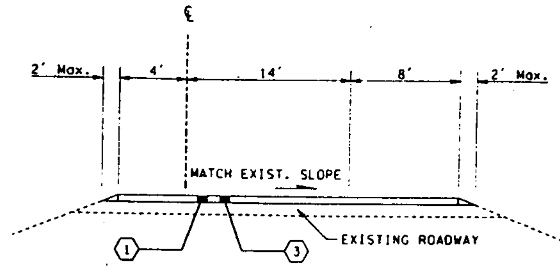


ROADWAY SECTION G

FT. LEWIS I/C

STA. NF 10+00.00 TO STA. NF 10+15.30* (41st Division Dr.)
STA. NF 10+24.30* TO STA. NF 10+35.50 (41st Division Dr.)

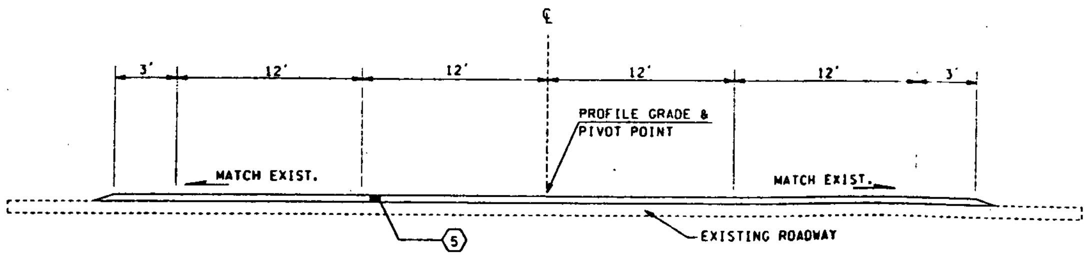
*SEE NOTE 4, SHEET R1



ROADWAY SECTION I

MOUNTS RD. I/C

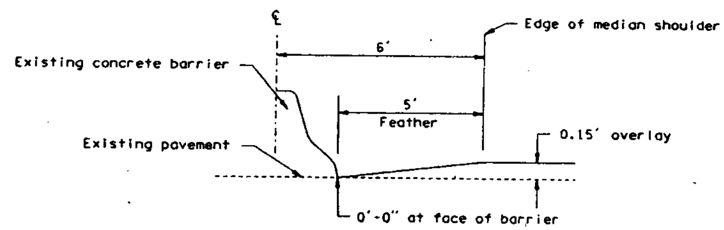
STA. A 1+00 TO STA. A 1+66
STA. SB 562+25 TO STA. WS 1+60



ROADWAY SECTION H

STA. AR6 0+76* TO STA. AR6 1+96 (Gravelly Lake Dr.)

*SEE NOTE 4, SHEET R1



FEATHERING DETAIL

See Special Provision: FEATHERING

SEE LEGEND SHEET R1

DESIGNED BY L. SCHMITT/G. LYDON	12-21-92			REGION NO. 10	STATE WASH	FED. AID PROJ. NO. IM-005-3(802)	PROGRAM DEVELOPMENT DIVISION
ENTERED BY A. LEINGANG				JOB NUMBER 93W022			
CHECKED BY H. DIEP/J. VESPER				CONTRACT NO.			
PROJ. ENGR. D. HILDERBRANT							
DIST. ADM. G. DEMICH							
	DATE	DATE	REVISION	BY			

Washington State Department of Transportation

/plans2/hoh/w93022a.dgn

SR 5
NISQUALLY RIVER TO GRAVELLY LAKE I/C

ROADWAY SECTIONS

R3

SHEET 7 OF 84 SHEETS

Appendix C
Test Reports

Table C-1. Asphalt Mix Samples

Sieve	AR4000W Hveem Mix Design	AR4000W Average Daily Sample	PBA-6 Hveem Mix Design	PBA-6 Average Daily Sample	PBA-6GR Hveem Mix Design	PBA-6GR Average Daily Sample
3/4"	100	100	100	100	100	100
1/2"	94	95	94	91	94	94
3/8"	86	86	86	83	86	86
1/4"	64	67	64	65	64	68
#10	34	36	34	34	34	38
#40	16	17	16	16	16	17
#200	5.0	5.0	4.8	4.2	4.8	4.6
% AC	5.3	5.1	5.3	5.5	5.5	5.7
Rice Density	154.7	155.0	155.2	154.3	154.4	153.9
Bulk Density	149.7	149.2	148.7	149.1	147.1	148.9
% Voids	2.9	3.7	4.0	3.3	4.4	3.3
% VMA	14.4	14.7	15.2	15.1	16.3	15.4
%VMA Filled	79.9	74.8	73.7	78.2	73.0	78.6
Stability	40	39	38	44	36	40
In Place Density		92.5		92.1		92.1
Core Voids						
Set A		9.74		6.52		8.87
Set B		7.17		5.47		8.80

COMPARISON OF PERFORMANCE OF THREE MIXES USING THE RSST-CH (DRAFT)

Submitted to:

LINDA M. PIERCE
WASHINGTON STATE DEPARTMENT OF TRANSPORTATION

Submitted by:

JORGE B. SOUSA
APPLIED PAVING TECHNOLOGY, INC.
HOUSTON, TEXAS

APRIL 20 , 1994

ACKNOWLEDGMENTS

APT express their appreciation for the collaboration and efforts of the following: Bob McGennis, Mike Anderson and Dan Quire from the Asphalt Institute.

DISCLAIMER

Introduction

The purpose of this project was to compare the shear resistance of three mixes containing three types of binders (PBA-6, PBA-6GR and AR4000W). The comparisons were executed on cored specimens.
The existing project was constructed in 1968 with 0.25 ft asphalt concrete pavement over 0.80ft asphalt treated base. In 1980 a 0.06 ft open graded mix was placed. Over the last several years, studded tires began to wear down the open graded material to such an extent that rut depths in the range of 6 to 12 mm occurred. Therefore, the current contract involved milling approximately 1 in. to remove the rutting and then the placement of 0.15 ft ACP overlay with various asphalt types (AR4000W, PBA-6 and PBA-6GR).
The ESAL's on this section of roadway are approximately 21,000,000 with 1.6 percent ESAL growth rate. Since this is essentially a new overlay (in service for approximately 6 month) rutting, if any, is minimal. The purpose of this report is to present the expected ranking of performance.

Selection of Test Temperature

It has been WSDOT's experience that most of the ruts develop under the summer temperatures. To predict the possibility of this occurrence RSST-CH temperature was carefully selected. The average 7-day maximum surface pavement temperature at the site is 51.5 °C with a standard deviation, based on 42 years of records, of 2.2°C (obtained from the SHRP project data base). From these values the temperature at 2 in. depth was computed to be 46.3°C. Considering that this temperature is not to be exceed with a high degree of reliability two standard deviations were added (approx. 0.95% reliability) to that value to yield the test temperature of 50.7°C.

					SURF.	STD		TEMP	TEMP
ST.	COUNTY_ID	STATION	LONG.	LAT.	TEMP	DEV	#	"@ 2 in.	"@2in. + 2 SIGMA
					(C)	(C)	Years	(C)	(C)
WA	THURSTON	OLYMPIA WSO AP	122.90	46.97	51.5	2.2	42	46.3	50.7

Table 1 - Selection of Test Temperature

Laboratory Tests

Test Selection

Rutting (permanent deformation) in an asphalt-concrete layer is caused by a combination of densification (volume change) and shear deformations, each resulting from repetitive application of traffic loads. For properly compacted pavements, shear deformations, caused primarily by large shear stresses in the upper portions of the asphalt-aggregate layer(s), are dominant. Repetitive loading in shear is required in order to accurately measure, in the laboratory, the influence of mix composition on resistance to permanent deformation. Because the rate at which permanent deformation accumulates increases rapidly with higher temperatures, laboratory testing must be conducted at temperatures simulating the highest levels expected in the paving mix in service.

To predict permanent deformation, laboratory tests must be capable of measuring properties under states of stress that are encountered within the entire rutting zone, particularly near the pavement surface. Since there is an infinite number of states of stress it is impossible to simulate them all with a single test given the non-linear and viscous behavior of the material. For this reason several tests have been proposed to determine a constitutive law for asphalt concrete (Sousa et al. 1993). However, if a single test is to be performed to rapidly screen and evaluate the resistance of various mixes to permanent deformation, then that test should be sensitive to the most important aspects of mix behavior and executed under conditions that most significantly affect that behavior. The repetitive simple shear test at constant height on 6 in. (0.15 m) diameter by 2 in. (0.05 m) height cylindrical specimen is proposed as an effective test to evaluate the rutting propensity of a mix. Several reasons lead to the selection of this test:

1- *Specimen Geometry*: a) A 6 in. diameter by 2 in high specimen can easily be obtained from any pavement section by coring, or from any compaction method (i.e., gyratory, rolling wheel, kneading, etc.); b) the state of stress is relatively uniform for the loads applied; c) the magnitude of loads required in testing such specimens is easily achievable by hydraulic equipment. If large stone mixes have to be studied, then 8 in. diameter by 3 in. high specimens should be used.
2- *Rotation of Principal Axes*: It is the simplest test that permits controlled rotation of principal axes of strain and stress which are important in studying rutting.
3- *Repetitively Applied Loads*: Studies have indicated that to capture the rutting phenomena, application of repetitive loads is required given the viscous nature of the binder (mixes behave differently at different loading rates) and also the granular nature of the aggregate (aggregates behave differently under static and repetitive loads).
4- *Dilation*: One of the most important aspects controlling the stability of a mix is dilation. Under shear strains densely compacted mixes tend to dilate (just like dense sands). If dilation is constrained (as it is in the pavement to some degree by the adjacent material) then confining stresses are generated. It is in part due to the development of these confining stresses that a mix derives its stability against shear strains. Mixes with little tendency to dilate will have a higher propensity for rutting. In the constant height simple shear test the development of axial stresses is fully dependent on the dilatancy characteristics of a mix. As permanent shear strains increase the mix will develop (due to dilation) more or less axial stresses depending on the aggregate type, structure, texture and void level. The axial stresses developed this way tend to stabilize the mix. In this test configuration a mix will resist permanent deformation either by relying on high binder stiffness to minimize shear strains or by aggregate structure stability

imparted by the development of axial stresses due to dilation. These two mechanisms are the most important ones that provide resistance to permanent deformation in a mix (mixes with some modified binders can have additional dilation forces caused by modifier dilatancy resulting from shear strain rates). A well compacted mix with a good granular aggregate will develop high axial forces at very small shear strain levels. Poorly compacted mixes can also generate similar levels of axial stresses but they will require much higher shear strains.

Stiffer binders help in resisting permanent deformation as the magnitude of the shear strains is reduced under each load application. The rate of accumulation of permanent deformation is strongly related to the magnitude of the shear strains. Therefore, a stiffer asphalt will improve rutting resistance as it minimizes shear strains in the aggregate skeleton.

In the constant height simple shear test these two mechanisms are free to fully develop their relative contribution to the resistance of permanent deformation as there are not constrained by imposed axial or confining stresses. Recognizing that pavement rutting is predominantly a shear flow phenomenon, it seems reasonable to impose shear stresses and allow the material to develop its own axial stress (representing to some extent the in-situ conditions where dilation is restrained by adjacent material).

Test Procedure

The testing system fabricated by Cox & Sons, Inc. Colfax, California, as been presented by Sousa, Tayebali et al. 1993 was used. It basically consists of two orthogonal tables which are mounted on bearings. The tables are connected to two hydraulic actuators which are controlled using servo-valves under feedback closed-loop digital algorithms. To insure that the shear and axial forces are transmitted to the specimen, aluminum caps are glued to the parallel faces of the specimen. A gluing device was also developed by Cox and Sons, Inc. to insure that the caps faces are glued parallel.

This equipment can accommodate several specimen sizes but for permanent deformation evaluation the recommended specimen size for shear testing is a cylinder with 6 in.

diameter by 2 in height. If large stone mixes are to be tested the recommended specimen size is 8 in. diameter by 3 in. height.

To execute a repetitive simple shear tests at constant height the vertical actuator maintains the height of the specimen constant using as feedback the output of an LVDT measuring the relative displacement between the specimen caps. The horizontal actuator under control by the shear load cell applies haversine loads corresponding to a 10 psi shear stress magnitude with a 0.1 sec loading time and 0.6 sec rest period.

Experience with a wide range of mixes tested at different temperatures and stress levels demonstrated that the 10 psi shear stress magnitude was a reasonable level at which good mixes would exhibit some permanent deformation while poor mixes would not fail excessively fast.

Tests were executed until 5% shear strain was reached or up to 5000 cycles. Prior to testing specimens were conditioned with 100 cycles of 1 psi haversine loading with a 0.1 sec. loading and 0.6 sec. rest period. The purpose of this preconditioning was instrumentation setup.

Specimen Preparation

Cores were obtained from the Washington State Department of Transportation and were labeled with numbers (1, 2, 4, 5, 7 and 8). Two inch thick specimens were cut out of these field cores with a blade saw. Then, the specific gravities of the specimens were determined with and without using parafilm (see Table 2).

Specimen Number	Binder	Air Voids (no parafilm)	Air Voids (with parafilm)
1	PBA-6	6.52	7.60
2	PBA-6	5.47	5.83
4	PBA-6GR	8.87	9.48
5	PBA-6GR	8.80	9.60
7	AR4000W	9.74	11.46
8	AR4000W	7.17	8.54

Table 2 - Summary of air void contents measurements

The specimens were let to dry before being glued to the caps. A DEVCON 5 minute plastic steel putty was used to glue the specimens to the caps, which was let to cure for several hours before testing.

Each specimen was placed in an oven having the same temperature as the mean highest 7-day maximum pavement temperature (at 2 in. depth) for at least two hours (but no more than four hours) before being tested.

Test Results

The RSST-CH was performed in each of the specimens. The tests were performed at 10 psi stress amplitude (with 0.1 second loading time and 0.6 second rest period) and at 7-day maximum pavement temperature encountered at the 2-inch depth. Figure 1 exhibits a graph of the permanent shear strain versus number of cycles obtained from the tests. It is apparent that some mixes really deform faster than others and that not only do they have different slopes but also different intercepts.

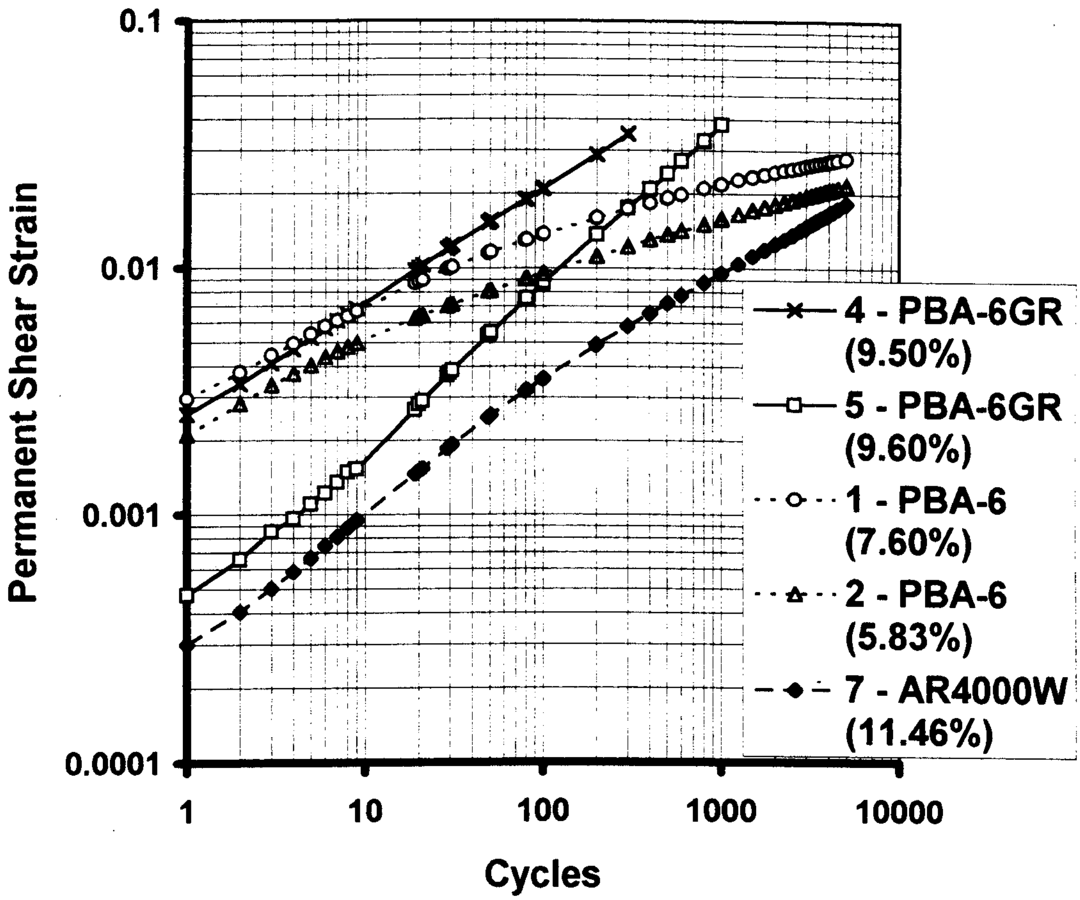


Figure 1 - Permanent Shear Strain versus number of cycles

It can be seen the following rank (from more resistant to less resistant to permanent deformation):

1- AR4000W

2- PBA-6

3- PBA-6GR

However, it can be noted that they have different air voids (measured with parafilm). It could be concluded that the AR4000W is likely to exhibit a better performance, than the others mixes, because even at a higher void content has higher shear resistance (for a given number of cycles accumulates less permanent deformation). However if one extrapolates the results of the PBA-6 to 4 or 5% permanent shear strain it can be observed that the better aggregate interlock, due to the lower air void content, does help.

Specimen Number	Binder	Air Voids (with parafilm)	Number of Cycles to 0.36 % permanent shear strain	ESALS to 0.4 in. rut depth	Number of Years to 0.4 in rut depth
1	PBA-6	7.60	30000	1.34E7	9.2
2	PBA-6	5.83	60000	2.34E7	16.1
4	PBA-6GR	9.48	300	3.26E5	0.2
5	PBA-6GR	9.60	900	7.92E5	0.5
7	AR4000W	11.46	27000	1.23E7	8.5
8	AR4000W	8.54	n/a	n/a	n/a

Table 2 - Summary of test results

The values in column E where obtained from the following equation (Sousa and Solaimanian, 1994):

$$\log\,(\textbf{RSST-CH Cycles}) = -\,4.36 + 1.240\,\log\,(\textbf{ESAL}) \qquad (1)$$

This relationship was obtained with an R^2=0.80 and relates RSST-CH cycles with ESALs in the field.

The values in column F were obtained dividing the values in column E by the annual year ESAL (1,450,000). The annual growth rate in ESAL was not considered.

It must be noted that those are conservative values. It is expected that it will actually take longer to reach those rut depths. It can be state, with a 95% reliability, that a 0.4 in. rut depth will not be reached in less than the time presented in Column F.

This analysis does not consider the fact that, as the mixes densifies it increases shear resistance, and that, with time, the binder in the mix ages, which also contributes to an increase in rut resistance. Therefore the values presented in Column F are a worst case scenario where the mix would not densify and it would not age.

From this data it can be concluded that the PBA6-6GR will exhibit more rutting than the AR4000W given that they have similar air void contents and the AR4000W is an order of magnitude more resistant. Unfortunately core number 8 got distorted by mistake so there are no duplicates for this data.

If one considers that as the mix densifies the increase in shear resistance varies as presented in Figure 2, then comparisons at a similar air void contents can be made. It must be noted that those are extrapolations based on one data point and assume a similar behavior for the other mixes. Actual performance might be different. In fact it is very likely that the slopes of the ESAL versus air void content extrapolations are different. This Figure is presented only to illustrate the analysis. It would be interesting to collect data from field cores after some more traffic densification and determine the actual slope of those lines.

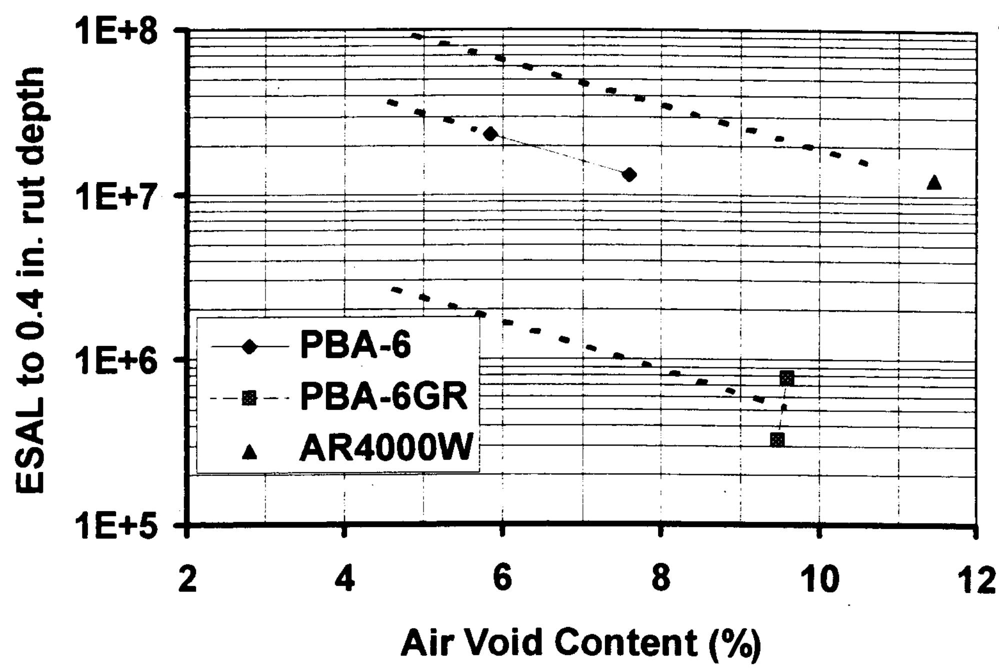


Figure 2 - Comparison of expected mix behavior with air void variation.

A comparison of performance at 5% air void content was made based on the assumptions above. The results are presented in Table 3.

Mix	ESALs to 0.4 in. rut depth	Years to 0.4 in. rut depth
PBA-6	3.0E7	20.7
PBA-6GR	2.4E6	1.7
AR4000W	9.0E7	62.0

Table 3 - Comparison of performance at 5% air void content.

It can be stated, with a 95% confidence level (assuming the ESAL - air void content variation), that in the case of the PBA-6GR mix a rut depth of 0.4 in. will not occur in the first 1.7 years. Based on this analysis is very unlikely that the other mixes (PBA-6 and AR4000W) will develop excessive rut during the life of the project.

It can not be concluded which mix will eventually perform better. At the high air void content where the tests where actually executed most of the contribution to the resistance to permanent deformation is due to the binder films. As the mix densifies with traffic the aggregate skeleton start to play a more important role. For an absolute comparison of behavior mixes should be tested at air void contents at which they exhibit their highest shear resistance. For dense graded mixes that value is between 3 and 4% (with parafilm). It is however very likely that the worst performing mix will be the one with PBA-6GR.

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Rutting Evaluation of Cores from IH-5 in Washington

Prepared for
Washington Department of Transportation

Prepared by
Robert B. McGennis
Asphalt Institute Research Center
Lexington, KY

February 16, 1995

BACKGROUND

In 1994, the Washington DOT placed an asphalt concrete overlay on Interstate 5 from the Nisqually River to the Gravelly Lake Road interchange. The project was approximately 10 miles in length from mileposts 114.97 to 124.64. The overlay was composed of a Washington DOT wearing course mixture, referred to as “Mix Class A.” The mixture contained a variety of asphalt binders. Figure 1 shows the mixture gradation and Table 1 shows project information.

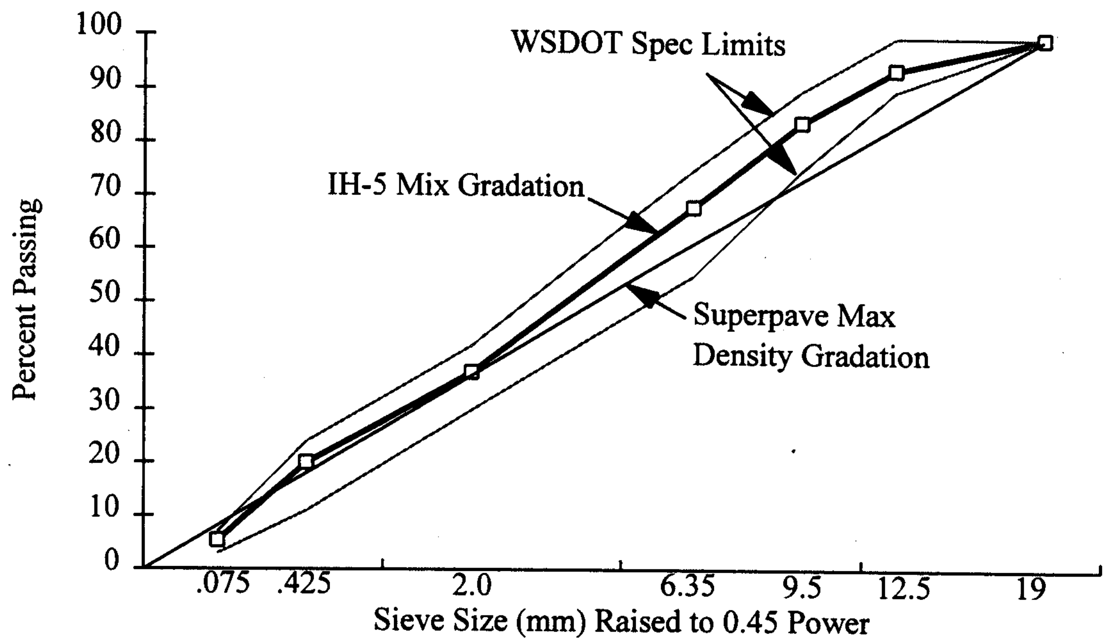

Figure 1. IH-5 Mixture Gradation

Table 1. IH-5 Project Data

Beg MP	End MP	ACP Depth, in	Base Material, in			10-Year ESALs	Binder Type
			ATB	CTB	Cr. Stone		
114.97	115.42	9.8	-	-	2.40	8,698,000	
115.42	115.85	1.92	9.60	-	-	8,698,000	
115.85	116.73	8.88	3.00	-	-	8,695,000	AR-4000W
116.73	116.99	4.66	-	6.00	3.00	8,831,000	
116.99	119.38	5.88	-	6.00	3.00	9,076,000	
120.27	120.53	5.28	-	6.00	3.96	8,440,000	
120.53	121.16	10.08	-	-	3.00	8,396,000	PBA-6GR
121.16	121.41	10.92	-	6.00	3.00	8,335,000	
121.41	124.19	8.04	-	6.00	3.00	8,900,000	PBA-6

The WSDOT desired to estimate the rutting performance of these mixtures. Of interest was the potential difference in rutting performance between the sections containing different binders. Additionally, WSDOT desired to begin mobilizing Superpave technology to analyze their mixtures. Consequently, WSDOT shipped numerous cores to the Asphalt Institute Research Center in Lexington, Kentucky for evaluation. This report describes the results of the evaluation.

WORK PLAN

To make use of Superpave technology, it was decided to use a rutting evaluation procedure proposed by Sousa (*1*). Because the Sousa approach does not require substantial quantities of materials and is suited for testing cores, it was considered ideal for the WSDOT evaluation. This analysis uses the Superpave shear tester to perform the repeated simple shear test at constant height (RSST-CH). The protocol for this test is outlined by AASHTO provisional standards (*2*). This test is not a required procedure in Superpave mixture design and analysis. However, it is allowed under the Superpave system as an optional check test for the purpose of identifying the propensity of asphalt mixtures to exhibit permanent deformation. For the WSDOT evaluation, the standardized test procedure from AASHTO was used. However, evaluation of the test results was by Sousa's method.

The RSST-CH procedure subjects 150 mm diameter by 50+ mm height specimens to a haversine shear load to achieve a controlled shear stress level of 68 kPa. When the repeated shear load is applied, the test specimen seeks to dilate. The signal from an axially oriented LVDT is used as feedback by the vertical actuator to apply sufficient axial load to keep the specimen from dilating. Thus, throughout the shear load application, the specimen height is maintained constant. A load cycle consists of 0.7-second, which is comprised of 0.1-second shear load application followed by 0.6-second rest period. Test specimens are subjected to 5000 load cycles or until the permanent shear strain reaches five percent. Data acquisition during the test is dictated by the AASHTO procedure.

To perform this testing, the overlay lift was split from lower layers by a masonry saw. Although 36 cores were secured by WSDOT, only a limited number could be tested using this approach. That is because specimen instrumentation for the RSST-CH requires a specimen that is at least 50 mm in height. Only eight of the 36 specimens met this requirement. Figure 2 shows the instrumentation geometry for the RSST-CH. Because of the 38 mm distance between the transducer mounting screws, a minimum specimen height of 50 mm is necessary. This meant that any specimen less than 50 mm in height could not reasonably be tested with this instrumentation geometry.

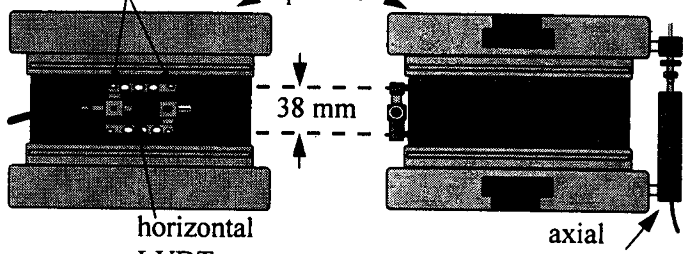

Figure 2. RSST-CH Specimen Instrumentation

Sousa's method of analysis requires that log permanent shear strain be plotted as a function of log load cycles. Estimated rut depth is computed by:

$$\text{Rut Depth (mm)} = 280 \times \text{permanent shear strain (strain units)}$$

Thus, 12.5 mm rut depth corresponds to 0.045 or 4.5 percent shear strain. Sousa relates RSST-CH load cycles to achieve this shear strain level to ESALs by:

$$\text{ESALs} = 10^{((\text{Log(cycles)} + 4.36)/1.24)}$$

By using this equation, a designer can estimate whether a proposed mixture will furnish the desired level of rutting performance.

Examination of the Superpave weather database (*3*) for Washington showed that the 98 percent reliability value for maximum pavement temperature ranged from 43° (Clallam Bay 1) to 62° C (Smyrna). The project site, which is in the Olympic region exhibits values in the range from 52° to 55°. For the 164 weather stations in Washington this value averaged 53.8° with a standard deviation of 4.1°. These temperature values are for a depth of 20 mm. The 53.8° average can be converted to surface temperature by:

$$T_{20\,mm} = (T_{surf} + 17.78)(0.9545) - 17.78$$

where $T_{20\,mm}$ is 53.8°. Substituting 53.8° into this equation yields an average surface temperature of 57.2° for Washington. This value plus two standard deviations [57.2° + 2(4.1°)] yields 65.4°, which is the 98 percent reliability high surface temperature for Washington. In other words, there would only be a two percent chance that the maximum pavement surface temperature in all of Washington would exceed 65.4°. For an average nominal layer thickness of 38 mm, the resulting temperature near the bottom of the average layer, near the zone of maximum shear stress, would be calculated by:

$$T_d = T_{surf}(1 - 0.063d + 0.007d^2 - 0.0004d^3)$$

In this equation, T_d and T_{surf} are in °F and d is in inches. Substituting 149.7° F (65.4° C) for T_{surf} and 1.5 inches (38 mm) for d, this equation yields a temperature of 58.7° C. This value would represent a statewide average value for high temperature pavement conditions near the bottom of a nominal layer thickness of 38 mm. Thus, for this study, a testing temperature of 58° C was chosen as a value that represented a statewide average high temperature and a statewide average layer thickness.

ANALYSIS OF RESULTS

For the purpose of this analysis, it was assumed that 12.5 mm of rutting was a critical level. Thus, for each core tested, the number of cycles to achieve 4.5 percent permanent shear strain ($N_{4.5}$) was determined. Once determined, $N_{4.5}$ was input into the equation listed above to arrive at an estimated number of ESALs to achieve 12.5 mm rut depth. An example of this analysis for one core specimen is shown in Figure 3.

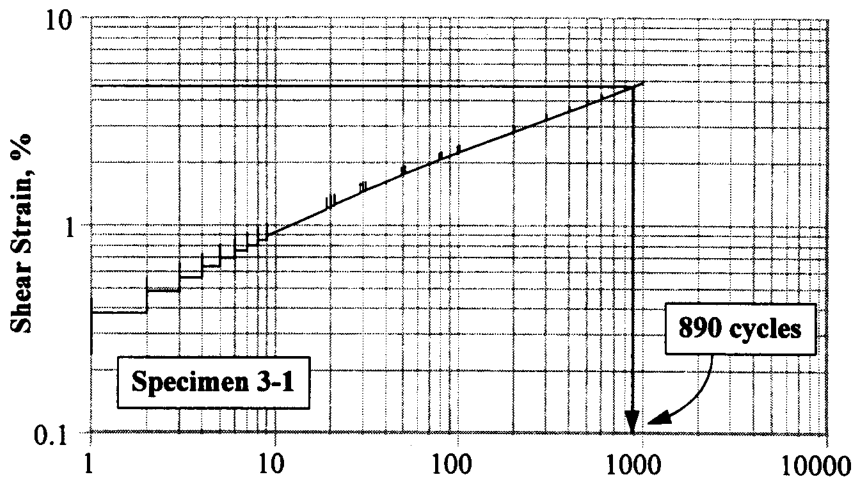

Figure 3. Sample Plot of Permanent Shear Strain versus Number of Cycles

For this test specimen, 890 cycles were required to achieve 4.5 percent shear strain. Consequently, the expected number of ESALs that would result in 12.5 mm rutting is:

$$\text{ESALs} = 10^{((\text{Log}(890) + 4.36)/1.24)} = 784{,}638$$

Table 2 shows the estimated ESALs corresponding to 12.5 mm rut depth for all of the specimens tested as part of this study. Because air void content has a profound effect on rutting susceptibility, this data is also shown in Table 2. The air void content of the cores was fairly homogeneous.

Table 2. Estimated ESALs for 12.5 mm Rut Depth

Specimen	No. Cycles	ESALs	Air Voids, %	Binder Type
1-1	700	646,492	6.8	PBA-6 GR
2-5	700	646,492	7.1	
3-1	890	784,638	4.7	PBA-6
4-2	1000	861,954	7.0	
4-4	310	335,190	7.0	
5-1	720	661,347	6.6	AR-4000W
5-3	640	601,419	6.6	
6-1	560	540,020	6.8	
6-6	800	719,997	6.8	

The most notable aspect of the performance data shown in Table 2 is the relatively low ESALs predicted to achieve 12.5 mm of rut depth. The number of ESALs shown in this table are roughly that which the project would receive in one year. The IH-5 overlay from which these materials were secured has been in service for less than one year. Conversations with WSDOT personnel confirmed that the projects do not appear to be exhibiting signs of permanent deformation. While this project was constructed in August 1994, which means that they have yet to experience a full summer season, WSDOT personnel indicate that past experience with these types of mixtures does not suggest an early, profound rutting failure.

The most likely cause of this evidently faulty rutting performance prediction lies in the method used to predict ESALs to rutting. The equations used to make these predictions were developed by Sousa and others using materials that were not specific to Washington. In addition, the testing temperature was probably 2° to 3° higher than what would have otherwise been used in an analysis of actual project conditions rather than the statewide averages that were used.

Because of the limited number of test specimens, there was no way to perform a rigorous statistical analysis of the data. However, observing the information presented in Table 2 does not suggest significant performance differences among the three types of binders.

CONCLUSIONS AND RECOMMENDATIONS

No information was secured as part of this investigation that would suggest any short- or long-term performance differences among the three binders used by WSDOT. RSST-CH testing using the Sousa performance prediction method at a statewide average high pavement temperature predicted early rutting failure for all three binders. Actual project conditions would have required a slightly lower testing temperature. Nevertheless, WSDOT personnel believe that even at high pavement temperatures such as 58° C used in this study, none of the binders would result in early rutting.

If the Superpave shear tester is to be of maximum use to WSDOT for an *analysis* tool, it will be necessary to develop alternate specimen instrumentation geometry. The current approach will only work if lifts with compacted thicknesses in excess of 50 mm. It is common in Washington, as well as other states, to use nominal layer thicknesses of less than 50 mm. It should be noted that this limitation exists with the normal Superpave Level 2 or 3 analysis as well as with the Sousa approach. This limitation of the Superpave shear tester does not affect its utility as a *design* tool since laboratory specimens can easily be fabricated to any desired height.

Assuming that the specimen instrumentation limitations can be overcome, it is likely that WSDOT will have to develop locally calibrated rutting prediction relationships if the RSST-CH is to be used as a check test. No data was presented in this study to confirm or deny the veracity of the Superpave Level 2 or 3 performance prediction approach. It is possible that approach will be more accurate for Washington conditions and materials.

WSDOT should continue monitoring the test sections to observe rutting performance. Should WSDOT decide to develop locally calibrated performance prediction relationships, this overlay project will provide a good starting point.

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